Rules for Classification and Construction

IV Industrial Services

6 Offshore Technology

4 Structural Design

Edition 2007

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Germanischer Lloyd Aktiengesellschaft

Head Office
Vorsetzen 35, 20459 Hamburg, Germany
Phone: +49 40 36149-0
Fax: +49 40 36149-200
headoffice@gl-group.com

www.gl-group.com

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Section 1

Environmental Conditions

A. Basic Considerations

Environmental conditions give rise to loads imposed on the structure by natural phenomena including wind, current, waves, earthquake, snow, ice, and earth movement. Environmental loads also include the variation in hydrostatic pressure and buoyancy on structural components caused by changes in the water level due to waves and tides. Environmental loads shall be anticipated from any direction unless knowledge of specific conditions makes a different assumption more reasonable.

The unit shall be designed for the appropriate loading conditions that will produce the most severe effects on the structure. Environmental loads, with the exception of earthquake load, shall be combined in a manner consistent with the probability of their simultaneous occurrence.

1. Determination

Pertinent meteorological and oceanographic conditions affecting a unit's operating site shall be defined. The corresponding environmental design data should be prepared to develop the descriptions of normal and extreme environmental conditions. Environmental conditions may be determined by wind, sea currents, sea waves, sea level, climatic conditions, temperature and fouling, sea ice, sea bed conditions, and other influences as applicable.

2. Normal environmental design conditions

Normal environmental conditions are those conditions that are expected to occur frequently during the life of the unit. Normal environmental conditions, important both during the construction and the service life of a unit, consider the most adverse possible effect during the installation and the operation of the unit.

3. Extreme environmental design conditions

3.1 Extreme environmental conditions occur rarely during the life of the unit. Extreme design conditions are important in formulating design loads for the unit when out of operation, while resting on its site with all equipment secured in a seaworthy condition. Design loads may be specified on the basis of statistical observations if available. Probabilistic estimations, if not specified in the Rules, are to be approved by Germanischer Lloyd.

3.2 Fixed marine installations shall be designed to meet limiting environmental conditions valid for the designated operating area.

3.3 Mobile units shall be designed and operated to meet limiting environmental conditions as specified in the operating manual.

4. Estimation of design parameters

Estimation of design parameters on the basis of environmental design conditions, e.g., estimation of wave particle velocity or acceleration on the basis of wave height and period, if not carried out as specified in the Rules, is to be approved by Germanischer Lloyd.

5. Superposition of parameters

Superposition of different environmental design parameters is to be based on physical relations or on stochastic correlations between these parameters or between the environmental phenomena to which they belong.

6. Simplifying assumptions

Simplifying assumptions in these Rules are made in accordance with the specification of related safety factors for structural design. If estimates of environmental design parameters are based on assumptions that are more appropriate for the design case than those specified in the Rules, a reduction of the safety factor may be approved by Germanischer Lloyd, see also Section 3, D.

B. Wind

Although wind loads are dynamic in nature, offshore structures respond to them in a nearly static fashion. However, a dynamic analysis of the unit/installation is indicated when the wind field contains energy at frequencies near the natural frequencies of the unit/installation (be it fixed or moored to the ocean bottom). Sustained wind speeds should be used to determine global loads acting on the unit/installation, and gust speeds should be used for the design of individual structural elements.

1. Wind properties

Wind speed changes with both time and height above sea level. Therefore, the averaging time and height shall be specified. Common reference times are one minute, ten minutes, or one hour. The common reference height is ten (10) meters. Wind forces should be computed using the one (1) minute mean wind speed, and appropriate formulas and coefficients may be derived from applicable wind tunnel tests.

2. Wind force direction

Wind forces shall be considered from any direction relative to the structure.

3. Mean wind speed

The mean wind speed at the reference height of 10 m, averaged over time *t*, may be estimated by the formula

$$u(t) = C_t \cdot u\,(t_r)$$

where

$u(t)$ = mean wind speed at a reference height of 10 m [m/s]

C_t = wind speed averaging time factor

= $[1 - 0{,}047 \ln (t / t_r)]$

$u(t_r)$ = reference wind speed [m/s]

t = averaging time [minutes]

t_r = reference time

= 10 [minutes]

Wind speed averaging time factors C_t for selected averaging times t are given in Table 1.1:

Table 1.1 Wind speed averaging time factors C_t

Averaging time t	C_t
3 seconds	1,249
5 seconds	1,225
15 seconds	1,173
1 minute	1,108
10 minutes	1,000
1 hour	0,916

4. Minimum wind speed for stability calculations of mobile offshore units

For the requirements for stability calculations of mobile offshore units please refer to Chapter 2, Section 7, B.4.

5. Sustained wind speed

The greatest one (1) minute mean wind speed, expected to occur over a return period of 100 years and related to a reference level of 10 m above sea level, is generally referred to as the sustained wind speed, u_S. This sustained wind speed is to be used for the determination of global loads.

6. Gust wind speed

The greatest three (3) second mean wind speed, expected to occur over a return period of 100 years, is referred to as the gust wind speed, u_G. This gust wind speed is to be used for the determination of local loads. The gust wind speed is related to the sustained wind speed as follows:

$$u_G = 1{,}137 \cdot u_S$$

The gust wind component may be considered as a zero mean random wind component which, when superimposed on the constant, average wind component yields the short-term wind speed.

The wind in a 3 second gust is appropriate to determine the maximum static wind load on individual members; 5 second gusts are appropriate for maximum total loads on structures whose maximum horizontal dimension is less than 50 m; and 15 second gusts are appropriate for the maximum total static wind load on larger structures. The one minute sustained wind is appropriate for total static superstructure wind loads associated with maximum wave forces for structures that respond dynamically to wind excitation, but which do not require a full dynamic wind analysis. For structures with negligible dynamic response to winds, the one-hour sustained wind is appropriate for total static superstructure wind forces associated with maximum wave forces.

7. Statistics of wind speed

The Weibull distribution may be used to describe the statistical behavior of the average wind speed u(z, t), referred to a fixed height and an averaging time, as follows:

$$Pr\,(u) = 1 - \exp\,[-(u / u_0)^c]$$

where

$Pr\,(u)$ = cumulative probability of u

u = u(z, t)

= wind speed [m/s]

u_0 = Weibull scale parameter

c = Weibull slope parameter

Gust wind speed may be assumed to follow the Weibull distribution.

8. Wind direction

The wind direction shall generally be assumed to be identical with the dominant direction of wave propagation.

C. Sea Currents

1. Sea currents are characterized as:

– near-surface currents, i.e., wind/wave generated currents, see 2.

– sub-surface currents, i.e., tidal currents and thermosaline currents, see 3.

– near-shore currents, i.e., wave induced surf currents, see 4.

2. Near-surface currents

For near-surface currents, the design velocity may be estimated as follows:

$$u_W(z) = k(z) \cdot u_S$$

where

$u_W(z)$ = near surface current velocity [m/s]

$k(z)$ = factor depending linearly on the vertical coordinate z

= 0,01 for $z = 0$ m

= 0 for $z \geq -15$ m

= to be obtained by linear interpolation for $0 > z > -15$ m

z = vertical coordinate axis above mean sea level [m]

u_S = sustained wind speed used for design [m/s]

= one (1) minute mean at z = 10 m as defined under B.5. [m/s]

3. Sub-surface currents

For sub-surface currents the design velocity is to be based on the current velocity at sea level (z = 0), which shall be based on observed values provided by competent institutions. The data are subject to approval by Germanischer Lloyd. The vertical velocity distribution for $0 \geq z \geq - d$ may be determined as follows:

$$u_{SS}(z) = [(z + d) / d]^{1/7} \cdot u_{S0}(0)$$

where

$u_{SS}(z)$ = sub-surface current velocity [m/s]

d = water depth [m]

z = vertical coordinate axis [m]

u_{S0} = current velocity at sea level [m/s]

Linear superposition of $u_{SS}(z)$ and $u_w(z)$ is applicable.

4. Near-shore currents

For near-shore currents, which have a direction parallel to the shore line, the design velocity at the location of breaking waves may be estimated as follows:

$$u_{nS} = 2 \cdot s \cdot \sqrt{g \cdot H_B}$$

where

u_{nS} = near-shore current velocity [m/s]

s = beach slope

= $\tan \alpha$

α = inclination of beach

g = 9,81 m/s²

H_B = breaking wave height [m]

= $b / [1 / d_B + a / (g \cdot T_B^2)]$

a = $44 [1 - \exp(-19 \cdot s)]$

b = $1,6 / [1 + \exp(-19 \cdot s)]$

d_B = water depth at the location of the breaking wave [m]

T_B = period of the breaking wave [s]

For very small beach slopes, H_B may be estimated from

$$H_B = 0,8 \cdot d_B \text{ [m]}$$

5. Current blockage

For structures that are more or less transparent, the current velocity in the vicinity of a structure is reduced by blockage. The presence of the structure causes the incident flow to diverge, with some of the incident flow going around the structure rather than through it.

The degree of blockage depends on the kind of structure. For dense fixed space frame structures it will be large, while for some kinds of transparent floaters it will be small. Approximate current blockage factors for typical jacket-type structures are given in Table 1.2.

For structures with other configurations, a current factor can be calculated according to C.2.3.1 b4 of API RP 2A-WSD. Factors less than 0,7 should not be used unless empirical evidence supports them.

Table 1.2 Current blockage factors

Number of legs	Current heading	Blockage factor
3	All	0,90
4	End-on	0,80
	Diagonal	0,85
	Broadside	0,80
6	End-on	0,75
	Diagonal	0,85
	Broadside	0,80
8	End-on	0,70
	Diagonal	0,85
	Broadside	0,80

6. Combined wave and current kinematics

Wave kinematics, adjusted for directional spreading and irregularity, should be combined vectorially with the current profile, adjusted for blockage. As the current profile is specified only to storm mean water level, it shall be stretched or compressed to the local wave elevation. For slab current profiles, simple vertical extension of the current profile from storm mean water level to the wave elevation is acceptable. For other current profiles, linear stretching is acceptable. In linear stretching, the current at a point with elevation z, above which the wave surface elevation is η, is obtained from the specified current profile at elevation z'. The elevations *z* and *z'* are related as follows:

$$(z + z') = \frac{(z + z') \cdot d}{(d + \eta)}$$

where d is the storm water depth. Points z and η are both positive above and negative below storm mean water level.

7. Marine growth

All structural members, conductors, risers, and appurtenances shall be increased in cross-sectional area to account for marine growth thickness. Also, elements with circular cross-sections shall be classified as either smooth or rough, depending on the amount of marine growth expected to have accumulated at the time of loading.

8. Conductor shielding

Depending on the configuration of the structure and the number of well conductors, forces caused by a steady current with negligible waves may be reduced if the conductors are closely spaced. A shielding factor, to be applied to the current force for the conductor array, can be estimated according to Table 1.3, in which S is the center-to-center spacing of the conductors in the current direction and D is the diameter of the conductors, including marine growth:

Table 1.3 Shielding factors

S/D	Shielding factor
1,75	0,450
2,00	0,500
2,50	0,625
3,00	0,500
3,50	0,875
4,00 and larger	1,000

For other values of S/D, linear interpolation may be applied.

D. Sea Waves

1. Design criteria

The design environmental criteria shall be developed from the environmental information. These criteria may be based on risk analysis where prior experience is limited. For new and relocated structures that are manned during the design event, or where the loss of or severe damage to the structure could result in severe damage, a 100-year recurrence interval should be used for oceanographic design criteria. Consideration may be given to reduced design requirements for the design or relocation of other structures that are unmanned or evacuated during the design event, that have either a shorter design life than the typical 20 years, or where the loss of or severe damage to the structure would not result in a high consequence of failure.

2. Design conditions

Two alternative methods may be used to specify design conditions for sea waves, namely, the deterministic method based on the use of an equivalent design wave or the stochastic method based on the application of wave spectra.

The deterministic method describes the seaway as regular periodic waves, characterized by wave period (or wave length), wave height, wave direction, and possible shape parameters.

The deterministic wave parameters may be based on statistical methods.

3. Two-dimensional wave kinematics of regular waves

Analytical or numerical wave theories may describe the wave kinematics. For a specified wave period T, wave height h, and storm water depth d, two-dimensional regular wave kinematics can be calculated using appropriate wave theory. The following theories are mentioned:

– linear (Airy) wave theory, where a sine function describes the wave profile

– stokes 5^{th} order wave theory, appropriate for high waves

– stream function theory, where wave kinematics are accurately described over a broad range of water depths

– solitary wave theory for shallow waters

Other wave theories, such as Chappelear, may be used, provided an adequate order of the solution is selected.

In the (H/gT^2, d/gT^2) plane, regions of applicability of various theories are shown in Fig. 1.1 as functions of wave period T, wave height H, and storm water depth d (g = acceleration of gravity).

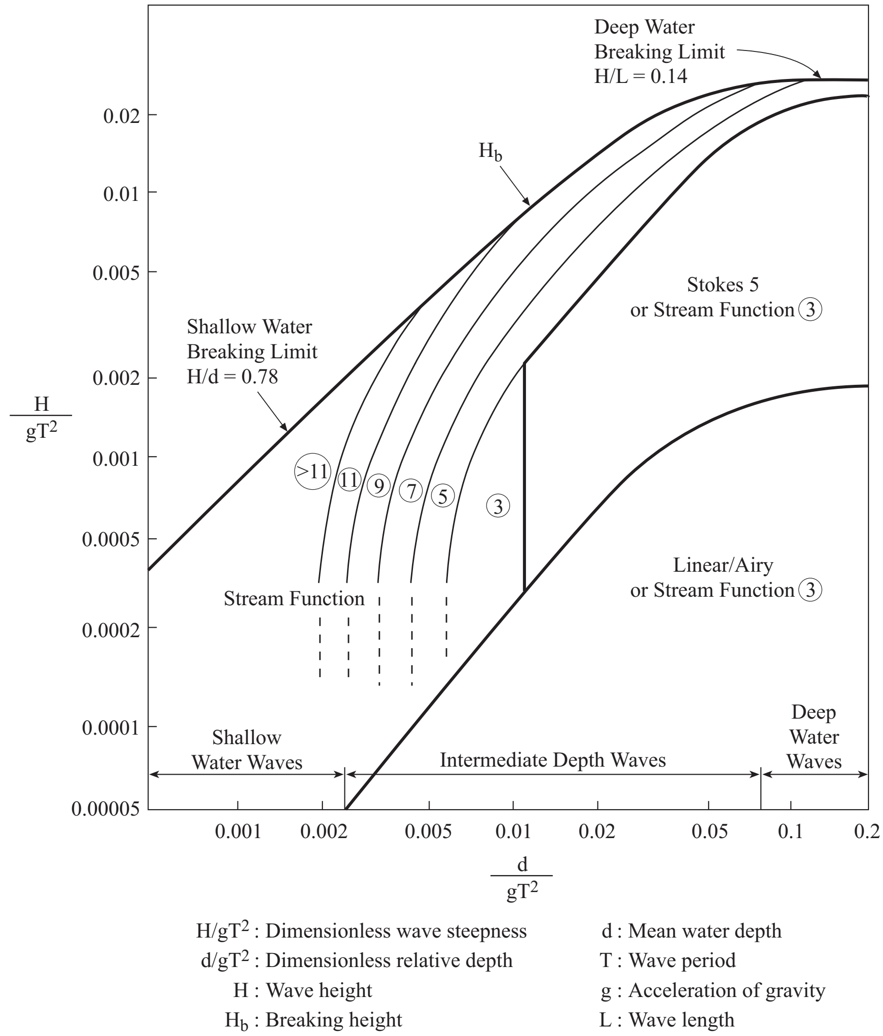


Fig. 1.1 Regions of applicability of wave theories

3.1 Equivalent design wave

Wave loads on a platform are dynamic in nature. For most design water depths presently encountered, these loads may be adequately represented by their static equivalents. For deeper waters or where platforms tend to be more flexible, a load analysis involving the dynamic action of the structure is required.

3.2 Design wave period and design wave height

The design wave period T_D, is associated with the highest wave. In any given storm, there is only one maximum wave, and the associated period is the most probable period of the maximum wave. For this reason it is common to give a range of wave periods for T_D that are independent of the peak period T_p. The peak period is the value associated with the "peak" of the wave spectrum. A range of design wave periods is specified as follows:

$$3,5 \cdot \sqrt{H_{1/3}} < T_D < 4,5 \cdot \sqrt{H_{1/3}}$$

where

T_D = design wave period [s]

$H_{1/3}$ = significant wave height [m]

The design wave period T_D shall not exceed 25 s.

The design wave height H_D, may be estimated as follows:

$$H_D = \sqrt{0,5 \cdot \ln\left(\frac{3600}{T_D}\right)}\ H_{1/3}$$

This method generally applies to deep water waves, i.e. waves of periods T that satisfy the following condition:

$$d/\left(g \cdot T^2\right) > 0,06$$

where

d = water depth [m]

Design wave parameters for wind-induced transitional water waves, i.e. water waves of periods T that satisfy the condition

$$0{,}002 < d/\left(g \cdot T^2\right) < 0{,}06$$

may be defined from information on extreme wind speed and fetch, using relevant theories subject to approval by Germanischer Lloyd.

4. Short-term wave conditions

Short-term stationary natural sea states may be described by a wave spectrum, i.e. by the power spectral density function of the vertical sea surface elevation. Short-term wave conditions are defined as seaways whose representative spectrum does not change for a brief but not closely specified duration. Wave spectra may be given in tabular form, as measured data, or in analytic form.

The two parameter modified Pierson-Moskowitz spectrum and the JONSWAP spectrum are most frequently applied. Both spectra describe sea conditions relevant for the most severe sea states. Application of other wave spectrum formulations is to be approved by Germanischer Lloyd.

4.1 The Pierson-Moskowitz spectrum

The principal natural sea state parameters are as follows:

– The significant wave height $H_{1/3}$ is defined as the average of the one-third highest wave heights in a record of stationary sea surface elevations.

– The characteristic wave period T_1 is defined as the average observed wave period in a record of stationary sea surface elevations.

The visually observed wave height H_v and the visually observed wave period T_v may be considered approximately equal to $H_{1/3}$ and T_1, respectively.

The following expression describes the wave energy spectral density:

$$S(\omega) = 173 \frac{H_{1/3}^2}{T_1^4 \cdot \omega^5} \exp\left[-\frac{691}{T_1^4 \cdot \omega^4}\right]$$

$$= 0{,}313 \frac{H_{1/3}^2 \cdot \omega_p^4}{\omega^5} \exp\left[-\frac{5}{4}\left(\frac{\omega_p}{\omega}\right)^4\right]$$

where

$S(\omega)$ = spectral density [m^2s]

ω = circular wave frequency [s^1]

= $2\pi/T$

T = wave period [s]

ω_p = circular spectral peak frequency [s^1]

= $2\pi/T_p$

T_p = modal period [s]

= period at which the spectrum is a maximum

4.2 The JONSWAP spectrum

The JONSWAP spectrum is also frequently applied. In addition to the significant wave height $H_{1/3}$ parameters are needed to describe the spectral shape, namely, the peakedness parameter γ and the spectral shape parameter σ.

The following expression describes the wave energy spectral density:

$$S(\omega) = 155 \frac{H_{1/3}^2}{T_1^4\ \omega^5} \exp\left[-\frac{944}{T_1^4\ \omega^4}\right] \gamma^{\exp\left[-\frac{0{,}191\omega T_1 - 1}{\sqrt{2}\,\sigma}\right]^2}$$

$$= 0{,}205 \frac{H_{1/3}^2\ \omega_p^4}{\omega^5} \exp\left[-\frac{5}{4}\left(\frac{\omega_p}{\omega}\right)^4\right] \gamma^{\exp\left[-\frac{(\omega-\omega_p)^2}{2\sigma^2\omega_p^2}\right]}$$

where

$S(\omega)$ = spectral density [m^2s]

γ = peakedness parameter

= 3,30 for a mean spectrum

σ = spectral shape parameter

= 0,07 if $\omega \le 5{,}24/T_1$

= 0,09 if $\omega > 5{,}24/T_1$

T_1 = average observed or mean period [s]

4.3 Spectral moments

The spectral moments, m_n, of order n are defined as follows:

$$m_n = \int_0^\infty \omega^n S(\omega)\, d\omega \quad \text{for } n = 0, 1, 2, \ldots$$

where

$S(\omega)$ = wave energy spectral density [m^2s]

ω = circular wave frequency [s^1]

= $2\pi/T$

= wave period [s]

The following quantities may be defined in terms of the spectral moments:

– Significant wave height:

$$H_{1/3} = \sqrt{m_0} \quad [m]$$

– Average observed wave period (center of gravity of the wave spectrum):

$$T_1 = 2\pi \frac{m_0}{m_1} \qquad [s]$$

– Zero-crossing period (average period between successive crossings):

$$T_0 = 2\pi\sqrt{\frac{m_0}{m_2}} \quad [s]$$

– Crest period (average period between successive wave crests):

$$T_C = 2\pi\sqrt{\frac{m_2}{m_4}} \quad [s]$$

– Significant wave slope:

$$s = \frac{2}{\pi \cdot g} \cdot \frac{m_2}{\sqrt{m_0}} \approx \sqrt{\frac{\gamma}{\pi}}$$

where

γ = peakedness parameter

= 3,30 for a mean spectrum

4.4 Spectral density and moments in terms of wave frequency

The wave spectral density may also be given in terms of wave frequency f in [Hz]. The relationship is

$$S(f) = 2\pi \cdot S(\omega)$$

The moments of the wave energy spectral density may also be given in terms of the wave frequency f in [Hz]. The relationship is

$$m_n(f) = \int_0^\infty f^n \cdot S(f) \cdot df = (2\pi)^n \cdot m_n$$

4.5 Other wave periods

The following wave periods are also in use:

– The significant wave period, $T_{1/3}$, is the average period of the 1/3 highest wave periods.

– The maximum wave period, T_{max}, is the period of the wave with maximum height.

4.6 Wave spreading and short-crested sea states

The wave spectra given above are uni-directional. However, real waves travel in many different directions. Directional short-crested wave spectra may be derived from a uni-directional wave spectrum as follows:

$$S(\omega, \alpha) = S(\omega).\, W(\alpha)$$

where

$S(\omega,\gamma)$ = directional short-crested wave energy spectral density

α = angle between direction of elementary wave trains and primary wave direction

$W(\alpha)$ = directionality function

= const · $\cos^s \alpha$

The value of the power constant *s* shall reflect an accurate correlation to the actual sea state. For typically occurring conditions in the open ocean s = 2 is appropriate.

For design purposes it is usual to assume that the secondary wave directions are spread over –90° < α < + 90°. Energy conservation requires that the directionality function fulfils the requirement

$$\int_{\alpha_{min}}^{\alpha_{max}} W(\alpha) = 1$$

4.7 Wave period relationships

For the Pierson-Moskowitz spectrum, wave periods are related to each other by the factors given in Table 1.4:

Table 1.4 Factors relating wave periods for the Pierson-Moskowitz spectrum

	T_1	T_0	$T_{1/3}$	T_p	T_{max}
T_1	1	1,086	0,88	0,77	0,74
T_0	0,92	1	0,81	0,71	0,68
$T_{1/3}$	1,14	1,24	1	0,88	0,85
T_p	1,30	1,41	1,14	1	0,96
T_{max}	1,35	1,46	1,18	1,04	1

For example:

$T_1 = 0{,}77 \cdot T_p = 1{,}09 \cdot T_0$ and

$T_0 = 0{,}71 \cdot T_p$

$T_p = 1{,}30 \cdot T_1$

For the JONSWAP spectrum, wave periods are related to each other as follows:

$T_1 = 0{,}83 \cdot T_p = 1{,}07 \cdot T_0$ and

$T_0 = 0{,}78 \cdot T_p$

Note that the modal period of the Pierson-Moskowitz spectrum differs from the modal period of the JONSWAP spectrum:

For the Pierson-Moskowitz spectrum, $T_p = 1{,}30 \cdot T_1$.

For the JONSWAP spectrum, $T_p = 1{,}20 \cdot T_1$.

5. Response in natural seas

5.1 Response from different wave components

A useful consequence of linearly predicted response is that results from regular waves of different amplitudes, wave lengths and propagation directions can be superimposed to obtain the response in natural seaways made up of a large number *N* of regular waves of different length and height. Thus, the response from different wave components can be written as

$$\sum_{n=1}^{N} \varsigma_{an} \cdot H^2(\omega) \cdot \sin\left[\omega_n \cdot t + \delta(\omega_n) + \varepsilon_n\right]$$

where

$|H(\omega_n)|$ = response amplitude per unit wave amplitude (transfer function)

$\delta(\omega_n)$ = a phase angle associated with the response

Both $|H(\omega_n)|$ and $\delta(\omega_n)$ are functions of the frequency of oscillation ω_n. The response can be any linear wave-induced motion of or load on the structure.

5.2 Variance of the response

In the limit, as N approaches infinity, the variance of the response, σ_r, is found as follows:

$$\sigma_r^2 = \int_0^\infty S(\omega) \cdot H^2(\omega) \cdot d\omega$$

The Rayleigh probability function may be used to approximate the probability density function for the maxima (peak values) of the response, p(R):

$$p(R) = \frac{R}{\sigma_r^2} \exp\left(-0{,}5 \cdot R^2 / \sigma_r^2\right)$$

5.3 Most probable largest value during a short-term seastate

During a short-term seastate of significant wave height $H_{1/3}$ and mean wave period of the sea spectrum T_1 the most probable largest value R_{max} over time t is then

$$R_{\max} = \sqrt{2.\sigma_r^2 . \ln(t / T_1)}$$

Strictly speaking, the mean period of the response variable should be used instead of the average observed (mean) wave period T_1. However, for linear wave-induced motions and loads the difference when estimating R_{max} is small and can be neglected.

5.4 Long-term probability of the response

Short-term response predictions are sometimes considered inadequate for the design and reliability analysis of offshore structures, because the assumption of stationarity (of a statistically invariant record) restricts the validity of the analysis. A long-term probability defines events and extreme value statistics for a period on the order of 20 to 100 years, as opposed to a few hours for the short-term probability.

The long-term prediction considers all sea states the structure is expected to encounter during its design lifetime. This may be accomplished in the form of the frequency of occurrences of all possible sea states or the long-term wave data over the entire life of the structure, i.e. in the form of a wave scatter diagram that conveniently describes a family of sea states, each characterized by the wave spectrum parameters ($H_{1/3}$, T_1) or ($H_{1/3}$, T_1, γ) as defined in 4.1 and 4.2, respectively.

The long-term method provides a rational criterion of acceptability in the form of a uniform likelihood that a given response level will be exceeded during the lifetime of the structure.

The long-term prediction can be performed in the frequency domain for a linear system or in the time domain for a nonlinear system.

Combining the Rayleigh distribution and a joint probability ($H_{1/3}$,T_1) from a wave scatter diagram listing M significant wave height intervals and K spectral peak periods yields the long-term probability of the response:

$$P(R) = 1 - \sum_{j=1}^{M} \sum_{k=1}^{K} \exp\left(-\frac{R^2}{2 \cdot \sigma_{rjk}^2}\right) \cdot p_{jk}$$

where

M = number of significant wave height intervals

K = number of spectral peak periods listed in the wave scatter diagram

P(R) = long-term probability that the peak value of the response does not exceed R

σ_{rjk} = standard deviation of the response in the seastate ($H_{1/3}$,T_1) of interval j and mean period k

p_{jk} = probability of occurrence of seastate ($H_{1/3}$,T_1) of interval numbers j and k

6. Derived wave parameters

Derived wave parameters, such as wave particle velocities and accelerations, may be used to determine drag and inertial forces on underwater portions of offshore drilling units or structures that are operating or situated in locations where the water depth is considered deep. Subject to approval by Germanischer Lloyd, other appropriate methods to determine these forces will be considered, provided they are referenced and supported by calculations. Linear (Airy) wave theory is applicable to define these parameters if specific limitations of $H/(gT^2)$ are observed, see 3.1 and 3.2.

6.1 Surface wave profile

The wave profile is defined as follows, see Fig. 1.2:

$$z = 0,5 \cdot h_w \cdot \cos(k \cdot x - \omega \cdot t) \quad \text{when} \quad d > \lambda/2$$

where

z = vertical coordinate of wave surface

k = $2\pi/\lambda$

ω = $2\pi/T$

λ = wave length [m]

T = wave period [s]

t = time [s]

h_w = wave height, crest to trough [m]

h = distance below surface [m]

x = distance from the origin [m]

d = depth from still-water level to bottom [m]

6.2 Water particle velocity for deep water waves

Using a right handed coordinate system x-y-z with x- and y-axes lying in the undisturbed sea surface and the vertical z-axis directed vertically upward (Fig. 1.2), the horizontal and vertical particle velocities at time *t* are defined as follows:

Horizontal:

$$v_x = 0,5 \cdot \omega \cdot h_w \cdot e^{-kh} \cdot \cos(k \cdot x - \omega \cdot t) \ \text{[m/s]}$$

Vertical:

$$v_z = -0,5 \cdot \omega \cdot h_w \cdot e^{-kh} \cdot \sin(k \cdot x - \omega \cdot t) \ \text{[m/s]}$$

The wave propagates along the horizontal *x*-axis.

6.3 Water particle acceleration for deep water waves

The horizontal and vertical particle accelerations at time t are defined as follows:

Horizontal:

$$a_x = 0,5 \cdot \omega^2 \cdot h_w \cdot e^{-kh} \cdot \sin(k \cdot x - \omega \cdot t) \ \text{[m/s]}$$

Vertical:

$$a_z = -0,5 \cdot \omega^2 \cdot h_w \cdot e^{-kh} \cdot \cos(k \cdot x - \omega \cdot t) \ \text{[m/s]}$$

6.4 Wave pressure

The total wave pressure at any point at distance h below the surface is the static pressure ρgh plus the wave dynamic pressure given as follows:

$$p = 0,5 \cdot \rho \cdot g \cdot h_w \cdot e^{-kh}.\cos(k \cdot x - \omega \cdot t)$$

where

p = wave dynamic pressure [kN/m^2]

g = acceleration of gravity [m/s^2]

ρ = mass density of water [kNs^2/m^4]

λ = $gT^2/2\pi$

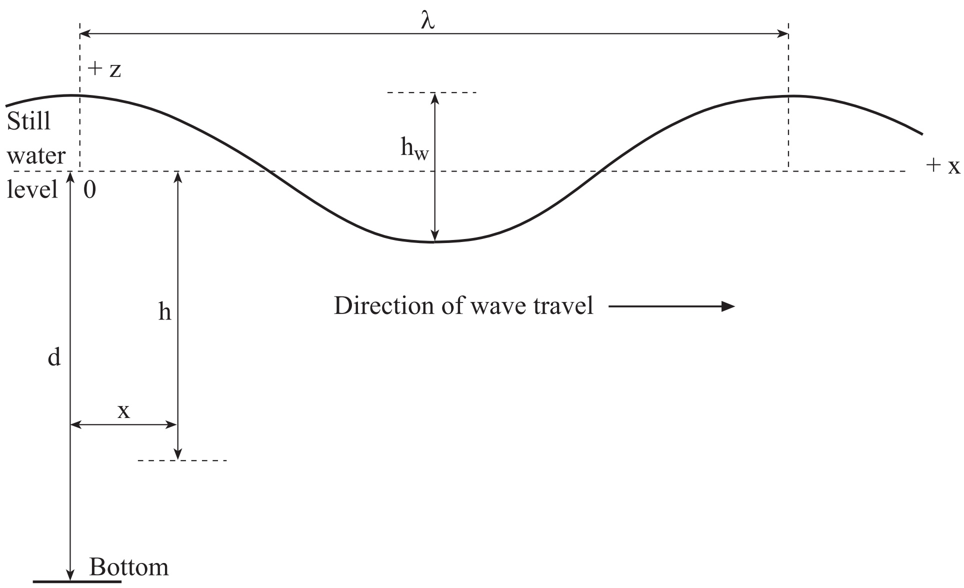


Fig. 1.2 Regular wave surface profile

E. Sea Level

1. The highest still water level to be used for design is defined by the water depth d at the highest astronomical tide plus the water level elevations due to storm surge.

2. The smallest still water level to be used for design is defined by the smaller value of either the water depth d due to the lowest astronomical tide minus the water level decline due to storm surge or the chart datum, whichever is applicable.

3. The highest wave elevation above still water level ζ^* defined in 1. or 2. is given as

$$\zeta^* = \delta \cdot H_D \quad [m]$$

where

H_D = wave height (see D.3.2) [m]

δ = wave elevation coefficient

The wave elevation coefficient δ is a function of the wave period T_D (see D.3.2) and the water depth d as given in Table 1.5.

Table 1.5 Wave elevation coefficient δ

$d/(gT_D^2)$	$H_D/(gT_D^2)$					
	0,02	0,01	0,005	0,001	0,0005	0,0001
≥ 0,2	0,60	0,55	0,50	0,50	0,50	0,50
0,02	-	0,68	0,58	0,52	0,50	0,50
0,002	-	-	-	0,87	0,80	0,68

4. Water depth and statistical wave data shall be provided by a competent Authority. The application of the data is subject to approval by Germanischer Lloyd.

5. Tsunamis

Tsunamis are generated by large and sometimes distant earthquakes or undersea fault movements or by large sea floor slides triggered by earthquakes or by volcanic eruptions. Traveling through deep water, tsunamis are long surface waves with low height and pose little hazard to floating or fixed offshore structures. When they reach shallow water, the wave form pushes upward from the bottom to make a rise and fall of water that can break in shallow water and wash inland with great power. Tsunamis travel great distances very quickly and can affect regions not normally associated with such events. The likelihood of tsunamis affecting the location of the structure shall be considered.

The greatest hazard to offshore structures results from inflow and outflow of water in the form of waves and currents. These waves can be significant in shallow water, causing substantial actions on structures. Currents caused by the inflow and outflow of water can cause excessive scour problems.

F. Climatic Conditions, Temperature and Marine Growth

1. Design temperature

1.1 Estimates of design temperatures of air and water shall be based on relevant data that are provided by competent institutions. Preferably, 1 percent and 99 percent fractiles shall be used as lowest and highest design temperatures, respectively. Least square fits of the data may be used to obtain a stochastic model.

1.2 If design air temperatures are less than 0 °C, the amount of snow and rain turning to ice on exposed surfaces is to be specified.

1.3 If design air temperatures are less than –2 °C, the amount of ice accretion on free surfaces, e.g., lattice structures, as result of seawater spray is to be specified.

2. Other environmental factors

Depending on circumstances, other environmental factors can effect operations and can consequently influence the design of structures. Appropriate data shall be compiled, including, where appropriate, records and or predictions of

- precipitation
- humidity
- fog
- wind chill
- salinity of sea water
- oxygen content of sea water

3. Marine growth

3.1 The thickness and kind of marine growth depends on location, age of structure, and the maintenance regime. Experience in one area of the world cannot necessarily be applied to another. Where necessary, site-specific studies shall be conducted to establish the likely thickness and the water depth dependence of marine growth.

3.2 The influence of marine growth on hydrodynamic actions is caused by increased dimensions and increased drag coefficients of rough surfaces. Components with circular cross sections shall be classed as either “smooth” or “rough,” depending on the thickness and kind of marine growth expected to accumulate on them.

Structural components can be considered hydrodynamically smooth if they are either above the highest still-water level or sufficiently deep, such that marine growth is sparse enough to ignore the effect of roughness. Site-specific data should be used to establish the extent of the hydrodynamically rough zones.

G. Sea Ice and Icebergs

1. The probability of occurrence of sea ice and icebergs in the operating area shall be established.

2. If seasonal sea ice is probable in the operating area, the modes of occurrence and the mechanical properties of the ice shall be determined by a competent Authority upon agreement with Germanischer Lloyd.

3. If permanent sea ice is probable in the operating area, thickness and extension as well as mechanical properties and drifting characteristics shall be determined by a competent Authority upon agreement with Germanischer Lloyd.

4. This data shall be used to determine design characteristics of the installation as well as possible evacuation procedures.

H. Sea Bed

1. Structures founded on the sea bed may be affected by gradual or transient changes of the sea bed. These changes shall be accounted for because they influence the validity of design parameters (compare E.) and affect the structural integrity.

2. Sea bed stability

As outlined in Section 7, it will have to be determined whether, owing to topography and soil configuration, the possibility of slope failure or slides, cavity failure, or erosion phenomena have to be considered. Settlement and soil liquefaction generally have to be taken into account for gravity foundations.

3. Seismic activities

Where seismic activity is to be expected, two different earthquake levels have to be distinguished:

– A strength level earthquake (SLE) represents a level not unlikely to happen in the region of the planned installation. For this level the strength of the structure is used up to the minimum nominal upper yield stress R_{eH}, but the main functions of the offshore installation can still be fulfilled and, after a sufficient checking operation, can be continued.

– A ductility level earthquake (DLE) represents an extreme level that normally puts the offshore installation out of operation, but still allows the main elements of the installation to keep their position, and the structure is thus able to survive. This should make it possible for the crew to save themselves or to be evacuated and should avoid extreme damage of the installation and to the environment.

The characteristics of these two earthquake levels (acceleration intensity, duration) shall be established by a competent Authority and agreed upon with Germanischer Lloyd, compare Section 2, G. and Section 3, C.

Section 2

Design Loads

A. Basic Considerations

1.1 Design loads for marine structures include environmental loads, permanent loads, functional loads, and accidental loads.

1.2 Loads, or load effects such as stresses, may result from direct action (imposed forces) or indirect action (imposed or constrained deformations, such as settling or temperature gradients). Loads of the latter category are usually termed deformation loads.

In the context of these Rules (Loading Conditions, Section 3, C.), such loads may generally be grouped into one of the categories listed above.

2. These Rules specify - explicitly or implicitly - nominal or design loads to be used in structural analysis.

3. Design values of permanent loads shall be vectorially added to the design values of other loads as listed in 1.1.

4. Design values of environmental and functional loads generally occur at different times. Thus, vectorial superpositions of design values of these loads are conservative, and consideration of the influence of the respective random load processes to define design values of superimposed loads is acceptable. The following rule may be applied to stochastically independent design values [1]:

$$F_D = \sqrt{\left(\sum F_{Di}{}^2\right)} \qquad i = 1,2,3......, I$$

where

F_D = stochastically independent design value of the load under consideration

F_{Di} = independent design load value for load component i

I = number of independent design loads F_{Di} considered

Other superposition principles, especially for nonlinear superposition of correlated random load processes, may be applied upon agreement with Germanischer Lloyd.

1 The term "design load" is used here in connection with deterministic allowable stress/global safety factor design methods, see Section 3, B.1.2 and D. According to the definitions given in Section 3, B.3., in case of semi-probabilistic design using partial safety factors, the "design loads" defined in this Section (2) are characteristic or representative loads, still to be multiplied by load factors γ_f.

B. Environmental Loads

1. Environmental loads include all loads attributable to environmental conditions as specified in Section 1:

- wind loads, see 2.
- sea current loads, see 3.
- wave loads, see 4.
- vibratory loads caused by vortex shedding, see 5.
- loads caused by climatic conditions, see 6.
- loads caused by sea ice, see 7.
- mooring loads, see D.2.
- Other environmental loads, such as loads caused by sea bed movements, earthquakes, etc., as applicable.

2. Wind loads

2.1 Wind pressure

A design value of wind pressure on structural elements at height *z* above still-water level may be calculated according to the following formula:

$$q = \tfrac{1}{2}\,\rho \cdot u^2 \cdot C_S \cdot C_H$$

where

q = wind pressure [kPa]

u = design wind speed, averaged over time t [m/s]

= u(z,t), see Section 1, B.3.

ρ = density of air

= 0,001224 for dry air [$kN \cdot s^2/m^4$]

z = coordinate for height above sea level [m]

C_S = shape coefficient from Table 2.1

$$C_H = \left(\frac{z}{10\,m}\right)^{\frac{2}{10}} \quad \text{in general}$$

C_H may be derived from Chapter 2 – Mobile Offshore Units, Section 7, Table 7.2 for stability calculations for floating units.

In the absence of data indicating otherwise, the following shape coefficients (C_S) are recommended for perpendicular wind approach angles with respect to each projected area.

Table 2.1 Wind pressure shape coefficients C_S

Shape	C_S
Spherical shapes	0,4
Cylindrical shapes (all sizes)	0,5
Large flat surfaces (hull, deckhouses, smooth deck areas)	1,0
Drilling derrick	1,25
Wires	1,2
Exposed beams and girders under deck	1,3
Small parts	1,4
Isolated shapes (cranes, beams, etc.)	1,5
Clustered deckhouses and similar structures	1,1

Shape coefficients for shapes or combination of shapes that do not readily fall into the specified categories are subject to special consideration or may be provided by competent institutions. These values are to be approved by Germanischer Lloyd.

The values of the height coefficient (C_H) depend on the height of the center of pressure of all wind exposed surfaces above sea level.

In calculating the wind overturning moment for fixed platforms and jack-up rigs in elevated mode, the lever for the wind overturning force is to be taken vertically from the seabed to the centre of pressure of all wind exposed surfaces.

In calculating the wind overturning (heeling) moment for mobile offshore units, the lever for the wind overturning force is to be taken vertically from the centre of lateral resistance of the underwater body of the unit to the centre of pressure of all wind exposed surfaces. (The unit shall be assumed floating free of mooring restraints.)

Wind heeling moments derived from wind tunnel tests on a representative scale model of the structure may be considered as alternatives. The determination of such heeling moments shall include lift and drag effects.

2.2 Shielding

Shielding may be considered when the object lies close enough behind another object. In such a case, shielding factors listed in Table 1.3 may be used.

2.3 Wind force

The design wind load may be calculated according to the following formula:

$$F = q \cdot A$$

where

F = wind force [kN]

q = wind pressure [kPa]

A = wind projected area of all exposed surfaces in either upright or heeled condition [m^2]

The exposed surfaces are to be taken in either the upright or heeled condition.

2.4 Recommendations

– For units with columns, the projected areas of all columns are to be included without shielding.

– Areas exposed because of heel, such as underdecks, are to be included using the appropriate shape coefficients.

– Cranes, isolated houses, structural shapes, etc., are to be calculated individually, using the appropriate shape coefficients of Table 2.1.

– Open truss structural components, such as lattice structures, derrick towers, crane booms, and certain kinds of masts may be approximated by taking 60% of the projected area of one side, using the shape coefficient of 1,25.

2.5 Design wind loads on the whole structure may be estimated by superposition of all design wind loads on individual structural elements, or by wind tunnel tests with a model of the entire structure.

2.6 Minimum wind loads for mobile drilling units

For mobile units the sustained wind velocities specified by the Owner (Designer) are not to be less than 25,8 m/s (50 knots).

However, for unrestricted services, the wind criteria for intact stability given are also to be applied for structural design considerations, for all modes of operation, whether afloat or supported by the sea bed. Where wind tunnel data obtained from tests on a representative model of the unit by a recognized laboratory are submitted, these data will be considered for the determination of pressures and resulting forces.

3. Sea current loads

3.1 Where current is acting alone (i.e., no waves), a design value of sea current pressure on structural elements at depth *z* below the still-water level is defined as follows:

$$q_D(z) = ½\,\rho\,u_D^2\,(z) \quad \text{for} \quad 0 \ge z \ge -d$$

where

$q_D(z)$ = sea current pressure [kPa]

$u_D(z)$ = design sea current speed [m/s]

ρ = density of seawater

= 1,025 [kNs^2/m^4]

z = coordinate for height above sea level [m]

d = depth from still-water to bottom [m]

The design current speed $u_D(z)$ is either $u_W(z)$, $u_{SS}(z)$ or $u_{nS}(z)$, as defined in Section 1, C.2., C.3. or C.4., or a superposition of these values, as applicable.

3.2 Using $q_D(z)$ as defined above, design sea current loads F(z) may be calculated according to the following formula:

$$F = q_D(z) \cdot A$$

where

F = current force [kN]

qD(z)= current pressure [kPa]

A = current projected area [m²]

Values of the current projected area A may be corrected for effects of marine growth, as applicable, compare Section 1, F.3.

3.3 Consideration shall be given to the possible superposition of current and waves. In those cases where this superposition is deemed necessary, the current velocity shall be added vectorially to the wave particle velocity. The resultant velocity is to be used to compute the total force.

4. Wave loads

4.1 General

4.1.1 The design wave criteria have to be specified by the Owner / Designer. A design value of wave load on a structural element (local wave load) or on the entire structure (global wave load) may be estimated by either using a design wave method, as outlined in 4.2 to 4.6, or a spectral method, as outlined in 4.7.

4.1.2 Consideration is to be given to waves of less than maximum height where, due to their period, the effects on various structural elements may be greater.

4.1.3 Forces caused by the action of waves on the installation/unit are to be taken into account in the structural design, with regard to forces produced directly on the immersed elements and forces resulting from heeled positions or accelerations due to motions. Theories used to calculate wave forces and to select relevant force coefficients are to be acceptable to Germanischer Lloyd.

4.1.4 Forces on appurtenances such as boat landings, fenders or bumpers, walkways, stairways, grout lines, and anodes shall be considered for inclusion in the hydrodynamic model of the structure. Forces on some appurtenances may be important for local member design. Appurtenances are generally modeled by non-structural members that contribute equivalent wave forces.

4.2 Design wave methods

Methods to estimate design wave loads depend on the hydrodynamic characteristics of the structure and on the way the structure is held on location, as outlined in Table 2.2.

4.3 Method I.1: Rigidly positioned, hydrodynamically transparent structures

4.3.1 The Morison formula may be used to calculate the force exerted by waves on a cylindrical object, such as a structural member. The wave force can be considered as the sum of a drag force and an inertia force, as follows:

$$F = F_D + F_I = \tfrac{1}{2}\,\rho \cdot C_D \cdot A \cdot U \cdot |U| + C_M\,\rho\,V\,(\delta U/\delta t)$$

where

F = hydrodynamic force vector per unit length acting normal to the axis of the member [kN/m]

F_D = drag force vector per unit length acting on the member in the plane of the member axis and U [kN/m]

Table 2.2 Summary of design wave methods

Kind of positioning	Hydrodynamic characteristics: I. Transparent [1]	Hydrodynamic characteristics: II. Compact [2]
1. Rigid [3]	Method I.1 (see 4.3)	Method II.1 (see 4.5)
2. Flexible [4]	Method I.2 (see 4.4)	Method II.2 (see 4.6)

1 A structure is considered hydrodynamically transparent if wave scattering can be neglected, i.e., if the structure does not significantly modify the incident wave. This condition is satisfied if $D/L < 0{,}2$, where D is a characteristic dimension of the structure (e.g., diameter of structural members, such as platform legs) in the direction of wave propagation, and *L* is the wave length.

2 A structure is considered hydrodynamically compact if scattering of sea waves cannot be neglected. This condition occurs if $D/L > 0{,}2$. If D/L is about 0,2, a design value may be defined as the smaller one of the design values obtained from the alternative application of methods I and II.

3 A structure is considered rigidly positioned if the wave-induced deviation from its equilibrium position in still-water is negligibly small when compared to the wave height.

4 A structure is considered flexibly positioned if the wave-induced deviation from its still-water equilibrium position is of the order of the wave height or more.

F_I = inertia force vector per unit length acting normal to the axis of the member in the plane of the member axis and $\delta U/\delta t$ [kN/m]

C_D = drag coefficient

ρ = density of water [kNs^2/m^4]

A = projected area normal to the cylinder axis per unit length

= D for circular cylinders [m]

V = displaced volume of the cylinder per unit length

= π $D^2/4$ for circular cylinders [m^2]

D = effective diameter of circular cylindrical member, including marine growth [m]

U = component of the velocity vector (due to wave and/or current) of the water normal to the axis of the member [m/s]

| U | = absolute value of U [m/s]

C_M = inertia coefficient

$\delta U/\delta t$ = component of the local acceleration vector of the water normal to the axis of the member [m/s^2]

Note that the Morison equation, as stated here, ignores the convective acceleration component in the inertia force calculation as well as lift forces, impact related slam forces, and axial Froude-Krylov forces.

4.3.2 The vectorial superposition of local wave loads in a phase correct manner defines global wave loads on the structure. Total base shear and overturning moment are calculated by a vector summation of local drag and inertia forces due to waves and currents, dynamic amplification of wave and current forces (see Section 3, H.), and wind forces on the exposed portions of the structure (see 2.).

Slam forces can be neglected because they are nearly vertical. Lift forces can be neglected for jacket-type structures because they are not correlated from member to member. Axial Froude-Krylov forces can also be neglected.

The wave crest shall be positioned relative to the structure so that the total base shear and overturning moment have their maximum values. The following aspects need to be considered:

– The maximum base shear may not occur at the same wave position as the maximum overturning moment.

– In special cases of waves with low steepness and an opposing current, maximum global structure force may occur near the wave trough rather than near the wave crest.

– Maximum local member stresses may occur for a wave position other than that causing the maximum global structure force. For example, some members of conductor guide frames may experience their greatest stresses due to vertical drag and inertia forces, which generally peak when the wave crest is far away from the structure centerline.

4.3.3 If the Morison equation is used to calculate hydrodynamic loads on a structure, the variation of C_D as a function of Reynolds number, Keulegan-Carpender number, and roughness shall be taken into account in addition to the variation of the cross-sectional geometry:

Reynolds number $R_n = U_{max}D/\nu$

Keulegan-Carpenter number $K_C = U_{max}T/D$

Relative roughness $= k/D$

where

D = diameter [m]

T = wave period [s]

U = maximum of velocity U defined in 4.3.1 [m/s]

ν = kinematic viscosity of water

$\approx$ $1{,}4 \cdot 10^{-6}$ at 10° C [m^2/s]

k = roughness height [m]

4.3.4 As guidance in determining drag coefficients, surface roughness values listed in Table 2.3 may be used.

Table 2.3 Surface roughness

Surface	k
Steel, new uncoated	$5 \cdot 10^{-6}$
Steel, painted	$5 \cdot 10^{-6}$
Steel, highly rusted	$3 \cdot 10^{-3}$
Concrete	$3 \cdot 10^{-3}$
Marine growth	$5 \cdot 10^{-3}$ to $5 \cdot 10^{-2}$

4.3.5 For smooth circular cylinders in steady flow, where the cylinder length is significantly larger than the cylinder diameter, Table 2.4 lists design values of hydrodynamic coefficients C_D and C_M for two classes of Reynolds number.

Table 2.4 Hydrodynamic coefficients C_D and C_M for circular cylinders in steady flow conditions

Reynolds number (R_n)	C_D	C_M
$< 10^{-5}$	1,2	2,0
$> 10^{-5}$	0,7	1,5
1. Corrections due to marine growth have to be made, as applicable. 2. C_D and C_M values for more complicated structural elements shall be provided by competent institutions. These values are to be approved by Germanischer Lloyd.		

4.3.6 For circular cylinders in supercritical oscillatory flow at high values of K_C, with in-service marine roughness, Table 2.5 lists design values of hydrodynamic coefficients C_D and C_M.

Table 2.5 Hydrodynamic coefficients C_D and C_M for circular cylinders in oscillatory flow conditions

Surface condition	C_D	C_M
Multiyear roughness, k/D > 1/100	1,05	1,8
Mobile unit (cleaned), k/D < 1/100	1,0	1,8
Smooth member, k/D < 1/10000	0,65	2,0
The smooth values will normally apply 2 m above the mean water level and the rough values 2 m below the mean water level. The roughness for a mobile unit applies when marine growth is removed between submersions of members.		

4.3.7 For several cylinders close together, group effects may be taken into account. If no adequate documentation of group effects for the specific case is available, the drag coefficients for the individual cylinder may be used.

4.3.8 The hydrodynamic modeling of jack-up legs may be carried out by utilizing the 'detailed' or the 'equivalent' technique. The 'detailed' technique models all relevant members with their own unique descriptions for the Morison term values and with the correct orientation to determine relative normal velocities and accelerations and the corresponding drag and inertia coefficients, as defined in 4.3.6. The hydrodynamic model of the 'equivalent' technique comprises one 'equivalent' vertical tubular located at the geometric center of the actual leg. The corresponding (horizontal) relative velocities and accelerations are applied together with equivalent hydrodynamic coefficients (C_D and C_M) and equivalent diameters (D). The model shall be varied with elevation, as necessary, to account for changes in dimensions, marine growth thickness, etc.

For non-tubular geometries, such as split tube and triangular leg chords, the appropriate hydrodynamic coefficients may, in lieu of more detailed information, be assessed from relevant literature and/or appropriate interpretation of (model) tests. The SNAME (2002) Technical & Research Bulletin 5-5A "Guidelines for Site Specific Assessment of Mobile Jack-Up Platforms" is an example of a document that provides such information.

4.4 Method I.2: Flexibly positioned, hydrodynamically transparent structures

4.4.1 Depending on the kind of structural elements considered in a strength analysis, this method requires calculation of wave-induced first order motions (see 4.4.2) as well as second order motions of the moored structure (see 4.4.3).

4.4.2 Loads on structural elements not belonging to the mooring system

Wave-induced design loads used for strength analysis of structural elements that do not belong to the mooring system may be estimated by a phase correct vectorial superposition of all local wave loads acting on the rigidly positioned (fixed) structure according to 4.3, Method I.1. The corresponding design loads for all motion-induced loads may be calculated on the basis of a linear motion analysis. Such an analysis shall account for all relevant degrees of freedom of the structure being considered. Typical offshore structures may be classified according to the number of degrees of freedom of rigid-body motions as listed in Table 2.6:

Table 2.6 Degrees of freedom of different kinds of offshore structures

Degrees of freedom	Kind of structure
6	Drill ships, supply vessels, tugs Anchored or dynamically positioned semisubmersibles Loading buoys, storage tanks Floating cranes, pipelaying vessels
3	Tension-leg platforms Guyed towers
2	Articulated towers

For a six degree-of-freedom system the motion analysis may generally be based on the following linear motion equations:

$$\sum_{j=1}^{6}\left[-\omega_D^2.(M_{jk}+A_{jk})+i.\omega_D.B_{jk}+C_{jk}\right]\cdot s_{Dk}=F_j$$

for $j=1,\ 2\ldots 6$

where

j = index denoting the direction of the force

k = index denoting the direction of the motion

ω_D = circular frequency of motion

= $2\pi / T_D$

T_D = period of the design wave

M_{jk} = generalized mass matrix

A_{jk} = generalized added mass matrix

B_{jk} = generalized damping matrix

C_{jk} = generalized restoring force matrix

s_{Dk} = generalized vector of motion amplitudes

F_j = generalized global wave-induced load vector, determined according to 4.3

To solve these equations for the motion components s_{Dk}, the generalized masses M_{jk} of the structure (including mass moments of inertia for i, j > 3), the respective global hydrodynamic added masses A_{jk}, the global damping coefficients B_{jk} (viscous damping for hydrodynamically transparent structures), and the global hydrostatic (restoring) coefficients C_{jk} need to be defined. These global hydromechanic coefficients may be obtained from calculation of local loads on all structural elements below the still-water line. Linear superposition of all local hydromechanic coefficients determines the required global hydromechanic coefficients. Details can be found in related text books. Appropriate computer codes or a compatible procedure may be applied upon agreement with Germanischer Lloyd.

After determination of the motion components, motion induced local loads on all structural elements can be estimated and added to the local wave loads as determined for the rigidly positioned (fixed) structure, as applicable, finally yielding the resulting wave induced (first order) local design loads on structural elements.

4.4.3 Loads on structural elements belonging to the mooring system

Wave-induced design loads used for strength analyses of structural elements that belong to the mooring system shall be estimated by superposition of wave loads due to first and second order (drift) motions. The second order motion analyses shall include restoring forces of the mooring system, which may be linearized upon agreement with Germanischer Lloyd. Simulation techniques with statistical evaluation or equivalent spectral analysis methods are acceptable. Details are subject to approval by Germanischer Lloyd.

Selected model tests are recommended to determine (validate) the influence of damping.

4.5 Method II.1: Rigidly positioned, hydrodynamically compact structures

4.5.1 Linear wave-induced loads on large body, hydrodynamically compact structure shall be determined by diffraction theory. Diffraction theories may be based on sink-source methods or finite fluid volume methods. For simple geometric shapes, analytical solutions may be used. For surface piercing bodies, results from sink-source methods shall be checked to avoid unreliable predictions in the neighborhood of irregular frequencies. Details can be found in related text books. Appropriate computer codes or a compatible procedure may be applied upon agreement with Germanischer Lloyd.

Model experiments are recommended if a new structural concept is to be analyzed and the loads cannot be adequately predicted by state of the art methods.

4.5.2 Wave-induced loads on structures comprising large volume parts and slender members may be obtained by combining the use of the wave diffraction theory and the Morrison equation. The modified velocities and accelerations caused by the large volume parts, however, should be accounted for when using the Morrison equation.

4.5.3 Hydrodynamic interaction between large volume parts should be accounted for.

4.5.4 In the vicinity of large bodies the free surface elevation (i.e., wave height) may be increased. This increase shall be accounted for not only in the wave-induced load calculation, but also when estimating the under deck clearance.

4.5.5 In many cases second order wave-induced loads may be important for the design of hydrodynamically compact structures.

– In addition to first order (linear) loads, mean (nonlinear) second order wave (drift) loads and nonlinear loads varying in time with twice the first order (wave) frequency act on the structure. Wave-induced load effects of higher order than two are usually neglected.

– In a natural (irregular) seaway, composed of an infinite number of elementary regular wave components, the resulting second order wave-induced loads contain three components, namely, the mean (drift) loads, loads varying in time with the difference frequencies of component waves (slowly varying drift loads), and loads varying in time with the sum frequencies of component waves (high frequency loads).

– The sum frequency loads may be important in considering wave load effects on certain offshore structures, such as deep water gravity structures.

– Second order loads shall be determined by a consistent second order theory or by appropriate model tests.

4.6 Method II.2: Flexibly positioned, hydrodynamically compact structures

4.6.1 Depending on the kind of structural elements considered in a strength analysis, this method requires calculation of wave-induced first order motions (see 4.5.1) as well as second order motions of the moored structure (see 4.5.5).

4.6.2 Wave induced local design loads used for strength analysis of structural elements that do not belong to the mooring system may be estimated on the basis of the linear motion analysis as outlined in 4.4.2, with F_j representing the components of the global wave load vector determined according to 4.5.

Contrary to 4.4.2, global hydrodynamic coefficients A_{jk} and B_{jk} may be determined directly by diffraction theory based methods. Details can be found in related text books. Appropriate computer programs or compatible procedures may be used upon agreement with Germanischer Lloyd.

4.6.3 The second order (slowly varying) difference frequency wave-induced loads (see 4.5.5) may be important for the design of mooring or dynamic positioning systems as well as for offshore loading systems. For hydrodynamically compact structures with a small waterplane area, the slow drift forces may cause large vertical motions.

Second order loads shall be determined by a consistent second order theory or by appropriate model tests.

4.7 Spectral methods may be used to obtain the response in natural (irregular) seas by linearly superposing results from regular wave components of different amplitudes, wave lengths, and propagation directions, assuming the natural seaway is made up of a large number of regular waves of different length and height. The procedure to follow is outlined in Section 1, D.5.

4.8 Wave impact loads

Structural members in the splash zone are susceptible to loads caused by wave-induced impact (slamming) when the member is submerged. These loads are difficult to estimate accurately and should generally be avoided by structural design measures. Long horizontal members in the splash zone are particularly susceptible to impact related loads.

4.9 For a cylindrical shaped structural member the slamming force per unit length may be calculated as follows:

$$F_S = \tfrac{1}{2} \cdot \rho \cdot C_S \cdot D \cdot v^2$$

where

F_S = slamming force per unit length in the direction of the velocity v [kN/m]

ρ = density of water [kNs^2/m^4]

C_S = slamming coefficient

D = member diameter [m]

v = relative velocity normal to member surface [m/s]

For a smooth circular cylinder the slamming coefficient shall be at least $C_S = 3{,}0$.

4.9.1 The slamming pressure space averaged over a broader area (several plate fields) may be obtained as follows:

$$p_S = \tfrac{1}{2} \cdot \rho \cdot C_{ps} \cdot v^2$$

where

p_S = space averaged slamming pressure [kPa]

ρ = density of water [kNs^2/m^4]

C_{ps} = space averaged slamming pressure coefficient

v = relative velocity normal to member surface [m/s]

For a smooth circular cylinder the space averaged slamming pressure coefficient shall be at least $C_{ps} = 3{,}0$. For a flat bottom the slamming pressure coefficient shall be at least $C_{ps} = 6{,}3$.

4.9.2 Space averaged slamming pressure coefficients for other shapes shall be determined using recognized numerical and/or experimental methods, taking into account three-dimensional effects. Interface capturing techniques of the volume-of-fluid (VOF) type, for example, are appropriate numerical methods as they take into account viscosity, flow turbulence, and deformation of the free surface.

4.9.3 Fatigue damage caused by wave slamming shall be taken into account. The fatigue damage caused by slamming has to be added to the fatigue contribution from other viable loads.

4.10 Impact related pressure from breaking waves

Breaking waves causing impact pressures on vertical surfaces shall be considered. In the absence of more reliable methods, the procedure described in 4.8 may be used to calculate this impact pressure. The coefficient C_S depends on the configuration of the area exposed to this impact related pressure. The impact velocity v shall correspond to the most probable largest wave height.

4.11 Air gap

4.11.1 Air gap for units

The air gap is defined as the minimum clearance between the underside of the unit in the elevated position and the calculated crest of the design wave with adequate allowance for safety. It is usually not practical to change the air gap in preparation for a storm and, therefore, the air gap for an intended operation shall be determined based on a suitable return period. Omnidirectional guideline wave heights with a return period of 100 years are recommended to compute wave crest elevations above storm water level, even if a lower return period is used for other purposes.

For jack-up platforms, the air gap shall be 1,20 m or 10 percent of the combined storm tide, astronomical tide, and height of the maximum wave crest above mean low water level, whichever is less. An additional air gap shall be provided for any known or predicted long-term seafloor subsidence.

4.11.2 Air gap for installations

The air gap is the clearance between the highest water surface that occurs during the extreme environmental conditions and the lowest part not designed to withstand wave impingement. For fixed platforms, the air gap to be maintained shall be at least 1,5 m.

5. Vibratory loads due to vortex shedding

Flow past a structural member may cause unsteady flow patterns from vortex shedding, leading to vibrations of the member normal or in line to its longitudinal axis. Such vibrations shall be investigated.

At certain critical flow conditions, the vortex shedding frequency may coincide with or be a multiple of the natural frequency of motion of the member, resulting in harmonic or subharmonic resonance vibrations.

The vortex shedding frequency in steady flow with Keulegan-Carpender number K_C (see 4.3.3) greater than 40, such as in a current, may be obtained as follows:

$$f = S_n \cdot (U / D)$$

where

f = vortex shedding frequency [HZ]

S_n = Strouhal number

U = local flow velocity normal to the member axis [m/s]

D = diameter of the member [m]

At certain critical velocity ranges, the vortex shedding will be such as to lock-in to the natural frequency of the member. This lock-in to the natural frequency can occur for flows in line as well as transverse to the member.

Vortex shedding on structural members may also be generated in waves. Vortex shedding occurs in that part of the wave motion where the acceleration is small. Vortex shedding in waves falls into two categories. For $K_C > 40$, the vortex shedding is of the same kind as in a steady current. For $6 < K_C < 40$, vortex shedding frequency depends on the type of wave motion. One distinguishes between two limiting cases:

– If the wave motion can be considered regular, the vortex shedding frequency is a multiple of the wave frequency.

– If the wave motion is irregular, the vortex shedding frequency is the same as the in a steady current, and the frequency depends on S_n.

For irregular waves characterized by a narrow band spectrum, the vortex shedding frequencies are a combination of these two limiting cases.

To reduce the severity of flow-induced oscillations caused by vortex shedding, either the member's shape or its structural properties should be changed. This can be achieved by altering the member's mass or the member's damping characteristics. As a result, its natural frequency will be changed so as to lie outside the range for the onset of resonant vortex shedding.

Spoiling devices are often used to suppress vortex shedding lock-on. The underlying principle is to either reduce drag on the member or to cause the shed vortices to become uneven over the length of the member. Drag can be reduced by adding streamlined fins and splitter plates that break the oscillating flow pattern. Uneven vortex shedding can be induced by making the member irregular, i.e., by adding spoiling devices, such twisted fins, helical strakes, or ropes wrapped around the member. Tests should determine the efficiency of the spoiling device to be used.

6. Loads due to climatic conditions

6.1 Snow and ice accretion

6.1.1 If icing is possible, and ice or snow covers parts of the structure, the weight of ice or snow shall be added to the permanent load under any loading conditions.

6.1.2 Snow can settle on both horizontal surfaces and, if the snow is sufficiently wet, on non-horizontal windward parts of installations or units. On horizontal surfaces, dry snow is blown off as soon as any thickness accumulates, while wet snow can remain in position for several hours. Snow covering on surfaces inclined more than 60° to the horizontal can be disregarded. Snow covering on surfaces inclined between 0 and 60° can be reduced linearly. Snow can freeze and remain as ice only on vertical surfaces.

6.1.3 Ice on the topsides of installations or units can accumulate through a number of mechanisms:

– freezing of old wet snow

– freezing sea spray

– freezing fog and super cooled cloud droplets

– freezing rain

Where ice covering is caused mainly by sea water spray, it may be taken to decrease linearly to zero from a level corresponding to the highest wave elevation to a level corresponding to 60 m above the highest wave elevation.

The effects of topside icing on the stability of floating structures and on the operation of emergency equipment are particular aspects that shall be considered when designing for operations in cold climates.

6.1.4 Loads caused by ice and/or snow on open decks and externally exposed walls are to be specified according to recommendations of competent independent authorities/institutions. In absence of specific information, new snow may be assumed to have a density of 100 kg/m³, and the average density of ice formed on the structure may be taken as having a density of 900 kg/m³.

6.2 Temperature influences

Stresses in and deformations of the structure or parts of it, which are induced by extreme temperature gradients in the structure, shall be added to the permanent load induced stresses and deformations under operating conditions, where they are deemed to be relevant.

6.3 Marine growth

6.3.1 Marine growth may be considerable in some areas and shall be taken into account, e.g., when assessing the wave and current loads acting on submerged parts of the structure. Relevant information shall be submitted to Germanischer Lloyd for verification, see also 4.3.6.

6.3.2 Thickness of marine growth shall be assessed according to local experience. If no relevant data are available, a thickness of 50 mm may be chosen for normal climatic conditions.

7. Loads due to sea ice

7.1 Forces exerted on a structure by ice are to be evaluated for their effect on local structural elements and for global effects on the structure as a whole.

7.2 Ice loads are to be evaluated for a range of ice-structure interactions. The range of interactions is determined by the ice environment of the area of operations, see Section 1, G., and may include the following aspects:

- ice pressure loads from continuous first or multi-year level ice
- loads from collision with first and/or multi-year ridges within the ice field
- impact loads caused by drifting ice floes (sea ice or glacial ice)
- impact loads caused by icebergs

7.3 Ice load evaluations are to include the forces exerted by ice on rubble ice or other ice pieces that are in firm contact with or held by the structure. This is of particular concern for multi-legged structures and for structures designed to cause ice failure in modes other than crushing.

7.4 The brittle nature of ice can lead to periodic dynamic loading, even during ice-structure interactions where this is not initially apparent. Dynamic amplification of the structure's response in its natural vibratory modes shall be evaluated.

7.5 When calculating ice loads, the following parameters specifying the condition of the ice have to be considered:

- thickness of a level ice sheet
- temperature or temperature gradient in the ice
- orientation of the ice crystals
- bulk salinity
- total porosity of the ice (brine volume, gas pockets, voids)
- density
- ice strength
- loading rate
- scale effects (size of structure/ice thickness)

7.6 Any attempt to determine ice loads on offshore structures shall be based on probabilistic methods. Ice parameters and corresponding forces shall be analyzed over the entire range of possible parameter values by using simulation techniques, such as the Monte Carlo simulation.

7.7 Exclusively using deterministic methods for ice loads, only considering a maximum ice thickness, introduces uncertainties of the following kind:

- randomness of every natural process
- incomplete ability to describe natural processes in engineering models
- human participation in engineering processes

In case of ice-structure interaction, the most significant uncertainties arise from the available models. It should always be considered that none of the publicly available methods delivers reliable results.

7.8 Model tests are recommended for evaluation of global loads and confirmation of expected ice failure modes.

7.9 Further details are described in Germanischer Lloyd Rules IV – Industrial Services, Part 6 – Offshore Technology, Chapter 7 – Guideline for the Construction of Fixed Offshore Installations in Ice Infested Waters.

C. Permanent Loads

1. Permanent loads are loads acting throughout the lifetime of the unit/installation or during prolonged operating periods. Such loads may comprise the weight of structures (dead load), equipment, permanent ballast, and effects of hydrostatic pressure exerted on parts of the submerged structure (buoyancy).

2. Permanent Loads shall be clearly stated and accounted for in the design documents and calculations.

3. In cases where loads/weights may be acting over longer periods, but not necessarily at all times (e.g., when using certain kinds of equipment), these cases may have to be investigated to account for the most unfavorable condition.

D. Functional Loads

1. Functional loads are variable loads caused by normal operations in a variable manner as follows:

– weight of tools and mobile equipment

– stores, frequently varying ballast, fuel, wastes

– loads from operations of cranes and other conveyance equipment

– loads from transport operations, e.g., helicopters

– mooring/fendering loads from vessels serving the unit/installation

2. Loads on windlasses, etc., exerted by mooring lines on mobile units, shall generally be regarded as functional loads, although they are mainly due to environmental influence. Regarding permissible stresses, these mooring loads shall be attributed to loading conditions 2 or 3 as applicable, see Section 3, C. and Chapter 2 – Mobile Offshore Units, Section 8.

3. Deck loads, weight of equipment, etc. shall be specified by the Owner/Designer. The specification shall also contain indications of permissible load combinations and limitations regarding the overall weight of the structure or installation.

Any such limitations shall also be stated in the Operating Manual, see Chapter 2 – Mobile Offshore Units, Section 1, C. and Chapter 3 – Fixed Offshore Installations, Section 1, C. The most unfavorable distribution of loads is to be accounted for in the structural analysis.

4. If not otherwise specified, the following minimum loading on deck surfaces is to be taken:

– crew spaces, walkways, etc.: 4,5 kN/m^2

– access platforms: 3 kN/m^2

– working areas: 9 kN/m^2

– storage areas: 13 kN/m^2

– helicopter platform: 2 kN/m^2

For railings and foot rails, a horizontal load in form of a moving single load of 0,3 kN/m has to be assumed.

The above loads on walkways, stairs, access platforms, and similar areas shall be fully considered for the local design; but for the overall design of the installation/unit, a minimum of 50 % of the live loads on these areas shall be considered.

5. Layout plan for loads

As indicated in Chapter 2 – Mobile Offshore Units, Section 1, C.2. and Chapter 3 – Fixed Offshore Installations, Section 1, C.2., a layout plot plan, including weights, shall be prepared for each design. This plan is to show the permanent design loads, see C., and functional maximum design loads (uniform loads as well as concentrated loads) for all areas and for each mode of operation.

E. Accidental Loads

1. General

1.1 Accidental loads are loads not normally occurring during the installation and operating phases, but which shall be taken into account, depending on location, type of operations, and possible consequences of failure. Causes for accidental loads within this context may be the following:

– collisions (other than normal mooring impacts referred to under D.1.)

– falling or dropped objects

– failing or inadequate crane operations

– explosions, fire

– strength level earthquakes according to Section 1, H.3.

1.2 The installation/unit shall be so designed and, if necessary, protected that the consequences of damage are acceptable and that an adequate margin of safety against collapse is maintained.

2. Collision of vessels with offshore installations

2.1 Collision zone

Accidental damage shall be considered for all exposed elements of an installation in the collision zone. The vertical extent of the collision zone shall be assessed on the basis of visiting vessel draft, maximum operational wave height, and tidal elevation.

2.2 Accidental impact energy

2.2.1 Total kinetic energy

The total kinetic energy involved in accidental collisions can be expressed as follows:

$$E = \tfrac{1}{2} \cdot a \cdot \Delta \cdot v^2$$

where

E = total kinetic energy [kJ]

Δ = displacement of the vessel [t]

a = vessel added mass coefficient

= 1.4 for sideways collision

= 1.1 for bow or stern collision

v = impact speed [m/s]

The minimum value for E shall be as follows:

– 14 MJ for sideways collisions and

– 11 MJ for bow or stern collisions

These minimum values correspond to a vessel of 5000 t displacement with an impact speed of 2 m/s. A reduced impact energy may be acceptable if the size of the visiting vessels and/or their operations near the offshore installation are restricted. In this instance, a reduced vessel size and/or a reduced impact speed may be considered.

2.2.2 Impact speed

The reduced impact speed may be estimated from the following empirical relationship:

$$v = ½ \cdot H_S$$

where

v = impact speed [m/s]

H_S = significant wave height [m]

Here the significant wave height H_S is to be taken equal to the maximum permissible significant wave height for vessel operations near to the installation.

2.2.3 Energy absorbed by the structure

The energy absorbed by the installation during the collision impact will be less than or equal to the impact kinetic energy, depending on the relative stiffness of the relevant parts of the installation and the impacting vessel that the installation comes in contact with, the mode shape of the collision, and the vessel's operation. These factors may be taken into account when considering the energy absorbed by the installation.

For a fixed steel jacket structure, it is recommended that the energy absorbed by the installation should not be taken less than 4 MJ unless a study of the collision hazards and consequences specific to the installation demonstrates that a lower value is appropriate. Examples of such structures are small, occasionally manned and infrequently supplied installations, or structures where there is a restriction on the size of the visiting vessel.

For vessels of less than 5000 t displacement, the minimum energy to be absorbed by the installation may be reduced to a value given by the following relationship:

$$E_a = 0.5 + \Delta^2 (4{,}2 \times 10^{-7} - 5{,}6 \text{ x } 10^{-11} \Delta)$$

E_a = energy absorbed by the installation [MJ]

Δ = displacement of the vessel [t]

The energy absorbed by the installation may be taken less than this value if the stiffness of the impacted part of the installation is large compared to the stiffness of the impacting part of the vessel, as for example in collisions involving concrete installations or fully grouted installation elements. In such cases the effects of impact loading shall be considered in detail as in the following.

2.3 Accidental impact loading

In cases where the stiffness of the impacted part of the installation is large compared to stiffness of the impacting part of the vessel, as for example in collisions involving concrete installations or fully grouted elements, the impact energy absorbed locally by the installation may be low. In such cases, it is important to examine damage caused by the impact force.

The impact force may be calculated as follows:

$$F = P_0$$

or

$$F = v \cdot \sqrt{c \cdot a \cdot \Delta}$$

whichever is the less, where

F = impact force [kN]

P_0 = minimum crushing strength or punching shear, as appropriate, of the impacting part of the vessel and the impacted part of the installation [kN]

v = impact speed [m/s]

c = total spring stiffness of the vessel and installation related to the impact point [kN/m]

a = vessel added mass coefficient, see 2.2.1

Δ = displacement of the vessel [t]

3. The regulations of competent Authorities and/or Administrations have to be observed, particularly regarding collisions.

4. According to Section 3, C.4., loading condition 4 – Accidental loads, the choice of accident cases and the determination of loads will be considered on a case to case basis.

5. Consequences of damage

The primary structure shall be designed to ensure that accidental damage does not cause complete collapse. Damaged members shall be considered to be totally ineffective unless it can be shown by analysis or tests that they retain residual load-carrying capacity.

F. Transportation and Installation Loads

1. General

Marine operations for transportation and installation are routine operations of limited duration. Such operations depend on the type of the offshore installation and shall include all the transient movements and other activities where the structure or operation is at risk in the marine environment. Samples of such operations are the following:

- load-out from shore to barge or vessel
- float-out from drydock
- construction and outfitting afloat
- wet or dry towage and other marine transportation
- installation
- jacket launching
- float-over of topsides
- piling, grouting, connection to permanent moorings
- modifications to an existing structure
- total or partial decommissioning and removal of the structure

2. Loads

The loads for these phases have to be defined on a case by case basis according to the type of installation and the situation on site. In determining these loads in an actual case, the following aspects shall be considered:

- The required equipment, vessels, and other means involved are to be designed and checked for adequate performance with respect to their intended use.
- Redundancy in the equipment shall cover possible breakdown situations.
- Operating weather conditions, chosen at values smaller than the specified design criteria, are to be forecasted for a period long enough to complete the required marine operations.
- The marine operations are planned, in nature and in duration, such that accidental situations, breakdowns, or delays have a low probability of occurrence and are covered by detailed contingency actions.
- Adequate documentation has been prepared for a safe step-by-step execution of the operation, with clear indications of the organization and adopted chain of command.
- Marine operations are to be conducted by suitably experienced and qualified personnel.

G. Earthquake Loads

1. Seismic actions

The characteristics of earthquake levels (acceleration intensity, duration, etc.) shall be established by a competent Authority and agreed upon with Germanischer Lloyd.

2. Seismic analysis

2.1 General

Two levels of seismic loading on a structure shall be considered in accordance with 1.:

2.1.1 Strength level earthquake

Strength level earthquakes (SLE) shall be assessed as an ultimate strength condition (ULS). In zones with low to moderate seismic activity, the action effects obtained from an analysis in which the structure is modeled as linear and elastic will normally be such that the structural design can be performed based on conventional linear elastic strength analyses, employing normal design and detailing rules for the reinforcement design.

2.1.2 Ductility level earthquake

Provided the structure survives, it is permissible to assess ductility level earthquakes (DLE) with ductile behavior of the structure, assuming extensive plastic deformation occurs.

If, under the DLE event, ductile response of specific components of the structure is predicted or considered in the analysis, such components shall be carefully detailed to ensure appropriate ductility characteristics and structural constraints. Expected best estimate of stress/strain parameters associated with ductile behavior may be adopted in the analysis. Due consideration shall be given to the effects of excessive strength with respect to the transfer of forces acting at adjoining members as well as to the design of those failure modes of such members that are not ductile, such as shear failure. For those cases where the structure can be designed to the DLE event by applying normal ULS criteria, no special detailing for ductility is required.

2.1.3 Seismic response

Seismic events may be represented by input response spectra or by time histories of significant ground motion. Where the global response of the structure is essentially linear, a dynamic spectral analysis shall normally be appropriate. Where nonlinear response of the structure is significant, a transient dynamic analysis shall be performed.

Section 3

Principles for Structural Design

A. General

1. Scope

This Section defines the principles for structural design of fixed offshore installations. Subsections A. - C. are applicable to structures in general. Subsections D. - I. refer to steel structures. If structures made of aluminium alloys shall be provided, the design should follow the GL Rules I – Ship Technology, Part 1 – Seagoing Ships, Chapter 1 – Hull Structures or recognized standards, like DIN 18800. The requirements for concrete structures are given in Section 5.

The design principles for mobile offshore units are defined in Chapter 2, Sections 2 to 6.

2. General design considerations

2.1 Structural integrity throughout the lifetime of an offshore installation depends on the thoroughness of design investigations, on the quality of materials and manufacture, and on the in-service inspection and maintenance. This Section deals with the required level of design investigations.

2.2 Serviceability and functional considerations may have influence on the structural design. They will generally be specified by the Owner/Operator of the installation, but will be taken into account in the design review by GL where relevant.

2.3 Loads and loading conditions should generally be treated in accordance with Sections 1 and 2. More specific or especially adapted load criteria/ values shall be well documented and agreed upon. The load cases and combinations to be considered in the design calculations shall cover the most unfavourable conditions likely to occur.

2.4 The dynamic nature of some loads acting on the structure has to be taken into account by suitable calculation methods where these loads are considered important, e.g. because they produce considerable accelerations of (parts of) the structure. In some cases dynamic forces can be accounted for quasi-statically, i.e. using adequate increments on static forces acting simultaneously, see I.

2.5 Strength/stress analysis may be carried through according to different, recognized methods and standards, see B., and Chapter 1, Section 1, E.4. However, it shall be ascertained in every case that the design fundamentals and standards used are consistent and compatible for one structure or installation, regarding the considerations listed in 2.1.

2.6 Structural redundancy shall be considered at an early stage. As redundancy is not explicitly taken account of in the design methods currently in use, with the exception of the categories grouping of structural elements (special, primary, secondary - see Section 4, A.), the consequence of a failure of individual structural elements will have to be specially considered.

3. Avoidance of wave impact

Where structures are not or cannot be suitably designed to resist wave impact, the latter shall be avoided by providing sufficient distance between the probable highest wave elevation and the lower edge of the structure (e.g. lowest closed deck). A clearance - "air gap" - of 1,5 m is recommended. In case of structures not connected rigidly to the sea floor, their vertical motions shall be accounted for. For mobile units, see also Chapter 2, Section 2 and 3.

4. Corrosion and wear allowances

Determination of scantlings, using the following indications or equivalent design methods, is based on the supposition that an accepted corrosion protection system is provided for, see Section 6. Otherwise adequate corrosion allowances are to be agreed upon, depending on the environmental and operating conditions. The same applies to structural elements prone to wear, e.g. by chafing of cables or chains.

5. Loading conditions

Loading conditions occurring during construction, transport or sea installation may have to be considered in the design, see also C.2. and Section 10.

B. Design Methods and Criteria

1. Choice of design concept/methods

1.1 Structural strength calculations may generally be based on linear elastic theory. However, non-linear relations between loads and load effects shall be properly accounted for, where they are found to be important. Where plastic design is used, the limitations described in 4. are to be observed.

1.2 Where deterministic methods incorporating global safety factors ("allowable stress design") are used, they shall comply with the requirements given in C. to G.

1.3 Semi-probabilistic design methods, usually in association with limit state and partial safety factor design, see 2. and 3., may be applied using adequate codes. The relevant data and safety factors shall be agreed upon with GL.

1.4 Probabilistic methods may be used in special cases only, after consultation with GL.

2. Design criteria, limit states

2.1 A structure may become unsafe or unfit for use by damage or other changes of state according to different criteria. They may be defined by "Limit States", i.e. states of loading, straining (deformation) or other impairment, at which a structure, or structural component, looses its planned operability or function. The limit states are usually classified as follows.

2.2 Ultimate limit states

Ultimate limit states are related to the maximum load carrying capacity of a structure or structural component, which may be reached by:

– loss of equilibrium (e.g. toppling) of the structure, considered as a rigid body

– yielding and/or fracture of material, see D. and E.

– buckling (structural instability)

– fatigue (formation of cracks after repeated loading), see H.

The design requirements for the ultimate limit state (structural) may be proven by elastic or, where applicable, by plastic theory, see 4.

2.3 Serviceability limit states

Serviceability limit states refer to the impairment of normal use or operation by changes or effects other than structural failure, such as

– excessive deformation or displacement

– unacceptable vibrations, see I.

– leakage (loss of tightness)

2.4 Limit state of progressive collapse

The limit state of progressive collapse is related to a collapse of a structural system after accidental damage such as:

– dropped objects

– collisions

– explosions

This limit state may also be considered for the evaluation of the "maximum credible earthquake", see Section 1, H.3.

2.5 Corrosion may lead to either an ultimate or serviceability limit state.

3. Definition of safety format

3.1 In connection with deterministic as well as semi-probabilistic design methods the safety condition is based on statistically representative "characteristic values" of the loading effects and of the resistance of the structure, as well as on safety factors as defined below.

3.2 In deterministic design the safety condition may be expressed as follows:

$$\gamma_g \cdot S_C / R_C \leq 1$$

S_C = characteristic loading effect (e.g. a stress) resulting from loads/load combinations as defined in these Rules, see C. and Section 2.

R_C = characteristic resistance (for instance, the yield strength, the ultimate strength or the critical buckling force or stress)

γ_g = global safety factor (taking account of all uncertainties in loads and resistance estimation)

= numerical values are given in D. to G.

3.3 In semi-probabilistic design methods the safety condition may be expressed as follows:

$$S_d / R_d \leq 1$$

S_d = S(F)

= function, e.g. stress, of the total design loading effect

F = $\sum(F_i \cdot \gamma_{fi})$

= total design loading effect consisting of various characteristic loads F_i, multiplied by individually defined load factors γ_{fi} which take account of the uncertainties of the loads and the probability of their simultaneous occurrence

R_d = R_C / γ_m

R_C = characteristic resistance, see 3.2

γ_m = partial safety factor for materials (covering the possible deviations of material properties for characteristic, statistically determined values)

Numerical values for γ_f and γ_m are to be agreed with GL.

4. Plastic design

4.1 A design using additional plastification resistance may be accepted provided the following conditions are met:

– It shall be possible to establish failure mechanisms with well-defined "hinges". At plastic hinge locations, the cross-section of the members shall have sufficient rotation capacity to enable the required plastic hinge to develop without local buckling.

– The material employed at the points of possible plastic deformations shall be sufficiently ductile.

– The detail construction and/or the nature of the loads shall be such that fatigue is prevented.

4.2 Plastic design may be suitable e.g. for structures intended for collision protection, and for earthquake design.

4.3 When plastic design is applied, the safety factors γ_g or γ_m respectively, have to be increased by a factor of 1,15.

5. Modelling of the structure

5.1 The calculation model ("idealization") used has to take account of all main load bearing and stiffening components, and of the relevant supporting and constraining effects.

The degree of subdivision (detailing) should take account of the geometry of the structure and its influence on the load distribution and introduction, of the distribution of external loads, and of the expected stress pattern, see 5.3.

Elements or members considered as being of "secondary" importance may nevertheless have to be accounted for where they have an influence on stress distribution or dynamical properties of the structure.

5.2 Where modelling is made by means of beam elements, the actual rigidities are to be accounted for as precisely as practicable, particularly in way of connections (joints). The effective width of associated plating may be chosen according to accepted standards, see also D. and F.

5.3 Stress concentration effects shall be carefully investigated, either by using adequate calculation methods (e.g. finite elements), or by applying stress concentration factors where proven and applicable design data exist.

5.4 The soil stiffness is to be represented as precisely as practicable when designing structures resting on or penetrating into the sea floor, see also Section 7.

5.5 For calculations analysing the dynamic behaviour of the whole structure (global behaviour), the mass distribution may generally be simulated in a simplified form, using equivalent or lumped masses. Details may have to be agreed upon, see also I.

6. Model tests

Model investigations (tests) may be accepted as a supplement to structural analysis provided that a recognized institution is involved and the particulars have been agreed. In special cases model tests may be required.

7. Superposition of load effect components

Stresses or other load effects resulting from relevant global and local effects shall be added according to their degree of simultaneousness, see also C. and Section 2, A.4. Stresses of different type (e.g. tension, shear) and different direction shall be combined according to recognized criteria (equivalent stress, v. Mises).

C. Loading Conditions

1. Global or partial safety factors as defined in B. and D. to G. are related to specified loading conditions and load combinations, see 4. and Table 3.1. For kinds and definition of loads see Section 2.

2. The following loading conditions refer to the intended service - operation phase - of the installation. For the construction and installation phase, and eventually for a demobilization phase, safety factors or allowable stresses will be agreed upon according to the probability of occurrence of the relevant loads, the importance of the structural element and the possible consequences of failure. Generally, the inherent safety for the construction and installation phase shall not be less than according to loading condition 3.

3. Limiting operating conditions

The limiting operating conditions, i.e. environmental conditions tolerable during specified operations, and the kind of (limited) operations to continue during extreme environmental conditions, will generally be defined by the Owner or Operator, checked by GL within the design review, and established in the operating instructions.

4. Definition of loading conditions

Seven loading conditions as summarized in Table 3.1 and defined in the following are generally to be considered:

4.1 Loading condition 1: Permanent loads

This (still water) loading condition covers all gravity, buoyancy and hydrostatic loads which would normally act continuously, and shall also take care of variable gravity loads which may be assumed to be in existence during a prolonged period of time, such as ballast and weight of equipment.

Table 3.1 Loading conditions, load combinations

Loading condition	Type of load									
	Dead load	Equip-ment	Vari-able / funct. Loads	Buo-yancy	Environ-mental loads (red.) [5]	Environ-mental loads (extr.)	Impact / collision	Strength level earth-quake	Ductil-ity level earth-quake	Dyn. Load (see I.)
1. Perma-nent loads	×	×	--	× [1]	--	--	--	--	--	--
2. Operating loads	×	×	×	×	×	--	× [6]	--	--	×
3. Extreme environ-mental loads	×	×	× [2]	×	--	×	--	--	--	×
4. Accidental loads	×	×	--	×	--	--	× [3]	× [3]	--	--
5. Transpor-tation/ In-stallation	×	×	--	× [1]	× [4]	--	--	--	--	×
6. Ductility level earth-quake	×	×	--	×	--	--	--	--	×	×
7. Marine Operations	×	×	× [1]	× [1]	× [4]	--	--	--	--	× [7]

1 if applicable
2 functional loads may be reduced
3 alternatively impact collision or strength level earthquake
4 maximum permissible environmental conditions to be defined
5 recurrence period to be defined individually
6 impact during normal operations, e.g. mooring of attending vessels
7 dynamic amplification for lifting see Section 10, E.2.2

For fixed structures this loading condition is generally not relevant and need not to be investigated.

4.2 Loading condition 2: Operating loads

This condition includes the loads as specified in 4.1, plus other variable functional loads such as from crane or drilling operations, supply vessels operating around the installation and those environmental loads which, according to 3., are to be taken into account during normal, unrestricted use of the installation. If not specified individually, a 10 year recurrence period is considered to be acceptable for the environmental loads.

4.3 Loading condition 3: Extreme environmental loads

This loading condition includes the loads as specified in 4.1, the most unfavourable (extreme) environmental loads and those functional loads which, according to 3., are assumed to be also acting in such environment.

According to the principle mentioned in Section 1, A. 3., several possible load combinations or cases may have to be considered.

4.4 Loading condition 4: Accidental loads

This loading condition is intended to cover unusual loads and damage resulting from accidents, such as collisions or fire, in addition to permanent loads, to be defined in the individual case. The kinds of accidents and extent of damages will also have to be agreed upon, possibly together with the competent Administration(s), depending on local conditions. The general philosophy is that immediate total failure of the structure is prevented so that emergency and rescue operations are possible.

An acceptable supposition as to the assessment of associated environmental loads prevailing after the accident is a recurrence period of one year.

In earthquake regions, a "strength level earthquake" (see Section 1, H.3.) shall be considered under this loading condition as an alternative to an impact or collision. The associated environmental and functional loads shall be agreed upon in the individual case.

4.5 Loading condition 5: Transportation/ installation

This loading condition is only temporary and covers the defined environmental conditions during transportation and installation. The situation during transport and installation or de-installation has to be investigated individually case by case.

4.6 Loading condition 6: Ductility level earthquake

This loading condition covers an extreme level of earthquake, defined in Section 1, H.3. The earthquake forms the excitation for the dynamic loads to be investigated according to I. This loading condition cannot be analysed by the allowable stress method.

4.7 Loading condition 7: Marine operations

This loading condition covers operations associated with moving or transporting an offshore structure or part thereof during the construction, installation or abandonment process, compare Section 10.

D. Allowable Stress Design

1. Design concept

When using the conventional deterministic dimensioning method, normally based on linear elastic theory and global safety factors ("allowable stress design"), the allowable stresses shall be taken as follows [1]. The allowable stresses are generally related to the minimum specified yield strength of the material. For materials without a defined yield strength, special agreements will be made.

Please note, that this concept is not applicable for loading condition 6.

As generally defined in B.3.2, the design concept is based on the conditions:

$$\gamma_g \cdot \sigma / R_{eH} \le 1 \quad \text{and} \quad \gamma_g \cdot \tau / R_{eH} \le 1$$

σ, τ = stresses resulting from characteristic loads, see B.3.2

R_{eH} = minimum specified yield strength

= not to be taken greater than $0{,}75 \times R_m$

R_m = minimum tensile strength

γ_g = global safety factor, see 2.

[1] The allowable stress design should generally be applied to mobile offshore units, see Chapter 2, Sections 2 to 6, in accordance with the IACS "Unified Requirements".

2. Safety factors

2.1 In the case of undisturbed stress distribution, i.e. in girders, frames and other structural components where the distribution and magnitude of stresses is reliably obtained from the calculations mentioned under 1., the safety factors γ_g may be chosen according to Table 3.2.

Table 3.2 Global safety factors γ_g (allowable stress design)

Definition of stress	Loading condition acc. to C.4.			
	1	2 / 7	3 / 5	4 [1]
Axial and bending stress (σ_a, σ_b)	1,67	1,45	1,25	1,15
Shear stress (τ)	2,5	2,16	1,90	1,82
Equivalent stress (σ_{eq}) [2]	1,45	1,25	1,10	1,05

[1] Preliminary design value, may have to be specially agreed depending on circumstances (see C.4.4)

[2] The equivalent (combined, v. Mises) stress may be calculated as follows:

$$\sigma_{eq} = (\sigma_x^2 + \sigma_y^2 - \sigma_x \cdot \sigma_y + 3 \cdot \tau^2)^{0,5}$$

(or relevant expression in case of 3-dimensional state of stress)

2.2 The factors are not applicable to local plate bending under lateral pressure. The total stress, including local plate bending, may reach R_{eH} for loading condition 3.

3. In case of predominant compression stresses, the possibility of buckling is to be investigated, see G.

4. Where calculations are carried through, e.g. using finite element methods, in order to determine stress concentrations, see B.5., values up to the yield strength may be admitted for loading condition 3, provided that the material employed at such points corresponds to category "special steel" (see Section 4) and that fatigue considerations (see 5.) are not essential.

5. The allowable stresses may have to be reduced in case of loading repeated with great frequency (fatigue). The admissible stresses may then be obtained according to H.

6. A reduction of admissible stresses may also be necessary

– in case of unfavourable inspection and testing conditions
– in case of not sufficiently well known loads
– where the consequences of a possible failure are deemed to be extremely negative (lack of redundancy, A.2.6.)

7. A higher stress level may be admitted in exceptional cases where the calculations have a high degree of accuracy and the load prediction is extremely conservative.

8. For the admissible stress, other formats than that given in 1. above may be accepted provided that an equivalent safety level is ensured.

9. Specific structural details such as joints of tubes or rolled sections, may be designed according to proven standards and codes (e.g. API-RP 2 A WSD), see also E., Tubular Joint Design. Fatigue considerations will have to be observed additionally, see H.

10. Admissible stresses for elements of special purpose installations such as cranes or drilling rigs may be taken according to acknowledged regulations relating to such equipment. The applicability of relevant loading conditions, in comparison with those defined in C., shall be carefully checked.

E. Tubular Joint Design

1. General

Tubular joints are of primary importance for offshore steel structures.

The following statements are considering the allowable stress design.

In order to avoid stress concentrations and fabrication induced defects the requirements of Section 4 are to be observed. The strength of simple joints without gussets, diaphragms or stiffeners may be investigated as shown in the following.

2. Design punching shear stresses

The design punching shear stress acting in the chord has to be considered for each load component separately and may be calculated as follows (according to API-RP 2 A WSD):

$$\tau_d = (t / T) \cdot \sin \Theta \cdot (\sigma \cdot \gamma)$$

σ = nominal axial stress in brace

= in plane bending stress in brace: σ_{IPB}

= out of plane bending stress in brace: σ_{OPB}

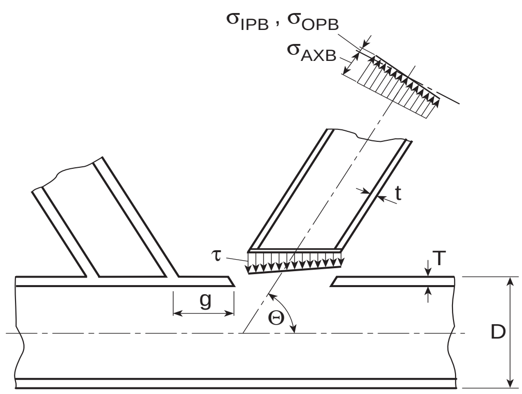
= see Fig. 3.1

γ = γ_g according to Table 3.2

Each stress component has to be considered separately for punching shear evaluation.

3. Allowable shear stresses

3.1 The design punching shear stresses shall not exceed the smaller of the two admissible values calculated according to 3.2 and 3.3.



Θ = brace angle (measured from chord)
g = gap
t = brace thickness
T = chord thickness
d = brace diameter
D = chord diameter
σ = design stress components (brace)

Fig. 3.1 Brace connection; definitions

3.2 Weld connection

The design punching shear shall not exceed the shear resistance (τ_a) of the weld connection:

$$\tau_D \le \tau_a = R_{eH} / \sqrt{3}$$

3.3 Interaction of shear stresses

3.3.1 The punching shear resistance of the chord (τ_p) has to be calculated for each brace load component separately, i.e. axial load (AX), in plane bending (IP), out of plane bending (OP).

The following basic equation is applicable:

$$\tau_P = K_c \cdot K_g (1 / (0{,}3 \cdot D / T) \cdot R_{eH}$$

Factor K_c accounts for nominal longitudinal stresses in the chord:

$$K_c = 1{,}0 - \lambda \cdot A^2 \cdot D / (2 \cdot T)$$

Values for λ depending of the different type of load in brace:

λ = 0,03 for AX

= 0,045 for IP

= 0,021 for OP

$$A = \sqrt{\sigma_{AXC}^2 + \sigma_{IPC}^2 + \sigma_{OPC}^2} / R_{eH}$$

σ^2_{AXC}, σ^2_{IPC}, σ^2_{OPC} = design stress components in the chord

K_c = 1,0 if all combined stresses in the chord (axial plus bending) are tensile

K_g = factor accounting for joint geometry and brace loading

= to be calculated as given in Table 3.3.

Table 3.3 Factor K_g

Type of joint [1]	Type of load in the brace			
	AX Tension	**AX compression**	**IP Bending**	**OP Bending**
K	$[1{,}1 + 0{,}2 / (d/D)] \cdot K_f$		$3{,}72 + 0{,}67 / (d/D)$	$[1{,}37 + 0{,}67 / (d/D)] \cdot K_d$
T, Y	$1{,}1 + 0{,}2 / (d/D)$			
Cross	$1{,}1 + 0{,}2 / (d/D)$	$[0{,}75 + 0{,}2 / (d/D)] \cdot K_d$		

$K_f = 1{,}8 - 0{,}1 \cdot g / T$ if $D / (2 \cdot T) \le 20$

$K_f = 1{,}8 - 4 \cdot g / D$ if $D / (2 \cdot T) > 20$

In no case K_f shall be taken less than 1,0

$K_d = 0{,}3 / [d/D \cdot (1 - 0{,}833 \cdot d/D)]$ if $d/D > 0{,}6$

$K_d = 1{,}0$ if $d/D \le 0{,}6$

[1] all joints: without diaphragm

3.3.2 Design stresses resulting from combined axial and bending stresses in the brace shall be checked according to the following interaction equations:

$$\sqrt{(\tau_d / \tau_p)_{IP}^{\,2} + (\tau_d / \tau_p)_{OP}^{\,2}} \le 1{,}0$$

$$\left|\tau_d / \tau_p\right| + (2/\pi) \cdot \arcsin \sqrt{(\tau_d / \tau_p)_{IP}^{\,2} + (\tau_d / \tau_p)_{OP}^{\,2}} \le 1{,}0$$

τ_d = design punching shear stress, see 2.

τ_p = see 3.3.1

4. If a joint geometry with increased wall thickness is needed, i.e. joint cans or brace stubs are provided, the requirements of Section 4 are to be observed regarding welding and fabrication details, see Fig. 3.2.

5. Overlapping joints

5.1 Definition

Overlapping joints are joints in which part of the load is transferred directly from one brace to another brace through their common weld connection.

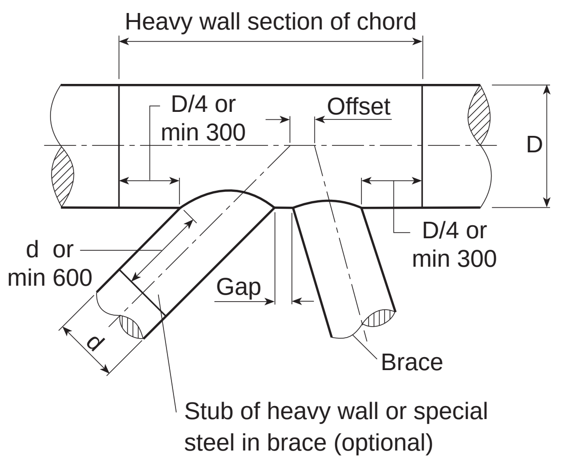


Fig. 3.2 Heavy wall section of chord

5.2 Design

5.2.1 The overlap shall be designed for at least 50 % of the load component F_p acting perpendicular to the chord, see Fig. 3.3.

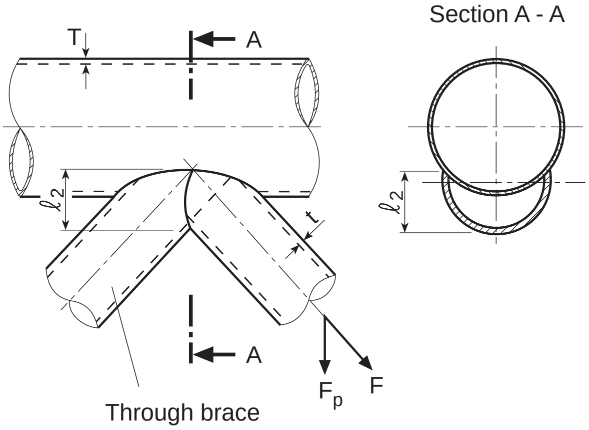


Fig. 3.3 Overlapping joint

5.2.2 In no case the brace wall thickness shall exceed the chord wall thickness.

5.2.3 Where the braces carry substantially different loads and/or one brace is thicker than the other, the heavier brace shall, if possible, be the through brace with it’s full circumference welded to the chord.

5.2.4 Moments caused by eccentricity of the brace centrelines shall be accounted for in the structural analysis.

5.3 Punching shear resistance

For overlapping joints in which brace moments are insignificant, the axial design load component perpendicular to the chord F_{pd} shall not exceed the punching shear resistance R_p:

$$F_{dp} \le R_p$$

$$F_{dp} = F_p \cdot \gamma$$

F_p = nominal axial load perpendicular to the chord, see Fig. 3.3

γ = γ_g for allowable stress design, see Table 3.2

R_p = $\tau_{pax} \cdot T \cdot \ell + 2 \cdot \tau_a \cdot t \cdot \ell_2$

τ_{pax} = punching shear strength for axial loads as defined in 3.3.1

T = chord wall thickness

ℓ = circumference for that portion of the brace which contacts the cord

τ_a = shear resistance of the weld connection, see 3.2

t = the lesser of weld throat thickness or wall thickness t of the thinner brace

ℓ_2 = see Fig. 3.3

6. Complex joints

Complex joints not covered in E. shall be designed on the basis of appropriate theoretical or experimental investigations.

F. Effective Width of Plating

1. Frames and stiffeners

Generally, the spacing of frames and stiffeners may be taken as effective width of plating.

2. Girders

2.1 The effective width of plating e_m of frames and girders may be determined according to Table 3.4 considering the type of loading.

Special calculations may be required for determining the effective width of one-sided or non-symmetrical flanges.

2.2 The effective cross sectional area of plates is not to be less than the cross sectional area of the face plate.

2.3 The effective width of stiffeners and girders subjected to compressive stresses may be determined according to G.3.2.2, but in no case to be taken greater than determined by 2.1.

3. Cantilevers

Where cantilevers are fitted at a greater spacing the effective width of plating at the respective cross section may approximately be taken as the distance of the cross section from the point on which the load is acting, however, not greater than the spacing of the cantilevers.

G. Buckling

1. General indications

1.1 Structural elements subjected to predominantly compressive or shear stresses, which may also result from bending or torsion, are to be investigated for overall and/or local buckling failure as indicated below.

1.2 Deformations due to loads (e.g. lateral load producing bending) or manufacture (fabrication tolerances, welding distortion) shall be taken into account where they may be important, using e.g. second order theory. On the other hand, buckling calculations may result in special manufacturing instructions regarding tolerable imperfections, see 4.2.2 and Section 4, B. and C.

1.3 Local buckling may be admitted where the structure is designed with sufficient redundancy, i.e. where adjoining elements are designed to take over the load originally carried by the buckled part with sufficient safety. Serviceability considerations have to be kept in mind, however, see B.2.3.

1.4 Detailed buckling analysis may be carried through according to recognized codes and standards such as DIN 18800, AISC (Specification for the Design, Fabrication and Erection of Structural Steel for

Table 3.4 Effective width of plating e_m for frames and girders

l / e	0	1	2	3	4	5	6	7	≥ 8
e_{m1} / e	0	0,36	0,64	0,82	0,91	0,96	0,98	1,00	1,0
e_{m2} / e	0	0,20	0,37	0,52	0,65	0,75	0,84	0,89	0,9

e_{m1} is to be applied where girders are loaded by uniformly distributed loads or else by not less than 6 equally spaced single loads.

e_{m2} is to be applied where girders are loaded by 3 or less single loads.

Intermediate values may be obtained by direct interpolation.

l = length between zero-points of bending moment curve, i.e. unsupported span in case of simply supported girders and 0,6 x unsupported span in case of constrained of both ends of girder

e = width of plating supported, measured from centre to centre of the adjacent unsupported fields

Buildings, and Manual of Steel Construction). Some specifications are given in the following, e.g. regarding the correlation between the load conditions defined in these Rules for Offshore Technology and the buckling standard applied. See also A.5.

1.5 In connection with the present Rules, using the conventional deterministic dimensioning method, the global safety factor γ_{gb} for buckling analyses shall be taken from Table 3.5.

In the following safety factors are used as defined in B.3.

For plastic design see B.4.

Table 3.5 Buckling safety factors γ_{gb}

	Loading condition			
	1	**2 / 7**	**3 / 5**	**4**
Safety factor γ_{gb}	1,5	1,5	1,3	1,1

2. Buckling of bar elements

2.1 Buckling modes

Depending on type of element (cross-section), slenderness, type and application of load(s), and boundary conditions, several buckling modes are possible. The critical mode, corresponding to the lowest buckling load, will have to be established by comparative calculations.

2.2 Overall buckling

2.2.1 General remarks

Overall buckling of bar elements is usually to be investigated for the flexural or combined flexural-torsional mode.

For hollow sections and for hot rolled steel members with the types of cross section commonly used for compression members the relevant buckling mode under compression load is generally flexural. Specifications for the proof of buckling strength are given in the following.

However, in some cases, i.e. open, thin-walled cross sections, the torsional or flexural-torsional modes have to be considered by application of relevant standards as mentioned in 1.4.

For rolled I-sections the proof of flexural-torsional buckling strength is not necessary, if

– bending occurs about the z-axis only (vertical axis parallel to the web)

– bending occurs about the y-axis (horizontal axis parallel to the flanges) and the compression flange is supported in y-direction at distances c:

$$c \le (\pi / 2) \cdot \sqrt{E / R_{eH}} \cdot i_z \cdot M_p / (\gamma_{gb} \cdot M_y) \quad [\text{mm}]$$

E = modulus of elasticity

= $2{,}06 \cdot 10^5$ N/mm² for steel

= $0{,}69 \cdot 10^5$ N/mm² for aluminium

i_z = radius of gyration about z-axis of the flange including 20 % of the web area [mm]

M_y = maximum bending moment within the length considered (y-axis)

M_p = plastic resistance bending moment, see 2.2.2.4

γ_{gb} = see 1.5.

2.2.2 Definitions

2.2.2.1 Buckling length l

The buckling length l has to be established according to recognized standards and manuals depending on the end supports and constraint conditions.

For members effectively held in position laterally, normally the full member length is taken. If the member is constrained by relatively stiff adjacent elements, e.g. a brace between platform legs, 80 % of the member length may be assumed as buckling length.

2.2.2.2 Cross section properties

A = cross-sectional area

I = moment of inertia about the considered axis (normally the smallest I).

For thin-walled members the effective cross section properties have to be evaluated by taking into account the effective widths b' of the compression plate elements as follows:

For internal compression plate elements (supported at both sides):

$$b' = \left(\frac{1}{\bar{\lambda}_p} - \frac{0{,}22}{\bar{\lambda}_p^{\,2}}\right) \cdot b \quad \text{or} \quad b' = \frac{\ell}{3}, \quad \text{whichever is less}$$

For outstand compression plate elements (having one free edge):

$$b' = \frac{0{,}7}{\bar{\lambda}_p} \cdot b \qquad \text{or} \quad b' = \frac{\ell}{6}, \quad \text{whichever is less}$$

b = unsupported plate breadth

$\bar{\lambda}_p$ is the reduced plate slenderness given by:

$$\bar{\lambda}_p = \sqrt{\frac{R_{eH}}{K \cdot \sigma_e}} \quad 2$$

K = buckling factor of the plate element of width b, assuming $\alpha \ge 1$ (see Table 3.7).

---

2 For greater economy, the maximum calculated compressive stress in the considered plate element, ($\sigma . \gamma$), may be used instead of R_{eH}, provided that this stress is based on the effective width of all compression elements.

σ_e = $0{,}9 \cdot E \cdot \left(\frac{t}{b}\right)^2$, ideal elastic buckling stress

t = nominal plate thickness.

If the cross section is subjected to an axial load N, the shift e of the neutral axis due to the reduced effective widths b' has to be taken into account by means of the additional moment ΔM:

$$\Delta M = (\gamma_{gb} \cdot N) \cdot e$$

2.2.2.3 The elastic buckling force (Euler) is

$$N_e = \pi^2 \cdot E \cdot I / l^2$$

For thin-walled members the moment of inertia I shall be based on the effective cross section.

2.2.2.4 Plastic resistance

Plastic compression resistance force:

$$N_p = A \cdot R_{eH}$$

Plastic resistance bending moment:

$$M_p = W_p \cdot R_{eH}$$

where

W_p = plastic section modulus.

For thin walled members the cross sectional area and the elastic section modulus of the effective cross section have to be applied.

The effect of shear forces on the plastic section modulus has to be considered.

2.2.2.5 Reduced slenderness $\bar{\lambda}$:

$$\bar{\lambda} = \sqrt{\frac{N_p}{N_e}}$$

2.2.2.6 Reduction factor κ for flexural buckling (buckling curves):

$$\kappa = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \le 1 \quad \text{for } \bar{\lambda} > 0{,}2$$

$$\kappa = 1 \quad \text{for } \bar{\lambda} \le 0{,}2$$

where

$$\phi = 0{,}5 \cdot \left[1 + \alpha \cdot (\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$$

α = factor depending on the appropriate buckling curve

curve a: $\alpha = 0{,}21$

curve b: $\alpha = 0{,}34$

curve c: $\alpha = 0{,}49$

curve d: $\alpha = 0{,}76$

The selection of the buckling curve is shown in Fig. 3.4 for various cross sections.

Fig. 3.5 shows the four different buckling curves.

2.2.3 Buckling under axial compression

The axial compression force N in the structural element, according to the first order elastic theory, is to fulfil the condition

$$(\gamma_{gb} \cdot N) / (\kappa \cdot N_p) \le 1$$

where κ and N_p are as defined in 2.2.2.

2.2.4 Buckling under axial compression and lateral bending

The proof of buckling strength for a structural element subjected to axial compression force N and bending moment M, according to the first order elastic theory, is to be carried out using the following formula:

$$(\gamma_{gb} \cdot N) / (\kappa \cdot N_p) + \beta_m \cdot (\gamma \cdot M) / M_p + \Delta n \le 1$$

where κ, N_p and M_p are as defined in 2.2.2 (κ and M_p are related to the considered bending axis, normally the weakest axis)

β_m = moment coefficient as given in Fig. 3.6, where $N_{ki} = N_e$

Δ_n = $0{,}25 \cdot \kappa^2 \cdot \bar{\lambda}^2$

Values for β_m less than unity may be applied only for structural elements without transverse loads, with rigid supports at both ends, constant cross section and constant axial load.

For pipes the bending moment M shall be the maximum resultant bending moment

$$M = \sqrt{M_y^2 + M_z^2}$$

For the consideration of bi-axial bending of other sections the relevant standards as mentioned in 1.4 should be applied.

2.2.5 Changes of cross section and/or axial load

For members with changes of cross section and/or axial load along the length, all relevant cross sections have to be checked using the respective sectional properties, axial forces and bending moments.

Alternatively, the check of buckling strength may be carried through by application of relevant standards taking into account the effects of changes of cross section and/or load.

2.3 Local buckling

2.3.1 Bars or columns incorporating thin-walled parts in their cross sections may fail by local plate or shell buckling. Such local failure may lead to subsequent overall buckling.

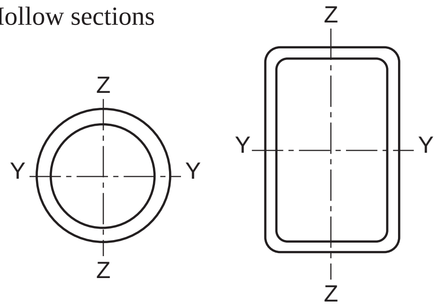
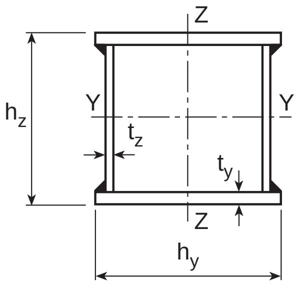
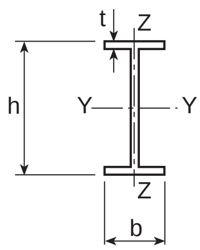
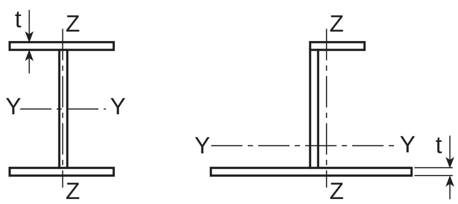
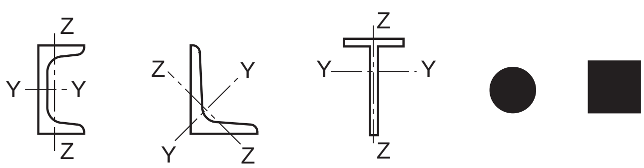
Cross section		**Buckling perpendicular to axis**	**Buckling curve (Fig. 3.5)**
Hollow sections 	hot formed	Y - Y Z - Z	a
	cold formed	Y - Y Z - Z	b
Welded box sections 	Generally	Y - Y Z - Z	b
	Thick welds and $h_y/t_y < 30$ $h_z/t_z < 30$	Y - Y Z - Z	c
Rolled I sections 	h/b > 1,2; t ≤ 40 mm	Y - Y Z - Z	a b
	h/b > 1,2; 40 < t ≤ 80 mm h/b ≤ 1,2; t ≤ 80 mm	Y - Y Z - Z	b c
	t > 80 mm	Y - Y Z - Z	d
Welded I and ⊥ sections 	t ≤ 40 mm	Y - Y Z - Z	b c
	t > 40 mm	Y - Y Z - Z	c d
U, L, T and solid sections 		Y - Y Z - Z	c
Sections not contained in this table shall be classified analogously			

Fig. 3.4 Selection of buckling curve for different cross sections (according to DIN 18800)

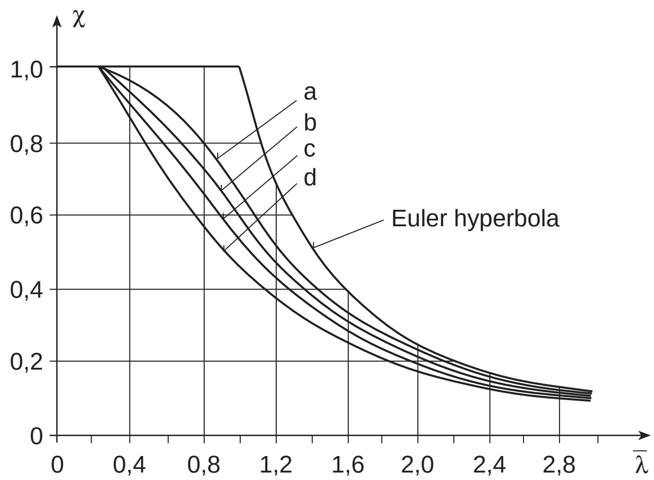


Fig. 3.5 Buckling curves a - d

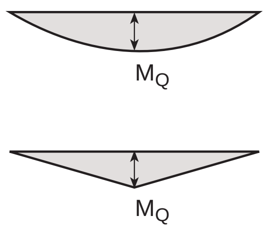
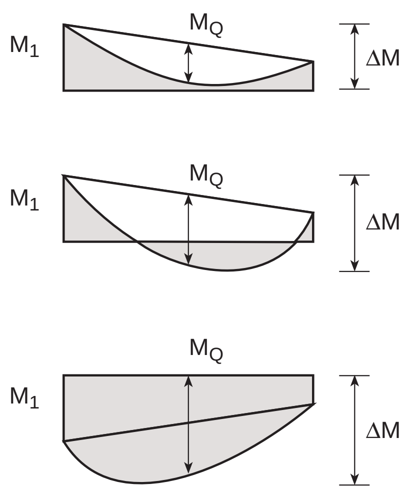
Moment distribution	**Bending moment coefficient β_m**
End moments  $-1 \leq \psi \leq 1$	$\beta_{m,\psi} = 0,66 + 0,44\,\psi$ but $\geq \beta_{m,\psi} \geq 1 - \dfrac{1}{\dfrac{N_{ki}}{\gamma_{gb} \cdot N}}$ and $\beta_{m,\psi} \geq 0,44$
Moments from lateral loads 	$\beta_{m,Q} = 1,0$
Moments from lateral loads with end moments 	$\psi \leq 0,77$: $\beta_m = 1,0$ $\psi > 0,77$: $\beta_m = \dfrac{M_Q + M_1 \cdot \beta_{m,\psi}}{M_Q + M_1}$

Fig. 3.6 Moment coefficient β_m

Maximum b/t values for built-up sections, for which buckling need not be considered, are given in the following:

Flatbar stiffeners and free flanges:

$$\frac{b}{t} = 0{,}4 \cdot \sqrt{\frac{E}{R_{eH}}}$$

Web plates:

$$\frac{b}{t} = 1{,}2 \cdot \sqrt{\frac{E}{R_{eH}}}$$

(b and t as defined in 2.2.2.2)

The b/t limit values are valid for shear stresses not exceeding $0{,}2 \cdot R_{eH}$.

2.3.2 Checks for local buckling can be carried through as indicated in 3. or 4. under consideration of the stress distribution over the cross section and boundary conditions.

2.3.3 In addition, web crippling has to be considered in way of supports and other points of concentrated load transfer.

3. Buckling of plane and curved plate panels [3]

3.1 Definitions

a = length of single or partial plate field [mm]

b = breadth of single plate field [mm]

α = aspect ratio of single plate field

= a / b

n = number of single plate field breadths within the partial or total plate field

t = nominal plate thickness [mm]

σ_x = membrane stress in x-direction [N/mm²]

σ_y = membrane stress in y-direction [N/mm²]

τ = shear stress in the x-y plane [N/mm²]

Compressive and shear stresses are to be taken as positive, tension stresses are to be taken as negative.

Note

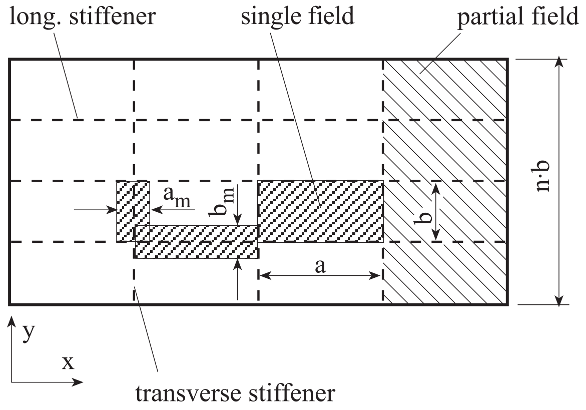
If the stresses in the x- and y-direction contain already the Poisson - effect, the following modified stress values may be used:

$$\sigma_x = (\sigma_x^* - 0{,}3 \cdot \sigma_y^*) / 0{,}91$$

$$\sigma_y = (\sigma_y^* - 0{,}3 \cdot \sigma_x^*) / 0{,}91$$

σ_x^*, σ_y^* = *stresses containing the Poisson effect.*

[3] The calculation method is based on DIN Standard 18800.



longitudinal: stiffener in the direction of the length a
transverse : stiffener in the direction of the breadth b

Fig. 3.7 Definitions

Table 3.6 Correction Factor F_1

1,0 for stiffeners sniped at both ends
Guidance values where both ends are effectively connected to adjacent structures*: 1,05 for flat bars 1,10 for bulb sections 1,20 for angle and tee-sections 1,30 for girders of high rigidity (e.g. bottom transverses)
* Exact values may be determined by direct calculations.

ψ = edge stress ratio according to Table 3.7

F_1 = correction factor for boundary condition at the longitudinal stiffeners according to Table 3.6

σ_e = reference stress

$= 0{,}9 \cdot E \cdot (t / b)^2$ [N/mm²]

E = Young's modulus

$= 2{,}06 \cdot 10^5$ N/mm² for steel

$= 0{,}69 \cdot 10^5$ N/mm² for aluminium alloys

R_{eH} = nominal yield point [N/mm²] for hull structural steel

= 0,2 % proof stress [N/mm²] for aluminium alloys

γ_{gb} = safety factor according to Table 3.5

For constructions of aluminium alloys the safety factors are to be increased in each case by 0,1.

λ = reference degree of slenderness

$= \sqrt{R_{eH} / (K \cdot \sigma_e)}$

K = buckling factor according to Tables 3.7 and 3.8.

In general, the ratio plate field breadth to plate thickness shall not exceed b/t = 100.

Table 3.7 Plane plate fields

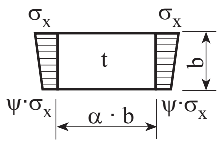
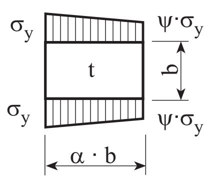
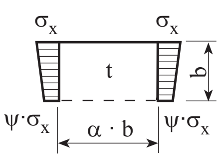
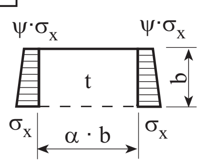
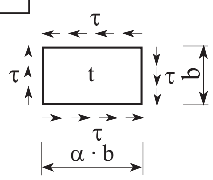
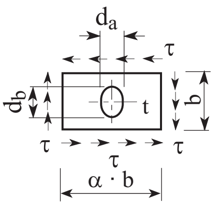
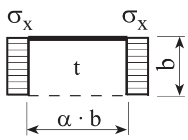
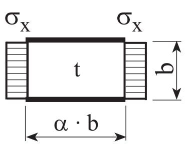
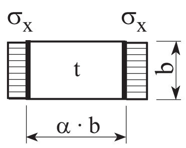
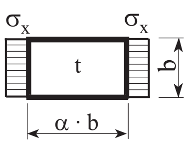
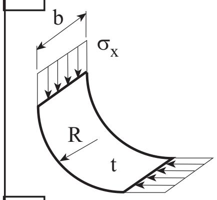
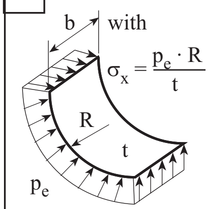
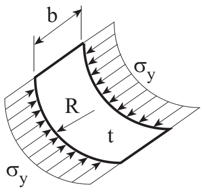
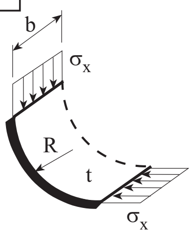
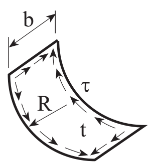
Load case	Edge stress ratio ψ	Aspect ratio α	Buckling factor K	Reduction factor κ
1 	$1 \ge \psi \ge 0$	$\alpha > 1$	$K = \frac{8,4}{\psi + 1,1}$	$\kappa_x = 1$ for $\lambda \le \lambda_c$ $\kappa_x = c\left(\frac{1}{\lambda} - \frac{0,22}{\lambda^2}\right)$ for $\lambda > \lambda_c$ $c = (1,25 - 0,12\psi) \le 1,25$ $\lambda_c = \frac{c}{2}\left(1 + \sqrt{1 - \frac{0,88}{c}}\right)$
	$0 > \psi > -1$		$K = 7,63 - \psi\,(6,26 - 10\,\psi)$	
	$\psi \le -1$		$K = (1 - \psi)^2 \cdot 5,975$	
2 	$1 \ge \psi \ge 0$	$\alpha \ge 1$	$K = F_1\left(1 + \frac{1}{\alpha^2}\right)^2 \frac{2,1}{(\psi+1,1)}$	$\kappa_y = c\left(\frac{1}{\lambda} - \frac{R+F^2\,(H-R)}{\lambda^2}\right)$ $c = (1,25 - 0,12\psi) \le 1,25$ $R = \lambda\left(1 - \frac{\lambda}{c}\right)$ for $\lambda < \lambda_c$ $R = 0,22$ for $\lambda \ge \lambda_c$ $\lambda_c = \frac{c}{2}\left(1 + \sqrt{1 - \frac{0,88}{c}}\right)$ $F = \left(1 - \frac{\frac{K}{0,91} - 1}{\lambda_p^2}\right) c_1 \ge 0$ $\lambda_p^2 = \lambda^2 - 0,5$ $\quad 1 \le \lambda_p^2 \le 3$ $c_1 = 1$ for σ_y due to direct loads $c_1 = \left(1 - \frac{F_1}{\alpha}\right) \ge 0$ for σ_y due to bending (in general) $c_1 = 0$ for σ_y due to bending in extreme load cases (e. g. w. t. bulkheads) $H = \lambda - \frac{2\lambda}{c\,(T + \sqrt{T^2 - 4})} \ge R$ $T = \lambda + \frac{14}{15\lambda} + \frac{1}{3}$
	$0 > \psi > -1$	$1 \le \alpha \le 1,5$	$K = F_1\left[\left(1 + \frac{1}{\alpha^2}\right)^2 \frac{2,1\,(1+\psi)}{1,1} - \frac{\psi}{\alpha^2}\,(13,9 - 10\,\psi)\right]$	
		$\alpha > 1,5$	$K = F_1\left[\left(1 + \frac{1}{\alpha^2}\right)^2 \frac{2,1\,(1+\psi)}{1,1} - \frac{\psi}{\alpha^2}\left(5,87 + 1,87\,\alpha^2 + \frac{8,6}{\alpha^2} - 10\,\psi\right)\right]$	
	$\psi \le -1$	$1 \le \alpha \le \frac{3\,(1-\psi)}{4}$	$K = F_1\left(\frac{1-\psi}{\alpha}\right)^2 5,975$	
		$\alpha > \frac{3\,(1-\psi)}{4}$	$K = F_1\left[\left(\frac{1-\psi}{\alpha}\right)^2 3,9675 + 0,5375\left(\frac{1-\psi}{\alpha}\right)^4 + 1,87\right]$	
3 	$1 \ge \psi \ge 0$	$\alpha > 0$	$K = \frac{4\,(0,425 + 1/\alpha^2)}{3\,\psi + 1}$	$\kappa_x = 1$ for $\lambda \le 0,7$ $\kappa_x = \frac{1}{\lambda^2 + 0,51}$ for $\lambda > 0,7$
	$0 > \psi \ge -1$		$K = 4\left(0,425 + \frac{1}{\alpha^2}\right)(1 + \psi) - 5 \cdot \psi\,(1 - 3,42\,\psi)$	
4 	$1 \ge \psi \ge -1$	$\alpha > 0$	$K = \left(0,425 + \frac{1}{\alpha^2}\right) \frac{3 - \psi}{2}$	

Table 3.7 Plane plate fields *(continued)*

Load case	Edge stress ratio ψ	Aspect ratio α	Buckling factor K	Reduction factor κ
5 	—	$\alpha \geq 1$	$K = K_\tau \cdot \sqrt{3}$	
			$K_\tau = \left[5{,}34 + \frac{4}{\alpha^2}\right]$	
		$0 < \alpha < 1$	$K_\tau = \left[4 + \frac{5{,}34}{\alpha^2}\right]$	$\kappa_\tau = 1$ for $\lambda \leq 0{,}84$ $\kappa_\tau = \frac{0{,}84}{\lambda}$ for $\lambda > 0{,}84$
6 	—		$K = K' \cdot r$ K' = K according to load case 5 r = Reduction factor $r = (1 - \frac{d_a}{a})(1 - \frac{d_b}{b})$ with $\frac{d_a}{a} \leq 0{,}7$ and $\frac{d_b}{b} \leq 0{,}7$	
7 	—	$\alpha \geq 1{,}64$	$K = 1{,}28$	$\kappa_x = 1$ for $\lambda \leq 0{,}7$
		$\alpha < 1{,}64$	$K = \frac{1}{\alpha^2} + 0{,}56 + 0{,}13\,\alpha^2$	$\kappa_x = \frac{1}{\lambda^2 + 0{,}51}$ for $\lambda > 0{,}7$
8 	—	$\alpha \geq \frac{2}{3}$	$K = 6{,}97$	
		$\alpha < \frac{2}{3}$	$K = \frac{1}{\alpha^2} + 2{,}5 + 5\,\alpha^2$	
9 	—	$\alpha \geq 4$	$K = 4$	$\kappa_x = 1$ for $\lambda \leq 0{,}83$
		$4 > \alpha > 1$	$K = 4 + \left[\frac{4 - \alpha}{3}\right]^4 2{,}74$	$\kappa_x = 1{,}13\left[\frac{1}{\lambda} - \frac{0{,}22}{\lambda^2}\right]$ for $\lambda > 0{,}83$
		$\alpha \leq 1$	$K = \frac{4}{\alpha^2} + 2{,}07 + 0{,}67\,\alpha^2$	
10 	—	$\alpha \geq 4$	$K = 6{,}97$	
		$4 > \alpha > 1$	$K = 6{,}97 + \left[\frac{4 - \alpha}{3}\right]^4 3{,}1$	
		$\alpha \leq 1$	$K = \frac{4}{\alpha^2} + 2{,}07 + 4\,\alpha^2$	

Explanations for boundary conditions
- - - - - - plate edge free
——— plate edge simply supported
▬▬▬ plate edge clamped

Table 3.8 Curved plate field $R/t \le 2500$ [1]

Load case	Aspect ratio b/R	Buckling factor K	Reduction factor κ
1a 	$\frac{b}{R} \le 1{,}63\sqrt{\frac{R}{t}}$	$K = \frac{b}{\sqrt{R \cdot t}} + 3\,\frac{(R \cdot t)^{0{,}175}}{b^{0{,}35}}$	$\kappa_x = 1$ [2] for $\lambda \le 0{,}4$ $\kappa_x = 1{,}274 - 0{,}686\,\lambda$ for $0{,}4 < \lambda \le 1{,}2$ $\kappa_x = \frac{0{,}65}{\lambda^2}$ for $\lambda > 1{,}2$
1b  p_e = external pressure in [N/mm²]	$\frac{b}{R} > 1{,}63\sqrt{\frac{R}{t}}$	$K = 0{,}3\,\frac{b^2}{R^2} + 2{,}25\left(\frac{R^2}{b \cdot t}\right)^2$	
2 	$\frac{b}{R} \le 0{,}5\sqrt{\frac{R}{t}}$	$K = 1 + \frac{2}{3}\,\frac{b^2}{R \cdot t}$	$\kappa_y = 1$ [2] for $\lambda \le 0{,}25$ $\kappa_y = 1{,}233 - 0{,}933\,\lambda$ for $0{,}25 < \lambda \le 1$ $\kappa_y = 0{,}3/\lambda^3$ for $1 < \lambda \le 1{,}5$ $\kappa_y = 0{,}2/\lambda^2$ for $\lambda > 1{,}5$
	$\frac{b}{R} > 0{,}5\sqrt{\frac{R}{t}}$	$K = 0{,}267\,\frac{b^2}{R \cdot t}\left[3 - \frac{b}{R}\sqrt{\frac{t}{R}}\right] \ge 0{,}4\,\frac{b^2}{R \cdot t}$	
3 	$\frac{b}{R} \le \sqrt{\frac{R}{t}}$	$K = \frac{0{,}6 \cdot b}{\sqrt{R \cdot t}} + \frac{\sqrt{R \cdot t}}{b} - 0{,}3\,\frac{R \cdot t}{b^2}$	as in load case 1a
	$\frac{b}{R} > \sqrt{\frac{R}{t}}$	$K = 0{,}3\,\frac{b^2}{R^2} + 0{,}291\left(\frac{R^2}{b \cdot t}\right)^2$	
4 	$\frac{b}{R} \le 8{,}7\sqrt{\frac{R}{t}}$	$K = K_\tau \cdot \sqrt{3}$ $K_\tau = \left[28{,}3 + \frac{0{,}67 \cdot b^3}{R^{1{,}5} \cdot t^{1{,}5}}\right]^{0{,}5}$	$\kappa_\tau = 1$ for $\lambda \le 0{,}4$ $\kappa_\tau = 1{,}274 - 0{,}686\,\lambda$ for $0{,}4 < \lambda \le 1{,}2$ $\kappa_\tau = \frac{0{,}65}{\lambda^2}$ for $\lambda > 1{,}2$
	$\frac{b}{R} > 8{,}7\sqrt{\frac{R}{t}}$	$K_\tau = 0{,}28\,\frac{b^2}{R\sqrt{R \cdot t}}$	

Explanations for boundary conditions:
- - - - - - plate edge free
———— plate edge simply supported
▬▬▬ plate edge clamped

[1] For curved plate fields with a very large radius the κ-value need not to be taken less than one derived for the expanded plane field.

[2] For curved single fields. e.g. the bilge strake, which are located within plane partial or total fields, the reduction factor κ may taken as follow:

Load case 1b: $\kappa_x = 0{,}8/\lambda^2 \le 1{,}0$: load case 2: $\kappa_y = 0{,}65/\lambda^2 \le 1{,}0$

3.2 Proof of single plate fields

3.2.1 Proof is to be provided that the following condition is complied with for the single plate field $a \cdot b$:

$$\left(\frac{|\sigma_x| \cdot S}{\kappa_x \cdot R_{eH}}\right)^{e_1} + \left(\frac{|\sigma_y| \cdot S}{\kappa_y \cdot R_{eH}}\right)^{e_2} - B\left(\frac{\sigma_x \cdot \sigma_y \cdot S^2}{R_{eH}^2}\right)$$

$$+\left(\frac{|\tau| \cdot S \cdot \sqrt{3}}{\kappa_\tau \cdot R_{eH}}\right)^{e_3} \le 1{,}0$$

Each term of the above condition shall be less than 1,0.

The reduction factors κ_x, κ_y and κ_τ are given in Table 3.7 and/or 3.8.

Where $\sigma_x \le 0$ (tension stress), $\kappa_x = 1{,}0$

Where $\sigma_y \le 0$ (tension stress), $\kappa_y = 1{,}0$

The exponents e_1, e_2 and e_3 as well as the factor B are calculated or set respectively as defined in Table 3.9.

3.2.2 Effective width of plating

The effective width of plating may be determined by the following formulae:

$b_m = \kappa_x \cdot b$ for longitudinal stiffeners

$a_m = \kappa_y \cdot a$ for transverse stiffeners

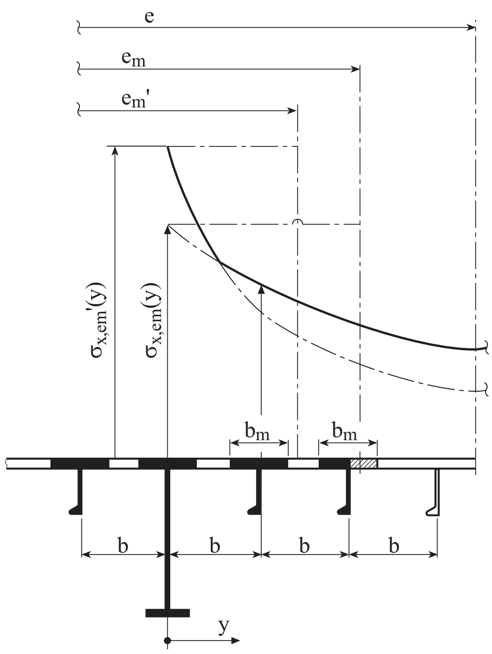
see also Fig. 3.7.

The effective width of plating is not to be taken greater than the value obtained from F.2.1.

Note

The effective width $e`_m$ of the stiffed flange plates of girders may be determined approximately as follows:

Stiffening parallel to web of girder:



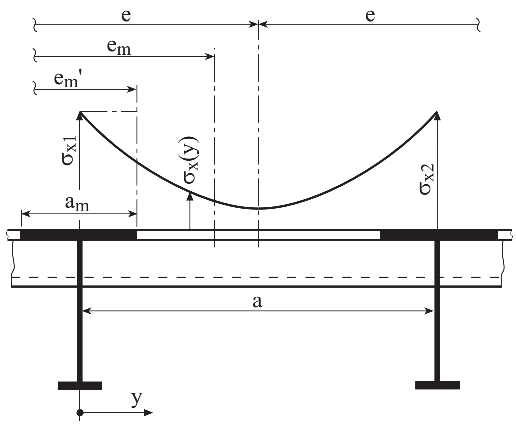
b $< e_m$

e'_m $= n \cdot b_m$

n = *integer number of the stiffener spacing b inside the effective width e_m according to Table 3.4 in F.2.1*

= $int\,(e_m / b)$

Stiffening vertical to web of girder:



a $\ge e_m$

e'_m $= a_m \,.\, e_m / e$

e = *width of plating supported according F.2.1.*

for $b \ge e_m$ or $a < e_m$ respectively, b and a shall be exchanged.

a_m and b_m for flange plates are in general to be determined for $\psi = 1$.

Stress distribution between two girders:

$$\sigma_x(y) = \sigma_{x1} \cdot \left\{1 - \frac{y}{e}\left[3 + c_1 - 4 \cdot c_2 - 2\frac{y}{e}\left(1 + c_1 - 2c_2\right)\right]\right\}$$

$$c_1 = \frac{\sigma_{x2}}{\sigma_{x1}} \qquad 0 \le c_1 \le 1$$

$$c_2 = \frac{1{,}5}{e} \cdot \left(e''_{m1} + e''_{m2}\right) - 0{,}5$$

σ_{x1}, σ_{x2} = *normal stresses in flange plates of adjacent girder 1 and 2 with spacing e*

e''_{m1} = *proportionate effective width of e_{m1}' and e_{m1} respectively of girder 1 within the distance e*

e''_{m2} = *proportionate effective width of e_{m2}' and e_{m2} respectively of girder 2 within the distance e*

y = *distance of considered location from girder 1*

Scantlings of plates and stiffeners are in general to be determined according to the maximum stresses σ_x(y) *at girder webs and stiffeners respectively. For stiffeners under compression arranged parallel to the girder web with spacing b no lesser value than 0,25 · R_{eH} shall be inserted for σ_x(y=b).*

Shear stress distribution in the flange plates may be assumed linearly.

Table 3.9 Exponents $e_1 - e_3$ and factor B

Exponents e_1 - e_3 and factor B	plate field	
	plane	curved
e_1	$1+\kappa_x^4$	1,25
e_2	$1+\kappa_y^4$	1,25
e_3	$1+\kappa_x \cdot \kappa_y \cdot \kappa_\tau^2$	2,0
B σ_x and σ_y positive (compression stress)	$(\kappa_x \cdot \kappa_y)^5$	0
B σ_x and σ_y negative (tension stress)	1	–

3.2.3 Webs and flanges

For non-stiffened webs and flanges of sections and girders proof of sufficient buckling strength as for single plate fields is to be provided according to 3.2.1 above.

Note

*Within 0,6 **L** amidships the following guidance values are recommended for the ratio web depth to web thickness and/or flange breadth to flange thickness:*

flat bars: $\frac{h_w}{t_w} \le 19{,}5\,\sqrt{k}$

angle, tee and bulb sections:

web: $\frac{h_w}{t_w} \le 60{,}0\,\sqrt{k}$

flange: $\frac{b_i}{t_f} \le 19{,}5\,\sqrt{k}$

b_i = b_1 or b_2 according to Fig 3.8, the larger value to be taken.

3.3 Proof of partial and total fields

3.3.1 Longitudinal and transverse stiffeners

Proof is to be provided that the continuous longitudinal and transverse stiffeners of partial and total plate fields comply with the conditions set out in 3.3.2 and 3.3.3.

3.3.2 Lateral buckling

$$\frac{\sigma_a + \sigma_b}{R_{eH}} S \le 1$$

σ_a = uniformly distributed compressive stress in the direction of the stiffener axis [N/mm²]

= σ_x for longitudinal stiffeners

= σ_y for transverse stiffeners

σ_b = bending stress in the stiffeners

$$= \frac{M_o + M_1}{W_{st} \cdot 10^3} \quad [\mathrm{N/mm^2}]$$

M_0 = bending moment due to deformation w of stiffener

$$= F_{Ki} \frac{p_z \cdot w}{c_f - p_z} \quad [\mathrm{N \cdot mm}]$$

$$(c_f - p_z) > 0$$

M_1 = bending moment due to the lateral load p for continuous longitudinal stiffeners:

$$= \frac{p \cdot b \cdot a^2}{24 \cdot 10^3} \quad [\mathrm{N \cdot mm}]$$

for transverse stiffeners:

$$= \frac{p \cdot a\,(n \cdot b)^2}{c_s \cdot 8 \cdot 10^3} \quad [\mathrm{N \cdot mm}]$$

p = lateral load [kN/m²] according to Section 2

F_{Ki} = ideal buckling force of the stiffener [N]

$$F_{Kix} = \frac{\pi^2}{a^2} E \cdot I_x \cdot 10^4 \quad \text{for long. stiffeners}$$

$$F_{Kiy} = \frac{\pi^2}{(n \cdot b)^2} \cdot E \cdot I_y \cdot 10^4 \quad \text{for transv. stiffeners}$$

I_x, I_y = moments of inertia of the longitudinal or transverse stiffener including effective width of plating according to 3.2.2. [cm⁴]

$$I_x \ge \frac{b \cdot t^3}{12 \cdot 10^4}$$

$$I_y \ge \frac{a \cdot t^3}{12 \cdot 10^4}$$

p_Z = nominal lateral load of the stiffener due to σ_x, σ_y and τ [N/mm²]

for longitudinal stiffeners:

$$p_{zx} = \frac{t_a}{b}\left(\sigma_{x1}\left(\frac{\pi \cdot b}{a}\right)^2 + 2 \cdot c_y \cdot \sigma_y + \sqrt{2}\,\tau_1\right)$$

for transverse stiffeners:

$$p_{zy} = \frac{t_a}{a}\left(2 \cdot c_x \cdot \sigma_{x1} + \sigma_y \left(\frac{\pi \cdot a}{n \cdot b}\right)^2 \left(1 + \frac{A_y}{a \cdot t_a}\right) + \sqrt{2}\,\tau_1\right)$$

σ_{x1} = $\sigma_x \cdot \left(1 + \dfrac{A_x}{b \cdot t_a}\right)$

c_x, c_y = factor taking into account the stresses vertical to the stiffener's axis and distributed variable along the stiffener's length

= $0{,}5\,(1 + \Psi)$ for $0 \le \Psi \le 1$

= $\dfrac{0{,}5}{1 - \Psi}$ for $\Psi < 0$

Ψ = edge stress ratio according to Table 3.7

A_x, A_y = sectional area of the longitudinal or transverse stiffener respectively [mm²]

τ_1 = $\left[\tau - t\sqrt{R_{eH} \cdot E\left(\dfrac{m_1}{a^2} + \dfrac{m_2}{b^2}\right)}\right] \ge 0$

for longitudinal stiffeners:

$\dfrac{a}{b} \ge 2{,}0 \;:\; m_1 = 1{,}47 \quad m_2 = 0{,}49$

$\dfrac{a}{b} < 2{,}0 \;:\; m_1 = 1{,}96 \quad m_2 = 0{,}37$

for transverse stiffeners:

$\dfrac{a}{n \cdot b} \ge 0{,}5 \;:\; m_1 = 0{,}37 \quad m_2 = \dfrac{1{,}96}{n^2}$

$\dfrac{a}{n \cdot b} < 0{,}5 \;:\; m_1 = 0{,}49 \quad m_2 = \dfrac{1{,}47}{n^2}$

w = $w_o + w_1$

w_o = assumed imperfection [mm]

$\dfrac{a}{250} \ge w_{ox} \le \dfrac{b}{250}$ for long. stiffeners

$\dfrac{n \cdot b}{250} \ge w_{oy} \le \dfrac{a}{250}$ for transv. stiffeners

however $w_o \le 10$ mm

Note

For stiffeners sniped at both ends w_o shall not be taken less than the distance from the midpoint of plating to the neutral axis of the profile including effective width of plating.

w_1 = deformation of stiffener due to lateral load p at midpoint of stiffener span [mm]

In case of uniformly distributed load the following values for w_1 may be used:

for longitudinal stiffeners:

$$w_1 = \frac{p \cdot b \cdot a^4}{384 \cdot 10^7 \cdot E \cdot I_x}$$

for transverse stiffeners:

$$w_1 = \frac{5 \cdot a \cdot p\,(n \cdot b)^4}{384 \cdot 10^7 \cdot E \cdot I_y \cdot c_s^2}$$

c_f = elastic support provided by the stiffener [N/mm²]

c_{fx} = $F_{Kix} \cdot \dfrac{\pi^2}{a^2} \cdot (1 + c_{px})$ for long. stiffeners

$$c_{px} = \frac{1}{1 + \dfrac{0{,}91 \cdot \left(\dfrac{12 \cdot 10^4 \cdot I_x}{t^3 \cdot b} - 1\right)}{c_{x\alpha}}}$$

$c_{x\alpha}$ = $\left[\dfrac{a}{2\,b} + \dfrac{2\,b}{a}\right]^2$ for $a \ge 2\,b$

= $\left[1 + \left(\dfrac{a}{2\,b}\right)^2\right]^2$ for $a < 2\,b$

c_{fy} = $c_s \cdot F_{Kiy} \cdot \dfrac{\pi^2}{(n \cdot b)^2} \cdot (1 + c_{py})$

for transv. stiffeners

c_s = factor accounting for the boundary conditions of the transverse stiffener

= 1,0 for simply supported stiffeners

= 2,0 for partially constraint stiffeners

$$c_{py} = \frac{1}{1 + \dfrac{0{,}91 \cdot \left(\dfrac{12 \cdot 10^4 \cdot I_y}{t^3 \cdot a} - 1\right)}{c_{y\alpha}}}$$

$c_{y\alpha}$ = $\left[\dfrac{n \cdot b}{2\,a} + \dfrac{2\,a}{n \cdot b}\right]^2$ for $n \cdot b \ge 2\,a$

= $\left[1 + \left(\dfrac{n \cdot b}{2\,a}\right)^2\right]^2$ for $n \cdot b < 2\,a$

W_s = section modulus of stiffener (longitudinal or transverse) [cm³] including effective width of plating according to 3.2.2

If no lateral load p is acting the bending stress σ_b is to be calculated at the midpoint of the stiffener span for that fibre which results in the largest stress value. If a lateral load p is acting, the stress calculation is to be carried out for both fibres of the stiffener's cross sectional area (if necessary for the biaxial stress field at the plating side).

Note

Longitudinal and transverse stiffeners not subjected to lateral load p have sufficient scantlings if their moments of inertia I_x and I_y are not less than obtained by the following formulae:

$$I_x = \frac{p_{zx} \cdot a^2}{\pi^2 \cdot 10^4} \left(\frac{w_{ox} \cdot h_w}{\frac{R_{eH}}{S} - \sigma_x} + \frac{a^2}{\pi^2 \cdot E} \right) \quad [cm^4]$$

$$I_y = \frac{p_{zy} \cdot (n \cdot b)^2}{\pi^2 \cdot 10^4} \left(\frac{w_{oy} \cdot h_w}{\frac{R_{eH}}{S} - \sigma_y} + \frac{(n \cdot b)^2}{\pi^2 \cdot E} \right) \quad [cm^4]$$

3.3.3 Torsional buckling

3.3.3.1 Longitudinal stiffeners

$$\frac{\sigma_x \cdot S}{\kappa_T \cdot R_{eH}} \le 1{,}0$$

$$\kappa_T = 1{,}0 \quad \text{for} \quad \lambda_T \le 0{,}2$$

$$= \frac{1}{\phi + \sqrt{\phi^2 - \lambda_T^2}} \quad \text{for} \quad \lambda_T > 0{,}2$$

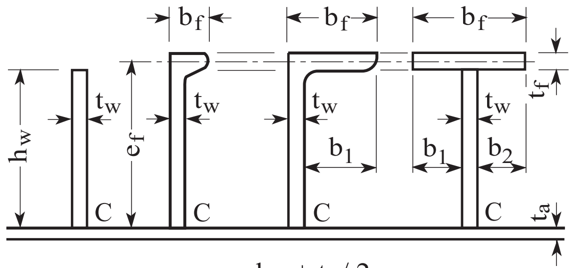
$$\phi = 0{,}5 \left(1 + 0{,}21 \left(\lambda_T - 0{,}2\right) + \lambda_T^2\right)$$

λ_T = reference degree of slenderness

$$= \sqrt{\frac{R_{eH}}{\sigma_{KiT}}}$$

$$\sigma_{KiT} = \frac{E}{I_P} \left(\frac{\pi^2 \cdot I_\omega \cdot 10^2}{a^2} \varepsilon + 0{,}385 \cdot I_T \right) \quad [N/mm^2]$$

For I_p, I_T, I_ω see Fig. 3.8 and Table 3.10.



$e_f = h_w + t_f / 2$

Fig. 3.8 Definitions of stiffeners

I_P = polar moment of inertia of the stiffener related to the point C [cm^4]

I_T = St. Venant's moment of inertia of the stiffener [cm^4]

I_ω = sectorial moment of inertia of the stiffener related to the point C [cm^6]

ε = degree of fixation

$$= 1 + 10^{-4} \sqrt{\frac{a^4}{I_\omega \left(\frac{b}{t^3} + \frac{4\, h_w}{3\, t_w^3} \right)}}$$

h_w = web height [mm]

t_w = web thickness [mm]

b_f = flange breadth [mm]

t_f = flange thickness [mm]

A_w = web area $h_w \cdot t_w$

A_f = flange area $b_f \cdot t_f$

Table 3.10 Definitions of moments of inertia for stiffeners

Section	I_P	I_T	I_ω
Flat bar	$\frac{h_w^3 \cdot t_w}{3 \cdot 10^4}$	$\frac{h_w \cdot t_w^3}{3 \cdot 10^4} \left(1 - 0{,}63 \frac{t_w}{h_w}\right)$	$\frac{h_w^3 \cdot t_w^3}{36 \cdot 10^6}$
Sections with bulb or flange	$\left(\frac{A_w \cdot h_w^2}{3} + A_f \cdot e_f^2\right) 10^{-4}$	$\frac{h_w \cdot t_w^3}{3 \cdot 10^4} \left(1 - 0{,}63 \frac{t_w}{h_w}\right)$ + $\frac{b_f \cdot t_f^3}{3 \cdot 10^4} \left(1 - 0{,}63 \frac{t_f}{b_f}\right)$	for bulb and angle sections: $\frac{A_f \cdot e_f^2 \cdot b_f^2}{12 \cdot 10^6} \left(\frac{A_f + 2{,}6\, A_w}{A_f + A_w}\right)$ for tee-sections: $\frac{b_f^3 \cdot t_f \cdot e_f^2}{12 \cdot 10^6}$

3.3.3.2 Transverse stiffeners

For transverse stiffeners loaded by compressive stresses and which are not supported by longitudinal stiffeners, proof is to be provided in accordance with 3.3.3.1 analogously.

4. Buckling of shell elements

4.1 General indications

4.1.1 Shell buckling investigations according to acknowledged standards are based on assumptions e.g. regarding geometrical imperfections and residual stresses, which have to be considered in the design and manufacturing process (see also 1.4).

4.1.2 Local discontinuities affecting stress distribution, such as weld seams, openings and welded-on fittings, shall be carefully investigated for their possible influence on buckling initiation. Local stiffening may be necessary.

4.1.3 For the buckling check of long unstiffened cylindrical shells (tubes) the procedure given in 4.2 may be adopted, in accordance with DIN 18800.

For short cylindrical shells or other types of loads not covered by the formulae given below the same standard may be referred to.

4.1.4 A buckling check with the help of other calculation methods, e.g. FEM, is generally acceptable provided the influence of imperfections and the nonlinear material behaviour are accounted for.

4.1.5 As an approximation the buckling behaviour of shells may be calculated using the formulae for plane plate panels, see 3.

In this case the curvature of the shell need not to be considered. The actual boundary conditions and shell membrane stresses are to be accounted for.

4.2 Buckling of long unstiffened cylindrical shells

4.2.1 The following basic requirements have to be fulfilled:

$$\sigma_d = \gamma_{bg} \cdot \sigma_{x,\varphi} \le \sigma_u$$

σ_d = design stress

σ_x, σ_φ = characteristic stress component due to axial loads (x) or external pressure (φ)

σ_u = ultimate buckling stresses, see 4.2.5

γ_{bg} = global safety factor given in Table 3.5

4.2.2 Limitation of imperfections for cylindrical shells

4.2.2.1 The reduction factors $\kappa_{1,2}$ given in 4.2.5 apply when the tolerances given in the following are not exceeded.

4.2.2.2 Local imperfections

The values "e" of the imperfections are measured from a straight rod, Fig. 3.9 a, c or a curved template, Fig. 3.9 b, d held anywhere over the weld, respectively against any meridian and against any parallel circle.

The length ℓ_m of the rod and the template shall be chosen according to Fig. 3.9.

The imperfection shall not exceed one percent of the measuring length:

$$e \le 0{,}01 \cdot \ell_m$$

ℓ_m = see Fig. 3.9

r = radius of shell curvature

ℓ = unsupported length (i.e. length between bulkheads or ring stiffeners).

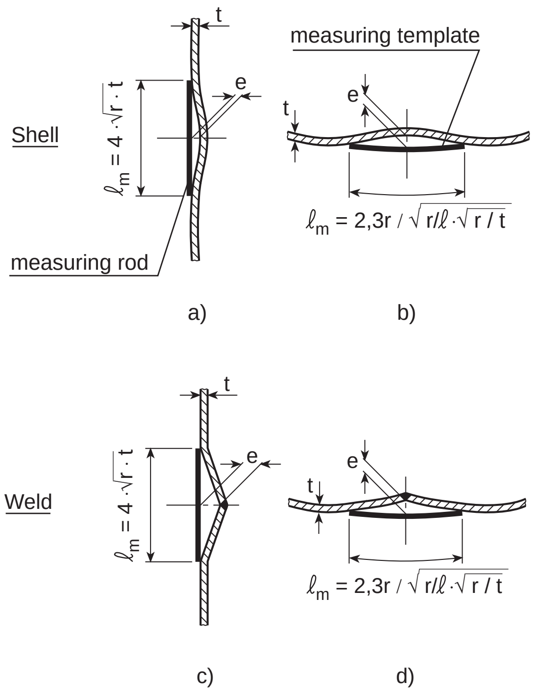


Fig. 3.9 Imperfections, measuring length ℓ_m

4.2.2.3 Out-of-roundness

The out-of-roundness shall fulfil the requirement:

$$e = 2 \cdot \frac{\max d - \min d}{\max d + \min d} \cdot 100 \le 0{,}5$$

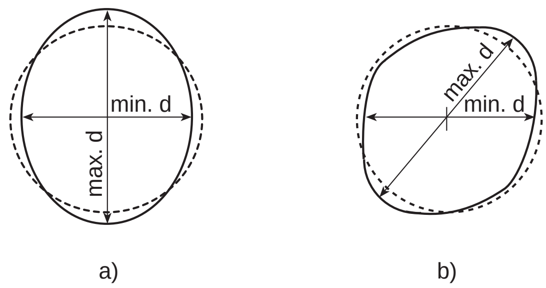
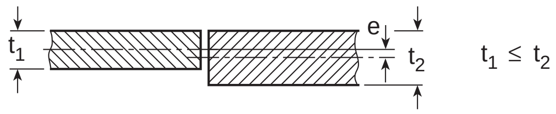


Fig. 3.10 Definition of out-of-roundness

4.2.2.4 Misalignment of butt joints

The misalignment shall be limited by the following condition:



$e \leq 0{,}2\ t_1$; max. 3 mm

$e \leq 1$ mm for $t_1 \leq 6{,}5$ mm

Fig. 3.11 Definition of misalignment

4.2.2.5 Corrective measures

If the imperfections specified above are exceeded, possible corrective measures have to be decided for each case.

In case the imperfections exceed the specified limiting imperfections by up to two times, the buckling calculation has to be repeated with reduced factors $\kappa_{1,2}$ as follows:

$$\kappa_{1,2_{red}} = \kappa_{1,2} \cdot \left[1 - \frac{\bar{\lambda}_{sx,\varphi}}{3}\left(\frac{\text{exist. e}}{\text{allow. e}} - 1\right)\right]$$

for $\bar{\lambda}_{sx,\varphi} < 1{,}5$

$$\kappa_{1,2_{red}} = \kappa_{1,2} \cdot \left[1{,}5 - 0{,}5\frac{\text{exist. e}}{\text{allow. e}}\right]$$

for $\bar{\lambda}_{sx,\varphi} \geq 1{,}5$

$\kappa_{1,2_{red}}$ = need not to be taken smaller than 0,3

$\kappa_{1,2}$ = reduction factors, see 4.2.5

$\underline{\lambda}_{sx,\varphi}$ = reduced slenderness for axial or circumferential loads, see 4.2.5

4.2.3 Ideal buckling stress for axial compressive loads

4.2.3.1 The following formulae apply to cylinders which are fixed in radial direction.

No buckling check is required if

$$\frac{r}{t} \leq \frac{E}{40\ R_{eH}} \cdot C_x$$

C_x = see 4.2.3.2

4.2.3.2 The ideal buckling stress is to be calculated according to the following formula:

$$\sigma_{xi} = 0{,}605 \cdot C_x \cdot E \cdot t / r$$

For cylinders with short and medium lengths, i.e.

$$\frac{\ell}{r} \leq 0{,}5\sqrt{\frac{r}{t}},$$

C_x = 1,0 is to be used

l = buckling length, see 2.2.2

For long cylinders, i.e.

$$\frac{\ell}{r} > 0{,}5\,\frac{r}{t},$$

the factor C_x is to be calculated as follows:

$$C_x = 1 - \frac{0{,}4\sqrt{\frac{\ell}{r}} - 0{,}2}{\eta} \geq 0{,}6$$

η reflects the end restraint condition and is to be chosen as follows:

both ends clamped: η = 6

one end simply supported - one end clamped: η = 3

both ends simply supported: η = 1

4.2.3.3 For long cylinders the bar buckling requirements of 2. are additionally to be observed.

For very slender cylinders (tubular bars), with

$$\frac{\ell}{r} > 6\,\frac{r}{t},$$

no check of local buckling is required.

4.2.4 Ideal buckling stress for circumferential compressive loads

4.2.4.1 No buckling check is required if

$$\frac{r}{t} \leq \sqrt{\frac{E}{23\ R_{eH}}}$$

4.2.4.2 For cylinders with short and medium lengths, i.e.

$$\frac{\ell}{r} \leq 1{,}63 \cdot C_\varphi \cdot \sqrt{\frac{r}{t}},$$

the ideal buckling stress is to be calculated as follows:

$$\sigma_\varphi = 0{,}92 \cdot C_\varphi \cdot E\,\frac{r}{\ell}\left(\frac{t}{r}\right)^{1{,}5}$$

C_φ = see Table 3.11

Table 3.11 Factor C_φ

Support Condition	C_φ
Both ends clamped	1,5
One end simply supported, one end clamped	1,25
Both ends simply supported	1,0
One end free, one end clamped	0,6

4.2.4.3 For long cylinders, i.e.

$$\frac{\ell}{r} > 1{,}63 \cdot C_\varphi \cdot \sqrt{\frac{r}{t}},$$

$$\sigma_\varphi = E\left(\frac{t}{r}\right)^2 \cdot \left[0{,}275 + 2{,}03 \cdot \left(\frac{C_\varphi}{\frac{\ell}{r}\sqrt{\frac{t}{r}}}\right)^4\right]$$

4.2.5 Ultimate buckling stress

The ultimate buckling stress is determined with regard to inelastic material behaviour and geometric as well as structural imperfections as follows.

a) Calculate the reduced slenderness of the shell,

$$\bar{\lambda}_{sx} = \sqrt{\frac{R_{eH}}{\sigma_{x_i}}}$$

$$\bar{\lambda}_{s\varphi} = \sqrt{\frac{R_{eH}}{\sigma_{\varphi_i}}}$$

b) The ultimate buckling stresses, σ_u, are calculated for the different directions as follows:

$$\sigma_{xu} = \kappa_1 \cdot R_{eH}$$

$$\sigma_{\varphi u} = \kappa_2 \cdot R_{eH}$$

c) The reduction factors $\kappa_{1,2}$ have to be established as a function of the reduced slenderness:

$$\kappa_1 = 1{,}0 \qquad \text{for } \bar{\lambda}_{sx} \le 0{,}2$$

$$\kappa_1 = 1{,}2 - \bar{\lambda}_{sx} \qquad \text{for } 0{,}2 < \bar{\lambda}_{sx} < 0{,}9$$

$$\kappa_1 = \frac{0{,}223}{\bar{\lambda}_{sx}^{2,8}} \qquad \text{for } \bar{\lambda}_{sx} \ge 0{,}9$$

$$\kappa_2 = 1{,}0 \qquad \text{for } \bar{\lambda}_{s\varphi} \le 0{,}4$$

$$\kappa_2 = 1{,}274 - 0{,}686 \cdot \bar{\lambda}_{s\varphi} \qquad \text{for } 0{,}4 < \bar{\lambda}_{s\varphi} < 1{,}2$$

$$\kappa_2 = \frac{0{,}65}{\bar{\lambda}_{s\varphi}^{2}} \qquad \text{for } \bar{\lambda}_{s\varphi} \ge 1{,}2$$

d) A decrease of these reduction factors may be required if the limiting imperfections are exceeded, see 4.2.2.5.

4.2.6 Combined axial compression and hydrostatic pressure

For combined loading, axial compressive stresses and circumferential stresses due to external pressure, the following interaction equation shall be satisfied:

$$\left(\frac{\gamma \cdot \sigma_x}{\sigma_{xu}}\right)^{1,25} + \left(\frac{\gamma \cdot \sigma_\varphi}{\sigma_{\varphi u}}\right)^{1,25} \le 1{,}0$$

4.3 Stiffened cylindrical shells

4.3.1 The buckling check of stiffened shells may be performed according to 3. above, or to acknowledged codes.

4.3.2 Stiffening of cylindrical elements shall be so designed that buckling of the shell panel between stiffeners would occur prior to stiffener failure.

4.3.3 Local buckling of a panel between stiffeners may be checked analogously to a plate panel, see 3., if:

$$\frac{b^2}{r \cdot t} \le 6 \quad (b:\ \text{stiffener spacing})$$

In other cases the indications in the codes mentioned may be followed.

H. Fatigue Strength

1. General

1.1 Alternative

Alternatively to the method defined in the following, the fatigue assessment according to the recommended practice of the American Petroleum Institute (API) RP 2A-WSD can be accepted by GL.

1.2 Definitions

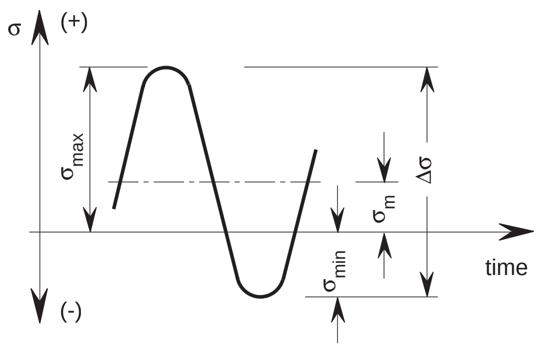


Fig. 3.12 Stress cycle

$\Delta\sigma$ = applied stress range (σ_{max} - σ_{min}) [N/mm^2]

σ_{max} = maximum upper stress of a stress cycle [N/mm^2]

σ_{min} = maximum lower stress of a stress cycle [N/mm^2]

$\Delta\sigma_{max}$ = applied peak stress range within a stress range spectrum [N/mm^2]

σ_m = mean stress (σ_{max} / 2 + σ_{min} / 2) [N/mm^2]

$\Delta\sigma_p$ = permissible stress range [N/mm^2]

$\Delta\tau$ = corresponding range for shear stress [N/mm^2]

n = number of applied stress cycles

N = number of endured stress cycles according to S-N curve (= endured stress cycles under constant amplitude loading)

$\Delta\sigma_R$ = fatigue strength reference value of S-N curve at 2 cycles of stress range [N/mm^2] = detail category number according to Table 3.14)

f_t = correction factor for plate thickness

f_c = correction factor for corrosion effects

f_m = correction factor for material effects

f_R = correction factor for mean stress effect

f_W = correction factor for weld shape effect

f_i = correction factor for importance of structural element

f_S = additional correction factor for hot spot stress analysis

f_n = factor considering stress spectrum and number of cycles for calculation of permissible stress range

$\Delta\sigma_R$ = corrected fatigue strength reference value of S-N curve at $2 \cdot 10^6$ stress cycles [N/mm^2]

γ_{fat} = safety factor

D = cumulative damage ratio

1.3 Scope

A fatigue strength analysis is to be performed for all structures which are predominantly subjected to cyclic loading, in order to ensure structural integrity and to assess the relative importance of possible fatigue damages for the establishment of an efficient inspection programme. The extent of the analysis is influenced by the local stress range and/or the number of cycles due to fluctuating loads on the structure. Wave loading is the main source of potential fatigue cracking. However, any other cyclic loading may contribute to fatigue failure and should therefore be considered.

The notched details, i.e. the welded connections at tubular joints, plates and beams as well as notches at free plate edges are to be considered individually. The fatigue strength assessment is to be carried out either on the basis of a permissible peak stress range for standard stress spectra, see 6. or on the basis of a cumulative damage ratio, see 5.

1.4 Application

The rules are applicable to structural steels up to a specified yield strength of 700 N/mm^2. Bolts are acceptable up to a tensile yield strength of 1000 N/mm^2. Other materials such as cast steel, and hybrid joints, can be treated in an analogous manner by adopting appropriate stress concentration factors and design S-N curves.

1.5 Low cycle fatigue

Low cycle fatigue problems in connection with extensive cyclic yielding have to be specially considered. When applying the following rules, the calculated nominal stress range should not exceed 1,5 times the minimum nominal upper yield point. In special cases the fatigue strength analysis may be performed by considering the local elasto-plastic stresses.

1.6 Limitation of fatigue assessment

No fatigue assessment is required if one of the following conditions is satisfied:

1.6.1 The maximum stress range $\Delta\sigma_{max}$ due to wave loads satisfies the following criterion and adequate protection exists in the case of corrosive environment:

$$\gamma_{fat} \cdot \Delta\sigma_{max} \leq \Delta\sigma_R \cdot f_t \cdot f_W \cdot f_m \cdot f_R \cdot f_i \cdot f_S$$

Additional stress ranges due to other load effects may be admitted if they remain below the endurable stress range at $N = 2 \cdot 10^8$ cycles, see 7.1.

The above criterion is based on the conservative assumption that the long term distribution of stress range resembles a Weibull distribution with a shape parameter $h \leq 2$ and that the total number of cycles is below 10^9, see 7.1.

1.6.2 The expected number of cycles during the life-time of the structure is less than

$$2 \cdot 10^6 \cdot [\Delta\sigma_R \cdot f_t \cdot f_C \cdot f_W \cdot f_m \cdot f_R \cdot f_i \cdot f_S / (\gamma_{fat} \cdot \Delta\sigma_{max})]^3$$

$\Delta\sigma_{max}$= the maximum nominal stress range or, for tubular joints, the maximum hot spot stress range, see 3. [N/mm^2]

γ_{fat} = safety factor according to Table 3.12

$\Delta\sigma_R$ = the fatigue strength reference value, i.e. the category number of the detail considered (minimum 36, see Table 3.14 or, for tubular joints, $\Delta\sigma_R$ = 100 N/mm^2

$f_t \cdot f_C \cdot f_W \cdot f_m \cdot f_R \cdot f_i \cdot f_S$ see 7.2

Table 3.12 Safety factors on stress range for the fatigue assessment

Structural detail is	part of a "fail-safe" structure	part of a non "fail-safe" structure
easily accessible	γ_{fat} = 1,00	γ_{fat} = 1,25
not easily accessible (e.g. under water)	γ_{fat} = 1,15	γ_{fat} = 1,35

1.7 Uncertainties

It should be noted that uncertainties may exist in the determination of the stresses at the considered location as well as in the fatigue resistance of the structural details. While performing a fatigue analysis, special care should be taken to ensure that the calculated stresses are not underestimated, and the fatigue strength not overestimated.

1.8 Design and fabrication

In order to achieve a good fatigue behaviour, the structure shall be designed and fabricated such that stress concentrations are reduced to a minimum. Plate thickness, structural stress concentration, weld shape and post-welding treatment as well as corrosion are some of the factors affecting the fatigue behaviour of structural details. Detailed information is given in the following.

2. Fatigue assessment procedure

2.1 General

The fatigue assessment shall verify that within an acceptable probability level the performance of the structure during its design life is satisfactory so that fatigue failure is unlikely to occur. An assessment should be made for every potential crack location. The fatigue assessment shall be carried out either in terms of calculated damage by comparing the applied damage ratio to the limit damage ratio, see 5., or in terms of maximum permissible stresses for standard stress distributions, see 6. In both cases the fatigue strength is based on design S-N curves as described in 7.

2.2 Influence of stress range

The number of stress cycles which may be endured by the structural element depends mainly on the magnitude of the stress ranges $\Delta\sigma$. The influence of the mean stress σ_m is generally small and may be taken care of by the correction factor f_R defined in 7.2.

2.3 Long term distribution

A long term distribution of stress range due to the load effects has to be established in terms of complete cycles using an appropriate cycle counting method. Range pair or "rainflow" counting is recommended.

The long term distribution of stress range shall take into account all stress fluctuations of relevance to the fatigue behaviour during the planned life of the structure. Some important sources of cyclic stresses are waves, wind, currents, varying hydrostatic pressure, crane loads, deck live loads and mechanical vibration. Construction, transport and installation loads may also be of relevance to the fatigue life.

2.4 Wave climate

Based on the wave climate expected over the long term either a spectral or a deterministic analysis of the structural response due to wave forces has to be performed. Within a spectral analysis the transfer functions have to be developed with a wave steepness appropriate for the wave climate to take the non linear effects properly into account. Within a deterministic analysis the representative sea states, wave heights and wave directions have to be chosen carefully with respect to their contribution to fatigue damage. Dynamic amplifications shall be considered for sea states having significant energy near the natural period of the platform or structure under consideration, see also I. Impact loads due to slamming have to be taken into account in the splash zone.

2.5 Types of stress

The fatigue strength analysis is, depending on the detail considered, based on one of the following types of stress:

– For tubular joints with full penetration welds the stress to be used in the fatigue analysis shall be the hot spot stress σ_S at the weld toe as defined in 3. The hot spot stress can be derived by calculating the nominal stress in the structural member (normally by a frame analysis of the structure) and applying appropriate stress concentration factors (SCF).

– For other welded joints the fatigue strength analysis is normally based on the nominal stress σ_n at the structural detail considered and on an appropriate detail classification as given in Table 3.14, which defines the detail category (or $\Delta\sigma_R$).

– In Table 3.15 $\Delta\sigma_R$ values for steel are given for some intersections of longitudinal frames of different shape and webs, which can be used for the assessment of the longitudinal stresses.

– For those welded joints, for which the detail classification is not possible or where additional stresses occur, which are not or not adequately considered by the detail classification, the fatigue strength analysis may alternatively be performed also on the basis of the hot spot stress σ_S as described in 3.4.

– For notches of free plate edges the notch stress σ_k, determined for linear-elastic material behaviour, is relevant, which can normally be calculated from a nominal stress σ_n and a theoretical stress concentration factor K_t. The fatigue strength is determined by the detail category (or $\Delta\sigma_R$) according to Table 3.14, type 29 and 30.

2.6 Safety factor

For the fatigue assessment procedure the stress ranges shall be multiplied by a safety factor γ which covers uncertainties of the procedure, the accessibility of the detail considered and the consequences of failure. Concerning the consequences of failure, the following two types of structures are to be considered:

– "fail-safe" structures, with reduced consequences of failure, i.e. local failure of a component does not result in a catastrophic failure of the structure

– "non fail-safe" structures, where local failure of a component leads rapidly to a catastrophic failure of the structure

The safety factors to be applied are given in Table 3.12.

3. Hot spot stress definition

3.1 General

As mentioned above, tubular joints shall be assessed on the basis of the hot spot stress range at the weld toe. This is the largest stress value at the intersection of the brace and chord and is defined as the extrapolation of the structural or geometric stress distribution to the weld toe, see Fig. 3.13. With this definition the hot spot stress incorporates the effects of the overall geometry (structural or geometric stress concentration) but omits the influence of the notch at the weld toe (local stress concentration). The hot spot stress has to be considered in connection with a particular design S-N curve as described in 7.1, which reflects the microscale effect occuring at the weld toe.

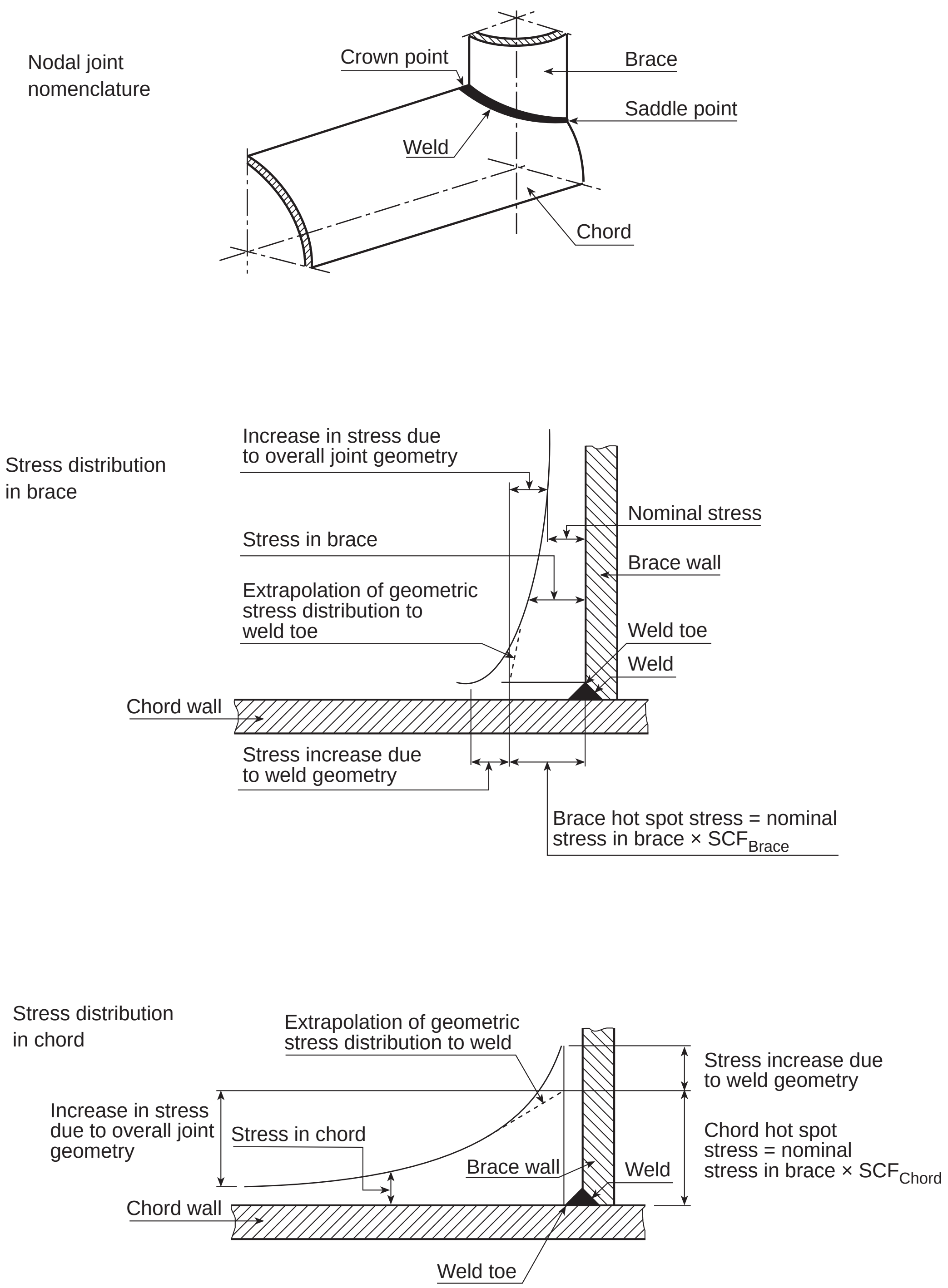


Fig. 3.13 Example of hot spot stresses in a tubular joint

3.2 Stress concentration factors

The hot spot stress can be calculated as the sum of nominal stresses due to the individual load components (i.e. axial load, in-plane and out-of-plane bending moment), each multiplied by a corresponding stress concentration factor (SCF). Stress concentration factors for the individual load components may be derived from finite element analyses, model tests or semi-empirical parametric equations based on such methods. Parametric equations should be used with caution in view of their accuracy and inherent limitations.

At least four different locations around the chord/ brace intersection shall be considered because the individual load components produce different stress peaks at the crown and saddle points, see Fig. 3.13. Both the chord and the brace side of the weld shall be checked in the fatigue analysis.

3.3 Alternative stress concentration factors

For tubular joints also stress concentration factors SCF according to M. EFTHYMIOU [4] or A. ALMAR-NAESS [5] can be accepted.

3.4 Complex geometry

Tubular joints with complex geometry (braces overlapping or stiffened by gusset plates or ring stiffeners) have to be investigated by a special analysis.

3.5 Plate structures

In plate structures, the hot spot stress at a weld toe is defined accordingly. It can be determined by measurements or numerically, e.g. by the finite element method using shell or volumetric models under the assumption of linear stress distribution over the plate thickness. This stress, containing membrane and bending components, may be linearly extrapolated to the weld toe over two points, 0,5 $\times$ t and 1,5 $\times$ t away from the weld toe (t = plate thickness).

3.6 Root failure

In addition to the assessment of the hot spot stress at the weld toe, the fatigue strength with regard to root failure has to be considered for partial penetration welds by application of the respective detail classification, see also 4.3 (and, e.g. type 23 in Table 3.14).

4. Nominal stress definition and classification of other details

4.1 Detail categories

Corresponding to their notch effect, welded joints are normally classified into detail categories considering particulars in geometry and fabrication, including subsequent quality control, and definition of nominal stress. Table 3.14 shows the detail classification based on recommendations of the International Institute of Welding (IIW) giving the detail category number (or $\Delta\sigma_R$), representing the fatigue strength reference value (at $2 \cdot 10^6$ cycles) for structures made of steel.

It has to be noted that some influence parameters cannot be considered by the detail classification and that a large scatter of fatigue strength has therefore to be reckoned with.

4.2 Classification of details

Details which are not contained in Table 3.14 may be classified either on the basis of hot spot stresses, see 3., or, else, by reference to published experimental work or by carrying out special fatigue tests, assuming a sufficiently high confidence level, see 7.1, and taking into account the correction factors as given in 7.2.8.

4.3 Nominal stress

Regarding the definition of nominal stress, the arrows in Table 3.14 indicate the location and direction of the stress for which the stress range is to be calculated. The potential crack location is also shown in Table 3.14. Depending on this crack location, the nominal stress range has to be determined by using either the cross sectional area of the parent metal or the weld throat thickness, respectively.

Bending stresses in plate and shell structures have to be incorporated into the nominal stress, taking the nominal bending stress acting at the location of crack initiation.

Note:

The factor K_S for the stress increase at transverse butt welds between plates of different thickness (see type 5 in Table 3.14) can be estimated in a first approximation as follows:

$$K_S = t_2 / t_1$$

t_1 = *smaller plate thickness*

t_2 = *larger plate thickness*

Additional stress concentrations which are not characteristic of the detail category itself, e.g. due to cut-outs in the neighbourhood of the detail, have also to be incorporated into the nominal stress.

4.4 Combination of normal and shear stress

In the case of combined normal and shear stress the relevant stress range may be taken as the range of the principal stress at the potential crack location which acts approximately perpendicular to the crack front as shown in Table 3.14.

4.5 Shear stresses

Where solely shear stresses are acting, the largest principal stress $\sigma_1 = \tau$ may be used in combination with the relevant detail category.

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4 EFTHYMIOU, M.: “Development of SCF formulae and generalised influence functions for use in fatigue analysis” Recent Developments in Tubular Joint Technology, OTJ ’88, London, plus updates

5 A. ALMAR-NAESS: “Fatigue Handbook – Offshore Steel Structures” TAPIR forlag, N-7005 Trondheim

5. Calculation of cumulative damage ratio

5.1 If the fatigue strength analysis is based on the calculation of the cumulative damage ratio, the stress range spectrum expected during the envisaged service life is to be established, see 2., and the cumulative damage rate D is to be calculated as follows:

$$D = \sum_{i=1}^{I} (n_i / N_i) \quad I = 1, ...I$$

I = total number of blocks of the stress range spectrum for summation (normally $I \geq 20$)

n_i = number of stress cycles in block i

N_i = number of endured stress cycles determined from the corrected design S-N curve, see 7. taking $\Delta\sigma = \gamma_{fat} \cdot \Delta\sigma_i$

$\Delta\sigma_I$ = stress range of block i

γ_{fa} = safety factor, see Table 3.12

5.2 Limit damage ratio

In general the design fatigue life of each joint and member shall be at least the planned service life of the structure. This means that the cumulative damage ratio D should not exceed the limit damage ratio of 1.

6. Permissible stresses for standard distributions of long term stress range

6.1 Standard distribution

For standard distributions of long term stress range the calculation can be simplified by using tabulated values of permissible peak stress ranges meeting the requirements stated above.

In many cases the two-parameter Weibull distribution applies having a cumulative stress distribution in the following form (see also Fig. 3.14):

$$\Delta\sigma = \Delta\sigma_{max} \ (1 - \log n / \log n_{max})^{1/h}$$

n = number of stress cycles exceeding $\Delta\sigma$

$\Delta\sigma_{max}$ = maximum stress range which is exceeded once within n_{max} stress cycles

n_{max} = total number of stress cycles

h = shape parameter (h = 1 corresponds to a linear distribution in a $\Delta\sigma$ - log n diagram)

6.2 Two-parameter distributions

For the two-parameter Weibull distributions, the permissible peak stress range can be calculated as follows:

$$\Delta\sigma_p = f_n \cdot \Delta\sigma_{Rc}$$

$\Delta\sigma_{Rc}$ = detail category or fatigue strength reference value, respectively, corrected according to 7.2

f_n = factor as given in Table 3.13

The peak stress range of the spectrum shall not exceed the permissible value, i.e.

$$\Delta\sigma_{max} \leq \Delta\sigma_p$$

The Table 3.13 is based on a cumulative damage ratio D = 1, see 5., and on the S-N curves as shown in Fig. 3.15 and 3.16, type "M". Therefore, the permissible stresses in Table 3.12 are not applicable to structural details without adequate corrosion protection.

7. Design S-N curves

7.1 Description of the design S-N curves

7.1.1 The design S-N curves for the calculation of the cumulative damage ratio according to 5. are shown in Fig. 3.15 for welded joints and in Fig. 3.16 for notches at free plate edges. The S-N curves represent the lower limit of the scatter band of 95 % of all test results available (corresponding to 97,5 % survival probability) considering further detrimental effects in large structures.

To account for different influence factors, the design S-N curves have to be corrected according to 7.2.

7.1.2 The S-N curves represent sectionwise linear relationships between log Δs and log N:

$$\log N = 6{,}69897 + m \cdot Q$$

Q = $\log (\Delta\sigma_R / \Delta\sigma) - 0{,}39794 / m_0$

m = slope exponent of S-N curve

m_0 = inverse slope in the range $N \leq 5 \cdot 10^6$

m_0 = 3 for welded joints

m_0 = for free plate edges, see Fig. 3.16

The S-N curves for detail category 160 forms the upper limit also for free plate edges with detail categories 100 – 140 in the range of low numbers of stress cycles, see Fig. 3.16.

7.1.3 For the fatigue strength analysis based on hot spot stress, the S-N curves shown in Fig. 3.15 apply with the following reference values:

$\Delta\sigma_R$ = 100 for K-butt welds with fillet welded ends, e.g. tubular joints or joints according to type 21 in Table 3.14, and for fillet welds which carry no load or only part of the load of the attached plate, e.g. type 17 in Table 3.14

$\Delta\sigma_R$ = 90 for fillet welds, which carry the total load of the attached plate, e.g. type 22 in Table 3.14

For butt welds, the values given for types 1 to 6 in Table 3.14 apply.

The correction of the design S-N curves according to 7.2 shall include the factor f_S.

Table 3.13 Factor f_n for the determination of the permissible stress range for Weilbull stress range spectra

Shape parameter h	Welded Joints ($m_o = 3$) n_{max} =			Plates Edges type 28 ($m_o = 5$) n_{max} =			type 29 ($m_o = 4$) n_{max} =			type 30 ($m_o = 3{,}5$) n_{max} =		
	10^7	10^8	10^9	10^7	10^8	10^9	10^7	10^8	10^9	10^7	10^8	10^9
0.5	(17,1)	(10,7)	6,86	9,27	7,62	6,16	(12,3)	9,14	6,73	(14,4)	9,94	6,88
0.6	(12,3)	7,50	4,79	7,78	6,17	4,85	9,69	6,94	5,01	(10,9)	7,27	4,98
0.7	9,45	5,65	3,62	6,63	5,12	3,97	7,88	5,51	3,96	8,63	5,64	3,85
0.8	7,56	4,49	2,90	5,74	4,36	3,36	6,59	4,55	3,26	7,07	4,57	3,13
0.9	6,26	3,71	2,41	5,06	3,79	2,91	5,65	3,86	2,78	5,96	3,83	2,64
1.0	5,33	3,16	2,06	4,52	3,36	2,58	4,94	3,36	2,43	5,15	3,30	2,28
1.2	4,11	2,44	1,61	3,74	2,76	2,13	3,96	2,69	1,96	4,05	2,60	1,82
1.4	3,36	2,01	1,34	3,21	2,36	1,83	3,33	2,26	1,66	3,36	2,22	1,53
1.6	2,86	1,72	1,15	2,84	2,08	1,63	2,89	1,97	1,46	2,89	1,87	1,33
1.8	2,51	1,52	1,02	2,56	1,88	1,48	2,57	1,76	1,31	2,56	1,66	1,18
2.0	2,25	1,37	0,91	2,34	1,72	1,36	2,33	1,60	1,20	2,30	1,51	1,07

The values given in parentheses may be applied for interpolation.

For interpolation between any pair of values (n_{max1}; f_{n1}) and (n_{max2}; f_{n2}), the following formula may be applied

$$\log f_n = \log f_{n1} + \log\left(n_{max}/n_{max1}\right) \frac{\log\left(f_{n2}/f_{n1}\right)}{\log\left(n_{max2}/n_{max1}\right)}$$

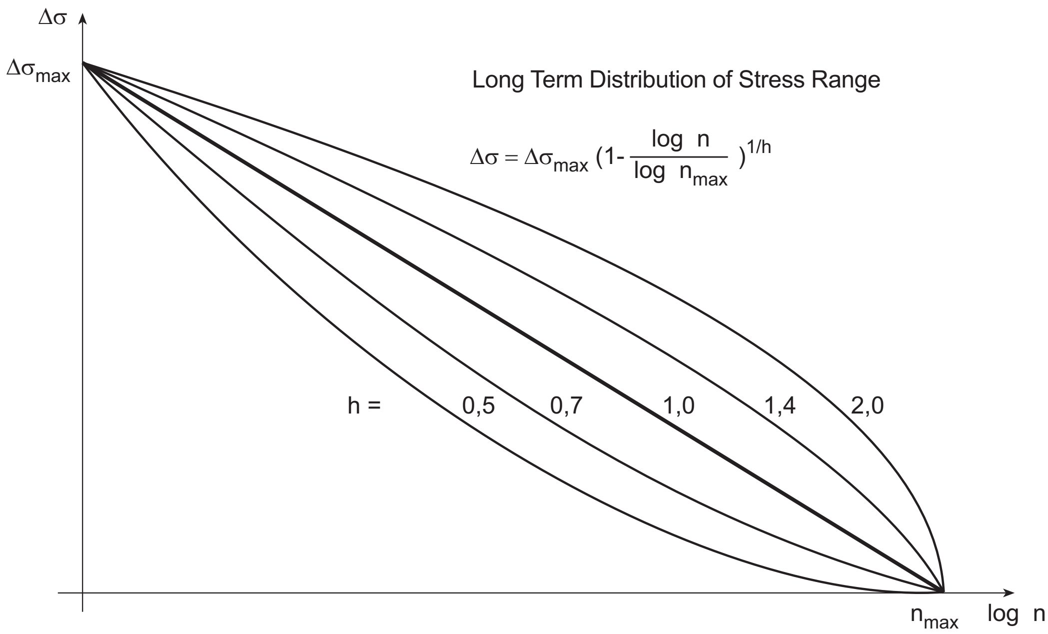


Fig. 3.14 Two-parameter Weibull distribution of stress

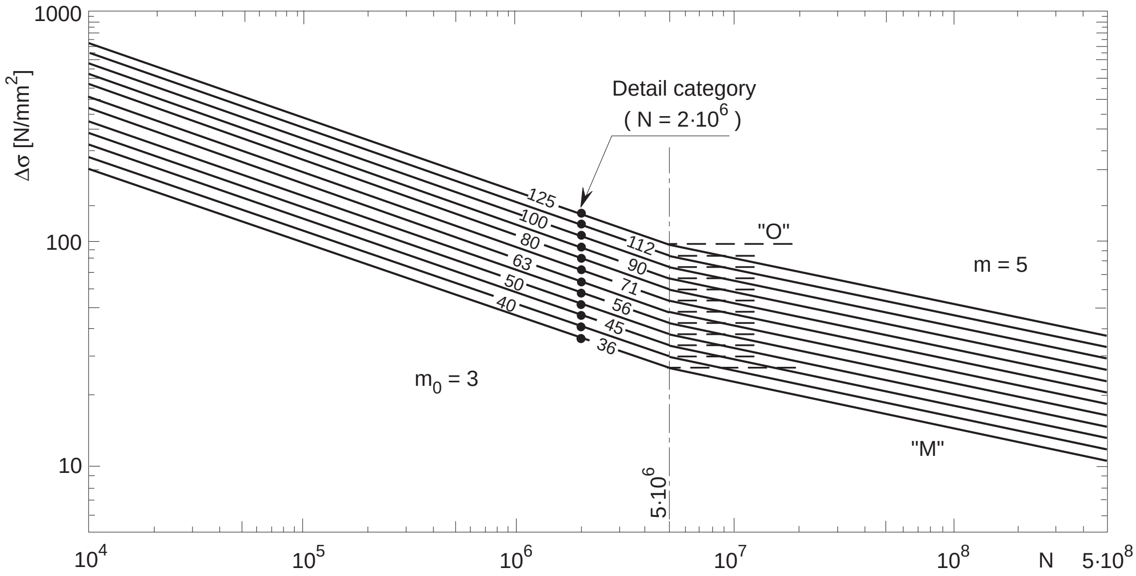


Fig. 3.15 S-N curves, welded joints

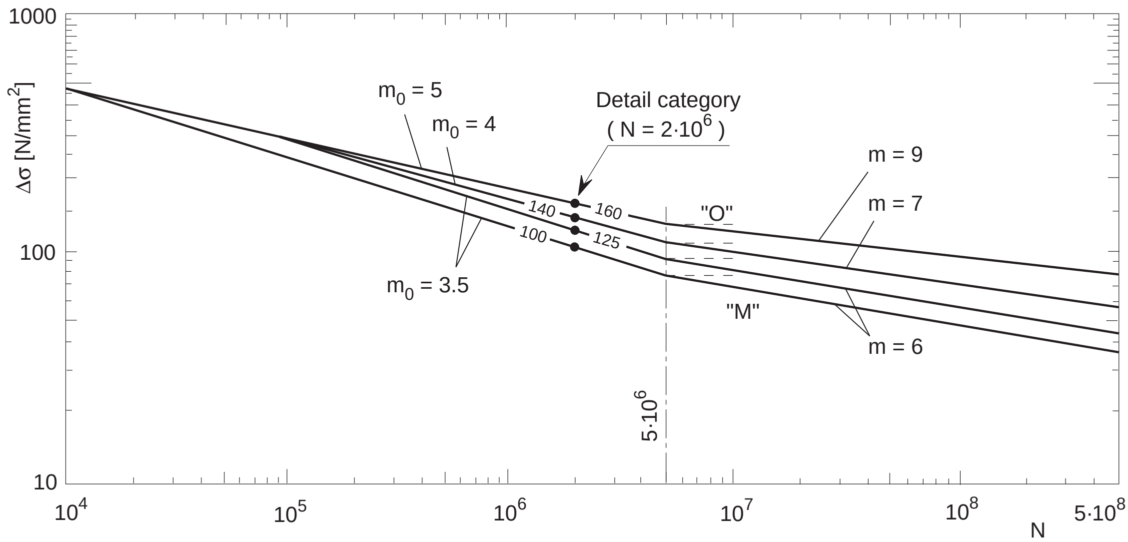


Fig. 3.16 S-N curves, plate edges

7.1.4 For structures in a mildly corrosive environment or in corrosive environment where an adequate protection is provided and which are subjected to variable stress ranges, the S-N curves shown by the solid lines in Fig. 3.15 and Fig. 3.16 have to be applied (S-N curves of type "M"), i.e.

$$m = m_0 \qquad \text{for } Q \le 0$$

$$m = 2 \cdot m_0 - 1 \qquad \text{for } Q > 0$$

7.1.5 For stress ranges of constant magnitude in non-corrosive environment the stress range given at N = 5 . 10^6 cycles may be taken as fatigue limit (S-N curves of type "O" in Fig. 3.15 and 3.16), thus:

$$m = m_0 \quad \text{for } Q \le 0$$

$$m = \infty \quad \text{for } Q > 0$$

7.1.6 For unprotected members exposed to corrosive environment the curves have to be extended without a change in slope and without fatigue limit, thus

$$m = m_0 \quad \text{for all values of Q}$$

7.1.7 The minimum weld quality levels required for the use of detail classification according to Table 3.14 are defined in Section 4. If any of these levels are not achieved, the use of the S-N curves is not appropriate. In such cases, the fatigue assessment shall be carried out by suitable adaption of these Rules.

7.2 Correction of the reference value of the design S-N curve

7.2.1 A correction of the reference value of the S-N curve (or detail category) is required to account for additional influence factors on fatigue strength as follows:

$$\Delta\sigma_{Rc} = f_t \cdot f_C \cdot f_W \cdot f_m \cdot f_R \cdot f_i \cdot f_S \cdot \Delta\sigma_R$$

f_t, f_C, f_W, f_m, f_R, f_i, f_S as defined in 7.2.2 – 7.2.8

For the description of the corrected design S-N curve, the formulae given in 7.1.3 may be used, replacing $\Delta\sigma_R$ by $\Delta\sigma_{Rc}$.

7.2.2 Thickness effect (f_t)

For welded connections oriented transversely to the direction of the applied stress with plate thickness t [mm], exceeding 25 mm at the potential crack locations:

$$f_t = (25 / t)^{0,25}$$

For all other cases:

$$f_t = 1{,}0$$

7.2.3 Corrosion effect (f_C)

For parts of a structure in corrosive environment without adequate corrosion protection:

$$f_C = 0{,}7$$

For all other cases:

$$f_C = 1{,}0$$

7.2.4 Material effect (f_m)

For welded joints it is generally assumed that the fatigue strength is independent of steel strength i.e.:

$$f_m = 1{,}0$$

For free plate edges the effect of the material's yield stress is accounted for as follows:

$$f_m = 1 + (R_{eH} - 235) / 1200$$

R_{eH} = minimum nominal upper yield stress of the steel [N/mm²]

7.2.5 Effect of mean stress (f_R)

The correction factor is calculated as follows:

– in the range of tensile pulsating stresses:

$$f_R = 1{,}0$$

$$\sigma_m \geq \Delta\sigma_{max} / 2$$

– in the range of alternating stresses:

$$f_R = 1 + c \cdot (1 - 2 \cdot \sigma_m / \Delta\sigma_{max})$$

$$-\Delta\sigma_{max} / 2 \leq \sigma_m \leq \Delta\sigma_{max} / 2$$

– in the range of compressive pulsating stresses:

$$f_R = 1 + 2 \cdot c$$

$$\sigma_m \leq -\Delta\sigma_{max} / 2$$

c = 0 for welded joints subjected to constant stress cycles

= 0,15 for welded joints subjected to variable stress cycles

= 0,3 for free plate edges

7.2.6 Effect of weld shape (f_W)

In normal cases:

$$f_W = 1{,}0$$

A factor $f_W > 1{,}0$ applies for welds treated e.g. by grinding. By this surface defects such as slag inclusions, porosity and crack-like undercuts shall be removed and a smooth transition from the weld to the base material shall be achieved. Final grinding shall be performed transversely to the weld direction. The depth should be approx. 0,5 mm larger than that of visible undercuts. For ground weld toes of fillet and K-butt welds:

$$f_W = 1{,}15$$

For butt welds ground flush either the corresponding detail category has to be chosen, e.g. type 1 in Table 3.14, or a weld shape factor

$$f_W = 1{,}25$$

may be applied in case of effective protection from sea water corrosion.

For endings of stiffeners or brackets, e.g. type 14 or 16 in Table 3.14, which have a full penetration weld and are completely ground flush to achieve a notch-free transition, the following factor applies:

$$f_W = 1{,}4$$

The assessment of a local post-weld treatment of the weld surface and the weld toe by other methods has to be agreed on in each case.

7.2.7 Influence of importance of structural element (f_i)

The influence of importance is usually covered by the safety factor γ_{fat}, see Table 3.12, i.e.:

$$f_i = 1{,}0$$

For notches at plate edges in general the following correction factor is to be used which takes into account the radius of rounding:

$$f_i = 0{,}9 + 5 / r \leq 1{,}0$$

r = notch radius [mm]; for elliptical roundings the mean value of the main half axes may be taken

7.2.8 Hot spot stress analysis (f_S)

If the hot spot stress is assessed, the following additional correction factor is to be taken into account which describes further influencing parameters such as e.g. pre-deformations:

$$f_S = k^*_m / [k_m - (k_m - 1) \cdot \Delta\sigma_{s,b} / \Delta\sigma_{s,max}]$$

$\Delta\sigma_{s,max}$ = applied peak stress range within a stress range spectrum

$\Delta\sigma_{s,b}$ = bending portion of $\Delta\sigma_{s,max}$

k_m = stress increase factor due to pre-deformations under axial loading, at least:

= 1,3 for butt welds, transverse stiffeners or tee-joints (corresponding to types 1 – 6, 17 and 21 – 22 in Table 3.14)

= 1,45 for cruciform joints (corresponding to types 21 and 22 in Table 3.14)

= 1,0 in all other cases (incl. tubular joints)

k^*_m = stress increase factor already contained in the fatigue strength reference value $\Delta\sigma_R$:

= 1,3 for butt welds (corresponding to types 1 – 6 in Table 3.14)

= 1,0 in all other cases (incl. tubular joints)

For simplification $f_S = k^*_m / k_m$ may be applied.

I. Dynamic Analysis

1. General

An investigation of the dynamic behaviour of the structure is required in case of risk of resonance of global or local structural vibration modes with energy-rich dynamic loads (periodical excitation forces).

2. Sources of excitation

Dynamic loads may be imposed by

– environmental loads such as waves, wind, currents, earthquake

– by variable functional loads from drilling or process equipment

– from the propulsion system in case of mobile units

3. Global design

3.1 Fixed structures

For fixed structures a dynamic analysis will be necessary in case of a platform natural frequency close to the frequency of energy-rich design sea states.

The global dynamic behaviour will generally only have to be accounted for in case of eigenperiods larger than 3 seconds.

3.2 Special structures

Dynamic analyses will generally be necessary for structures with large motions, e.g. articulated towers or guyed ("compliant") structures.

Also for structures installed in seismically active areas a dynamic investigation is required to know the inertia induced forces.

4. Local design

4.1 Larger structural components

The design of larger structural components, e.g. flare boom, helicopter deck, and of local structural members shall take account of possible dynamic loads, e.g. due to wind, wave, current, ice-flow or wave slamming.

4.2 Slender structures

Due regard shall be given to dynamic loads on slender structures such as radio towers and flare booms. Wind induced vortex shedding and also variable wind speeds (gusts) may cause dynamic stress amplifications and reduce the fatigue life.

4.3 Rotating equipment

For structural components supporting rotating equipment with free excitation forces and moments resonance shall be avoided. If this is not possible, the dynamic stress level shall be proven in a forced vibration investigation.

4.4 Earthquake loading

For structures which are subject to earthquake loading the eigen-frequencies of appurtenances or equipment supports should be substantially larger than the frequencies of the basic structure modes, see also 6.

5. Structural analysis

5.1 Structural modelling

The structural modelling shall take account of the requirements outlined in B.5.

5.2 Structure/foundation interaction

Special regard shall be given to the structure / foundation interaction. Compared to dynamic fatigue investigation for extreme environmental response a possible reduction of the foundation stiffness has to be considered.

5.3 Hydrodynamic masses

Besides all effective structural masses, the hydrodynamic masses, accounting for increased member thickness due to marine growth, and water enclosed in submerged members are to be considered.

5.4 Viscous damping

Equivalent viscous damping may be used for dynamic investigations of fixed steel structures. As an estimate two to three percent of critical damping may be applicable.

5.5 Method of analysis

5.5.1 The analysis method to be used will depend on the type of loading and the structural response.

5.5.2 Response spectral analysis will be used for structures with a linear elastic response to random loading, e.g. due to non-deterministic wave loads, provided a linearisation of the non-linear load effects is possible. The method may be appropriate to determine dynamic amplification effects due to extreme wave loads on fixed jacket structures and also to perform fatigue damage accumulation calculations.

5.5.3 Time history analysis, e.g. by direct integration, will have to be used for dynamic problems with non-linear nature.

6. Earthquake analysis

6.1 Strength and stiffness of structure

For seismically active areas the strength and stiffness of offshore structures shall be adequate to ensure that no significant damage will occur due to the expected design earthquake. [6]

[6] As a guideline for the design of a platform for earthquake ground motion the use of American Petroleum Institute API RP2A “Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms" is recommended.

6.2 Resistance against earthquake loading

An earthquake analysis is not required for areas where the design horizontal ground acceleration (strength level) is less than 0,05 g, provided other significant environmental loads are to be accounted for, so that sufficient resistance against earthquake loading is ensured.

6.3 Dynamic response

The analysis of the dynamic response should be performed using recognized procedures such as

– response spectrum analysis

– time history analysis, see 5.5

Generally a three-dimensional structure model shall be used for the analysis.

6.4 Modal maxima

When the response spectrum analysis is applied for the combination of the modal maxima, the use of the "Complete Quadratic Combination" (CQC) method is recommended.

6.5 Earthquake induced response

The earthquake induced structural response has to be combined with other load components acting permanently on the structure, see C. The member stresses shall not exceed the limit values given in D., loading condition 3.

Table 3.14 Catalogue of details

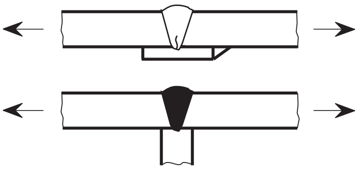
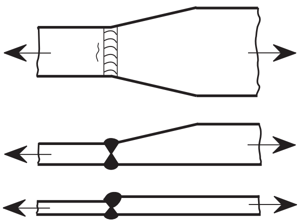
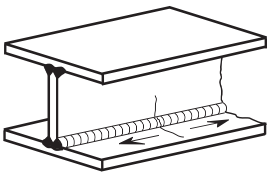
Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	Detail category $\Delta\sigma_R$	
			Steel	**Al**
1		Transverse butt weld ground flush to plate, 100 % NDT (Non-Destructive Testing)	112	45
2		Transverse butt weld made in shop in flat position, max. weld reinforcement 1 mm + 0,1 × weld width, smooth transitions, NDT	90	36
3		Transverse butt weld not satisfying conditions for joint type No. 2, NDT	80	32
4		Transverse butt weld on backing strip or three-plate connection with unloaded branch Butt weld, welded on ceramic backing, root crack	71 80	25 28
5		Transverse butt welds between plates of different widths or thickness, NDT as for joint type No. 2, slope 1 : 5 as for joint type No. 2, slope 1 : 3 as for joint type No. 2, slope 1 : 2 as for joint type No. 3, slope 1 : 5 as for joint type No. 3, slope 1 : 3 as for joint type No. 3, slope 1 : 2 For the third sketched case the slope results from the ratio of the difference in plate thicknesses to the breadth of the welded seam. Additional bending stress due to thickness change to be considered.	 90 80 71 80 71 63	 32 28 25 25 22 20
6		Transverse butt welds welded from one side without backing bar, full penetration root controlled by NDT not NDT For tubular profiles $\Delta\sigma_R$ may be lifted to the next higher detail category.	 71 45	 28 18
7		Partial penetration butt weld; the stress is to be related to the weld throat sectional area, weld overfill not to be taken into account	36	12
8		Continuous automatic longitudinal fully penetrated butt weld without stop/start positions (based on stress range in flange adjacent to weld)	125	50

Table 3.14 Catalogue of details ***(continued)***

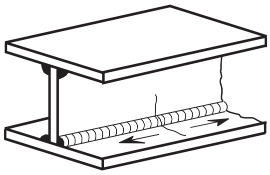
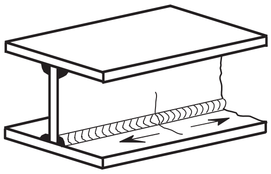
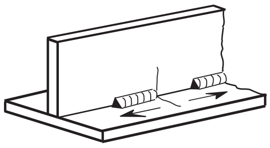
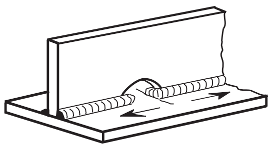
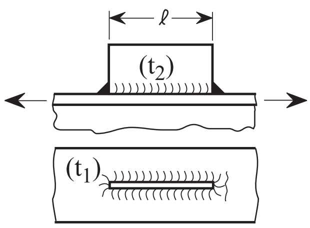
Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	Detail category $\Delta\sigma_R$	
			Steel	**Al**
9		Continuous automatic longitudinal fillet weld without stop/start positions (based on stress range in flange adjacent to weld)	100	40
10		Continuous manual longitudinal fillet or butt weld (based on stress range in flange adjacent to weld)	90	36
11		Intermittent longitudinal fillet weld (based on stress range in flange at weld ends) In presence of shear τ in the web, the detail category has to be reduced by the factor (1 – $\Delta\tau$ / $\Delta\sigma$), but not below 36 (steel) or 14 (Al).	80	32
12		Longitudinal butt weld, fillet weld or intermittent fillet weld with cut outs (based on stress range in flange at weld ends) If cut out is higher than 40 % of web height In presence of shear τ in the web, the detail category has to be reduced by the factor (1 – $\Delta\tau$ / $\Delta\sigma$), but not below 36 (steel) or 14 (Al). ***Note*** *For Ω-shaped scallops, an assessment based on local stresses in recommended.*	71 63	28 25
13		Longitudinal gusset welded on beam flange, bulb or plate: $\ell \leq 50$ mm $50 \text{ mm} < \ell \leq 150$ mm $150 \text{ mm} < \ell \leq 300$ mm $\ell > 300$ mm For $t_2 \leq 0{,}5\ t_1$, $\Delta\sigma_R$ may be increased by one category, but not over 80 (steel) or 28 (Al); not valid for bulb profiles. When welding close to edges of plates or profiles (distance less than 10 mm) and/or the structural element is subjected to bending, $\Delta\sigma_R$ is to be decreased by one category.	 80 71 63 56	 28 25 20 18

Table 3.14 Catalogue of details *(continued)*

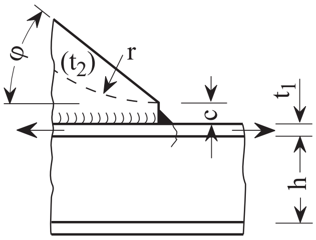
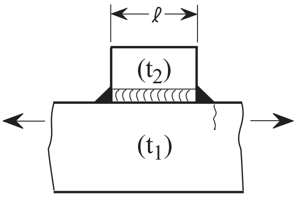
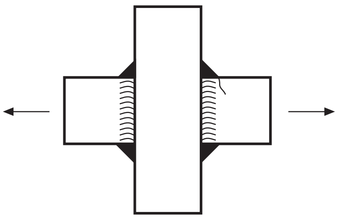
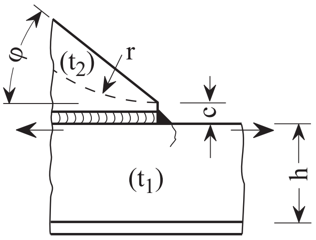
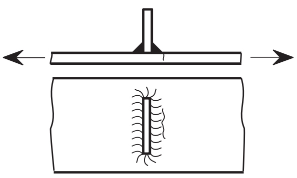
Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	Detail category $\Delta\sigma_R$ Steel	Detail category $\Delta\sigma_R$ Al
14		Gusset with smooth transition (sniped end or radius) welded on beam flange, bulb or plate; $c \le 2\,t_2$, max. 25 mm $r \ge 0{,}5\,h$ $r < 0{,}5\,h$ or $\varphi \le 20°$ $\varphi > 20°$ see joint type 13 For $t_2 \le 0{,}5\,t_1$, $\Delta\sigma_R$ may be increased by one category; not valid for bulb profiles. When welding close to the edges of plates or profiles (distance less than 10 mm), $\Delta\sigma_R$ is to be decreased by one category.	 71 63	 25 20
15a		Longitudinal flat side gusset welded on plate or beam flange edge $\ell \le 50$ mm 50 mm $< \ell \le$ 150 mm 150 mm $< \ell \le$ 300 mm $\ell >$ 300 mm For $t_2 \le 0{,}7\,t_1$, $\Delta\sigma_R$ may be increased by one category, but not over 56 (steel) or 20 (Al). If the plate or beam flange is subjected to in-plane bending, $\Delta\sigma_R$ has to be decreased by one category.	 56 50 45 40	 20 18 16 14
15b		Transverse butt weld at crossing flanges. Crack starting at butt weld. For crack of throughgoing flange see detail 15a.	50	20
16		Longitudinal flat side gusset welded on plate edge or beam flange edge, with smooth transition (sniped end or radius); $c \le 2\,t_2$, max. 25 mm $r \ge 0{,}5\,h$ $r < 0{,}5\,h$ or $\varphi \le 20°$ $\varphi > 20°$ see joint type 15 For $t_2 \le 0{,}7\,t_1$, $\Delta\sigma_R$ may be increased by one category.	 50 45	 18 16
17		Transverse stiffener with fillet welds (applicable for short and long stiffeners)	80	28

Table 3.14 Catalogue of details *(continued)*

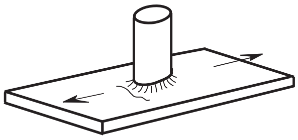
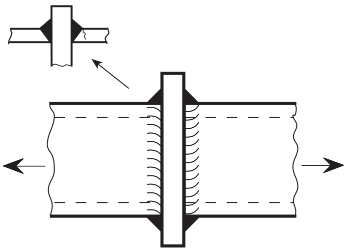
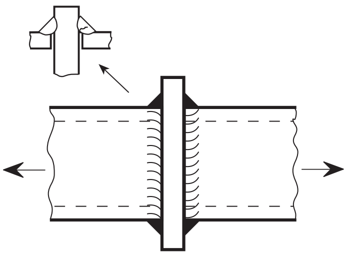
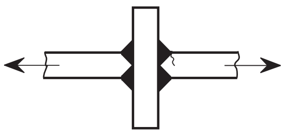
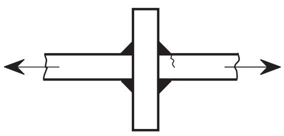
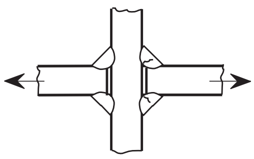
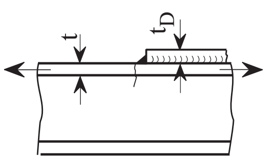
Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	Detail category $\Delta\sigma_R$	
			Steel	Al
18		Non-load-carrying shear connector	80	28
19		Full penetration weld at the connection between a hollow section (e.g. pillar) and a plate, for tubular section for rectangular hollow section	 56 50	 20 18
20		Fillet weld at the connection between a hollow section (e.g. pillar) and a plate, for tubular section for rectangular hollow section The stress is to be related to the weld sectional area.	 45 40	 16 14
21		Cruciform or tee-joint K-butt welds with full penetration or with defined incomplete root penetration cruciform joint tee-joint	 71 80	 25 28
22		Cruciform or tee-joint with transverse fillet welds, toe failure (root failure particularly for throat thickness a < 0,7 · t, see joint type 23) cruciform joint tee-joint	 63 71	 22 25
23		Welded metal in transverse load-carrying fillet welds at cruciform or tee-joint, root failure (based on stress range in weld throat), see also joint type No. 22	36	12
24		End of long doubling plate on beam, welded ends (based on stress range in flange at weld toe) $t_D \le 0{,}8\ t$ $0{,}8\ t < t_D \le 1{,}5\ t$ $t_D > 1{,}5\ t$ The following features increase $\Delta\sigma_R$ by one category accordingly: – reinforced ends according to Section 19, Fig. 19.4 – weld toe angle ≤ 30 ° – length of doubling ≤ 150 mm	 56 50 45	 20 18 16

Table 3.14 Catalogue of details ***(continued)***

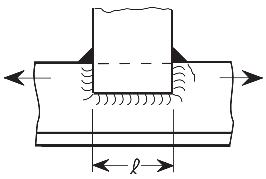
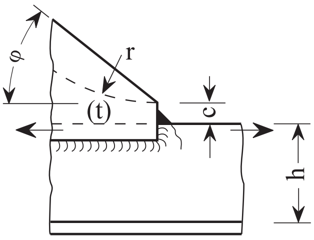
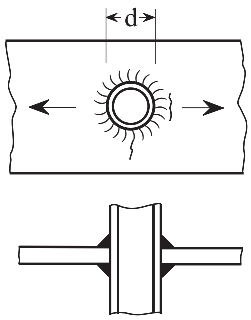
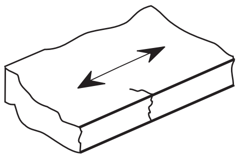
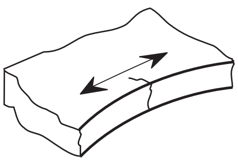
Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	Detail category $\Delta\sigma_R$	
			Steel	**Al**
25		Fillet welded non-load-carrying lap joint welded to longitudinally stressed component. – to bulb section or flat bar – to angle section For $\ell > 150$ mm, $\Delta\sigma_R$ has to be decreased by one category, while for $\ell \le 50$ mm, $\Delta\sigma_R$ may be increased by one category. If the component is subjected to bending, $\Delta\sigma_R$ has to be reduced by one category.	 56 50	 20 18
26		Fillet welded lap joint with smooth transition (sniped end with $\varphi \le 20°$ or radius) welded to longitudinally stressed component. – to bulb section or flat bar – to angle section $c \le 2$ t, max. 25 mm	 56 50	 20 18
27		Continuous butt or fillet weld connecting a pipe penetrating through a plate d ≤ 50 mm d > 50 mm ***Note*** *For large diameters an assessment based on local stress is recommended.*	 71 63	 25 22
28		Rolled or extruded plates and sections as well as seamless pipes, no surface or rolling defects	160 ($m_0 = 5$)	71 ($m_0 = 5$)
29		Plate edge sheared or machine-cut by any thermal process with surface free of cracks and notches, cutting edges broken or rounded. Stress increase due to geometry of cut-outs to be considered	140 ($m_0 = 4$)	40 ($m_0 = 4$)

Table 3.14 Catalogue of details *(continued)*

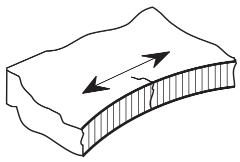
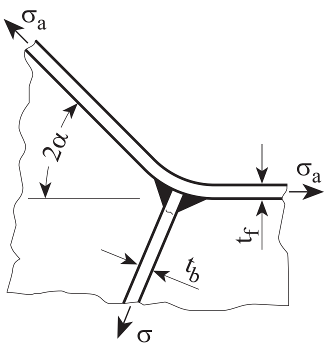
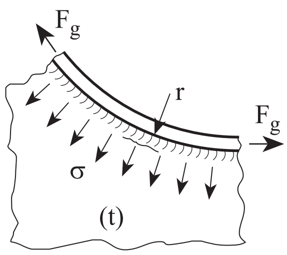
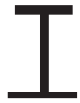
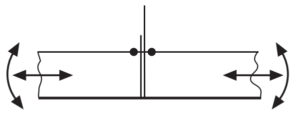
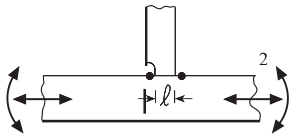
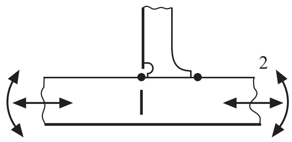
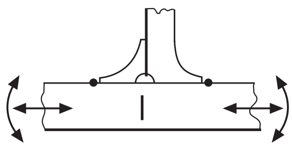
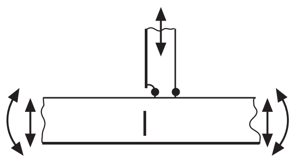
Type No.	Joint configuration showing mode of fatigue cracking and stress σ considered	Description of joint	Detail category $\Delta\sigma_R$	
			Steel	**Al**
30		Plate edge not meeting the requirements of type 29, but free from cracks and severe notches. Machine cut or sheared edge: Manually thermally cut: Stress increase due to geometry of cut-outs to be considered.	 125 (m_0 = 3,5) 100 (m_0 = 3,5)	 36 (m_0 = 3,5) 32 (m_0 = 3,0)
31		Joint at stiffened knuckle of a flange, to be assessed according to type 21, 22 or 23, depending on the type of joint. The stress in the stiffener at the knuckle can normally be calculated as follows: $$\sigma = \sigma_a \frac{t_f}{t_b} 2\sin\alpha$$	—	—
32		Unstiffened flange to web joint, to be assessed according to type 21, 22 or 23, depending on the type of joint. The stress in the web is calculated using the force F_g in the flange as follows: $$\sigma = \frac{F_g}{r \cdot t}$$ Furthermore, the stress in longitudinal weld direction has to be assessed according to type 8 – 10. In case of additional shear or bending, also the highest principal stress may become relevant in the web, see B.1.4.	—	—
Partly based on Recommendations on Fatigue of Welded Components, reproduced from IIW document XIII-1965-03 / XV-1127-03, by kind permission of the International Institute of Welding.				

Table 3.15 Various intersections

Joint configuration **Loads** **Locations being at risk for cracks**	**Description of joint**	**Detail category** $\Delta\sigma_R$ **steel**			
			 [1]	 [1]	
	Watertight intersection	71	71	71	71
	With heel stiffener	45 [3] (50) [3]	56 [3] (50) [3]	56 [3] (45) [3]	63 [3]
	With heel stiffener and integrated bracket	45	56	56	63
	With heel stiffener and integrated bracket and with backing bracket	50 (56)	63 (56)	63 (50)	71
	With heel stiffener but considering the load transferred to the stiffener	80 [4]	71 [4] 45 [5]	71 [4] 45 [5]	71 [4] 45 [5]

( ) Values for overlapping connection
[1] Additional stresses due to asymmetric sections have to be observed, see Section 3,L.
[2] To be increased by one category, when longitudinal loads only
[3] For ℓ > 150 mm to be decreased by one category
[4] Stress increase due to eccentricity and shape of cut out has to be observed
[5] Valid for stress in fillet weld connection

Section 4

Steel Structures

A. Materials

1. General

1.1 The materials for fixed offshore installations are to fulfil the minimum requirements detailed in this Section. Beyond this, the GL Rules II – Materials and Welding, Part 1 – Metallic Materials, are to be observed as far as applicable.

1.2 For the vessel structure of mobile offshore units directly the GL Rules II – Materials and Welding, Part 1 – Metallic Materials are to be observed. But for elements of special offshore equipment, e.g. jack-up legs, semi-submersibles, etc., the requirements of this Section have still to be applied. The design principles for mobile offshore units are defined in Chapter 2, Sections 2 to 6.

1.3 Materials for the steel structure shall exclusively be supplied by manufacturers approved by GL for this purpose. Such approval is to be applied for in writing with GL Head Office.

1.4 As far as possible, steels suitable for welding are to be employed in accordance with recognized standards. If other steels are intended to be used, the relevant specifications with all details required for appraisal are to be submitted to GL for approval.

2. Categories of structural members

2.1 In the choice of materials for the different members of the steel structure the criteria explained below are to be observed, see also Section 3, A.2.5:

– importance of the member within the structure (consequence of failure, redundancy)

– character of load and stress level (static or dynamic loads, residual stresses, stress concentrations, direction of stresses in relation to the rolling direction of the material, etc.)

– design temperature

– chemical composition (suitability for welding)

– yield and tensile strength of the material (dimensioning criteria)

– ductility of the material (resistance to brittle fracture at given design temperature)

– through-thickness properties (resistance to lamellar tearing)

Additional properties, such as corrosion resistance, may have to be considered.

2.2 Structural members

Depending on the importance of the structural member and on the type of load and the stress level, a structure can be subdivided into the following component categories:

2.2.1 Special structural members

These are members essential to the overall integrity of the structure and which, apart from a high calculated stress level, are exposed to particularly arduous conditions (e.g. stress concentrations or multi-axial stresses due to the geometric shape of the structural member and/or weld connections, or stresses acting in the through-thickness direction due to large-volume weld connections on the plate surface).

Note:

This applies e.g. to the cans of tubular nodes, thick-walled deck-to-leg and column connections, thick-walled points of introduction or reversal of forces.

2.2.2 Primary structural members

These are members participating in the overall integrity of the structure or which are important for operational safety and exposed to calculated load stresses comparable to the special structural members, but not to additional straining as mentioned above.

Note:

These are, for example, all other tubes of tubular truss work structures (jacket legs, bracing, piles), riser support structures, boat landing structures, girders and beams in the deck, self-supporting plating, helicopter decks, module support structures, crane columns and cranes, etc.

2.2.3 Secondary structural members

These are all structural members of minor significance, exposed to minor stresses only, and not coming under the above categories of "special" and „primary“.

Note:

These are, for example, non-structural walls, stairs, pedestals, mountings for piping and cables, etc.

2.3 The categories of structural members as per 2.2 are to be fixed in the design stage and indicated in the construction documentation.

3. Selection criteria of steels

3.1 Structural steels are defined depending on their minimum yield strength in the following strength classes:

– **normal strength** (mild) **steels** with minimum yield strength up to 275 N/mm^2

– **higher strength steels** with minimum yield strength over 275 N/mm^2, up to 360 N/mm^2

– **high strength steels** with minimum yield strength or 0,2 % proof stress over 360 N/mm^2

Note:

In particular for reasons of resistance to fatigue, for special and primary structures, fine grained structural steels, suitable for welding, with nominal yield strengths not exceeding 360 N/mm² are recommended.

High-strength steels having nominal yield strengths (or 0,2 % proof stresses) exceeding 460 N/mm² may be employed in technically motivated exceptional cases only, with GL's consent.

3.2 The strength class chosen for the respective structural member and/or the kind of material corresponding to it are to be indicated in the construction documentation. The same applies to the special requirements possibly having to be met by the material.

3.3 Steels with improved through-thickness properties are to be employed where structural members are rigid and/or particularly thick and where high residual welding stresses are to be expected, e.g. due to large-volume single-bevel butt joints or double-bevel butt joints with full root penetration, simultaneously implying high stresses acting in the through-thickness direction of the materials.

This will normally be the case where special structural members are concerned.

3.4 Depending on the structural member category, the design temperature and the material thickness, all steels to be employed for the structure have to meet the requirements listed in 4., in particular those for impact energy.

3.5 Steels for special and primary structural members have to undergo an approval test by GL at the manufacturers works. The scope of testing will be fixed from case to case. An approval test may be dispensed with, where standardized steels with nominal yield points of up to 360 N/mm² are concerned, which do not have to meet any special requirements.

4. Requirements for steels

4.1 Manufacturing procedures

All steels shall be made by the basic oxygen process or basic electric arc process, or by other methods approved by GL. The open hearth furnace process shall not be used.

Steel shall be as follows:

– steels with an impact temperature, see Table 4.7, of ≥ 0 °C shall be semi-killed as a minimum, i.e. equivalent to the deoxidation method FN of Standard EN 10025

– steels with an impact test temperature, see Table 4.7, of < 0 °C shall be fully killed, i.e. equivalent to the deoxidation method FF of Standard EN 10025

– all Z35 or Z30 steels according to Standard EN10164 or ASTM A370 shall be fully killed and fine grained

4.2 Supply condition and heat treatment

Steels intended for special and primary structural members shall be supplied as follows:

– steels with minimum yield strength up to 255 N/mm^2: normalized / normalized rolled (N)

– steels with minimum yield strength over 255 and up to 360 N/mm^2: normalized / normalized rolled (N), thermomechanically rolled (M) or quenched and tempered (Q)

– steels with minimum yield strength over 360 N/mm^2: thermo-mechanically rolled (M) or quenched and tempered (Q)

as defined in Standard EN 10225.

The following thickness limits shall apply:

– normalized / normalized rolled (N):
  up to 250 mm

– thermo-mechanically rolled (M):
  up to 150 mm for normal and higher strength steels
  up to 100 mm for high strength steels

– quenched and tempered (Q):
  up to 150 mm for higher strength steels
  up to 100 mm for high strength steels

The minimum rolling reduction ratio of concast material for plates shall be 4:1, except for piling, where it shall be 3:1.

4.3 Chemical composition and suitability for welding

The steels shall conform to the requirements for chemical composition, as determined by heat analysis, prescribed in the standards of Table 4.1 to Table 4.3 or equal.

The Carbon Equivalent CEV and Pcm of the heat analysis shall be calculated by both of the following equations:

$$CEV = C + Mn / 6 + (Cr + Mo + V) / 5 + (Ni + Cu) / 15$$

$$Pcm = C + Si / 30 + (Mn + Cu + Cr) / 20 + Ni / 60 + Mo / 15 + V / 10 + 5B$$

The maximum Carbon Equivalent CEV and Pcm shall not exceed the values prescribed in the relevant standards or the values of Table 4.1 (the smaller value shall be applied). CEV and Pcm computed on product shall not exceed the CEV and Pcm computed on heat by more than 0,02 % and 0,01 % respectively.

4.4 Mechanical properties

4.4.1 Tensile test properties

The requirements for tensile strength R_m, yield strength R_{eH} or 0,2 % proof stress $R_{p0,2}$ and for elongation at break, as stated in the GL Rules II – Materials and Welding, relevant standards and/or approved materials specifications, are to be observed; see also 3.1.

Beyond this, for steels with required notch bar impact tests at test temperature ≤ –20 °C, see Table 4.7, the yield strength ratios R_{eH} / R_m respectively $R_{p0,2} / R_m$ as defined in Table 4.2 shall not be exceeded.

4.4.2 Impact energy

The minimum values listed in Table 4.6 apply, proof of which is to be furnished at the test temperature T_T indicated in Table 4.7.

4.4.3 Through-thickness properties

Through thickness testing shall be carried out in the final heat treatment condition. Testing is not required- for a product thickness below 25 mm. The test shall be in accordance with EN 10164 or ASTM A370 and the test results as follows:

– the through-thickness tensile strength shall be not less than 80 % of the minimum specified tensile strength

– a minimum reduction of area Z = 30 %

4.5 Stress relieving treatment

4.5.1 General

Where the thickness, complexity or high level of stresses of the welded assemblies will require, or at least indicate, the probable requirement of stress relieving treatment after welding, the corresponding steels shall be ordered with tests on their physical properties (at least for tensile, impact properties) after such a treatment or after welding without such a treatment (see 4.5.5, CTOD).

Table 4.1 Maximum Carbon Equivalent

Steel strength class	CEV			Pcm		
	N	M	Q	N	M	Q
Normal strength steel	0,42	0,38	--	--	--	--
Higher strength steel	0,45	0,43	0,43	0,24	0,24	0,24
High strength steel	0,45	0,45	0,45	0,25	0,25	0,25

Table 4.2 Yield strength ratios

Steel strength class	Material quality N		Material quality M, Q	
	t ≤ 16 mm	t > 16 mm	t ≤ 16 mm	t > 16 mm
High strength	0,87	0,85	0,93	0,90
Higher strength	0,87	0,85	0,93	0,90
Normal strength	0,80	0,80	0,80	0,80

Table 4.3a Appropriate steels for plates (special structural members / high strength steel)

Structural member category	Steel strength class	Standard	Designation of material	Min. yield strength [MPa] / at thickness [mm]	Min. average Charpy value [J]	Min. single Charpy value [J]	Standard Charpy test temp. [°C]	Lowest Charpy test temp. [°C]	Through thickness value [%]	Max. S content [%]	Max. P content [%]	Max. thickness [mm]
Special members with Through Thickness Properties acc. to 2.2.1	High strength steel	EN 10225	S 460G2 + Q	460/16	60 (T) [1]	42 (T) [1]	–40 °C	–	Z 35 [1]	0,007	0,02	100
		EN 10225	S 460G2 + M	460/16	60 (T) [1]	42 (T) [1]	–40 °C	–	Z 35 [1]	0,007	0,02	100
		EN 10225	S 420G2 + Q	420/16	60 (T) [1]	42 (T) [1]	–40 °C	–	Z 35 [1]	0,007	0,02	100
		EN 10225	S 420G2 + M	420/16	60 (T) [1]	42 (T) [1]	–40 °C	–	Z 35 [1]	0,007	0,02	100
		API Spec 2Y	Grade 60(+Q)	414/all	48 (T)	41 (T)	–40 °C	–60 °C [1]	Z 30 [1]	0,006 [1]	0,03	100
		API Spec 2W	Grade 60(+M)	414/all	48 (T)	41 (T)	–40 °C	–60 °C [1]	Z 30 [1]	0,006 [1]	0,03	100
Special members without Through Thickness Properties acc. to 2.2.1		EN 10225	S 460G1 + Q	460/16	60 (T) [1]	42 (T) [1]	–40 °C	–	–	0,010	0,02	100
		EN 10225	S 460G1 + M	460/16	60 (T) [1]	42 (T) [1]	–40 °C	–	–	0,010	0,02	100
		EN 10225	S 420G1 + Q	420/16	60 (T) [1]	42 (T) [1]	–40 °C	–	–	0,010	0,02	100
		EN 10225	S 420G1 + M	420/16	60 (T) [1]	42 (T) [1]	–40 °C	–	–	0,010	0,02	100
		API Spec 2Y	Grade 60(+Q)	414/all	48 (T)	41 (T)	–40 °C	–60 °C [1]	–	0,010	0,03	100
		API Spec 2W	Grade 60(+M)	414/all	48 (T)	41 (T)	–40 °C	–60 °C [1]	–	0,010	0,03	100

Remarks: [1] options or supplementary requirements of the standard (T) transversal

Table 4.3b Appropriate steels for plates (special structural members / higher and normal strength steel)

Structural member category	Steel strength class	Standard	Designation of material	Min. yield strength [MPa] / at thickness [mm]	Min. average Charpy value [J]	Min. single Charpy value [J]	Standard Charpy test temp. [°C]	Lowest Charpy test temp. [°C]	Through thickness value [%]	Max. S content [%]	Max. P content [%]	Max. thickness [mm]
Special members with Through Thickness Properties acc. to 2.2.1	Higher strength steel	EN 10225	S 355 G8 + M	355/25	50 (T) [1]	35 (T) [1]	–40 °C	–	Z 35 [1]	0,007	0,02	100
		EN 10225	S 355 G8 + N	355/25	50 (T) [1]	35 (T) [1]	–40 °C	–	Z 35 [1]	0,007	0,02	150
		EN 10225	S 355G10 + N	355/25	50 (T) [1]	35 (T) [1]	–40 °C	–	Z 35 [1]	0,005	0,015	150
		EN 10225	S 355G10 + M	355/25	50 (T) [1]	35 (T) [1]	–40 °C	–	Z 35 [1]	0,005	0,015	100
		API Spec 2W	Grade 50&50T(+M)	345/all	41 (T)	34 (T)	–40 °C	–60 °C [1]	Z 30 [1]	0,006 [1]	0,03	150
		API Spec 2Y	Grade 50&50T(+Q)	345/all	41 (T)	34 (T)	–40 °C	–60 °C [1]	Z 30 [1]	0,006 [1]	0,03	150
		API Spec 2H	Grade 50 (+N)	345/all	41 (T)	34 (T)	–40 °C	–60 °C [1]	Z 30 [1]	0,006 [1]	0,03	100
		API Spec 2W	Grade 42 (+M)	290/all	34 (T)	27 (T)	–40 °C	–60 °C [1]	Z 30 [1]	0,006 [1]	0,03	150
		API Spec 2Y	Grade 42 (+Q)	290/all	34 (T)	27 (T)	–40 °C	–60 °C [1]	Z 30 [1]	0,006 [1]	0,03	150
		API Spec 2H	Grade 42 (+N)	289/all	34 (T)	27 (T)	–40 °C	–60 °C [1]	Z 30 [1]	0,006 [1]	0,03	100
Special members without Through Thickness Properties acc. to 2.2.1		EN 10225	S 355 G7 + M	355/25	50 (T) [1]	35 (T) [1]	–40 °C	–	–	0,01	0,02	100
		EN 10225	S 355 G7 + N	355/25	50 (T) [1]	35 (T) [1]	–40 °C	–	–	0,01	0,02	150
		EN 10225	S 355 G9 + N	355/25	50 (T) [1]	35 (T) [1]	–40 °C	–	–	0,01	0,02	150
		EN 10225	S 355 G9 + M	355/25	50 (T) [1]	35 (T) [1]	–40 °C	–	–	0,01	0,02	100
		EN 10028-3	P 355 NL 2	355/16	40 (T)	28 (T)	–20 °C	–	–	0,01	0,02	150
		EN 10028-6	P 355 QL 1	355/50	40 (T)	28 (T)	–20 °C	–	–	0,01	0,02	150
		EN 10028-6	P 355 QL 2	355/50	40 (T)	28 (T)	–40 °C	–	–	0,01	0,02	150
		API Spec 2W	Grade 50&50T(+M)	345/all	41 (T)	27 (T)	–40 °C	–60 °C [1]	–	0,01	0,03	150
		API Spec 2Y	Grade 50&50T(+Q)	345/all	41 (T)	27 (T)	–40 °C	–60 °C [1]	–	0,01	0,03	150
		API Spec 2H	Grade 50 (+N)	345/all	41 (T)	27 (T)	–40 °C	–60 °C [1]	–	0,01	0,03	100
		API Spec 2W	Grade 42 (+M)	290/all	34 (T)	27 (T)	–40 °C	–60 °C [1]	–	0,01	0,03	150
		API Spec 2Y	Grade 42 (+Q)	290/all	34 (T)	27 (T)	–40 °C	–60 °C [1]	–	0,01	0,03	150
		API Spec 2H	Grade 42 (+N)	289/all	34 (T)	27 (T)	–40 °C	–60 °C [1]	–	0,01	0,03	100
	Normal	EN 10028-3	P 275 NL 2	275/16	40 (T)	28 (T)	–20 °C	–	–	0,01	0,02	150

Remarks: [1] options or supplementary requirements of the standard (T) transversal

Table 4.3c Appropriate steels for plates (primary and secondary structural members)

Structural member category	Steel strength class	Standard	Designation of material	Min. yield strength [MPa] / at thickness [mm]	Min. average Charpy value [J]	Min. single Charpy value [J]	Standard Charpy test temp. [°C]	Lowest Charpy test temp. [°C]	Through thickness value [%]	Max. S content [%]	Max. P content [%]	Max. thickness [mm]
Primary members	Steels for "special" members may be used. The required options on supplementary requirements according to remark 1 (like Z 35, Z 30, (T)) are not necessary. The Charpy values-transversal of special members may be used for Charpy values-longitudinal of "primary" members at 20 °C deeper.											
	High strength steel	EN 10025-4	S 460 M	460/16	55 (L)	38 (L)	+20 °C	–	–	0,025	0,030	16
		EN 10025-4	S 460 ML	460/16	51 (L)	35 (L)	–10 °C	–	–	0,020	0,025	16
		EN 10025-4	S 420 M	420/16	47 (L)	33 (L)	0	–	–	0,025	0,030	40
		EN 10025-4	S 420 ML	420/16	47 (L)	33 (L)	–20 °C	–	–	0,020	0,025	40
	Higher strength steel	EN 10225	S 355 G2 + N	355/16	50 (L)	35 (L)	–20 °C	–	–	0,030	0,035	20
		EN 10225	S 355 G3 + N	355/16	50 (L)	35 (L)	–40 °C	–	–	0,025	0,030	40
		EN 10225	S 355 G5 + M	355/16	50 (L)	35 (L)	–20 °C	–	–	0,030	0,035	20
		EN 10225	S 355 G6 + M	355/16	50 (L)	35 (L)	–40 °C	–	–	0,025	0,030	40
		EN 10028-3	P 355 N	355/16	45 (L)	31 (L)	–20 °C	–	–	0,015	0,025	150
		EN 10028-3	P 355 NL1	355/16	40 (L)	28 (L)	–40 °C	–	–	0,015	0,025	150
		EN 10028-5	P 355 M	355/40	27 (T) [3]	19 (T) [3]	–20 °C [3]	–	–	0,020	0,025	63
		EN 10028-5	P 355 ML1	355/40	27 (T) [3]	19 (T) [3]	–40 °C [3]	–	–	0,015	0,020	63
		EN 10028-6	P 355 Q	355/40	27 (T) [3]	19 (T) [3]	–20 °C [3]	–	–	0,015	0,025	150
		EN 10025-3	S 355 N	355/16	40 (L)	28 (L)	–20 °C	–	–	0,025	0,030	250
		EN 10025-3	S 355 NL	355/16	40 (L)	28 (L)	–30 °C	–	–	0,020	0,025	250
		EN 10025-4	S 355 M	355/16	40 (L)	28 (L)	–20 °C	–	–	0,025	0,030	63
		EN 10025-4	S 355 ML	355/16	40 (L)	28 (L)	–30 °C	–	–	0,020	0,025	63
		EN 10025-2	S 355 K2	355/16	40 (L)	28 (L)	–20 °C	–	–	0,025	0,025	30
		EN 10025-2	S 355 JO	355/16	40/27 (L) [2]	28/19 (L) [2]	+20 °C [2]	–	–	0,030	0,030	30
		EN 10025-2	S 355 J2	355/16	40/27 (L) [2]	28/19 (L) [2]	0 °C [2]	–	–	0,025	0,025	30

Remarks:

1 options or supplementary requirements of the standard

2 steels with Charpy values of 27 J may be used as 40 J at 20 °C higher test temperature or 30 J at 10 °C higher test temperature

3 e.g. 27 J (T) ≈ 40 J (L) (see table 6 in EN 10028-3)

(T) transversal

(L) longitudinal

Table 4.3c Appropriate steels for plates (primary and secondary structural members) *continued*

Structural member category	Steel strength class	Standard	Designation of material	Min. yield strength [MPa] / at thickness [mm]	Min. average Charpy value [J]	Min. single Charpy value [J]	Standard Charpy test temp. [°C]	Lowest Charpy test temp. [°C]	Through thickness value [%]	Max. S content [%]	Max. P content [%]	Max. thickness [mm]
Primary members	Normal strength steel	EN 10028-3	P 275 NL1	275/16	30 (L)	21 (L)	–50 °C	–	–	0,015	0,025	150
		EN 10025-3	S 275 N	275/16	40 (L)	28 (L)	–20 °C	–	–	0,025	0,030	250
		EN 10025-3	S 275 NL	275/16	31 (L)	21 (L)	–40 °C	–	–	0,020	0,025	250
		EN 10025-4	S 275 M	275/16	40 (L)	28 (L)	–20 °C	–	–	0,025	0,030	150
		EN 10025-4	S 275 ML	275/16	31 (L)	21 (L)	–40 °C	–	–	0,020	0,025	150
		EN 10025-2	S 275 JO	275/16	30/27 (L) [1, 2]	21/19 (L) [1, 2]	+10 °C	–	–	0,030	0,030	150
		EN 10025-2	S 275 J2	275/16	30/27 (L) [1, 2]	21/19 (L) [1, 2]	–10 °C	–	–	0,025	0,025	150
		EN 10025-2	S 235 JR	235/16	27 (L)	19 (L)	+20 °C	–	–	0,035	0,035	250
		EN 10025-2	S 235 JO	235/16	27 (L)	19 (L)	0 °C	–	–	0,030	0,030	250
		EN 10025-2	S 235 J2	235/16	27 (L)	19 (L)	–20 °C	–	–	0,025	0,025	250
Secondary members		Steels for "special" and "primary" members may be used.										

Remarks:
1 options or supplementary requirements of the standard
2 steels with Charpy values of 27 J may be used as 40 J at 20 °C higher test temperature or 30 J at 10 °C higher test temperature
3 e.g. 27 J (T) ≈ 40 J (L) (see table 6 in EN 10028-3)

(T) transversal
(L) longitudinal

Table 4.4 Appropriate steels for tubes and hollow sections

Structural member category	Steel strength class	Standard	Designation of material	Min. yield strength [MPa] / at thickness [mm]	Min. average Charpy value [J]	Min. single Charpy value [J]	Standard Charpy test temp. [°C]	Lowest Charpy test temp. [°C]	Through thickness value [%]	Max. S content [%]	Max. P content [%]	Max. thickness [mm]
Special members with Through Thickness Properties acc. to 2.2.1	High strength steel	EN 10225	S 420 G6 + Q	420/20	60 (T) [1]	42 (T) [1]	–40	–	Z 35 [1]	0,007	0,025	40
Special members without Through Thickness Properties acc. to 2.2.1		EN 10225	S 460 G6 + Q	460/20	60 (T) [1]	42 (T) [1]	–40	–	–	0,010	0,025	40
Special members with Through Thickness Properties acc. to 2.2.1	Higher strength steel	EN 10225	S 355 G15 + N	355/20	50 (T) [1]	35 (T) [1]	–40	–	Z 35 [1]	0,007	0,025	40
		EN 10225	S 355 G15 + Q	355/20	50 (T) [1]	35 (T) [1]	–40	–	Z 35 [1]	0,007	0,025	40
Special members without Through Thickness Properties acc. to 2.2.1		EN 10225	S 355 G14 + N	355/20	50 (T) [1]	35 (T) [1]	–40	–	–	0,010	0,025	40
		EN 10225	S 355 G14 + Q	355/20	50 (T) [1]	35 (T) [1]	–40	–	–	0,010	0,025	40
Primary members	Steels for "special" members may be used. The required options on supplementary requirements according to remark 1 (like Z 35, Z 30, (T) are not necessary. The Charpy values-transversal of special members may be used for Charpy values-longitudinal of "primary" members at 20 °C deeper.											
	High strength steel	EN 10225	S 460 G 5 + Q	460/20	60 (L)	42 (L)	–40	–	–	0,015	0,025	20
		EN 10225	S 420 G 5 + Q	420/20	60 (L)	42 (L)	–40	–	–	0,015	0,025	20
	Higher strength steel	EN 10225	S 355 G13 + N	355/20	50 (L)	35 (L)	–40	–	–	0,015	0,025	20
		EN 10225	S 355 G13 + Q	355/20	50 (L)	35 (L)	–40	–	–	0,015	0,025	20
		EN 10225	S 355 G 1 + N	355/20	50 (L)	35 (L)	–20	–	–	0,030	0,035	20
	Normal strength steel											
Secondary members		Steels for "special" and "primary" members may be used.										

Remarks: [1] options or supplementary requirements of the standard (T) transversal
(L) longitudinal

Table 4.5 Appropriate steels for rolled sections

Structural member category	Steel strength class	Standard	Designation of material	Min. yield strength [MPa] / at thickness [mm]	Min. average Charpy value [J]	Min. single Charpy value [J]	Standard Charpy test temp. [°C]	Lowest Charpy test temp. [°C]	Through thickness value [%]	Max. S content [%]	Max. P content [%]	Max. thickness [mm]
Special members with Through Thickness Properties acc. to 2.2.1	High strength steel	EN 10225	S 460 G4 + M	460/16	60 (T) [1]	42 (T) [1]	–40	–	Z 35	0,007	0,020	63
		EN 10225	S 420 G4 + M	420/16	60 (T) [1]	42 (T) [1]	–40	–	Z 35	0,007	0,020	63
	Higher strength steel	EN 10225	S 355 G12 + N	355/16	50 (T)	35 (T)	–40	–	Z 35	0,007	0,020	63
		EN 10225	S 355 G12 + M	355/16	50 (T)	35 (T)	–40	–	Z 35	0,007	0,020	63
Primary members	Steels for "special" members may be used. The required options on supplementary requirements according to remark 1 (like Z 35, Z 30, (T) are not necessary. The Charpy values-transversal of special members may be used for Charpy values-longitudinal of "primary" members at 20 °C deeper.											
	High strength steel	EN 10225	S 460 G3 + M	460/16	60 (L)	42 (L)	–40	–	–	0,015	0,025	63
		EN 10225	S 420 G3 + M	420/20	60 (L)	42 (L)	–40	–	–	0,015	0,025	63
		EN 10025-4	S 460 M	460/16	55 (L)	38 (L)	+20	–	–	0,025	0,030	16
		EN 10025-4	S 460 ML	460/16	51 (L)	35 (L)	–10	–	–	0,020	0,025	16
		EN 10025-4	S 420 M	420/16	47 (L)	33 (L)	0	–	–	0,025	0,030	40
		EN 10025-4	S 420 ML	420/16	47 (L)	33 (L)	–20	–	–	0,020	0,025	40
	Higher strength steel	EN 10225	S 355 G11 + N	355/16	50 (L)	35 (L)	–40	–	–	0,015	0,025	63
		EN 10225	S 355 G11 + M	355/16	50 (L)	35 (L)	–40	–	–	0,015	0,025	63
		EN 10225	S 355 G4 + M	355/16	50 (L)	35 (L)	–20	–	–	0,030	0,035	40
		EN 10225	S 355 G1 + N	355/16	50 (L)	35 (L)	–20	–	–	0,030	0,035	40
		EN 10025-3	S 355 N	355/16	40 (L)	28 (L)	–20	–	–	0,025	0,030	250
		EN 10025-3	S 355 NL	355/16	40 (L)	28 (L)	–30	–	–	0,020	0,025	250
		EN 10025-2	S 355 K2	355/16	40 (L)	28 (L)	–20	–	–	0,025	0,025	30
		EN 10025-2	S 355 JO	355/16	40/27(L) [2]	28/19(L) [2]	+20 [2]	–	–	0,030	0,030	30
		EN 10025-2	S 355 J2	355/16	40/27(L) [2]	28/19(L) [2]	0 [2]	–	–	0,025	0,025	30
	Normal strength steel											
Secondary members		Steels for "special" and "primary" members may be used.										

Remarks:
1 options or supplementary requirements of the standard
2 steels with Charpy values of 27 J may be used as 40 J at 20 °C higher test temperature or 30 J at 10 °C higher test temperature

(T) transversal
(L) longitudinal

Table 4.6 Impact energy requirements [1, 2, 3]

	Minimum nominal yield strength [4] [MPa]		**235**	**275**	**290**	**355**	**420**	**460**
Charpy V-Energy [Joule]	Transversal: Special category	Minimum average	27	30	34	40	48	50
		Minimum single	19	21	24	28	34	35
	Longitudinal: primary + secondary category	Minimum average	27	30	34	40	47	50
		Minimum single	19	21	24	28	33	35

1 average value from 3 specimens, one single value may be below
2 steels with Charpy values of 27 J may be used as 40 J at 20 °C higher test temperature or 30 J at 10 °C higher test temperature
3 e.g. 27 J(T) ≅ 40 J(L) (see Table 6, EN 10028-3)
4 intermediate values may be interpolated

Table 4.7 Test temperatures T_T for notch bar impact test [2]

Category of sea (Temperature range)	**Steel category**	**Thickness t of product [mm]**							
		t ≤ 12,5		**12,5 < t ≤ 25**		**25 < t ≤ 50**		**t > 50**	
Polar sea e.g. Alaska Sea, Bering Sea (T_D [1] ≤ –20 °C)	T_D	–40 °C	–20 °C	–40 °C	–20 °C	–40 °C	–20 °C	–40 °C	–20 °C
	Special	–40 °C	–20 °C	–60 °C	–40 °C	–60 °C	–60 °C	–60 °C	–60 °C
	Primary	–40 °C	–20 °C	–40 °C	–20 °C	–60 °C	–40 °C	–60 °C	–60 °C
	Secondary	–20 °C	–20 °C	–20 °C	–20 °C	–40 °C	–20 °C	–60 °C	–40 °C
Cold sea e.g. North sea, Baltic Sea, Irish Sea (–15 °C ≤ T_D ≤ 0 °C)	T_D	–15 °C	0 °C	–15 °C	0 °C	–15 °C	0 °C	–15 °C	0 °C
	Special	–15 °C	0 °C	–35 °C	–20 °C	–55 °C	–40 °C	–55 °C	–40 °C
	Primary	–15 °C	0 °C	–15 °C	0 °C	–35 °C	–20 °C	–55 °C	–40 °C
	Secondary	+20 °C	+20 °C	+20 °C	+20 °C	–15 °C	+20 °C	–35 °C	–20 °C
Temperate sea e.g. English Channel, Bay of Biscay, West Mediteranean (0 °C ≤ T_D ≤ +15 °C)	T_D	0 °C	+15 °C	0 °C	+15 °C	0 °C	+15 °C	0 °C	+15 °C
	Special	0 °C	0 °C	–20 °C	–20 °C	–40 °C	–40 °C	–40 °C	–40 °C
	Primary	0 °C	0 °C	0 °C	0 °C	–20 °C	–20 °C	–40 °C	–40 °C
	Secondary	+20 °C	+20 °C	+20 °C	+20 °C	0 °C	0 °C	–20 °C	–20 °C
Warm sea e.g. Gulf of Guinea, Arabian and Persian Gulf, Indian Sea, East Mediteranean (T_D > +15 °C)	T_D	+15 °C	+25 °C	+15 °C	+25 °C	+15 °C	+25 °C	+15 °C	+25 °C
	Special	0 °C	0 °C	0 °C	0 °C	–20 °C	–20 °C	–40 °C	–40 °C
	Primary	+20 °C	+20 °C	+20 °C	+20 °C	0 °C	0 °C	–20 °C	–20 °C
	Secondary	+20 °C	+20 °C	+20 °C	+20 °C	+20 °C	+20 °C	0 °C	0 °C

1 T_D = design temperature to be determined in accordance with Section 1, F.1
2 In general a test temperature T_T of less than –60 °C is not to be stipulated.
3 For intermediate temperatures T_D the test temperatures may be interpolated.

4.5.2 Reference thickness and thickness limits for stress relieving treatment

Stress relieving treatment is required for materials of a Reference Thickness exceeding 50 mm for Special Category and high strength steels of Primary Category members joined by full penetration welds.

In a full penetration weld (e.g. butt weld, T, K, Y welds, etc.) the Reference thickness is that thickness which governs the thickness of the weld. For example: thickness of thinner member in a butt weld; thickness of stub in stub/can junction in a tubular node.

4.5.3 Post weld heat treatment (PWHT)

This heat treatment shall be defined by the steel producing mill and shall be transmitted to GL for information, see EN 10225, Chapter 8.3.2. The holding time shall be 2 minutes per mm of thickness. Acceptance tests shall then be performed on samples having undergone the heat treatment under simulated conditions. The results obtained shall comply with requirements stated in this specification concerning steel qualities and grades.

4.5.4 Peening

Peening of local area as weld toe or weld transition is an acceptable method to improve fatigue life of structures.

This treatment shall be defined by the distributor of the peening equipment and transmitted to GL for information. Acceptance tests shall then be performed on samples having undergone the peening under simulated conditions. The results obtained shall comply with requirements stated in this specification concerning steel qualities and grades.

4.5.5 Crack tip opening displacement (CTOD)

CTOD testing is required for weld bevel thickness above 50 mm and shall meet a requirement of minimum 0,25 mm (as-welded) and 0,20 mm (PWHT), at –10 °C for design temperature down to –15 °C (Table 4.7: Cold sea) for splash zone or above (0 °C for submerged parts). CTOD-testing of welds shall be carried out with the fatigue notch tip positioned in the coarse grained region of the HAZ and in the weld.

Other test temperatures, especially for a design temperature < –15 °C (see Table 4.7: Polar sea), shall be prescribed by the Designer and approved by GL. (For CTOD-testing see EN 10225 and BS 7448.)

4.6 Impact energy after strain ageing

Steels that are to be subject to a cold or warm forming process (below 250 °C) of more than 5 % strain, either heat treatment shall be performed, or strain ageing tests shall be carried out according to the following procedure:

– Material shall be strained locally to the design deformation.

– Material shall be heated at 250 °C for 1 hour

– Set of 3 impact test specimens shall be tested from the base material in the strained plus heated condition. The notch shall be located within the strained portion, in the part which has received the highest strain.

– The impact test temperature shall be in accordance to Table 4.7.

– The impact energy shall comply with Table 4.6 and not be more than 25 % lower than the impact energy for the material before straining and strain ageing.

If straining is performed at a temperature above 250 °C, it shall be documented that the material properties, weldability, weld and HAZ properties satisfy the material Certificate and this Section.

4.7 Non-destructive testing

4.7.1 Plates and rolled sections

Plates and rolled sections of H and I shaped type (or equivalent) shall be ultrasonic inspected in accordance with EN 10160. UT scanning shall be carried out over 100 % of webs and flanges of rolled sections and edges and body of plates.

Each plate and rolled section of “Special members with Through Thickness Properties” according with 2.2.1 and 4.6 shall be inspected as per “Class S_1/E_2“of EN 10160 or equivalent.

Plates and rolled sections of “Special and primary members without Through Thickness Properties” shall be inspected as per “Class S_0/E_1” of EN 10160 or equivalent.

4.7.2 Welded and seamless tubes

This Section applies to structural tubulars manufactured by pipe mills or tubular workshops which are qualified to produce tubulars according to API specifications.

4.7.2.1 Welded tubes

Steel plates shall be ultrasonically tested in accordance with the requirements of 4.7.1. No welding repairs shall be permitted on plates.

4.7.2.2 Seamless tubes

All tubulars shall be NDT tested over their full length. Ultrasonic testing shall apply for wall thickness of 10 mm and above (see EN 10225, Chapter 8.5.3.2).

4.8 Inspections

Unless otherwise fixed or agreed, plates for special structural members are to be tested individually per rolled length. For all other products the inspection lots as indicated in the standards and/or GL Rules II - Materials and Welding apply.

Inspections of materials for special and primary structural members are to be carried out by GL Surveyors. For materials for secondary structural members delivery with Inspection Certificate 3.1 according to EN 10204 or equivalent may be agreed.

4.9 Identification and marking

4.9.1 All products are to be marked according to the standard such as to enable their clear identification and correlation to the inspection documentation. The minimum components for marking are to be the manufacturer's symbol, heat number and steel grade. Products tested by GL are to be additionally marked with the GL test stamp.

4.9.2 Products which cannot be clearly identified will be rejected, unless the properties stipulated can be determined by re-inspection.

5. Steel forgings and castings

5.1 General

Where steel forgings or castings are intended to be employed for special and primary structural members, relevant material specifications containing all details required for assessment are to be submitted to GL. For primary and secondary structural members appropriate materials in accordance with the standards may be employed.

5.2 Approval test

Steel forgings and castings for special structural members are to be subjected to an approval test by GL. The same applies to primary structural members, where no standardized materials are intended to be employed.

5.3 Supply conditions

All products are to be supplied in heat-treated condition. Heat treatment may comprise

– normalizing

– normalizing and tempering

– quenching and tempering

5.4 Chemical composition and suitability for welding

The composition of forged and cast carbon and carbon manganese steel grades suitable for welding has to meet the minimum requirements defined in Tables 4.8 and 4.9. The values apply to both ladle and product analysis.

Table 4.8 Forged structural steel

Min. Yield strength [MPa]	Max. S content [%]	Max. P content [%]	Max. CEV [%]	Max. P_{cm} [%]
460	0,007	0,015	0,55	0,27
420	0,007	0,015	0,47	0,22
355	0,005	0,015	0,45	0,22

Table 4.9 Cast structural steel

Min. Yield strength [MPa]	Max. S content [%]	Max. P content [%]	Max. CEV [%]	Max. P_{cm} [%]
460	0,005	0,012	0,55	0,28
420	0,008	0,012	0,48	0,26
310	0,008	0,012	0,41	0,24

Weldability data are required for a weld bevel thickness above 25 mm. The documentation of weldability shall be in accordance with EN 10225, Annex E. The heat input shall be 3,5 ± 0,2 kJ/mm (EN 10225, Table E.3).

5.5 Material properties

Product testing shall be carried out for each cast and forged item above 1 ton weight. For smaller forgings or large series another test frequency may be agreed upon. The results obtained shall comply with Tables 4.2, 4.6 and 4.7.

B. Fabrication

1. General

1.1 Fabrication, other than welding, shall be in accordance with a national or regional fabrication specification that complements the design code used. Fabrication tolerances shall be compatible with design assumptions, except where specific service requirements dictate the use of more severe control over the deviations assumed in the design.

1.2 The minimum requirements for the fabrication and construction of steel structures are defined herein. These requirements analogously apply to work carried out on the steel structure prior to or during transport, installation and operation and for repairs. For welding see C. and for inspection and testing see D.

2. Standards, specifications

The application of relevant standards, codes, guidelines, specifications, etc. (e.g. ANSI/AWS D 1.1 "Structural Welding Code-Steel") may be agreed to by GL. Should such standards, codes, guidelines, specifications, etc. contradict the present rules, the latter shall take precedence over all others.

3. Fabrication specification, schedule

A specification containing the main aspects of fabrication and construction is to be submitted to GL for approval. A time schedule is to be submitted for information. For welding procedure and NDT specifications, see C.3.1 and D.6.4 respectively.

4. Quality assurance and control

4.1 Manufacturers will be responsible for compliance with these rules, with the approved drawings and specifications as well as with any other conditions which may be agreed or stated in the approvals. The inspections performed by GL do not relieve manufacturers of this responsibility.

4.2 Manufacturers are to introduce a quality assurance system which ensures that the materials, fabrication and construction are in accordance with these rules and with the approved drawings and specifications or any other conditions stated in the approvals.

4.3 The quality control shall be performed by competent personnel with defined responsibilities covering all aspects of quality control including welding and NDT. The quality assurance department shall be independent from the manufacturing departments.

An organization chart, including proof of qualification of the personnel in charge of control, is to be submitted to GL.

4.4 During fabrication and construction controls are to be carried out, inter alia, as to

– the storage and marking of materials

– the preparation and assembly of components

– the accuracy to size of the components and the overall structure

For checking of welding see C.6.1 and for the final inspections including NDT see D.1.

4.5 The quality assurance is to also cover all work which may affect the integrity and strength of the structure, including the fitting of auxiliary structures and equipment to the structure, fairing and repair work, as well as corrosion protection measures.

4.6 All quality assurance measures are to be recorded, thus proving the kind, scope and results of the checks. A summary analysis of results is to offer a clear picture of the progress of work, including possible rework and controls. The records and analysis are to be presented to the GL Surveyor for approval at reasonable intervals.

5. Deviations, defects and repair work

5.1 Any deviations from the rules, approved drawings and specifications or conditions agreed or stated in the approvals require prior approval by GL. The same applies analogously to major repairs. A proposal for intended repair work is to be prepared by the manufacturer and to be presented to GL for approval, together with a report on defects. Repair work shall be started only following approval of the repair procedure by GL. For admissible tolerances see 15.

5.2 For any such substitutions and revisions made during fabrication, suitable records shall be documented by the fabricator and listed as corrections to the fabrication drawings. The responsibility for the compilation of these records with other documentation related to the construction and inspection of the structure and the retention of these permanent records shall be specified by the Owner.

6. Identification and storage of materials

6.1 All materials (plates, shapes, tubes, etc.) for main (special and primary) structural parts are to be clearly marked as to enable them to be definitely identified on the basis of the pertinent Certificates.

6.2 Non-identified materials are to be rejected, unless compliance with these rules and/or the approved specifications has been verified by renewed testing. The number and type of tests shall be in accordance with A.4. or A.5. respectively.

6.3 Manufacturers are to introduce a system enabling, them to trace the material within the structure. During each stage of manufacture correlation of the pertinent materials Certificates to each individual component shall be possible.

6.4 All materials and structural members are to be stored and handled such as to exclude their surface finish and properties being unduly affected. For the storage of welding consumables and auxiliary materials see C.2.3.

7. Surface and edge preparation of materials and structural members

7.1 When applying thermal (cutting) procedures, the possibility of hardening of the basic material and – depending on the materials employed - variation of the strength and/or toughness values are to be taken into account. For flame straightening, see 13.

7.2 Excessive flame cutting drag lines, (e.g. due to burner failure) are to be ground notch-free with smooth transitions. Where possible, weld repairs are to be avoided and require approval by the GL Surveyor.

7.3 Plates or tubes shall be prepared by machining or by thermal cutting (flame, arc gouging, plasma cutting, etc.) followed by mechanical cleaning where appropriate in accordance with an internationally accepted standard. The same applies analogously to shear cuts, the fins of which are to be rough-planed in all cases.

Surfaces to be welded shall be visually examined to ensure freedom from lamination, carbon deposits, cracks, slag inclusions and cutting notches. Any such defect shall be dressed out by mechanical cleaning. If the depth of excavation exceeds 5 mm, a weld repair shall be made using an approved welding procedure. The surface of the steel within 25 mm of the proposed weld shall be dry, clean and free from scale or paint, except that the use of a protective primer may be qualified by use in a welding procedure qualification test. The clean area shall be increased to 75 mm where required for ultrasonic testing or to avoid noxious fumes.

Flame cut edges that are not subsequently welded shall be ground to remove nicks and burrs. The use of flame gouging or “washing” is not permitted.

7.4 Where practicable, penetrations through main members (e.g. for piping or cables) are to be drilled. Flame-cut penetrations shall, under all circumstances, be completely ground notch-free. The edges are to be chamfered or rounded off.

7.5 The surface of piles and followers shall be untreated bare metal, free from mill varnish, oil and paint, except for specific markings. The area of the surface used for markings shall be the minimum required consistent with adequate identification.

8. Cold and hot forming

8.1 Cold forming resulting in permanent deformation (elongation) by more than 3 % should be avoided and normally requires performance of a strain ageing test of the base material with retesting of mechanical properties. For welding in cold formed areas, see C.5.5.

8.2 The forming of T-sections into flanged ring stiffeners of any diameter may require mechanical tests to demonstrate that the properties required by the application are maintained after forming and welding.

Acceptance of wedges or other techniques to assist rolling will be subject to the visual acceptability of the product and to acceptable mechanical test results from regions of maximum strain.

8.3 Hot forming should normally be performed at a temperature not exceeding 700 °C, unless subsequent heat treatment is carried out and mechanical properties as specified are proved by retests in the final condition. Hot forming is not allowed for TMCP steels.

9. Splices

9.1 Splices shall be fabricated using full penetration welds and subject to the following restrictions. The location of all splices shall be shown in as-built drawings.

9.2 Pipe splices should be in accordance with the requirements of ANSI/API Spec 2B, Section 3.8. Pipes used as beams should also be subject to the requirements of 9.3.

Circumferential welds are not permitted within cones and node stubs unless specified on design or approved shop drawings.

Ring stiffeners shall be at least 100 mm from circumferential seams except that , where this is not possible, the welds shall overlap by at least 10 mm. Exceptions are rings used to stiffen cone-cylinder junctions which shall not be offset and the welds shall be overlapped by at least 10 mm to avoid coincident location of weld toes.

9.3 Beams

Segments of beams with the same cross-sections may be spliced. Either straight or staggered splices may be used for beam or plate splices. Splices should be full penetration.

Splices in plate girders and sections shall not be less than one meter or twice the depth of the beam apart whichever is greater.

Longitudinal stiffeners shall be at least 100 mm from longitudinal seams.

For staggered splices, splices in girder and column flanges shall be at least 100 mm from the web splice.

10. Fitting and assembly

10.1 Excessive jamming is to be avoided during fitting and assembly. Major deformations of individual members are to be faired prior to further assembly. The welding-on of assembly aids is to be restricted to a minimum. For weld preparation, assembly and tack welding, see C.6.3.

10.2 Components which are to be spliced, shall be aligned as accurately as possible. Special attention shall be paid to the alignment of (butting) members, which are interrupted by transverse members. If necessary, such alignment shall be facilitated by drilling check holes in the transverse members which are later seal-welded. The fit-up is to be checked before welding.

For allowable tolerances, see 15.

10.3 When members are slotted to receive gusset plates, the slot should be 305 mm or twelve times the member wall thickness, whichever is greater, from any circumferential weld. To avoid notches the slotted member should be drilled or cut out and ground smooth at the end of the slot with a diameter of at least 3 mm greater than the width of the slot. Where the gusset plate passes through the slot, the edge of the gusset plate should be ground to an approximately half round shape to provide a better fit-up and welding condition.

10.4 All components should be trial fitted before welding and grinding or buttering of the weld preparation at the installation site should be avoided for special and primary steel fabrication. Where root gaps are out of tolerance, the joint shall be dismantled to permit local grinding, or buttering followed by grinding, back to the required profile. The depth of buttering on any member shall not exceed 15 mm. Alternatively the requirements of 5. may be applied to establish an appropriate repair procedure.

10.5 Where butt welds are to be made between members of different thickness, a transition using a taper of 1:4 shall be provided. This taper may be wholly or partially within the width of the weld. When necessary, the thicker member shall be tapered.

10.6 Joints in grating and mechanical joints in deck plates shall be at support points unless the design includes other appropriate details. Welded joints in deck plate should preferably offset from welds to the supporting structures. Mismatches greater than 2 mm at grating and deck plate joints shall be ground down and recoated.

10.7 Rat-holes, penetrations and other cut-outs shall be avoided wherever possible. Rat-holes may be permitted provided they are approved on shop drawings and the following criteria shall apply:

– Permanent rat-holes shall have a radius of 50 mm or twice the plate thickness, whichever is larger, and shall be ground smooth. Where rat-holes of these dimensions would jeopardize structural integrity rat-holes less than this radius may be considered. Fillet welds shall be returned through the rat holes.

– Temporary rat holes (those approved subject to their being reinstated or sealed) shall have a radius not less than twice the plate thickness and shall be reinstated to an approved procedure. Filling with weld metal is not permitted.

– The radius of any approved cut-outs other than rat-holes shall not be less than 100 mm. Where necessary, the area shall be checked by ultrasonic testing before cutting to ensure that no cut is made within 300 mm of internal stiffeners. The original plate shall be clearly marked and shall be welded back into the cut-out, unless other proposals are agreed.

11. Weather protection

Prefabrication is to be performed as far as possible in weather-protected workshops. For assembly the components and in particular the areas, in which welding is performed, are to be protected by provisional coverings against the effects of weather (e.g. rain and wind). For welding under low ambient temperatures and pre-heating see C.6.4.

12. Removal of temporary attachments and auxiliary material

12.1 Clamping plates, aligning ties, scaffolding and other auxiliary materials, temporarily welded on structural members, shall be limited as much as practicable.

12.2 Where attachments are necessary, the following requirements apply.

– temporary attachments shall not be removed by hammering or arc-air gouging. Attachments to leg joint cans, skirt sleeve joint cans, brace joint can, brace stub ends and joint stiffening rings shall be flame cut to 3 mm above parent metal and mechanically ground to a smooth flush finish with the parent metal

– attachments on all areas that will be painted shall be removed as above, prior to any painting

– attachments to all other areas, not defined above, shall be removed by flame cutting just above the attachment weld (maximum 6 mm above weld); the remaining attachment steel shall be completely seal welded

– attachments to aid in the splicing of legs, braces, sleeves, piling, conductors, etc, shall be removed to a smooth, flush finish

12.3 If during rough-planning the surface of the structural members is damaged, the areas concerned are to be gouged flat and ground notch-free and subsequently checked for cracks. Where possible, repair by welding is to be avoided; otherwise, approval by the GL Surveyor is required.

13. Flame straightening

Flame straightening shall not impair the properties of the materials nor of the welded joints and shall be avoided completely, where practicable. In cases of doubt, proof of satisfactory performance of thermal treatment may be demanded.

Note:

Standard flame straightening carried out on higher-strength (normalized) steels may generally be regarded as acceptable, provided that the straightening temperature does not exceed 700 °C and that localized overheating and abrupt cooling (e.g. with water) are avoided. Prior to flame straightening of high-strength (quenched and tempered) structural steel, special agreement based on steel manufacturers recommendations is required.

14. Heat treatment

If heat treatment is required, it is to be performed in accordance with a procedure specification, listing in detail:

– structural member, dimensions and material

– heating facilities and insulation

– control devices and recording equipment

– heating and cooling rates (temperature gradients)

– holding range and time

The specification is to be submitted to GL for approval. For post-weld heat treatment, see C.6.8. Regarding MCP steels see 8.2 (heat treatment not allowed).

15. Fabrication tolerances

15.1 General

15.1.1 Allowable fabrication and construction tolerances shall be specified depending on the significance of the structural members and the loads acting on them. They shall be submitted to the Owner for acceptance. The resulting specification is to be submitted to GL for approval. A general guidance for allowable tolerances is given in 15.2 to 15.12.

15.1.2 For weld imperfections see C.6.6 and D.6.4. In case of fatigue assessment the detail classification of the different welded joints as given in Table 3.13 is based on the assumption that the fabrication of the structural detail or welded joint, respectively, corresponds in regard to external defects at least to quality group B according to DIN EN ISO 5817 and in regard to internal defects at least to quality group C.

15.2 Length tolerances

The length tolerances are defined in Table 4.10.

Table 4.10 Length tolerances according to DIN EN ISO 13920

Nominal length [mm]	Special and primary structures	Secondary structures
2 – 30	± 1	± 2
120 – 400	± 1	± 2
401 – 1000	± 2	± 3
1001 – 2000	± 3	± 4
2001 – 4000	± 4	± 6
4001 – 8000	± 5	± 8
8001 – 12000	± 6	± 10
12001 – 16000	±7	± 12
16001 – 20000	± 8	± 14
above 20000	± 9	± 16

15.3 Misalignment of butt (weld) joints

The misalignment e shall not exceed the following, compare Fig. 4.1:

$e \leq 0{,}10 \cdot t_1$, for special structures

$e \leq 0{,}15 \cdot t_1$, for primary structures

$e \leq 0{,}30 \cdot t_1$, for secondary structures

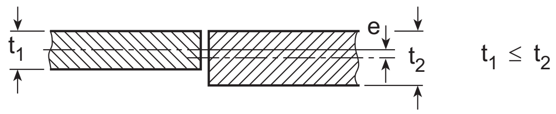


Fig. 4.1 Definition of misalignment at butt joints

15.4 Misalignment of cruciform (weld) joints

The misalignment e shall not exceed the following, compare Fig. 4.2:

$e \leq 0{,}15 \cdot t$, for special structures

$e \leq 0{,}30 \cdot t$, for primary structures

$e \leq 0{,}50 \cdot t$, for secondary structures

t is the smallest thickness of t_1, t_2 and t_3.

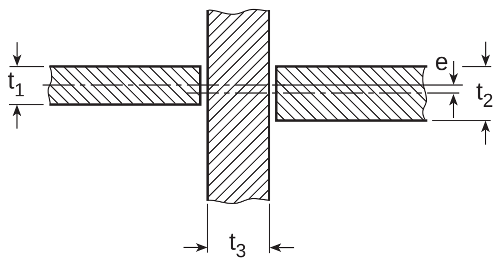


Fig. 4.2 Definition of misalignment at cruciform joints

15.5 Roundness of tubular members

15.5.1 The external circumference shall not differ from the design or calculated circumference by more than ± 0,30 % of the specified external diameter.

15.5.2 The difference between the measured maximum and minimum diameters shall not exceed 1 % of the specified diameter or 6 mm, whichever is less.

15.5.3 The maximum deviation from the nominal diameter, measured at a stiffener or a bulkhead shall not exceed 0,50 %.

15.5.4 Lower values and/or additional limitations may be required for masts, crane pillars and similar components of lifting appliances.

15.5.5 The global and local tolerances for circular cylinder shells (e.g. of flotation tanks, buoyancy structures, etc.) exposed to outer pressure are to be calculated and specified in each single case depending on maximum pressure, diameter to wall thickness ratio and stiffener distance to diameter ratio.

15.5.6 For limitations regarding buckling, see Section 3, G.4.

15.5.7 For deviations of longitudinal stiffeners see 15.9 and 15.10

15.5.8 Local deviations out of roundness and out of straightness.

The misalignment e shall be (see Fig. 4.3):

$$e \leq 0{,}01 \cdot \frac{g}{1 + g/r}$$

A circular template or straight rod held anywhere on the shell.

g = length of template or rod

The length of the circular template shall be the smallest of:

– s

– $1{,}15 \cdot \text{sqrt}(l \cdot \text{sqrt}(r \cdot t))$

– $\pi \cdot \dfrac{r}{2}$

s = stiffener spacing (of longitudinal stiffeners)

= distance between rings or bulkheads

The length of the straight rod shall be taken equal to the smallest of:

– l

– $4 \cdot \text{sqrt}(r \cdot t)$

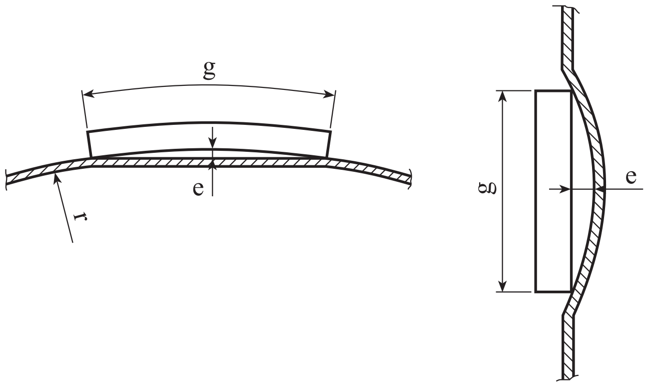


Fig. 4.3 Definition of misalignment of a circular template or straight rod

15.6 Straightness of (prefabricated) tubular members and girders or beams

The deviation of straightness e shall be ≤ 0,001 · L or 3 mm, whichever is greater, for compression members, e.g. columns, primary truss members and girders or beams with non-hollow cross sections, compare Fig. 4.4.

The deviation of straightness shall be e ≤ 0,002 · L or 5 mm, whichever is greater, for girders or beams with hollow cross sections and for all other applications.

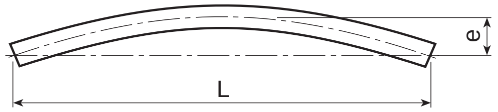


Fig. 4.4 Definition of straightness

15.7 Deviation from the theoretical axis in tubular nodes

If the specified (designed) eccentricity x is less D/4, the deviations are defined as follows, compare Fig. 4.5:

– x as built < x designed (x_1 in Fig. 4.5):
  $e_1 \leq x_1$, but separation requirement 50 mm minimum (see Fig. 4.5) to be observed

– x as built > x designed (x_2 in Fig. 4.5):
  $e_2 \leq 0{,}1 \cdot x_2$, maximum 50 mm

If the specified (designed) eccentricity x > D/4, which has to be considered in the strength calculations, the deviations are:

– $e_{1,2} \leq 0{,}1 \cdot x_{1,2}$, maximum 50 mm
  separation requirement of 50 mm minimum to be observed, see Fig. 4.5

15.8 Bulges in plating

The deviations e of bulges in plating from straight line are defined as follows, compare Fig. 4.6

$$e \leq 0{,}004 \cdot s$$

s = stiffener distance

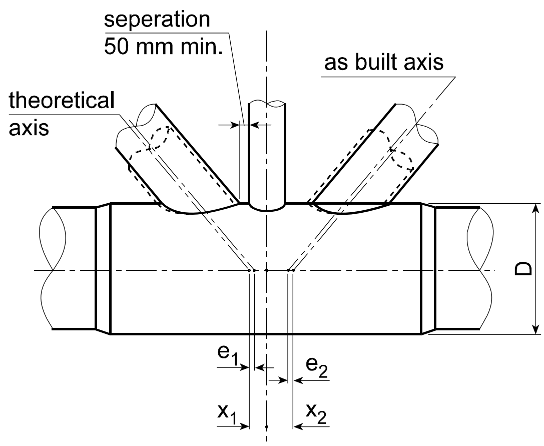


Fig. 4.5 Deviations at tubular nodes

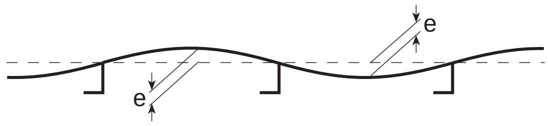


Fig. 4.6 Definitions of bulges in plating

15.9 Deformation of plane, stiffened platings

The permissible deviation e of deformations in plane, stiffened platings from straight line is defined as follows, compare Fig. 4.7:

$$e \leq 0{,}0015 \cdot L$$

Dimensions of e and L in [m].

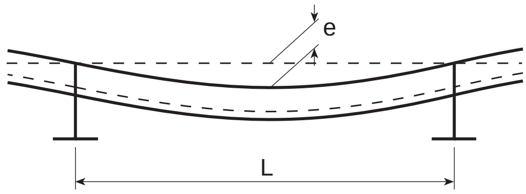


Fig. 4.7 Definition of plating deformation

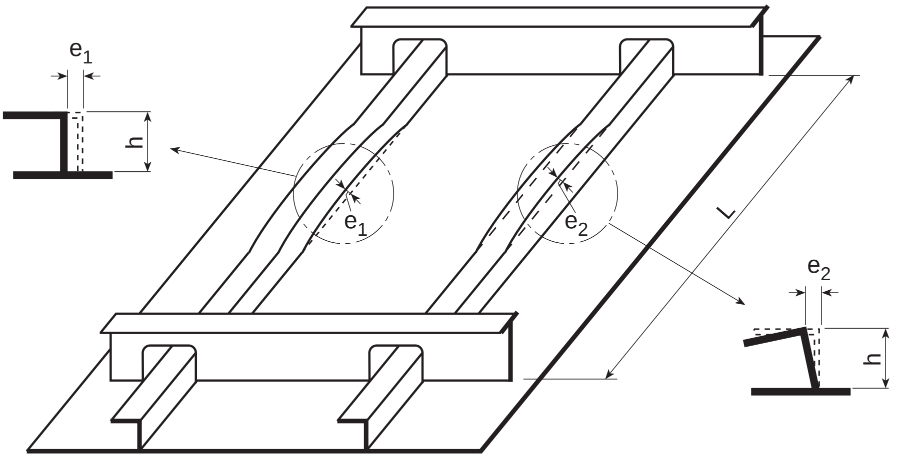


Fig. 4.8 Definition of deviations of stiffeners between two supports

15.10 Lateral miss-positioning or deflection of stiffeners between supporting members

The permissible lateral miss-positioning or deflection of stiffeners between two supporting members is defined as follows, compare Fig. 4.8:

$e_1 \leq 1 + 0{,}015 \cdot L$, maximum 5 mm

$e_2 \leq 1 + 0{,}01 \cdot h$, maximum 3 mm

e_1, e_2, L, h are dimensions in [mm]

Lower values may be required for reasons of buckling strength.

15.11 Deviation and/or distortion of girders

The permissible deviation and/or distortion of girders is defined as follows, compare Fig. 4.9, left:

$e_1 \leq 1 + 0{,}01 \cdot h$ maximum 3 mm

$e_2 \leq 0{,}01 \cdot h$ maximum 2 mm

e_1, e_2, h dimension [mm]

The permissible deflection of the face plate to the girder plane is defined as follows, compare Fig. 4.9, right:

$e_3 \leq 1 + 0{,}01 \cdot b$ maximum 3 mm

$e_4 \leq 0{,}01 \cdot b$ maximum 2 mm

e_3, e_4, b dimension [mm]

Lower values may be required for reasons of buckling strength.

15.12 Lateral deflection and inclination of (as built) columns

The lateral deflection e_1, compare Fig. 4.10, shall be in accordance with the requirements defined in 15.6 if L = H [mm].

The permissible deflection e_2 shall be less or equal $0{,}0015 \cdot H$.

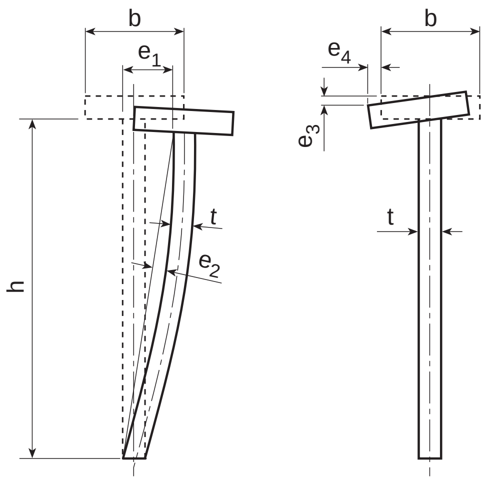


Fig. 4.9 Definition of deviations of girders

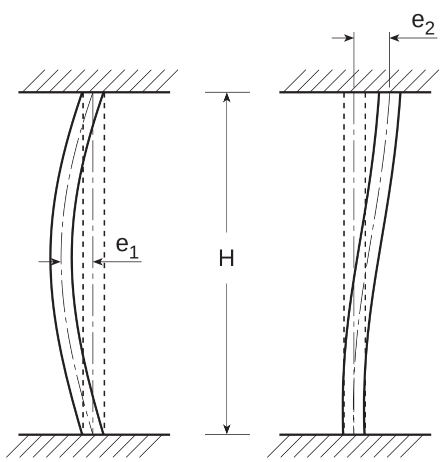


Fig. 4.10 Definition of column deviations

15.13 Tolerances for trusses

Tolerances for trusses are to be specified analogously to those given in 15.7 (deviation from the theoretical axis in tubular nodes), 15.11 (deviation and/or distortion of girders) and 15.12 (lateral deflection and inclination of as built columns). Special consideration is to be given to areas of introduction or deviation of forces, e.g. at stabbing points or connections of crane pedestals.

C. Welding

1. General requirements

1.1 Scope, welding rules

These Rules define the general conditions for the preparation and performance of all welding work including quality assurance measures. It is taken for granted that all further details of welding (not listed here) will be laid down in the welding specifications analogously to the GL Rules II – Materials and Welding, Part 3 – Welding, in the following referred to as "GL Welding Rules", or to other standards, codes, etc. recognized by GL.

1.2 Standards, codes

1.2.1 The standards, codes, etc. mentioned below or in the GL Welding Rules constitute an integral part of these Rules and require no special consent. The version current at the time when these Rules are issued shall be applied. New editions of the standards, codes etc. may be used in agreement with GL.

1.2.2 The application of other standards, codes, etc. may be agreed by GL. They are to be listed in the construction particulars, e.g. in the welding specifications, and presented to GL on request. The actual version of these standards shall be applied; otherwise the year of publication shall be indicated.

1.2.3 If any of these standards, codes, etc. contradict these Rules, the latter shall take precedence over all the others. Any deviations require approval by GL from case to case.

1.3 Terms and definitions

The terms and definitions as contained in recognized standards, codes, etc. are to be employed. The standard or code used is to be indicated in the design and/or construction documentation and presented to GL on request. Any other deviating terms and definitions are to be separately explained in the welding specifications.

1.4 Welding works equipment

The manufacturers (welding works) shall have suitable equipment and plant for the satisfactory performance of welding work. Apart from perfectly functioning cutting and welding machines and tools, this equipment shall include dry, heatable storage facilities, redrying ovens and heated holding boxes for the welding consumables, and equipment for preheating and temperature control.

1.5 Welding supervision

The preparation and performance of welding operations shall be supervised by specially trained and experienced welding supervisors. Proof of the latters' professional qualifications is to be furnished to GL; their tasks and responsibilities are to be defined and made known to GL. For more details see GL Rules II – Materials and Welding, Part 3 - Welding.

1.6 Performance of qualification tests

Qualification tests are to be performed under the supervision of GL on the manufacturers' premises. Fabrication conditions shall be simulated as far as possible. Non-destructive and destructive tests may – in agreement with GL - be performed on the premises of another independent, neutral testing body. Unless otherwise stated, sampling, specimens and performance of tests shall be in accordance with the GL Rules II – Materials and Welding, Part 3 -Welding.

1.7 Documentation

The manufacturer is to produce and to submit to GL prior to production, a documentation covering the main aspects of welding work as follows:

– drawings showing all weld details
– welding procedure specifications
– non-destructive testing specifications and plans of the welds
– procedure qualification records
– welding supervisors' qualification
– welders qualification Certificates
– welders identification system

For documentation on materials, see A.4.9 and B.6., for weld inspection and NDT, see D.1.7.

2. Welding consumables

2.1 Approval of welding consumables

2.1.1 All welding consumables and auxiliary materials (covered electrodes, wire electrodes, wire-gas combinations or wire-flux combinations) intended to be used on offshore steel structures shall be approved by GL in accordance with the GL Welding Rules for the respective range of application (base materials, welding positions, heat treatment condition, etc.)

2.1.2 Welding consumables and auxiliary materials not approved by GL according to the GL Welding

Rules may be qualified - with special agreement by GL - in the course of the welding procedure qualification tests. In this case, one round tensile test specimen is to be additionally taken lengthwise, mainly from the weld metal of the butt welds.

Such qualifications are, however, limited to the user's work in the range of application according to the approved welding procedure specifications, and remain valid for not longer than one year, unless repeat tests are carried out in accordance with the GL Welding Rules.

2.2 Selection of welding consumables

2.2.1 The welding consumables and auxiliary materials are to be chosen such that in the weld connection, including the areas of transition to the base material, at least the same mechanical properties as those specified for the base materials are achieved. When different steels are to be joined, the welding consumables and auxiliary materials are to be chosen analogously to those for hull structural steels in the GL Welding Rules.

2.2.2 Welding consumables and auxiliary materials with a controlled low hydrogen content in the weld metal (symbols HH or HHH according to the GL Welding Rules) shall be used for welding of higher and high strength steels (yield strength > 285 N/mm²), for all steels having a carbon equivalent ≥ 0,42 %, and are recommended for all highly stressed and thick-walled components exposed to low temperatures as well as for steel forgings and castings.

Welding consumables and auxiliary materials other than low hydrogen types may only be used with special agreement by GL, except for secondary structures.

2.3 Storage and redrying of welding consumables

2.3.1 All welding consumables and auxiliary materials (except shielding gases) are to be stored - as far as possible in the original packages - in a storage room, where a minimum temperature of 18 °C and a humidity of the atmosphere of less than 60 % is maintained. The manufacturer's recommendations are to be observed. The welding consumables and the auxiliary materials shall be properly identifiable up to their final use.

2.3.2 Prior to use, low hydrogen covered electrodes, folded flux core wire electrodes and fluxes are to be redried (baked) in accordance with the manufacturer's instructions (observing the maximum baking time specified).

Unless otherwise specified, the baking temperature should be 250 to 350 °C, the baking time ≥ 2 hours, but not longer than 12 hours in total. Repeated baking is permissible, as long as the maximum drying period of 12 hours is not exceeded.

2.3.3 At the place of work, the welding consumables and auxiliary materials are to be protected against the effects of weather (moisture) and contamination. Low hydrogen covered electrodes and fluxes are to be kept in heated holding boxes at temperatures between 100 and 150 °C. All unidentifiable, damaged, wet, rusty or otherwise contaminated welding consumables and auxiliary materials are to be discarded.

3. Welding procedure specification and qualification

3.1 General

Detailed welding procedures shall be prepared for all welding covered by these rules. The welding procedure may be based on previously qualified welding procedures provided that all the specified requirements can be fulfilled.

3.2 Welding procedure specification WPS

A welding procedure specification (WPS) is a specification based on a WPQR and accepted in accordance with those requirements. The WPS is the pWPS revised to reflect the welding variables qualified by the WPQ. All production welding shall be performed in accordance with a WPS.

WPS are to be prepared and submitted to GL for review. The WPS are to contain all essential data on the preparation and performance of weldings, such as

– materials (steel grades)
– joint and groove design
– joint preparation (cutting method) and tacking
– welding process and parameters
– welding consumables and auxiliary materials
– welding positions and progressions
– preheating, working temperatures (heat input)
– weld build-up (welding sequence)
– root treatment (back gouging)
– postweld (heat) treatment

WPS are to be submitted for new as well as for repair welds. For specifications to be submitted concerning inspection procedures, see D.6.4.

3.3 Preliminary welding procedure specification pWPS

A preliminary welding procedure specification (pWPS) shall be prepared for each new welding procedure qualification. The pWPS shall specify the ranges for all relevant parameters.

The pWPS shall be submitted for review and acceptance by the purchaser prior to commencing the welding procedure qualification.

3.4 Welding procedure qualification WPQ

3.4.1 The manufacturers (welding works) shall prove their ability to apply the specified welding procedures in a sufficient manner and with satisfactory results, in conjunction with the materials and welding consumables actually used. Such proof is to be provided prior to fabrication.

3.4.2 Unless otherwise agreed, a welding procedure qualification (WPQ) shall be proved by a welding procedure qualification test (WPQ-Test) under actual or simulated fabrication conditions, and witnessed by GL.

3.4.3 In exceptional cases, GL may recognize in part or in whole existing welding procedure qualifications based on tests witnessed and approved by another competent and independent authority. Therefore, adequate documentation, including complete test reports, is to be submitted to GL.

3.4.4 The approval for a welding procedure qualification is restricted to the manufacturer and to the welding works or working unit (e.g. barge), where the test welds were performed. The approval is valid within the limitations of the essential variables according to 3.6.

3.4.5 The approval for the use of the qualified procedure is normally valid for the fabrication period of the structure. Where the fabrication has been interrupted for more than 3 months, GL may require production tests or repeated welding procedure qualification tests.

3.4.6 Production tests or repeated welding procedure qualification tests may also be required if the conditions under which the approval was granted (e.g. personnel premises) do not longer apply, or in the event of doubts as to satisfactory execution of the welding.

3.5 Welding procedure qualification tests

3.5.1 Welding procedure qualification tests are to be carried out in accordance with the GL Welding Rules or - with the consent of GL - subject to other recognized standards or codes (e.g. ANSI/AWS Structural Welding Code (Steel) D.1.1). They comprise the welding of test specimens and their non-destructive as well as mechanical and technological testing as per Table 4.11.

3.5.2 For the welding procedure qualification tests a programme is to be prepared and submitted to GL for approval. The different base materials and welding consumables, prematerials (e.g. plates or tubes), plate and/or wall thicknesses, seam forms and weld positions, as well as the effects of seam preparation and possibly back-gouging at the workshop are to be taken into account.

3.5.3 The base materials for the test assemblies shall be of the same type as those used for the steel construction and shall be selected such that the carbon content and the carbon equivalent value are in the upper range of the specified values. The rolling direction shall be parallel to the weld seam.

3.5.4 Non-destructive testing of the test assemblies is to be performed in the same manner as specified in D. Sampling of test specimen and mechanical testing shall comply with the GL Welding Rules or with the standard or code applied. The results shall meet the requirements according to Table 4.12.

3.5.5 GL may require additional tests (e.g. fracture mechanic tests for higher and high tensile steels and/or heavy welded joints) as well as other test conditions (e.g. test temperatures for the Charpy impact tests), or may specify other (more stringent) acceptance requirements.

3.6 Welding procedure qualification record WPQR

The welding procedure qualification record (WPQR) shall be a record of all welding data, including seam preparation, back gouging, preheating, interpass temperatures, a possible post weld treatment and subsequent non-destructive or destructive test results. The WPQR shall be submitted for review and agreement prior to start of production. The complete WPQR with satisfactory test results qualifies a pWPS to a WPS.

3.7 Limits application of qualified welding procedures

3.7.1 Approved welding procedures may be used within the limitations of the essential variables regarding

- materials
- thickness
- tube diameter (if applicable)
- joint configuration
- groove design
- welding process
- welding positions and progression
- welding consumables
- welding parameters
- preheating and working temperatures
- number and sequence of layers
- post-weld heat treatment

as laid down in the following for welding above water. For welding under water see 3.9.5. GL may specify other limits if the peculiarities of a welding procedure, the material, the joint configuration and thicknesses or other parameters have a considerable effect on the properties of a weld.

Table 4.11 Welding procedure qualification tests - types and number of tests

Joint type	Thickness [mm]	Non-destructive testing	Mechanical-technological tests				Others
			Transverse tensile test specimen	Transverse bend test specimen	Charpy impact test specimen	Macro etchings/ hardness test specimen	
Butt welds (Plates and tubes)	t ≤ 30	100 % X-ray + MP		1 face [2] 1 root	4 × 3 [3, 5]	2/2 rows [6]	
	t > 30	100 % UT + MP	2 [1]	2 side	6 × 3 [3, 4, 5]	2/3 rows [6, 7]	
T-joints (full penetration)	t ≤ 30	100 % UT + MP	–	–	4 × 3 [3, 5]	2/2 rows [6]	
	t > 30				6 × 3 [3, 4, 5]	2/3 rows [6, 7]	
T-joints (filled welds)	all	100 % MP	–	–	–	2/2 rows [6]	see 3.5.5
Tubular joints (nodes)	t ≤ 30	100 % UT + MP	–	–	4 × 3 [3, 5]	3/2 rows [6]	
	t > 30				6 × 3 [3, 4, 5]	3/3 rows [6, 7]	

[1] Additional round tensile test specimen to be taken longitudinally from weld metal if welding consumables are not approved by GL, see 2.1.2.

[2] For > 20 mm 2 side bend test specimens may be chosen instead of 1 face and 1 root bend test specimen.

[3] Charpy impact test specimen to be sampled approximately 1 mm below base material surface at the capping side, notch positioned perpendicular to surface.

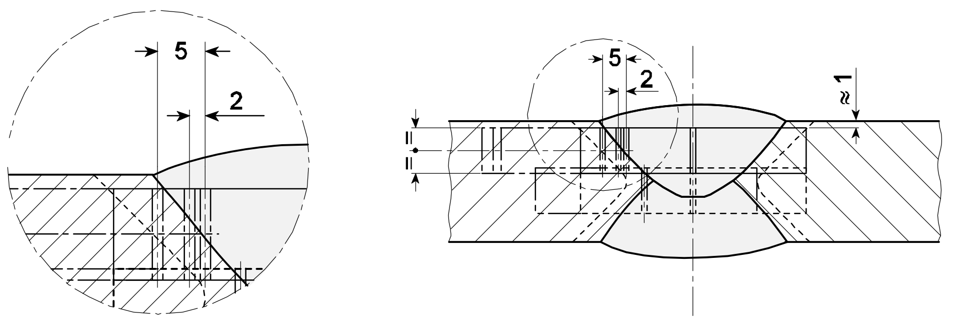
- in the centre of the weld
- on the fusion line (50 % weld metal, 50 % heat affected zone)
- 2 mm from fusion line in the heat affected zone
- 5 mm from fusion line in the heat affected zone.

see sketch below, T- and tubular joints analogously, see 3.5.4 below

[4] Additional charpy impact test specimen to be sampled in the root region with the notch positioned perpendicular to surface

- in the centre of the weld
- on the fusion line (50 % weld metal, 50 % heat affected zone).

see sketch below, T- and tubular joints analogously, see 3.5.4 below



[5] If different welding consumables or welding processes are used for the same joint, impact testing shall be performed for all related areas of the weld.

[6] One hardness measurement row 1 mm below each surface, measuring points concentrated near fusion line in the heat affected zone with some spots in the weld metal and base material.

[7] Additionally 1 row in the root area.

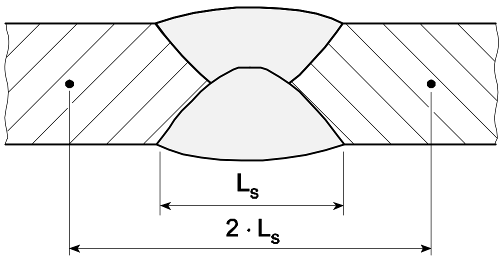
Table 4.12 Welding procedure qualification tests - acceptance requirements

Type of test	Acceptance requirements
Non-destructive testing	The soundness of the welds is to comply with the acceptance requirements laid down in D. for special and primary structures
All weld metal tensile tests [1]	Minimum yield strength and tensile strength as specified for the base material to be welded [2].
Transverse tensile tests	The ultimate strength of the welded joint is to be at least equal to the minimum tensile strength specified for the base material.
Face/root and side bend tests	Bending angle 180°. Mandrel diameter 3 × t for normal strength (mild) and higher strength steels, 4 × t for high strength steels. Minor defects (e.g. pore exposures) up to a maximum length of 3 mm may be tolerated. Bending elongation (Gauge length = 2 × width of the weld) [3] shall meet the requirements specified for the base material.
Charpy impact test (V-notch)	Average and minimum absorbed energy for each notch position (centre of weld, fusion line and heat affected zone) shall meet the base metal requirements. Test temperature as specified for the base material.
Hardness tests	The maximum hardness shall not exceed 350 HV 5 (10) at any position of the welded joint section for normal strength (mild) and higher strength steels.
Macro etchings	The welded joint shall show a regular profile with smooth transitions to the base material and without significant undercuts or excessive reinforcement. Incomplete penetration, lack of fusion and cracks are not acceptable. Minor porosity or slag inclusions may be tolerated.

1 If required according to section 2.1.2.

2 The tensile strength of all weld metal for welding high strength steels may be up to 10 % below the minimum tensile strength of the base material, provided that the results obtained with the transverse tensile test specimen across the weld meet the requirements. Special consideration is to be given for material/wall thicknesses ≥ 50 mm.

3 2 L_S according to EN 910, see sketch below.



3.7.2 Changes in the variables as specified in 3.7.3 to 3.7.14 and 5.5, respectively, are to be considered as being essential and in general require performance of a new or additional procedure qualification test unless otherwise agreed. When a combination of welding processes is used, the variables applicable to each process shall apply. See also ANSI/AWS Structural Welding Code (Steel) D 1.1, Section 4.7.

3.7.3 Materials

Changes in the chemical composition significantly affecting weldability, or in the type of quality grade regarding mechanical properties.

3.7.4 Thickness

Changes in plate/wall thicknesses beyond the ranges given in Fig. 4.11.

3.7.5 Tube diameter

Changes from larger (outside) diameter to smaller diameters exceeding 0,6 × outside diameter of the tested tube. Tubes with diameters greater than 600 mm are to be equated with plates. Special agreements may be made for fully mechanized welding procedures.

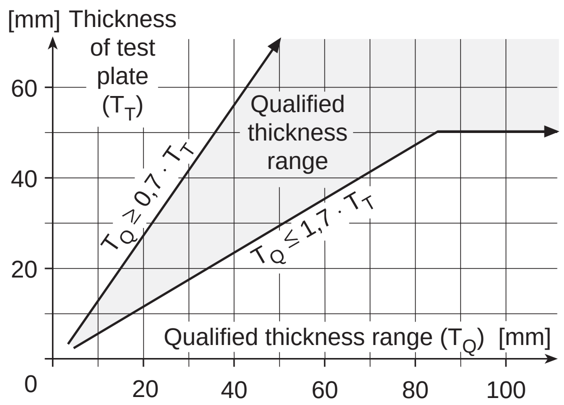


Fig. 4.11 Qualified thickness range

3.7.6 Joint configuration

Change from butt welds to T-joints and tubular joints and vice versa. Butt joints in tubes of more than 600 mm diameter are to be equated with plates and may be covered by welding procedure qualification tests on plates in the relevant positions (see also Table 4.11).

3.7.7 Groove design

Change from double side welding to single side welding steady change of included angle of more than 15° and change of specified root face and root gap significantly affect steady fusion and penetration conditions.

3.7.8 Welding process

Any change**.**

3.7.9 Welding position and progression

Change from one principal welding position and progression to another, unless covered by the inclusions given in Table 4.13.

Table 4.13 Qualified welding positions

Test weld position [1]		Qualified positions [1]		
Joint configuration	**Plate or tube position**	**Butt welds**		**Fillet welds Plates and tubes**
		Plates	**Tubes**	
Plates, butt welds	F/1G	F/1G	F/1G [2]	–
	H-V/2G	H-V/2G	F/1G and H-V/2G [2]	–
	V/3G	V/3G	–	–
	O/4G	O/4G	–	–
Tubes, butt welds [3]	F/1G	F/1G		–
	H-V/2G	F/1G and H-V/2G		–
	TH/5G	F/1G, V/3G and O/4G		–
	TI/6G	F/1G, H-V/2G, V/3G and O/4G [4]		–
	TR/6GR	F/1G, H-V/2G, O/4G and TI/6G [4]		–
Fillet welds	F/1F	–	–	F/1F
	H/2F	–	–	F/1F and H/2F
	V/3F	–	–	V/3F
	O/4F	–	–	O/4F

1 Welding positions: (see Annex B)

F/1G/1F = Flat (downhand) position
H/2F = Horizontal (fillet) weld
H-V/2G = Horizontal-vertical (groove) weld
V/3G/3F = Vertical (upward)position

O/4G/4F = Overhead position
TH/5G = Tube horizontal fixed
TI/6G = Tube inclined (45°) fixed
TR/6GR = Tube inclined (45°), fixed, with restriction ring (simulating T-, Y-or K-connections)

not applicable, for information only: V-D/3G/3F = Vertical (downward) position

2 Qualifies also for butt welds in tubes with outside diameters of more than 600 mm.

3 Tubular joints (node sections) are to be qualified separately.

4 To be applied for welders qualification tests only.

3.7.10 Welding consumables

Any change to another type or quality grade within the welding consumables (covered or wire electrodes, wire-flux or wire-gas combinations) approved by GL, and any change to welding consumables not approved by GL.

Change to electrode/wire diameters other than used in the procedure qualification test.

3.7.11 Welding parameters

Any change of type of current and polarity. Change beyond ± 15 % each of mean voltage and mean amperage and change beyond ± 10 % of welding speed. Changes of heat input (kJoules per cm) for quenched and tempered steels beyond ± 10 % of specified range.

3.7.12 Preheating and working temperatures

Changes ± 25 % beyond specified minimum or maximum preheating and working temperature range.

3.7.13 Number and sequence of layers

Change of more than ± 25 % of the specified number of passes. If the cross section of the groove is changed, it is permissible to change the number of passes in proportion to the cross section. Change from welding with temper beads to welding without temper beads.

3.7.14 Postweld heat treatment

Change from as welded condition to postweld heat treated condition and vice versa. Change beyond specified temperature range, holding time and heating/cooling rates (temperature gradients).

3.8 Qualification for tubular node connections

3.8.1 For full penetration groove welds between tubes and/or plates or girders (T-, Y- or K-joints) forming node connections, welding procedure qualification tests are to be performed analogously to the requirements stated above and following the indications given in 3.8.2 to 3.8.4. A detailed test programme is to be prepared by the manufacturers in agreement with GL from case to case.

3.8.2 The test programme shall cover the different joint configurations, tube diameters and wall thicknesses, angles included between the axes of the tubes as well as the different welding positions for each individual groove type. If not otherwise required, the test pieces may be formed as a brace to can connection according to Fig. 4.12.

3.8.3 When the diameter "D" (of the can) exceeds 600 mm, this tube may be replaced by a plate of same or equivalent steel grade and of relevant size and similar thickness, but not less than 25 mm. When other types of welding (double side, partial penetration

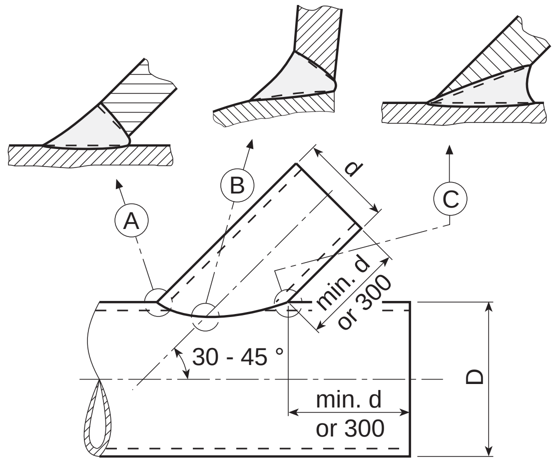


Fig. 4.12 Tubular node test piece

groove or fillet weldings) are intended and approved for fabrication, these types are to be qualified separately. Double side welding may be covered by single side welding qualification with GL's consent in each individual case.

3.8.4 The joint is to be tested by ultrasonic and magnetic particle method. From each position marked with a circle (A, B, C) in Fig. 4.12 one set of test specimens as required in Table 4.9 is to be prepared and tested. Charpy impact test specimens are to be taken from positions "A" and "C" only and those from fusion line and heat affected zone shall be placed on the bracing side, if not required otherwise. The acceptance requirements given in Table 4.10 apply.

3.8.5 In addition to the limitations and essential variables given in 3.4, for tubular node connections the following limitation shall apply: Change of more than 5° from the included angle between the axes of the tubes or girders to smaller angles requires requalification when the included angle is 45° or less.

3.9 Qualification for underwater welding

3.9.1 Underwater welding, to be used for construction or repair, is to be qualified by a test, following the scheme given for welding above water, conducted under actual welding conditions at the welding site or simulated at a test facility. The test is to be carried out at the same water depth or at similar conditions as for the actual welding operation.

3.9.2 For underwater welding generally a low hydrogen process shall be used, carried out in a dry chamber under pressure equivalent to the water depth or under one atmosphere pressure (habitat welding).

Underwater wet welding, where the arc is working in the water or in a small gas-filled cavern, is normally restricted to minor or temporary repair work in shallow waters and requires GL’s consent in each individual case.

3.9.3 Prior to the qualification test, a detailed welding procedure specification is to be prepared by the manufacturer and submitted to GL for review. In addition to the particulars mentioned in 3.1, the following data are to be specified:

– maximum and minimum water depth

– chamber (habitat) atmosphere (gas composition and pressure)

– maximum humidity in the chamber (habitat)

– temperature fluctuations inside the chamber

– electrode transport and monitoring system

– inspection schedule and methods

3.9.4 Based on the welding procedure specification according to 3.9.3 a welding procedure qualification test programme is to be established by the manufacturer (welding works) and submitted to GL for approval. The manufacturer shall also document his previous experience with similar welding work, otherwise an extended testing may be required by GL. Testing and acceptance requirements given in 3.5 apply analogously.

3.9.5 In addition to the limitations and essential variables given in 3.4, for underwater welding the following variations are considered to be essential:

– change from habitat welding to wet welding and vice versa

– change in water depth (working pressure) beyond the agreed range

– change in habitat atmosphere (gas composition)

– change in welding consumables

– change in welding current/polarity

– change in welding parameters (amperage, voltage, travel speed) of more than 10 % (mean value)

Regarding production tests to be performed prior to actual welding see 6.9 and 6.10.3.

3.10 Evaluation of test results, requirements, repeat test specimens, test reports

3.10.1 Designation of test results

3.10.1.1 To ensure that the description and evaluation of welding processes and positions, test results, etc. are as clear and uniform as possible, use shall be made of the terminology and symbols in the relevant standards (e.g. ISO 857, EN ISO 6947, EN 26520/ISO 6520, EN 25817/ISO 5817, EN 30042/ISO 10042) and, for internal defects, Table 4.1 in GL Welding Rules, Chapter 2 – Design, Fabrication and Inspection of Welding Joints, Section 4. The position of a defect or fracture shall be indicated and may be designated as follows:

WM = in the weld metal

FL = in the transition zone (fusion line)

HAZ = in the heat-affected zone (of the base material)

BM = in the base material

3.10.2 Requirements, repeat test specimens

3.10.2.1 The requirements are specified in Table 4.12.

3.10.2.2 If, in the tests, individual specimens fail to meet the requirements or the failure of these specimens is due to localised defects in the specimen or deficiencies in the testing equipment, it is sufficient to test two repeat test specimens or sets of repeat specimens in each case from the same test piece, which shall then meet the requirements.

3.10.2.3 In the testing of notched bar impact test specimens, unless otherwise specified in a particular case, the average value of three specimens shall apply; none of the individual values may be less than 70 % of the required value. If these conditions are not met and the average value is not less than 85 % of the required value, three repeat test specimens may be tested and the results added to the values originally obtained. The new average value from these six specimens shall then meet the requirements. If the average value of the first three specimens is less than 85 % of the required value, six repeat test specimens shall be tested, the average value of which shall meet the requirements.

3.10.2.4 If the requirements are not met by a sizeable number of specimens and/or in several areas of testing, the causes of the failures shall be investigated. When the faults have been cured, new test pieces shall be welded and fully tested.

3.10.3 Reports, storage times

3.10.3.1 Reports (cf. GL Rules II – Materials and Welding, Part 3 – Welding, Chapter 1 – General Requirements, Proof of Qualifications, Approvals, Annex B) shall be prepared of all trial welds and tests and submitted to GL in duplicate, signed by the tester and the testing supervisor.

3.10.3.2 The debris of test pieces, specimens and the test documentation are to be kept until all the tests and inspections are concluded by the confirmation of approval issued by GL. For the storage time of documents relating to the non-destructive testing of welds (e.g. radiographs), see GL Welding Rules, Chapter 2, Section 4.

3.11 Range and period of validity

3.11.1 Works and sub-works

3.11.1.1 Welding procedure approvals are generally non-transferable. GL may allow exceptions in the case of a nearby branch works where the welding work is carried out under the constant supervision of the main works, provided that the fabrication work is performed under the same conditions and the same specified welding processes are used. GL may, however, require proof as to whether the welding processes are being applied correctly and the mechanical properties are adequate by means of non-destructive tests and/or simplified production tests.

3.11.1.2 Welding procedure tests performed in a workshop are in general not simultaneously valid for welding in the field. In such cases, the welding procedure test shall be repeated in full or in part under field conditions as determined by GL. GL may dispense with repeat testing by prior agreement if the qualitative properties of the field welds are demonstrated by production tests.

3.11.2 Period of validity

3.11.2.1 A welding procedure approval is generally valid without limit of time. This is, however, always provided that the conditions under which it was granted do not change significantly.

3.11.2.2 The welding procedure approval is tied to the approval of the welding shop to perform welding work and expires when the approval of the welding shop expires. For renewal of the welding shop approval document (cf. Chapter 1 – General Requirements, Proof of Qualifications, Approvals, Section 2, A.4. of GL Welding Rules), it shall be demonstrated to GL that the approved welding processes have not been changed in the current production run and have been used without any significant defects.

3.11.2.3 For the production tests necessary in individual fields (e.g. steamboiler, pressure vessel) of application to maintain the validity of a welding procedure approval, please refer to Chapter 1, Section 2, A.3. of GL Welding Rules. GL will check the aforementioned conditions in the course of the three-yearly renewal of the welding shop approval; cf. Chapter 1, Section 2 of GL Welding Rules.

3.11.2.4 GL may revoke part or all of a welding procedure approval and require a fresh welding procedure test or fresh production tests if doubts arise as to whether a welding process is being applied correctly or safely or if defects in or damage to the welds made by this process lead to the conclusion that the quality of the welded joints is inadequate.

4. Welders qualification

4.1 Qualification test requirements

4.1.1 Manual or semi-mechanized (hand guided) welding shall be performed by qualified welders. Welders' qualification is to be tested according to GL Welding Rules or a recognized code or standard, agreed by GL. The welders' qualification tests are to be witnessed by GL. GL may accept qualification tests witnessed by another independent authority.

4.1.2 The manufacturers (welding works) are required to maintain a list, card index or the like furnishing complete information about the number, names, identification marks, welders' qualification levels and dates of the initial and repeat tests passed by the welders. Copies are to be submitted to GL, the relevant original documentation (test reports) is to be retained at the welding works, for examination by GL on demand.

4.1.3 Welding operators applying fully mechanized welding processes such as the submerged arc welding process generally need not pass a welder's performance test. They shall have been instructed and trained for work at the plant and shall mark their welds for later identification. As far as possible, the Operators shall be involved in the welding procedure qualification tests. GL may require production tests to prove their capability.

4.2 Scope of tests and limitations

4.2.1 Welders who are to perform welding work on offshore steel structures should be properly trained and are to be tested as comprehensively as possible. Specialization and limitation to tests for one particular kind of welding should be the exception for clearly defined purposes (e.g. for welding of single side root runs without backings). Table 4.14 states test groups of reasonable types and positions and the relevant scopes of testing. Any deviations there from are to be agreed with GL in each individual case.

4.2.2 Welders' qualification tests are to be performed with the respective welding process for those kinds of materials (plates, tubes), joint types (butts, fillets, T-, K- and Y-connections) and welding positions (see Annex A) which will be employed during later manufacture. The details and further data are to comply with those stated in the welding procedure specifications. Regarding test types and position limitations Table 4.13 shall apply analogously. GL may specify other types and limitations if the welder has to be specially skilled for performing the welding work.

4.2.3 Plate thickness, tube diameters and wall thicknesses of the test pieces shall be in accordance with Table 4.15

4.2.4 Additional to the limitations mentioned before, for the following variations it is considered essential that they shall require requalification by a new or additional performance test:

– Material: Change from normal strength (mild) steel to higher strength steel, or change from higher strength steel to high strength steel

– Joint type: Change from double side welding to single side welding

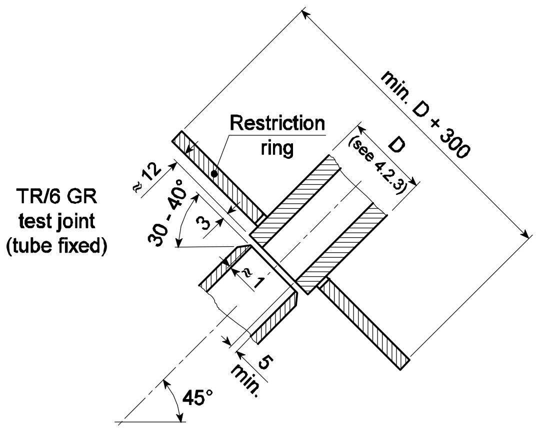
– Welding process: Any change

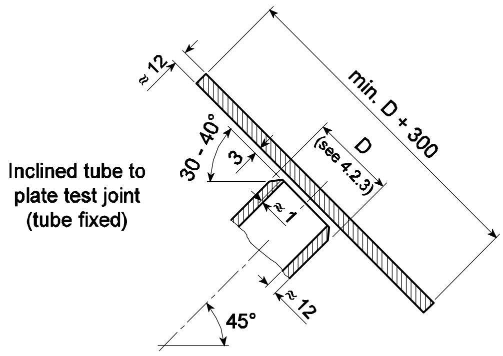
– Welding position and progression: Any change outside the limitations given in Table 4.13

Table 4.14 Welders qualification test and groups; scope of testing

Test group (Type test)	Test joints and positions [1]	Visual inspection	Type and number of tests		
			NDT	Bend tests[2]	Macro etchings
PB (plate, butt joint) [3, 4]	1 plate butt weld in pos. H-V/2G 1 plate butt weld in pos. V/3G 1 plate butt weld in pos. O/4G	×	≤ 30 mm: X-ray	≤ 20 mm: 1 root + 1 face of each position	1 of each position
TB (tube, butt joint) [3, 4]	1 tube butt weld in pos. H-V/2G 1 tube butt weld in pos. TH/5G or 1 tube butt weld in pos. TI/6G	×	> 30 mm: UT	> 20 mm: 2 side of each position	
TC (tubular T-, Y-, or K-connection) [3, 4]	1 tube butt weld in pos. TR/6GR or 1 tubular T-joint (inclined tube to plate) or equivalent [5]	×	X-ray for TR/6GR; UT for T-joint	–	1 of each position (min. 3)

1 For the welding positions of butt and fillet joints see Table 4.7, Footnote [1] and Annex II, for tubular connections see sketches below.





2 The root and face bend tests should be performed in the form of notched bend tests according to ISO 9606-1 or nick break tests to force the break in the weld metal.

3 All butt and tubular welds are normally to be performed in the single side technique without backing. Where the tests are performed as a double side welding, this shall be denoted by the letter „X" appended to the test group designation and mentioned in the test Certificate.

4 Test group TC covers TB and PB, test group TB covers PB. Test group PB qualifies also for butt welds in tubes with outside diameters of more than 600 mm.

5 If the included angle between the axes of the tubes is 30° or less and the weld is to be performed from single side, GL may require relevant tubular node joints or similar tube to plate joints with full penetration groove welds.

Table 4.15 Test piece dimensions and range of applicability

	Test piece dimensions	Range of applicability
Plate/wall thickness "t"	up to 20 mm	0,5 t – 2 t
	more than 20 mm	all thicknesses greater than 10 mm
Tube diameter "D"	up to 600 mm	tested diameter – 100 / + 200 mm
	more than 600 mm	all diameters greater than 500 mm

4.3 Qualification of underwater welders

4.3.1 For underwater welding only those welders may be qualified, who are well trained, experienced and qualified for relevant welding above water. Prior to the qualification tests under water, the welders are to be given sufficient training to get familiar with the underwater welding conditions such as pressure, atmosphere, temperatures, etc.

4.3.2 The test welds are to be performed using materials comparable in chemical composition (weldability) and strength to those used in the actual welding job, and with the welding process in question under actual or simulated diving conditions.

4.3.3 Type and scope of testings are to be based on the actual welding job analogously to those given above and will be decided from case to case. Limitations given above apply analogously, the limitation of applicable (greater) water depth for welding will be decided in each single case.

4.4 Validity of qualification Certificates

4.4.1 The validity of a welder's qualification Certificate is two years, provided that during this period the welder has been working under the supervision of GL and the welder's work is monitored by visual and/or non-destructive testing. GL may demand annual repeat tests if monitoring mainly takes the form of visual inspections or if the results of NDT cannot be correlated to the welder.

4.4.2 A repeat test is required where a welder who has been tested in more than one welding process has not used the process in question for a period exceeding six months but has meanwhile used another process. A repeat test is also required if a welder has not performed any welding work as defined in 4.1.1 for a period exceeding three months.

GL may demand a repeat test at any time if reasonable doubts should arise as to a welder's performance.

5. Design of weld connections

5.1 General requirements

5.1.1 The drawings, specifications, etc. shall provide clear and complete information regarding location, type, size and extent of all welds. The drawings shall also clearly distinguish between shop and field welds. Symbols and signs used to specify welding joints shall be in accordance with recognized standards or codes (to be mentioned in the drawings, specifications, etc.) or shall be explained in these documents. Special weld (groove) details shall be shown by sketches and/or relevant remarks.

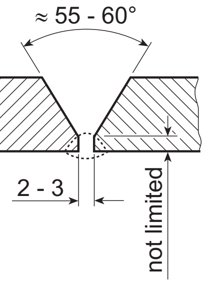
5.1.2 It is recommended that standard welding details, symbols, etc., including any workmanship or inspection requirements, are to be laid down in standard drawings submitted to GL for general approval. Dimensions of welding should not form a part of those standard drawings, but shall be designated in the contract or shop drawings for each individual structural part or welding respectively.

5.1.3 All welds shall be planned and designed in such a way that they are readily accessible during fabrication and can be executed in an adequate welding position and welding sequence, see also 6.5. Welded joints which are subject to NDT-inspection shall be placed and designed to facilitate application of the required inspection procedure, so that tests offering reliable results may be carried out.

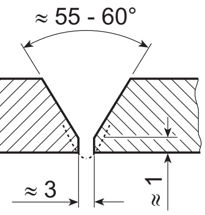
5.2 Weld shapes and dimensions

5.2.1 Weld shapes (groove configurations) shall be in accordance with GL Welding Rules and other recognized standards and codes, such as EN 12345/ISO**,** EN 22553/ISO 2553 or EN 29692/ISO 9692 or ANSI/AWS Structural Welding Code (Steel) D 1.1. As far as possible, full penetration butt or T-joints shall be designed as double side weldings with the possibility of sufficient root treatment (gouging) instead of single side weldings. Fillet welds shall be made on both sides of the abutting plate or web. Typical single side bevelled groove weld shapes are shown in Fig. 4.13.

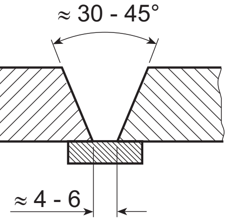
5.2.2 Weld shapes shall be designed to ensure that the proposed weld type and quality (e.g. full penetration) can be satisfactorily achieved under the given fabrication conditions. Failing this, provisions shall be made for welds which are easier to execute. The (possibly lower) load-bearing capacity of these welds shall be taken into consideration in the dimensional design. According to Fig. 4.18, the effective throat thickness of a partial penetration groove weld shall be the depth of bevel less 3 mm, if not otherwise proved in way of welding procedure qualification tests and agreed by GL.



Groove weld with back welding,
root gouged (double side welding)



Groove weld without back welding,
root gouged (double side welding)



Groove weld with backing flat
bar, for minor purposes only

Fig. 4.13 Typical single side bevelled groove weld shapes

5.2.3 Special weld shapes, differing from those laid down in the above rules and standards (e.g. single side, full penetration groove welds in tubular T-, Y- or K-connections) or weld shapes for special welding processes shall have been proved in connection with the welding procedure qualification tests according to 3.3 and 3.5 and are to be approved by GL. Typical welds of tubular T-, Y- and K-connections with and without back welding are shown in Fig. 4.14 and 4.15.

5.2.4 The weld contour is to be designed with smooth transitions tangent to the parent material in order to achieve the actual calculated fatigue life. The requirements regarding the weld contour, including additional treatment, such as grinding of weld toes to a smooth profile depending on the actual detail category (see Section 3, H.), are to be laid down in the drawings and/or specifications. A typical improved weld contour of a T-joint is shown in Fig. 4.16. The cap layer should be made with single passes (no excessive weave technique), starting at base material and making the last passes in temper bead technique on top of the weld to avoid excessive hardness in the heat affected zone.

5.2.5 Dimensions (effective throat thicknesses and lengths) of welds are to be determined in context with the static and fatigue strength calculations and are to be specified in the drawings submitted to GL for approval.

The effective throat thickness of a complete penetration groove weld shall be the thickness of the thinner part to be joined. For partial penetration groove welds, the drawings shall specify the groove depth applicable to the effective throat thickness required for the welding procedure to be used (see 5.2.2 and Fig. 4.18), as well as the calculated effective throat thickness.

5.2.6 The throat thickness of fillet welds is the size "a" in Fig. 4.17. In the drawings, fillet weld dimensions may be specified by the size "a" or by the leg length "I" which is a · 2 .

The throat thickness of fillet welds shall not exceed 0,7 times the lesser thickness of the parts to be welded (generally the web thickness). The minimum throat thickness is defined by the expression:

$$a_{min} = \sqrt{\frac{t_1 + t_2}{3}} \text{ [mm], but not less than 3 mm}$$

t_1 = lesser (e.g. the web) plate thickness [mm]

t_2 = greater (e.g. the flange) plate thickness [mm]

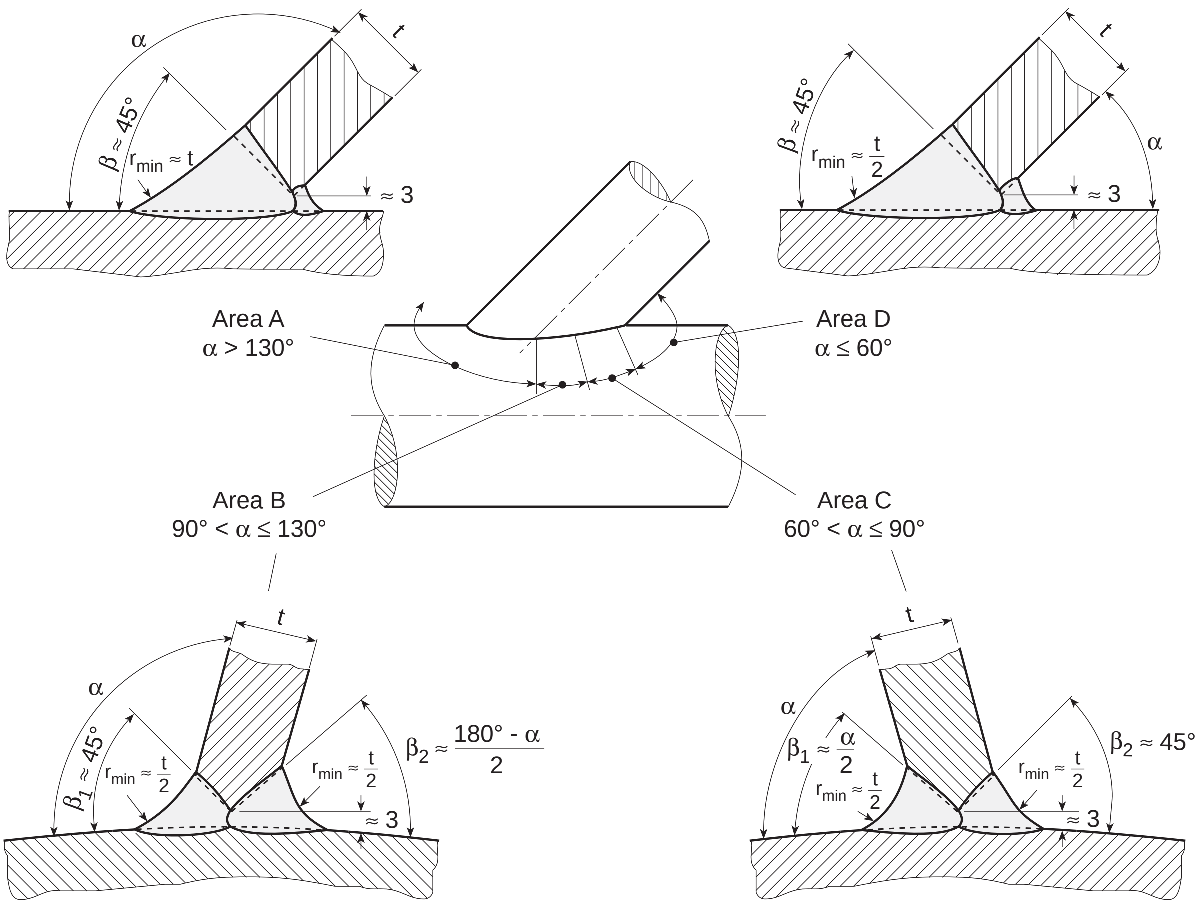


Fig. 4.14 Typical tubular weld connections with back welding

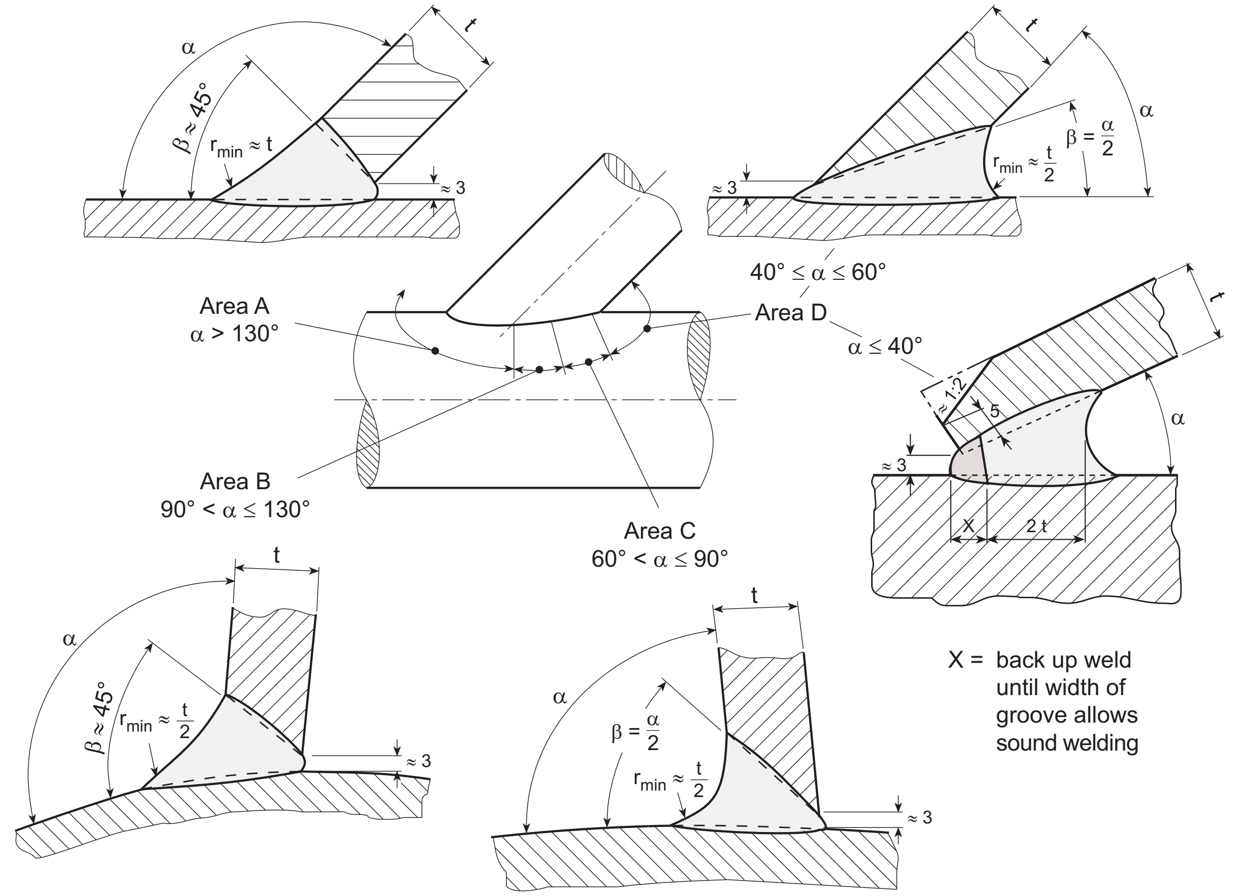


Fig. 4.15 Typical tubular weld connections without back welding

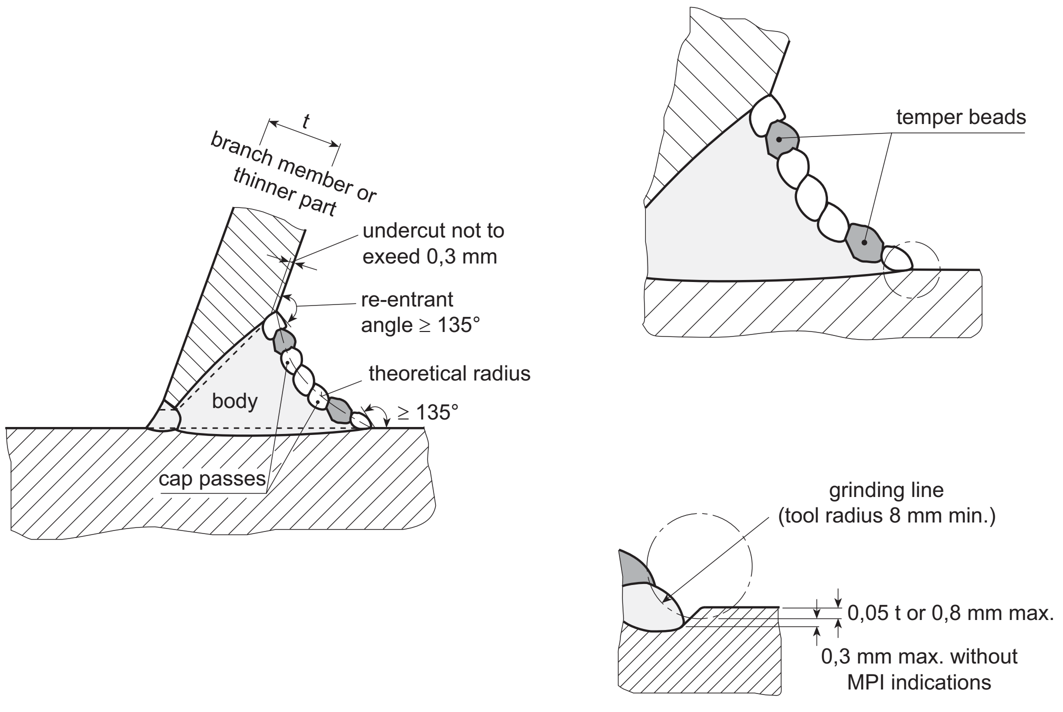


Fig. 4.16 Typical improved weld contour

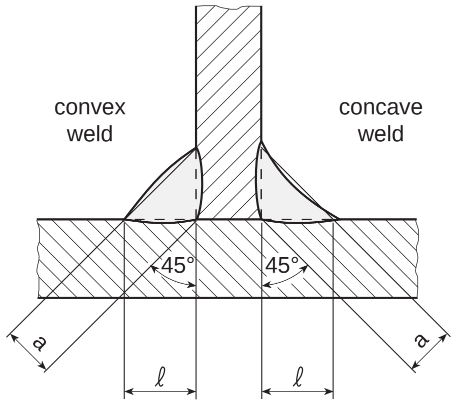


Fig. 4.17 Effective throat thickness “a” and leg length “l” of fillet welds

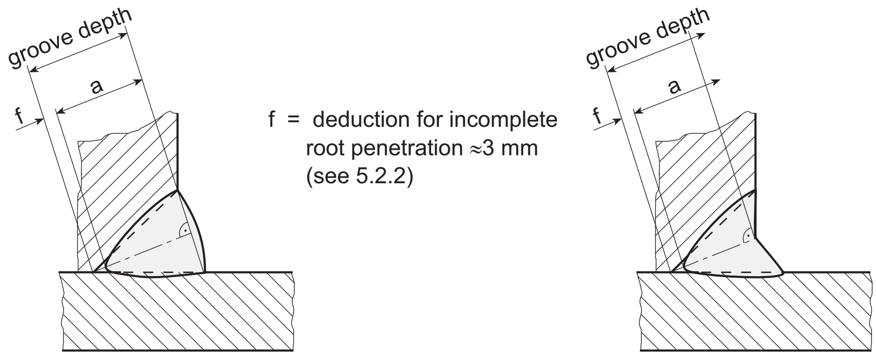


Fig. 4.18 Effective throat thickness “a” of partial penetration groove welds

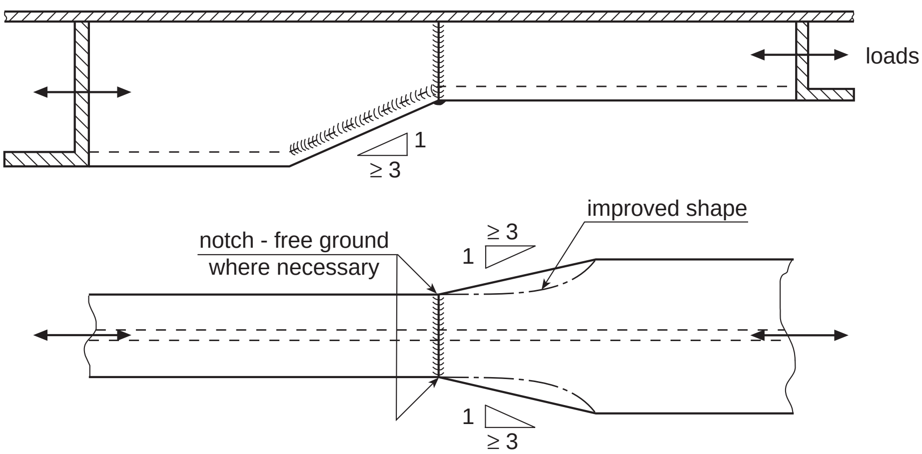


Fig. 4.19 Transitions between components with differing dimensions

5.3 Details of welded joints

5.3.1 All welded joints on primary members shall be designed to provide a stress flow as smooth as possible without major internal or external notches, discontinuities in rigidity and obstructions to strains.

The transition between differing component dimensions e.g. of girders or stiffeners shall be smooth and gradual. The length of the transition should be at least 3 times the difference in depth, see Fig. 4.19.

5.3.2 The requirements given in 5.3.1 applies in analogous manner to the welding of secondary members onto primary (or special) supporting members. The ends should have smooth transitions to the main structure, see Fig. 4.20.

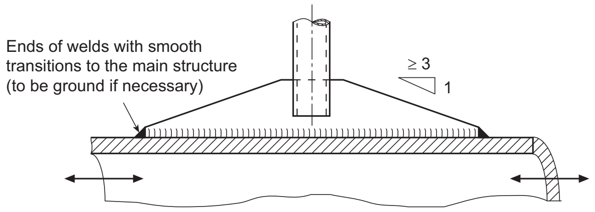


Fig. 4.20 Ends of secondary members welded on to the main structures

5.3.3 Where the plate or wall thickness differ more than 3 mm at joints mainly stressed perpendicularly to the (butt) weld direction, the difference shall be smoothened by bevelling and/or buttering according to Fig. 4.21 prior to butt welding. Differences in surface levels up to 3 mm may be equalized within the weld.

5.3.4 If a misalignment due to component or plate respective wall thickness tolerances cannot be eliminated by adjusting (e.g. in case of circumferential butt welds in tubes) and the difference "X" is not greater than one quarter of the thinner plate or wall thickness, the misalignment may be equalized by chamfering or built up welding according to Fig. 4.22. Where the difference is greater and/or highly fatigue stressed joints are involved, equalization is to be decided and agreed upon with GL in each individual case.

5.3.5 Where platings (including girder web and flange plates) or tube walls are subjected locally to increased stresses, thicker plates are to be inserted in preference to doubler platings. This applies analogously to flanges, stuffing-boxes, hubs, bearing bushes or similar components to be welded into platings. The minimum size of those reinforcement plates shall be

$$D_{min} = 170 + 3\,(t - 10), \text{ but not less than } 170 \text{ mm}$$

D = the diameter of round inserts or the length/breadth of angular inserts [mm]

t = plating thickness [mm]

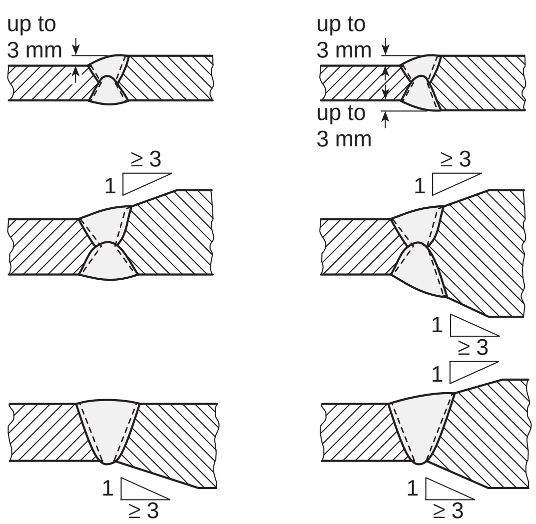


Fig. 4.21 Transition in butt welds of unequal thickness

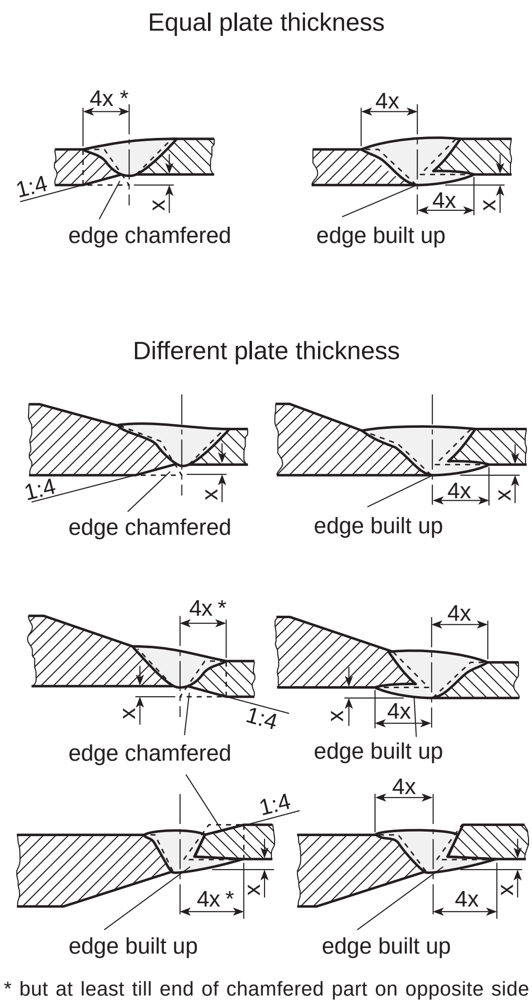


Fig. 4.22 Equalization of misalignment in butt welds

The corner radii of angular reinforcing plates shall be at least 50 mm. The weldings between those reinforcement plates or inserts and the plating shall be full penetration butt welds, and the transitions shall be in accordance with 5.3.3.

5.3.6 Doubler plates (cover plates) may be used to reinforce girder flanges or similar components in the area of maximum bending moments. Doubler plates shall be limited to one per flange, and their thickness "t" shall not exceed 1,5 times the thickness of the flange, their width "w" should not exceed 20 times their thickness. The ends of doubler plates shall be extended beyond the calculated, theoretical start or end conforming to Fig. 4.23. The fillet welds connecting the doubler plate to the flange shall be continuous welds. The welds at the end of the doubler plates shall conform to Fig. 4.23.

5.3.7 Local clustering of welds and short distances between welds are to be avoided. Adjacent butt welds should be separated from each other by a distance of at least

$$50 \text{ mm} + 4\,t$$

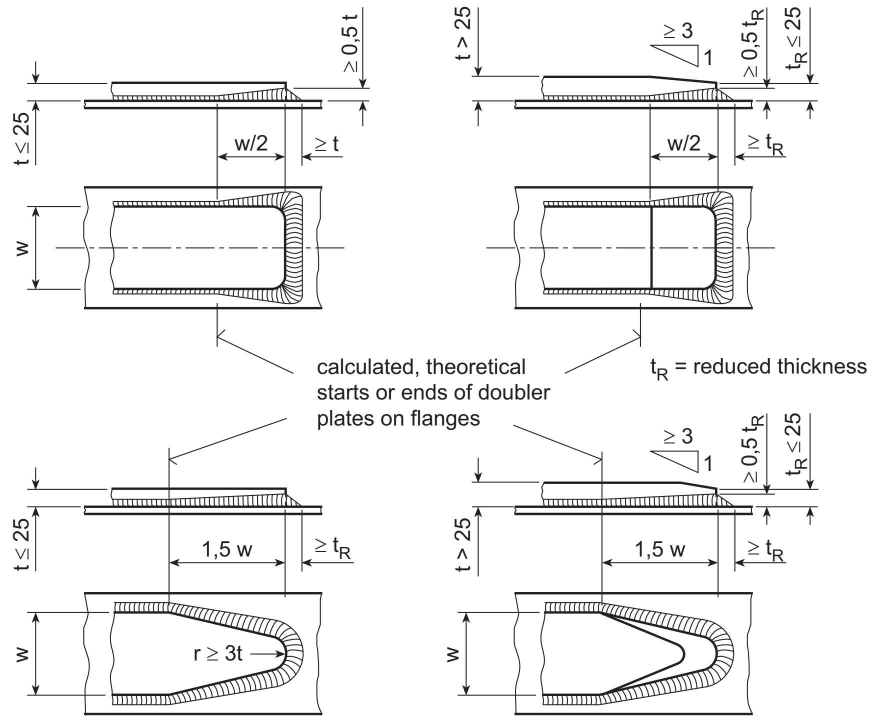


Fig. 4.23 Configuration and welding of the ends of doubler plates on flanges

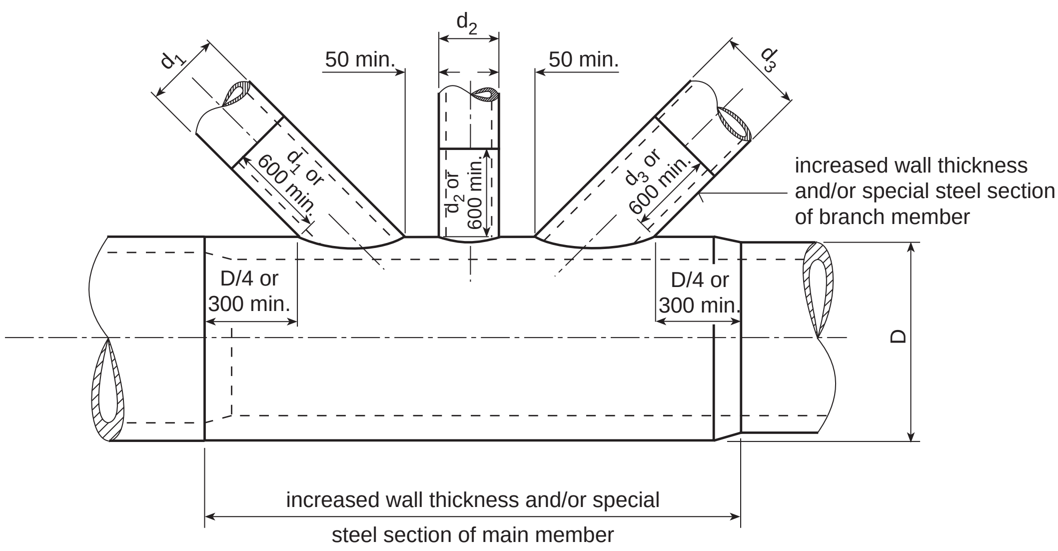


Fig. 4.24 Separation of welds in tubular joints

where t = plate thickness. Fillet welds should be separated from each other and from butt welds by a distance of at least

30 mm + 2 t.

The width of sections (strips) of plate to be replaced should, however, be at least 300 mm or 10 t, whichever is greater.

5.3.8 Where tubular joints (node sections) without overlapping requires increased wall thicknesses and/or special steel in the main member (chord, can) and/or in the branch members (bracing stubs), the separation of the welds shall be in accordance with Fig. 4.24. The greater measures apply, see also B.15.7.

5.3.9 In overlapping tubular joints (node sections), in which part of the load is transferred directly from one branch member (bracing) to another through their common weld, the overlapping of the welds shall be at least in accordance with Fig. 4.25. The greater measure applies. The actual length of the overlapping weld is to be determined by calculation. The heavier brace shall preferably be the through brace, with its full circumference welded to the chord.

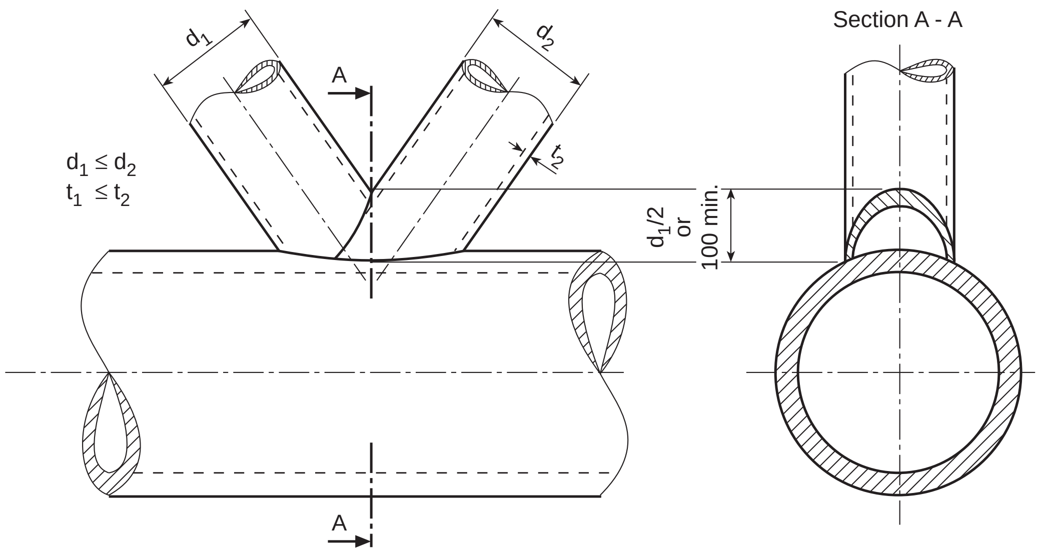


Fig. 4.25 Separation of welds in overlapping tubular joints

5.4 Calculation of welded joints

5.4.1 Any calculation relating to welded joints which is stipulated in these rules or prescribed as an alternative to the rules governing dimensions, shall be performed in accordance with the GL Rules I – Ship Technology, Part 1 –Seagoing Ships, Chapter 1 – Hull Structures, Section 19, C. and Section 20. Calculations conforming to other rules or standards are subject to the prior consent of GL.

5.4.2 Proof by calculation of adequate dimensioning (a general stress analysis) is required where, with mainly static loading, the thickness of butt welds are not equivalent to the plate thickness, and in the case of throat thicknesses of fillet welds.

5.4.3 For welded joints subjected to mainly dynamic loads, the permissible loading shall be determined by reference to the range of stress variations, the global loading conditions, the limit stress ratio and the notch category (proof of fatigue strength). The notch category is a function of the geometrical configuration of the welded joint. It also depends on the proof of the absence of serious internal notches (welding defects).

5.4.4 Tubular joints

Tubular joints shall be designed using proven procedures, e.g. as shown in API-RP 2A (Section 3-5). See also Section 3, E.

For non-standard or complex joints, appropriate analytical and/or experimental investigations may be required.

For all tubular joints which are subject to cyclic loading, a fatigue analysis shall be performed, see Section 3, H.

5.5 Welding in cold formed (bent) areas, bending radii

5.5.1 Wherever possible, welding should be avoided at the cold formed sections with more than 3 % permanent elongation [1] and in the adjacent areas where structural steels with a tendency towards strain ageing are used.

5.5.2 Welding may be performed at the cold formed sections and adjacent areas of hull structural steels and comparable structural steels provided that the minimum bending radii are not less than those specified in Table 4.16.

Table 4.16 Minimum bending radii

Plate thickness t	Minimum inner bending radius r
4 mm or less	1 × plate thickness
8 mm or less	1,5 × plate thickness
12 mm or less	2 × plate thickness
24 mm or less	3 × plate thickness
over 24 mm	5 × plate thickness
Note *The bending capacity of the material may necessitate a larger bending radius.*	

1 Elongation ε in the outer tensile-stressed zone:
$100 / (1 + 2r/t)$ [%]
r = inner bending radius [mm]
t = plate thickness [mm]

5.5.3 For other steels and other materials, where applicable, the necessary minimum bending radius shall, in case of doubt, be established by test. Proof of adequate toughness after welding may be stipulated for steels with minimum yield strengths of more than 355 N/mm^2 and plate thicknesses of 30 mm and above which have undergone cold forming resulting in 2 % or more permanent elongation.

6. Performance of welding, workmanship

6.1 General requirements

6.1.1 All welding is to be performed observing the applicable paragraphs of this Section as well as the common actual standard of welding technology to achieve the required quality. Compliance with these rules and with any other conditions, which may be agreed or stated in the drawings and specifications or imposed in connection with approvals as well as with good workmanship, is the responsibility of the manufacturers. Inspections performed by GL do not relieve the manufacturers of this responsibility.

6.1.2 The preparation and performance of all welding operations shall be supervised by manufacturer's welding supervisors (see 1.5 above). Regarding the qualification and the functions of the welding supervising personnel see GL Welding Rules. In the event of any deviations from the above stated operating conditions, the welding supervisory personnel shall, in agreement with GL, take steps to ensure the constant and adequate quality of the welds. For inspection personnel see D.5.

6.2 Overweldable shop primers

6.2.1 Overweldable shop primers which are applied to plates, sections, etc. prior to welding and are not removed shall be tested and approved in accordance with the GL Welding Rules, Chapter 1 – General Requirements, Proof of Qualifications, Approvals, Section 6.

6.2.2 By suitable checks- on the thickness of the coating especially - and by sample production tests in the course of normal fabrication welding, shops shall ensure that the welded joints suffer no unacceptable deterioration (cf. GL Welding Rules, Chapter 1 – General Requirements, Proof of Qualifications, Approvals, Section 6, C.)

6.3 Weld preparation, assembly, tack welding

6.3.1 Machining, oxygen cutting, air carbon arc gouging, oxygen gouging, chipping or grinding may be used for joint preparation, back gouging or the removal of unacceptable work or metal, except that oxygen gouging shall not be used on normalized or quenched and tempered steels.

6.3.2 When preparing and assembling components, care shall be taken to ensure compliance with the weld shapes and root openings (air gaps) specified in the drawing and specifications. With single and double-bevel T joints, especially, attention shall be paid to an adequate root opening in order to achieve sufficient root penetration.

6.3.3 The root opening shall not exceed twice the specified gap. If the gap exceeds this value locally over a limited area, the gap may be reduced by build-up welding of the groove edges, subject to the prior consent of the Surveyor. With fillet welds, the "a" dimension shall be increased accordingly, or a single or double-bevel weld shall be made if the air gap is large. Inserts and wires may not be used as fillers.

6.3.4 With the Surveyor's agreement, large gaps - except in highly stressed areas of special structural members - may be closed by means of a strip of plate with a width of at least 10 times the plate thickness or 300 mm, whichever is the greater.

6.3.5 Members to be welded shall be brought into correct alignment (see also B.10.) and held in position by bolts, clamps, wedges, guy lines, struts, and other suitable devices, or by tack welds until welding has been completed. The use of jigs and fixtures is recommended where practicable. Suitable allowances shall be made for warpage, shrinkage and for expansion due to ambient changes, e.g. direct sunlight.

6.3.6 Clamping plates, temporary ties, aligning pins, etc. shall be made of structural steel of good weldability and should not be used more than necessary. They are to be carefully removed to prevent damage to the surface of members when the components have been permanently welded.

6.3.7 Clamping plates, temporary ties, aligning pins, etc. shall not be welded to components or areas subject to particularly high stresses, nor shall they be welded to the edges of flange plates of girders or similar members. The same applies to the welding of handling lugs and other auxiliary fixtures.

6.3.8 Tack welds should be used as sparingly as possible and should be made by trained operators using qualified welding procedures and consumables. Where their quality does not meet the requirements applicable to the subsequent welded joint, they are to be carefully removed before the permanent weld is made. All tack welds which will form part of the permanent weld shall be cleaned, ground down to a feather edge at both ends and visually inspected prior to welding of the root pass.

6.3.9 Particularly with mechanized welding processes - and invariably when end craters and defects at the start and end of the weld have to be avoided – run-in and run-off plates of adequate section shall be attached to components and cleanly removed on completion of the weld.

6.3.10 Surfaces to be welded shall be clean and dry. Any scale, rust, cutting slag, grease, paint or dirt is to be carefully removed before welding (for over-weldable shop primers, see 6.2). Moisture is to be removed by preheating. Edges (groove and root faces) are to have a smooth and uniform surface.

6.4 Weather protection, preheating, heat input during welding

6.4.1 Weather protection, welding at low temperatures

6.4.1.1 The area in which welding work is performed is to be sheltered from wind, damp and cold, particularly if out of doors. Where gas-shielded arc welding is carried out, special attention is to be paid to ensuring adequate protection against draughts. When working in the open under unfavourable weather conditions it is advisable to dry welding edges by heating.

6.4.1.2 At ambient temperatures below +5 °C, additional measures shall be taken, such as shielding of components, extensive preliminary heating and preheating, especially when welding with a relatively low heat input (energy input per unit length of weld), e.g. when laying down thin fillet welds or in the case of rapid heat dissipation, e.g. when welding thick-walled components. Wherever possible, no welding should be performed at ambient temperatures below –10 °C.

6.4.2 Preheating for the welding of ferritic steels

6.4.2.1 The need for preheating of ferritic steels and the preheating temperature depend on a number of factors. Among these are especially important:

- the chemical composition of the base material (carbon equivalent) and the weld metal
- the thickness of the workpiece and the type of weld joint (two or three dimensional heat flow)
- the welding process and the welding parameters (energy input per unit length of weld)
- the shrinkage and transformation stresses
- the temperature dependence of the mechanical properties of the weld metal and the heat affected zone
- the diffusible hydrogen content of the weld metal

6.4.2.2 The operating temperature to be maintained (minimum preheating temperature and maximum interpass temperature) for (hull) structural steels may be determined in accordance with EN 1011-2. Guide values for the preheating temperature are contained in Figures 4.26 and 4.27 shown below for two different energy inputs per unit length of weld [2] and hydrogen contents HD [3] of the weld metal, together with the various carbon equivalents CET [4].

Note

Table 4.17 gives guide values for the carbon equivalents CET [4] of some of the standard grades of steel. Basis were the information of the steel manufacturers. In case of doubt CET has to calculate by the actual analysis.

6.4.2.3 Table 4.18 contains guide values for preheating high temperature Mo or CrMo alloy steels (used for steam boilers) in accordance with the GL Rules II – Materials and Welding; cf EN 1011-2.

6.4.2.4 Table 4.19 contains guide values for preheating nickel steels tough at sub-zero temperatures in accordance with the GL Rules II – Materials and Welding. For details of this and also particulars relating to the use of austenitic or nickel-based welding consumables, cf. EN 1011-2.

Table 4.17 Guide values for the carbon equivalent CET

Steel grades	CET [% in weight]	
	Average value [1]	Maximum value [1]
GL–A	0,27	0,28
GL–E	0,26	0,27
GL–D36	0,33	0,34
GL–E36TM	0,27	0,28
GL–D40	0,27	0,28
GL–E40TM	0,24	0,25
S275NL	0,25	0,27
S460NL	0,34	0,36
S460ML (TM)	0,27	0,28
S690QL	0,26	0,38
S890QL	0,38	0,41
2C22	0,26	0,29
34CrMo4	0,49	0,55
GS20Mn5	0,34	0,41

[1] For product thicknesses up to 50 mm.

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[2] Energy input per unit length of weld:

$$E = \frac{U[\text{volts}] \cdot I[\text{amps}] \cdot \text{welding time}[\text{min}] \cdot 6}{\text{length of weld}[\text{mm}] \cdot 100} \left[\frac{\text{kJ}}{\text{mm}}\right]$$

[3] HD 5 = max. 5 ml diffusible hydrogen per 100 g of weld metal

HD 15 = max. 15 ml diffusible hydrogen per 100 g of weld metal

[4] Carbon equivalent:

$$CET = C + \frac{Mn + Mo}{10} + \frac{Cr + Cu}{20} + \frac{Ni}{40} \quad [\%\ \text{in weight}]$$

The above formula for calculating the carbon equivalent CET in accordance with EN 1011-2 can be applied to steels which have yield strengths ranging from 300 to 1000 MPa and to the following chemical composition: 0.05 – 0,32 % C, max. 0,8 % Si, 0,5 – 1,9 % Mn, max. 0,75 % Mo, max. 1,5 % Cr, max. 0,7 % Cu, max. 2,5 % Ni, max. 0,12 % Ti, max. 0,18 % V, max. 0,005 % B, max. 0,06 % Nb.

Table 4.18 Guide values for preheating high-temperature steels (used for steam boilers)

Category in accordance with DIN V 1738 (CR 12187)	Steel grade	Thickness [mm]	Minimum preheating temperature [°C] given an H_2 content of the weld metal of		
			≤ 5 ml/100 g	> 5 – ≤ 10 ml/100 g	> 15 ml/100 g
1.2	16Mo3	≤ 15 > 15 – ≤ 30 > 30	20 20 75	20 75 100	100 100 not permitted
5.1	13CrMo4-5	≤ 15 > 15	20 100	100 150	150 not permitted
5.2	10CrMo9-10 11CrMo9-10	≤ 15 > 15	75 100	150 200	200 not permitted

Table 4.19 Guide values for preheating nickel steels tough at sub-zero temperatures

Category in accordance with DIN V 1738 (CR 12187)	Steel grade	Thickness [mm]	Minimum preheating temperature [°C] given an H_2 content of the weld metal of	
			≤ 5 ml/100 g	> 5 – ≤ 10 ml/100 g
7.2	12Ni14 (3,5 % Ni)	> 10	100	150
7.3	12Ni19 (5 % Ni)	> 10	100	not permitted
	X8Ni9 (9 % Ni)	> 10	100	not permitted
	X7Ni9 (9 % Ni)	> 10	100	not permitted

Table 4.20 Effect of the various factors on the level of preheating

Shift in the preheating temperature to lower values	Factors influencing preheating	Shift in the preheating temperature to higher values
low alloying element content	chemical composition of the base material (hardenability), e.g. expressed by the carbon equivalent	higher alloying element content
thin	thickness of the workpiece or component (heat dissipation, rigidity, residual stress condition)	thick
butt joints (2 planes), thick (multiple run) welds	type of joint, weld shape and dimensions, heat input, heat dissipation	T-joints (3 planes) thin (single-run) welds
high	ambient or workpiece temperature (heat dissipation)	low
high	heat input (energy input per unit length of weld) during welding	low
low	hydrogen content of the weld metal (type and rebaking of the welding consumables and auxiliary materials)	high

6.4.2.5 Depending on the complexity of the component, the welding process applied, the level of the residual stresses in the component and the (low) ambient temperature, the preheating temperatures shall be increased or the boundary wall thicknesses reduced as appropriate. For the effect of the various factors on the preheating temperature level, see Table 4.20.

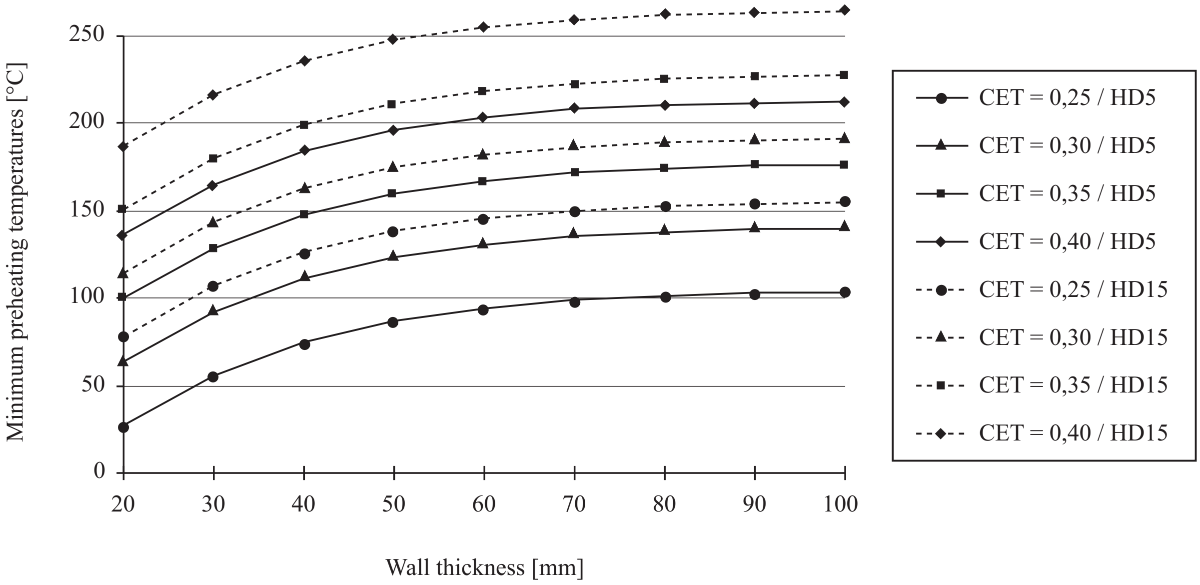

Fig. 4.26 Minimum preheating temperatures (operating temperatures) applicable to welding processes with a relatively low heat input (energy input per unit length [1] E ≈ 0,5 kJ/mm) as a function of the carbon equivalent CET [3] of the base material and the hydrogen content of the weld metal

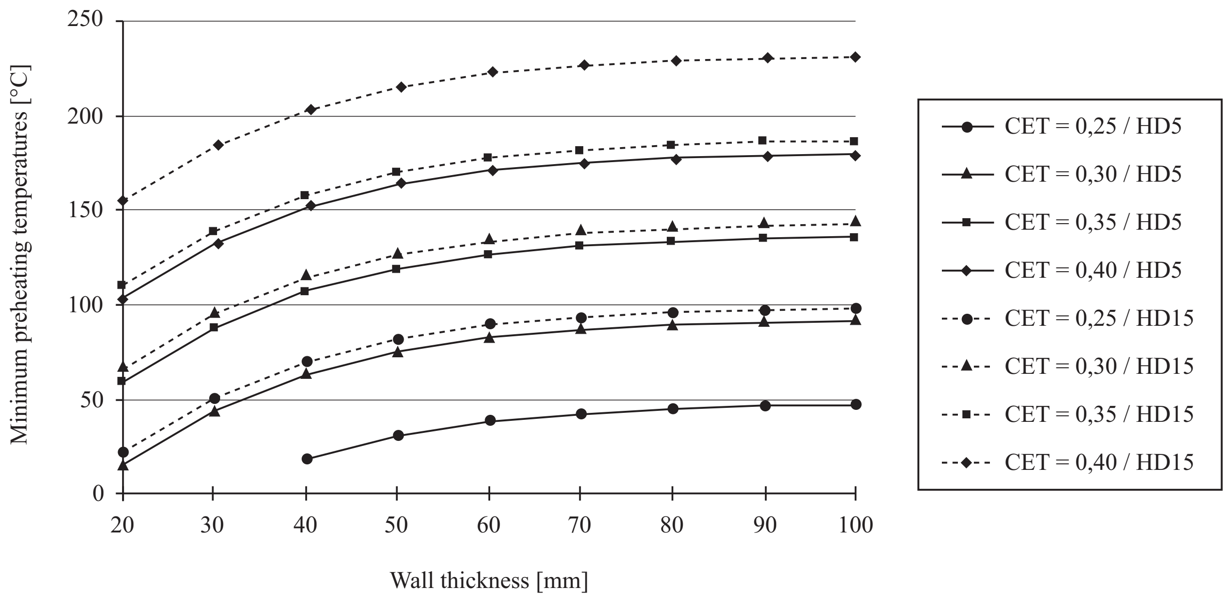

Fig. 4.27 Minimum preheating temperatures (operating temperatures) applicable to welding processes with a relatively high heat input (energy input per unit length [1] E ≈ 3,5 kJ/mm) as a function of the carbon equivalent CET [3] of the base material and the hydrogen content of the weld metal

6.4.2.6 If the temperature of the workpiece is lower than the minimum operating temperature calculated on the basis of the above data, preheating is called for. Various methods are available:

– continuous heating prior to and during welding

– alternate heating and welding

– heating only prior to the start of welding, if the heat input during welding is sufficient to maintain the minimum operating temperature.

The heating method may be chosen at will, provided that it does not harm the material by localized overheating or cause a nuisance by making the welding area contaminated.

6.4.2.7 Preheating is always necessary for tack and auxiliary welds whenever preheating is needed for the rest of the welding. Possible exceptions to this rule are tack and auxiliary welds where it can be guaranteed that subsequent welds are remelted the heat affected zone, for instance tacks for submerged arc welds.

6.4.2.8 Irrespective of the information given above, preheating is always necessary when making major auxiliary erection welds, e.g. when welding on handling lugs and when welding very large wall thicknesses and also thick-walled castings and forgings.

6.4.2.9 Preheating shall be applied uniformly throughout the thickness of the plate or component over a distance of four times the plate thickness, minimum of 100 mm, on both sides of the weld. Localized overheating is to be avoided. Preheating with gas burners should be performed with a gentle, though not sooty, flame in order to prevent dirt being deposited in the area of the weld. For details on the recording of the preheating temperature, cf. EN IS0 13916.

6.4.2.10 To prevent cold cracks in higher-strength and high-strength (quenched and tempered) steels, thick-walled components or components of complex design, it is advisable to use measures which give the hydrogen introduced into the weld metal during welding sufficient time to escape. The following methods are well established:

– maintenance of a specific minimum preheating and interpass temperature throughout the welding operation

– delayed cooling after welding

– holding at approx. 250 °C prior to cooling (hydrogen-reducing heat treatment) or

– heat treatment immediately after welding (without cooling in between)

6.4.2.11 Where structural steels or fine-grained structural steels have undergone thermo-mechanical processing (TM steels), the need for and degree of preheating shall be decided on separately on the basis of the carbon equivalent and the results of the approval or welding procedure tests as applicable. Drying of the areas to be welded by heating may be sufficient.

6.4.3 Monitoring interpass temperatures

The guide values contained in Table 4.21 for the interpass temperatures relating to the various steels shall not be significantly exceeded.

6.4.4 Welding with controlled heat input per unit length of weld

In addition to controlling the preheating and interpass temperature, the heat input per unit length of weld shall be controlled during welding, especially in the case of weldable, high-strength (quenched and tempered) fine-grained structural steels. The heat input per unit length of weld shall not fall below or exceed the values indicated by the steel manufacturer or those used in the welding procedure tests and specified in the welding procedure specifications (WPS) by any significant amount.

6.4.5 Preheating and heat input during the welding of other steels or metallic materials

6.4.5.1 Preheating is not normally required for austenitic materials. Preheating may be necessary for austenitic-ferritic materials. A maximum permitted interpass temperature which is normally between 150 °C and 180 °C shall be complied with in order to prevent hot cracks.

6.4.5.2 Ferritic and stainless martensitic steels shall be adequately preheated and welded using controlled heat input per unit length of weld. Guide values for the preheating and interpass temperatures are prescribed in EN 1011-3.

Table 4.21 Guide values for the maximum interpass temperature during welding

Category in accordance with DIN V 1738 (CR 12187)	Steel grades	Maximum interpass temperature [°C]
1.1	Normal-strength hull structural steels and comparable structural steels	250
1.2	Higher-strength structural steels and comparable structural steels	250
1.2	High-temperature, low Mo alloy steels	250
2	Normalised or thermo-mechanically processed fine-grained steels with yield strengths of > 360 N/mm²	250
3	Quenched and tempered or precipitation-hardened (excluding stainless) steels with yield strengths of > 360 N/mm²	250
5	Steels with a max. Cr content of 10 % and a max. Mo content of 1,2 %	350
7	Nickel alloy steels with a max. Ni content of 10 %	250

6.4.5.3 Preheating is not normally required for welding aluminium alloys, but should not exceed 50 °C. A maximum permitted interpass temperature of 100 °C to 120 °C shall be complied with in order to prevent undesirable phase dispersion. EN 1011-4 contains guide values for the preheating temperature to be applied and the interpass temperature.

6.5 Welding positions, welding sequence

6.5.1 Welding should be performed in the optimum welding position, and welding in unfavourable positions is to be limited to the indispensable minimum. For similar and repetitive welding operations it is advisable to use a (rotary) jig enabling all welds, as far as practicable, to be made in the downhand position. Vertical-downward welding may not be used to join special and primary structural members.

6.5.2 The welding sequence shall be chosen to allow shrinkage to take place as freely as possible and to minimize shrinking stresses. In special cases, GL may require to set down the assembly procedure and welding sequence in an appropriate schedule.

6.6 Weld quality requirements

6.6.1 The requirements to be met by the weld, e.g. seam geometry, surface quality, admissible external and internal flaws, depend on the significance of the respective welds for the overall integrity of the structure. The operability of individual components as well as the kind of loading (static-dynamic) and the stress level shall also be considered.

6.6.2 The requirements to be met by the welds are to be laid down in the plans for production, inspection and testing, e.g. drawings, specifications, etc. (see also D.6.4) on the basis of relevant standards, e.g. ISO 5817/EN 25817, and taking into account the criteria as stated in 6.6.1 (see ISO 5817, Table 1). These particulars are to be presented to GL for review and approval. For inspection and testing of welds, see D.

6.7 Repair welding

6.7.1 For repair welds generally the requirements are identical to those applicable to the original weld. Prior to carrying out extensive repair welds or the repetition of repair welds, e.g. owing to inadequate results of non-destructive inspections, consultation with the GL Surveyor shall be sought. The kind and scope of envisaged repair measures have to be agreed with him. For examination of repair welds, see D.6.7.

6.7.2 Where possible, minor surface defects have to be repaired by grinding only, ensuring smooth transitions from the ground area to the work piece or welding seam surface. Extensive defects have to be gouged carefully, examined and welded. Due consideration of welding procedure specifications shall be made. If in one seam section several flaws requiring repair are located close to each other, the whole seam section has to be gouged and welded new.

6.7.3 In general, major repair welds at parts which have already undergone postweld heat treatment, necessitate another heat treatment of the whole part. GL may agree to local postweld heat treatment or - in the case of minor repair welds - dispense with it completely; this has to be decided in each individual case.

6.8 Postweld heat treatment

Note

Stress-relief annealing will be dealt with in the following under the heading of "postweld heat treatment". Should other kinds of postweld heat treatment be intended, then agreement shall be sought from GL, who will consider the conditions on a case to case basis.

6.8.1 Postweld heat treatment should generally be performed at structural welds of joint thicknesses of 50 mm and over, unless adequate fracture toughness can be proved in the as welded condition. For restrained joints of complicated design, postweld heat treatment may be required for thicknesses less than 50 mm.

6.8.2 Any heat treatment is to be performed in accordance with a procedure specification, to be prepared by the manufacturers and submitted to GL for approval. This specification shall detail heating facilities, insulation, control devices and recording equipment, as well as heating and cooling rates, temperature gradients, holding temperature ranges and times.

6.8.3 Wherever possible, post-weld heat treatment is to be carried out in an enclosing furnace. Where this is impractical, local heat treatment may be performed with the consent of GL. When local heat treatment is performed, an area extending over at least 3 times the material thickness on both sides of the weld is to be kept at the specified temperature.

6.8.4 Any heat treatment cycle is to be recorded using thermocouples equally spaced externally and - whenever possible - internally throughout the heated area. The heat treatment records are to be submitted to the GL Surveyor. Regarding inspections and tests after heat treatment see D.6.5.

6.9 Production tests

6.9.1 Production tests, i.e. tests with test pieces welded simultaneously at specified intervals during fabrication, may be called for where the base material, the welding process or the loading conditions require that proof be provided of the sufficient mechanical characteristics of the welded joints produced under fabrication conditions.

6.9.2 Production test pieces shall be welded and tested in a manner analogous to that prescribed in 3.3 in connection with welding procedure qualification tests. The scope of the tests and the requirements to be met shall be determined on a case to case basis.

6.10 Underwater welding

6.10.1 Underwater welding is generally to be carried out in a large chamber from which the water has been evacuated (habitat welding). Underwater wet welding, characterized by the arc working in the water or in a small gas-filled confinement, is normally restricted to minor repair work at secondary structures only in shallow water and requires the special consent of GL from case to case.

6.10.2 Underwater welding is to be performed in accordance with a detailed procedure specification to be prepared by the manufacturers and submitted to GL for approval. It is to be carried out using especially qualified welding procedures (see 3.6) and by special qualified welders (see 4.3). The transfer and handling of welding consumables as well as redrying (baking) and storage in the welding habitat shall be considered in the welding procedure specification.

6.10.3 Production test welds are to be carried out prior to commencing the underwater welding at the site in a manner which, as far as possible, reproduces the actual welding conditions, thus checking that all systems are properly functioning, and results in sound welds. The production test welds are to be visually inspected and non-destructively tested prior to production welding. Subsequently they are to be subjected to mechanical and technological testings. Type and number of test pieces and tests are to be specified and agreed to by GL from case to case.

6.10.4 Visual inspections and non-destructive testings are to be carried out on the actual production welds by competent operators, applying qualified and approved procedures (see also D.). The methods and extent of inspections and testings are to be specified and agreed by GL from case to case. The finished welds are to fulfill the requirements specified for the individual structural parts or welds respectively.

6.10.5 All underwater welding work, including qualifications, production tests, actual welding data and inspection and test results are to be recorded. The records are to be submitted to GL for review and shall be kept for 5 years (cf. D.1.7).

D. Inspection and Testing

1. General requirements

1.1 Inspection and testing shall be performed by the manufacturers after finishing the steel structures or parts of it. These inspections and tests shall ensure that the structure and welds meet the requirements of these Rules, the approved drawings and specifications or any other requirements stated or agreed upon.

1.2 Inspection and testing are to be carried out in the fabrication, construction, and erection phases. For the quality control required during fabrication and construction, see B.4.

1.3 Manufacturers will be responsible for the due performance of the inspections and testings as required, as well as for compliance with the requirements laid down in the documents. Inspections and testings performed by GL do not relieve manufacturers of this responsibility.

1.4 The GL Surveyor shall have the opportunity of witnessing and observing the inspections and tests conducted, or of conducting inspections and tests himself. To this effect the Surveyor shall have permission to enter production areas and building sites and be assisted, wherever possible.

1.5 Following inspection and testing by the manufacturers the structures or parts thereof are to be presented to the GL Surveyor for final checking. Parts are to be presented in suitable sections enabling proper access for inspection normally before painting. The Surveyor may reject components that have not been adequately inspected and tested by the manufacturers, and may demand their resubmission upon successful completion of such inspections and corrections by the manufacturers.

1.6 Assessments of defects (e.g. weld defects), which are identified by their nature, location, size and distribution, shall take due account of the requirements applicable to the welded joints (see 6.4). Inspection or testing results respectively shall be evaluated by the testing department and/or the welding supervisory staff and submitted to GL for review. The final assessment, together with the right of decision as to the acceptance or repair of defects in the material or weld shall rest with the GL Surveyor.

1.7 Reports shall be prepared on all (initial and repeat) inspections and tests, and these shall be submitted to the GL Surveyor together with other documentation (e.g. radiographs) for review. The reports shall contain all the necessary details relating to the particular test method used (see 7.5.5, 8.7.3, 8.7.4, 9.4.2 and 9.4.3), the position at which the test was performed and the results obtained. The reports shall at any time, during fabrication and construction, as well as during erection and installation, permit the unequivocal tracing of tested areas and test results. Test reports and documentation shall be kept for 5 years.

2. Standards, codes

2.1 The standards, codes, etc. mentioned in the following constitute an integral part of these Rules and require no special consent. The version in force on the date of publication of these Rules shall be applied. New editions of the standards, codes etc. may be used in agreement with GL.

2.2 The application of other equivalent standards, codes, etc. (e.g. ANSI/AWS D 1.1 "Structural Welding Code-Steel") may be agreed to by GL. They are to be listed in the construction particulars, e.g. in the inspection and testing plans and specifications, and presented to GL on request.

2.3 Where the standards, codes, etc. are contradictory to these Rules, the latter shall automatically take precedence. Any deviations from these Rules require approval by GL on a case to case basis.

3. Terms and definitions

The terms and definitions as contained in recognized standards, codes, etc. are to be employed. The standard or code used is to be indicated in the inspection and testing documentation and presented to GL on request. Any other deviating terms and definitions are to be explained separately in the inspection and testing specifications.

4. Testing equipment

4.1 The testing facilities and equipment employed have to meet the latest technical standards and have to be in good condition.

If manufacturers do not possess equipment of their own, they may use test equipment of third parties, e.g. testing institutes. GL has to be informed accordingly, see also item 5.3.

4.2 The equipment has to be calibrated and/or gauged and the respective Certificates have to be on hand.

4.3 GL may check the equipment or insist on having the equipment checked in the presence of the Surveyor. Ultrasonic testing equipment and accessories may be checked within the scope of examining the testing personnel as per 5.4.

5. Inspection personnel, supervisors

5.1 Inspection personnel (inspectors)

5.1.1 The non-destructive weld tests may only be performed by persons trained in the use of the test method concerned and possessing adequate practical experience. GL shall be supplied with appropriate documentary proof of such training and experience, e.g. conforming to EN 473/ISO 9712.

5.1.2 Inspection of welds by ultrasonic means shall only be performed by inspectors holding a DGZfP [5] certificate U.2.1 (or equivalent, e.g. ASNT [6] Level II) and having at least 2 years of proven practical testing experience who are recognized by GL.

5.1.3 For such recognition, GL may require verification of the suitability of the ultrasonic inspection personnel and of the test appliances and the test method under practical conditions in the works. In exceptional cases and where necessary for a restricted field of use, GL may, following successful verification, also recognize inspectors who do not hold the certificates specified in 1.2.

5.1.4 Application for such verification shall be made to GL Head Office, accompanied by the following information and documents:

- documentary proof of the professional training of the inspection personnel and, where applicable, the inspection supervisors
- a description of the test equipment (appliances, probes, etc.)
- a description of the test method (instrument setting, angles and scanning directions, instrument sensitivity, etc.)
- method of determining the size of defects
- form of the inspection report

After successful verification, recognition may be linked to authorization of the inspector for the independent performance of certain tests and inspections (materials, weld shapes) under his personal responsibility. The decision lies with GL.

Note:

The recognition and authorization of an inspector normally covers the inspection of normal butt and corner joints (e.g. the joints uniting deck stringers and sheer strakes) or approximately right-angled T-joints in hull structural steels and/or other comparable structural steels. For the performance of further (more difficult) tests (e.g. on other materials and/or on acute-angled tube connections and weld shapes of comparable complexity), the authorization shall be subject to special review and supplementation.

5.2 Inspection supervisors

5.2.1 An appropriately qualified works inspection supervisor shall be available for scheduling and monitoring the performance of the non-destructive weld tests and evaluating the results. The name of the inspection supervisor shall be given to GL; proof of his qualifications (e.g. DGZfP [5] Stage 3, ASNT [6] Level III or, for welding supervisors, to DIN EN ISO 14731 standard with additional NDT training) shall be submitted to GL.

5.2.2 The inspection supervisor is responsible for ensuring that the non-destructive weld tests are competently and conscientiously carried out and recorded by suitable inspectors in accordance with these Rules, the relevant standards and the approved inspection schedule.

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5 Deutsche Gesellschaft für Zerstörungsfreie Prüfung (German Association for Non-destructive Testing).

6 American Society for Non-destructive Testing.

5.2.3 When using the services of outside inspection bodies, the works shall ensure that the above conditions are satisfied and shall inform GL accordingly.

6. Methods and extent of inspection and testing, schedule

6.1 The methods of inspection and testing to be applied in each instance shall be selected with due consideration for the test conditions (shape and size of the weld, nature and location of possible defects, accessibility) so that any defects may be reliably detected. The methods of inspection and testing shall be agreed with GL. GL may stipulate that two or more inspection and testing techniques respectively be used in conjunction with each other.

6.2 A complete visual inspection shall be performed after finishing the steel structure or parts of it. The inspection shall ensure completeness, correct dimensions and satisfactory workmanship, meeting the requirements and the standards of good practice. For quality control during fabrication and construction work, see B.4. A written report shall be prepared, confirming the visual inspection and indicating major corrections. The report is to be submitted to the GL Surveyor prior to the final checking according to 1.5.

6.3 Non-destructive inspections and testings (NDT) are to be performed at the weld connections according to their importance, considering overall integrity of the structure, type of load, stress level and their later (in-service) accessibility. A minimum scope of non-destructive inspections and testings is given in Table 4.22. For the categorization of structural members see A.2.; the relevant weld connections are to be subdivided accordingly.

6.4 The respective inspection and testing plans as well as relevant specifications shall be compiled by the manufacturers for each structure, taking into consideration their individual design and stress conditions. These plans and specifications shall contain the scope and methods of inspection and testings as well as the acceptance limits for surface and internal weld defects. These limits are intended to be used for the assessment of the welds in accordance with common standards, e.g. ISO 5817 (see C.6.6).

Particular attention is to be paid to materials and processes (possibilities for faults) besides the stresses.

Plans and specifications are to be submitted to GL for approval.

6.5 All non-destructive testings of complicated and heavy structures (e.g. tubular node joints and other, thickwalled F-, Y- and K-connections) or of higher-strength and especially high-strength structural steels shall be performed not earlier than 48 hours after weld completion in order to be able to detect delayed cracking. Where components are subjected to post-weld heat treatment, the non-destructive testings shall be performed after this heat treatment.

6.6 Should the inspections and tests reveal defects of any considerable extent, the scope of the tests shall be increased. Unless otherwise agreed, tests shall then be performed on two further sections of weld of the same length for every weld section tested and found to be in need of repair. Where it is not certain that a defect is confined to the section of weld under test, the adjoining weld sections shall be additionally tested. GL may stipulate further inspections and tests, especially in the event of doubts as to the professionally competent and satisfactory execution of the welds.

6.7 Repaired welds shall be re-inspected. Where welds have been completely remade, retesting at least equal in scope to the initial inspections shall be performed in accordance with the Surveyor's instructions. Re-inspections are to be identified as such in test reports and on radiographs, e.g. by a letter "R" (= repair) placed next to the reference of the radiograph.

Table 4.22 Minimum scope of non-destructive inspections and testings [1]

Category / Type of connection	Special structural members (welding)			Primary structural members (welding)			Secondary structural members (welding)		
	RT	**UT**	**MT**	**RT**	**UT**	**MT**	**RT**	**UT**	**MT**
Butt welds	10 % [2]	100 % [2]	10 %	10 %	100 % [2]	10 %	spot	spot	spot
T-joints (full penetration)	–	100 %	100 %	–	100 %	100 %	–	spot	5 %
T-joints	–	[3]	100 %	–	[3]	100 %	–	–	spot
Fillet welds	–	–	10 %	–	–	10 %	–	–	spot

[1] All welds which will become inaccessible or very difficult to inspect in service are to be non-destructive tested over their full length.

[2] Up to weld thicknesses of 30 mm ultrasonic inspection (UT) may be replaced by radiographic inspection (RT) up to an amount of 100 %.

[3] Where partial penetration T-joints are admissible in highly stressed areas, an ultrasonic inspection to determine the size of incompletion and the soundness of the welds may be required.

7. Radiographic inspection

7.1 Radiation sources, appliances

7.1.1 Wherever possible, X-ray units shall be used as radiation sources for radiographic inspections. The radiation energy (tube voltage) should lie within the energy limits specified in DIN EN 1435 / ISO 1106. Allowing for the differences in thickness of the component, the radiation energy (tube voltage) should be kept as low as possible within the permissible working range so as to obtain a high-contrast image.

7.1.2 Where justified in exceptional cases (e.g. by lack of accessibility), gamma ray sources - preferably Ir 192 - may be used, subject to the consent of GL in each instance.

7.2 Films, intensifying screens

7.2.1 Class C5 [7] films may be used for X-raying steel. Class C3 or C4 films are to be used for the radiographic inspection when using gamma rays.

7.2.2 Front and rear 0,02 mm lead screen shall normally be used when radiographing steel. During radiography, the film and the screens shall be kept in intimate contact in suitable cassettes, packs, etc.

7.2.3 The use of salt intensifying screens and fluorometal screens is not allowed.

7.3 Radiographic parameters

7.3.1 The radiographic parameters prescribed in DIN EN 1435 / ISO 1106 for test category A (General Inspection Procedure) are to be applied. For radiographic inspection using X-rays and a film length of 480 mm, the distance between the film and the focal point should normally be 700 mm and in any case not less than the length of the film. In special cases GL may stipulate test category B (Higher-Sensitivity Inspection Procedure).

7.3.2 Any impurities on the surface of the workpiece liable to impair the interpretation of the image are to be removed prior to the radiographic inspection, together with any visible welding defects and damage. Traces of auxiliary welds are also to be removed. In special cases (e.g. where the surface of the weld is very rough or where the image quality is subject to special requirements), grinding of the weld faces may be required.

7.3.3 In order to determine the image quality, at least one image quality indicator according to DIN EN 462 Part 1 and 3 (wire indicator) shall, for each radiograph, be laid on the side of the weld away from the film and facing the radiation source, and shall be radiographed together with the weld. Should this be impossible, the image quality indicator may, with the consent of GL and after the preparation of comparative radiographs designed to determine the changed index of image quality, be fixed to the workpiece on the side close to the film (i.e. between the film and the weld). The film image shall be marked to indicate that this arrangement was used.

7.3.4 Each film image shall be clearly and unmistakably marked by lead figures or letters simultaneously irradiated and depicted on the film. This identification shall be the same as that given in the test plan and shall enable any defects found to be readily located. The marking is to be located outside the weld area being inspected (weld width plus at least 10 mm on each side).

7.4 Film processing, density, image quality

7.4.1 The films shall be processed in properly equipped darkrooms in such a way as to avoid any blemishes which interfere with their evaluation (e.g. fog density, scratches, dark crescent-shaped marks due to kinks in the film, etc.).

The instructions and recommendations issued by the film and chemicals manufacturers are to be followed. Premature interruption of the developing process and the reduction with chemicals of overexposed films is not allowed.

7.4.2 The density "D" of radiographic images shall be at least 2,0 over the entire area for evaluation. The upper limit value depends on the brightness of the film viewers available for the evaluation, but should not exceed 2,5 with 3,0 as the maximum figure. Wider density differences within a single radiograph are to be avoided.

7.4.3 The image quality shall be determined with an image quality indicator prescribed in 7.3.3 and in accordance with DIN EN 462, Part 1. For category A inspection (see 7.3.1), image quality B is desirable for steel, with image quality A as the minimum requirement. In the case of test category B, image quality B shall be attained. The criterion in each case is the smallest wire of the image quality indicator which is still visible in the area of the weld.

7.5 Viewing conditions, evaluation

7.5.1 Viewers with a luminous density to DIN EN 25580 / ISO 5580 sufficient for the required film density shall be used for the examination and evaluation of radiographs. Stops shall be fitted to enable the field of view to be adapted to the film size for, or capable of, evaluation. The brightness shall be adjustable.

7.5.2 The viewing and evaluation of radiographs should take place in a dimly lit though not completely darkened room. Evaluation should only be performed after a sufficient period has been allowed for adaptation. Bright, dazzling areas within the field of view are

[7] Classification acc. to DIN EN 584-1

to be screened. The use of magnifying glasses for the detection of fine details may be beneficial.

7.5.3 The symbols given in Table 4.23 are to be used to identify weld defects in the test report.

Table 4.23 Symbols denoting defects (Extract from EN 26520/ISO 6520)

Reference number	Symbol acc. to IIW	Defect description [1]
100	E	Crack
101	Ea	Longitudinal crack
102	Eb	Transverse crack
104	Ec	Crater crack
2011	Aa	Gas Pore
2015	Ab	Elongated cavity
2016	Ab	Worm hole
2024	K	Crater pipe
301	Ba	Slag inclusion
304	H	Metallic inclusion
4011	–	Lack of side wall fusion
4012	–	Lack of inter-run fusion
4013	–	Lack of fusion at the root
402	D	Incomplete penetration
5011	F	Continuous undercut
5012	F	Localized undercut
5013	–	Root concavity (see also 515)
502	–	Excessive weld metal
503	–	Excessive convexity
504	–	Excessive penetration
507	–	Linear misalignment
510	–	Burn through
511	–	Incompletely filled groove
515	–	Root concavity (see also 5013)
517	–	Poor restart

[1] Explanation and illustration see EN 26520/ISO 6520

7.5.4 Initial evaluation shall be performed by the testing department and/or the welding supervisor of the manufacturers. Thereafter the films (the initial and the follow-up radiographs) shall be submitted to the GL Surveyor for assessment together with the test reports. The assessment shall be made with due regard to the details provided above by the instructions "acceptable" or "not acceptable".

7.5.5 The following information is to be given in the test report, together with explanatory sketches where necessary:

– works number, component, test schedule number, inspection positions

– radiation source and size of tube focus or of emitter

– tube voltage or activity at time of inspection

– radiographic arrangement of DIN EN 1435 / ISO 1106, position of wire penetrameter

– thickness of workpiece or weld, as appropriate

– type of film and nature and thickness of intensifying screens

– test category, image quality index and image quality class

– symbols denoting defects and assessment.

The test report shall also indicate whether the information relates to an initial radiograph or to a follow-up inspection after repair work has been carried out (cf. 6.6).

8. Ultrasonic testing

8.1 Test appliances and accessories

8.1.1 The test appliances, probes and other accessories (calibration and reference blocks for adjusting the sensitivity, reference scales, etc.) shall conform to the state of the art and to the relevant standards (e.g. DIN 54120, DIN EN 27963 / ISO 2400, DIN EN 1714 or DIN EN 583-1).

8.1.2 All possible echo heights within the range of the instrument sensitivity used shall be capable of being determined with the aid on an amplification control calibrated in dB and a suitable scale marking on the display. The interval between the switching stages shall not exceed 2 dB. Instruments not equipped with a calibrated amplification control may not be used.

8.1.3 Stepless controls shall enable the ranges of adjustment available on the instrument to be juxtaposed, as far as possible without any intervening gap. Within each individual range the time sweep shall be continuously adjustable.

8.1.4 With regard to the geometrical characteristics of the sound field, especially the incidence and squint angles, the test frequency and the resolution, the probes shall lie within the tolerances specified in the standards mentioned above. The incidence and squint angles shall not in either case deviate by more than 2° from the nominal value or from the centre line of the probe. The angle of incidence and the probe index (of angle beam probes) shall be verified.

8.2 Calibration, sensitivity setting

8.2.1 The distance signal (time sweep) may be optionally calibrated in projection distances "a", shortened projection distances "a'", sonic distances "s" or, if possible, depth positions "b". Unless otherwise agreed, calibration in shortened projection distances "a'" shall be preferred for weld inspections, or in sonic distances "s" for parts of complex shape (e.g. tubular joints acc. to Fig. 4.24).

8.2.2 For calibrations in accordance with 8.3.1 a calibration block according to DIN 54120 or DIN EN 27963 shall be used for testing structural steels. Appropriate calibration or reference blocks shall be used for materials having other sound velocities (e.g. high alloy steels). Bore holes used for calibration shall not be larger than 2 mm and shall lie parallel to the testing surface. Where possible, calibration should not be performed at edges.

8.2.3 Depending on the intended method of echo height definition, the sensitivity setting shall be performed using calibration reflectors of known shape, position and size (e.g. large flat reflectors, side-drilled holes) in accordance with the provisions of DIN EN 583-2. Unless otherwise agreed, the DGS method [8] of inspection shall be used. With the DGS method, the sensitivity setting is to be carried out in accordance with the instrument manufacturer's instructions using calibration blocks to DIN 54120 and DIN EN 27963 / ISO 2400.

Flat bottom holes and grooves should not be used as calibration reflectors.

8.2.4 If necessary (e.g. for defects close to the surface), the sensitivity setting is to be corrected in accordance with DIN EN 583-2. In testing unalloyed and low alloy structural steels and where the sonic distances are not too great (cf. DIN EN 583-2), the sound attenuation may normally be disregarded. A transfer correction to determine the coupling differences between the surface of the reference block and that of the test piece shall, however, be performed in every case. The value of the transfer correction shall be stated in the test report.

8.2.5 For more efficient detection of defects it is recommended that the work should be performed with a test sensitivity (search sensitivity) increased by approximately 6 dB over the chosen registration level (see 8.6). However, the registration level setting is generally to be used when evaluating defect indications. All echo indications to be registered shall attain at least 20 % of the display height even at the maximum sonic distance (cf. DIN EN 583-2). In the case of electrogas-welded seams, the inspection shall normally be performed with a sensitivity increased by 12 dB, and this fact shall be expressly stated in the test report with a reference to the welding process (e.g. EG + 12 dB).

8 Distance Gain Size method

8.3 Surface preparation, coupling

8.3.1 On both sides of the welded seam (cf. 8.5.1) the testing surfaces shall be smooth and free from impurities liable to interfere with coupling. Rust, scale and weld spatter are to be removed so that the probes lie snugly against the surfaces, which should if necessary be ground. Firmly adhering paint need not be removed provided that it does not interfere with the inspection and quantitative allowance can be made for the resulting loss of sensitivity when evaluating the echo heights.

8.3.2 Where angle beam probes have to be applied to the surface of the weld for the inspection of transverse defects (see 8.5.3), this shall also be prepared in the manner of the testing surfaces described above. Notches, grooves and the like lying across the beam axis which produce false indications and may impair the test are to be removed.

8.3.3 Coupling to the testing surfaces prepared in accordance with 8.4.1 should be as uniform as possible and shall not vary by more than ± 4 dB. If greater variations are found, the state of the surface shall be improved. Where greater variations cannot be avoided, this fact shall be stated in the test report. Flowing water, cellulose glue, oils, fats or glycerine may be used as coupling media.

8.4 Test directions, angle of incidence

8.4.1 Unless otherwise agreed or stipulated, testing for longitudinal defects shall be performed from one surface and from both sides of the weld, as shown in Fig. 4.28. The testing area shall embrace the weld metal itself and an area on both sides of the seam equal to about 1/3 of the wall thickness, subject to a minimum of 10 mm and maximum of 20 mm. The testing area shall encompass a width equal at least to the full skip distance plus twice the length of the probe.

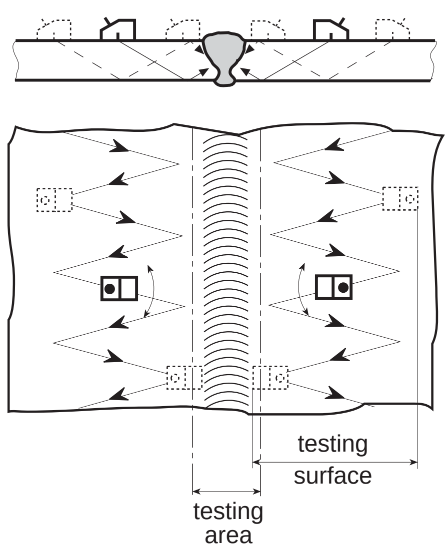


Fig. 4.28 Testing for longitudinal defects in butt welds

8.4.2 Depending on the weld geometry and the possible orientation of defects it may be expedient to perform the test from both surfaces or (e.g. in the case of bevels) from only one side of the seam. With corner and T-joints, the test shall normally be performed both from the side of the web using an angle probe and from that of the continuous (flange) plate using a standard probe, as shown in Fig. 4.29. Such probe arrangements differing from 8.4.1 shall be especially noted in the test report. The same applies in analogous manner to curved surfaces such as shown in Fig. 4.30 in case of tubular connections.

8.4.3 Testing for transverse defects is to be performed from both sides of the weld in two directions along the seam as shown in Fig. 4.31 or - where the test requirements are more stringent - on the ground surface of the weld.

GL may require that testing for transverse defects be performed with two probes connected in parallel.

Where welds are made with a large weld pool (as in electroslag welding), testing for oblique defects shall also be performed at an angle of approximately 45° (cf. DIN EN 1714).

8.4.4 With plate thicknesses (weld thicknesses) of less than 30 mm, testing may be performed with an angle of incidence of 70°. With thicknesses of 30 mm and over, two angles of incidence (70° and 45° or 60°) shall be used. Where the surface is curved, the necessary angle of incidence shall be determined in accordance with DIN EN 583-2. With very large wall thicknesses (above about 100 mm) the inspection shall be performed using a tandem technique (with fixed, mechanical coupling of two similar probes) for different depth zones.

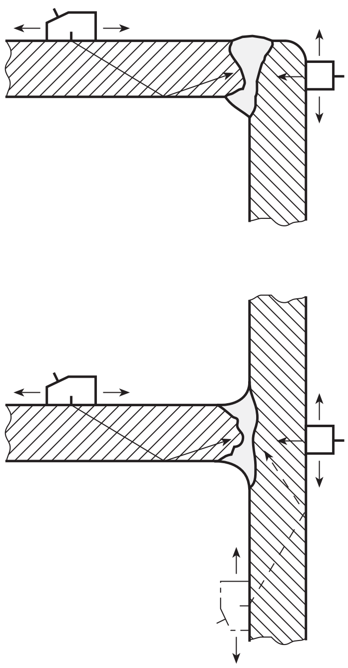


Fig. 4.29 **Testing for longitudinal defects in corner and T-joints**

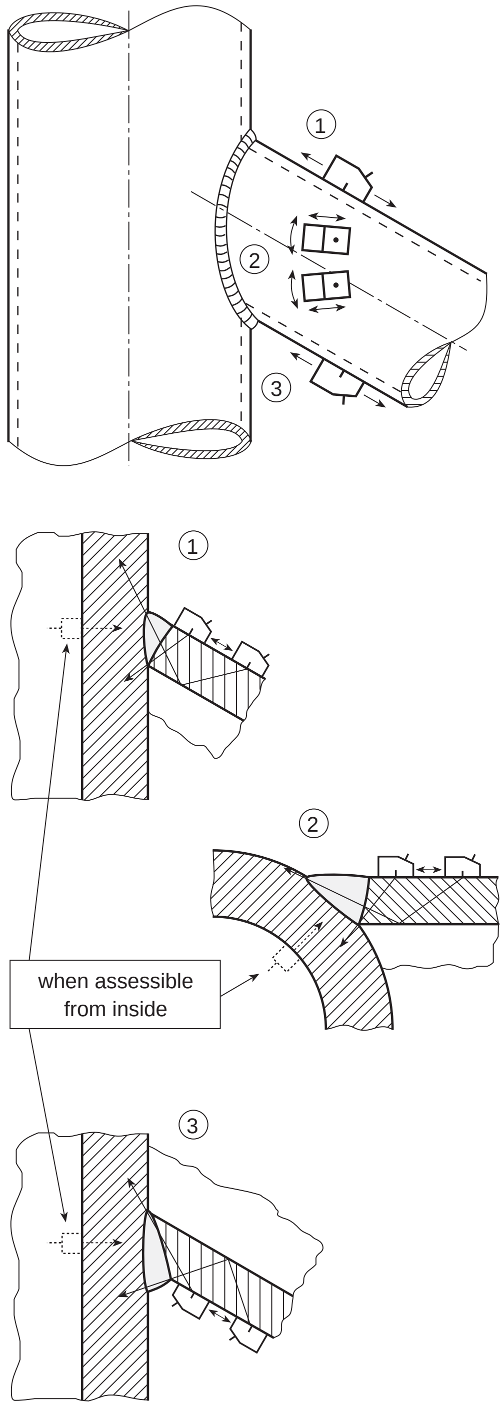


Fig. 4.30 **Testing for longitudinal defects in tubular joints**

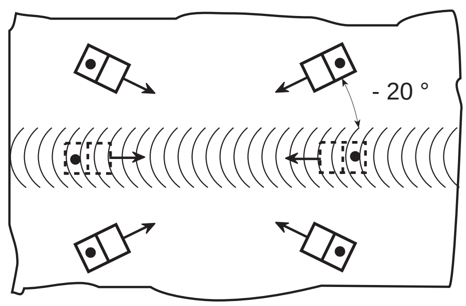


Fig. 4.31 **Testing for transverse defects**

8.5 Registration level, evaluation of echo indications, acceptance limits

8.5.1 For tests carried out by the DGS method, the registration level (reference reflector size) for longitudinal and transverse defects is given by the diameters of the disc shaped reflectors specified in Table 4.24 in relation to the wall thickness (weld thickness).

Where the thickness is greater than 60 mm, the registration level will be determined on a case to case basis. For tandem testing the registration level shall be determined by a 6 mm diameter disc shaped reflector. For other methods of echo height definition (e.g. the reference block method) the registration level shall be determined in accordance with DIN EN 583-2.

Table 4.24 Registration levels

Wall thickness (weld thickness)	Diameter of disc 4 MHz	Shaped reflectors 2 MHz
≥ 10 up to 15 mm	1,0 mm	1,5 mm
> 15 up to 20 mm	1,5 mm	2,0 mm
> 20 up to 40 mm	2,0 mm	3,0 mm
> 40 up to 60 mm	3,0 mm	4,0 mm

8.5.2 The registration of non-form-related echo indications, which are observed when inspecting welded joints and whose echo heights attain or exceed the registration level (reference reflector size) specified in 8.5.1, is required only where expressly stipulated by GL or where subsequent repeat tests have to be performed. Otherwise only those echo indications shall be registered which exceed the repair limit value specified in 8.5.4.

8.5.3 For the classification of echo indications it shall be stated by how many dB the maximum echo height of the reflections found differs from the registration level defined in 8.5.1. In the case of the DGS method, the size of the (substitute) disc shaped reflector may also be stated. Further characteristics to be stated are the registration lengths and half-value depths in accordance with DIN EN 1714. The location of reflexions shall be defined by coordinates indicating the ‘longitudinal and transverse distances from a reference point’ and the ‘depth position’.

8.5.4 Echo indications produced by longitudinal defects which exceed the acceptance limit values shown in Table 4.25 (excess of registration lengths and/or echo heights above the registration level shown in Table 4.24) shall be regarded as weld defects which shall be repaired. Continuous echo indications which point to systematic weld defects (such as root defects due to incomplete penetration or rows of pores) call for repairs even if the repair limit values are not attained. Echo indications which point to the presence of cracks necessitate repairs in every case.

8.5.5 Echo indications produced by transverse defects shall in every case count as weld defects requiring repair unless they can be unequivocally associated with the indications produced by longitudinal defects.

8.5.6 Where the evaluation of echo indications gives rise to doubt regarding the need for repair, recourse may be had to radiographic inspection to help in the assessment (cf. 6.1). However, echo indications obtained with welded seams 30 mm or more in thickness which exceed the repair limit values specified in Table 4.25 invariably necessitate repair, even if radiographic inspection fails to reveal any defects or fails to reveal them clearly.

8.5.7 During testing of one side welded tubular intersection seams as per Fig. 4.30 the root area has to be specially assessed. Here, the acceptance limits as per Table 4.25 are not applicable. Depending on the envisaged seam shape, the details of analysis of indications from the root area have to be agreed with GL from case to case.

8.6 Test reports

8.6.1 Complete test reports as prescribed in DIN EN 1714 and containing the information listed below shall be prepared for all ultrasonic tests in accordance with the test schedule (cf. 6.4) The test reports shall enable the inspections to be repeated identically. They shall be signed by the person performing the test and countersigned by the supervisor.

8.6.2 The test reports shall contain the following details:

- clear identification of the test piece, the material, the welded joint inspected together with its dimensions and location (sketch to be provided for complex weld shapes and testing arrangements) and the welding process
- indication of any other rules (e.g. specifications, standards or special agreements) applicable to the inspection
- place and time of the inspection, testing body and identification of the person performing the test

8.6.3 Test reports shall contain at least the following specific details relating to the inspection:

- make and type of test equipment
- make, type nominal frequency and angle of incidence of probes
- distance calibration (testing range)
- sensitivity setting (calibration reflector used, instrument sensitivity, registration level)
- correction values (for defects close to surface, transfer correction)
- test sensitivity

Table 4.25 Acceptance limits for ultrasonic testing

Evaluation group acc. to DIN EN 25817/ ISO 5817 [1]	Wall thickness [2] [mm]	Longitudinally orientated indications			Transversely orientated indications [4]		
		Number of indications per m seam length	Registration length [3] [mm]	Permissible excess echo height [dB]	Number of indications per m seam length	Registration length [3] [mm]	Permissible excess echo height [dB]
B	10...15	10 and 3 and 1	10 20 10	6 6 12	3	10	6
	> 15...20	10 and 3 and 1	10 20 10	6 6 12	3	10	6
	> 20...40	10 and 3 and 1	10 25 10	6 6 12	3	10	6
	> 40	10 and 3 and 1	10 30 10	6 6 12	3	10	6
C	10...20	10 and 3 and 1	15 30 10	6 6 12	3	10	6
	> 20...40	10 and 3 and 1	15 40 10	6 6 12	3	10	6
	> 40	10 and 3 and 1	15 50 10	6 6 12	3	10	6
D	10...20	10 and 3 and 1	15 50 10	6 6 12	5	10	6
	> 20...40	10 and 3 and 1	15 50 10	6 6 12	5	10	6
	> 40	10 and 3 and 1	20 50 10	6 6 12	5	10	6

1 See C.6.6, D.6.4.

2 Where the wall thicknesses differ, the lesser wall thickness (weld thickness excluding the weld reinforcement) is meant.

3 Where the wall thickness exceeds 60 mm, it is to be divided into test zones, each 60 mm thick, and is to be inspected from both faces. Each test zone is to be assessed separately. Test zones may overlap.
Continuous echo indications, which point to systematic weld defects (e.g. root defects due to incomplete penetration or rows of pores), call for repairs even if the repair limit values are not attained.

4 Echo indications of transverse defects are in any case to be regarded as indicating welding defects requiring repair. This does not apply where echo indications can be unequivocally correlated to longitudinal defects.

– surface preparation, coupling media

– testing surfaces, test directions, angles of incidence

8.6.4 The test results - where these are to be stated in the report (cf. 8.5.2) - shall, wherever possible, be tabulated or shown on sketches with the following details:

– coordinates of defects with stated reference point

– maximum excess echo height (+ ... dB) above the given registration level (reference reflector size) or, where applicable, the diameter of the corresponding (substitute) disc shaped reflector

– defect characteristics (registration length, half-value depth)

Where echo indications below the acceptance limit values shown in Table 4.25 are also registered, each defect thus identified is to be allocated on assessment (e.g. acceptable or not acceptable).

9. Magnetic particle and dye penetrant inspection

9.1 Test methods, test equipment, test media

9.1.1 Wherever possible, the magnetic particle method should be used for testing magnetic materials for surface cracks. The use of the dye penetrant method for magnetic materials is to be restricted to unavoidable, exceptional cases and requires GL's consent on each occasion. Non-magnetic materials (e.g. austenitic stainless steels and non-ferrous metals) are to be tested by the dye penetrant method.

9.1.2 The test equipment and test media used shall conform to the state of the art and to the relevant standards (e.g. DIN 54130, 54131, 54132 and DIN EN 571 Part 1). The magnetizing equipment shall be provided with markings or measuring devices which indicate the magnetizing current and field strength at any time. GL may stipulate that measurements be performed to verify these data. On request, proof shall be furnished to GL of the suitability of the test media employed.

9.2 Magnetic particle inspection

9.2.1 Wherever possible, magnetization shall be effected by passing a current through the workpiece or, in the case of minor localized inspections, by yoke magnetization using electromagnets or, if necessary, permanent magnets.

In special cases (e.g. where burn marks have to be avoided at all costs or for circumferential welds), it may be expedient to effect magnetization with a live conductor (a cable or coil). A combination of different methods of magnetization for the detection of variously orientated defects is allowed.

9.2.2 Where the current is passed through the workpiece, alternating, direct, impulse or surge current can be used. AC or DC magnets may be used for yoke magnetization. Where the magnetizing current is passed through the workpiece, fusible supply electrodes should be used to prevent burn marks. Where AC is used, fusible electrodes are obligatory.

9.2.3 The effective magnetizing (tangential) field strength shall be at least 20 A/cm (25 Oe), but shall not exceed 50 A/cm (62,5 Oe). The adequacy of the magnetization shall be checked at the time of the test by suitable means (e.g. test indicator or with a tangential field strength meter).

9.2.4 Magnetic particles suspended in suitable, readily volatile vehicle liquids shall be used as test media for revealing the leakage flux due to discontinuities in the material. These magnetic particles may be black or fluorescent. Where black magnetic particles are used, the surface to be tested shall be coated with a permanent white paint, applied as thinly as possible, to provide a contrast. The proportion of magnetic particles in the vehicle liquid shall conform to the manufacturer's instructions and shall be verified (e.g. by means of a test indicator or by a separation test using a glass centrifuge vessel to ASTM D 96-73, Fig. 6). Dry test media may only be used for tests at elevated temperatures (e.g. on root passes).

9.2.5 The testing surfaces shall be free from loose scale, rust, weld spatter and other impurities. Notches, grooves, scratches, edges, etc. which can produce false indications, are to be removed prior to the test.

9.2.6 Magnetization shall be effected, as shown in Fig. 4.32, in two different directions including an angle of not less than 60° and not more than 90° so as to enable variously orientated defects to be located.

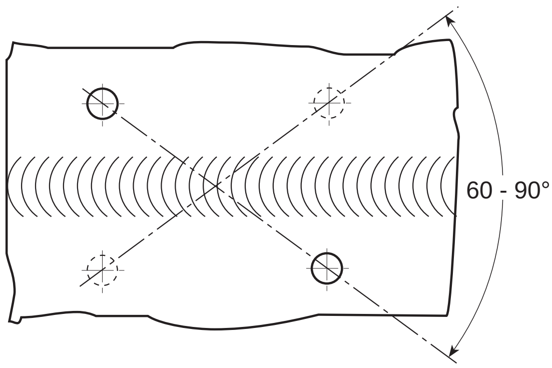


Fig. 4.32 Directions in which magnetization is to be effected

9.2.7 Magnetization shall be continued at least as long as the testing surface is being sprayed with magnetic particle suspension and for as long thereafter as any movement of the vehicle liquid can be detected, subject to a minimum of 5 seconds. Testing under conditions of residual magnetization is not permitted.

9.2.8 Every accumulation of magnetic particles, not due to a false indication, indicates a discontinuity or crack in the material which is to be registered in the test report and repaired. In the case of small cracks (e.g. end crater cracks) this can be done by grinding. Larger cracks are to be ground out and repair welded.

9.3 Dye penetrant inspections

9.3.1 Where possible, coloured dyes should be used as penetrant media, although fluorescent penetrants may also be used. The penetrant remover and the developer shall be compatible with the penetrant used. On request, proof shall be furnished to GL of the suitability of the inspection system.

9.3.2 To allow the penetrant to enter any defects present, the testing surfaces shall be completely freed from scale, rust, greases, oils, paints or electrodeposits before the penetrant is applied. During this operation care shall be taken to ensure that defects are not mechanically sealed by preliminary cleaning. The testing surfaces shall be dry. The temperature of the workpiece shall be between 5 °C and 50 °C.

9.3.3 Any method of applying the penetrant may be used. Care shall be taken to ensure that the testing surface is completely wetted throughout the entire penetration time. The penetration time shall be chosen in accordance with the manufacturer's instructions but shall not be less than 15 minutes for workpiece temperatures of 15 °C and over or less than 30 minutes where the temperature is below 15 °C. The penetrant shall not become dry during the penetration period.

9.3.4 Following penetration, the surplus penetrant shall be completely removed from the testing surface in such a way as to leave behind the portion lodged in possible defects. It is advisable first of all to wipe off the surplus penetrant with a cloth and only thereafter quickly to remove the remains with sparing use of the penetrant remover. The testing surface should then be dried as quickly as possible (max. 50 °C).

9.3.5 The developer is to be applied evenly and as thinly as possible immediately after removal of the surplus penetrant and drying. The testing surface should be just covered. The developing time should be about the same as the time allowed for penetration. Visual inspection for defects shall begin as the developer is applied, but the final inspection can only take place after the expiry of the developing time. 9.2.8 applies in analogous manner to the assessment.

9.3.6 Should an unequivocal assessment of the indications be impossible, the entire inspection procedure, starting with preliminary cleaning, shall be repeated. Where necessary, the surface quality shall also be improved. The repeat inspection shall be performed using the same system as on the first occasion. The conditions specified in standard DIN EN 571 part 1 are also applicable.

9.4 Test reports

9.4.1 Complete test reports with full details of the performance of the test and the defects found shall be prepared for all surface crack inspections in accordance with the test schedule (cf. 6.4). These test reports shall be signed by the person performing the test and by the test supervisor.

9.4.2 Test reports relating to magnetic particle inspections shall include the following details:

– details of the structural component and weld concerned

– method of magnetization

– type of current, and where appropriate the amperage used for magnetization

– test arrangement (magnetizing equipment, distance between electrodes or poles)

– test media

– test results

– place and time of the inspection, testing body and identification of the person performing the test

9.4.3 Test reports relating to penetrant medium inspections shall include the following details:

– details of the structural component and weld concerned

– test media (type, brand name)

– description of the test procedure (temperature of the workpiece, penetrant acting time, etc.)

– test results

– place and time of the inspection, testing body and identification of the person performing the test

Test reports shall conform to the form provided in Appendix A to DIN EN 571 Part 1.

Section 5

Concrete Structures

A. General

1. Scope

1.1 This Section on concrete structures refers to loadbearing and stiffening components of offshore structures made of ordinary or heavyweight concrete with closed texture. It is applicable to plain, reinforced and pre-stressed concrete structures.

1.2 The requirements of this Section may also be applied to barge type units made of concrete, which otherwise, for example regarding loads, would be designed according to the GL Rules I – Ship Technology, Part 1 – Seagoing Ships, Chapter 1 – Hull Structures.

2. Standards and safety concepts

2.1 The safety concept, structural analysis and design of structures shall follow the current state of the art, with reference to the German Standards DIN 1045-1, Plain, reinforced and pre-stressed concrete structures and DIN 1055-100-01, Actions on structures-basis of design, safety concept and design rules.

The partial safety factors for actions on structures and to determine the resistance at ultimate limit state shall be defined according to DIN 1045-1, Edition 07.2001, Section 5.3.3.

2.2 Design, analysis and construction may also be based on acknowledged international or national standards for concrete structures, using other safety concepts (probalistic or semi-probalistic concepts) than those mentioned above.

However, only one uniform safety concept is to be used throughout for one particular offshore installation.

2.3 Acknowledged standards

For example the following acknowledged standards and regulations may be used:

- ACI Standards 318, Building Code Requirements for Structural Concrete with Commentary
- ACI Report 357 R-84, Guide for the Design and Construction of Fixed Offshore Concrete Structures
- British Standard BS 8110, The Structural Use of Concrete
- FIP Recommendations, Design and Construction of Concrete Sea Structures
- Norsk Standard NS 3437, Concrete Standard Design and Detailing Rules, 6th edition 2003 and quoted normative references Part 1 – 4, Chapter 2

The standards intended to be used for design, analysis and construction are to be indicated to GL in good time, and their application is to be approved by GL in each individual case.

2.4 Additional standards

If the acknowledged standards do not cover all the areas of design, analysis and construction, additional appropriate standards are to be incorporated. This applies in particular to manufacture, testing and quality control of materials. Exceptions can be agreed with GL.

2.5 Other design principles

Principles of construction and design, material qualities or methods of stress analysis differing from those in the acknowledged standards may be adopted if their practicability is proven by tests, experiments or theoretical investigations, and if it is shown that they satisfy the safety requirements of these Rules.

3. Documents to be examined

3.1 Scope

The documents to be examined include the building specifications, the drawings and the structural design calculations (static and dynamic analysis).

3.2 Building specification

The building specification shall contain the information necessary for understanding the drawings and calculations, and for construction, transport, installation (especially when using pre-fabricated structural members) and supervision of the construction.

Design parameters and materials shall be described in specifications. Construction, transport and installation procedures shall also be described.

3.3 Drawings

The drawings shall show clearly and conveniently the dimensions of the components and their reinforcement. The strength class of the concrete is to be stated. The reinforcement drawings shall also contain data on the concrete cover, the type of reinforcement steel and the number, diameter, shape and position of the reinforcing bars.

3.4 Stability

The stability and adequate dimensioning of the structure and its components shall be analysed and presented in a convenient and readily checkable form.

B. Materials

1. Cements

For plain concrete of the higher strength classes and reinforced concrete, cement as specified in the standards shall be used (e.g. to DIN 1164).

2. Aggregates

2.1 Aggregates shall comply with the standards (e.g. DIN 4226). The combined aggregate shall be as coarse-grained and dense-graded as practicable. The maximum particle size is to be selected as a function of the spacing between the reinforcing bars and of the concrete cover, so that placing and compaction of the concrete is possible, see F.3.

2.2 Under no circumstances may aggregate with alkali-reactive constituents be used.

2.3 The granulometric composition of the aggregate is to be in accordance with the requirements of the standards and in general will be based on grading curves. Characteristic values for the grading and the water demand may be used.

2.4 Aggregates with steady and discontinuous grading curves (gap gradings) may be used.

3. Admixtures

3.1 Admixtures may be used in concrete and cement mortar only if it has been proven by tests that they do not unfavourably affect the important properties of the concrete and do not affect the corrosion protection of the reinforcement. The conditions specified in the test Certificate are to be maintained. Special consistency checks for the concrete to be produced may be requested by GL in individual cases.

3.2 Chlorides, substances containing chlorides or any other substances causing steel corrosion shall not be added to reinforced or pre-stressed concrete.

4. Water

4.1 The water used shall contain no constituents capable of unfavourably affecting hardening or other properties of the concrete or the corrosion protection of the reinforcement. Where there is any doubt, investigation of the suitability of the water will be necessary.

4.2 Sea water may be used only in special cases for plain concrete, agreement on this is required.

5. Concrete

5.1 Concrete is classified into strength classes, e.g. in DIN 1045-1, Table 9 and 10, on the basis of its compressive strength at an age of 28 days. The compressive strength is determined on cubes with 15 cm side length with reference to DIN EN 206-1/DIN 1045-2.

5.2 The nominal strength is based on the 5 % fractile of the statistical parent population. It is regarded as having been attained if the compressive strength of each cube does not fall below the values of row 2 in tables 9 and 10 of DIN 1045-1. In the case of concrete of the same composition and manufacture, however, one of 9 successive cubes may fall below the values of row 2 of tables 9 and 10 by not more than 20 %.

5.3 A series consists of three cubes made from three different batches from the mixer. The series strength is regarded as having been attained if the average compressive strength of each series of three consecutive cubes does not fall below the values according to DIN EN 206-1.

5.4 Concrete exposed to sea water shall be of strength class e.g. according DIN 1045-1, Table 3, item 3 and 4.

5.5 The ultrafines content of concrete consisting of cement and particles up to 0,25 mm is to be so selected that the concrete is properly workable and achieves a closed texture.

5.6 The concrete shall contain so much cement that the required compressive strength, and in reinforced and pre-stressed concrete an adequate degree of protection of the steel against corrosion can be achieved.

5.7 The water/cement ratio shall be in accordance with, e.g. DIN EN 206-1 and DIN 1045-2.

5.8 Consistency of fresh concrete is to be specified before the start of work, having regard to the conditions for placing and working the concrete.

6. Concrete with special properties

6.1 Concrete which is exposed to frequent and abrupt alternations of freezing and thawing in a moisture-saturated condition shall have high frost resistance. This requires frost-resistant aggregate and waterproof concrete. If the concrete frequently comes into contact with de-icing salts or similar chemicals, a sufficient quantity of an air-entraining additive shall be added so that the mean air content in the fresh concrete makes up approximately 4 % of the volume. The required minimum concrete strength class shall be in accordance with e.g. DIN 1045-1, Table 3, item 5.

6.2 The resistance of concrete to chemical attack depends to a great extent on its density and its water/ cement ratio. If there is a strong likelihood of chemical attack the concrete shall be dense enough to ensure that the maximum depth of water penetration under test (e.g. to DIN 1048) is not greater than 3 cm in accordance with DIN 1045-1, Table 3, item 6.

6.3 For concrete which is exposed to attack by water containing more than 400 mg of SO_4 per litre, cement with high sulphate resistance is to be used.

6.4 Concrete for placing under water (underwater concrete) shall flow as a coherent mass so as to obtain a closed even texture without compaction. It is preferable to use continuous gradings and a sufficiently high content of ultrafine particles, see also D.4.

7. Injection grout

7.1 Injection grout for bonded tendons is made from cement, water and admixtures, and its purpose is to ensure bonding between the pre-stressing steel and the structure by enveloping the pre-stressing steel and filling in all the hollow spaces in the stressing channel, and to protect the steel against corrosion.

7.2 In general only Portland cement is to be used. Tap water from the public supply is suitable in the majority of cases for preparing injection grout. Sea water shall not be used. The additives shall be tested for suitability.

8. Reinforcing steel

The diameter, shape, strength properties and distinctive marks of reinforcing steel for concrete shall comply with the standards (e.g. DIN 488).

9. Pre-stressing steel

9.1 The properties of the pre-stressing steel are to be attested by Certificates from the manufacturer's mill. In particular data and test results are to be submitted on the composition of the steel, the method of manufacture, the stress-strain curve, the elastic limit, the yield point, the tensile strength and the creep limit. These documents may be substituted by an official authorisation.

9.2 Regarding the pre-stressing procedure, see C.7.2.

C. Verification of Quality

1. General requirements

The contractor's supervisor is responsible for the execution and interpretation of the tests, specified below, for taking due account of the results of such tests in the execution of the job and for submitting the results to GL.

2. Cement and admixtures

For each delivery it should be checked that the information on the package and on the delivery note conforms to the particulars in the technical documents. In the case of admixtures attention is to be paid to the test mark on the package.

3. Aggregate

3.1 Aggregate should be regularly checked visually with regard to its granulometric composition and other significant properties. In doubtful cases the aggregate should be examined more thoroughly.

3.2 Sieve tests are necessary when the first delivery is affected and whenever there is a changeover to a different supplier.

4. Concrete

4.1 The testing procedure and the manufacture and storage of the test specimens shall comply with the standards (e.g. DIN 1048).

4.2 Preliminary test

The purpose of the preliminary test is to establish, before the concrete is used, what composition it has to have in order to attain the required properties.

4.3 Quality control test

The purpose of the quality control test is to establish that the concrete made for use on the job attains the required properties. The compressive strength, the water/cement ratio and the consistency of the concrete are to be confirmed.

4.4 For the quality control test of the compressive strength, a series of three test specimens is to be made for each 500 m³ of concrete and at least on every seventh day of concreting (cf. B.5.3).

5. Injection grout

5.1 Preliminary tests of the compressive strength are to be carried out for each section of the construction before commencement of the injection work on specimens at least 7 days old.

5.2 A suitable number of quality control tests is to be made during injection work e.g. according to DIN 1045 and quoted normative references.

5.3 The flowability, change of volume, compressive strength and the resistance to premature setting of the grout are to be verified for each section of the construction.

6. Reinforcing steel

6.1 For each delivery of reinforcing steel it shall be checked that the steel bars have the distinctive mark - as laid down in the standard (e.g. DIN 488) – for the steel group and also the steelworks mark. If it does not, a yield point of only 220 N/mm^2 shall be assumed for the steel. Exceptions may be approved if the properties of the steel are confirmed by an independent institution.

6.2 Welding on reinforcing steel may only be carried out by firms possessing the required specialists and welding supervisors. Only welders who are especially qualified to weld reinforcing steel may be employed. Before welding begins, preliminary tests (procedure tests) are to be carried out under local manufacturing conditions, and during welding monitoring checks are to be undertaken. The individual requirements of the standard (e.g. DIN 4099) are to be followed.

7. Pre-stressing steel

7.1 Pre-stressing steel may only be used if the Certificates from the steel mill as to its properties are available, or if an official authorisation is obtained. The requirements contained therein, e.g. as to additional tests or special procedures during installation, are to be followed when verifying quality.

7.2 An approved pre-stressing method statement, covered by ministerial decree of the country of origin, shall be permanently available at the construction site.

8. Supervision of the work

8.1 The contractor's supervisor shall ensure that the work is properly carried out in accordance with the examined documents, particularly in respect of

– accuracy of dimensions of the components

– safe construction of form- and falsework

– adequate quality of the materials used

– conformity of reinforcing and pre-stressing steel with the examined drawings

8.2 GL is to be notified, as far as is practicable 48 hours in advance of commencement of the work, of the planned initial concreting, recommencement of concreting work after any extensive interruption, and of any major welding work on the site.

8.3 Depending on the nature and extent of the work to be carried out, reports are to be continuously prepared by the contractor's supervisor on the site for all processes incidental to quality and stability. These reports shall be available during the construction period on the site and are to be submitted to the supervising Surveyor on request.

D. Placing and Curing the Concrete

1. Handling the concrete

1.1 The method of handling and conveying and the composition of the concrete shall be so adjusted as to obviate segregation.

1.2 Fall tubes determining the point of deposit are to be used in column and wall formwork.

1.3 Transport pipes for pumped concrete are to be chosen and laid in such a way that there is no break in the flow of concrete inside the pipes.

2. Placing and working the concrete

2.1 The reinforcing bars shall be densely embedded in concrete.

2.2 The concrete shall be compacted as completely as possible.

2.3 Individual concreting sections are to be determined before commencement of the concreting work. Construction joints are to be designed in such a way that all the stresses occurring can be withstood. In waterproof concrete the joints shall also be waterproof.

3. Curing the concrete

3.1 Until it has hardened, concrete shall be protected from harmful influences. This applies also to mortar and concrete placed in joints between pre-cast concrete components.

3.2 In order to slow down the shrinkage of the young concrete and to ensure that it will harden properly at the surface also, it shall be kept moist. Further curing procedure is given in DIN 1045-1, Table 3.

3.3 Sea water shall not be used for curing.

4. Concreting under water

4.1 Underwater concrete in accordance with DIN 1045-2 and DIN 1045-3 is to be placed uninterruptedly by means of fixed hoppers in such a manner that it does not come into contact with the water on the delivery path. The water in the foundation pit shall be still.

4.2 At water depths of up to 1 m the concrete may be placed by careful forward feeding along a natural slope.

4.3 Underwater concrete is, alternatively, allowed to be made by injecting grout with low segregating tendency from below into a mass of aggregate.

5. Concreting in cold or hot weather

5.1 In cold weather and during frost the concrete shall have a certain minimum temperature at the time of placing and be protected for a given period against heat loss and drying.

5.2 Fresh concrete shall not be poured onto frozen concrete members.

5.3 In hot weather the influence of the sun on the fresh concrete has to be taken into account (e.g. by covering). Curing shall start as soon as possible taking into account high temperatures.

5.4 The temperature of the fresh concrete may be lowered by cooling the aggregate and water. In hot climates, the maximum allowable ambient temperature for pouring shall be agreed with GL.

E. Formwork and Falsework

1. Formwork

1.1 Formwork and moulds shall as far as practicable be made to the precise dimensions and be tight enough to ensure that fine mortar ("grout") in the concrete does not escape through the joints when the concrete is placed or compacted.

1.2 Formwork and falsework are to be designed so as to carry safely all loads on it until the concrete hardens. The influence of the filling rate and the method of compacting the concrete have to be taken into account.

2. Dismantling the falsework

2.1 Dismantling may only start if the contractor's supervisor has confirmed that the concrete is sufficiently strong and has ordered removal.

2.2 Special care is requested in the case of components that have been exposed to low temperatures after placing of the concrete.

2.3 Auxiliary supports shall be left or be erected immediately after dismantling the falsework in order to minimise displacements due to creep or shrinkage of the concrete.

F. Reinforcement

1. Installing of reinforcement

1.1 Substances that may affect bonding, such as dirt, grease, ice, loose rust, are to be removed from the steel before it is used.

1.2 The steel bars are to be joined in a rigid framework and kept in their proposed positions by means of spacers not affecting the corrosion protection, so that they are not displaced when the concrete is placed or compacted.

2. Concrete cover to reinforcement

2.1 The concrete cover of each reinforcing bar or stirrup shall not be less than the values on all sides according to DIN 1045-1, Table 4.

2.2 In the case of secondary loadbearing members of pre-cast concrete components lower values may be selected for the concrete cover on the basis of the standards and with the agreement of GL.

2.3 For mobile units capable of being drydocked, a reduction of the concrete cover may be agreed upon.

3. Bars and tendons

3.1 The clear distance between parallel reinforcing bars shall be at least 2 cm and may not be less than the bar diameter. Lap joints should be staggered, where possible, and at no point in area of major stress shall all bars overlap, with reference to DIN 1045-1, Section 12.8.

3.2 The spacing of tendons shall be such as to enable the concrete to be placed and compacted satisfactorily.

The minimum requirements shall follow DIN 1045-1, Section 12.10.

4. Anchorage

4.1 The anchoring of reinforcing steel is to be carried out in accordance with the standards. The usual anchoring elements are: straight anchorages, U hooks, L hooks, loops, anchorage attachments and welded-on transverse bars / plates (T-headed bars).

4.2 The anchoring of pre-stressed members shall comply with the authorisation for the pre-stressing process or be verified by calculation for the stresses specified in the standards.

5. Splices in reinforcement

5.1 Tensile splices in reinforcement

Tensile splices in the reinforcement can be formed by laps with or without hooks at the bar ends, by loops, by welded-on transverse bars / plates (T-headed bars), by welding or by screwed connections.

5.2 Tensile splices in cross section

Tensile splices in cross sections subject to bending stresses are, as far as practicable, to be disposed outside the areas of fully exposed steel cross sections. The splices are to be distributed as uniformly as possible over the entire reinforcement area and shall not be laid next to each other in the longitudinal direction. Attention shall be paid to achieving adequate transverse reinforcement in the vicinity of splices.

5.3 Compressive splices

Compressive splices in reinforcing bars can be formed by laps, welding, screwed connections, contact splices.

5.4 Design of splices

The specific requirements of the standards are applicable to the design of the splices. In the case of welding splices special standards are to be used (e.g. DIN 4099).

6. Arrangement of reinforcement in flexural components

6.1 Parameters

The arrangement of the reinforcement depends on the line of tensile force, on the magnitude of the shear loading, and on the type of shear reinforcement. Reinforcing bars no longer needed to cope with tensile forces may be bent up and may be used for shear reinforcement.

6.2 End supports

Freely rotatable or only feebly restrained end supports are to have at least one third of the largest field reinforcement anchored behind the theoretical support line.

6.3 Special supports

Intermediate supports of continuous slabs and beams, end supports with attached brackets, clamped supports and frame corners are to have at least one quarter of the largest field reinforcement led behind the support forward edge. In order to absorb unanticipated loads (e.g. effect of fire, settling of supports), as far as practicable a portion of the field reinforcement is to be laid interlaced through or spliced positively (force locking).

6.4 Shear reinforcement

The shear reinforcement is to be distributed approximately in accordance with the course of the shear stresses. Stirrups acting as shear reinforcement shall go around the tensile reinforcement and, where appropriate, also the compressive reinforcement, and extend over the entire height of the cross section.

7. Components loaded in compression

In so far as reinforcing bars are used for compressive reinforcement, they are to be made safe against buckling by means of stirrups in accordance with the relevant requirements of the standards. The maximum values given in the standards for compressive reinforcement shall not be exceeded.

8. Pre-stressing steel

8.1 Surface condition

Pre-stressing steel should be clean and free of rust when installed. Pre-stressing steels with a slight film of rust may be used. (The term "slight film of rust" refers to a coating of rust that can be removed by wiping with a dry rag. However, removal of the rust in this manner need not be carried out.)

8.2 Conditions for handling

Pre-stressing steel is to be cut to length and assembled in the dry. Until installation is finished, pre-stressing members are to be stored clear of the floor in a dry place, and are to be protected against contact with harmful chemicals and moisture. Kinks or damage by machinery are to be avoided when laying out and installing pre-stressing members. Particular care is to be taken that pre-stressing steel is also protected against corrosion in the period between laying down and manufacture of the bond. In the case of short periods it may be sufficient to seal off the ends of the strands and to avoid local accumulation of moisture.

8.3 Checking before concrete is placed

Before the concrete is placed, strands or ducts are to be checked for kinks, dents or other damage; if there is any risk of impairing the tensioning process or if there are any leaks, remedial measures shall be taken.

8.4 Sealing of splices

Special attention is to be paid to the sealing of splices. It is to be ensured, during placement of the concrete, that the pipes are not damaged.

8.5 Welding

Welding onto pre-stressing steel is not allowed. Pre-stressing steel is to be protected from welding heat and falling weld metal (e.g. by means of resistant jackets).

9. Application of the pre-stress

9.1 Condition for pre-stressing

Concrete may only be pre-stressed when it is strong enough to be able to absorb the resultant stresses, including the loads at the anchorage points. This pre-requisite is regarded as fulfilled if the hardness test shows that the cube strength has reached 90 % of the nominal strength.

9.2 Earlier portion of pre-stressing

It may be expedient, in order to avoid shrinkage cracks or for early stripping of individual components, to apply a portion of the pre-stress as soon as practicable. This is only admissible if a hardness test shows that the cube strength has reached 50 % of the nominal strength according to DIN 1045-1, Table 9 and 10. In this case the partial pre-stressing forces and the concrete stresses in the rest of the component may not be more than 30 % of the admissible stresses or the admissible pre-stressing force for anchoring, respectively.

9.3 Equipment for pre-stressing

Equipment used for pre-stressing is to be tested before first use, and then generally every six months, in order to ascertain what deviations from the desired value are occurring during use. Account is to be taken of the extent to which these deviations depend on external factors (e.g. on temperature during oil pressing).

9.4 Tensioning programme

A precise tensioning programme is to be prepared and to be attached to the strength calculations. The tensioning programme shall contain, in addition to the time sequence for tensioning each member, data on the stressing force and tendon elongation with regard to compression of the concrete, friction and slip.

9.5 Pre-stressing sequence

The pre-stressing sequence is to be so selected that no inadmissible loads occur. All the measurements carried out during jacking are to be recorded in the tensioning report.

G. Principles of Calculation

1. Necessary verifications

1.1 The stability, loadbearing capacity and durability of the loadbearing structure are to be investigated for all load conditions occurring during erection and use. The following calculations are required:

For the ultimate limit states:

– strength of the loadbearing system in accordance with 2.1

– stability of the loadbearing system in accordance with 2.2

– safety of cross section against fracture in accordance with 2.3

For the serviceability limit states:

– crack width control in accordance with 3.1

– control of deflection in accordance with 3.2

1.2 Further investigations may also be required, e.g. for dynamic loads or fatigue phenomena in materials as a result of frequently alternating loads.

1.3 For prestressed concrete the checks according to H.2. are also to be carried out.

2. Ultimate limit states

2.1 Strength of the loadbearing system

2.1.1 Verification of strength

The strength of the loadbearing system and its components is to be verified by a recognized method. The cross-sectional values may be determined according to the theory of elasticity, disregarding the formation of cracks in the concrete and the effect of the reinforcement. Limit load analysis may be used with the simplification of regrouping forces and moments in the cross section relative to the distribution according to the theory of elasticity, provided equilibrium is ensured.

2.1.2 Model tests

Model tests can be used to verify the strength and distribution of cross-sectional values if they are carried out by an institution with relevant experience.

2.1.3 Avoidance of collapse

Structures in which the failure or defectiveness of one component can lead to the collapse of further components are to be avoided, see Section 3, A.2.5.

2.2 Stability of the loadbearing system

2.2.1 Analytical verification

The safety of the loadbearing system and its components against instability is to be carefully investigated. If it is not already apparent that there is sufficient stiffness and stability, an analytical verification is necessary incorporating stiffening structural components. Account is to be taken here of geometrical inaccuracies in the system and undesired eccentricities of loads.

2.2.2 Highly flexible components

Where the loadbearing or stiffening components are highly flexible, attention shall also be paid to the alterations in shape when determining the cross-sectional values (second order theory).

2.2.3 Safety against buckling

The safety of slender compressive members against buckling is to be verified in accordance with the standards.

2.3 Safety of cross sections against fracture

2.3.1 Safety margin

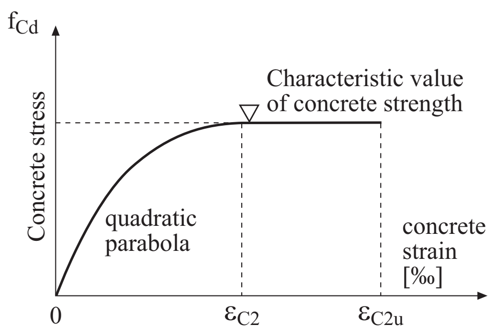
The dimensions shall ensure an adequate safety margin between the working load and the calculated ultimate load, bearing in mind the non-proportional relationship between stress and strain. There is adequate safety if the sectional values for the relevant loading condition, multiplied by the global safety factors, are not greater than the calculated ultimate loads for the section under consideration. The bending moment and longitudinal force are to be used in the most unfavourable combination and may be multiplied by the same global safety factor.

2.3.2 Strains in proportion to neutral axis

The following dimensioning rules apply to cross sections where the strains of the individual fibres are in proportion to their distance from the neutral axis. This prerequisite is regarded as satisfied if the distance between points of zero bending moment is at least twice the girder height or the cantilever length is at least equal to the girder height.

2.3.3 Stress/strain relationship

The relationship between stress and strain for concrete can be assumed to be as shown in Fig. 5.1. For steel the modulus of elasticity can be uniformly taken as 206 kN/mm² up to the yield point, beyond that point the stress remains constant. Other approximations for the stress-strain behaviour of materials are admissible but shall be agreed.



f_{Cd} = design unconfined concrete compressive strength

ε_{C2} = compressive strain in concrete on reacting limit strength

ε_{C2u} = max. compression strain in concrete

Fig. 5.1 Design values for stress and strain of concrete

2.3.4 The maximum steel strain in the cross-section shall not exceed a limiting value which may be selected at, for instance, 5 %.

2.3.5 Any accompanying action of the concrete in tension may not be taken into account.

2.3.6 The safety factor and the nominal value of the concrete strength in accordance with Fig. 5.1 shall be selected according to the acknowledged standards. However, the corresponding load conditions of the standards shall conform with those of these Rules.

2.3.7 Global safety factors

For working stress design following global safety factors can be used:

– 1,75 if failure of the section occurs with advance warning

– 2,1 if failure of the section occurs without advance warning

– 1,0 for forces due to imposed deformations

These values may be reduced by the factors given in Table 5.1, as adjustment to the loading conditions defined in Section 3, C.

Table 5.1 Reduction of safety factors for different loading conditions

Loading condition acc. to Table 3.1		Reduction factor
Number	Designation	
2	Operating loads	1,0
3	Extreme environmental loads	0,9
4	Accidental loads	0,8

2.3.8 Shear and torsion

In the case of shear force and torsion the safety margin is regarded as maintained if the stresses occurring at the working load are limited in the manner prescribed in the acknowledged standards.

3. Serviceability limit states

3.1 Limiting the crack width

3.1.1 Crack widths in concrete structures at the working loads are to be so controlled, by suitable selection of the percentage reinforcement, steel stress and bar diameter, that serviceability and durability are not affected.

Additional requirements are to be laid down for water tight or oil tight structures such as tanks.

3.1.2 The application of the acknowledged standards regarding the method of estimating the crack width, and regarding the admissible crack widths, are to be agreed in each case with GL.

3.2 Limiting the deflection

3.2.1 The extent of shape alterations of components under the working load (due, for instance, to deflections or displacements) is to be so limited that damage is prevented and serviceability is not affected.

3.2.2 The calculation can be based on a constant modulus of elasticity of the concrete for compression and tension.

3.2.3 The influence of creep and shrinkage of the concrete, and also temperature variations, are to be taken into account in determining shape alterations if they have a substantial effect.

H. Calculations for Pre-stressed Structures

1. The calculation of pre-stressed structures, depending on the pre-stressing method shall be verified according to DIN 1045-1, Section 8.7.

2. Verifications

2.1 The verifications required for reinforced concrete, see G.1., are to be carried out for pre-stressed concrete taking into account the pre-stressing of the steel.

2.2 Parameters

It is to be verified in addition that, at the working load, the stresses specified in the acknowledged standards are not exceeded. This verification may be undertaken on the assumption that the relationship between stress and strain is linear. The effects of pre-stressing, constant load, live load, temperature, creep, shrinkage, and constrainment due to settlement of the soil are also to be investigated separately, as far as practicable, and to be used to determine the most unfavourable loadings.

The condition directly after application of the pre-stress, the condition at the most unfavourable live load and with partial creep and shrinkage, and the condition with the most unfavourable live load after creep and shrinkage have ceased should in general also be verified separately.

2.3 Tensile forces

The tensile forces occurring in the concrete at the working load are to be absorbed by unstressed or by pre-stressed reinforcement, the stresses envisaged for this in the acknowledged standards being maintained. As an approximation, the calculation may be based on the uncracked condition of the cross section instead of on the cracked condition.

2.4 Pre-stressing steel

The stress-strain curve of the pre-stressing steel is to be derived from the authorization or the Certificate from the manufacturer's works, it being assumed, however, that the stress does not increase further above the yield point.

The admissible stresses are to be obtained from the recognized standards and are to be agreed with GL before use.

Section 6

Corrosion Protection

A. General

1. Scope

1.1 This Section covers corrosion protection of fixed or mobile steel and concrete structures. Design, calculation methods, material selection, fabrication, installation and commissioning of the corrosion protection system are subject to approval by GL in connection with the overall Certification procedure.

1.2 The DIN 81249 and ISO 15156 series standards shall apply to the selection of materials and the design of offshore structures and equipment components in case the technical designer has to consider the corrosion behavior of an unprotected metallic material in sea water or sea atmosphere and if this metallic material is listed in one of these standards.

1.3 The corrosion protection of fixed offshore steel structures, including platforms (jackets), tension leg platforms (TLPs) and subsea templates shall be conform to ISO 12495 or NACE RP0176 supplemented with the requirements listed below.

1.4 The corrosion protection of mobile floating structures which are static during their usual operation conditions include barges, jack-ups, semi-submersible platforms, storage tankers, buoys and appurtenances, such as chains shall be conform to ISO 13173 or NACE RP0176 supplemented with requirements below. It does not cover the cathodic protection of ships.

1.5 The corrosion protection of steel in concrete shall be conform to NORSOK M-503.

2. Terms, definitions

2.1 General

The basic terms and definitions for corrosion of metals and alloys in ISO 8044 and ISO 15156, the general principles of cathodic protection in ISO 12473 and paint systems in ISO 12944 shall be applied. For the different special structure types the terms and definitions in according with ISO 12495, 13173 and NORSOK M-503 shall be used.

2.2 Anode

Anode is an Electrode at which anodic reaction predominates. (ISO 8044)

2.3 Anodic reaction

Anodic reaction is an electrode reaction equivalent to a transfer of positive charge from the electronic conductor to the electrolyte. An anodic reaction is an oxidation process. An example common in corrosion is: $Me \rightarrow Me^{n+} + ne^{-}$ (ISO 8044)

2.4 Atmospheric zone

Zone located above the splash zone, i.e. above the level reached by the normal swell, whether the structure is moving or not. (ISO 12495)

2.5 Back e.m.f.

Back Electro **M**otive **F**orce = Voltage produced in a conductor that tends to neutralize the present voltage. The back e.m.f. is also the naturally occurring open circuit potential difference between the anode and the cathode in sea water.

2.6 Boot topping

Boot topping is the section of the hull between light and fully loaded conditions, which may be intermittently immersed. (ISO 13173)

2.7 Bracelet anode

Anode shaped as half-rings to be positioned on tubular items. Two half-ring anodes will have to fit together to become a bracelet anode. (prEN 12496)

2.8 Buried zone

Zone located under the mud line. (ISO 12495)

2.9 Calcareous deposits

Calcareous deposits are minerals precipitated on the steel cathode because of the increased alkalinity caused by cathodic polarization. Well formed calcareous deposits reduce the rate of diffusion of dissolved oxygen in the sea water to the steel surfaces and thereby reduce the current density necessary to maintain cathodic polarization. (ISO 12473)

2.10 Cathode

Cathode is an electrode at which cathodic reaction predominates. (ISO 8044)

2.11 Cathodic disbonding

Failure of adhesion between a coating and a metallic surface that is directly attributable to the application of cathodic protection. (ISO 12473)

2.12 Cathodic protection

Cathodic protection is an electrochemical protection by decreasing the corrosion potential. (ISO 8044)

2.13 Cathodic reaction

Cathodic reaction is an electrode reaction equivalent to a transfer of negative charge from the electronic conductor to the electrolyte. A cathodic reaction is a reduction process, e.g.: $Ox + ne^- \rightarrow Red$. (ISO 8044)

2.14 Coating breakdown factor

The coating breakdown factor is the anticipated reduction in cathodic current density due to the application of an electrically insulating coating when compared to that of bare steel. (ISO 12473)

2.15 Conductor pipes

Conductor pipes form the first installed casing of an offshore well. (ISO 12495)

2.16 Corrosion protection

Corrosion protection means modification of a corrosion system so that corrosion damage is reduced. (ISO 8044)

2.17 Corrosion – resistant alloy (CRA)

Alloy intended to be resistant to general and localized corrosion of oilfield environments that are corrosive to carbon steels. (ISO 15156)

2.18 Crevice corrosion

Locally more intensified corrosion in crevices. It results from corrosion cells caused by differing concentrations within the corrosive medium, especially by differences of the oxygen concentration between the crevice and the environment. As a result of the corrosion cell formation pH value reductions as well as increased chloride concentrations occur within the crevices (DIN 81249-1).

2.19 Critical crevice temperature (CCT)

The CCT is that temperature at which, in a 6% $FeCl_3$ solution, crevice corrosion first occurs (see ASTM G 48).

2.20 Critical pitting temperature (CPT)

CPT is that temperature at which, in a 6% $FeCl_3$ solution, pitting corrosion first occurs (see ASTM G 48).

2.21 Current density

Current density is the current per unit area of the electrode. (ISO 8044)

2.22 Current drain

Current drains are components which are not considered to require cathodic protection but will drain current from the system. (NORSOK M-503)

2.23 Dielectric shield

Alkali resistant organic coating applied to the structure being protected in the immediate vicinity of an impressed current anode to enhance the spread of cathodic protection and minimize the risk of hydrogen damage to the protected structure in the vicinity of the anode. (ISO 12473)

2.24 Driving potential

The driving potential is the difference between the structure/electrolyte potential and the anode/electrolyte potential. (ISO 12473)

2.25 Duplex stainless steel

Duplex stainless steel is a stainless steel whose microstructure at room temperature consists primarily of a mixture of austenite (Face-centred cubic crystalline phase of iron-based alloys) and ferrite (Body-centred cubic crystalline phase of iron-based alloys). (ISO 15156).

2.26 Extended tidal zone

Extended tidal zone is a zone including the tidal zone, the splash and the transition zone. (ISO 12495)

2.27 Ferritic steel

Ferritic steel is steel whose microstructure at room temperature consists predominantly of ferrite (body-centred cubic crystalline phase of iron-based alloys). (ISO 15156)

2.28 Flush mounted anode

Flush mounted anodes are designed to limit hydrodynamic effects when fitted to the structure. (prEN 12496)

2.29 Galvanically – induced hydrogen stress – cracking (GHSC)

Cracking that results due to the presence of hydrogen in a metal, induced in the cathode of a galvanic couple, and tensile stress (residual and/or applied). (ISO 15156)

2.30 Galvanic protection

Galvanic protection is an electrochemical protection in which the protecting current is obtained from a corrosion cell formed by connecting an auxiliary electrode to the metal to be protected. (ISO 8044)

2.31 H.A.T.

Level of the highest astronomical tide. (ISO 12495)

2.32 Hydrogen (induced) stress cracking (HSC, also known as HISC)

Cracking that results from the presence of hydrogen in a metal and tensile stress (residual and/or applied).

HSC describes cracking in metals that are not sensitive to SSC (Stress Corrosion Cracking) but which may be embrittled by hydrogen when galvanically coupled, as the cathode, to another metal that is corroding actively as an anode. The term galvanically induced HSC has been used for this mechanism of cracking. (ISO 15156)

2.33 Holiday

As holiday a coating discontinuity is understood that exhibits electrical conductivity when exposed to a specific voltage. (ISO 21809-1)

2.34 Immersed zone

Zone located below the extended tidal zone and above the mud line. In case of floating structures, zone located below the waterline at draught corresponding to normal working conditions. (ISO 12495)

2.35 Impressed current protection

Impressed current protection is an electrochemical protection in which the protection current is supplied by an external source of electric energy. (ISO 8044)

2.36 Insert

Insert is the form over which the anode is cast and which is used to connect the anode to the structure requiring protection (prEN 12496).

2.37 J-tube

Curved tubular conduit designed and installed on a structure to support and guide one or more pipeline risers or cables. (ISO 12495)

2.38 L.A.T.

L.A.T. is the level of the lowest astronomical tide. (ISO 12495)

2.39 Marine sediments

Top layer of the sea bed composed of water saturated solid materials of various densities. (ISO 12495)

2.40 M.T.L.

Mean tide level (also known as M.S.L or M.W.L.). (ISO 12495)

2.41 Pitting corrosion

Type of corrosion with the anodic formed on the material surface being considerably smaller than the cathodic regions. Pitting corrosion frequently occurs if the protective layer of a metal is locally damaged or if the passive layer of a stainless steel is penetrated e.g. by chloride ions (DIN 81249-1).

2.42 Pitting resistance equivalent number (PREN)

PREN is a number, developed to reflect and predict the pitting resistance of a CRA, based upon the proportions of Cr, Mo, W and N in the chemical composition of the alloy.

PREN = % Cr + 3,3 % Mo +16 % N (ISO 15156, 12473)

2.43 Protection current density

Current protection density is the density that is required to maintain the corrosion potential in a protection potential range. (ISO 8044)

2.44 Reference electrode

Reference electrode is an electrode, having a stable and reproducible potential that is used as a reference in the measurement of electrode potentials. (ISO 8044)

2.45 Resistivity of an electrolyte

Resistivity is the resistance of an electrolyte of unit cross section and unit length. It is expressed in ohm · metres ($\Omega \cdot$ m). The resistivity depends, amongst other things, upon the amount of dissolved salts in the electrolyte. (ISO 12473)

2.46 Riser

Vertical or near vertical portion of an offshore pipeline between the platform piping and the pipeline at or below the seabed, including a length of pipe of at least five pipe diameters beyond the bottom elbow, bend or fitting. (ISO 12495)

2.47 Salinity

Amount of inorganic salts dissolved in the sea water. The standardized measurement is based on the determination of the electrical conductivity of the sea water. Salinity is expressed in grammes per kilogramme or in ppt. (ISO 12495)

2.48 Splash zone

Height of the structure which is intermittently wet and dry due to the wave action just above the H.A.T. (ISO 12495)

2.49 Submerged zone

The submerged zone is a zone including the buried zone, the immersed zone and the transition zone. In case of floating structures, the zone includes the immersed and the buried zones. (ISO 12495)

2.50 Stand off anode

Stand off anodes are designed to be off set a certain distance from the object on which they are positioned. The distance is typically 0,3 metre. (prEN 12496)

2.51 Tidal zone

Zone located between the L.A.T. and the H.A.T. (ISO 12495)

2.52 Transition zone

Zone located below the L.A.T. and including the possible level inaccuracy of the platform installation and a depth with a usually higher oxygen content due to the normal swell. (ISO 12495)

2.53 Underwater hull

Underwater hull is the part of the hull vital for its stability and buoyancy of a floating structure, i.e. below the light water line. (ISO 13173)

3. Structural design

3.1 Design parameters

Some factors that may influence the design of a corrosion protection system shall be outlined. More detailed information is given in each of the referred Standards in A.1. and A.2.1. When designing a corrosion protection system, it is important to ensure that the whole structure is adequately protected and that e.g. areas are not substantially over polarized, uniform distribution of current and restriction are placed on the sitting of the anodes (e.g. in way of nodes) and it may be advantageous to use coatings. The detailed design of offshore structures shall include corrosion protection. This shall involve:

– type of production installation
– material selection
– specification of coating
– cathodic protection (sacrificial or impressed current)
– selection of anode material and type
– reliability
– maintenance
– monitoring

The major parameters affecting corrosion protection are:

– dissolved oxygen content
– wind
– earthquake
– vibrations
– waves
– currents
– sand-, ice-loads, formation and accretion
– temperature
– marine growth
– salinity

B. Material Selection

1. General

The intensity of corrosion of an unprotected steel structure in seawater varies markedly with position relative to the sea levels as shown in Fig. 6.1. The splash zone above the mean tide level (M.T.L) is the most severely attacked region due to continuous contact with highly aerated sea water and the erosive effects of spray, waves and tidal actions. Corrosion rates as high as 0,9 mm/y at Cook Inlet, Alaska, and 1.4 mm/y in the Gulf of Mexico have been reported. Cathodic protection in this area is ineffective because of lack of continuous contact with the seawater, the electrolyte, and thus no current flows for much of the time. Corrosion rates of bare steel are often also very high at a position just below M.T.L in a region that is very anodic relative to the tidal zone, due to powerful differential aeration cells which form in the well aerated tidal region.

Protection of a steel structure can be achieved by various means; each corrosion zone being separately considered:

– atmospheric zone
– extended tidal zone
– immersed zone
– buried zone

Three generally accepted methods are

– cathodic protection
– painting or coating
– and sheathing

Corrosion in the atmospheric zone is typically controlled by the application of a protective coating system.

For the extended tidal zone, with the highest damage/corrosion rates NACE RP0176 provide a list of Corrosion Control Measures, e.g. additional Corrosion Allowance, Steel Wear Plates, Alloy-clad (65Ni-Cu alloy 400 or 90/10 Cu-Ni alloy) and diverse coatings, like Vulcanized Chloroprene (thickness of 6 to 13 mm), High-Build Organic Coatings filled with silica glass-flake or fiberglass (thickness of 1 to 5 mm) or Thermal-sprayed Aluminum.

In the past, conventional protective coatings were seldom applied to structure in the submerged zone. However, increased current requirement and anode weight restriction can affect the decision to coat complex structures to be installed in deeper waters with higher current density requirements, in shielded areas as large conductor bundles and/or on structures with extended design lives.

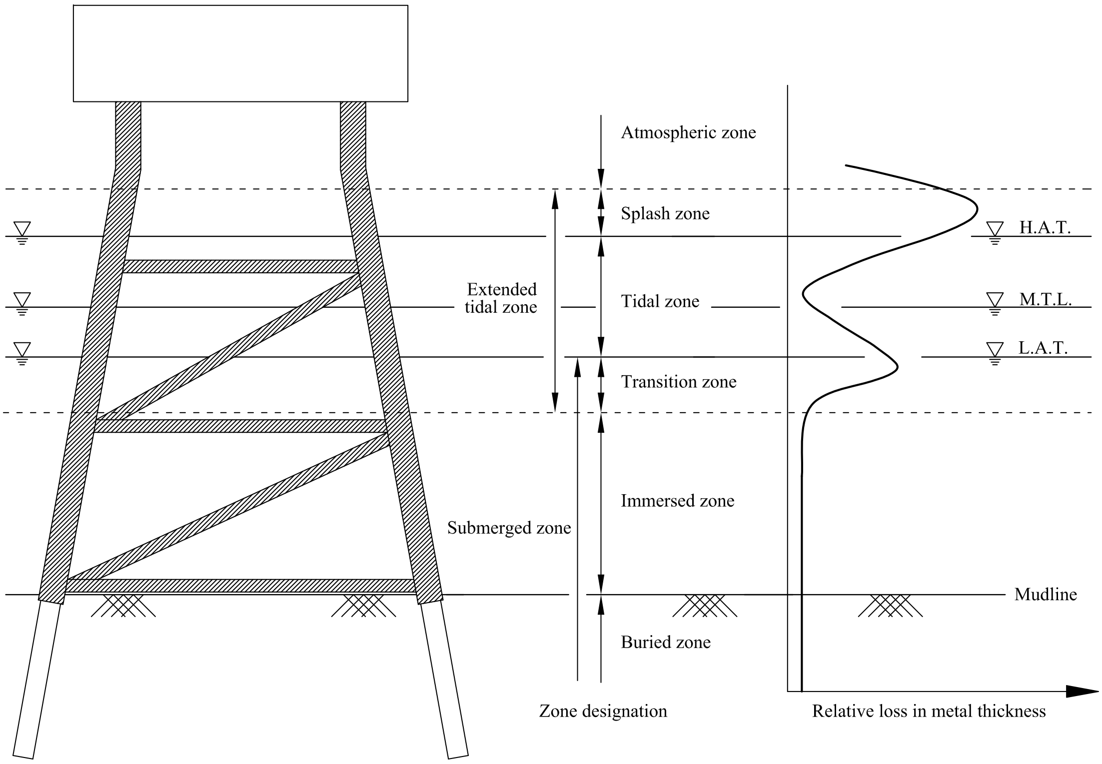


Fig. 6.1 Schematic representation of levels, zones and thickness loss in sea water environment

This clause outlines some of the more important metals which may be protected by cathodic protection in sea water and gives guidance for

– metal sheathing of extended tidal zone with 65%Ni-Cu alloy 400 and 90-10 Cu-Ni

– Critical Pitting and Crevice Temperature (CPT and CCT) at which the CRAs are susceptible to chlorides in seawater

– potentials recommended for protection of metals

However, in view of the wide range of alloys and applications it is essential that these values are used for general guidance only and for more specific recommendations the standards of A.1, A.2.1 and literature review may be necessary.

2. Metal sheathing

Metal sheathing with 65%Ni-Cu alloy 400 and 90-10 Cu-Ni has proved to be a very successful approach when applied to the legs and risers in the extended tidal zone.

2.1 65Ni-Cu (Alloy 400) is prone to pitting under stagnant conditions (0-0,6 m/sec) but becomes passive in flowing seawater and high resistance even at 40 m/sec.

In early trials of 65Ni-Cu (Alloy 400) welded directly to the steel, it was assumed that corrosion of the anodic steel below the tidal zone would be accelerated because it is in direct contact with the more noble sheathing alloy. On the contrary, steel below the tidal zone was found to be cathodic relative to the noble alloy sheathing material, since the sheathing alloy became polarized to the potential of the adjacent steel below. Hence the submerged steel below the sheathed pilling corroded at a lower rate than the submerged steel on unsheathed bare steel because the resulting galvanic current between the sheathed tidal zone and the submerged steel below it is lower.

2.2 90-10 Cu-Ni is an established alloy for seawater systems and recognized for its unique combination of high resistance to corrosion and macrofouling.

Maximum resistance to biofouling relies on 90-10 Cu-Ni being freely exposed and not galvanically or cathodically protected by less noble materials. It is thought that this allows the availability of free copper ions in the surface film to inhibit the growth of macrofouling although some microfouling will colonize. Attachment of the sheathing material to the steel structure by welding or mechanical fasteners will result in cathodic polarization of the sheath material and a reduction in the antifouling capability of the 90-10 Cu-Ni- alloy. Therefore it is necessary to electrically insulate the sheath from the steel jacket members to gain the full advantage of the biofouling resistance properties of the alloy. Electrical insulation can be achieved by pumping cement or an epoxy into the annular space between the component and the sheath or, more simply, by use of an elastomer or rubber-base insulator.

2.3 Long term data has shown complete protection of the steel behind the sheathing. The corrosion rate of the 65Ni-Cu (Alloy 400) and 90-10 Cu-Ni was very low and uniform; after 10 years no measurable loss of thickness of the sheathing itself in the case of the directly welded and insulated steel. Coating should be applied and maintained on the upper steel/65Ni-Cu or steel/90-10 Cu-Ni interface and any steel area above in the atmospheric zone. The accumulation of biofouling on insulated Cu-Ni was 1-4% of the bare steel. In the case of directly welded sheathing, there were over 50 and 60% reduction in the biofouling mass after 5 and 10 years exposure respectively representing a significant reduction in structure weight and wave loading (see also ISO 12473 Chapter 8.3.2.3 Cooper alloys).

3. High strength steel and corrosion resistant alloys (CRAs)

3.1 For deep water developments, high pressure, high temperature, sour gas and condensate fields there is a need for high strength and corrosion resistant alloys (CRAs). Although CRAs are considered to be immune from corrosion, they are really only resistant to general corrosion in oil and gas production fluids compared to carbon and low alloy steels. Internal corrosion problems in steels are essentially related to corrosion by CO_2 and H_2S. ISO 15156 Part 3 lists the CRAs that are acceptable and the conditions for which they may be used in terms of pH, H_2S partial pressure and chloride levels (see also Chapter 5, Section 15). External corrosion problems increases at the anode with

– an increasing free corrosion potential difference

– an increasing cathodic/anodic area ration

– an increasing specific electrical conductivity and an increasing temperature of the sea water

The types of corrosion attack depend upon the material used, the component design, and the operating conditions. Uniform corrosion and shallow pit formation usually occur on unalloyed steels and low- alloy steels. Pitting and crevice corrosion generally occurs on materials that are passivatable and tend to form a surface layer (high-alloy steels, aluminum and aluminum alloys as well as copper and copper alloys).

3.2 The pitting resistance equivalent number (PREN) is a measure of the pitting corrosion resistance of stainless steels as well as chrome and molybdenum nickel-base alloys. The larger the value of the PREN, the better the resistance to pitting and crevice corrosion in seawater. Using the following formula, various materials can be ranked based upon their chemistry.

$$PREN = \%Cr + 3.3 \cdot \%Mo + 16 \cdot \%N$$

Crevice corrosion is very similar to pitting corrosion. However, since the tighter crevice allows higher concentration of corrosion products, it is more insidious than pitting. This drives the pH value lower. The end result is that crevice corrosion can happen at temperatures 30° –50°C lower than pitting in the same environment.

The Tables 6.1 to 6.3 present some of the more important metals used for subsea collecting systems.

Table 6.1 Reference materials

UNS, M-No.	Standard	Grade, class	Material-types
S32205	ASTM A 240	2205	Duplex 22Cr
1.4462	EN 10028-7	X2CrNiMoN 17-12-2	Duplex 22Cr
S32750	ASTM A 240	2207	Super-Duplex 25Cr
1.4410	EN 10028-7	X2CrNiMoN 25-7-4	Super-Duplex 25Cr
S30400	ASTM A 240	304	Austenic SS
1.4301	EN 10028-7	X5CrNi 18-10	Austenitic SS
S31603	ASTM A 240	316 L	Austenitic SS
1.4404	EN 10028-7	X2CrNiMo 17-12-2	Austenitic SS
N06625		625	Nickel-base- Alloy
2.4856	DIN 17744	NiCr22Mo9Nb	Nickel-base- Alloy
N08825		825	Nickel-base- Alloy
2.4858	DIN 17744	NiCr21Mo	Nickel-base- Alloy

Table 6.2 Chemical composition of the reference materials

UNS M-No.	Weight, %, max.								
	C	Mn	Si	P	S	Cr	Ni	Mo	Fe	others
S32205	0,03	2,0	1,0	0,03	0,02	22,0-23,0	4,5-6,5	3,0-3,5	Rest	N=0,1
1.4462	0,03	2,0	1,0	0,035	0,015	21,0-23,0	4,5-6,5	2,5-3,5	Rest	N=0,1
S32750	0,03	1,2	0,8	0,035	0,02	24,0-26,0	6,0-8,0	3,0-5,0	Rest	N=0,2
1.4410	0,03	2,0	1,0	0,035	0,015	24,0-26,0	6,0-8,0	3,0-4,5	Rest	N=0,2
S30400	0,08	2,0	0,7	0,045	0,03	18,0-20,0	8,0-10,5	-	Rest	N=0,1
1.4301	0,07	2,0	1,0	0,045	0,015	17,0-19,5	8,0-10,5	-	Rest	N=0,1
S31603	0,03	2,0	0,7	0,045	0,03	16,0-18,0	10-14	2,0-3,0	Rest	N=0,1
1.4404	0,03	2,0	1,0	0,045	0,015	16,5-18,5	10-13	2,0-2,5	Rest	N=0,1
N06625	0,1	0,5	0,5	0,015	0,015	20,0-23,0	Rest	8,0-10,0	5,0	Nb4,1
2.4856	0,1	0,5	0,5	0,02	0,015	20,0-23,0	58	8,0-10,0	5,0	Nb,Co
N08825	0,05	1,0	0,5	-	0,03	19,5-23,5	38-46	2,5-3,5	Rest	Ti=1,2
2.4858	0,02	1,0	0,5	0,02	0,015	19,5-23,5	38-46	2,5-3,5	Rest	Co,Cu

Table 6.3 Mechanical properties, PREN, CPT and CCT

UNS M-No.	$R_{0,2}$ MPa min.	R_m MPa	A_5 %	PREN	CPT °C [1, 2]	CCT °C [1, 2]
S32205	450	640	25	31-37	~30	~20
1.4462	460	640	25	31-38	~30	~20
S32750	550	800	20	38-44	~50-60	~25-35
1.4410	530	730	20	38-46	~50-60	~25-35
S30400	210	515-690	45	20-22	< 0	< 0
1.4301	210	520-720	45	19-21	< 0	< 0
S31603	220	515-690	45	18-20	< 0	< 0
1.4404	220	530-680	40	18-20	< 0	< 0
N06625	410	800-1000	30	46-56	~60-90	~40-60
2.4856	410	800	30	46-56	~60-90	~40-60
N08825	240	590-750	30	28-35	~20-40	< 0-15
2.4858	235	550	30	28-30	~20-25	< 0

Notes

1 DIN 81249

2 Klöwer, J., Schlerkmann, H., Pöpperling, R.: H_2S Resistant Materials for Oil & Gas Production. Paper No. 01004-2001 by NACE International

3.3 The potential susceptibility of some alloys used for jumpers, manifolds, valves, etc. to HISC due to polarization from cathodic protection of carbon steel subsea flowlines or subsea structures shall be considered. Within the potential range of cathodic protection by e.g. Al-anodes (i.e. –0,80 to –1.10V Ag/AgCl/ seawater) the production of hydrogen increases exponentially towards the negative potential limits. Forgings, casts and welds are more prone to HISC than wrought materials due to the course microstructure allowing HISC to propagate.

The potential susceptibility of HISC of ferritic and ferritic- pearlitic structural steels with specified minimum yield strength above 700 MPa or 300/350 HV, ferritic-austenitic Duplex stainless steels and certain precipitation hardening nickel based alloys with hardness above 300/350 HV due to polarization from cathodic protection shall be considered. Whilst this is the subject of current research, guidance will be provided in Table 6.4 (see also ISO 12473, Chapter 8.3, Other metallic materials).

Table 6.4 **Summary of potential versus Ag/AgCl/ sea water reference electrode recommended for the cathodic protection of various metals in sea water**

Material	**Min. negative potential [Volt]**	**Max. negative potential [Volt]**
High strength steel: R_{eH} ≤ 700 MPa and ≤350 HV: Aerobic environment Anaerobic environment	 –0,80 –0,90	 –1,10 –1,10
High strength steel: R_{eH} > 700 MPa or >350 HV:	–0,80	–0,95 [1]
Stainless steel: Austenitic steel: PREN ≥ 40 Austenitic steel: PREN < 40 Ferritic-austenitic Duplex:	 –0,30 –0,60 [2] –0,60 [2]	 No limit No limit –0,90 [3, 1]
Al Mg and Al Mg Si alloys	–0,80	–1,10
Copper alloys without Al Copper alloys with Al	–0,45 to –0,60 –0,45 to –0,60	No limit –1,10
Nickel base alloys	–0,20	4

Notes

1 For materials susceptible to HISC the simultaneous presence of at least two conditions is needed:
- more negative than –0,83 V
- high level of strain in the material (> 80% of R_{eH} for global and local stress; 67% of R_{eH} for regions influenced by residual stresses)

2 For most applications these potentials are adequate for the protection of crevices although higher potentials may be considered.

3 Forgings, casts and welds are more prone to HISC than wrought materials due to the course microstructure allowing HISC to propagate preferentially in the ferritic phase.

4 High strength nickel copper and nickel chromium iron alloys can be susceptible to HISC and potentials that result in significant hydrogen evolution should be avoided.

R_{eH} = minimum specified yield strength

C. Coating Selection

1. General

1.1 The most suitable form of coating depends on the type of structure and its environment. In the selection of a coating, the aim should be to achieve overall economy in the combined cost of the protected structure and of the initial and running costs of the protection schemes. Due regard should be paid to the design life of the structure and the economics of repairing the coating should this become necessary. More information is given in the respective Standards, see A.1.

1.2 While organic coatings are not entirely impermeable to oxygen and water they do restrict corrosion when applied to the surface of a metal. The bulk of the corrosion on a painted surface does not occur beneath the intact coating but at the base of holidays and pin holes. If cathodic protection is applied to a painted surface, the coating acts as a substantial resistive barrier to current flow and where the current flows, it is to the holidays or pin holes. In terms of cathodic protection the presence of a coating improves current distribution and reduces current demand and interference effects.

1.3 Coating on offshore structures can be divided into four major areas:

– atmospheric zone

– extended tidal zone

– immersed zone

– buried zone

Fig. 6.1 presents the levels, zones and profile of the thickness loss resulting from corrosion of an unprotected steel structure in seawater.

Corrosion in the atmospheric zone is typically controlled by the application of a protective coating system (see NACE RP0176 - Table 3 and ISO 12944 - Corrosion Category C5-M). Table 6.5 illustrates typical coating systems used in the atmospheric zone for a design life of more than 15 years.

For the extended tidal zone, with the highest damage/corrosion rates (until 100 times higher than in the atmospheric zone and three times higher than in the immersed zone) NACE RP0176 provide diverse coatings, like Vulcanized Chloroprene (thickness of 6 to 13 mm), High-Build Organic Coatings filled with silica glass-flake or fiberglass (thickness of 1 to 5 mm) or Thermal-sprayed Aluminum. ISO 12944 provide coating systems for sea and brackish water (Corrosion Category Im2).

In the past, conventional protective coatings were seldom applied to structure in the immersed and buried zone. However, increased current requirement and anode weight restriction can affect the decision to coat complex structures to be installed in deeper waters with higher current density requirements, in shielded areas as large conductor bundles and/or on structures with extended design lives.

Table 6.6 illustrates typical coating systems used in the extended tidal and immersed and buried zone.

1.4 Organic coatings promote the formation of a dense calcareous deposit at coating holidays and bare areas because the initial current density may be relatively high at such locations. However, the solubility of potential film- forming calcareous deposits normally increases with decreasing temperature such that colder waters might not allow the formation of a protective calcareous deposit or could require higher initial current density to achieve polarization. This includes deep water applications for which the formation of calcareous deposits may be slow.

1.5 The application of coatings may not be suitable for parts of submerged structures requiring frequent inspection for fatigue cracks, e.g. critical welded nodes of jacket structures**.**

For components of materials sensitive to HISC by CP (see B.3.), an organic coating should always be considered as a barrier to hydrogen adsorption.

2. Coatings and coating breakdown factors

2.1 Coating breakdown factor is the anticipated reduction in cathodic current density due to the application of an electrically insulating coating when compared to that of bare steel. If the Coating breakdown factor is zero, the coating is 100% electrically insulating, thus decreasing the cathodic current density to zero. Coating breakdown factor 1 means that the coating has no current reducing properties.

An initial coating breakdown factor (fci) related mainly to mechanical damage occurring during the installation of the structure should be considered and a coating deterioration rate shall be thereafter evaluated in order to take into account the coating aging and possible small mechanical damage occurring to the coating during the structure life.

2.2 Due to possible interactions between the cathodic protection and the coating, all coatings to be used in combination with cathodic protection shall be tested beforehand to establish that they have adequate resistance to cathodic disbondment. Alkali is generated on the cathodically protected surface and this may result in the cathodic disbondment of the coating in way of any coating defects.

2.3 The conventional coatings of the oleoresinous or alkyd types are attacked by alkali, i.e. they are subject to saponification and are not recommended for use in association with cathodic protection. The use of polyvinyl butyral shop primers has also caused loss of adhesion when combined with cathodic protection.

The Coating Categories I to VII of Table 6.6 und the recommended constants k_1 and k_2 of Table 6.7 are based on recommended values of ISO 12495, 13173, 15589 and NORSOK M-503 as well the acceptance criteria of the coating standards ISO 12944, inclusive ISO 20340 and ISO 21809-1.

Table 6.5 Typical coating systems used in the atmospheric zone (design life >15 years)

Coating system No. [1]	Primer coats				Intermediate/top coats			Total NDFT [µm]
	Binder	Type	No. of coats	NDFT [µm]	Binder	No. of coats	NDFT [µm]	
S 7.04	EP, PUR	div.	1-2	80	EP,PUR	3-4	240	320
S 7.06	EP, PUR	Zinc-R	1	250	EP,PUR	1	250	500
S 7.09	EP, PUR	Zinc-R	1	40	EP,PUR	3-4	280	320
S 7.10	EP, PUR	Zinc-R	1	40	CTV	3	360	400
S 7.11	EP, PUR	Zinc-R	1	40	CTE	3	360	400
S 7.14	ESI	Zinc-R	1	80	EP,PUR	2-4	240	320

Notes
1 = ISO-12944-5, Corrosion Cat. C5-M
CTE = Epoxy Tar
CTV = Vinyl Tar Qualification tests ISO 20340-para 8.2
EP = Epoxy
ESI = Ethylsilicate
NDFT = Nominal Dry Film Thickness
PUR = Polyurethane
Zinc-R = Zinc-rich

Table 6.6 Typical coating systems used in the extended tidal, immersed and buried zone

Coating system No.	Primer coat				Intermediate/top coats			Total NDFT [µm]	fcf	Coating category
	Binder	Primer	No. of coats	NDFT [µm]	Binder	No. of coats	NDFT [µm]			
S8.03mod[8]	EP	div.	1	20					1.0	I
S8.01mod[8]	EP,PUR	zinc-R	1	40	EP,PUR	1-3	210	250	0.8	II
S8.04mod[8]	EP	div.	1	80	EP,PUR	1-2	170	250	0.8	II
S8.01mod[8]	EP,PUR	zinc-R	1	40	EP,PUR	2-3	260	300	0.5	III
S8.03mod[8]	EP	div.	1	40	CTE	3	320	360	0.5	III
S8.04mod[8]	EP	div.	1	80	EP,PUR	2	240	320	0.5	III
S8.07mod[8]	CTE	div.	1	120	CTE	2	180	300	0.5	III
S8.11mod[8]	CTPUR	div.	1	170	CTPUR	1	170	340	0.5	III
S8.02[7]	EP,PUR	zinc-R	1	40	CTPUR	4	500	540	0.32	IV
S8.05[7]	EP	div.	1	80	EP	1	400	480	0.32	IV
S8.06[1, 7]	EP	div.	1	800	–	–	–	800	0.32	IV
S8.08[7]	CTE	div.	1	120	CTE	3	380	500	0.32	IV
S8.09[7]	CTE	div.	1	500	–	–	–	500	0.32	IV
S8.10[1, 7]	CTE	div.	1	1000	–	–	–	1000	0.32	IV
3LPE-Class A1/B1[9]	EP[2 or 3]	div.	1	25[2]or 125[3]	Adhesive[4] 150µm + PE[5]	2	–	1800-3200[6)]	0,16	V
3LPP-Class C1[9]	EP[2 or 3]	div.	1	25[2]or 125[3]	Adhesive[4] 150µm + PP[5]	2	–	1300-2500[6]	0,1	VI
Rubber[10]								6000	0,07	VII

Notes:

1 solvent free
2 liquid
3 powder (FBE)
4 powder sprayed or extruded
5 applied by extrusion
6 thickness is function of pipe weight
7 ISO 12944, Qualification tests ISO 20340-para 8.2
8 ISO 12944 modified: reduced intermediate/top coat thickness, Qualification tests ISO 20340-para 8.2
9 ISO 21809-1
10 Vulcanized chloroprene (NACE RP0176)

CTE = Epoxy Tar
CTPUR = Polyurethane Tar
EP = Epoxy
FBE = Fusion Bond Epoxy
fcf = Final cathodic breakdown factor for design life of 30 years
NDFT = Nominal Dry Film Thickness
PE = Polyethylene
PP = Polypropylene
PUR = Polyurethane
Zinc-R = Zinc-rich
3LPE = Three layer polyethylene coating
3LPP = Three layer polypropylene coating

Table 6.7 Recommended constants k_1 and k_2 for calculation of cathodic breakdown factors

Zones (Fig.1)	Constants	Coating categories (Table 3)						
		I	II	III	IV	V	VI	VII
Extended	k_1	0,10	0,05	0,05	0,02	0,01	0,01	0,005
Tidal	k_2	0,10	0,025	0,015	0,01	0,005	0,003	0,002
Immersed	k_1	0,10	0,05	0,02	0,01	0,01	0,01	0,005
	k_2	0,05	0,015	0,015	0,01	0,003	0,002	0,0015

2.4 Calculation of cathodic breakdown factors:

$$fci = k_1 \quad (1)$$

$$fcm = k_1 + k_2 \text{ x } t/2 \quad (2)$$

$$fcf = k_1 + k_2 \text{ x } t \quad (3)$$

If fcm or fcf exceeds 1, fcm = 1 or fcf = 1 shall be applied in the design. (Coating breakdown factor 1 means that the coating has no current reducing properties.)

k_1 = constant for the initial condition of coating depending mainly on mechanical damage occurring during the installation of the structure mechanical damage

k_2 = constant for coating damage depending mainly on marine growth (incl. cleaning operations to remove such growth) and the erosion effects of waves and currents

t = design life (years)

fci = initial cathodic breakdown factor

fcm = mean (or maintenance or average) cathodic breakdown factor

fcf = final (or re-polarization) cathodic breakdown factor

D. Cathodic Protection

1. General

In order to achieve the cathodic protection criteria on the whole structure it is necessary to consider the electrical current demand on each part of the structure.

The electrical current demand of each part of the structure is the product of its steel surface area multiplied by the electrical current density required.

The current density required is not the same for all parts of the structure as the environmental conditions are variable. Therefore, the following areas and parts should be considered, referring to zones as defined in Fig. 6.1.

– areas located in the tidal and transition zone (usually coated or cladded)

– areas located in the immersed zone

– areas located in the buried zone

– wells to be drilled; a current allowance per well shall be considered, depending on projected size, depth and cementing of the wells

– Neighboring structures and pipelines in electrical contact with the fixed steel offshore structure to be protected.

2. Design current densities

The selection of design current densities may be based on experiences from similar structures in the same environment or from specific tests and measurements.

The electrical current density required for cathodic protection depends upon the kinetics of the electrochemical reactions and varies with parameters such as listed in A.3.1.

For each particular set of environmental condition (see Table 6.8) and surface condition of the structure (see Table 6.6, 6.7), the following electrical current densities shall be evaluated:

– initial electrical current density required to achieve the initial polarization of the structure, i.e. to achieve the lowering of the steel potential down to value within the range recommended in Table 6.4

– mean (or maintenance or average) electrical current density required to maintain this polarization level on the structure

– final (or re-polarization) electrical current density required for a possible re-polarization (i.e. for re-establishing the potential to the initial polarization level) of the structure after severe storms or cleaning operations (i.e. marine growth)

Table 6.8 presents a general guide to the design of CP systems in major offshore petroleum producing areas for steel, stainless steel, aluminum and other metallic materials. These data may be used as a starting point for investigation prior to selection of final design parameters for a specific application. For special cases and deep water Table 6.9 and 6.10 shall be used.

Table 6.8 Typical design current density values of bare metal surface in immersed zone until 100m water depth

Current density levels	Code [1]	Geographic area	Water temp. [2]	Wave action [3]	Lateral Water Flow	Design current density [mA/m²]		
						Initial	Mean	Final
Very high	NACE	Cook Inlet	2	Low	High	430	380	380
High	ISO	North Sea > 62 °N	–1 - 4	High	Med.	220	100	130
	NACE	North Sea 55 - 62 °N	0 - 12	High	Med.	180	90	120
	NACE	Brazil	15 - 20	Med.	High	180	65	90
Med.	NACE	North Sea < 55 °N	0 - 12	High	Med.	150	90	100
	NACE	U.S. West coast	15	Med.	Med.	150	90	100
Low	NACE	Arabian Gulf	30	Med.	Low	130	65	90
	NACE	Australia	12 - 18	High	Med.	130	90	90
	NACE	West Africa	5 - 21	Low	Low	130	65	90
	ISO	India				130	70	90
Very low	NACE	Gulf of Mexico	22	Med.	Med.	110	55	75
	NACE	Indonesia	24	Med.	Med.	110	55	75
	NACE	South China Sea	30	Low	Low	100	35	35
	ISO	Mediterranean Sea				110	60	80
	ISO	Adriatic Sea				110	60	80

Notes
1 NACE Standard RP0176 and ISO 12495 or 13173
2 Surface (sea level) water temperature
3 Turbulence Factor

3. Cathodic protection systems

Cathodic protection can be achieved using:

– the galvanic (or sacrificial) anodes system

– the impressed current system

– a combination of both cathodic protection systems (Hybrid Systems)

For permanently installed offshore structures, galvanic anodes are usually preferred.

However, due to weight and drag forces caused by galvanic anodes, impressed current cathodic protection systems are sometimes chosen for permanently installed floating structures.

When using hybrid systems the galvanic anodes should provide cathodic protection during float out, initial installation and subsequently during the impressed current systems shutdowns.

Galvanic anodes should also be installed in areas where it may be difficult to achieve adequate level of polarization by the impressed current system due to shielding effects.

As the electrical current demand is not constant with time, the cathodic protection system shall be able to deliver the re- polarization current density required for short periods throughout the life of the structure.

4. Galvanic anodes system

4.1 Design considerations

The three design electrical current densities as defined in 2. shall be considered:

– The initial electrical current density shall be used to verify that the output current capacity of new anodes, i.e. their initial dimensions, is adequate to obtain a complete initial polarization of the structure within a few weeks.

– The mean (or maintenance or average) electrical current density shall be used to determine the weight of the anodes. This current density is required to maintain an adequate polarization level of the structure during its design life.

– The final (or re-polarization) electrical current density shall be used to verify that the output current capacity of the anodes when they are consumed to an extent commensurate with their utilization factor, i.e. their final usable dimensions, is adequate to re-polarize the structure after severe storms or after marine growth cleaning operations.

Table 6.9 Typical design current density values for special cases

Case No.	Code [4]	Description	Design Current Density [mA/m^2]		
			Initial	Mean	Final
1	ISO	Bare metal in extended tidal zone	1,2 × Table 6.8	1,2 × Table 6.8	1,2 × Table 6.8
2	NACE	Deep-water structures depth > 100 m	Table 6.10	Table 6.10	Table 6.10
3	ISO	Bare metal in marine sediment	25	20	20
4	NACE	Freely flooded compartments	Table 6.8	Table 6.8	Table 6.8
5	NACE	Closed compartments with free access to air	Table 6.8	Table 6.8	Table 6.8
6	NACE	Closed and sealed flooded compartments	0	0	0
7	NORSOK	Current Drain by mud mats, skirt and piles [1]	20	20	20
8	NORSOK	Open pile ends [2]	Table 6.8	Table 6.8	Table 6.8
9	ISO	Current Drain per platform wells	5000	5000	5000
10	NORSOK	Current Drain per subsea wells	8000	8000	8000
11	NORSOK	Current Drain to anchor chain	Table 6.8 and 6.9 [3]	Table 6.8 and 6.9 [3]	Table 6.8 and 6.9 [3]
12	NORSOK	For components heated by an internal fluid for each °C exceed 25°C, additional to base values	1	1	1
13	NORSOK	For embedded steel in concrete structures exposed on one side in immersed zone	2	2	2
14	NORSOK	For embedded steel in concrete structures exposed on both sides in immersed zone	1	1	1
15	NORSOK	For embedded steel in concrete structures exposed on one side in extended tidal zone	3	3	3
16	NORSOK	For embedded steel in concrete structures exposed on both sides in extended tidal zone	1,5	1,5	1,5
17		For Al- and Zn- components or coated with Al or Zn	10	10	10
18		For Al- and Zn- components or coated with Al or Zn and heated by an internal fluid for each °C exceed 25°C, additional to case 17	2	2	2

Notes:

1 based on the outer (sediment exposed) external surface area

2 The top internal surface shall be included for a distance of 5 times the internal diameter

3 Current drain to anchor chains shall be accounted for by 30 m for systems with mooring point topside only. For systems with mooring point below the water level, the seawater exposed section above this point shall also be included. A current drain of 30 m shall also be included for cathodic protection of anchoring arrangements using chains.

4 NACE Standard RP0176, ISO 12495 or 13173, and NORSOK M-503

Table 6.10 Typical design current density values of bare metal surface in immersed zone > 100m water depth (NACE RP0176) [1]

Temperature °C	Design Current Density [mA/m^2]		
	Initial	Mean	Final
4 °C	350	140	300
5 °C	300	85	250
6 °C	250	75	200
7 °C	210	65	160
8 °C	170	60	120
9 °C	140	55	90
10 °C	120	52	70
11 °C	115	48	65
12 °C to 17 °C	110	45 to 40	60
18 °C to 23 °C	100	40 to 35	50
24 °C to 30 °C	100 to 90	35 to 30	50 to 40

Notes:

1 For deep-water structures different design current density values should be used for different temperature zones. To optimize the design the structure should be split up into separate zones over which the temperature does not vary by more than 1°C at temperatures ≤ 11°C and 5°C at temperatures > 11°C. The depth average temperature of each interval should be used to assess the required current densities.

A large variety of shapes and sizes can be used to deliver protective electrical current to the structure in order to optimize the electrical current distribution (see 4.3).

The performance of galvanic anodes in sea water depends critically upon the alloy composition, particularly when aluminum or zinc alloys are used (see prEN 12496, NACE RP0387 and NACE RP0492)

The electrochemical properties of the anodic material shall be documented or determined by appropriate tests.

The information required includes:

– the driving voltage to polarized steel, i.e. the difference between closed circuit anode potential and the positive limit of the protection potential criterion

– the practical electrical current capacity (Ampere hour per kilogram = Ah/kg) or consumption rate (kilogram per Ampere per year = kg/Ay)

– the susceptibility to passivation

– the susceptibility to intergranular corrosion

4.2 Galvanic anode materials

The cathodic protection calculation shall take into account the closed circuit potential and practical current capacity values guaranteed by the anode Supplier and verified during the qualification procedure of pEN 12496 and parameters specified by NORSOK M-503, and NACE RP0387 or NACE RP0492. For the Aluminum and Zinc anodes specified in pEN 12496 the requirements in Table 6.11 and 6.12 shall apply.

4.2.1 Aluminium alloy anodes

Aluminum alloy anodes can be used in sea water or in marine sediments.

The behavior of certain aluminum alloys may be adversely affected when covered with mud and particularly at low current output.

The alloys may suffer intergranular corrosion even at low temperatures.

The use of practical current capacity greater than the values of Table 6.12 should be justified by long term testing like free running test of minimum 12 month (NORSOK M-503). Closed circuit resistance shall be adjusted to give a nominal anodic current density of 1.0 ± 0.1 A/m^2. Minimum 16 samples from full scale anodes shall be used.

The electrochemical characteristics shall be documented for seawater at 5 - 12 °C.

Table 6.11 Typical chemical composition in % weight for Al- and Zn anodes

Elements	Alloy A2 (pEN 12496)	Alloy Z1 (pEN 12496)
Zinc (Zn)	2 -6	Remainder
Aluminum (Al)	Remainder	0,1 -0,5
Indium (In)	0,01 -0,03	0
Cadmium (Cd)	0	0,025 – 0,07
Silicon (Si)	0,10 max	0
Iron (Fe)	0,12 max	0,005 max
Copper (Cu)	0,005 max	0,005 max
Lead (Pb)	0	0,006 max

Table 6.12 Design values for galvanic anodes

Anode type according to prEN 12496	Density [kg/m^3] ζ	Internal fluid temp. °C	Seawater		Sediment	
			Potential [mV/Ag/AgCl/ seawater] Ea	Current capacity [Ah/kg] ε	Potential [mV/Ag/AgCl/ seawater] Ea	Current capacity [Ah/kg] ε
Alloy A2 (Aluminum alloy)	2725	5 - 30	–1050	2000	–1000	1730
		40				1450
		50				1200
		60				900
		70				650
		80				400
Alloy Z1 (Zinc)	7130	5 - 30	–1030	780	–980	750
		30 - 50				580
Ea = Closed circuit anode potential						

4.2.2 Zinc alloy anodes

Zinc alloy anodes can be used in sea water or in marine sediments.

Certain alloys may suffer intergranular corrosion particulary at elevated temperatures.

At elevated temperature zinc alloys may undergo a reduction in driving potential and current capacity and should not be used at temperatures exceeding 50 °C unless supported by appropriate tests.

For jackets equipped with mud mats, the part of the mats facing downwards shall only be protected by zinc alloy anodes.

4.3 Anode shapes and utilization factors

4.3.1 Stand-off anodes

Stand-off anodes shall be used as far as possible with a minimum distance to the steel surface of 300 mm (see Fig. 6.2). The insert should be preferably cylindrical in order to avoid gas entrapment during the alloy casting. The physical tolerances, anode weight, anode dimensions and straightness, steel inserts and metallurgical requirements shall be in accordance to prEN 12496 or NACE RP0387.

The dimensions of the insert and the support are to be designed taking into account the weight of the anodic alloy and with the various environmental loading conditions as detailed in ISO 12495 Chapter 5.5.4 – Structural considerations.

4.3.2 Flush-mounted anodes

Flush-mounted anodes are designed to limit hydrodynamic effects when fitted to the structure (see Fig. 6.2). The physical tolerances, anode weight, anode dimensions and straightness, steel inserts and metallurgical requirements shall be in accordance to prEN 12496 or NACE RP0387.

Zn anodes located underneath mud mats shall be flush-mounted**.**

Flush-mounted anodes may be recommended for the protection of mobile floating units.

4.3.3 Bracelet anodes

Bracelet anodes are designed to reduce the wave loads on steel jackets. Bracelet anodes are shaped as half-rings to be positioned on tubular items. Two half-ring anodes will have to fit together to become a bracelet anode (see Fig. 6.2). The physical tolerances, anode weight, anode dimensions and straightness, steel inserts and metallurgical requirements shall be in accordance to prEN 12496 or NACE RP0492.

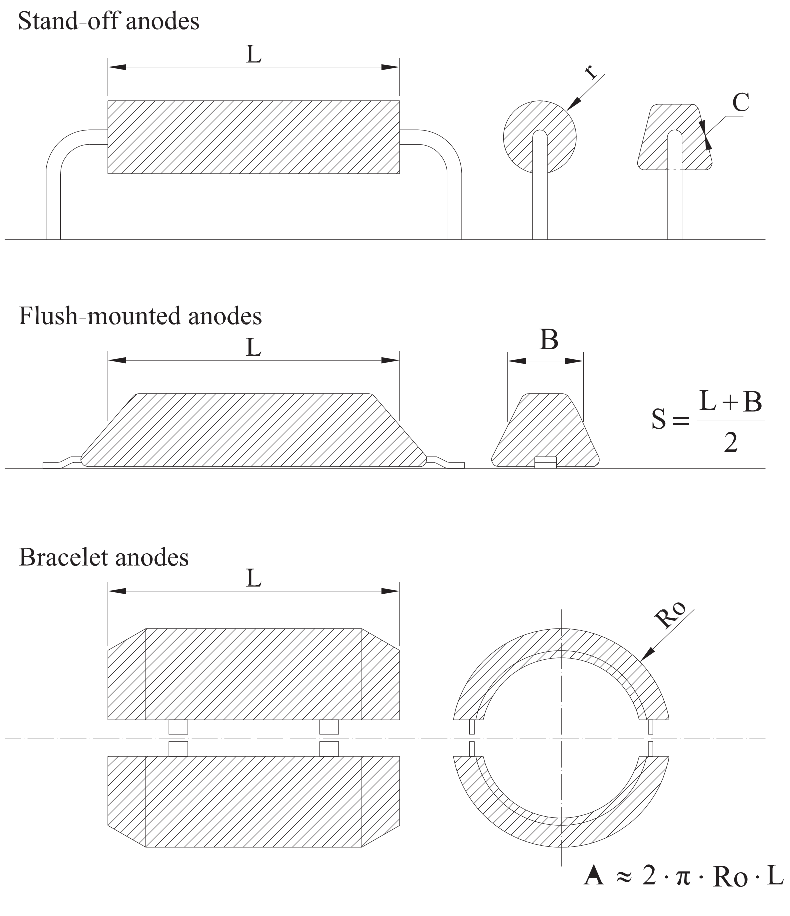


Fig. 6.2 Typical anode shapes

4.3.4 Utilization factor

The utilization factor is determined by the proportion of anodic material that may be consumed before the anode ceases to provide the required current output. The shape of the anode will affect the utilization factor.

Table 6.13 Recommended anode utilization factors

Anode type	Anode utilization factors u
Stand-off anodes, if $L \geq 4 \cdot r$	0,90
Stand-off anodes, if $L < 4 \cdot r$	0,85
Flush-mounted anodes	0,90
Bracelet anodes	0,80

4.4 Calculation of anode resistance

The anode resistance, Ra shall be in accordance with the following formulas:

Stand-off anodes, if $L \geq 4 \cdot r$:

$$Ra = \cdot \frac{\rho}{2 \cdot \pi \cdot L}\left[\ln \frac{4 \cdot L}{r} - 1\right] \tag{4}$$

Stand-off anodes, if $L < 4 \cdot r$:

$$Ra = \frac{\rho}{2 \cdot \pi \cdot L} \cdot \left[\ln\left\{\frac{2 \cdot L}{r}\left(1 + \sqrt{1 + \left(\frac{r}{2 \cdot L}\right)^2}\right)\right\} + \frac{r}{2 \cdot L} - \sqrt{1 + \left(\frac{r}{2 \cdot L}\right)^2}\right] \tag{5}$$

Flush-mounted anodes:

$$Ra = \frac{\rho}{2 \cdot S} \tag{6}$$

Bracelet anodes:

$$Ra = \frac{0{,}315 \cdot \rho}{\sqrt{A}} \tag{7}$$

Ra = anode resistance [ohm]

ρ = sea water/sediment resistivity [ohm m] (see Table 6.14)

L = length of the anode (m)

r = radius of the anode [m] (for other shapes than cylindrical, $r = C/2 \cdot \pi$, where C [m] = cross section periphery)

S = arithmetic mean of anode length and width [m]

A = exposed surface area of anode[m^2]

Table 6.14 Sea water resistivity as a function of temperature, salinity and density

Temperature [°C]	Salinity [ppt]	Density [kg/m^3]	Resistivity [ohm m]
-5	30	1024	0,47
-5	35	1028	0,40
-5	40	1032	0,34
0	30	1024	0,41
0	35	1028	0,35
0	40	1032	0,29
5	30	1024	0,35
5	35	1028	0,30
5	40	1032	0,24
10	30	1023	0,30
10	35	1027	0,26
10	40	1031	0,21
15	30	1022	0,27
15	35	1026	0,23
15	40	1030	0,18
20	30	1021	0,24
20	35	1025	0,21
20	40	1029	0,17
25	30	1020	0,22
25	35	1023	0,19
25	40	1027	0,16
30	30	1018	0,20
30	35	1022	0,18
30	40	1026	0,15

4.5 Sea water resistivity and temperature

4.5.1 Temperature has a significant influence on sea water resistivity, dissolved oxygen and calcareous deposit formation.

As the temperature along with salinity of sea water has a significant effect on the resistivity of sea water and as this is directly related to the effective resistance of galvanic anodes, it should be taken into account in the design of cathodic protection systems.

In the open sea the salinity does not vary significantly and temperature is the main factor. The relation between resistivity and temperature at a salinity of 30 to 40 ppt (parts per thousand) is given in Table 6.14. Fig. B.1 of ISO 12495 shall be used for sea water densities between 1000 kg/m^3 and the values of Table 6.14.

4.5.2 Compared to sea water, the resistivity of marine sediments is higher by a factor ranging from about 2 for very soft clays to 5 for sand. Unless sediment data for the location are available, the highest factor shall be assumed for calculation of the resistance of any buried anodes.

4.6 Cathodic protection calculation and design procedure

The purpose of this paragraph is primarily the clarification of the Cathodic Protection (CP) Calculation Procedure. The procedure will be demonstrated by using two different examples:

Example 1: Tripod

A tripod with mudmats, built from carbon steel, located in West Africa, designed for a life time of 30 years

Example 2: PLET (Pipeline end termination unit)

Carbon steel subsea structure (PLET =Pipeline end termination unit) connected to carbon steel flow line and jumper with Super-Duplex 25Cr flow mete, located in the East Mediterranean Sea in 500 m water depth, sea bed temperature 13°C, internal fluid temperature 50°C, designed for a life time of 25 years

4.6.1 Subdivision of the object in sections

The object has to be subdivided into sections using the zone definitions of Fig. 6.1.

Example 1: Tripod

The four following zones shall be separately considered:

- *atmospheric zone*
- *extended tidal zone*
- *immersed zone*
- *buried zone*

The following text, Figures and Tables shall be considered:

Fig.6.1: *Zones and profile of the thickness loss of an unprotected steel structure in sea water*

B.1. and B.2.: *Methods of corrosion protection in the four zones*

C.1. and C.2.: *Coating selection for the four zones (Tables 6.5, 6.6, and 6.7)*

D.1. and D.2.: *The subdivision shall consider all sections which influence the cathodic protection current demand and the different design current densities (Tables 6.8, 6.9, and 6.10).*

Possible solution:

Atmospheric Zone will not drain current from the CP system. This zone shall not include in the CP system and shall be protected minimum with a coating system of Table 6.5.

Extended Tidal Zone may be protected with 90-10 Cu-Ni on rubber-base insulator and will not drain current from the CP system.

Section 1: Immersed Zone may be protected with coating systems of coating category IV of Table 6.6.

Section 2: Mudmat underside may be none coated

Example 2: PLET

The following zones shall be considered:

- *immersed zone*

The following text, Figures and Tables shall be considered:

B.1. and B.2.: *Material Selection for deep water applications (Tables 6.1, 6.2, 6.3, 6.4)*

C.1. and C.2.: *Coating Selection for deep water structures and piping components (Tables 6.6, 6.7)*

D.1. and D.2.: *The Subdivision shall consider all sections which influence the cathodic protection current demand and the different design current densities (Tables 6.8, 6.9, 6.10).*

Possible solution:

Section 1: Subsea structure (PLET) may be protected with a coating system of coating category IV

Section 2: Flow line may be protected with a coating system of coating category V

Section 3: Jumper with Super-Duplex 25Cr flow meter may be protected with a coating system of coating category IV, because the CPT and CCT of Super-Duplex 25Cr are equal or lower than the fluid temperature, so this material shall cathodic protected.It shall be considered that Super-Duplex 25Cr is potential susceptible to HISC due to over polarization. Within the potential range of cathodic protection by Al-anodes (i.e. -0.80 to -1.10V Ag/AgCl/seawater) the production of hydrogen increases exponentially towards the negative potential limit. Forgings, casts, like flow meters and welds are more prone to HISC than wrought materials due to the course microstructure allowing HISC to propagate preferentially in the ferrit phase. I.e. the anodes shall be located as far as possible away from the Super-Duplex 25Cr component.

Section 1 shall be protected in accordance with this Section and the Sections 2 and 3 in accordance with Section 8 of the GL Rules IV – Industrial Services, Part 8 – Pipelines, Chapter 1 – Rules for Subsea Pipelines and Risers.

4.6.2 Surface area of sections

For each section, with or without coating the surface areas shall be calculated separately.

Example 1: Tripod

Solution:

Section 1: A1 = 659 m^2

Section 2: A2 = 238 m^2

Example 2: PLET

Solution:

Section 1: A1 = 288 m^2

4.6.3 Calculation of current demand

For each section the following required current demands shall be calculated:

$$Ici1 = Ac1 \cdot ici1 \cdot fci1 \quad (8)$$

$$Icm1 = Ac1 \cdot icm1 \cdot fcm1 \quad (9)$$

$$Icf1 = Ac1 \cdot icf1 \cdot fcf1 \quad (10)$$

Ici1 = required initial current demand of section 1 [mA]

Icm1 = required mean current demand of section 1 [mA]

Icf1 = required final current demand of section 1 [mA]

Ac1 = surface area of section 1 [m^2]

ici1 = required initial design current density of section 1 [mA/m^2]

icm1 = required mean design current density of section 1 [mA/m^2]

icf1 = required final design current density of section 1 [mA/m^2]

fci1 = initial coating breakdown factor, if section 1 is coated; fci1=1, if bare metal surface

fcm1 = mean coating breakdown factor, if section 1 is coated; fci1=1, if bare metal surface or fcm1 >1.

fcf1 = final coating breakdown factor, if section 1 is coated; fci1 = 1, if bare metal surface or fcf1 >1

Example 1: Tripod

The following clauses, figures and tables shall be considered:

C.2.: *Calculation of cathodic breakdown factors (Table 6.7)*

D.1. and D.2.: Required design current densities of geographic areas and in specific areas of the structure (Tables 6.8, and 6.9)

Solution:

*Section 1**:** Coating category IV*

Selection of k_1 and k_2 (Table 6.7: Immersed zone):

$k_1 = 0{,}01$, $k_2 = 0{,}01$

Selection of design current densities (Table 6.8: West Africa):

ici1= 130 mA/m^2, icm1= 65 mA/m^2, icf1= 90 mA/m^2

Calculation of cathodic breakdown factors:

$fci1 = k_1 = 0{,}01$

$fcm1 = k_1 + k_2 \cdot t/2 = 0{,}01 + 0{,}01 \cdot 30/2 = 0{,}16$

$fcf1 = k_1 + k_2 \cdot t = 0{,}01 + 0{,}01 \cdot 30 = 0{,}31$

Calculation of required current demands: Ici1, Icm1 and Icf1 for A1 = 659 m^2

$Ici1 = Ac1 \cdot ici1 \cdot fci1 = 856{,}7$ mA

$Icm1 = Ac1 \cdot icm1 \cdot fcm1 = 6853{,}6$ mA

$Icf1 = Ac1 \cdot icf1 \cdot fcf1 = 18386{,}1$ mA

Section 2: Mudmat underside with bare metal surface

Selection of design current densities (Table 6.9: Case No. 3: Bare metal in marine sediment):

ici2 = 25 mA/m^2, icm2 = 20 mA/m^2, icf2 = 20 mA/m^2

Calculation of required current demands Ici2, Icm2 and Icf2 for A2 = 238 m^2:

$Ici2 = A2 \cdot ici2 = 5950$ mA

$Icm2 = A2 \cdot icm2 = 4760$ mA

$Icm3 = A2 \cdot icf2 = 4760$ mA

Example 2: PLET

The following clauses, figures and tables shall be considered:

C.2.: *Calculation of cathodic breakdown factors (Table 6.7)*

D.1. and D.2.: Required design current densities in deep water (Table 6.10).

Solution:

*Section 1**:** Coating category IV.*

Selection of k_1 and k_2 (Table 6.7: Immersed zone):

k_1= 0,01, k_2=0,01

Selection of design current densities (Table 6.10):

ici1= 110 mA/m^2, icm1= 44 mA/m^2, icf1= 60 mA/m^2

Calculation of cathodic breakdown factors:

$fci1 = k_1 = 0{,}01$

$fcm1 = k_1 + k_2 \cdot t/2 = 0{,}01 + 0{,}01 \cdot 25/2 = 0{,}135$

$fcf1 = k_1 + k_2 \cdot t = 0{,}01 + 0{,}01 \cdot 25 = 0{,}6$

Calculation of required current demands: Ici1, Icm1 and Icf1 for A1 = 288 m^2

$Ici1 = Ac1 \cdot ici1 \cdot fci1 = 316{,}8$ mA

$Icm1 = Ac1 \cdot icm1 \cdot fcm1 = 1710{,}7$ mA

$Icf1 = Ac1 \cdot icf1 \cdot fcf1 = 4492{,}8$ mA

4.6.4 Selection of anode materials and shapes

4.1, 4.2, und 4.3 shall be considered.

Example 1: Tripod

Solution:

Stand-off aluminum alloy anodes for immersed zone (see Fig. 6.2)

Flash-mounted zinc alloy anodes on mud mats facing downwards (see Fig. 6.2).

Example 2: PLET

***Solution*:**

Stand-off aluminum alloy anodes placed on subsea structure

4.6.5 Calculation of anode mass

Calculation of the required net anode mass (Ma) for the required mean current demand Icm (see 4.6.3) for each section.

The Table 6.8 (Geographic area, Water temperature), 4.1, 4.2 (Table 6.12), und 4.3 (Table 6.13) shall be considered.

$$\text{Ma} = \text{Icm} \cdot \text{t} \cdot 8760 / 1000 \cdot \text{u} \cdot \varepsilon \qquad (11)$$

Ma = net anode mass [kg]

Icm = required mean current demand [mA] (4.6.3)

t = design life (years)

8760 = refers to hours per year

u = anode utilization factor (Table 6.13)

ε = anode current capacity [Ah/kg] (Table 6.12)

Example 1: Tripod

Solution:

Section 1:

Selection of Anode current capacity ε1 (1 = section 1) in seawater, considering anode material (Table 6.12: Alloy A2) and sea water temperature (Table 6.8: West Africa water temperature 5 -21): ε1 = 2000 [Ah/kg]

Selection of utilization factor u1 (1 = section 1) considering Anode Type Stand-off with L ≥ 4 · r (Table 6.13): u1 = 0,9

Calculation of net anode mass Ma1 of section 1:

Ma1 = Icm1 · t · 8760 / 1000 · u1 · ε1 = 6853,6 · 30 · 8760/ 1000 · 0,9 · 2000 = 1000 kg

where: Icm1 = 6853,6 mA, t = 30 years

Section 2:

Selection of Anode current capacity ε2 (2 = section 2) in sediment, considering anode material (Table 6.12: Alloy Z1) and sea water temperature (Table 6.8: West Africa water temperature 5-21): ε2 = 750 (Ah/kg)

Selection of utilization factor u2 (2 = section 2) considering Anode Type Flush-mounted (Table 6.13): u2 = 0,9

Calculation of net anode mass Ma1 of section 2:

Ma2 = Icm2 · t · 8760 / 1000 · u2 · ε2 = 4760 · 30 · 8760 / 1000 · 0,9 · 750 = 1849 kg

where: Icm2 = 4760 mA, t = 30 years

Example 2: PLET

Solution:

Section 1:

Selection of Anode current capacity ε1 (1 = section 1) in seawater, considering anode material (Table 6.12: Alloy A2) and subsea water temperature of 13 °C: ε1 = 2000 (Ah/kg)

Selection of utilization factor u1 (1 = section 1) considering Anode Type Stand-off with L ≥ 4 · r (Table 6.13): u1 = 0,9

Calculation of net anode mass Ma1 of section 1:

Ma1 = Icm1 · t · 8760 / 100 · u1 · ε1 = 1710 · 25 · 8760 / 1000 · 0,9 · 2000 = 208 kg

where: Icm1 = 1710 mA, t = 25 years

4.6.6 Calculation of anode number

4.6.6.1 Acceptance criteria

The calculation shall be iterative, starting with a commercially available product sizing, e.g. example Tripod, section 1 with Ma1 = 1000 kg, N = 7 (stand-off anode with anode net weight of ma = 150 kg). The result of the iteration shall be:

.1 The number of initial (new) anodes (Ni) of one section multiplied with the initial anode current output (Iai_o) of one anode shall be equal or larger than the required initial current demand (Ici) of the section.

$$\text{Ici} \leq \text{Ni} \cdot \text{Iai}_o \qquad (12)$$

.2 The Number of final (consumed) anodes (Nf) of one section multiplied with the final anode current output (Iafo) of one anode shall be equal or larger than the required final current demand (Icf) of the section.

$$\text{Icf} \leq \text{Nf} \cdot \text{Iaf}_o \qquad (13)$$

.3 The Number of anodes (N) based on mean current output of one section multiplied with the mean current capacity (Ca_o) of one anode shall be equal or larger than the required mean current output of the section.

$$\text{Ca} = \text{N} \cdot \text{Ca}_o \geq \text{Icm} \cdot \text{t} \cdot 8760 \qquad (14)$$

The following clauses, Figures and Tables shall be considered:

B.3., Table 6.4: Protective Potentials for various metals in sea water

4.2, Table 6.12: Closed circuit anode potential Ea

4.4, 4.5, and Table 6.14: Calculation of Anode Resistance

In the case of stand-off anodes the dimensions of the insert and the support are to be designed taking into account the weight of the anodic alloy and with the various environmental loading conditions as detailed in ISO 12495 Chapter 5.5.4 – Structural considerations, see 4.3.

The design protective potential Ec for all metals of Table 6.4 in seawater and anaerobic environment (sediment, mud) shall be –800 mV.

Iteration with N

$$Iai_o = \frac{(Ec - Ea)}{Rai} \qquad (15)$$

$$Iaf_o = \frac{(Ec - Ea)}{Raf} \qquad (16)$$

$$Ca_o = mam \cdot \varepsilon \cdot u \qquad (17)$$

Iai_o = initial current output of one anode [mA]

Iaf_o = final current output of one anode [mA]

Ea = closed circuit anode potential [mV] (see Table 6.12)

Ec = Design protective potential [–800 mV]

Rai = initial anode resistance of the new anode [ohm] (4.4)

Raf = final anode resistance consumed to its utilization factor u [ohm]

Ca_o = Anode mean current capacity of one anode [A·h]

mam = mean anode net mass of one anode [kg]

u = Anode utilization factor (Table 6.13)

ε = Anode current capacity [Ah/kg] (Table 6.12)

4.6.6.2 Calculation of initial and final anode resistance

The final net anode mass maf and final volume Vaf is given by:

$$maf = mai \cdot (1 - u) \qquad (18)$$

$$Vaf = \frac{maf}{\zeta} \qquad (19)$$

mai = initial anode net mass of the new anode [kg]

maf = final anode net mass consumed to its utilization factor u [kg]

ζ = Density of the anode material [kg/m³] (Table 6.12)

Vaf = final volume of the final anode [m³]

The final dimensions and final anode resistance Raf of the different anode shapes can be calculated as follows:

Stand-off anodes:

$$Vaf = \frac{maf}{\zeta} = \frac{(daf^2 - dac^2)\cdot\pi}{4}\cdot Laf \qquad (20)$$

daf = final diameter of the anode (daf = 2 · raf) [m]

dac = core diameter [m]

raf = final radius of the anode [m]

Laf = final length of the anode shall be: Laf = Lai · 0,9 [m]

Lai = initial length of the anode [m]

$$daf = \sqrt{\left(\frac{maf}{\zeta} + \frac{dac^2\cdot\pi\cdot Lai\cdot 0,9}{4}\right)\cdot\frac{4}{\pi\cdot Lai\cdot 0,9}} \qquad (21)$$

Raf calculation with the equations (4) and (5) of 4.4:

Stand-off anodes, if L ≥ 4 · r:

$$Raf = \frac{\rho}{2\cdot\pi\cdot Laf}\left[\ln\frac{4\cdot Laf}{raf} - 1\right] \qquad (22)$$

Stand-off anodes, if L < 4 · r:

$$Raf = \frac{\rho}{2\cdot\pi\cdot Laf}\left[\ln\left\{\frac{2\cdot Laf}{raf}\left(1+\sqrt{1+\left(\frac{raf}{2\cdot Laf}\right)^2}\right)\right\} + \frac{raf}{2\cdot Laf} - \sqrt{1+\left(\frac{raf}{2\cdot Laf}\right)^2}\right] \qquad (23)$$

Raf = final anode resistance [ohm]

ρ = sea water resistivity [ohm · m] (see Table 6.8: water temp. and Table 6.14)

Laf = Lai · 0,9 = final length of the anode [m]

raf = daf/2 = final radius of the anode (for other shapes than cylindrical, assuming that the final anode shape is cylindrical) [m]

Flush-mounted anodes:

The shape of flush-mounted anodes are almost similar to the non cylindrical stand-off anodes (see Fig. 6.2). The final shape shall be assumed to be semi-cylindrical and the final length (Laf), final radius (raf) shall be calculated with the equations (20) and (21).

$$\text{Vaf} = \frac{\text{maf}}{\zeta} = \frac{(\text{daf}^2 - \text{dac}^2)\cdot\pi}{4}\cdot\text{Laf} \qquad (20)$$

daf = final diameter of the anode (daf = 2 · raf) [m]

dac = core diameter (for other shapes than cylindrical, dac = C/π, where C [m] = cross section periphery) [m]

raf = final radius of the anode [m]

Laf = final length of the anode shall be: Laf = Lai · 0,9 [m]

Lai = initial length of the anode [m]

$$\text{daf} = \sqrt{\left(\frac{\text{maf}}{\zeta} + \frac{\text{dac}^2\cdot\pi\cdot\text{Lai}\cdot 0{,}9}{4}\right)\cdot\frac{4}{\pi\cdot\text{Lai}\cdot 0{,}9}} \qquad (21)$$

Raf calculation with the equation (6) of 4.4:

$$\text{Raf} = \frac{\rho}{2\cdot\text{Saf}}$$

Raf = final anode resistance [ohm]

ρ = sea water resistivity [ohm · m] (see Table 6.14)

Laf = Lai · 0,9 = final length of the anode [m]

daf = final diameter of the anode (for other shapes than cylindrical, assuming that the final anode shape is cylindrical) [m]

Saf = (Laf + daf) / 2 = arithmetic final of anode length and diameter [m]

Bracelet anodes:

For bracelet anodes the final exposed area (Aaf) shall be assumed to be equivalent to the initial exposed area (Aai).

Raf calculation with the equation (7) of 4.4:

$$\text{Raf} = \frac{0{,}315\cdot\rho}{\sqrt{\text{Aaf}}}$$

Raf = final anode resistance [ohm]

ρ = sea water resistivity [ohm · m] (see Table 6.14)

Aaf = exposed final surface area of anode [m²]

4.6.6.3 Calculation of initial and final current output and mean current capacity of the anodes

Example 1: Tripod

Solution:

Section 1:

Selection of stand-off anode size (see Fig.6.2, not cylindrical), closed circuit anode potential Ea=-1050 mV (Table 6.12), design protective potential Ec = -800 mV, anode current capacity ε=2000 Ah/kg (Table 6.12) and Anode utilization factor u=0.9 (Table 6.13).

The following parameters may be selected, if Ma1= 1000 kg:

N = 7 (Number of anodes)

mai = 150 (initial anode weight) [kg]

ζ = 2725 (density of the anode material – Table 6.12) [kg/m³]

Lai = 1,77 (initial anode length) [m]

W = 0,184 (initial anode width) [m]

D = 0,184 (initial anode depth) [m]

dac = 0,06 (core diameter) [m]

u = 0,9

ρ = 0,24 (average water temperature of West Africa acc. to Table 6.8: 13°C); salinity may be 35 ppt; resistivity 0,24 [Ohm·m] acc. to Table 6.14) [Ohm · m]

Calculation of Rai according to formula (4):

$$rai = \frac{C}{2\cdot\pi} = 0{,}117\ [m] \qquad C = 2\cdot W + 2\cdot D = 0{,}736\ [m]$$

$$Rai = \frac{\rho}{2\cdot\pi\cdot Lai}\left[\ln\frac{4\cdot Lai}{rai} - 1\right] = 0{,}067\ [Ohm]$$

Calculation of Raf according to formular (22):

$$maf = mai\cdot(1-u) = 15\ [kg] \qquad \text{(see (18))}$$

$$Vaf = \frac{maf}{\zeta} = 0{,}0055\ [m^3] \qquad \text{(see (19))}$$

maf = final anode net mass consumed to its utilization factor u [kg]

Vaf = final volume of the final anode [m³]

$$daf = \sqrt{\left(\frac{maf}{\zeta} + \frac{dac^2\cdot\pi\cdot Lai\cdot 0{,}9}{4}\right)\cdot\frac{4}{\pi\cdot Lai\cdot 0{,}9}}$$

$$= 0{,}089\ (m) \qquad \text{(see (21))}$$

daf = *final diameter of the anode (daf = 2 · raf) [m]*

raf = *final radius of the anode [m]*

Laf = *final length of the anode shall be: Laf = Lai · 0,9 [m]*

Lai = *initial length of the anode [m]*

$$Raf = \frac{\rho}{2 \cdot \pi \cdot Laf}\left[ln\frac{4 \cdot Laf}{raf} - 1\right] = 0{,}095 \text{ (ohm) (see (22))}$$

4.6.6.4 Examination of fulfillment of acceptance criteria:

Example 1: Tripod

Solution:

Section 1:

Ec = – *800 mV*

Ea = –*1050 mV*

Iai_o = *initial current output of one anode [mA]*

Iaf_o = *final current output of one anode [mA]*

Raf = *final anode resistance consumed to its utilization factor u [ohm]*

Ca_o = *Anode mean current capacity of one anode [A·h]*

mam = *mean anode net mass of one anode [kg]*

ε = *2000 Ah/kg (anode current capacity)*

$$Iai_o = \frac{(Ec - Ea)}{Rai} = 3730 \text{ mA} \qquad \text{(see (15))}$$

$$Iaf_o = \frac{(Ec - Ea)}{Raf} = 2632 \text{ mA} \qquad \text{(see (16))}$$

$$Ca_o = mam \cdot \varepsilon \cdot u = 270\,000 \text{ A·h} \qquad \text{(see (17))}$$

Acceptance Criteria:

.1 $Ici1 \leq N \cdot Iai_o = 7 \cdot 3730 = 26110$ *mA;*
$Ici1 = 856{,}7$ *mA;* *(see (12))*

.2 $Icf1 \leq N \cdot Iaf_o = 7 \cdot 2632 = 18424$ *mA;*
$Icf1 = 18386$ *mA;* *(see (13))*

.3 $Ca = N \cdot Ca_o \geq Icm1 \cdot t \cdot 8760 = 1801126\ A \cdot h;$
$Ca = 1890000\ A \cdot h;$ *(see (14))*

If the equations 13, 14 and 15 cannot be fulfilled for the initially selected dimensions and/or numbers, another anode size and/or numbers shall be selected and the calculation repeated until the criteria are fulfilled.

4.7 Location of anodes

Based on the calculated number of anodes required for different zones of the structure, for the distribution of anodes ISO 12495 and 13173 and the following aspects shall be considered:

– anodes shall be uniformly distributed

– anodes for the splash zone shall be located on the upper level of immersed zone

– anodes shall be located nearer complex and critical points such as node areas, but not closer than 600 mm to nodes

– anodes protecting conductor pipes shall be uniformly distributed at the different levels in their immediate proximity on the conductor guide frames

– anodes corresponding to the current drain of wells shall be uniformly distributed over all the immersed part of the structure

– on legs, the anodes shall face the centre of the structure

– on diagonals, the anodes shall be alternately placed on the upper and lower surface if more than one anode is required

– on horizontal bracings at different levels, the anodes shall be installed alternately facing up and down with the exception of the uppermost level where they shall be mounted facing downwards

– The adequacy of the current distribution can be improved using a greater number of anodes, which have a lower individual electrical current output.

The location of all individual anodes shall be shown on drawings. With the exception of stand-off anodes, coating with minimum a coating category II (Table 6.6) shall be applied to the anode surface towards the protection object.

4.8 Anodes' inserts and attachments design

The method of attachment of the anodes to the structure shall be in accordance with ISO 12495 and 13173.

4.9 Documentation of Cathodic Protection Design Report

The Cathodic Protection Design Report shall contain the following parts:

– design criteria including the design life, the environment characteristics (see Tables 6.8 and 6.14), the protection criteria, the electrical current density requirements (see Tables 6.8, 6.9, and 6.10), the assumed values of the anode current output at these various periods, and the anode theoretical current capacity and closed circuit anode potential (see Table 6.12) incl. all relevant project specifications, codes, standards, literatures;

– subdivision of the object in sections incl. to all design data (see 4.6.1)

– surface area calculation of all sections (see 4.6.2)

– current demand calculation (see 4.6.3)

– calculation of minimum required net anode mass (see 4.6.5)

– calculation of minimum anode number (see 4.6.6)

– calculation of initial and final anode resistance (see 4.6.6.2)

– calculation of initial and final current output and mean current capacity of the anodes (see 4.6.6.3)

– examination on fulfillment of acceptance criteria (see 4.6.6.4)

– the number of galvanic anodes, their dimensions (see Fig. 6.2), weight, specification, chemical composition (see Table 6.11), practical consumption rate or current capacity (as measured during laboratory tests, see 4.2 and 4.3), utilization factor (see Table 6.13), as well as the manufacturer/supplier's references and documentation

– anode location drawings (see 4.7)

– anode attachment drawings (see 4.8)

4.10 Design of monitoring system

Permanently installed monitoring systems may be used with galvanic anodes system.

The monitoring system of a cathodic protection system measures and may be used to control the operating parameters and the effectiveness of the cathodic protection system. ISO 12495 and 13173 shall be used as a guideline.

4.11 Commissioning and surveying

The objectives of the commissioning and the survey of the cathodic protection system are:

– to ensure that the cathodic protection system functions in accordance with the intentions of the design at structure installation

– to establish if the cathodic protection system is continuing to perform in accordance with the design and that the structure remains satisfactorily protected from corrosion

– a cathodic protection survey of the entire structure shall be performed within three month for bare steel structure or one year on coated structures from the installation of the structure

– the survey should include the measurement of the potential at selected locations

– the actual location and status of the anodes should be verified and recorded

The documentation of commissioning and surveying shall include:

– the location of each and every anodes as checked during construction or after positioning, all discrepancies with the design location being highlighted in the as-built drawings, the method of attachment, the date of installation; these data should be updated during the life of the structure

– the description, specification and position of any electrical current or potential control or monitoring device, including type of reference electrode, measuring equipment, and connecting cables

– the commissioning results including the potential survey data

4.12 Safety and cathodic protection

The cathodic protection system shall comply with all safety standards and regulations related to electrical equipment that may apply to fixed steel offshore installations.

Herewith the safety hazards due to cathodic protection systems and related to diving personnel during their underwater operations are defined. The galvanic anodes system will be considered, in association with physical obstruction and evolution of dangerous gases.

The major risk from anodes is the entanglement of the diver's umbilical or life line around the anodes or the anode supports, and the mechanical damage to the equipment due to chafing. Thus the following galvanic anode and anode assembly characteristics are recommended (see Fig.6.2):

– stand-off anodes with J-shaped cylindrical stand off supports, protruding from anode extremities

– flush- mounted anodes

– bracelet anodes

Similarly, anode cable conduits, junction boxes, etc. shall be designed to exclude any sharp edges or corners or protruding extremities.

Polarization of the structure to potential more negative than –800 mV can result in the evolution of hydrogen gas at the steel surface. If the gas is allowed to collect in confined air spaces, such as cofferdams which are only part full of sea water, it may present a risk of explosion. All designs shall include an adequate venting to prevent the build up of hydrogen.

5. Impressed current system

5.1 Design considerations

The impressed current system for fixed steel offshore structures and floating structures usually includes a variable transformer rectifier and one or several anodes and normally a number of reference electrodes.

Manually adjustable rectifiers are generally selected because the current demand does not vary significantly with time.

Rectifiers with automatic potential control may be used in particular cases where the environment condition and the structure configuration induce large and frequent variations of the current demand.

Specific areas presenting particular situations may even require the consideration of a multi-zone control system in order to adapt and optimize the current distribution to the protection demands, the control of the protection of each zone being performed by a separate specific impressed current system.

The number and location of the anodes shall be determined in order to achieve, as far as is practical an adequate protective potential level.

Individual impressed current anodes usually deliver higher electrical currents than galvanic anodes, therefore with impressed current anodes either dielectric shields or larger anode to structure separation should be used to prevent localized over-polarization and improve the electrical current distribution to the cathode.

5.2 Direct current power source

The transformer rectifier or controllable DC generator shall be able to deliver the total current demand to the zone it is intended to protect.

The output voltage shall take into account the resistance of the electric circuit (cables, anodes) and the back e.m.f. (back electromotive force) between the anode and the cathode. The back e.m.f. is the naturally occurring open circuit potential difference between the anode and the cathode in sea water.

The transformer rectifier shall be able to deliver sufficient current to maintain the steel / sea water / reference electrode potential within the design range.

DC power source with automatic potential control shall deliver an electrical current when one of the reference electrodes used for the control of the DC power source leads to a potential reading less negative than the set minimum negative potential or more negative than the set maximum negative potential (see Table 6.4).

There shall be devices to limit the output current to each anode to a pre-set value.

A DC power source without current limitation circuits should have an effective shutdown in the event of an external short circuit.

5.3 Anode materials

Anodes used for impressed current systems can be of two main types: semi-consumable or inert anodes.

Platinum coated titanium and niobium anodes were claimed to be the most commonly used electrodes for cathodic protection of large offshore structures. Lead alloy anodes were listed as being popularly used to protect semi-submersible drilling rigs.

Impressed current anodes, unlike galvanic anodes, can be driven over a wide range of anode current densities, depending on the designer's preferences and the demands of a particular application. Because anode current loadings have a definite bearing on consumption rates, any meaningful listing of consumption rates for various anode materials shall necessarily include the approximate anode current density at which such rates have been established.

Table 6.15 lists consumption rates, associated anode current densities of various impressed current anode materials used for CP systems on offshore structures.

Platinum is the ideal permanent impressed current anode material. It is one of the most noble metals and in practically all environments forms a thin invisible film which is electrically very conductive. In addition, the exchange current densities of most anodic reactions on the Pt surface are greater than on other anode materials. Due to its high cost, platinum is applied as a thin coating (1–5 µm) on metallic substrates such as titanium, niobium and tantalum.

To avoid the dissolution of titanium at unplatinized locations on the surface, the operating voltage of the anode is limited by anodic breakdown potential of titanium which is in the range of 9 to 9,5 V in the presence of chlorides. Hence the maximum recommended operating voltage of platinized titanium anodes is 8 V. The corresponding maximum current density output is approximately 1 kA/m^2.

For cathodic protection systems where operating voltages are relatively high, niobium and tantalum based anodes are generally selected. This is because these two substrates have anodic breakdown potentials greater than 100 V in chloride containing electrolytes.

Table 6.15 Typical electrochemical characteristics for impressed current anodes

Anode materials	Consumption rate [g/A·y]	Max. current density [A/m^2]	Max. Voltage [V]
Platinized Titanium	0,004 to 0,0012	500 to 3000	8
Platinized Niob	0,004 to 0,0012	500 to 3000	50
Platinized Tantalum	0,004 to 0,0012	500 to 3000	100
Mixed Metal Oxide	0,0006 to 0,006	400 to 1000	8
Lead Silver	25 to 100	250 to 300	24
Chromium silicon iron	250 to 500	10 to 30	50

The rate of platinum consumption has been found to accelerate in the presence of AC current ripple. Most consumption was observed to occur with AC frequencies of less than 50 Hz. Ripples frequencies less than 100 Hz should be avoided. The repeated oxidation/ reduction processes result in the formation of a brownish layer of platinum oxide. To avoid the occurrence of this phenomenon, a single or a three phase full-wave rectification is recommended. The consumption rate of platinized anodes is also adversely affected by the presence of organic impurities such as sugar and diesel fuel.

5.4 Anode shapes and installation

Impressed current systems are more critical with respect to mechanical damage because relatively few anodes, each discharging a substantial amount of protective current, are involved. The loss of an anode can seriously reduce system performance.

The electrical connection between the anode lead cable and the anode body shall be watertight and mechanically sound.

Cable and connection insulating materials should be resistant to chlorine, hydrocarbons, and other deleterious chemicals.

Care shall be taken to provide suitable mechanical protection for both the anode and its connecting cable. On suspended systems, the individual anodes or anode strings may be equipped with winches or other retrieval means as a damage- preventing measure during severe storms or for routine inspection and maintenance. The loss of protection during these periods should be considered.

Acceptable methods of installing fixed-type impressed current anodes include, but are not limited to, the following:

5.4.1 Tubular, rod and wire anodes

Tubular, rod, and wire anodes can be installed at the lower ends of protective vertical steel pipe casings or conduits. Casings should be attached to above-water structure members and supported at repeating members below water. The anodes should be lowered through the casings (which protect the anode lead wires) and should be allowed to extend below a termination fitting at the bottom of each casing. This method provides a means of anode retrieval or replacement using the anode cable, without diver assistance. Marine growth or corrosion scale may make anode retrieval difficult.

5.4.2 Flush mounted anodes

Flush- mounted anodes, insulating-type holders can be attached directly to submerged structure members or to auxiliary structural members, such as vertical pipes, which can be removed for anode replacement. Properly designed systems of the latter type permit anode retrieval without diver assistance.

5.4.3 Stand-off anodes

Stand-off anodes can be installed on submerged structure members. Diver assistance is required for this type of anode replacement.

5.4.4 Bottom-installed anodes

Bottom-installed anodes are typically mounted on specially designed concrete sleds for stability; this minimizes the possibility of their becoming covered with mud or silt.

5.4.5 Location of anodes

All precautions shall be taken to avoid any leakage of water through the hull penetration. It is usually a requirement to fit a cofferdam. Typical anode mountings with cofferdam arrangement are given in ISO 13173 annex D.

Impressed current anodes should be located as far as practical from any structure member (usually a minimum distance of 1.5 m, but proportional to current magnitude (see Table 6.15). If a spacing of 1,5 m is not feasible a dielectric shield should be used to minimize wastage of protective current by localized overprotection.

Anode holder should be designed to avoid such wastage and to minimize the possibility of a short circuit between the anode and the structure.

There may be a delay of several months to a year or more between the time a structure is set until permanent electrical power becomes available. Plans should be made for either temporary power and early energizing of impressed current systems or a short-term galvanic anode system (see also 3.). Otherwise, serious corrosion of structure members, as well of as the underwater components of the impressed current system, can occur.

5.5 Dielectric shields

The objective of dielectric shields, and coatings used as dielectric shields, is to prevent extremely high current densities and current wastage in the vicinity of the anodes. This serves to promote more uniform protective current distribution.

Materials selected shall be suitable for the intended service.

Liquid applied coatings, fiberglass reinforced plastic, prefabricated plastic or elastomeric sheets can be used on the structure adjacent to the anodes, or incorporated into the anode assembly.

The electrochemical reactions occurring at the anode and cathode surfaces produce corrosive products and gases which may deteriorate the dielectric shield or include its disbanding.

The design of the dielectric shield should take into account the possible deterioration and aging characteristics of the materials.

Liquid applied coatings, prefabricated plastic or elastomeric sheets can be used on the structure adjacent to the anodes, or incorporated into the anode assembly.

5.6 Reference electrodes

Reference electrodes are used to measure the steel to electrolyte potential and may be used to control the output current of the cathodic protection system. They are either zinc or silver/silver chloride/sea water electrodes (see ISO 12473). Zinc electrodes are more robust whereas silver/silver chloride/sea water electrodes are more accurate.

When reference electrodes are used to control the impressed current system, they should be installed at locations determined by calculations or experience to ensure the potential of the structure is maintained within the design set limits (see Table 6.4).

Precautions should be taken in order to avoid any direct electrical contact between the electrodes and the steel structure.

5.7 Cathodic protection calculation and design procedure

The required current demand for each section of the object shall be calculated as described in 4.6.1 to 4.6.3.

For current demand determination of floating structures, the underwater surface area should always include the boot topping, but never the atmospheric zone.

To compensate for a less efficient current distribution (small number of anodes), the impressed current cathodic protection system shall be designed to be able to provide 1,25 to 1,5 times the calculated total current demand, which is the total design current demand.

Example 1: Tripod

The following clauses, figures and tables shall be considered:

C.2.: *Calculation of cathodic breakdown factors (Table 6.7)*

D.1. *and* D.2.: *Required design current densities of geographic areas and in specific areas of the structure (Tables 6.8, and 6.9).*

Solution:

Section 1: Coating category IV.

Selection of k_1 and k_2 (Table 6.7: Immersed zone):

$k_1 = 0{,}01,\ k_2 = 0{,}01$

Selection of design current densities (Table 6.8: West Africa):

$ici1 = 130\ mA/m^2,\ icm1 = 65\ mA/m^2,\ icf1 = 90\ mA/m^2$

Calculation of cathodic breakdown factors:

$fci1 = k_1 = 0{,}01$

$fcm1 = k_1 + k_2 \cdot t/2 = 0{,}01 + 0{,}01 \cdot 30/2 = 0{,}16$

$fcf1 = k_1 + k_2 \cdot t = 0{,}01 + 0{,}01 \cdot 30 = 0{,}31$

Calculation of required current demands: Ici1, Icm1 and Icf1 for A1=659m²

$Ici1 = Ac1 \cdot ici1 \cdot fci1 = 856{,}7\ mA$

$Icm1 = Ac1 \cdot icm1 \cdot fcm1 = 6853{,}6\ mA$

$Icf1 = Ac1 \cdot icf1 \cdot fcf1 = 18386{,}1\ mA$

Multiplied with a factor of 1,5 the minimum required total current demand shall be:

$Icf1tot = 1{,}5 \cdot Icf1 = 27579\ mA$

For any cathodic protection system, the size of the anodes shall be determined using Ohm's law. In the case of inert anodes, like platinized or mixed metal oxide anodes the anode resistance Ra will be stable during the life time, because no change of the dimensions will occur. In case of semi- consumable anodes, like lead silver and chromium silicon iron the initial and final anode resistance shall be considered (see 4.6.6).

$$Ia = \frac{(Ec - Ea)}{Ra} \qquad \text{(see 15)}$$

Ia = current output of anodes [mA]

Ea = Closed circuit potential (see Table 6.15) [mV]

Ec = Design protective potential (see Table 6.4) [mV]

Ra = anode resistance (4.4)

The anodic resistance is a function of the resistivity of the anodic environment and of the geometry (form and size) of the anode. The equations (4) and (5) for Stand-off anodes, (6) for Flush-mounted anodes and (7) for Bracelet anodes shall be used.

The acceptance criteria:

The maximum value of Ici, Icm, and Icf multiplied with a factor of 1,5 shall be smaller than the current output of the anodes.

5.8 Design of monitoring system

Permanently installed monitoring system shall be used with impressed current systems.

The monitoring system of a cathodic protection system measures and may be used to control the operating parameters and the effectiveness of the cathodic protection system.

The cathodic protection is an active system, i.e. it is effective only when the impressed current systems are operating and provide adequate polarization of the steel to achieve the protection criteria stated.

Therefore, the steel potential should be measured during the life of the structure in order to verify the adequacy of the protection.

The monitoring system may include the following:

– Permanent reference electrodes. The reference electrode should be installed and secured on each immersed level of the structure, close to the steel surface, and at locations as mentioned in 5.4. These reference electrodes shall be electrically insulated from the structure and connected to the data transmission system.

– Conduit systems, cables, junction boxes, data transmission systems shall be in accordance to IS0 12495

5.9 Potential measurements

The potential measurements shall be conducted in accordance to ISO 12495 or 13173.

5.10 Commissioning and surveying

For the commissioning of the impressed current cathodic protection system the following operations shall be carried out:

– check wiring to ensure correct polarity (negative terminal to structure)

– switching-on the transformer-rectifier(s) to supply direct current to the structure

– monitoring of the structure's potentials with the monitoring system until the required potential values are reached

A cathodic protection potential survey of the structure should be performed within one month of the commissioning of the cathodic protection system.

This survey should include the recording of the structure potential at selected locations. The actual location and status of anodes, cables, conduits, monitoring system should be verified and recorded.

5.11 Documentation

The following data should be kept for reference and permanently updated, if applicable:

– design criteria including the design life, the environment characteristics (see Tables 6.8 and 6.14), the protection criteria, the electrical current density requirements (see Tables 6.8, 6.9, and 6.10) the assumed values of the anode current output (Table 6.15) incl. all relevant project specifications, codes, standards, literatures

– subdivision of the object in sections incl. to all design data (see 4.6.1)

– surface area calculation of all sections (see 4.6.2);

– current demand calculation (see 4.6.3)

– calculation of anode resistance (see 4.4)

– calculation of the required current output of the anodes (see 5.7)

– examination on fulfillment of acceptance criteria (see 5.7)

– the number of anodes, their dimensions, specification (see 5.7 and Table 6.15), description of anodic equipment and connection, effective output current densities, associated consumption rates and allowable voltage (see Table 6.15), as well as the manufacturer/supplier's references and documentation

– the description and means of attachment of anodes, the composition and location of any dielectric shield, as well as the specification, characteristics and attachment method and through wall or through hull arrangements of the connecting cables (see 5.4 and 5.5)

– the location of each and every anodes as checked during construction or after positioning, all discrepancies with the design location being highlighted in the as-built drawings, the method of attachment, the date of installation; these data should be updated during the life of the structure (see 5.4)

– the location, detailed specification, drawings, and output characteristics of each DC power source with their factory test reports (see 5.2)

– the location, description and specification of any protection, potential control or monitoring device, including reference electrode, measuring equipment and connecting cables (see 5.6, 5.8, and 5.9)

– the commissioning results including the potential survey data; current and voltage output values of each DC power source and any adjustment made for non-automatic devices (see 5.10)

– the results of data recorded during periodic maintenance inspection including protection potential values, DC output values, maintenance data on DC power sources and downtime periods in order to follow the changes of the protection potential level status of the structure.

5.12 Safety and cathodic protection

The cathodic protection system shall comply with all safety standards and regulations related to electrical equipment that may apply to fixed or floating steel offshore installations.

This clause deals with safety hazards due to cathodic protection systems and related to diving personnel during their underwater operations. The galvanic anodes system will be considered, in association with physical obstruction, electric shock and evolution of dangerous gases.

The major risk from anodes is the entanglement of the divers umbilical or life line around the anodes or the anode supports, and the mechanical damage to the equipment due to chafing.

Similarly, anode cable conduits, junction boxes etc. should be designed to exclude any sharp edges or corners or protruding extremities.

In the event that the impressed current cathodic protection system equipment is defective, or the diver inadvertently makes direct contact with the active anode element, he may suffer an electric shock.

During diving operations not directly related to cathodic protection system, and any diving inspection carried out close to impressed current anodes, the DC supply of the anodes shall be switched off. However, diving cathodic protection inspection may be performed, with the impressed current system in operation, providing all relevant safety regulations and precautions are applied (see ISO 12495 annex D).

Polarization of the structure to potential more negative than –800 mV can result in the evolution of hydrogen gas at the steel surface. If the gas is allowed to collect in confined air spaces, such as cofferdams which are only part full of sea water, it may present a risk of explosion.

To avoid this hazard, the following measures should be taken:

– all designs to include an adequate venting to prevent the build up of hydrogen

– the structure to electrolyte potential to be kept at values less negative than the threshold value at which hydrogen evolution is not significant. This can be achieved by ensuring that a minimum distance should be kept between the structure and impressed current anodes.

Electrochemical reactions at the surface of impressed current anodes in sea water invariably result in the evolution of chlorine gas which is highly toxic and corrosive. If this is allowed to collect in confined air spaces above the water line, it may present a hazard to personnel and materials.

To avoid this hazard, all design shall prevent the build up of gas.

Section 7

Foundations

A. General

1. Scope

This Section applies to the foundations of fixed offshore structures and covers piled as well as gravity type designs. Requirements regarding self-elevating platforms (mobile units) see Chapter 2 – Mobile Offshore Units, Section 2.

2. Foundation design

The foundation design of fixed offshore structures shall consider all types of loads which may occur during installation and operation of the structure.

3. Soil

The soil behaviour and its interaction with the foundation shall be investigated with special regard to static, dynamic and transient loading.

4. Evolution of foundations

The design practise for foundations has proved to be very innovative and this evolution is expected to continue. Therefore circumstances may arise when the procedures described herein are insufficient on their own to ensure that a safe and economical foundation design is achieved. In such a case new requirements have to be discussed and agreed by GL.

B. Geotechnical Investigations

1. Field investigation

The field investigation shall be carried out at the actual site, defined as the area within which the structure may be installed, including accepted tolerance limits from theoretical centre. The field investigation programme shall comprise but not necessarily be limited to the following:

– site geology survey in order to establish geological conditions in general

– geophysical explorations for the purpose of extending the localized information from the borings and in situ testing

– bottom topography including registration of rocks or hard objects on the bottom

– soil sampling to ensure a sufficient number of good quality samples, that will allow the properties of all important layers to be determined

– in situ cone penetration tests (or other equivalent in situ test) performed, if possible, to a depth that will cover the critical shear zone

The actual extent of a geotechnical field investigation has to be set up individually for each site.

2. Boring vessel

2.1 Positioning system

The boring vessels shall be equipped with a positioning system which ensures an adequate accuracy of borehole positioning.

2.2 Laboratory and personnel

The boring vessel shall contain a high quality soil laboratory for classification and testing of soil specimens. The opening of samples on board, the visual inspection and the classification of the soil shall be performed by qualified geotechnical personnel.

3. Samples

The handling, storage and transport of soil samples shall be such as to retain a satisfactory sample quality.

C. General Design Considerations

1. Influences to be considered

The design of foundations shall take account of all effects which may be of influence on deformations, bearing capacity and installation of the foundation system.

2. Characteristics of soil layers

For each soil layer the physical properties shall be thoroughly evaluated with the help of in-situ and laboratory testing.

The main parameters for the foundation design are:

For sand:

– effective friction angle Φ' [degrees]

– submerged unit weight γ' [kN/m^3]

– Youngs modulus E [Mpa]

– initial soil modulus K [MN/m^3]

For clay:

– undrained shear strength c_u [kN/m^2]

– submerged unit weight γ' [kN/m^3]

– Youngs modulus E [Mpa]

– Strain at 50 % of failure [e_{50}]

3. Cyclic loads

3.1 The effect of cyclic loading, which may cause a reduction of shear strength and bearing capacity of the soil, shall be investigated in the case of gravity type foundations.

3.2 Especially for structures installed in seismically active areas the proneness of the soil to partial liquefaction and possibly resulting reduction in soil strength and stiffness have to be evaluated with great care.

4. Scour

4.1 The possibility of scour or undermining around the foundation shall always be investigated.

In case that scouring may occur,

– the foundation has to be protected by suitable means

– alternatively, the foundation has to be considered to be partly unsupported

4.2 If no other data are available for the specific site conditions, the scour depth at pile foundations may be estimated as 2,5 × d (d = pile diameter) for design purposes.

4.3 The design criteria shall be verified by regular surveys, see Chapter 1 – Classification, Certification and Surveys, Section 5. Countermeasures shall be taken in case of exceeding the limits established in the design.

5. Hydraulic stability

The hydraulic stability shall be investigated for those types of foundations which may exhibit significant hydraulic gradients within the supporting soils.

6. Soil stability

6.1 If necessary, due to existing slope or due to installation effects, the risk of slope failure or the possibility of a deep slip shall be investigated.

6.2 Total or effective stress analysis shall be used for soil stability investigations.

6.3 Especially for soft, normally consolidated clays or loose sand deposits consideration of seafloor instability is required.

6.4 Seabed movements due to action of waves, earthquake or operational effects, e.g. drilling, dredging, may cause reduced resistance or increased loading and shall be investigated.

7. Settlements and displacements

Long term settlements and displacements of the structure as well as the surrounding soil shall be analyzed.

8. Load deformation behaviour

The load deformation behaviour of a piled foundation is to be analyzed with special regard to possible structural interaction.

D. Pile Foundations

1. General

1.1 Among several existing types of pile foundations the following are most frequently used offshore:

– open ended and driven piles

– driven and underreamed piles

– drilled and grouted piles

Many other designs are used to suit the needs of the individual soil conditions.

1.2 For each type of pile foundation the soil-pile interaction and pile capacity have to be evaluated with due regard to site specific soil conditions.

2. Pile design, design methods

2.1 The design of piled foundations has to fulfil requirements for axial and lateral load capacity and stiffness.

2.2 Among others the following factors will affect the design: diameter, penetration, spacing, mudline restraint, material, pile footing and installation method.

2.3 The design method has to incorporate the pile geometry, properties and arrangement. It should be capable of simulating the non-linear properties of the soil and be compatible with the load deflection behaviour of the structure and pile foundation system.

2.4 Deflections and rotations shall be checked for individual piles and pile groups.

The design shall consider bending moments as well as axial and lateral loads.

3. Axially loaded piles

3.1 Design loads

The design of the pile penetration shall provide sufficient capacity for the maximum axial compression and tension loads. The loading conditions of Table 7.1 have to be investigated.

Table 7.1 Safety factors γ_g for different loading conditions (compare Table 3.2)

Loading conditions		Safety factor γ_g
No.	**Designation**	
2	Operating loads	2,0
3	Extreme environmental loads with specified drilling or producing loads	1,5
3	Extreme environmental loads with minimum weights	1,5

The design requirement may be expressed as follows:

$$\gamma \cdot F_A < \frac{R_K}{\gamma_m}$$

R_K = ultimate resistance

F_A = axial pile load

$\gamma = \gamma_g$ global safety factor as given in Table 7.1 when deterministic method is used

$\gamma = \gamma_f$ partial safety factor when limit state design is used

γ_m = 1,0 when deterministic method is used, see Section 3, B.3.

3.2 Calculation of ultimate resistance

For the calculation of the ultimate pile resistance various methods may be used. One acknowledged method is outlined in the following parts of this Section.

Other design methods may be used. Their applicability for the specific site conditions and their limitations shall be carefully checked and agreed upon with GL.

3.3 Ultimate resistance of piles in compression

The ultimate resistance of piles in compression R_d may be defined as follows:

$$R_d = R_f + R_p \tag{1}$$

R_f = skin friction resistance

R_p = total end bearing (tip) resistance

For plugged piles (i.e. closed end piles) R_d may be calculated as follows:

$$R_d = \Sigma f_{C0} \cdot A_0 + q \cdot A_p \tag{2}$$

f_{C0} = outer unit skin friction capacity for compression

A_0 = outer pile shaft area

q = unit end bearing capacity

A_p = gross end bearing area

$= 0{,}25 \cdot \pi \cdot d_0^2$

d_0 = outer pile diameter

For unplugged piles the following formula is to be used

$$R_d = \Sigma f_{C0} \cdot A_0 + \Sigma f_{Ci} \cdot A_i + q \cdot A_w \tag{3}$$

f_{ci} = inner unit skin friction for compression

A_i = inner pile shaft capacity area

A_W = net bearing area

$= \pi \cdot (d_0 - t) \cdot t$

t = pile wall thickness

Plugged condition is given if

$$\Sigma f_{Ci} \cdot A_i > q \cdot A_{Pi} \tag{4}$$

$A_{Pi} = 0{,}25 \cdot \pi \cdot d_i^2$

d_i = inner pile diameter

In this case equation (2) is to be used.

Unplugged condition prevails if

$$q \cdot A_{Pi} > \Sigma f_{Ci} \cdot A_i \tag{5}$$

Equation (3) applies in this case.

When calculating the design resistance of pile foundations the weight of the pile soil plug system shall be considered.

Due regard shall be given to the possibility that full resistance of skin friction and tip capacity may be mobilized at different deflections and are not necessarily additive.

3.4 Ultimate resistance of piles in tension

When calculating the ultimate resistance for pullout loads, the effective weight of the pile and the soil plug may be considered.

The pullout resistance is calculated from the expression

$$R_t = f_t \cdot A_0 + G'_s + G'_p \tag{6}$$

f_t = unit skin friction capacity for tension, see 3.5

G'_s = effective steel weight of pile

G'_p = effective weight of plug

It has to be observed that for tension loads the skin friction capacity normally is considerably lower than for compression. Unless it can be proven otherwise, the skin friction for tension should be taken not more than 2/3 of the skin friction for compression loading.

3.5 Skin friction and end bearing capacity

3.5.1 Driven piles

The design values for skin friction (f) and end bearing capacity (q) may be established either on the basis of test results or according to empirical methods.

For driven piles in cohesive and cohesionless soils a proven analytical method is described in API RP 2A [1].

3.5.2 Drilled and grouted piles

The skin friction of drilled and grouted piles in rock is limited by the shear strength of the rock. (Normally the unit skin friction will be much less than the shear strength of the rock.)

When establishing the design values, the installation methods, e.g. drilling or jetting, are to be taken into account, which may greatly affect the rock strength and rock/grout bond.

The capacity of steel/grout bond and shear key design has to be examined with great care, see 7.

3.5.3 End bearing capacity

The end bearing capacity should be specified on the basis of the triaxial shear strength. The bearing capacity factor shall be selected with good engineering judgement. Due regard shall be given to a reduction of bearing capacity in fractured rock.

3.5.4 Limits for skin friction and end bearing capacity

3.5.4.1 For silica sand the limit values shown in Table 7.2 for unit skin friction and unit end bearing capacity shall be observed unless it can be proven by in-situ tests that other values are applicable.

3.5.4.2 The selection of design parameters in calcareous sands or other soils which are easily crushable and compressible has to be done with great care because the skin friction may be reduced significantly.

The compressibility and carbonate content of calcareous soils may serve as an indicator for the risk of reduced side adhesion. Further properties, e.g. grain crushing and degree of cementation, are to be studied as a basis for the selection of the design parameters.

Table 7.2 Skin friction and end bearing capacities for silica sand

Type of soil	Unit skin friction capacity f [kPa] limit	Unit end bearing capacity q [Mpa] limit
Very dense sands and gravel	115	12,0
Dense sand and very dense silty sand	96	9,6
Medium dense sand and dense silty sand	81	4,8
Loose sand, medium dense silty sand and dense silt	67	2,9
Very loose sand, loose silt sand and medium dense silt	48	1,9

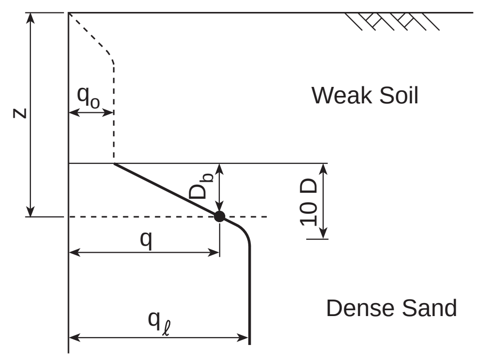
3.6 Piles driven in non-homogeneous soil

For piles penetrating non-homogeneous soils an average value of the tip resistance should be chosen as follows:

3.6.1 Pile tip penetrating a dense layer from a weak layer, see Fig. 7.1

For the unit end bearing capacity the following is valid

$$q = q_0 + D_b \cdot \frac{q_l - q_0}{10 \cdot D} \leq q_l$$



q_o limiting unit end bearing capacity in weak soil
q_ℓ limiting unit end bearing capacity in dense sand
D pile diameter
D_b depth of penetration into dense layer
z penetration depth

Fig. 7.1 Pile tip penetrating a dense layer from a weak layer

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[1] For flush piles (no inner driving shoe) the inner and outer unit skin friction can be assumed to be of the same magnitude.

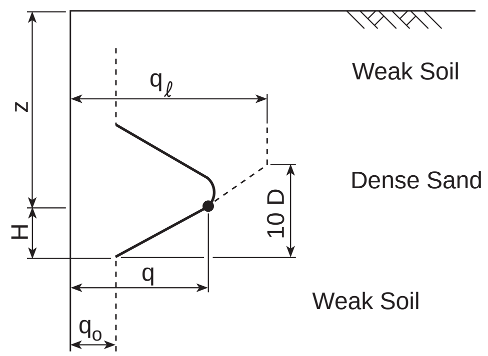
For piles with an inner driving shoe typical values for inner unit skin friction above the shoe are 50 % to 70 % of outer unit skin friction.

3.6.2 Pile tip penetrating a thin dense layer laying over a weak layer, see Fig. 7.2.

Possible punch-through effects are to be considered.

The unit end bearing capacity may be calculated as follows:

$$q = q_0 + H \cdot \frac{q_l - q_0}{10 \cdot D} \leq q_l$$



H distance of pile tip from weak soil layer

Fig. 7.2 Pile tip penetrating a thin dense layer laying over a weak layer

4. Laterally loaded piles

4.1 Task

It has to be ensured that the pile foundation is capable to sustain all static and cyclic loads acting in lateral direction.

The soil resistance in the vicinity of the mudline contributes significantly to the lateral capacity of a pile foundation.

Any disturbance of the soil e.g. due to scouring, or to the installation of piles or conductors has therefore to be considered with great care.

4.2 Design criteria

The design has to fulfil two main criteria:

4.2.1 The load-deflection behaviour has to meet the requirements of platform operability.

4.2.2 The allowable pile stresses shall not be exceeded.

4.3 p – y curve

A commonly accepted method makes use of the p-y curve. In this method the load (p)-deflection (y) relationship of the soil is established for each soil layer. Based on numerical methods the soil stiffness is used to calculate deflection and bending moments of the pile foundation system.

The p-y data may be derived using stress-strain data from laboratory tests on soil samples.

A proven numerical method to calculate the p-y curves is given in API RP 2A.

5. Pile groups

5.1 For pile groups with pile spacing of less than eight pile diameters, pile group effects have to be evaluated.

For axial pile loads the group capacity in clay may be less than the sum of the single pile capacities, whereas the group capacity in sand may be greater than the sum of the individual pile capacities.

5.2 For lateral loads the group will normally undergo larger deflection than a single pile under corresponding average loading.

5.3 The analysis of a pile group may be carried out according to acknowledged methods [2] as agreed with GL.

5.4 All piles of a pile group should end in the same soil layer.

6. Pile structure design

6.1 General

The pile structure design shall consider all types of loads which may occur during installation and operation phases.

6.2 Stresses in piles during installation

6.2.1 During pile installation the stresses including buckling shall not exceed the allowable values according to loading condition 2, see Section 3, D.

6.2.2 Consideration shall be given to axial loads and bending moments due to pile weight and full hammer weight. Bending moments due to pile batter eccentricities are to be accounted for. For vertical piles 2 % of pile and hammer weight shall be assumed as if acting laterally to account for possible impacts and eccentricities during hammer placement.

6.2.3 Consideration shall be given to the stresses that occur during driving. Combined stresses during driving, i.e. static plus dynamic portion, shall not exceed yield stress.

6.2.4 Pile wall thickness

6.2.4.1 The ratio of pile diameter and wall thickness shall be small enough so that local buckling during

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2 See e.g. Focht and Koch, OTC 1896, 1973. This procedure combines the subgrade reaction (p-y) analyses and elastic half space procedures. It results in modified p-y curves for an isolated pile to represent the single pile behaviour in a pile group.

installation is avoided. For piles driven in hard soils the following relation may serve as a guideline:

$$t = 6{,}35 + \frac{D}{100}$$

t = wall thickness [mm]

D = pile diameter [mm]

6.2.4.2 The pile wall thickness may vary along its length according to the stress level in the different sections. Thickened portions should be reasonably extended in order to allow for underdrive or overdrive.

6.2.4.3 For piles to be driven in hard soils a driving shoe should be provided with a length of about one pile diameter and a wall thickness of 1,5 times the value established according to 6.2.4.1.

6.3 Stresses in piles during platform operation

6.3.1 All loads resulting from the design load conditions of the platform are to be considered for the pile structure design.

Normally these loads will control the pile design in the mudline area.

6.3.2 The soil stiffness, see 4., has to be accounted for when calculating the bending moment distribution. Due regard shall be given to the effect of scour and lack of soil adhesion due to large pile deflections.

6.3.3 The axial load transfer may be calculated according to the "axial soil resistance - pile deflection" characteristics or alternatively according to the skin friction capacity of the pile, see 3.

The relevant safety factors specified in 3.1 have to be accounted for when calculating the axial load transfer.

6.3.4 The pile stresses during platform operation shall not exceed the allowable values according to Section 3, D.

For piles having large horizontal deflections a stress increase due to second order effects shall be considered.

6.3.5 For pile sections embedded in the soil normally column buckling need not to be investigated.

7. Grouted pile to structure connection

7.1 Platform loads may be transferred to leg and sleeve piles by grouting the annulus.

Shear keys may be added in order to improve the capacity of the steel grout bond.

7.2 Grouted pile to structure connections may be designed using parametric formulae, e.g. as provided by API RP2A. Due regard shall be given to the suitability of the formula for the geometric configuration investigated.

7.3 For geometries not covered by the existing formulae the strength of the grouted connection has to be proven by suitable calculation methods or by test.

7.4 Grout quality

7.4.1 Prior to the installation, the suitability of the grout mix has to be proven. The compressive strength has to be confirmed by laboratory tests on grout samples which were mixed and cured under field conditions.

7.4.2 The design grout strength has to be verified by a representative number of specimens taken during grouting operations. It has to be ensured that the specimens will be cured simulating in-situ conditions.

E. Gravity Type Foundations

1. General

1.1 Characterization

Gravity type foundations are characterized as foundations with relatively small penetration into the soil, compared to the width of the foundation bases, and relying predominantly on compressive contact with the supporting soil.

1.2 Skirts

If soil layers of sufficient strength are found at some depth below the sea bed, skirts may be required in order to ensure the suitability of the foundation (see e.g. 2.3.4.3). In some cases skirts will be used to protect the foundation against scouring.

1.3 Scope of design

The design of the foundation shall include the following considerations:

- stability
- static deformation
- dynamic behaviour
- hydraulic instability
- installation and removal

1.4 Design loads

The design foundation load shall not be greater than the bearing capacity for the relevant type of loading:

$$\gamma \cdot F_G \leq \frac{Q_K}{\gamma_m}$$

F_G = foundation load

Q_K = characteristic bearing capacity

γ, γ_m = see remarks in D.3.1 and Table 7.3

Table 7.3 Global Safety Factor γ

	Loading conditions acc. to Section 3, Table 3.1			
	2 Permanent loads	3 Operating loads	4 Accidental loads	On bottom stability (unpiled jacket)
Sliding	1,5	1,3	1,1	1,3
Bearing	2,0	1,5	1,25	1,5
Overturning	See 2.2			1,2
Buoyancy	1,25	1,1	1,05	1,1

The values shown in the Table may be used as well if cyclic loading effects have been taken into account. They may have to be increased when geotechnical data are uncertain.

2. Stability

2.1 Stability requirements

The foundation system shall ensure the stability in respect of overturning, bearing capacity and lateral sliding resistance.

For structures without skirts the contact stresses between foundation and soil shall always be compressive.

2.2 Overturning

For the calculation of overturning moments the most unfavourable load combination and extreme position of loads has to be accounted for.

The dimensions of foundation surfaces have to be chosen as follows:

The eccentricity e of the resultant of permanent loads shall be within the 1st core area of the bottom as defined in Fig. 7.3 to avoid gaping.

For the total loads and most unfavourable load conditions, the eccentricity e of the resultant has to be within the 2nd core area defined in Fig. 7.3. That means a maximum gaping is allowed only up to the centre of gravity of the bottom area.

For rectangular foundation areas:

Proof that the resultant is within the 1st core area:

$$\frac{e_x}{b_x} + \frac{e_y}{b_y} \le \frac{1}{6}$$

Proof that the resultant is within the 2nd core area:

$$\left(\frac{e_x}{b_x}\right)^2 + \left(\frac{e_y}{b_y}\right)^2 \le \frac{1}{9}$$

For circular foundation areas with radius r:

radius of 1st core area $r_e = 0{,}25\ r$

radius of 2nd core area $r_e = 0{,}59\ r$

2.3 Bearing capacity and sliding resistance

2.3.1 Different methods will be applicable to check the bearing capacity and sliding resistance of shallow foundations.

Provided the limitations given in 2.3.4 are fulfilled, the equations given in 2.4 and 2.5 may be used. When calculating the bearing capacity the effective foundation area according to 2.4.1 is to be used.

2.3.2 The bearing capacity and the sliding resistance of the foundation system shall be evaluated considering the following:

a) the shape of the foundation base

b) loads acting on the foundations and their variation in time

c) surface characteristics of sea bottom

d) geophysical characteristics of the soil layers concerned

e) possible rupture surfaces in the soil

f) effective stress reduction due to cyclic loading

g) pore pressure variation corresponding to the actual stress level of the soil.

2.3.3 For the bearing capacity analysis either undrained or drained condition applies.

For rapid loading, where no drainage and consequently no dissipation of excess pore pressure occur, an undrained analysis is to be performed. In this case the internal friction of soil is considered to be zero, $\Phi' = 0°$

The bearing capacity will depend on the undrained shear strength c of the soil.

Where, on the contrary, the rate of loading is slow enough, a complete drainage occurs and excess pore pressures will not develop. In this condition the bearing capacity of the foundation is determined by the

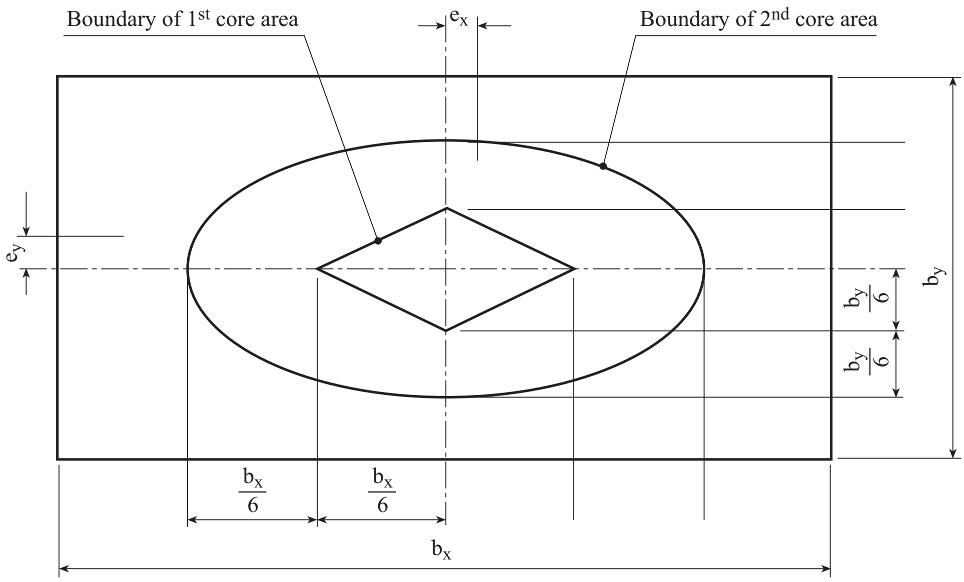


Fig. 7.3 Permissible eccentricity of permanent and total loads

drained shear strength of the soil, which may be determined from Mohr's failure envelope, i.e.

$$s = c + \sigma' \cdot \tan \Phi'$$

c' = effective soil cohesion

σ' = effective normal stress

Φ = effective angle of internal friction

2.3.4 Limitations

2.3.4.1 The formulae given under 2.4 and 2.5 may only be used provided the following limitations are observed:

– the soil is homogeneous, isotropic and fully plastic

– loading rates result in clearly drained or undrained conditions

– the actual loading is close to the simple conditions assumed in the stability formulae

– low torsional stress levels

– the foundation geometry is regular

2.3.4.2 It has to be observed that the effective soil stresses may be reduced due to cyclic loading or hydraulic gradients, induced for example by foundation rocking.

2.3.4.3 The sliding analysis according to 2.5 is only applicable where suitably spaced skirts are provided in order to ensure a horizontal failure plane in the soil rather than a failure at the interface of the foundation base and the soil.

2.3.4.4 Where the above conditions are not satisfied, more conservative methods of analysis and / or increased safety factors shall be used, or more refined techniques shall be adopted.

2.4 Bearing capacity

2.4.1 Bearing capacity, undrained condition

The maximum bearing capacity Q_V is given by the following formula:

$$Q_V = (c_u \cdot N_C \cdot K_C + \gamma \cdot d) \cdot A'$$

c_u = undrained shear strength of the soil [kN/m²]

N_C = dimensionless factor ($N_C = 5{,}14$ for $\Phi' = 0$, see Fig. 7.5)

K_C = correction factor which accounts for load inclination, shape of footing, penetration depth, inclination of the foundation base and of the ground surface, see 2.4.3

γ = total unit weight of soil [kN/m³]

d = penetration depth of foundation [m]

A' = effective area of the foundation, depending on the load eccentricity, see Fig. 7.4 [m²]

B' = effective breadth of the foundation

L' = effective length of the foundation

In the rather frequently encountered cases of

– vertical centric load

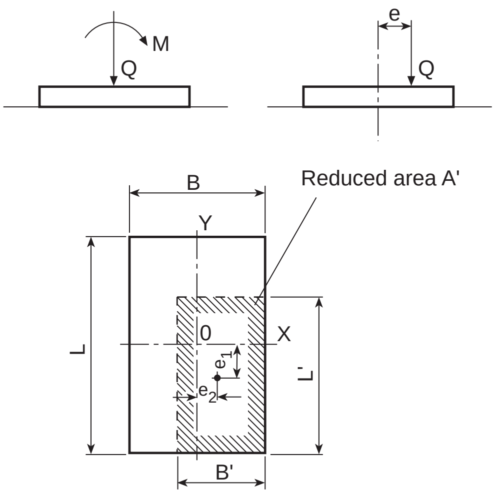
– horizontal foundation base

– horizontal sea bed

the above formula may be reduced as follows for circular or square footings:

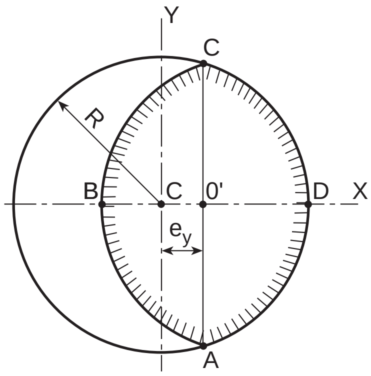
$$Q_{V0} = 6{,}17 \cdot c_u \cdot A$$

A = actual foundation area [m²]



Rectangular footings:

$A' = L' \cdot B',\ \ L' = L - 2e_1,\ \ B' = B - 2e_2$



Circular footings:

For a circular footing as shown above the effective area A' is considered to be two times the circular segment ADC:

$$A' = 2 \cdot \left[R^2 \cdot \cos^{-1}\left(\frac{e_y}{R}\right) - e_y \cdot \sqrt{R^2 - e_y^{\,2}} \right]$$

In addition, for further calculations, the effective area may be considered to be rectangular with a length-to-width ratio equal to the ratio of the line length $\overline{AC}$ to $\overline{BD}$. The effective dimensions are therefore:

$$A' = B' \cdot L'$$

where:

$$L' = \left(A' \cdot \sqrt{\frac{R + e_y}{R - e_y}} \right)^{0,5}$$

$$B' = L' \cdot \sqrt{\frac{R - e_y}{R + e_y}}$$

Fig. 7.4 Reduced footing area

2.4.2 Bearing capacity, drained condition

The maximum bearing capacity is given by the following formula:

$$Q'_V = (c' \cdot N_C \cdot K_C + q \cdot N_q \cdot K_q + 0{,}5 \cdot \gamma' \cdot B \cdot N_\gamma \cdot K_\gamma) \cdot A' \quad [kN]$$

c' = effective cohesion intercept of Mohr-Coulomb failure envelope [kN/m²]

$N_C = (N_q - 1) \cdot \cot an\, \Phi'$ see Fig. 7.5

$N_q = e^{\pi \cdot \tan\Phi'} \cdot \tan^2\left(45° + \frac{\Phi'}{2}\right)$ see Fig. 7.5

$N_\gamma = 2 \cdot (N_q + 1) \cdot \tan \Phi'$ see Fig. 7.5

Φ' = effective friction angle of Mohr-Coulomb failure envelope [°]

γ' = effective unit weight of soil (in water) [kN/m³]

q = effective overburden pressure [kN/m²]

$= \gamma' \cdot d$

d = penetration depth of foundation [m]

B = minimum lateral foundation dimension [m]

A' = effective area of foundation, see 2.4.1 [m²]

K_C , K_q , K_γ = correction factors, see 2.4.3

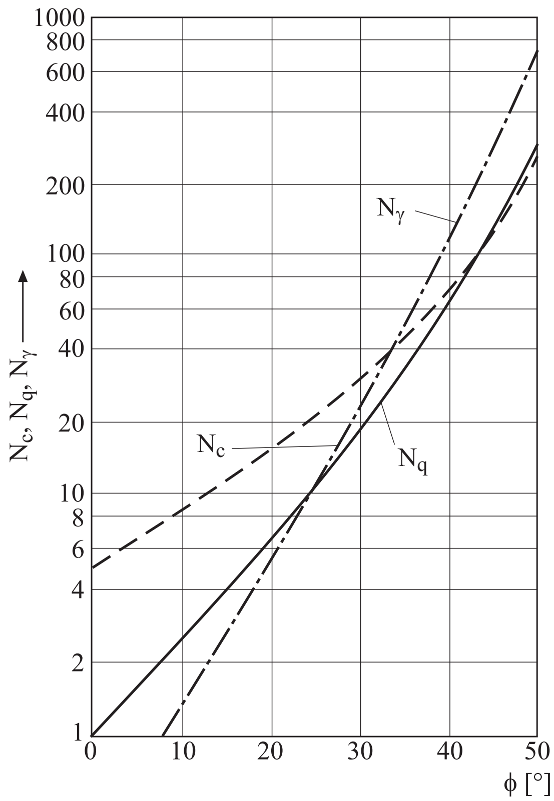


Fig. 7.5 Bearing capacity factors

For cohesionless soils and in case of centric load, simplified formulae may be used for circular or square footing, e.g.:

$$Q_V = 0{,}3 \cdot \gamma' \cdot B \cdot N_\gamma \cdot A \quad [kN]$$

B = breadth or diameter of the foundation [m]

A = actual foundation area [m²]

2.4.3 Calculation of correction factors

Correction factors K_C , K_q and K_γ are usually written as follows:

$$K_C = i_c \cdot s_c \cdot d_c \cdot b_c \cdot g_c$$

$$K_q = i_q \cdot s_q \cdot d_q \cdot b_q \cdot g_q$$

$$K_\gamma = i_\gamma \cdot s_\gamma \cdot d_\gamma \cdot b_\gamma \cdot g_\gamma$$

where i, s, d, b and g are individual correction factors related to load inclination, foundation shape, penetration, depth, base inclination and ground surface inclination, respectively.

Their recommended values are:

2.4.3.1 Inclination factors i

$$i_q = \left[1 - \frac{H}{Q + B' \cdot L' \cdot c \cdot \operatorname{cotan} \Phi'}\right]^m \qquad \Phi' > 0$$

$$i_\gamma = \left[1 - \frac{H}{Q + B' \cdot L' \cdot c \cdot \operatorname{cotan} \Phi'}\right]^{m+1} \qquad \Phi' > 0$$

$$i_c = i_q - \frac{1 - i_q}{N_C \cdot \tan \Phi'} \qquad \Phi' > 0$$

$$i_c = 1 - \frac{m \cdot H}{B' \cdot L' \cdot c \cdot N_C} \qquad \Phi' = 0$$

H = horizontal load [kN]

Q = vertical load

m = $m_L \cdot \cos^2 \delta + m_B \cdot \sin^2 \delta$

δ = angle between longitudinal axis of the base and force H

$$m_L = \frac{2 + L'/B'}{1 + L'/B'}$$

$$m_B = \frac{2 + B'/L'}{1 + B'/L'}$$

N_C = see Fig. 7.5

2.4.3.2 Shape factors s

– Rectangular base:

$$s_c = 1 + \frac{B'}{L'} \cdot \frac{N_q}{N_C}$$

$$s_q = 1 + \frac{B'}{L'} \cdot \tan \Phi'$$

$$s_\gamma = 1 - 0{,}4 \cdot \frac{B'}{L'}$$

N_q see Fig. 7.5

– Circular base with centric load:

$$s_c = 1 + \frac{N_q}{N_C}$$

$$s_q = 1 + \tan \Phi'$$

$$s_\gamma = 0{,}6$$

For a circular base with eccentric load, the formulae for an equivalent rectangular base shall be used.

2.4.3.3 Depth factors d

$$d_c = d_q - \frac{1 - d_q}{N_C \cdot \tan \Phi'}$$

$$d_q = 1 + 2 \cdot \tan \Phi' \cdot (1 - \sin \Phi')^2 \cdot \frac{d}{B'}$$

$$d_\gamma = 1{,}0$$

2.4.3.4 Base and ground surface inclination factors b and g

$$b_q = b_\gamma = (1 - \alpha \cdot \operatorname{tg} \Phi')^2 \qquad \Phi' > 0$$

$$b_c = b_q - \frac{1 - b_q}{N_C \cdot \operatorname{tg} \Phi'} \qquad \Phi' > 0$$

$$b_c = 1 - 2 \cdot \frac{\alpha}{N_C} \qquad \Phi' = 0$$

$$g_q = g_\gamma = (1 - \tan \beta)^2 \qquad \Phi' > 0$$

$$g_c = g_q - \frac{1 - g_q}{N_C \cdot \operatorname{tg} \Phi'} \qquad \Phi' > 0$$

$$g_c = 1 - 2 \cdot \frac{\beta}{N_C} \qquad \Phi' = 0$$

where α and β are the base and ground inclination angles, in radians, in respect to the horizontal plane.

2.5 Sliding resistance

The maximum sliding resistance is given by the following formulae.

– Undrained condition:

$$Q_H = c_u \cdot A \quad [kN]$$

– Drained condition:

$$Q'_H = c' \cdot A + Q \cdot \tan \Phi' \quad [kN]$$

A = actual foundation area

Q = vertical load

2.6 Soil reaction on foundation structure

Grouting of voids beneath the foundation base may be required in order to ensure the predetermined load distribution and sufficient bearing capacity. Materials and methods used for filling of voids have to be agreed upon with GL, see also Section 5.

3. Static deformations

3.1 Maximum foundation deformation

The maximum foundation deformation has to be investigated with special regard to the structural integrity and possible effects on components attached to the structure or embedded in the soil, e.g. risers, conductors.

3.2 Short time deformations for circular foundation

For the condition where the foundation of the structure base is circular, rigid, subject to static loads or to loads which may be considered as static, and rests on isotropic and homogeneous soil, the short time (undrained) deformations may be evaluated by the following formulae:

Vertical: $$u_v = Q \cdot \frac{1-\nu}{4 \cdot G \cdot R}$$

Horizontal: $$u_h = H \cdot \frac{7 - 8 \cdot \nu}{32 \cdot (1-\nu) \cdot G \cdot R}$$

Rocking: $$\delta_r = 3 \cdot M \cdot \frac{1-\nu}{8 \cdot G \cdot R^3}$$

Torsion: $$\delta_t = 3 \cdot \frac{M_t}{16 \cdot G \cdot R^3}$$

u_v, u_h = vertical and horizontal deformations [m]

Q, H = vertical and horizontal loads [kN]

δ_r, δ_t = overturning and torsional rotations [rad]

M, M_t = bending and torsional moments [kN · m]

G = elastic shear modulus of the soil [kN/m^2]

ν = Poisson's ratio of the soil

R = radius of the base [m]

3.3 Square foundations

These equations may also be used to estimate the response of square foundation shapes with equivalent area.

4. Dynamic behaviour

4.1 Dynamic loads

Dynamic loads due to waves, earthquake, etc. may significantly influence the integrity of the foundation. Their effect on the foundation behaviour has to be thoroughly evaluated.

4.2 Low stress level

In many cases, especially when the stress level is rather low, the dynamic foundation behaviour may be investigated using the continuous "half space" approach which assumes the soil to be a homogeneous, linearly elastic material.

4.3 Non-uniform soil profiles

In case of non-uniform soil profiles with the risk of energy reflection at the interfaces of soil layers, or of dynamic loads with large amplitudes which cause nonlinear soil behaviour, more appropriate analyses are required which are to be agreed by GL.

5. Hydraulic instability

5.1 Scour

Depending on current conditions (influence of the geometry of the structure) and soil conditions, measures shall be taken to prevent scouring and wash-out phenomena, e.g. with the help of scour skirts, concrete mats, rock riprap or other suitable means.

5.2 Piping

The foundation design shall take account of possible excessive hydraulic gradients in the soil with consequent formation of piping channels and a reduction of bearing capacity, see also 2.3.4.2.

6. Installation and removal of gravity foundations

6.1 Installation

6.1.1 Careful planning is necessary in order to ensure a proper installation of the foundation base.

When scour skirts and other installation aids (dowels, etc.) are required to penetrate into the sea bottom, a penetration analysis has to be performed using the soil characteristics of the relevant soil layers.

The requirements on ballasting facilities have to be investigated in order to ensure a well balanced seating of the foundation base without excessive disturbance of the supporting soil.

6.1.2 The resistance to penetration R [kN] of scour skirts and non-plugged dowels is given by their end resistance and skin friction resistance and may be calculated by the following formula:

$$R = K_p(d) \cdot A_p \cdot q_c(d) + A_S \cdot \int_0^d K_f(z) \cdot q_C(z) \cdot dz \text{ [kN]}$$

z = depth of the soil layer under consideration [m]

d = penetration depth [m]

K_p = empirical coefficient relating to end resistance

K_f = empirical coefficient relating to skin friction

q_C = cone penetration resistance [kN/m^2]

A_p = end area of scour skirt [m^2]

A_S = skin area of scour skirt per unit penetration depth [m^2/m]

For K_P and K_f the coefficients shown in Table 7.4 may be taken to calculate an upper limit of penetration resistance.

For penetration depths lower than 1 to 1,5 m, the values shown in Table 7.4 may be reduced by 25 to 50 % due to local piping or lateral movements of the platform.

When friction reducers both at the inside and outside skin of dowels are used, the coefficient K_f for sand may be reduced.

6.1.3 The end resistance of plugged dowels may be calculated in the same way as for plugged open ended piles, see D.3.3.

Table 7.4 Coefficients for penetration resistance

Type of soil	End resistance coefficient K_p	Skin friction coefficient K_f
Clay	0,6	0,05
Sand	0,6	0,003

6.2 Removal

6.2.1 It should be clarified between Owner/Operator, Designer and the relevant Administration whether, and to what extent, a removal of the structure will be required.

6.2.2 If a removal is planned, investigations are to be carried out regarding, e.g.,

– method and phases of removal

– environmental conditions

– risks involved

– necessary equipment to be provided during operations

– structural arrangements and mechanical devices (pipes, fittings, etc.) to be provided already during the construction phase, and measures required to ensure that removal operations will be possible at the expected time

Section 8

Cranes and Crane Support Structures

A. Scope of Application

1. General

1.1 This Section applies to cranes, crane pedestals and crane lashing equipment including their foundations and substructures on fixed offshore installations and mobile offshore units.

1.2 The Certification of cranes is mainly intended to confirm adequate strength of load bearing structural members and includes individual Certification of some essential mechanical components.

1.3 The Classification of cranes exceeds Certification, i.e. additionally includes individual Certification of some electrical and some more mechanical components which are deemed to be essential for a continuous and reliable operation.

2. Certified offshore installations/units

2.1 If an offshore installation/unit is to be certified by GL, also the cranes are to be certified by GL as outlined in 6. Crane support structures are covered by the Certification of the installation/unit.

2.2 Cranes which in exceptional cases will not get GL Certification have at least to be tested and examined to an extent as far as deemed necessary to verify the safety of the installation/unit as a whole.

The extent of testing and examination in such cases shall include load testing of cranes to verify the safety of crane support structures and may include, for example, other tests to verify the fire protection system of the installation or the stability of the floating unit.

3. Classed offshore installations/units

3.1 If an offshore installation/unit is to be classed by GL, cranes shall be certified by GL as outlined in 6. Upon special agreement the cranes may get an additional GL crane Class Certificate. Crane support structures are included in the Classification of the installation/unit.

3.2 Plan approval, testing and examination at the manufacturer or sub-supplier will be further extended to electrical and more mechanical components as described in 1.3.

4. Other applicable GL Rules

4.1 In addition to these Rules the GL Rules VI – Additional Rules and Guidelines, Part 2 – Life Saving Appliances, Lifting Appliances, Accesses, Chapter 2 – Rules for Lifting Appliances on Seagoing Ships and Offshore Installations (in the following abbreviated as "Lifting Appliance Rules") are the basis for Certification and Classification of cranes and for the crane support structures.

4.2 The Lifting Appliance Rules are dealing in detail with design and dimensioning requirements, plan approval, supervision during construction, Certification of components, tests and examinations as well as with accident prevention and the Certification and Classification system.

5. Further rules and regulations

5.1 For Certification, GL can agree to the application of other recognized Rules, regulations and/or standards. National regulations to be observed in addition to the GL Rules will remain unaffected.

5.2 If several rules, regulations and/or standards are to be applied, their order of valence is to be agreed with the Operator of the offshore installation/unit.

6. Crane duties

6.1 Sea lift duties

6.1.1 Certification as addressed in 1.2 is required for cranes serving sea lift duties like load handling to and from supply ships, barges or other floating bodies and subsea operations.

6.1.2 Crane Classification as addressed in 1.3 will only be carried out and certified upon special agreement with the Operator of the offshore installation/unit, see also F.3.

6.2 Platform duties

6.2.1 Deviating from 6.1.1 cranes serving platform duties only may be treated differently in agreement with the Operator of the offshore installation/unit.

In such cases it is to be distinguished between operating on the open deck or in enclosed spaces of fixed offshore installations or mobile offshore units.

6.2.2 If no special agreement is made with the Operator, GL will apply the Lifting Appliance Rules as deemed suitable. In case of cranes used only for subordinate duties, Manufacturer Inspection Certificates will be accepted. However, a thorough examination of crane support structures by the GL Surveyor and crane load testing in his presence is to be carried out on board of the installation/unit before cranes are being taken into use.

B. Environmental Conditions

1. General

The requirements defined in the following are to be used as a basis for material selection, design and dimensioning. They are of fundamental importance and are to be employed for the selection of materials and dimensioning of cranes and their support structures.

2. Environmental conditions to be applied

The environmental conditions specified for the installation/unit shall also apply to the cranes located on it.

3. GL Lifting Appliance Rules

In the absence of specified or specially agreed environmental conditions, GL will apply the requirements as stated in the Lifting Appliance Rules, except 4.

4. Crane operation in the North Sea or Arctic Regions

If the offshore installation/unit is located in these regions, the following conditions are recommended - as far as no other conditions are prescribed or requested by the Operator.

4.1 Wind speeds

The following mean wind speeds may be assumed for the purpose of determining the wind loads acting on the crane and on the crane support structure:

– 25 m/s for cranes in service

– 63 m/s for cranes out of service or in stowed position

4.2 Horizontal accelerations

4.2.1 If a crane is installed on a floating unit, the effects of the unit's dynamic motions (e.g. pitch/roll/ heave, etc.) shall be superimposed on the loads as given in the Lifting Appliance Rules.

4.2.2 The unit's motions at different wave heights for cranes in service as well as the worst case for cranes out of service shall be established by tests or motion response analysis, with the effect of the mooring system taken into consideration.

4.2.3 In the absence of any specific data for horizontal accelerations, the values stated in Table 8.1 may be used as a minimum for dimensioning of the crane, its sealashing system and their support structures.

4.3 Ice loads

In the case of possible ice accumulation, ice loads shall be included in the out of service load case. Ice loads shall be as specified for the offshore installation/unit or according to Table 8.2. Wind loads shall be determined on the basis of the increased area. In service only ice loads according to the Lifting Appliance Rules are allowed.

Table 8.1 Horizontal accelerations

Type of offshore unit	Horizontal acceleration [m/s^2] for different wave heights		
	$H_{1/3}$ = 2 m	$H_{1/3}$ = 4 m	$H_{1/3}$ = 6 m
Tension leg platform (TLP)	1,0	1,0	1,0
Semi-submersible unit	0,3	0,6	1,0
Floating Production Storage Offloading Unit (FPSO)	0,5	1,0	2,0
$H_{1/3}$ = significant wave height: average height of the highest one third of the individual wave heights in a short term constant state of sea, typically 3 hours.			

Table 8.2 Ice thickness and density

Height above sea level m	Ice thickness		Density [kg/m^3]
	Latitude 56 °N to 68 °N [mm]	Latitude North of 68 °N [mm]	
5 – 10	80	150	850
10 – 25	Linear reduction from 80 to 0	Linear reduction from 150 to 0	Linear reduction from 850 to 500
above 25	0	0	--

C. Cranes

1. General

1.1 In addition to or deviating from the Lifting Appliance Rules some special items are addressed in the following.

1.2 Failure Mode Analysis (FMA)

If defined by national regulations or by the Operator of the installation/unit, a Failure Mode Analysis is to be established on the basis of relevant rules, regulations or standards prescribed or agreed upon.

2. Cargo handling

2.1 Cranes for loading and discharging of offshore supply vessels shall be furnished with rating tables or curves which take into account the dynamics associated with the unit's and the vessel's motions.

2.2 Except when loads are determined and marked prior to lifting, each crane shall be fitted, to the satisfaction of GL, with a safety device to give the crane Operator a continuous indication of hook load and rated load for each radius. The indicator shall give a clear and continuous warning when approaching the rated capacity of the crane.

2.3 Considerations shall be given to the installation of limit switches to provide for the safe operation of the crane.

3. Conveyance of persons

3.1 General

3.1.1 Transport of persons may include:

- transfer between installations/units and other fixed or floating bodies
- transfer to and from working areas
- launching and recovery of rescue boats

3.1.2 Offshore cranes intended for the transport of persons shall fulfil relevant rules, regulations or standards as prescribed or agreed upon, but at least the following.

3.2 Safety factor

3.2.1 The SWL of cranes for the transport of persons shall be at least twice as high as the weight and the rated load of the transportation means used.

3.2.2 The SWL according to 3.2.1 is to be understood as the permissible load at the actual load radius and at wave height ($H_{1/3}$).

3.3 Secondary brake

3.3.1 In addition to the normal working brake, hoisting and luffing winches shall be equipped with a mechanically and operationally independent secondary brake with separate control circuits.

3.3.2 The secondary brake shall preferably act directly on the winch drum, but instead a fully independent load path will be considered acceptable.

3.3.3 Means shall be provided for the Operator to enable individual testing of the secondary brake.

3.4 Cylinders

3.4.1 Where cylinders are used for luffing, folding or telescoping, they shall be provided with a safety device, such as for example a shut-off valve. This safety device shall be capable of withstanding pressure shocks due to dynamic braking impact.

3.4.2 Alternatively each motion shall have two independent cylinders where each cylinder is independently capable of holding the weight and the rated load of the transportation means for persons.

3.5 Rescue of persons

3.5.1 Cranes designed for the conveyance of persons shall have an independent means for the rescue of the persons in their transportation means from any position.

3.6 Cranes/davits for survival craft/rescue boats

3.6.1 Cranes intended for launching and recovery of lifeboats or rescue boats shall comply with the requirements of the MODU Code**,** compare also Chapter 2 – Mobile Offshore Units, Section 9 and Chapter 3 – Fixed Offshore Installations, Section 5.

4. Derrick systems

Derrick systems for both in place or temporary use shall be dimensioned and designed according to the same principles as applied to cranes.

5. Crane sea lashing systems

5.1 All moving parts of cranes, especially on mobile offshore units, such as for example crane booms, machinery houses and crane gantries, shall be capable of seaworthy lashing.

5.2 Cranes in their stowed position and their stowage/lashing equipment shall be designed to withstand the combination of motions and/or wind forces applicable to the design of the offshore installation/unit on which the crane is installed.

6. Existing cranes

6.1 Existing crane Certification/Classification given by recognized organisations or IACS members may be accepted by GL. But at least a thorough examination of the crane conducted by a GL Surveyor is required. GL reserve the right to request further tests and trials.

6.2 Existing cranes without Certification according to 6.1 may get GL Certification on the basis of a thorough examination in conjunction with load and function testing conducted/supervised by a GL Surveyor.

6.3 In case of crane Classification being required a special document review as well as a specialist test and survey procedure is to be agreed with the Operator.

6.4 Missing Certificates for wire ropes, accessories and lifting attachments can only be reproduced on the basis of testing to destruction or load testing respectively.

D. Crane Support Structures

1. General

1.1 Crane support structures comprise the crane pedestals including the upper flange for the bolting of the slew ring and their connections to the steel construction of the installation/unit as well as reinforcements in way of sealashing systems.

1.2 Support structures do not include the slew rings with their bolting system or any other kind of slewing system.

2. Location

Crane support structures shall be attached at suitable locations on the installation/unit and connected to their substructures with minimal eccentricities.

3. Strength analysis

3.1 If need be crane pedestals shall be included in the analytical model of the primary structure of the installation/unit because their stiffness and crane loads may have a significant effect on load distribution and local stressing.

3.2 The angle of deflection at the top of the pedestal shall not exceed 1° for the most unfavourable case of loading.

E. Tests and Examinations on Site

1. Initial load testing and examination is required for the issuance of crane Certificates. In case of newly constructed cranes and support structures a full approval procedure in accordance with the Lifting Appliance Rules is required. For existing cranes see C.6.

2. To maintain the validity of the crane Certification, annual examination and five-yearly examination with load testing is required. The latter successfully conducted, leads to renewal of crane Certificates.

3. The initial and periodical tests and examinations shall generally be conducted as defined in the Lifting Appliance Rules if not otherwise agreed between GL and the Operator.

4. Slew rings

4.1 Slew rings which are constructed in accordance with these Rules and have got GL Certification are to be removed, dismantled and inspected internally on the occasion of the first five-year examination (non-destructive testing and examination of the running faces). Opening up may be dispensed with, if the slew rings undergo an approved condition monitoring or eddy current supervision.

4.2 Slew rings which do not fulfil the before mentioned conditions are to be inspected internally for the first time after 36 months. Depending on the result of the inspection renewed dismantling may be dispensed with entirely or a new date for dismantling will be set.

F. Documentation

1. Crane manual

A crane manual shall be provided for each crane and shall be readily available. This manual shall contain full information concerning:

- design standard, operation, erection, dismantling and transportation
- all limitations during normal and emergency operations with respect to SWL, safe working moment, maximum wind, maximum heel and trim, design temperatures and braking systems
- all safety devices
- diagrams for electrical , hydraulic and pneumatic systems and equipment
- materials used in construction, welding procedures and extent of non-destructive testing

– guidance on maintenance and periodic inspection

2. Certification

Certification of offshore cranes includes the issuance of a GL Register Book by a GL Surveyor as described in the Lifting Appliance Rules. In addition the GL Surveyor will add the test Certificate(s) for the crane(s) and the test Certificates for wire ropes, accessories and lifting attachments as the case may be.

3. Classification

In case of offshore crane Classification in addition to the crane Certification, as mentioned before, the acting GL Surveyor will make a relevant note in the Register Book and the Operator will get a crane Class Certificate from GL Head Office.

Section 9

Helicopter Facilities

A. General

1. Scope

1.1 This Section summarizes main design considerations relating to helicopter landing facilities. Aspects which are mostly aeronautically determined, like size and marking of the helicopter deck, clearances around the platform, sectors for approach and take-off have to be treated according to the relevant international and national regulations or codes, compare 2.

1.2 In this Section it is assumed that the structure of the helicopter deck is made of steel. If a structure made of aluminium alloys shall be provided, the design should follow the GL Rules I – Ship Technology, Part 1 – Seagoing Ships, Chapter 1 – Hull Structures or recognized standards, like standards of the American Petroleum Institute (API).

1.3 For electrical installations on helicopter facilities, see Chapter 6, Section 14.

2. Standards and regulations

Depending on the location of the offshore installation or the flag state of the offshore unit relevant national and international standards and regulations have to be fulfilled besides of these GL Rules. The following examples can be defined:

– ISO/CD 19901-3 Standard: Petroleum and Gas Industries – Specific Requirements for Offshore Structures – Topside Structure, 8.5

– IMO: Code for the Construction and Equipment of Mobile Offshore Drilling Units (MODU Code), Chapters 9 and 13

– IMO Res. A.855(20): Standards for on board Helicopter Facilities

– ICS (International Chamber of Shipping): Guide to Helicopter/Ship Operations

– CAP 437: Offshore Helicopter Landing Areas - Guidance on Standards, Civil Aviation Authority, Gatwick Airport South, West Sussex, RH6 0YR, UK

3. Helicopter data

For providing relevant helicopter facilities the Owner/Operator has to deliver the following information:

– types of helicopters to be operated

– geometrical main dimensions, especially length of fuselage, number and diameters of rotors, etc.

– total overall length of the helicopter when the rotors are turning (D-value)

– weight, weight distribution and wheel or skid configuration

– highest vertical rate of descent on the helicopter deck, e.g. because of a single engine failure, etc.

– data for winching operations, if applicable

– lashing systems to be provided

– possibility of an unserviceable helicopter stowed on the side of the deck while a relief helicopter is required to land, if applicable

– fuel used and type and capacity of refuelling equipment to be provided

– starting equipment, if applicable

4. Arrangement of the helicopter deck

4.1 For the arrangement of the helicopter deck the following aspects have to be considered:

– location on the installation/unit with respect to prevailing wind conditions, air turbulence and quality of the air flow due to adjacent structures

– hot gas thermal effects due to flare plumes or exhaust emissions, which may degrade helicopter performance by increasing the ambient temperature

– emergency (blowdown) systems which are designed to discharge hydrocarbon gases shall be designed that any emissions are controllable and are not initiated without consideration of sufficient warning to helicopter Operators

– clear approach and take-off sector as recommended in international or national standards, see 2.

– helidecks should be at or above the highest point of the main structure

– the obstacle-free sector should be positioned facing into the prevailing wind so that the helicopter can approach into wind with the deck in the right-hand quadrant as viewed from the helicopter and facilitating an into wind overshoot in the clear sector

– ready and protected access to and from the accommodation area without the need to pass through working areas

– effect of adjacent structures of one installation or vessel affecting the air quality and obstacle protected surfaces of another installation or vessel

4.1 In addition regarding the arrangement of the helicopter facilities within the whole installation or unit arrangement, applicable national regulations shall be observed, see 2.

5. Documentation to be submitted

5.1 Plans showing the arrangement, scantlings and details of the helicopter deck are to be submitted. The arrangement plan is to show the overall size of the helicopter deck and the designated landing area. If the arrangement provides for the securing of a helicopter or helicopters to the deck, the predetermined position(s) selected to accommodate the secured helicopter, in addition to location of deck fittings for securing the helicopter is to be shown.

5.2 The helicopter for which the deck is designed is to be specified and calculations for the relevant loading conditions are to be submitted.

5.3 Technical documentation for equipment, aviation fuel system and fire protection/fighting is to be provided.

B. Structure of the Helicopter Deck

1. General

1.1 The helicopter deck shall be dimensioned for the largest helicopter type expected to use the deck.

1.2 For scantling purposes, other loads (cargo, snow/ice, etc.) are to be considered simultaneously or separately, depending on the conditions of operation to be expected. Where these loads are not known, the data contained in 2. may be used as a basis.

1.3 The following provisions shall in principle apply to landing areas on special, pillar-supported landing decks or on the upper deck, superstructure deck or deckhouse of a fixed or mobile installation/unit.

2. Design loads

The following design load cases (LC) are to be considered:

2.1 Load case LC 1

Helicopter **lashed** on deck of mobile units, where helicopter deck is used in floating condition, with the following **vertical** forces acting simultaneously:

2.1.1 Wheel and/or skid force P acting vertically at the points resulting from the lashing position and distribution of the wheels and/or supports according to helicopter construction, see Fig. 9.1.

$$P = 0{,}5 \cdot G\,(1 + a_v)\ \ [\text{kN}]$$

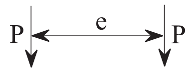


Fig. 9.1 Skid/wheel loads of a helicopter

G = maximum permissible take-off weight [kN]

P = evenly distributed force over the contact area f = 30 · 30 cm for single wheel or according to data supplied by helicopter manufacturers; for dual wheels or skids to be determined individually in accordance with given dimensions

e = wheel or skid distance according to helicopter types to be expected

a_v = acceleration addition to be defined primarily by a motion analysis

a_v = acceleration addition for non self-propelled units where the towing speed $v \le \mathbf{L}^{0,5}$ [kn] to be estimated as follows:

$a_v = 0{,}11 \cdot m$

$m = 1{,}61 - 3{,}05 \cdot x / \mathbf{L}$ $0 \le x / \mathbf{L} \le 0{,}2$

$= 1{,}0$ $0{,}2 < x / \mathbf{L} \le 0{,}7$

$= 1 + 8{,}7\,(x / \mathbf{L} - 0{,}7)$ $0{,}7 < x / \mathbf{L} \le 1{,}0$

L = length of the unit [m] as defined in Chapter 2, Section 1, B.4.

2.1.2 Evenly distributed vertical load over the entire helicopter deck, taking into account snow, cargo, personnel, etc.

$$p = 2{,}0\ \ \text{kN/m}^2$$

2.1.3 Vertical force on supports of the deck due to weight of helicopter deck M_e:

$$M_e\,(1 + a_v)\ \ [\text{kN}]$$

2.2 Load case LC 2

Helicopter **lashed** on deck with the following **vertical** and **horizontal** forces acting simultaneously:

2.2.1 Wheel and/or skid force P acting vertically at the points resulting from the lashing position and distribution of the wheels and/or supports according to helicopter construction, see Fig. 9.1.

$$P = 0{,}5 \cdot G \ \ [kN]$$

2.2.2 Vertical force on supports of the deck due to weight of helicopter deck:

$$M_e \ \ [kN]$$

2.2.3 Evenly distributed vertical load over the entire helicopter deck, taking into account snow, cargo, personnel, etc.

p = $2{,}0$ kN/m^2 for fixed installations and units

= $0{,}0$ kN/m^2 for floating units

2.2.4 Horizontal forces on the lashing points of the helicopter:

$$H = a_h \cdot G + W_{He} \ \ [kN]$$

a_h = acceleration on the helicopter in horizontal direction

= to be defined primarily by a motion analysis if no details are known, the following values may be used:

= 0 for fixed installations and units

= 0,6 for floating units

W_{He} = wind load on the helicopter at the lashing points

2.2.5 Horizontal force on supports of the deck due to weight and structure of helicopter deck:

$$H = a_h \cdot M_e + W_{St} \ \ [kN]$$

W_{st} = wind load on the structure of the helicopter deck, compare 2.3.4

2.3 Load case LC 3

Normal landing impact on **fixed** offshore installations and **floating** offshore units, with the following forces acting simultaneously:

2.3.1 Wheel and/or skid load P at two points simultaneously, at an arbitrary (most unfavourable) point of the helicopter deck (landing zone + safety zone), see Fig. 9.1.

$$P = 0{,}75 \cdot G \ \ [kN]$$

P to be increased by 15 % if the helicopter deck is part of a deckhouse with accommodations below.

2.3.2 Evenly distributed load over the entire helicopter deck, taking into account snow or other environmental loads:

$$p = 0{,}5 \ \ kN/m^2$$

2.3.3 Vertical force on supports of the deck due to deadweight of helicopter deck:

$$M_e \ \ [kN]$$

2.3.4 Horizontal force on supports of the deck due to structure of helicopter deck:

$$H = W_{St} \ \ [kN]$$

W_{st} = wind load on the structure of the helicopter deck for the wind velocity admitted for helicopter operation v_W'

v_W' = wind velocity, to be taken according to local weather conditions

= 25 m/s if no other relevant data available

2.4 Load case LC 4

Emergency/crash landing impact on **fixed** and **floating** offshore installations/units, with the following vertical force:

2.4.1 Wheel and/or skid load P at two points simultaneously, at an arbitrary (most unfavourable) point of the helicopter deck (landing zone + safety zone), see Fig. 9.1.

$$P = 1{,}25 \cdot G \ \ [kN]$$

2.4.2 Forces according to 2.3.2 to 2.3.4

3. Scantlings of structural members

3.1 Structural analysis

Structural analysis of the supporting structure shall be effected in accordance with Section 3 by direct calculations. Regarding construction and materials employed, see Section 4.

Proof of sufficient buckling strength is to be carried out in accordance with Section 3, G. for structures subjected to compressive stresses.

3.2 Permissible stresses

Permissible stresses for stiffeners, girders and substructure:

$$\sigma_{zul} = 235 / (k \cdot \gamma) \quad [N/mm^2]$$

k = material factor

= $295 / (R_{eH} + 60)$

γ = safety factor according to Table 9.1

Table 9.1 Safety factors for different load cases of helicopter decks

Structural element	γ		
	LC1 + LC2	LC3	LC4
Stiffeners (deck beams)	1,25	1,10	1,00
Main girders (deck girders)	1,45	1,45	1,10
Load bearing substructure (pillar system)	1,7	2,0	1,20

3.3 Plating

The thickness of the plating is not to be less than the greater of the two following values::

$$t = n \cdot c \cdot \sqrt{P \cdot k} + t_k \; [mm]$$

or

$$t = 1{,}1 \cdot a \cdot \sqrt{p \cdot k} + t_k \; [mm]$$

n = 1 in general

= 0,9 for crash landing of helicopters (LC 4)

P = total load in [kN] of one wheel or group of wheels with print area f on a plate panel F = a · b, see Fig. 9.2

= values for different load cases LC 1 - 4 according to 2.

a = width of smaller side of plate panel (in general beam spacing)

b = width of larger side of plate panel

F need not be taken greater than 2,5 a^2

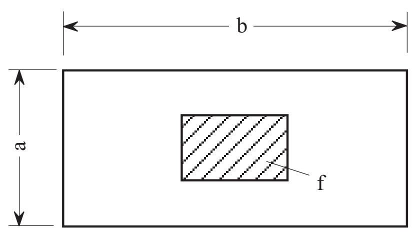


Fig. 9.2 Area of plate panel influenced by a wheel print

In case of narrowly spaced wheels these may be grouped together to one wheel print area.

t_K = corrosion allowance depending on material, protection, service and maintenance conditions, if any [mm]

c = factor to be determined as follows:

– for the aspect ratio b/a = 1 and for the range $0 < f / F \le 0{,}3$:

$$c = 0{,}95 \left\{1{,}87 - \sqrt{(f/F) \cdot (3{,}4 - 4{,}4 \cdot f / F)}\right\}$$

– for the aspect ratio b/a = 1 and for the range $0{,}3 < f / F \le 1{,}0$:

$$c = 0{,}95\,(1{,}20 - 0{,}40 \cdot f / F)$$

– for the aspect ratio b/a ≥ 2,5 and for the range $0 < f / F \le 0{,}3$:

$$c = 0{,}95 \left\{2{,}0 - \sqrt{(f/F) \cdot (5{,}2 - 7{,}2 \cdot f / F)}\right\}$$

– for the aspect ratio b/a ≥ 2,5 and for the range $0{,}3 < f / F \le 1{,}0$:

$$c = 0{,}95\,(1{,}20 - 0{,}517 \cdot f / F)$$

For intermediate values of b/a the factor c is to be obtained by direct interpolation.

p = evenly distributed pressure load [kN/m^2]

= values for different load cases LC 1 - 4 according to 2.

C. Helicopter Deck Equipment

1. Landing deck surface

1.1 Deck sheathing

The landing deck sheathing has to comply with the following requirements:

– resistant against increased mechanical impact at starting and landing procedure

– resistant against aircraft fuel, hydraulic and lubricating oils

– resistant against dry fire extinguishing powder and foams

– resistant against defrosting expedient and salt

– friction coefficient μ = 0,65 at minimum, to be checked periodically

Especially for normally unattended offshore installations guano and associated bird debris may become a major problem on the helicopter deck. Thus the friction surface may be destroyed and the essential visual aids will quickly become obliterated. Adequate cleaning operations or preventive measures have to be brought in place. No flights shall be undertaken to helicopter decks where essential visual aids for landing are insufficient.

1.2 Rope netting

Tautly-stretched rope netting should be provided to aid the landing of helicopters with wheeled undercarriages in adverse weather conditions. The intersections should be knotted or otherwise secured to prevent distortion of the mesh. It is preferable that the rope be 20 mm diameter sisal, with a maximum mesh size of 200 mm. The rope should be secured every 1,5 metres round the landing area perimeter and tensioned to at least 2225 N.

The location of the net should ensure coverage of the area of the aiming circle but should not cover helicopter deck markings.

For fixed offshore installations with no significant movements, provided the helicopter deck can be shown to achieve an average surface friction value of not less than 0,65, determined by a test method approved by GL, the helicopter deck netting may be not applied.

1.3 Helicopter lashing points

Sufficient flush fitting (when not in use) or removable semi-recessed lashing points shall be provided for fastening the maximum sized helicopter for which the helicopter deck is designed. They shall be so located and be of such strength and construction to secure the helicopter when subjected to weather conditions pertinent to the design considerations of the installation/unit. They shall also take into account, where significant, the inertial forces resulting from the movement of floating units.

1.4 Marking

The marking of the helicopter deck shall be done according to international or national regulations and standards, see A.2.

2. Wind direction indicator

A wind direction indicator shall be located on the installation/unit which, in so far as is practicable, indicates the actual wind conditions over the helicopter deck. Units on which night helicopter operations take place shall have provisions to illuminate the wind direction indicator, see Chapter 6, Section 14.

3. Personnel safety measures

3.1 Means of escape

At least two means of escape have to be provided from the helicopter deck. They shall be situated at the maximum possible distance from each other.

3.2 Safety net and railings

Safety nets for personnel protection shall be installed around the landing area except where adequate structural protection against falls exists. The netting used shall be of a flexible nature, with the inboard edge fastened level with, or just below, the edge of the helicopter landing deck. The net itself shall extend 1,5 m in the horizontal plane and be arranged so that the outboard edge is slightly above the level of the landing area, but by not more than 0,25 m, so that it has an upward and outward slope of at least 10°. For vessels subject to MODU Certification only a height of the outboard edge of 0,15 m is permissible. The net shall be strong enough to withstand and contain, without damage a 75 kg weight being dropped from a height of 1 m.

If handrails are used, they shall be retractable, collapsible and removable and painted in a contrasting colour scheme. Procedures shall be in place to retract, collapse or remove them prior to helicopter arrival. Once the helicopter has landed and the crew have indicated that passenger movement may commence, the hand rails may be raised and locked in position. The hand rails shall be retracted, collapsed or removed again prior to the helicopter take-off.

4. Drainage

Every helicopter deck shall have a drainage system which will direct any rainwater and fuel spills within its boundary to a safe place. Any distortion of the deck's surface due to, for example, loads from a helicopter at rest shall not modify the landing area drainage system to the extent of allowing spilled fuel to remain on deck. A system of guttering or a slightly raised kerb shall be provided around the perimeter to prevent spilled fuel falling on to other parts of the installation/unit and to conduct the spillage to an appropriate drainage system.

The capacity of the drainage system shall be sufficient to accept a maximum spillage of fuel on the deck. The calculation of the amount of spillage to be contained shall be based on an analysis of the helicopter type, fuel capacity, typical fuel loads and uplifts. The design of the drainage system shall preclude blockage by debris. The helicopter deck area shall be properly sealed so that spillage will only route into the drainage system.

5. Helicopter operations support equipment

Provision shall be made for equipment needed for use in connection with helicopter operations including:

– chocks and lashing strops/ropes (strops are preferable)

– heavy-duty, calibrated, accurate scales for passenger baggage and freight weighing

– a suitable power source for starting helicopters if helicopter shut-down is seen as an operational requirement

– equipment for clearing the helicopter landing area of snow and ice, if applicable, and other contaminants

D. Fire Protection and Fire Extinguishing Systems

1. Fire protection

1.1 Helicopter decks are to be of steel or equivalent fire resistant construction.

If the space below the deck is a high fire risk space, the deck shall be an "A 60" class division, compare Chapter 5, Section 10, B.

Helicopter decks of steel, aluminium or other non-combustible materials are to be constructed to the satisfaction of GL and shall be structural fire protected of at least “A-0” class as identified in Chapter 5, Section 10, B. GL may accept an air gap of at least 1 m between the deckhouse top and the underside of the helicopter deck as an alternative to the “A-0” requirement. Deckhouse tops directly below helicopter decks shall have no openings.

1.2 If aluminium or other low melting metal construction that is not made equivalent to steel are permitted, the following provisions for helicopter decks above a deckhouse or similar structure shall be satisfied:

– the deckhouse top and the bulkheads under the platform shall have no openings

– all windows under the platform shall be provided with steel shutters

– the required fire fighting equipment shall be provided according to 2. and be to satisfaction of the Administration

1.3 Safe drainage of fuel spills shall be provided according to C.4.

2. Fire extinguishing

In close proximity to the helicopter landing deck there shall be provided and stored near the means of access to that deck:

2.1 Helicopter facilities with no refuelling capabilities

2.1.1 At least two nozzles of dual-purpose type and hoses sufficient to reach any part of the helicopter deck. The operation of the fire main is not to interfere with the simultaneous operation of the foam system.

2.1.2 At least two dry powder extinguishers having a total capacity of not less than 45 kg:

2.1.3 CO_2-extinguishers of a total capacity of not less than 18 kg or equivalent. One of these extinguishers shall be so equipped as to enable it to reach the engine area of any helicopter using the deck. The back-up system is to be located so that the equipment would not be vulnerable to the same damage as the primary extinguishing system.

2.2 Helicopter facilities with refuelling capabilities

2.2.1 Fire fighting systems as defined in 2.1 and so arranged as to adequately protect both the helicopter deck and fuel storage areas.

2.2.2 A fixed low expansion foam system with monitors or foam making branch pipes capable of delivering foam to all parts of the helicopter deck in all weather conditions in which helicopters can operate. The system shall be capable of delivering a discharge rate of not less than 6 l/m² for at least 5 minutes for each square metre of the area contained within a circle of diameter D. D is the distance [m] across the main rotor and tail rotor in the fore-and-aft line of a helicopter with a single main rotor and across both rotors for a tandem rotor helicopter.

The foam agent shall meet the performance standards of ICAO 24 and be suitable for use with salt water.

GL may accept other fire-fighting systems which provide a fire-extinguishing capability at least as effective as the required foam application system.

2.3 Additional equipment

2.3.1 Two fireman's outfits with breathing apparatus in addition to those required by national regulations.

2.3.2 At least the following equipment, stored in a manner that provides for immediate use and protection from the environment:

– adjustable wrench

– blanket, fire resistant

– cutters bolt 600 mm

– hook, grab or salving

– hacksaw, heavy duty complete with 6 spare blades

– ladder

– life line 5 mm diameter × 15 m in length

– pliers, side cutting

– set of assorted screwdrivers

– harness knife complete with sheath

– rescue axe, large (non wedge or aircraft type)

– crowbar, large

– gloves, fire resistant

Sizes of this equipment shall be appropriate for the types of helicopter on which the design is based.

2.4 For further information see also Chapter 5, Section 10.

E. Aviation Fuel System

1. General

These Rules apply to aviation fuel with a flash point above or below 60° C.

2. Storage

2.1 General

2.1.1 For the storage of aviation fuel the general safety measures for fuel tanks are to be applied analogously, compare Chapter 5, Section 13a.

2.1.2 The aviation fuel has to be stored in separate dedicated tank(s). Fuel storage tanks are to be of approved metallic construction. Special attention is to be given to the design, mounting and securing arrangements and electrical earthing of the tank.

2.2 Arrangement of tanks

The arrangement of aviation fuel tanks has to comply with the following requirements:

– tanks have to be located as far as practicable from accommodation spaces, escape routes, embarkation stations and machinery spaces

– tanks have to be isolated by cofferdams from areas containing sources of vapour ignition

– tanks and associated equipment shall be protected against physical damage and from a fire in an adjacent space or area

– no fuel tanks are to be arranged forward of the collision bulkhead, only applicable for offshore units

– aviation fuel tanks may not be arranged directly at the shell or directly besides other tanks

– the fuel storage area shall be provided with arrangements whereby fuel spillage may be collected and drained to a safe location

– the storage and handling area is to be permanently marked

2.3 Tank equipment

2.3.1 The filling and outlet pipes, the sounding equipment, the mounting of devices and fittings as well as the ventilation and overflow equipment has to be provided in accordance with Chapter 5, Section 13a and 13b.

2.3.2 If the flash point of the fuel is below 60 °C the following requirements have to be complied with:

– Venting pipes have to be provided with pressure vacuum valves and flame arrestors of approved type. The location of the openings in relation to any source of ignition has to be approved according to the area classification, compare Chapter 6, Sections 13 and 14.

– Electrical equipment has to be explosion-protected, compare Chapter 6, Section 14.

2.4 Where portable fuel storage tanks are used, special attention shall be given to:

– design of the tank for the intended purpose

– mounting and securing arrangements

– electrical earthing

– inspection procedures

3. Fuel transfer system

3.1 General

3.1.1 For the handling of aviation fuel on the installation/unit, separate piping systems are to be provided, which are not connected to other fuel systems. For these Rules it is assumed that the refuelling is done on the helicopter landing deck.

3.1.2 The following functions are required:

– filling of the aviation fuel tank(s) of the installation/unit

– discharging from any of the tanks via the connections, with the fuel transfer pump

– transfer of fuel between any of the aviation fuel tanks, using the transfer pump, if applicable

– refuelling of the helicopter from the aviation fuel tank, using the refuelling pump

– flushing of the refuelling hoses to the aviation fuel tank

3.2 Piping and pumping arrangements

3.2.1 Electrical fuel pumping units and associated control equipment shall be of a type suitable for the location and potential hazard.

3.2.2 The tank outlet valve has to be directly at the tank. It has to be a quick-closing valve capable of being closed remote-controlled.

3.2.3 The fuel pumping unit shall be connected to one tank at a time and the piping between the tank and the pumping unit shall be of steel or equivalent material, as short as possible and protected against damage.

Piping connections have to be of approved type.

3.2.4 Compensators and hoses have to be of steel or have to be flame-resistant and have to be of approved type.

3.2.5 Piping and pumping arrangements have to be firmly connected to the hull/installation structure.

3.2.6 The pump has to be able to be controlled from the refuelling station. Fuel pumps shall be provided with means which permit shutdown from a safe remote location in the event of a fire. Where a gravity-fed fuelling system is installed, equivalent closing arrangements shall be provided.

3.2.7 A relief device has to be provided which prevents over-pressure in the refuelling hose.

3.2.8 The following items have to be provided in the system:

– fuel metering

– fuel sampling

– filters

– water traps

4. Requirements for the room containing the pump and filter unit (pump room)

The following requirements have to be met:

– The bulkheads and decks have to be of steel and have to be insulated to "A 60" standard towards adjoining spaces.

– Access to the room is only permitted from the open deck. There is no access permitted to other spaces from this room.

– The room has to be provided with a fire detection system and a fixed fire extinguishing system which can be released from outside this room.

– The room has to be provided with a mechanical ventilation of the extraction type which is separate from any other ventilation system. The fans have to be of non-sparking design. The capacity of the ventilation has to be sufficient for 20 air changes per hour, based on the gross volume of the room.

– Inside the room only explosion protected equipment is permitted (IIA, T3).

– Up to a distance of 3 m from openings to the room, possible sources of ignition and openings to other rooms containing possible sources of ignition are not permitted.

– An emergency shutdown of the pumps and release of the quick-closing valves have to be provided from a position located outside the pump room close to the refuelling station.

– Drip trays have to be provided below components where leakage can occur.

5. No smoking

"NO SMOKING" signs are to be displayed at appropriate locations.

6. Procedures and precautions

The procedures and precautions during refuelling operations shall be in accordance with good recognized practice.

F. Requirements for Winching

1. Winching operations

1.1 For any fixed offshore installation or any mobile offshore unit, attended or unattended, for which helicopters are a normal mode of transport of personnel, a helicopter landing area should be provided. Winching should not be adopted as a normal method of transfer.

1.2 If a regular delivery of supplies, like provisions, spare parts, etc., is planned, measures for a convenient material flow from the winching area has to be provided.

1.3 If winching operations are required, they shall be conducted in accordance with procedures agreed between the helicopter Operator, the Owner/ Operator of the offshore installation/unit and GL and shall be contained in the Operating Manual.

2. Winching areas

2.1 A winching area should, for operational effectiveness and safety, be located at the side or one end of an offshore installation/unit so that a large part of the manoeuvring zone can extend outside the installation/unit. The position of the operating area shall enable the pilot of the helicopter hovering over the winching area to have an unobstructed view of the installation/unit and be in position which will minimize the effect of air turbulence and flue gases. The area shall, as far as possible, be positioned clear of accommodation spaces, provide an adequate deck area for material and provide for safe access to the area from different directions.

2.2 In selecting a winching area the desirability of keeping the winching height to a minimum shall also be borne in mind. In routine operations a winching height greater than 12 m shall be avoided.

2.3 A winching area shall provide a "manoeuvring zone" in which a clear zone shall be centred. The sizes of these areas are to be defined by the state of location or flag state of the offshore installation/unit.

3. Winching above accommodation areas

Some installation/units may only be able to provide winching areas which are situated above accommodation spaces. Due to the constraints of operating above such an area only twin-engined helicopters shall be used for such operations and the following procedures adhered to:

– personnel shall be cleared from all spaces immediately below the helicopter operating area and from those spaces where the only means of escape is through the area immediately below the operating area

– safe means of access to and escape from the operating area shall be provided by at least two independent routes

– all doors, ports, skylights, etc. in the vicinity of the helicopter operating area shall be closed. This also applies to deck levels below the operating area.

– fire and rescue parties shall be deployed in a ready state but sheltered from the helicopter operating area

Section 10

Marine Operations

A. General

1. Marine operations, i.e. operations associated with moving or transporting an offshore structure or parts thereof during the construction, installation or abandonment process, may have decisive influence on the overall design and on the dimensioning of scantlings, see also Chapter 1 – Classification, Certification and Surveys, Section 1, B.5.2; Chapter 2 – Mobile Offshore Units, Section 1, C. and Chapter 3 – Fixed Offshore Installations, Section 1, C. Marine operations concerning structural integrity are therefore normally included in the design review by GL.

2. Where such operations, i.e. loadout, transportation, lifting/launching, upending, levelling, piling, floatover/lifting, mating and lowering/embedding, impose important loads or critical conditions on the structure, they have to be taken account of in the design.

The review and survey by GL will cover, as far as applicable:

– Location Survey and environmental conditions

– Loadout Analyses

– Transportation Analyses including motion response

– Lifting/Launching/Upending Analyses including on-bottom stability

– Lifting/floatover, Lowering and Touchdown Analyses including dynamic influences

Additional to above investigations Marine Warranty Survey (MWS) document review and on-site surveillance will cover as far as applicable:

– Condition Survey of vessels and equipment involved

– Loadout Procedure including quay condition, mooring and retrieval system

– Transportation Arrangement including seafastening, stability, towing

– Lifting/Launching/Upending Procedure (rigging, mooring, clearances)

– Installation Procedures including anchor handling, positioning, piling, grouting, mating

3. A complete Certification of the design and construction of an offshore installation will be based on surveys and attendance to all important transport and installation operations. The necessary extent of such attendance will be agreed upon in each individual case, and certified accordingly. However, the operations shall be conducted by the responsible personnel of the service company in charge, which is assumed to be competent and experienced for the respective tasks.

4. The Operating Manual shall cover all relevant procedures and limiting conditions, and is recommended to be reviewed and approved by GL, see Chapter 2 – Mobile Offshore Units, Section 1, C.3.1 and Chapter 3 – Fixed Offshore Installations, Section 1, C.3.1.

5. Construction phases of a (e.g. concrete) platform in a floating, moored condition are not understood to be within the scope of this Section.

B. Standards and Guidelines

Within the scope of Certification and Marine Warranty Survey the following standards are accepted by GL:

– NORSOK STANDARD: Applicable Chapters regarding Marine Operations

– ISO 19901-6:2004: Marine Operations (Draft)

– GL Rules VI – Additional Rules and Guidelines, Part 11 – Other Operations, Chapter 1 – Guidelines for Ocean Towage

– London Offshore Consultants Ltd./ Oilfield Publications Ltd.: Guidelines for Marine Operations

– Noble Denton International Ltd, London: Applicable Guidelines regarding Marine Operations

C. Loadout

1. Basic information

Loadout means the loading operation of structures from quay to board of a vessel/cargo barge, from quay into the water or from board of a vessel/cargo barge to board of another vessel/cargo barge.

To check the loadout condition of the structure the following information has to be available:

For skidding loadout sufficient support / structural integrity is to be checked based on fail or loss of one support. The settlement of the support is to be assumed by ± 25 mm.

2. Choice of loadout type

Considering the overall situation, the type of the loadout has to be chosen from existing possibilities.

2.1 Floating loadout

When loadout is established by floating up the structure, the following has to be considered:

– the barge to be used for floating up the structure shall be classed by a recognized Classification Society

– the loads induced on the barge during the loadout shall be checked for not exceeding the design level

– if tide influences the floating operation an easily readable tide gauge shall be provided adjacent to the loadout location

– the ballasting/deballasting system of the barge shall be able to compensate tidal influence during loadout

– weather and weather window shall be adequate for the floating up operation

2.2 Lifted loadout

Where lifted loadouts are used, the following has to be considered:

– weight control shall be performed by means of a well defined, documented system in accordance with current good practice

– the structure including the padeyes shall be analysed for the loads and reactions imposed during the lift

– loads imposed by shore-based cranes on the quay shall be within the allowable limits

– floating cranes shall be safely moored, if applicable

– the lifting operation shall be within the approved load curve of the crane

– after the loadout the padeyes and their connection into the structure shall be examined by the Surveyor and access thereto available

– minimum clearances between lifted items, spreader bar (if any) and crane boom shall be 1 m for onshore loadouts

2.3 Skidded loadout

Where skid shoes on a skidway are used for loadout, the following has to be considered:

– shear forces and bending moments on the bearing structure shall be within the limits defined by the designer

– maximum loadings on the skid shoes shall be within the limits defined by the designer

– sufficient articulation of skid shoes shall be provided to compensate for level and slope changes

– the skidway shall establish the vertical alignment of quay, linking beam and barge, tolerances on linking beam movement because of possible barge movement shall be defined

– lateral guides are to be provided along the full length of the skidway

– the pressures transferred by the skidway on quay, linking beam and barge shall be within allowable values

– a contingency plan shall be made for failure of the devices enabling the transfer movement (retrieval system by wire and winch, hydraulic jacks, strand jacks)

– active mooring lines to be steel wires

2.4 Loadout by trailers

Where trailers are used for loadout, the following has to be considered:

– shear forces and bending moments on the structure of the trailer shall be within the limits defined by the trailer manufacturer

– sufficient trailer stability shall be ensured to prevent toppling due to uneven surfaces and inclination

– maximum axle loadings shall be within the limits defined by the trailer manufacturer

– vertical alignment of quay, linking beam and barge shall be within a third of the hydraulic travel possibility of the wheels of the trailer; tolerances on linking beam movement because of possible barge movement shall be defined

– the footprint pressures of the trailer in the loadout area, linking beam and barge shall be within allowable values

– a contingency plan shall be made for the case of hydraulic leakage or power pack failure for propelled trailers or failure of wire and winch systems for non-propelled trailers

2.5 Grounded loadout

If local circumstances demand to ground the barge for the loadout operation, the following requirements have to be observed:

– the seabed, where the barge shall be grounded, has to be surveyed for the levels and the existence of debris on the ground

– in case the seabed levels are not equal enough, the structure of the barge has to be calculated for the unequal load distribution

– the barge shall be ballasted to ensure sufficient ground contact

– the skidway levels shall be controlled during loadout

– a plan has to be prepared for float-off operations after the loadout

3. Detailed investigation of loadout operation

For the loadout operation the following circumstances have to be assumed and respectively have to be investigated:

3.1 Sufficient strength of quay, linking beam and grillage.

3.2 Barge parameters are:

– adequate ballasting and stability of the barge with a minimum metacentric height of 1,0 m during loadout

– proven structure and strength of the barge for the actual transport task with a loading plan approval by a recognized Classification Society

– a maximum draft of the barge according to load line marking, which shall not be exceeded

– a clearance under the keel of 1,0 m shall be kept for floating operations. The minimum under keel clearance of 0,5 m is allowed only in lowest water levels and after underwater survey of the loadout area

3.3 Adequate retrieval and mooring systems have to be used.

– The safety factor for mooring wires shall be equal to $f_{moor} = 3$ based on the acting forces.

– retrieval system with a capacity equal to the pulling system

3.4 For transfer operations during skidded loadout the friction values according to Table 10.1 may be used.

Table 10.1 Coefficients of friction for sliding and rolling

Type of movement	Surface combination		Type of travel	
	Surface 1	Surface 2	Static	Dynamic
Sliding	Steel	Steel	0,22	0,16
	Steel	Teflon	0,18	0,07
	Stainless steel	Teflon	0,15	0,06
	Teflon	Wood	0,19	0,07
	Steel	Waxed wood	0,15	0,09
Rolling	Steel wheels	Steel	0,01	0,01
	Rubber tyres	Steel	0,02	0,02
	Rubber tyres	Asphalt	0,03	0,03
	Rubber tyres	Gravel	0,03	0,03

D. Transportation

1. Seafastening

For seafastening the following aspects have to be considered:

– design effects of the seafastening members to the structure to be investigated

– sufficient structural integrity of the seafastening members to be checked based on accelerations given in GL Guidelines VI – Additional Rules and Guidelines, Part 11 – Other Operations and Systems, Chapter 1 – Guidelines for Ocean Towage and using safety factors according to loading condition 2 in Section 3, Table 3.2

These are:

γ_g = 1,45 for axial and bending stresses

γ_g = 2,16 for shear stresses

γ_g = 1,25 for equivalent stresses

– no seafastening members are to be located on unsupported fields (stiffener support is required)

– the safety factor for lashing with wire ropes shall be equal to $f_{LASH} = 2{,}7$

$$F_{perm} \leq MBL / f_{LASH}$$

F_{perm} = permissible lashing force

MBL = minimum breaking load

– Non-destructive testing (NDT of welds on the seafastening member) to be carried out by recognized Third Party.

2. Towing

2.1 Sufficient structural integrity of the structure is to be checked based on the accelerations defined in the GL Guidelines VI – Additional Rules and Guidelines, Part 11 – Other Operations and Systems, Chapter 1 – Guidelines for Ocean Towage.

Towing applies for transportation on board of barges or for self floating structures (jackets or gravity bases).

For towing an adequate tug with sufficient bollard pull has to be provided. The required continuous static bollard pull of the tug is to be calculated based on the total environmental load acting on the barge and cargo as follows:

– wind speed = 40 kn

– significant wave height = 5 m

– current velocity = 1 kn

2.2 Cargo Barge Condition Survey

The following items will be checked by Marine Warranty Surveyor (MWS), if applicable:

– Class Certificate of a recognized Classification Society
– International Load Line Certificate
– towing equipment functionality
– availability of emergency towline (positioned, accessible, seafastened)
– barge ballasting system
– emergency bridle recovery line and buoy
– navigation lights working

2.3 Towing Vessel Condition Survey

The following items will be checked by MWS, if applicable:

– Class Certificate of a recognized Classification Society
– International Load Line Certificate
– IOPPC Certificate
– Safety Radio and Safety Equipment Certificate
– Bollard Pull Certificate
– main and spare towing wire Certificate according to 2.4
– navigational equipment and bridge machinery controls work efficiently
– suitable weather forecast available on board
– work boat or Zodiac with fuel is recommended for fast access to the barge

2.4 Towing arrangement

As part of the towing arrangement wires, chains, shackles, towing rings and delta plates/triangle plates are to be certified by a recognized organisation.

Requirements for main and spare towing wires:

– adequate length depending on location:

  Normal sea areas ≥ 900 m

  Benign sea areas ≥ 500 m

– MBL ≥ 3 × bollard pull of the tug when bollard pull of the tug < 40 t
– MBL ≥ 2,5 × bollard pull of the tug when bollard pull of the tug = 40 t to 90 t
– MBL ≥ 2 × bollard pull of the tug when bollard pull of the tug > 90 t
– hard eye thimbles

Requirements for towing bridle:

– two legs with an angle of 45° - 60° between the legs
– MBL of each leg ≥ 3 × bollard pull of the tug

Pennant wire with same strength of towing wire should be used between main towing wire and bridle connection.

2.5 Towing of gravity bases

For the tow to the mating or offshore site following requirements to be considered:

– metacentric height to be minimum 1 m
– maximum inclination not to exceed 5° taking the static inclination due to full towline pull or due to wind speed 20 m/s and maximum motions due to seastate H_s = 5 m into account
– average speed with 10 m/s head wind in seastate H_s = 2 m to be at least 1,5 knots

2.6 Towing of floating structures

For the tow of floating structures following requirements are to be considered:

– metacentric height to be a minimum of 1 m
– the structure has to be surveyed for integrity/leakages and water-tight/weather-tight closure means before commencing the towage
– fixings for towing wires and mooring chains shall be designed for a force 20 % in excess of the MBL of the wire/chain
– failure of a fixing point shall not lead to structural damage with possible downflooding

E. Lifting Operations

1. General

1.1 Design considerations

Overall dimensions, weight and centre of gravity of a module to be lifted, number of lifting points, arrangement and characteristics of lifting equipment, etc. should be considered at an early design stage in view of the means which will be available, and of the static and dynamic loads associated with the anticipated procedure.

1.2 Calculations

1.2.1 Parameters

The analysis of the lifting and lowering procedures shall take into account, where adequate, the elastic properties of both, the structure itself and the lifting equipment, the dimensional tolerances of the lifting slings, dimensional and weight tolerances of the lifted object, and the dynamic conditions which depend on the environment and the type of lifting gear employed. The weight of the rigging/lifting equipment (e.g. spreaders) shall be accounted for where relevant.

1.2.2 Additional external influences

Additional external influences and loads such as hydrodynamic and hydrostatic forces acting on submerged structures, wind, tug forces and foreseeable impact forces shall be considered where they are deemed to be important.

2. Load assumptions

2.1 Weight contingency factors

For lifted objects a weight contingency factor f_{CONT} [-] shall be considered as follows:

f_{CONT} = 1,03 for weighed weights

= 1,10 for calculated weights

For rigging equipment a rigging weight contingency factor f_{RIG} shall be considered as follows:

$$f_{RIG} = 1{,}03$$

2.2 Dynamic amplification f_{DAF}

2.2.1 A dynamic amplification factor f_{DAF} [-] depending on the mass M [t] of the lifted object according to Table 10.2 has to be considered for any controlled lifting operation using one hook.

2.2.2 In any case the dynamic amplification factor can be evaluated in a detailed motion analysis.

2.2.3 For lifting and lowering procedures involving floating objects and/or two or more crane barges, a special investigation of the motion characteristics of the coupled system will usually be necessary, unless it can be shown that the maximum forces likely to occur are sufficiently smaller than the lifting capacity provided.

Table 10.2 Dynamic amplification factor f_{DAF}

Mass	Unprotected areas offshore [1]	Sheltered areas (inshore)[1] and onshore
M ≤ 1000 t	$f_{DAF} = 1{,}15 + 0{,}15 \cdot \left(1 - \frac{M}{1000}\right)$	$f_{DAF} = 1{,}05 + 0{,}10 \cdot \left(1 - \frac{M}{1000}\right)$
M > 1000 t	1,15	1,05

[1] it is assumed that operations are carried out under defined, controlled weather conditions

2.3 Lifting weight W_L

The lifting weight is defined as follows:

$$W_L = M \cdot g \cdot f_{CONT} \cdot f_{DAF} \ \ [\text{kN}]$$

M = mass [t], for the lifted object

g = gravity acceleration (9,81 m/s^2)

f_{CONT} = weight contingency factor according to 2.1

f_{DAF} = dynamic amplification factor according to 2.2

2.4 Rigging weight W_R

The rigging weight is defined as follows:

$$W_R = M_R \cdot g \cdot f_{RIG} \cdot f_{DAF}^{*} \ \ [\text{kN}]$$

M_R = mass of all rigging equipment (slings, shackles, etc.)

g = gravity acceleration (9,81 m/s^2)

f_{RIG} = rigging weight contingency factor acc. to 2.1

f_{DAF} = dynamic amplification factor according to 2.2

* same f_{DAF} as for W_L to be applied

2.5 Hook load H

The hook load H may be determined as follows:

$$H = W_L + W_R \ \ [\text{kN}]$$

2.6 Inaccuracy of centre of gravity f_{COG}

Depending on the type of structure and possible accuracy of calculation (design stage), a tolerance factor f_{COG} [-] shall be chosen to take into account errors regarding the position of centre of gravity (COG).

Standard procedure should be the consideration of the worst position of the COG inside an envelope of 1 m × 1 m.

Possible tilting effects have to be considered.

Where this procedure can not be applied a minimum factor of

$$f_{COG} = 1{,}10$$

has to be considered.

2.7 Skew load factor f_{SKL}

The skew load factor f_{SKL} [-] is applied to account for dimensional tolerances of the slings and the influence of elasticity of slings length or properties.

For indeterminate (4-sling) lifts a skew load factor of:

$$f_{SKL} = 1{,}25$$

shall be applied to each diagonally opposite pair of lift points in turn.

For determinate (2- and 3-point) lifts the skew load factor is:

$$f_{SKL} = 1{,}00$$

2.8 Lift point resolved lift weight P_{RLW}

The lift point resolved lift weight P_{RLW} [kN] is the vertical load on each lift point (padeye / trunnion) taking into account the lift weight W_L and the geometric arrangement of the lifting points in relation to the COG only.

2.9 Resolved lift point load (sling load) P_S

The sling load P_S [kN] is the total load acting on the padeye in direction of the sling:

$$P_S = \frac{P_{RLW} \cdot f_{COG} \cdot f_{SKL}}{\sin(\alpha)}$$

α = angle between the sling and the horizontal plane.

2.10 Additional influences

Additional influences e.g. from horizontal forces, see 1.2.2, shall be especially considered. At the points of load application (padeyes), a force of at least 5 % of P_S is to be accounted for, acting perpendicular to the padeye plane.

2.11 Spreader frames

When using spreader frames, the sling forces may be calculated according to 2.9 and 2.10, α being the angle between sling and (horizontal) spreader plane.

3. Safety factors

3.1 Safety factors for lift points and lifted objects

3.1.1 Consequence factors f_{cons}

The following consequence factors f_{cons} [-] shall be applied to the lift points and supporting structure.

f_{cons} = 1,35 for lift points, attachments and supporting members, including those on spreaderframes

f_{cons} = 1,00 for other structural members

3.1.2 Global safety factor γ_g

The global safety factor γ_g is to be chosen according to Section 3, Table 3.2, for loading condition 7.

These are:

γ_g = 1,45 for axial and bending stresses

γ_g = 2,16 for shear stresses

γ_g = 1,25 for equivalent stresses

3.2 Safety factors for lifting equipment

For the safety factor of lifting equipment see 5.

4. Design of lifted object and lift points

4.1 General

The structure to be lifted, including the elements introducing the loads into the structure shall be designed according to the principles laid down in Section 3 and Section 4. The permissible stresses and safety factors, respectively, should generally be taken according to loading condition 7. See also 3.1 and Section 3, D.2.

4.2 Shock loading

Elements which may be subjected to shock loading during the lifting and lowering procedure shall be especially considered and duly strengthened. Protection by guides and in special cases fendering may be necessary.

4.3 Arrangement of padeyes

Padeyes shall be so arranged that the sling direction lies in the plane of the padeye main plate. A bending force according to 2.10 in the direction of the eye axis is to be considered. The padeye main plate shall be introduced into the structure, avoiding sling forces to be transmitted across girder or deck plates.

Where through thickness loading of plates cannot be avoided, the material used shall be documented to be free of laminations, with a recognized through-thickness designation.

Pinholes should be bored/reamed, and shall be designed to suit the shackle proposed. Adequate spacer plates shall be provided to centralise shackles.

4.4 Design of standard padeyes

Padeyes of usual configuration shall comply with the following indications, see Fig. 10.1.

The following formulae are valid for a relation between the pin diameter of the proposed shackle (d_p) and the diameter of the pinhole (d) of:

$$\frac{d_p}{d} \geq 0{,}95$$

4.4.1 Section 1-1:

Shear stress:

$$\tau = f_{cons} \frac{P_s \cdot 1000}{2 \cdot A_{S1}} \leq \frac{R_{eH}}{\gamma_g} \quad [\text{MPa}]$$

A_{S1} = shear area of Section 1-1, [mm²] related to force P_s (for $D / 2R \geq 0{,}8$, $A_{S1} = A_1$)

A_1 = area of Section 1-1 [mm²]

R_{eH} = minimum nominal upper yield strength [MPa]

γ_g = global safety factor for shear stresses according to 3.1
= 2,16

f_{cons} = consequence factor for lift points according to 3.1
= 1,35

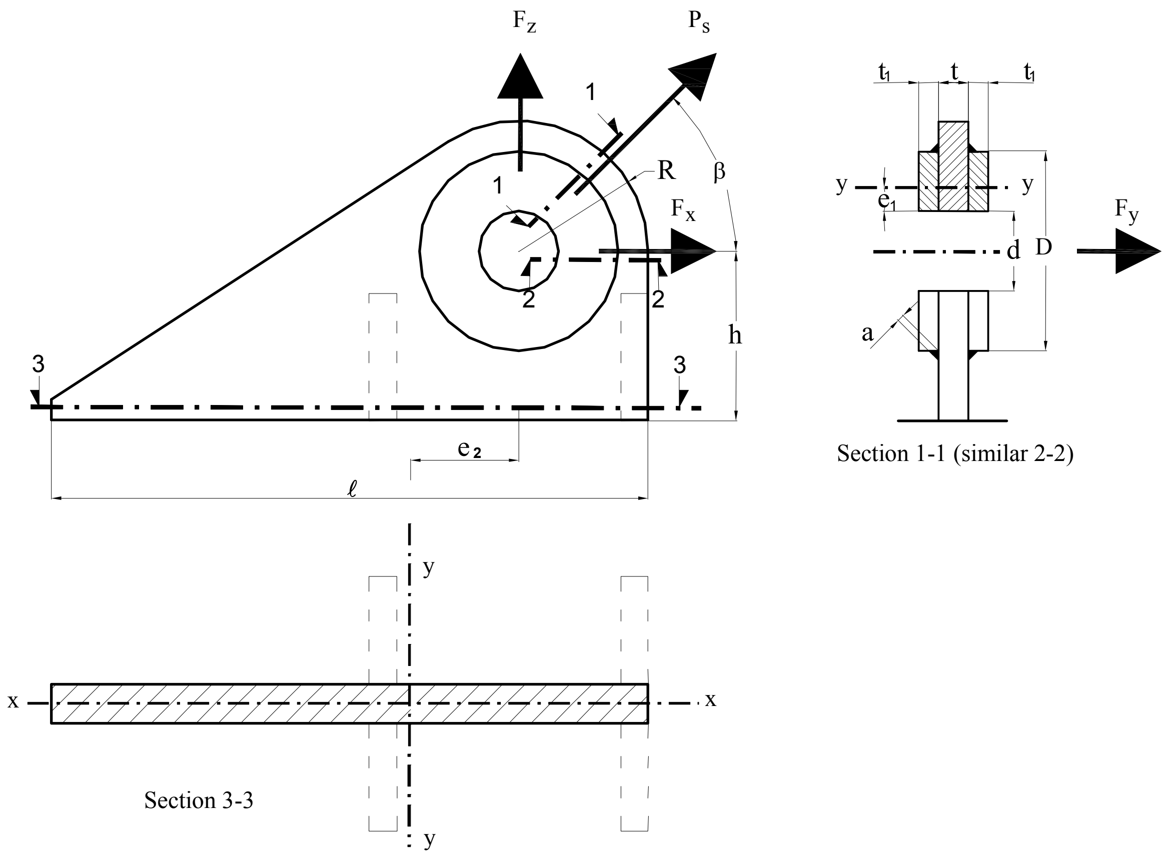


Fig. 10.1 Standard padeye

ℓ = base length [mm]

h = pin hole height above base [mm]

R = main plate radius [mm]

d = pin hole diameter [mm]

D = Cheek plate diameter [mm]

e_1 = neutral axis of section 2-2 related to pin hole edge [mm]

e_2 = pin hole eccentricity related to the neutral axis y-y of section 3-3 [mm]

t = main plate thickness [mm]

t_1 = cheek plate thickness [mm]

a = throat thickness of cheek plate weld [mm]

β = sling angle against x-axis

P_s = resolved padeye load (according to 2.9) [kN]

F_x = $P_S \cdot 1000 \cdot \cos(\beta)$ [N]

F_y = minimal $0{,}05 \cdot P_S \cdot 1000$ (see 2.10) [N]

F_z = $P_S \cdot 1000 \cdot \sin(\beta)$ [N]

4.4.2 Section 2-2:

Equivalent stress:

$$\sigma_{eq} = f_{cons} \cdot \sqrt{F_z^2 \cdot \left(\frac{1}{2 \cdot A_2} + \frac{d + 2 \cdot e_1}{15 \cdot Z_{2y}}\right)^2 + 3 \cdot \left(\frac{F_x}{2 \cdot A_{S2}}\right)^2}$$

$$\leq \frac{R_{eH}}{\gamma_g} \quad [\text{MPa}]$$

A_2 = area of Section 2-2 (similar 1-1) [mm²]

A_{S2} = shear area of Section 2-2,

related to force F_x (for D / 2R ≥ 0,8, A_{S2} = A_2) [mm²]

Z_{2y} = section modulus of section 2-2 related to axis y-y [mm³]

R_{eH} = minimum nominal upper yield strength [MPa]

γ_g = global safety factor for equivalent stresses according to 3.1

= 1,25

f_{cons} = consequence factor for lift points according to 3.1

= 1,35

4.4.3 Section 3-3:

Combined axial and bending stress:

$$\sigma_{a/b} = f_{cons} \cdot \left(\frac{F_z}{A_3} \pm \frac{F_z \cdot e_2 - F_x \cdot h}{Z_{3y}} \pm \frac{F_y \cdot h}{Z_{3x}}\right) \leq \frac{R_{eH}}{\gamma_g} \quad [\text{MPa}]$$

Location with highest stresses is relevant

A_3 = area of Section 3-3 [mm²]

$Z_{3x(y)}$ = section modulus of section 3-3 for location under consideration related to axis x-x or y-y, respectively [mm³]

γ_g = safety factor for axial and bending stresses according to 3.1

= 1,45

f_{cons} = consequence factor for lift points according to 3.1

= 1,35

Shear stress:

$$\tau_{(x)} = f_{cons} \cdot \frac{F_x}{A_{S3(x)}} \leq \frac{R_{eH}}{\gamma_g} \quad [\text{MPa}]$$

$$\tau_{(y)} = f_{cons} \cdot \frac{F_y}{A_{S3(y)}} \leq \frac{R_{eH}}{\gamma_g} \quad [\text{MPa}]$$

A_3 = area of Section 3-3 [mm²]

$A_{S3(x),(y)}$ = shear area of Section 3-3, related to force F_x or F_y [mm²], respectively (for rectangular cross section $t \times \ell$, without stiffening brackets, A_3 = $A_{S3(x)}$ = $A_{S3(y)} = t \cdot \ell$)

R_{eH} = minimum nominal upper yield strength [MPa]

γ_g = global safety factor for shear stresses according to 3.1

= 2,16

f_{cons} = consequence factor for lift points according to 3.1

= 1,35

Equivalent stress:

$$\sigma_{eq} = \sqrt{\sigma_{a/b}^2 + 3 \cdot \left(\tau_{(x)} + \tau_{(y)}\right)^2} \leq \frac{R_{eH}}{\gamma_g} \quad [\text{MPa}]$$

R_{eH} = minimum nominal upper yield strength [MPa]

γ_g = global safety factor for equivalent stresses according to 3.1

= 1,25

f_{cons} = consequence factor for lift points according to 3.1

= 1,35

4.4.4 Weld connection between cheek plate and main plate:

Shear stress:

$$\tau \cong f_{cons} \cdot \frac{P_s \cdot 1000}{D \cdot \pi \cdot a} \cdot \frac{t_1}{t + 2 \cdot t_1} \leq \frac{R_{eH}}{\gamma_g} \quad [\text{MPa}]$$

R_{eH} = minimum nominal upper yield strength [MPa]

γ_g = global safety factor for shear stresses according to 3.1

= 2,16

f_{cons} = consequence factor for lift points according to 3.1

= 1,35

Minimum weld thickness:

$$a_{min} = 1{,}5 \cdot \sqrt{\frac{t + t_1}{3}} \quad [\text{mm}]$$

but not less than 3 mm

4.5 Arrangement of trunnions

The diameter of the trunnion has to be chosen with respect to the minimum allowable bending radius of the proposed lifting sling, see Fig. 10.2. In general the diameter of the trunnion shall not be less than:

D_t = 9 · D_s

D_s = nominal sling diameter [mm]

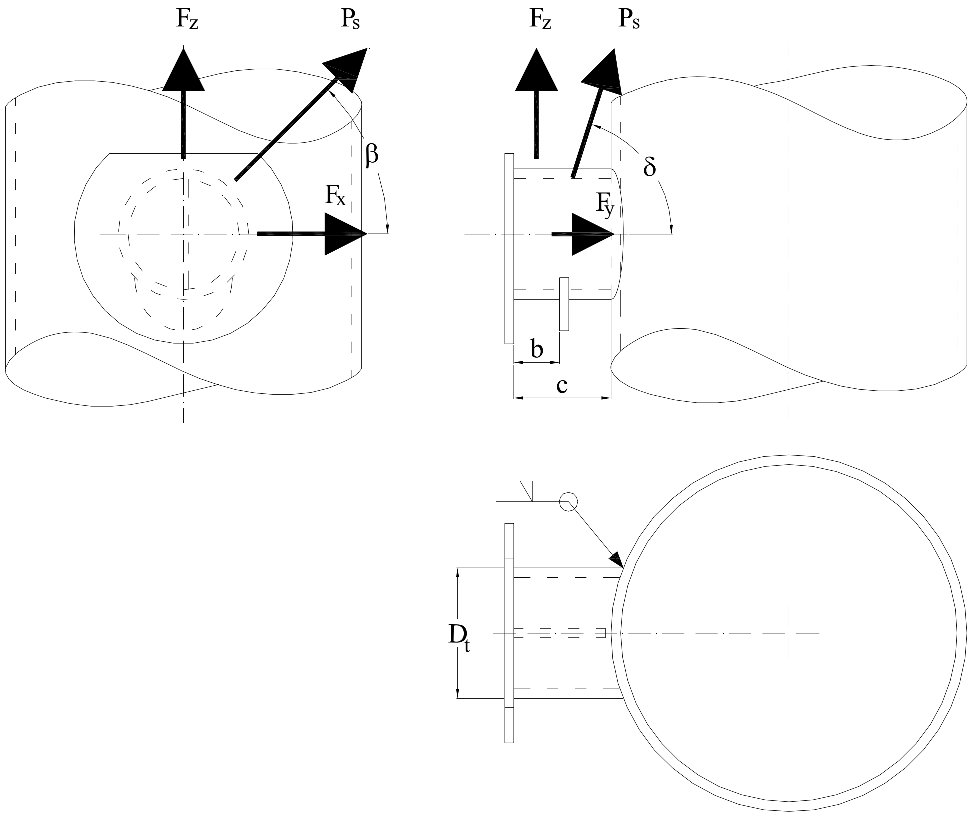


Fig. 10.2 Typical trunnion arrangement

D_t = trunnion diameter [mm]

b = clearance for sling between keeper plates [mm]

c = stub length of trunnion [mm]

β = sling angle against x-axis

δ = sling angle against y-axis

P_s = resolved trunnion load (according to 2.9) [kN]

Where practical aspects do not allow keeping this minimum diameter, a reduction of the allowable sling load has to be considered.

Adequate clearance shall be included between trunnion keeper plates, to allow for ovalisation of the sling section under load. In general, the width available for the sling shall be not less than:

$$b = 1{,}25 \cdot D_s + 25 \ \text{[mm]}$$

However, the practical aspects of the rigging and de-rigging operations may demand a greater clearance than this.

Through-thickness loading of the trunnions and their attachments to the structure shall be avoided if possible. If such loading cannot be avoided, the material used shall be documented to be free of laminations, with a recognized through-thickness designation.

4.6 Design of trunnions

Trunnions are subject to a strength investigation based on the following loads.

Safety factors are to be chosen according to 3.1

$$F_x = P_S \cdot 1000 \cdot \cos(\beta) \ \text{[N]}$$

$$F_y = P_S \cdot 1000 \cdot \cos(\delta), \text{ but not less than}$$

$$0{,}05 \cdot 1000 \cdot P_S \ \text{[N]} \quad \text{(see 2.10)}$$

$$F_z = \sqrt{\left(P_S \cdot 1000\right)^2 - F_x^2 - F_y^2} \ \text{[N]}$$

where z is the vertical direction:

$$F_Z = P_{RLW} \cdot 1000 \cdot f_{COG} \cdot f_{SKL}$$

P_{RLW} = Lift point resolved lift weight (according to 2.8) [kN]

4.6.1 Loadings on trunnion

The following loadings have to be considered for the design of the trunnion and the weld connection. The loads are acting simultaneously.

Bending moment:

$$M = \sqrt{F_x^2 + F_z^2} \cdot \left(c - \frac{b}{2}\right) \quad [\text{Nmm}]$$

Axial load:

$$F_A = F_y [\text{N}]$$

Shear force:

$$F_S = \sqrt{F_x^2 + F_z^2} \quad [\text{N}]$$

For abbreviations see Figure 10.2.

4.6.2 Weld connection of trunnion stub to shell

The trunnion stub shall generally be welded to the shell by means of a full penetration weld.

4.6.3 Shell and substructure

The shell and the substructure are also subject to strength investigation with the local loads as defined in 4.6.1, especially when the shell is unstiffened.

5. Lifting equipment

5.1 Standard lifting equipment

Where standard lifting equipment is used, it shall be designed or chosen according to Section 8. The adequacy for offshore operations will, however, be ascertained in every individual case.

5.2 Slings

5.2.1 Slings (steel wire ropes) and cables shall be of an approved type and certified by a recognized organization.

In the following it is assumed that the length of slings will be adjusted to actual as-built geometrical conditions of the object to be lifted, and that the sling length tolerances can be held within 0,25 % ($\Delta l_S / l_S \leq 0{,}0025$, l_S = nominal sling length).

5.2.2 Marine Warranty Survey (MWS) rigging approval

5.2.2.1 If the safe working load SWL of the slings is certified, then F_{SLING} or F_{CABLE} have to be less than SWL.

5.2.2.2 If MBL is certified the following safety factors have to be considered during lifting operation:

f_{SLING} = 2,25 with $F_{SLING} < MBL / f_{SLING}$

f_{CABLE} = 3,0 with $F_{CABLE} < MBL / f_{CABLE}$

5.2.2.3 If proof load PL is certified F_{SLING} or $F_{CABLE} < PL / 1{,}1$

5.3 Shackles

Shackles and similar lifting equipment are to be of an approved design. The safe working load indicated and certified shall at least correspond to the loads given under 2., excluding f_{DAF}.

Shackles shall not be subjected to bending.

5.4 Load distributing devices

Lifting beams, spreaders and similar load distributing devices shall be designed according to the loads as indicated in 2. The safety factors may be chosen according to Section 3, D.2., applying loading condition 7, see also 3.1.2.

Special attention shall be paid to buckling of members under compression and shear.

When using spreader bars for Marine Warranty Survey approval of one lift only, a proof load test Certificate is to be presented. If this is not possible a Design Certificate of a recognized institution, a Material Certificate 3.2 according to EN 10204 and NDT Reports of all seams are to be submitted for MWS approval, if applicable.

6. Offshore cranes and crane barges

6.1 Crane vessels

Barge or ship-mounted cranes intended for offshore lifting operations shall be of adequate type and design and certified accordingly. Regarding the crane(s), reference is made to Section 8.

The load curves of offshore cranes may sometimes include a certain amount of dynamic effects. This fact has to be confirmed by a recognized Classification Society or equivalent institution.

The barge or vessel itself shall be adequate for the sea area and kind of loads envisaged. Stability investigations covering the load cases planned will have to be presented.

6.2 Crane Vessel Condition Survey

The following items will be checked by MWS, if applicable:

– Class Certificate of recognized Classification Society

– International Load Line Certificate

– IOPPC (International Oil Pollution Prevention Certificate)

– Safety Radio and Safety Equipment Certificate

– Crane Certificate

– condition of navigational equipment and bridge machinery controls

– functionality of the radio equipment including portable items

– functionality of anchor line monitoring and indicators

– functionality of wind indicator

– suitable weather forecast available on board

– functionality of main and auxiliary hoists

– condition of tugger wires and winches

– functionality of load indicator in the crane cabin

– availability of wire and winch monitoring in the crane cabin

6.3 Two or more crane barges or vessels

Regarding operations involving two or more crane barges or vessels see 2.2.3.

7. Monitoring, measurements

7.1 During critical phases of lifting/lowering operations, monitoring e.g. by load or strain measurements may be advisable or necessary. In such cases review of the monitoring installation and control of the measurements during the operations will normally be part of the GL's survey procedure, and will be certified accordingly.

7.2 The minimum clearances during offshore lift between lifted items, spreader bar and crane boom shall be 5 m. Vertical clearance between lifted object and existing facilities shall be ≥ 2 m.

F. Installation Offshore

1. Jacket installation

1.1 General

The following requirements have to be followed for the operation of installing a steel jacket at its planned position on the seabed:

– A schedule shall be prepared indicating the duration of each phase of the installation operation. This shall show that the jacket can be made safe before the end of the forecasted favourable period. The installation can only be started if favourable weather conditions exist and are forecasted for the planned installation period.

– A control centre shall be established to deal with all information regarding the operation and from where all necessary commands can be given. The essential equipment for this centre shall be redundant.

– Prior to jacket lift or launch a check is required if no damage or leakage has occurred during the towing.

– Sufficient structural integrity during launching is to be checked using for skidded launch friction coefficients given in C.3.4 and for lifted launch contingency factor f_{CONT} given in E.2.1.

1.2 Lifted launch

1.2.1 Weight and centre of gravity of the jacket have to be calculated, locations of padeyes and corresponding loads to be defined. During the lifting operation E. has to be considered.

1.2.2 The launching operation includes a series of different load cases from the starting of the lift to the stage where the barge and the jackets float separately. Each load case shall be considered step by step. The first load case at the beginning of the lift shall be a low lift above the mountings of the jacking on the barge to see if weight and centre of gravity calculations are valid.

1.2.3 Necessary clearances from the jacket to crane vessel and barge during the lifting have to be provided depending also on accepted environmental conditions.

1.3 Skidded launch

1.3.1 The ballast system of the barge shall be suitable for the planned operations and has to be checked before the start.

1.3.2 The barge rocker arms shall be checked for structural integrity.

1.3.3 The launch operation shall take place in an area close to the final location of the jacket. The launching area has to be controlled in advance for any obstructions because launching above pipelines, templates, etc. is not recommended.

1.4 Floating condition of the jacket

1.4.1 The following requirements have to be fulfilled for the floating jacket:

– reserve buoyancy for lifted launch: 10 %

– reserve buoyancy for skidded launch: 15 %

1.4.1 The minimum calculated bottom clearance shall be 5 m or 10% of water depth, whichever is greater.

1.4.2 After launching of the jacket and prior to upending, an inspection shall be required to confirm that no damage and/or leakage have occurred to the jacket.

Draughts, trim and heel shall be checked and compared with the calculations.

1.4.3 The sufficient number of tugs shall be available to handle the floating jacket safely.

1.5 Upending

1.5.1 Hook assisted upending operations

Crane lifting forces shall be determined, based on the ballasting sequence. Loads on padeyes and related slings have to be under control.

The crane vessel's influence on the motion characteristics of the jacket shall be considered.

There shall be a minimum clearance of 3 metres between the jacket and the crane vessel at all times. A seabed clearance of at least 5 m has to be guaranteed at any stage of upending.

1.5.2 Self-upending (ballast assisted) structures

A team shall be sent on the jacket to connect control cables, hoses, etc.

The control of the ballast system during the upending operation shall be done from the control centre at an assisting vessel. Draught readings and measurements of the upending angle shall be done in regular intervals during the upending procedure.

Contingency plans shall be defined for the case of failure of valves, operator error and other unforeseen events. These plans shall allow to make the jacket safe again and afterwards to continue the upending operation.

The ballasting system shall give the ability to reverse the upending operation at any stage.

1.5.3 On bottom control / positioning

1.5.3.1 Seabed to be controlled by Remote Operated Vehicles (ROV) before and after positioning.

1.5.3.2 The on bottom stability of the positioned unpiled jacket shall be guaranteed by reaching the following safety factors:

– sliding: 1,3
– bearing: 1,5
– overturning: 1,2
– buoyancy: 1,1

2. Pile installation

2.1 General

Concerning operational feasibility the following aspects shall be considered:

– soil characteristics
– pile driving procedure
– sizes of driving and reserve hammers
– lifting equipment for hammers and piles
– lifting/upending procedure for piles

2.2 Installing the piles

The piles shall be installed in a sequence which achieves stability of the jacket in all stages of the installation.

For pile lifting E. has to be considered. Pile lifting and upending tools (if any) are to be certified by a recognized Certification Society.

A proper arrangement for locating and guiding the piles into the pile sleeves shall be established. Special importance gains this requirement for underwater operation.

Horizontal clearances between hammer, pile or follower and the jacket structure shall not be less than 1 m. Fabrication tolerances, clearances, deflections and pile sway have to be considered.

2.3 Pile grouting

No piling shall be performed after commencement of the pile grouting operation.

The limiting environmental criteria shall be established considering the grout system design.

The positioning system and the manoeuvrability of the vessel executing the grouting procedure shall be considered to avoid impact loads to the installed jacket. If necessary, appropriate fendering measures shall be provided.

3. Installation of gravity based foundations

3.1 General

Before the installation of gravity based foundations at the planned working location, the following has to be investigated:

– definition of environmental conditions for the suitable weather window
– the seabed at the location shall be controlled by Remote Operated Vehicles (ROV) and debris or obstructions shall be removed
– it shall be clarified if an underkeel clearance of at least 3 m can be achieved
– each phase of the installation procedure shall be considered step by step and critical conditions investigated in detail

3.2 Water ballast system

The water ballast system shall meet the following requirements:

– there shall be a second power supply available to drive all the water ballast pumps

– ballast water levels are to be constantly monitored with the required accuracy and it is recommended to have a back-up system available

– air venting systems are to have adequate monitoring and control

– the ballast system allow for fine adjustment of the trim of the foundation by operating different tanks within the foundation

– in the event of power or control failure all critical valves shall be closed or shall be opened as a safe condition of the foundation requires

3.3 Installation operation

During the installation the following requirements have to be observed:

– The foundation has to be placed at the location with the defined accuracy. Two totally independent positioning systems shall be applied.

– The verticality of the foundation has to be checked constantly and measures to be taken to remain within the inclination tolerances.

– The lowering operation has to be performed within the defined time window for a favourable weather forecast.

– The installation procedure has to be reversible. In case of inaccuracy of location and/or inclination, the foundation shall be able to flow up and be repositioned. This has also be possible for a change of the working location, if applicable.

4. Topside Installation

4.1 General

Before the mating operation of joining the heavy topside structure (supported by a barge) and the base structure, the following investigations have to be made:

– assumption of environmental conditions for the necessary weather window

– each phase of the mating operation shall be considered step by step and the most critical load cases investigated in detail

– the systems involved in the operation are to be tested with their essential functions

4.2 Mating operation by floatover

The following operations have to be executed in the defined sequence:

4.2.1 Gravity based structure

– ballasting of the gravity based structure to mating draught, if applicable

– ballasting of the barge with the topside to mating draught

– positioning of the topside above the base structure

– deballasting of the gravity based structure to contact with the topside lower edge plane

– fixing topside to base structure

– removal of barge

– deballasting of gravity based structure to towing draught, if applicable

4.2.2 Pile based structure

– ballasting of the barge with the topside to mating draught, if applicable

– positioning of the topside above the base structure

– ballasting of the barge to make contact of the topside lower edge plane with the base structure

– fixing topside to base structure

– ballasting and removal of barge

4.3 Mating operation by lifting

For lifting operations E. has to be considered.

The following operations have to be executed in the defined sequence:

– ballasting of the barge with the topside for lift-off

– preparing of the base structure for taking over the topside

– lifting the topside from the barge

– positioning of topside at the base structure

– fixing of topside to base structure

– removal of barge and cranes

G. Other Marine Operations for Marine Warranty Survey

1. Vessel positioning

The positioning system of vessels may contain anchors, an anchors and thrusters combination, or a DP (dynamic positioning) equipment.

The positioning shall be monitored and recorded during offshore operations.

1.1 Anchor handling

For positioning of vessels and barges by anchors an anchor handling procedure including anchor patterns and risk assessment has to be prepared.

The following specific criteria are to be considered for anchor handling operations:

– minimum clearance between anchor and pipeline/cable, if anchor line is crossing pipeline/cable: 200 m

– minimum clearance between anchor and pipeline/cable, if anchor line is not crossing pipeline/cable: 100 m

– usage of middle-line buoys if anchor line is crossing pipeline/cable

For anchor placement in exclusion zones a permit from the responsible third party is to be presented.

1.2 Dynamic positioning

The DP system shall meet the Germanischer Lloyd requirements for **DP 3** Notation. DP tests under normal working condition and in accordance with DP FMEA to be carried out prior to enter the 500 m safety zone of an offshore installation. The DP test shall be witnessed and approved by the Marine Warranty Surveyor.

2. Pipe/cable laying

2.1 Detailed pipe/cable laying procedures including start-up, laydown, crossing, abandonment and recovery to be prepared and submitted for MWS approval. These procedures shall be developed based on the pipe/cable stress analyses approved by a recognised third party.

2.2 Pipe/cable laying vessel condition survey

The following items will be checked by the Marine Warranty Surveyor, if applicable:

– Class Certificate of a recognized Classification Society

– International Load Line Certificate

– IOPPC Certificate

– Safety Radio and Safety Equipment Certificate

– Certificate for laying equipment

– condition of navigational equipment and bridge machinery controls

– functionality of the radio equipment including portable items

– functionality of anchor line monitoring and indicators

– condition of wind indicator

– suitable weather forecast available on board

– condition of pipe/cable handling/line-up systems

– condition of tensioners

– functionality of abandonment and recovery winches

– functionality of load indicator in the control cabin

– wire and winch monitoring in the control cabin available

– condition of stinger/ramp levelling

2.3 Following on-site surveillance pipe/cable laying will be carried out in following phases:

– start up

– laying operation, if required

– lay down

– crossings

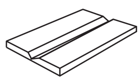
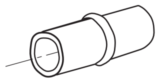
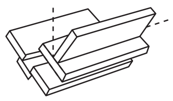
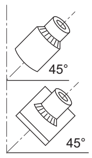
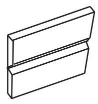
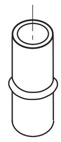
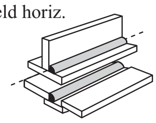
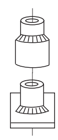
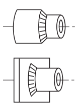
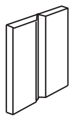
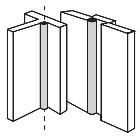
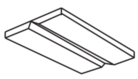
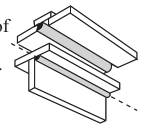
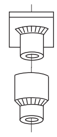
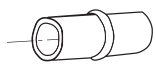
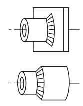
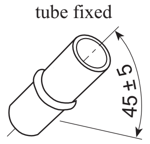
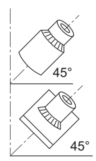
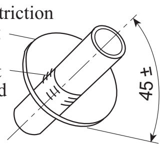
– abandonment and recovery

– riser/speed installation

2.4 For further information, see also Chapter 2 – Mobile Offshore Units, Section 5.

Annex A

Welding and Test Piece Positions

Welding position 1)	Welding progression	Configuration of joint or test piece			
		Plate groove	**Tube groove**	**Plate fillet**	**Tube fillet**
F/IG F/IF	any	Plates horizontal 	Axis horiz. tube rotated 	Throat of weld vertic. Axis of weld horiz. 	 Axis inclined (45°) tube rotated
H-V/2G H/2F	any	Plates vertical axis of weld horiz. 	Axis vertical. tube fixed 	One plate horizontal Axis of weld horiz. 	Axis vert. tube fixed  Axis horiz. tube rotated 
V/3G V-D/3G V/3F V-D/3F	upward downward 3) upward downward 3)	Plates vertical axis of weld vertical 	2)	Axis of weld vertical 	2)
O/4G O/4F	any	Plates horizontal 	2)	One plate horizontal Axis of weld horiz. 	Axis vertical tube fixed 
TH/5G TH/5F		n.a.	Axis horiz., tube fixed 	n.a.	Axis horiz., tube fixed 
TI/6G TI/6F	any, but vertical position only upward if not otherwise agreed 3)	n.a.	Axis inclined (45°), tube fixed 	n.a.	 Axis inclined (45°) tube rotated
TR/6GR		n.a.	Axis inclined (45°), tube fixed Restriction ring Test weld 	n.a.	n.a.

1) Explanations:

F/ ... = Flat (downhand) position
H-V/ .. = Horizontal-Vertical (groove) weld
H/ ... = Horizontal (fillet) weld
V/ ... = Vertical (upward) position
V-D/ ... = Vertical (downward) position
O/ ... = Overhead position
TH/ ... = Tube horizontal, fixed
TI/ ... = Tube inclined (45°), fixed
TR/ ... = Tube inclined (45°), fixed, with restriction ring (simulating T-, Y- or K- connections)

2) Included in position TH/5G/5F or TI/6G/6F respectively.

3) For welding the V-D/3G (3F)-position in the downward progression or the TH/5G(5F)-position in the 12 or 6 o'clock progression, which is normally not applicable for steel structures, special agreement is to be made.

Annex B

List of Standards, Codes, etc. Quoted

Chapter/ Section	IACS	IMO	ISO	EN	DIN	API	ASTM	Others
4/1						RP 2A-WSD		
4/2	(D.3.3.6)							SNAME T&RB 5-5A
4/3	UR				18800	RP 2A-WSD		AISC IIW XIII-1965-03 IIW XV -1127-03
4/4			857 1106 2400 2553 5580 5817 6520 6947 9606-1 9692 9712 10042 13916 13920 14731 22533	462 473 571 583-1 583-2 910 1011-2 1011-3 1011-4 1435 1714 5817 6947 10025 10028 10160 10164 10204 10225 12345 13916 13920 14731 22553 25580 25817 26520 27963 29692 30042	462 571 583-1 583-2 1435 1714 5817 14731 18800 25580 25817 27963 54120 54130 54131 V 1738	RP 2A Spec2B	A370	ANSI / AWS D1.1 BS 7448 CR 12187
4/5				206-1	206-1 488 1045-1 1045-2 1045-3 1048 1055-100-1 1164 4099 4226			ACI 318 ACI 357 R-84 BS 8110 FIP-Rec. NORSOK NS 3437
4/6			8044 12473 12495 12944 13173 15156 15589 20340 21809-1	10028-7 12496	17744 81249-1			ASTM A240 ASTM G48 NACE 01004-2001 NACE RP0176 NACE RP0387 NACE RP0492 NORSOK M503

- continued -

Chapter/ Section	IACS	IMO	ISO	EN	DIN	API	ASTM	Others
4/7						RP 2A		OTC 1896, 1973
4/8		MODU						
4/9	(D3.10.1) (D11.4.2) (D11.4.3) (D9.9.2)	MODU A.855 (20)	19901-3			Aluminium Structure		ICS CAP 437
4/10			19901-6:2004	10024				NORSOK J-003 NORSOK R-003 Noble Denton Int. and London Offshore Consultants Guidelines IOPPC

Explanation of abbreviations:

ACI	American Concrete Institute
AISC	Specification for the Design, Fabrication and Erection of Structural Steel for Buildings and Manual of Steel Construction
ANSI	American National Standardization Institute
API	American Petroleum Institute
ASTM	American Society for Testing and Materials
BS	British Standard
CAP 437	Offshore Helicopter Landing Areas – Guidance on Standards, Civil Aviation Authority Gatwick, UK
D1 - D12	Requirements concerning Mobile Offshore Drilling Units, Unified Requirement of IACS, 1996
DIN	Deutsches Institut für Normung (German Institute for Standardization)
EN	European Standards
FIP	Federation International de la Precontrainte
IACS	International Association of Classification Societies
ICS	Guidance to Helicopter/Ship Operations, International Chamber of Shipping
IEC	International Electrotechnical Commission
IIW	International Institute of Welding
IMO	International Maritime Organization of the United Nations
IOPPC	International Oil Pollution Prevention Certificate
ISM	MO International Safety Management Procedures, 1 July 2002
ISO	International Standardization Organization
MODU	Code for the Construction and Equipment of Mobile Offshore Drilling Units, issued by IMO
NORSOK	Norsk Sokkels Konkurranseposisjon (Norwegian Standard)
OTC	Offshore Technology Conference
SNAME	Society of Naval Architects and Marine Engineers, USA
(…..)	References in brackets are used, but not explicitly mentioned in text