NORSOK STANDARD

DESIGN OF STEEL STRUCTURES

N-004
Rev. 1, December 1998

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ERRATA TO

NORSOK STANDARD

DESIGN OF STEEL STRUCTURES,

N-004
REV 1, DEC 1998

Updated for misprints as noted per 1999-10-15, 2000-05-15, 2000-09-01 and 2000-11-10

- Page 27 (per 1999-10-15 and 2000-05-15):
  - At the end of paragraph 6.3.6. add: "**Torsional buckling of flanged ring stiffeners may be excluded as a possible failure mode provided that:**
    $$b \geq \frac{3.5h}{\sqrt{10 + \frac{Eh}{f_y r}}}$$
  - below equation 6.25. add "**N_{Sd} is negative if in tension**."
- Page 30 (per 1999-10-15):
  - add after equation 6.32**: "** $M_{Sd} = \sqrt{M_{y,Sd}^2 + M_{z,Sd}^2}$ **provided the same variation in $M_{y,Sd}$ and $M_{z,Sd}$ along the member length."**
- Page 32 (per 1999-10-15 and 2000-09-01):
  - equation 6.40 change γ_m to $\boldsymbol{\gamma_M}$**.**

- Above equation 6.41, change in the first inequality f_{cle} to $\mathbf{f_{he}}$ to read:

$$\sigma_{c,Sd} > 0{,}5\frac{f_{he}}{\gamma_M}$$

- Page 46 (per 1999-10-15):
  - In equation 6.71 change f_{cj} to $\mathbf{f_{clj}}$.

  In the description for f_{cj} change to:"**$\mathbf{f_{clj}}$= corresponding tubular or cone characteristic axial local compressive strength.**"

- Page 47 (per 1999-10-15):
  - In paragraph 6.5.4.1 change the reference to the equations in section 6.3.6.1 to "**equation 6.44 or 6.51 for method A or B respectively.**"

- Page 55 (per 1999-10-15):
  - In equation 6.87 change to: **"$\sigma_{x,Rd}=C_x \cdot f_y/\gamma_M$ "**

- Page 56 (per 1999-10-15):
  - In equation 6.91 change to **"$\sigma_{x,Sd} \le \sigma_{x,Rd}$; "**
    - The following line:
    "where $\sigma_{c,Sd}$= longitudinal compressive stress(tensile stress to be set equal to zero)"
    should be deleted.

- Page 60 (per 1999-10-15):
  - Change equation 6.106 by adding the index ***l*** for the parameters $\bar{\lambda}_\tau$ and $\mathbf{k}_l$ so the equation reads: " $\bar{\lambda}_{\tau l} = 0.035\frac{s}{t}\frac{1}{\sqrt{k_l}}\sqrt{\frac{f_y}{235}}$ "

- **Insert** new equation: " $\bar{\lambda}_{\tau g} = 0.035\frac{l}{t}\frac{1}{\sqrt{k_g}}\sqrt{\frac{f_y}{235}}$ "

- In equation 6.107 change inequality sign to
  ***l*<L$_G$** in first line

  and

  ***l*>L$_G$** for second line.

- In equation 6.108 invert the ratio l/s to read **s/l** for both lines so the equation reads:

$$k_l = 5.34 + 4\left(\frac{s}{l}\right)^2, \text{for } l \geq s$$
$$= 5.34\left(\frac{s}{l}\right)^2 + 4, \text{for } l < s$$

- Page 64 (per 2000-05-15):
  - In figure 6-17 the correct distance for e_f is to **center of web plate.**

- Page 68 (per 1999-10-15):
  - Add before equation 6.153 :

    " If $\frac{q_{Sd} l^2}{8} \geq N_{Sd} \cdot z^*$ then:"

  - After equation 6.154 add:

    " If $\frac{q_{Sd} l^2}{8} < N_{Sd} \cdot z^*$ then:

$$\frac{N_{Sd}}{N_{ks,Rd}} - \frac{N_{Sd}}{N_{Rd}} + \frac{N_{Sd} \cdot z^* - \left|\frac{q_{Sd} l^2}{8}\right|}{M_{s,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \leq 1,$$

and

$$\frac{N_{Sd}}{N_{kp,Rd}} + \frac{N_{Sd} \cdot z^* - \left|\frac{q_{Sd} l^2}{8}\right|}{M_{p,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \leq 1\text{"}$$

- Page 71 (per 1999-10-15):

    - In the description for n_G change the word “stiffeners” to:
      “**stiffener spans**”

      Add to the listing of symbols:
      “**I_s = moment of inertia of stiffener and plate with plate width equal to s.**”

- Page 93 (per 1999-10-15):

    - Below equation 10.30. In the explanation to N_{Sd}:
      Change the wording “compression negative” to:
      “**compression positive**”.

Annex A:

- Page 138 (per 1999-10-15):

    - Equation A.3.23 exchange:
      ε_{cr} with $\boldsymbol{\varepsilon_y}$
      and
      ε_y with $\boldsymbol{\varepsilon_{cr}}$**.**

Annex B:

- Page 200 (per 1999-10-15):

    - In equation B.5,42 change index of parameter r_r to $\mathbf{r_f}$.

- Page 207 (per 2000-05-15):

    - In the description for l_T the parameter t should be **changed to h** in the square root so the correct description is:

l_T = for ring stiffeners: distance (arc length) between tripping brackets l_T need not be taken greater than $\pi\sqrt{rh}$ for the analysis
for longitudinal stiffeners: distance between ring frames.

- Page 208 (per 2000-05-15):

  - The sign of inequality in equation B.5.78 and B.5.83 should be changed from greater or equal **to less or equal**

Annex C:

- Page 222 (per 1999-10-15):

  - In the description of the symbols used in equation C.2.6 change:
    ”tref” with ”$\mathbf{t_{ref}}$”.

- Page 233 (per 2000-05-15):

  - Equation C.2.10 is valid for **T<2t**

- Page 238 (per 1999-10-15):

  - the numbering of paragraph C.2.3.6.10 should be changed to **C.2.6.3.10**.

- Page 242 (per 1999-10-15):

  - In Table C.2-4 add in the column for Tolerance requirement for Single side and Double side weld with hot spot at surface (the two lower rows in the table) the following:
    ” e ≤ min. (0.15t, 4 mm)”

- Page 261 (per 1999-10-15):

  - Insert after equation C.7.3 the following expression:

$$q(\Delta\sigma_0, h) = \frac{\Delta\sigma_0}{(\ln n_0)^{\frac{1}{h}}}$$

Annex K:

- Table of contents gives wrong page numbers. (per 1999-10-15)

- Page 330 (per 2000-11-10):

  - The numbering of the Notes to Table K.2-1 should be renumbered so b)in line two should be deleted and apply to line 4 “Outfitting structures”….Note d) should be changed to c). Starting with “If multilegged……” note d) should apply to note g),note e) apply to h) and f) g) and h) should be deleted.
    The notes should be:

a) Local areas of welds with high utilisation shall be marked with frames showing areas for
mandatory NDT when partial NDT are selected. Inspection categories depending on access for in-service inspection and repair.
b) Outfitting structures are normally of minor importance for the structural safety and
integrity. However, in certain cases the operational safety is directly influenced by the
outfitting and special assessment is required in design and fabrication.
A typical example is guides and supports for gas risers.
c) If multilegged jacket with corner legs supporting a foundation systems;
Upper part of corner legs and inner legs: DC 4, III, B or C
Lower part of corner legs: DC 2, II, A or B
d) If multilegged jacket where each leg supporting a foundation system: DC 4, III, B or C
e) If one or two pile(s) per leg: DC2, II.

Pages 348 – 355 (per 1999-10-15):

- Paragraph K.6.1.2 up to paragraph K6.4 are given wrong item numbers, shall be read as **K.6** instead of K.1

Annex L:

Table of contents gives wrong page numbers. (per 1999-10-15)

Annex M:

Table of contents gives wrong page numbers. (per 1999-10-15)

Annex N:

Table of contents gives wrong page numbers. (per 1999-10-15)

FOREWORD

NORSOK (The competitive standing of the Norwegian offshore sector) is the industry initiative to add value, reduce cost and lead-time and eliminate unnecessary activities in offshore field developments and operations.

The NORSOK standards are developed by the Norwegian petroleum industry as a part of the NORSOK initiative and supported by OLF (The Norwegian Oil Industry Association) and TBL (Federation of Norwegian Engineering Industries). NORSOK standards are administered and issued by NTS (Norwegian Technology Standards Institution).

The purpose of NORSOK standards is to contribute to meet the NORSOK goals, e.g. by replacing individual oil company specifications and other industry guidelines and documents for use in existing and future petroleum industry developments.

The NORSOK standards make extensive references to international standards. Where relevant, the contents of a NORSOK standard will be used to provide input to the international standardisation process. Subject to implementation into international standards, the NORSOK standard will be withdrawn.

Annex A, B, C, K, L, M and N are normative parts of this standard.

INTRODUCTION

This NORSOK standard provides guidelines and requirements for how to design and document offshore steel structures. The standard is intended to fulfil NPD Regulations relating to loadbearing structures in the petroleum activities /1/. The design principles follow the requirements in ISO 13819-1.

The standard gives provisions for offshore structures and references Norwegian standard NS 3472 and NS-ENV 1993 1-1 Eurocode 3. Either code may be used for design of parts where relevant.

1 SCOPE

This standard specifies guidelines and requirements for design and documentation of offshore steel structures.

The standard is applicable to all type of offshore structures made of steel with a specified minimum yield strength less or equal to 500 MPa. For steel with higher characteristic yield strength see Chapter 12 Commentary.

2 NORMATIVE REFERENCES

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

DNV	Rules for classification of ships
DNV	Rules for planning and executions of marine operations.
ISO 13819-1	Offshore structures part 1: General requirements.
ISO 3010	Basis for design of structures - seismic action of structures.
NORSOK G-001	Soil investigation.
NORSOK J-003	Marine operations
NORSOK M-001	Materials selection.
NORSOK M-101	Structural steel fabrication.
NORSOK M-120	Material data sheets for structural steel.
NORSOK M-122	Cast and forged steel for structural components
NORSOK M-501	Surface preparation and protective coating.
NORSOK M-503	Cathodic protection.
NORSOK N-001	Structural design.
NORSOK N-002	Collection of metocean data.
NORSOK N-003	Action and action effects.
NORSOK N-005	Condition monitoring.
NORSOK S-001	Technical safety.
NORSOK U-001	Subsea structures and piping systems.
NORSOK Z-001	Documentation for operation.
NPD	Regulations relating to loadbearing structures in the petroleum activities.
NS 3472	Steel structures. Design rules
NS 3481	Soil investigation and geotechnical design for marine structures.
NS 2128	Weights engineering, terminology.
NS 2129	Weights engineering, requirements for weight reports.
NS 2130	Weights engineering, specification for weighing of major assemblies.
NS 2131	Weights engineering, specification for weight data from suppliers and weighing of bulk and equipment.
NS-ENV 1993 1-1	Eurocode 3: Design of steel structures part 1-1: General rules and rules for buildings

Note: The reference to DNV Rules applies to the technical provisions therein. Any requirement therein for classification, certification or third party verification is not part of this NORSOK standard and may be considered as a separate service. Wherever the terms: agreement, acceptance or consideration etc. appear in the DNV Rules they shall be taken to mean agreement, acceptance or consideration by the Client/Purchaser or any other specifically designated party. Likewise, any statement such as: to be submitted to DNV, shall be taken to mean: to be submitted to Client/Purchaser or any other specifically designated party.

3 DEFINITIONS, ABBREVIATIONS AND SYMBOLS

3.1 Definitions

Normative references	Shall mean normative in the application of NORSOK standards.
Informative references	Shall mean informative in the application of NORSOK standards.
Shall	Shall is an absolute requirement which shall be followed strictly in order to conform with the standard.
Should	Should is a recommendation. Alternative solutions having the same functionality and quality are acceptable.
May	May indicates a course of action that is permissible within the limits of the standard (a permission).
Can	Can-requirements are conditional and indicates a possibility open to the user of the standard.
Design Premises	A set of project specific design data and functional requirements which are not specified or are left open in the general standard.
Norwegian petroleum activities	Petroleum activities where Norwegian regulations apply.
Operator	A company or an association which through the granting of a production licence is responsible for the day to day activities carried out in accordance with the licence
Petroleum activities	Offshore drilling, production, treatment and storage of hydrocarbons.
Principal Standard	A standard with higher priority than other similar standards. Similar standards may be used as supplements, but not as alternatives to the Principal Standard.
Recognised classification society	A classification society with recognised and relevant competence and experience from the petroleum activities, and established rules and procedures for classification/certification of installations used in the petroleum activities.
Verification	Examination to confirm that an activity, a product or a service is in accordance with specified requirements.

3.2 Abbreviations

API	American Petroleum Institute.
BSI	British Standards Institution.
DNV	Det Norske Veritas.
ECCS	European Convention for Constructional Steelwork.
ISO	International Organisation for Standardisation.
NPD	Norwegian Petroleum Directorate.

3.3 Symbols

A	cross sectional area, accidental action, parameter, full area of the brace /chord intersection

A_c	cross sectional area of composite ring section, cracked area of the brace / chord intersection of a tubular joint
A_{Corr}	corroded part of the cross-section
A_{Crack}	crack area
A_e	effective area
A_f	cross sectional area of flange
A_G	cross-sectional area of girder, cross-sectional area of the grout
A_R	net area of circular part of a dented cylinder
A_s	effective cross sectional area of stiffener, gross steel area of a grouted, composite section
A_w	cross sectional area of web
B	hoop buckling factor
C	rotational stiffness, factor
C_e	critical elastic buckling coefficient
C_h	elastic hoop buckling strength factor
C_m	reduction factor
C_{my}, C_{mz}	reduction factor corresponding to the member y and z axis respectively
C_x, C_{xG}, C_{xs}	factors
C_y, C_{yG}, C_{ys}	factors
C_τ, $C_{\tau G}$, $C_{\tau s}$	factors
C_0	factor
$CTOD_c$	characteristic value of crack opening
$CTOD_{cd}$	design value of crack opening
D	outer diameter of chord, cylinder diameter at a conical transition, outside diameter
D_c	diameter to centeroid of composite ring section
D_e	equivalent diameter
D_j	diameter at junction
D_{max}	maximum measured diameter
D_{min}	minimum measured diameter
D_{nom}	nominal diameter
D_s	outer cone diameter
E	Young's modulus of elasticity, $2.1 \cdot 10^5$ MPa
E_G	modulus of elasticity of the grout
E_S	modulus of elasticity of the steel
F_{AR}	correction factor for axial resistance and bending resistance of a cracked tubular joint
G	shear modulus
I, I_z, I_s	moment of inertia, moment of inertia of undamaged section
I_c	moment of inertia for ring composite section
I_{ch}	moment of inertia
I_{cT}	moment of inertia of composite ring section with external hydrostatic pressure and with effective width of flange
I_{dent}	moment of inertia of a dented cross-section
I_e	effective moment of inertia
I_G	effective moment of inertia for the grout in a grouted, composite cross section
I_p	polar moment of inertia
I_{po}	polar moment of inertia = $\int r^2 dA$ where r is measured from the connection between the stiffener and the plate.
I_S	effective moment of inertia for the steel in a grouted, composite cross section

I_s	moment of inertia of web stiffener
K_a	factor
K_{Ic}	characteristic fracture toughness (mode I)
K_{Icd}	design fracture toughness
L	length, distance
L_1	distance to first stiffening ring in tubular section
L_c	distance to first stiffening ring in cone section along cone axis
L_G	length of girder
L_{Gk}	buckling length of girder
L_{GT}	distance between lateral support of girder
L_{GT0}	limiting distance between lateral support of girder
L_r	ring spacing
$M_{1,Sd}$	design bending moment about an axis parallel to the flattened part of a dented tubular section, design end moment
$M_{2,Sd}$	design bending moment about an axis perpendicular to the flattened part of a dented tubular section, design end moment
$M_{crack,Rd}$	bending resistance of a cracked tubular joint
$M_{dent,Rd}$	bending resistance of dented tubular section
$M_{g,Rd}$	bending resistance of the grouted member
$M_{p,Rd}$	design bending moment resistance on plate side
$M_{pl,Rd}$	design plastic bending moment resistance
M_{Rd}	design bending moment resistance
$M_{red,Rd}$	reduced design bending moment resistance due to torsional moment
M_{Sd}	design bending moment
$M_{s,Rd}$	design bending moment resistance on stiffener side
$M_{T,Sd}$	design torsional moment
$M_{T,Rd}$	design torsional moment resistance
$M_{y,Rd}$	design in-plane bending moment resistance
$M_{y,Sd}$	in-plane design bending moment
$M_{z,Rd}$	design out-of-plane bending moment resistance
$M_{z,Sd}$	out-of-plane design bending moment
$N_{c,Rd}$	design axial compressive resistance
$N_{can,Rd}$	design axial resistance of can
$N_{cg,Rd}$	axial compression resistance of a grouted, composite member
N_{cl}	characteristic local buckling resistance
N_{cl}	characteristic local buckling resistance
$N_{cl,Rd}$	design local buckling resistance
$N_{crack,Rd}$	axial resistance of a cracked tubular joint
$N_{dent,Rd}$	axial resistance of the dented tubular section
N_E	Euler buckling strength
$N_{E,dent}$	Euler buckling strength of a dented tubular member, for buckling in-line with the dent
N_{Eg}	elastic Euler buckling load of a grouted, composite member
N_{Ey},N_{Ez}	Euler buckling resistance corresponding to the member y and z axis respectively
$N_{ks,Rd}$	design stiffener induced axial buckling resistance
$N_{kp,Rd}$	design plate induced axial buckling resistance
N_{Sd}	design axial force
$N_{t,Rd}$	design axial tension resistance
$N_{tg,Rd}$	axial tension resistance of a grouted, composite section
N_{ug}	axial yield resistance of a grouted, composite member

$N_{x,Rd}$	design axial resistance in x direction
$N_{x,Sd}$	design axial force in x direction
$N_{y,Rd}$	design axial resistance in y direction
$N_{y,Sd}$	design axial force in y direction
P_{Sd}	design lateral force
Q	factor
Q_f	chord action factor
Q_g	chord gap factor
Q_u	strength factor
Q_{yy}	angle correction factor
Q_β	geometric factor, tubular joint geometry factor
R	radius, radius of chord, radius of conical transition
R_d	design resistance
R_k	characteristic resistance
S_d	design action effect
S_k	characteristic action effect
T	thickness of tubular sections, thickness of chord
T_c	chord-can thickness
T_n	nominal chord member thickness
V_{Rd}	design shear resistance
V_{Sd}	design shear force
W	elastic section modulus
W_{eG}	effective section modulus on girder flange side
W_{ep}	effective section modulus on plateside
W_{es}	effective section modulus on stiffenerside
W_R	elastic section modulus of the circular part of a dented cylinder
W_{tr}	elastic section modulus of a transformed, composite tubular cross section
Z	plastic section modulus
a	stiffener length, crack depth, weld throat, factor
a_i	initial crack depth
a_m	maximum allowable defect size
a_u	calculated crack depth at unstable fracture
b	plate span measured to centre of support plate, factor, width
b_e	effective width
c	factor
d	diameter of tubular cross section, brace diameter
d_0, d_1, d_2	distance
e	distance from neutral axis of the cylindrical part of the dented cylinder to the neutral axis of the original, intact cylinder
e_f	flange eccentricity
e_G	distance from centroid of dented grout section to the centroid of the intact grout section
e_S	distance from centroid of dented steel section to the centroid of the intact steel section
f_{bg}	characteristic bending strength of grouted member
f_c	characteristic axial compressive strength
f_{cg}	characteristic cube strength of grout
$f_{ch,Rd}$	design axial compressive strength in the presence of hydrostatic pressure
f_{cj}	corresponding tubular or cone characteristic compressive strength
f_{cl}	characteristic local buckling strength

$f_{cl,Rd}$	design local buckling strength, design local buckling strength of undamaged cylinder
f_{clc}	local buckling strength of conical transition
f_{cle}	characteristic elastic local buckling strength
f_{cr}	critical buckling strength
f_d	design yield strength
f_E	Euler buckling strength
f_{Epx}, f_{Epy}	Euler buckling strength corresponding to the member y and z axis respectively
$f_{Ep\tau}$	Euler buckling shear strength
f_{ET}	torsional elastic buckling strength
f_{Ey}, f_{Ez}	Euler buckling strength corresponding to the member y and z axis respectively
f_h	characteristic hoop buckling strength
f_{he}	elastic hoop buckling strength for tubular section
f_{hec}	elastic hoop buckling strength for cone section
$f_{h,Rd}$	design hoop buckling strength
f_k, f_{kx}, f_{ky}, f_{kp}	characteristic buckling strength
f_m	characteristic bending strength
$f_{m,Rd}$	design bending strength
$f_{m,Red}$	reduced bending strength due to torsional moment
$f_{mh,Rd}$	design bending resistance in the presence of external hydrostatic pressure
f_r	characteristic strength
f_T	characteristic torsional buckling strength
f_{TG}	characteristic torsional buckling strength for girders
$f_{th,Rd}$	design axial tensile resistance in the presence of external hydrostatic pressure
f_y	characteristic yield strength
$f_{y,b}$	characteristic yield strength of brace
$f_{y,c}$	characteristic yield strength of chord
g	gap
h	height
i	radius of gyration
i_e	effective radius of gyration
k, k_g,k_l, k_σ	buckling factor
l, l_L	length, element length
l_e	effective length
l_l	length of longitudinal web stiffener
l_t	length of transverse web stiffener
l_T	distance between sideways support of stiffener
m	modular ration of E_S/E_G
m_q	exponent in resistance equation for cracked tubular joints
n	number of stiffeners
p_f	lateral pressure giving yield in extreme fibre of a continuous stiffener
p_{Sd}	design hydrostatic pressure, design lateral pressure
p_0	equivalent lateral pressure
q_f	design lateral lineload
q_{Sd}	design lateral lineload
r	radius, factor
s	element width, stiffener spacing
s_e	effective width
t	thickness
t_b	bracket thickness

t_c	cone thickness
t_{eff}	effective thickness of chord and internal pipe of a grouted member
t_f	flange thickness
t_w	web thickness
z_p, z_t	distance
z^*	distance
α	coefficient, angle between cylinder and cone geometrical coefficient, factor
α	exponent in stability equation for dented tubular members
α	circumferential angle of dented / corroded / cracked area of tubular section
β	factor
γ	factor
γ_d	material factor to take into account model uncertainties
γ_f	partial factor for actions
γ_M	resulting material factor
γ_m	material factor to take into account uncertainties in material properties
γ_n	material factor to take into account the consequence of failure
δ	dent depth
$\bar{\delta}$	equivalent dent depth
δ_{max}	the sagging in the final state relative to the straight line joining the supports.
δ_0	the pre-camber
δ_1	the variation of the deflection of the beam due to the permanent loads immediately after loading
δ_2	the variation of the deflection of the beam due to the variable loading plus any time dependent deformations due to the permanent load
Δy_1	out-of-straightness of a dented member, measured perpendicular to the dent (along an axis parallel to the dent)
Δy_2	out-of-straightness of a dented member, measured in-line with the dent (along an axis perpendicular with to the dent)
ε	factor
η	hoop buckling factor
θ	angle
θ_c	the included angle for the compression brace
θ_t	the included angle for the tension brace
$\bar{\lambda}$	reduced slenderness, column slenderness parameter
$\bar{\lambda}_e$	reduced equivalent slenderness
$\bar{\lambda}_G$	reduced slenderness
$\bar{\lambda}_p$	reduced plate slenderness
$\bar{\lambda}_s$	reduced slenderness, shell slenderness parameter
$\bar{\lambda}_T$	reduced torsional slenderness
$\bar{\lambda}_{TG}$	reduced torsional slenderness for girders
$\bar{\lambda}_\tau$	reduced slenderness
μ	coefficient, geometric parameter
ν	Poisson's ratio
ξ	factor
ξ_C	correction factor for axial resistance of the dented tubular section
ξ_M	correction factor for bending resistance of the dented tubular section

ξ_δ, ξ_m	empirical factors for characteristic bending strength of grouted member
ρ	rotational stiffness factor
$\sigma_{a,Sd}$	design axial stress in member
$\sigma_{ac,Sd}$	design axial stress including the effect of the hydrostatic capped end axial stress
$\sigma_{at,Sd}$	design axial stress in tubular section at junction due to global actions
$\sigma_{c,Sd}$	maximum combined design compressive stress, design axial stress in the damaged cylinder
$\sigma_{equ,Sd}$	equivalent design axial stress within the conical transition
$\sigma_{h,Sd}$	design hoop stress due to the external hydrostatic pressure
$\sigma_{hc,Sd}$	design hoop stress at unstiffened tubular-cone junctions due to unbalanced radial line forces
$\sigma_{hj,Sd}$	net design hoop stress at a tubular–cone junction
$\sigma_{j,Sd}$	design von Mises' equivalent stress
$\sigma_{m,Sd}$	design bending stress
$\sigma_{mc,Sd}$	design bending stress at the section within the cone due to global actions
$\sigma_{mlc,Sd}$	local design bending stress at the cone side of unstiffened tubular-cone junctions
$\sigma_{mlt,Sd}$	local design bending stress at the tubular side of unstiffened tubular-cone junctions
$\sigma_{mt,Sd}$	design bending stress in tubular section at junction due to global actions
$\sigma_{my,Sd}$	design bending stress due to in-plane bending
$\sigma_{mz,Sd}$	design bending stress due to out-of-plane bending
$\sigma_{p,Sd}$	design hoop stress due to hydrostatic pressure
$\sigma_{tot,Sd}$	total design stress
$\sigma_{q,Sd}$	capped end axial design compression stress due to external hydrostatic pressure
$\sigma_{x,Sd}$	design stress in x direction
$\sigma_{y,Sd}$	design stress in y direction
$\sigma_{y1,Sd}$	larger design stress in y direction (tensile stresses taken as negative)
$\sigma_{y2,Sd}$	smaller design stress in y direction (tensile stresses taken as negative)
τ	factor
τ_{ceg}, τ_{cel}	elastic buckling stress
τ_{crg}, τ_{crl}	critical shear stress
τ_{Sd}	design shear stress
$\tau_{T,Sd}$	shear stress due to design torsional moment
ψ, ψ_x, ψ_y	factors
ϕ	factor

4 GENERAL PROVISIONS

All relevant failure modes for the structure shall be identified and it shall be checked that no corresponding limit state is exceeded.

In this standard the limit states are grouped into:

- Serviceability limit states
- Ultimate limit states
- Fatigue limit states
- Accidental limit states

For definition of the groups of limit states, reference is made to NPD Regulation relating to loadbearing structures in the petroleum activity or ISO 13819-1.

The different groups of limit states are addressed in designated chapters of this standard. In general, the design needs to be checked for all groups of limit states.

The general safety format may be expressed as:

$$S_d \leq R_d \qquad \textbf{(4.1)}$$

where:

S_d	=	$S_k \gamma_f$	Design action effect
R_d	=	$\dfrac{R_k}{\gamma_M}$	Design resistance
S_k	=	Characteristic action effect	
γ_f	=	partial factor for actions	
R_k	=	Characteristic resistance	
γ_M	=	$\gamma_m \cdot \gamma_n \cdot \gamma_d$	Resulting material factor
γ_m	=	Material factor to take into account uncertainties in material properties	
γ_n	=	Material factor to take into account the consequence of failure	
γ_d	=	Material factor to take into account model uncertainties.	

In this standard the values of the resulting material factor are given in the respective chapters.

General requirements for structures are given in NORSOK N-001.

Determination of actions and action effects shall be according to NORSOK N-003.

The steel fabrication shall be according to the requirements in NORSOK M-101.

5 STEEL MATERIAL SELECTION AND REQUIREMENTS FOR NON-DESTRUCTIVE TESTING

5.1 Design class

Selection of steel quality and requirements for inspection of welds shall be based on a systematic classification of welded joints according to the structural significance and complexity of joints. The main criterion for decision of design class (DC) of welded joints is the significance with respect to consequences of failure of the joint. In addition the geometrical complexity of the joint will influence the DC selection.

The selection of joint design class shall be in compliance with Table 5-1 for all permanent structural elements. A similar classification may also be used for temporary structures.

Table 5-1 Classification of structural joints and components

Design Class [1)]	Joint complexity [2)]	Consequences of failure
DC1	High	Applicable for joints and members where failure will have substantial consequences [3)] and the structure possesses limited residual strength. [4).]
DC2	Low	
DC3	High	Applicable for joints and members where failure will be without substantial consequences[3)] due to residual strength. [4).]
DC4	Low	
DC5	Any	Applicable for joints and members where failure will be without substantial consequences. [3)]

Notes:
1) Guidance for classification can be found in Annex K, L, M and N.
2) High joint complexity means joints where the geometry of connected elements and weld type leads to high restraint and to triaxial stress pattern. E.g., typically multiplanar plated connections with full penetration welds
3) "Substantial consequences" in this context means that failure of the joint or member will entail:
   - Danger of loss of human life;
   - Significant pollution;
   - Major financial consequences.
4) Residual strength means that the structure meets requirements corresponding to the damaged condition in the check for Accidental Damage Limit States, with failure in the actual joint or component as the defined damage. See Commentary.

5.2 Steel quality level

Selection of steel quality level for a structural component shall normally be based on the most stringent DC of joints involving the component. Through-thickness stresses shall be assessed.

The minimum requirements for selection of steel material are found in Table 5-2. Selection of a better steel quality than the minimum required in design shall not lead to more stringent requirements in fabrication.

The principal standard for specification of steels is NORSOK M-120, Material data sheets for rolled structural steel. Material selection in compliance with NORSOK M-120 assures toughness and weldability for structures with an operating temperature down to -14°C.

Cast and forged steels shall be in accordance with recognised standards.

If steels of higher specified minimum yield strength than 500 MPa or greater thickness than 150 mm are selected, the feasibility of such a selection shall be assessed in each case.

Traceability of materials shall be in accordance with NORSOK Z-001, Documentation for operation.

Table 5-2 Correlation between design classes and steel quality level

Design Class	**Steel Quality Level**			
	I	**II**	**III**	**IV**
DC1	X			
DC2	(X)	X		
DC3	(X)	X		
DC4	(X)		X	
DC5				X

(X) = Selection where the joint strength is based on transference of tensile stresses in the through thickness direction of the plate. See Commentary.

5.3 Welding and non-destructive testing

The required fracture toughness level shall be determined at the design stage for joints with plate thickness above 50 mm when steel quality level I and II are selected.

The extent of non-destructive examination during fabrication of structural joints shall be in compliance with the inspection category. The selection of inspection category for each welded joint shall be in accordance with Table 5-3 and Table 5-4 for joints with low and high fatigue utilisation respectively.

Welds in joints below 150 m waterdepth should be assumed inaccessible for in-service inspection.

The Principal Standard for welder and welding qualification, welding performance and non-destructive testing is NORSOK M-101, Structural Steel Fabrication.

Table 5-3 Determination of inspection category for details with low fatigue utilisation [1].

Design Class	Type of and level of stress and direction in relation to welded joint.	Inspection category [2]
DC1 and DC2	Welds subjected to high tensile stresses transverse to the weld. [6]	A
	Welds with moderate tensile stresses transverse to the weld and/or high shear stresses. [7]	B [3]
	Welds with low tensile stresses transverse to the weld and/or moderate shear stress. [8]	C [4]
DC3 and DC4	Welds subjected to high tensile stresses transverse to the weld. [6]	B [3]
	Welds with moderate tensile stresses transverse to the weld and/or high shear stresses. [7]	C [4]
	Welds with low tensile stresses transverse to the weld and/or moderate shear stress. [8]	D [5]
DC5	All load bearing connections.	D
	Non load bearing connections.	E

Notes:

1) Low fatigue utilisation means connections with calculated fatigue life longer than 3 times the required fatigue life (Design fatigue life multiplied with the Design Fatigue Factor DFF).
2) It is recommended that areas of the welds where stress concentrations occur be marked as mandatory inspection areas for B, C and D categories as applicable.
3) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category A.
4) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category B.
5) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category C.
6) High tensile stresses mean ULS tensile stresses in excess of 0.85 of design stress.
7) Moderate tensile stresses mean ULS tensile stresses between 0.6 and 0.85 of design stress.
8) Low tensile stresses mean ULS tensile stresses less than 0.6 of design stress.

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Table 5-4 Determination of inspection category for details with high fatigue utilisation [1].

Design Class	Direction of dominating principal stress	Inspection category [3]
DC1 and DC2	Welds with the direction of the dominating dynamic principal stress transverse to the weld (between 45°and 135°)	A [2]
	Welds with the direction of the dominating dynamic principal stress in the direction of the weld (between -45°and 45°)	B[4]
DC3 and DC4	Welds with the direction of the dominating dynamic principal stress transverse to the weld (between 45°and 135°)	B[4]
	Welds with the direction of the dominating dynamic principal stress in the direction of the weld (between -45°and 45°)	C[5]
DC5	Welds with the direction of the dominating dynamic principal stress transverse to the weld (between 45°and 135°)	D
	Welds with the direction of the dominating dynamic principal stress in the direction of the weld (between -45°and 45°)	E

Notes:

1) High fatigue utilisation means connections with calculated fatigue life less than 3 times the required fatigue life (Design fatigue life multiplied with the Design Fatigue Factor DFF).
2) Buttwelds with high fatigue utilisation and Stress Concentration Factor (SCF) less than 1.3 need stricter NDT acceptance criteria. Such criteria need to be developed in each case.
3) For joints in inspection categories B, C or D, the hot spot regions (regions with highest stress range) at welds or areas of welds of special concern shall be addressed with individual notations as mandatory for selected NDT methods.
4) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category A.
5) Welds or parts of welds with no access for in-service inspection and repair should be assigned inspection category B.

6 ULTIMATE LIMIT STATES

6.1 General

This chapter gives provisions for checking of Ultimate Limit States for typical structural elements used in offshore steel structures, where ordinary building codes lack relevant recommendations. Such elements are tubular members, tubular joints, conical transitions and some load situations for plates and stiffened plates. For other types of structural elements NS 3472 or NS-ENV 1993 1-1 (Eurocode 3) apply.

The material factor γ_M is 1.15 for ultimate limit states unless noted otherwise.

The material factors according to Table 6-1 shall be used if NS 3472 or Eurocode 3 is used for calculation of structural resistance:

Table 6-1 Material factors

Type of calculation	Material factor 1)	Value
Resistance of Class 1,2 or 3 cross-sections	γ_{M0}	1.15
Resistance of Class 4 cross-sections	γ_{M1}	1.15
Resistance of member to buckling	γ_{M1}	1.15
Resistance of net section at bolt holes	γ_{M2}	1.3
Resistance of fillet and partial penetration welds	γ_{Mw}	1.3
Resistance of bolted connections	γ_{Mb}	1.3

1) Symbols according to Eurocode 3

The ultimate strength of structural elements and systems should be evaluated by using a rational, justifiable engineering approach.

The structural analysis may be carried out as linear elastic, simplified rigid–plastic, or elastic-plastic analyses. Both first order or second order analyses may be applied. In all cases, the structural detailing with respect to strength and ductility requirement shall conform to the assumption made for the analysis.

When plastic or elastic-plastic analyses are used for structures exposed to cyclic loading (e.g. wave loads) checks must be carried out to verify that the structure will shake down without excessive plastic deformations or fracture due to repeated yielding. A characteristic or design cyclic load history needs to be defined in such a way that the structural reliability in case of cyclic loading (e.g. storm loading) is not less than the structural reliability for Ultimate Limit States for non-cyclic actions.

In case of linear analysis combined with the resistance formulations set down in this standard, shakedown can be assumed without further checks.

6.2 Ductility

It is a fundamental requirement that all failure modes are sufficiently ductile such that the structural behaviour will be in accordance with the anticipated model used for determination of the responses. In general all design procedures, regardless of analysis method, will not capture the true structural behaviour. Ductile failure modes will allow the structure to redistribute forces in accordance with the presupposed static model. Brittle failure modes shall therefore be avoided or shall be verified to have excess resistance compared to ductile modes, and in this way protect the structure from brittle failure.

The following sources for brittle structural behaviour may need to be considered for a steel structure:

1. Unstable fracture caused by a combination of the following factors: brittle material, a design resulting in high local stresses and the possibilities for weld defects.
2. Structural details where ultimate resistance is reached with plastic deformations only in limited areas, making the global behaviour brittle. E.g. partial butt weld loaded transverse to the weld with failure in the weld.
3. Shell buckling.
4. Buckling where interaction between local and global buckling modes occur.

In general a steel structure will be of adequate ductility if the following is satisfied:

1. Material toughness requirements are met, and the design avoids a combination of high local stresses with possibilities of undetected weld defects.
2. Details are designed to develop a certain plastic deflection e.g. partial butt welds subjected to stresses transverse to the weld is designed with excess resistance compared with adjoining plates.
3. Member geometry is selected such that the resistance does not show a sudden drop in capacity when the member is subjected to deformation beyond maximum resistance. (An unstiffened shell in cross-section class 4 is an example of a member that may show such an unfavourable resistance deformation relationship. For definition of cross-section class see NS 3472 or Eurocode 3).
4. Local and global buckling interaction effects are avoided.

6.3 Tubular members

6.3.1 General

The structural strength and stability requirements for steel tubular members are specified in this section.

The requirements given in this section apply to unstiffened and ring stiffened tubulars having a thickness $t \geq 6$ mm, $D/t < 120$ and material meeting the general requirements of Section 5 of this standard. In cases where hydrostatic pressure are present, the structural analysis may proceed on the basis that stresses due to the capped-end forces arising from hydrostatic pressure are either included in or excluded from the analysis. This aspect is discussed in the Commentary.

In the following sub-sections, y and z are used to define the in-plane and out-of-plane axes of a tubular member, respectively.

The requirements assume the tubular is constructed in accordance with the fabrication tolerances given in the NORSOK M-101.

The requirements are formulated for an isolated beam column. This formulation may also be used to check the resistance of frames and trusses, provided that each member is checked for the member forces and moments combined with a representative effective length. The effective length may in lieu of special analyses be determined according to the requirements given in this chapter. Alternatively the Ultimate Limit States for frames or trusses may be determined on basis of non-linear analyses taking into account second order effects. The use of these analyses requires that the assumptions made are fulfilled and justified.

Tubular members subjected solely to axial tension, axial compression, bending, shear, or hydrostatic pressure should be designed to satisfy the strength and stability requirements specified in Sections 6.3.2 to 6.3.6. Tubular members subjected to combined loads without hydrostatic pressure should be designed to satisfy the strength and stability requirements specified in Section 6.3.8. Tubular members subjected to combined loads with hydrostatic pressure should be designed to satisfy the strength and stability requirements specified in Section 6.3.9.

The equations in this section are not using an unique sign convention. Definitions are given in each paragraph.

6.3.2 Axial tension

Tubular members subjected to axial tensile loads should be designed to satisfy the following condition:

$$N_{Sd} \le N_{t,Rd} = \frac{A\, f_y}{\gamma_M} \qquad \textbf{(6.1)}$$

where

N_{Sd}	=	design axial force (tension positive)
f_y	=	characteristic yield strength
A	=	cross sectional area
γ_M	=	1.15.

6.3.3 Axial compression

Tubular members subjected to axial compressive loads should be designed to satisfy the following condition:

$$N_{Sd} \le N_{c,Rd} = \frac{Af_c}{\gamma_M} \qquad \textbf{(6.2)}$$

where

N_{Sd}	=	design axial force (compression positive)
f_c	=	characteristic axial compressive strength
γ_M	=	see section 6.3.7

In the absence of hydrostatic pressure the characteristic axial compressive strength for tubular members shall be the smaller of the in-plane or out-of-plane buckling strength determined from the following equations:

$$f_c = [1.0 - 0.28\bar{\lambda}^2] f_y \quad \text{for } \bar{\lambda} \le 1.34 \tag{6.3}$$

$$f_c = \frac{0.9}{\bar{\lambda}^2} f_y \quad \text{for } \bar{\lambda} > 1.34 \tag{6.4}$$

$$\bar{\lambda} = \sqrt{\frac{f_{cl}}{f_E}} = \frac{kl}{\pi i}\sqrt{\frac{f_{cl}}{E}} \tag{6.5}$$

where

f_{cl}	=	characteristic local buckling strength
$\bar{\lambda}$	=	column slenderness parameter
f_E	=	smaller Euler buckling strength in y or z direction
E	=	Young's modulus of elasticity, $2.1 \cdot 10^5$ MPa
k	=	effective length factor, see Section 6.3.8.2
l	=	longer unbraced length in y or z direction
i	=	radius of gyration

The characteristic local buckling strength should be determined from:

$$f_{cl} = f_y \quad \text{for } \frac{f_y}{f_{cle}} \le 0.170 \tag{6.6}$$

$$f_{cl} = \left(1.047 - 0.274\frac{f_y}{f_{cle}}\right) f_y \quad \text{for } \frac{f_y}{f_{cle}} > 0.170 \tag{6.7}$$

and

$$f_{cle} = 2 C_e E \frac{t}{D} \tag{6.8}$$

where

f_{cle}	=	characteristic elastic local buckling strength
C_e	=	critical elastic buckling coefficient = 0.3
D	=	outside diameter
t	=	wall thickness

For $\frac{f_y}{f_{cle}} > 0.170$ the tubular is a class 4 cross section and may behave as a shell. Shell structures may have a brittle structure failure mode. Reference is made to section 6.2. For class 4 cross sections increased γ_M values shall be used according to equation (6.22).

6.3.4 Bending

Tubular members subjected to bending loads should be designed to satisfy the following condition:

$$M_{Sd} \le M_{Rd} = \frac{f_m W}{\gamma_M} \qquad \textbf{(6.9)}$$

where

M_{Sd} = design bending moment
f_m = characteristic bending strength
W = elastic section modulus
γ_M = see section 6.3.7

The characteristic bending strength for tubular members should be determined from:

$$f_m = \frac{Z}{W} f_y \qquad \text{for } \frac{f_y D}{Et} \le 0.0517 \qquad \textbf{(6.10)}$$

$$f_m = \left(1.13 - 2.58\left(\frac{f_y D}{Et}\right)\right)\left(\frac{Z}{W}\right) f_y \qquad 0.0517 < \frac{f_y D}{Et} \le 0.1034 \qquad \textbf{(6.11)}$$

$$f_m = \left(0.94 - 0.76\left(\frac{f_y D}{Et}\right)\right)\left(\frac{Z}{W}\right) f_y \qquad 0.1034 < \frac{f_y D}{Et} \le 120\frac{f_y}{E} \qquad \textbf{(6.12)}$$

where

W = elastic section modulus
$= \frac{\pi}{32}\frac{\left[D^4 - (D-2t)^4\right]}{D}$
Z = plastic section modulus
$= \frac{1}{6}\left[D^3 - (D-2t)^3\right]$

For $\frac{f_y}{f_{cle}} > 0.170$ the tubular is a class 4 cross section and may behave as a shell. Shell structures may have a brittle structure failure mode. Reference is made to section 6.2. For class 4 cross sections increased γ_M values shall be used according to equation (6.22).

6.3.5 Shear

Tubular members subjected to beam shear forces should be designed to satisfy the following condition:

$$V_{Sd} \le V_{Rd} = \frac{A f_y}{2\sqrt{3}\,\gamma_M} \qquad \textbf{(6.13)}$$

where

V_{Sd} = design shear force
f_y = yield strength
A = cross sectional area
γ_M = 1.15

Tubular members subjected to shear from torsional moment should be designed to satisfy the following condition:

$$M_{T,Sd} \le M_{T,Rd} = \frac{2I_p f_y}{D\sqrt{3}\,\gamma_M} \qquad \textbf{(6.14)}$$

where

$M_{T,Sd}$ = design torsional moment
I_p = polar moment of inertia = $\frac{\pi}{32}\left[D^4 - (D-2t)^4\right]$

6.3.6 Hydrostatic pressure

6.3.6.1 Hoop buckling

Tubular members subjected to external pressure should be designed to satisfy the following condition:

$$\sigma_{p,Sd} \le f_{h,Rd} = \frac{f_h}{\gamma_M} \qquad \textbf{(6.15)}$$

$$\sigma_{p,Sd} = \frac{p_{Sd} D}{2t} \qquad \textbf{(6.16)}$$

where

f_h = characteristic hoop buckling strength
$\sigma_{p,Sd}$ = design hoop stress due to hydrostatic pressure (compression positive)
p_{Sd} = design hydrostatic pressure
γ_M = see section 6.3.7

If out-of-roundness tolerances do not meet the requirements given in NORSOK-M-101, guidance on calculating reduced strength is given in the Commentary.

$$f_h = f_y, \qquad \text{for } f_{he} > 2.44 f_y \qquad \textbf{(6.17)}$$

$$f_h = 0.7 f_y \left[\frac{f_{he}}{f_y}\right]^{0.4} \qquad \text{for } 2.44 f_y \ge f_{he} > 0.55 f_y \qquad \textbf{(6.18)}$$

$$f_h = f_{he}, \qquad \text{for } f_{he} \le 0.55 f_y \qquad \textbf{(6.19)}$$

The elastic hoop buckling strength, f_{he}, is determined from the following equation:

$$f_{he} = 2 C_h E \frac{t}{D} \tag{6.20}$$

where

C_h	=	0.44 t/D	for $\mu \geq 1.6D/t$
	=	$0.44\ t/D + 0.21\ (D/t)^3/\mu^4$	for $0.825D/t \leq \mu < 1.6D/t$
	=	$0.737/(\mu - 0.579)$	for $1.5 \leq \mu < 0.825D/t$
	=	0.80	for $\mu < 1.5$

and where the geometric parameter, μ, is defined as:

$$\mu = \frac{L}{D}\sqrt{\frac{2D}{t}} \qquad \text{and}$$

L = length of tubular between stiffening rings, diaphragms, or end connections

6.3.6.2 Ring stiffener design

The circumferential stiffening ring size may be selected on the following approximate basis:

$$I_c = f_{he} \frac{t L_r D^2}{8E} \tag{6.21}$$

where

I_c	=	required moment of inertia for ring composite section
L_r	=	ring spacing
D	=	diameter (See Note 2 for external rings.)

Notes:

1. *Equation (6.21) assumes that the yield strength of the stiffening ring is equal to or greater than that of the tubular.*
2. *For external rings, D in equation (6.21) should be taken to the centroid of the composite ring.*
3. *An effective width of shell equal to* $1.1\sqrt{D \cdot t}$ *may be assumed as the flange for the composite ring section.*
4. *Where out-of-roundness in excess of tolerances given in NORSOK-M-101 is permitted, larger stiffeners may be required. The bending due to out-of-roundness should be specially investigated.*

Local buckling of ring stiffeners with flanges may be excluded as a possible failure mode provided that the following requirements are fulfilled:

$$\frac{h}{t_w} \leq 1.1\sqrt{\frac{E}{f_y}}$$

and

$$\frac{b}{t_f} \leq 0.3\sqrt{\frac{E}{f_y}}$$

where

h = web height
t_w = web thickness
b = half the width of flange of T-stiffeners
t_f = thickness of flange

Local buckling of ring stiffeners without flanges may be excluded as a possible failure mode provided that:

$$\frac{h}{t_w} \le 0.4\sqrt{\frac{E}{f_y}}$$

6.3.7 Material factor

The material factor, γ_M, is given as:

$$\gamma_M = 1.15 \quad \text{for } \bar{\lambda}_s < 0.5 \tag{6.22}$$
$$\gamma_M = 0.85 + 0.60\bar{\lambda}_s \quad \text{for } 0.5 \le \bar{\lambda}_s \le 1.0$$
$$\gamma_M = 1.45 \quad \text{for } \bar{\lambda}_s > 1.0$$

where

$$\bar{\lambda}_s^2 = \frac{f_y}{\sigma_{j,Sd}}\left(\frac{\sigma_{c,Sd}}{f_{cle}} + \frac{\sigma_{p,Sd}}{f_{he}}\right) \tag{6.23}$$

$$\sigma_{j,Sd} = \sqrt{\sigma_{c,Sd}^2 - \sigma_{c,Sd}\sigma_{p,Sd} + \sigma_{p,Sd}^2} \tag{6.24}$$

where f_{cle} is calculated from (6.8) and f_{he} from (6.20). $\sigma_{p,Sd}$ is obtained from (6.16) and

$$\sigma_{c,Sd} = \frac{N_{Sd}}{A} + \frac{\sqrt{M_{y,Sd}^2 + M_{z,Sd}^2}}{W} \tag{6.25}$$

6.3.8 Tubular members subjected to combined loads without hydrostatic pressure

6.3.8.1 Axial tension and bending

Tubular members subjected to combined axial tension and bending loads should be designed to satisfy the following condition at all cross sections along their length:

$$\left(\frac{N_{Sd}}{N_{t,Rd}}\right)^{1.75} + \frac{\sqrt{M_{y,Sd}^2 + M_{z,Sd}^2}}{M_{Rd}} \le 1.0 \tag{6.26}$$

where

$M_{y,Sd}$ = design bending moment about member y-axis (in-plane)
$M_{z,Sd}$ = design bending moment about member z-axis (out-of-plane)
N_{Sd} = design axial tensile force

6.3.8.2 Axial compression and bending

Tubular members subjected to combined axial compression and bending should be designed to satisfy the following conditions at all cross sections along their length:

$$\frac{N_{Sd}}{N_{c,Rd}} + \frac{1}{M_{Rd}}\left\{\left[\frac{C_{my}M_{y,Sd}}{1-\frac{N_{Sd}}{N_{Ey}}}\right]^2 + \left[\frac{C_{mz}M_{z,Sd}}{1-\frac{N_{Sd}}{N_{Ez}}}\right]^2\right\}^{0.5} \le 1.0 \quad \textbf{(6.27)}$$

and

$$\frac{N_{Sd}}{N_{cl,Rd}} + \frac{\sqrt{M_{y,Sd}^2 + M_{z,Sd}^2}}{M_{Rd}} \le 1.0 \quad \textbf{(6.28)}$$

where

N_{Sd} = design axial compression force
C_{my}, C_{mz} = reduction factors corresponding to the member y and z axes, respectively
N_{Ey}, N_{Ez} = Euler buckling strengths corresponding to the member y and z axes, respectively
$N_{cl,Rd} = \frac{f_{cl} \cdot A}{\gamma_M}$ design axial local buckling resistance

$$N_{Ey} = \frac{\pi^2 EA}{\left[\frac{kl}{i}\right]_y^2} \quad \textbf{(6.29)}$$

$$N_{Ez} = \frac{\pi^2 EA}{\left[\frac{kl}{i}\right]_z^2} \quad \textbf{(6.30)}$$

k in Equations (6.29) and (6.30) relate to buckling in the y and z directions, respectively.

These factors can be determined using a rational analysis that includes joint flexibility and side-sway. In lieu of such a rational analysis, values of effective length factors, k, and moment reduction factors, C_m, may be taken from Table 6-2. All lengths are measured centreline to centreline.

Table 6-2 Effective length and moment reduction factors for member strength checking

Structural element	**k**	**C_m (1)**
Superstructure legs		
- Braced	1.0	(a)
- Portal (unbraced)	k(2)	(a)
Jacket legs and piling		
- Grouted composite section	1.0	(c)
- Ungrouted jacket legs	1.0	(c)
- Ungrouted piling between shim points	1.0	(b)
Jacket braces		
- Primary diagonals and horizontals	0.7	(b) or (c)
- K-braces(3)	0.7	(c)
- Longer segment length of X-braces(3)	0.8	(c)
Secondary horizontals	0.7	(c)

Notes:

1. *C_m values for the cases defined in Table 6-2 are as follows:*
   (a) 0.85
   (b) for members with no transverse loading,
   $C_m = 0.6 - 0.4\, M_{1,Sd}/M_{2,Sd}$
   where $M_{1,Sd}/M_{2,Sd}$ is the ratio of smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. $M_{1,Sd}/M_{2,Sd}$ is positive when the number is bent in reverse curvature, negative when bent in single curvature.
   (c) for members with transverse loading,
   $C_m = 1.0 - 0.4\, N_{c,Sd}/N_E$, or 0.85, whichever is less, and $N_E = N_{Ey}$ or N_{Ez} as appropriate.
2. *Use Effective Length Alignment Chart in Commentary.*
3. *At least one pair of members framing into the a K- or X-joint must be in tension if the joint is not braced out-of-plane. For X-braces, when all members are in compression, the k-factor should be determined using the procedures given in the Commentary.*
4. *The effective length and C_m factors given in Table 6-2 do not apply to cantilever members and the member ends are assumed to be rotationally restrained in both planes of bending.*

6.3.8.3 Interaction shear and bending moment

Tubular members subjected to beam shear force and bending moment should be designed to satisfy the following condition:

$$\frac{M_{Sd}}{M_{Rd}} \le \sqrt{1-\frac{V_{Sd}}{V_{Rd}}} \qquad \text{for } \frac{V_{Sd}}{V_{Rd}} \ge 0.4 \qquad \textbf{(6.31)}$$

$$\frac{M_{Sd}}{M_{Rd}} \le 1.0 \qquad \text{for } \frac{V_{Sd}}{V_{Rd}} < 0.4 \qquad \textbf{(6.32)}$$

6.3.8.4 Interaction shear, bending moment and torsional moment

Tubular members subjected to beam shear force, bending moment and torsion should be designed to satisfy the following condition:

$$\frac{M_{Sd}}{M_{Red,Rd}} \le \sqrt{1 - \frac{V_{Sd}}{V_{Rd}}} \qquad \textbf{(6.33)}$$

where

$M_{Red,Rd} = \dfrac{W f_{m,Red}}{\gamma_M}$

$f_{m,Red} = f_m \sqrt{1 - 3\left(\dfrac{\tau_{T,Sd}}{f_d}\right)^2}$

$\tau_{T,Sd} = \dfrac{M_{T,Sd}}{2\pi R^2 t}$

$f_d = \dfrac{f_y}{\gamma_M}$

R = radius of tubular member

γ_M = see section 6.3.7

6.3.9 Tubular members subjected to combined loads with hydrostatic pressure

The design provisions in this section are divided into two categories. In Method A it is assumed that the capped-end compressive forces due to the external hydrostatic pressure are not included in the structural analysis. Alternatively, the design provisions in Method B assume that such forces are included in the analysis as external nodal forces. Dependent upon the method used in the analysis, the interaction equations in either Method A or Method B should be satisfied at all cross sections along their length.

It should be noted that the equations in this section are not applicable unless Equation (6.15) is first satisfied.

For guidance on significance of hydrostatic pressure see Commentary.

6.3.9.1 Axial tension, bending, and hydrostatic pressure

Tubular members subjected to combined axial tension, bending, and hydrostatic pressure should be designed to satisfy the following equations in either Method A or B at all cross sections along their length.

Method A ($\sigma_{a,Sd}$ is in tension)

In this method, the calculated value of member axial stress, $\sigma_{a,Sd}$, should not include the effect of the hydrostatic capped-end axial stress. The capped-end axial compression due to external hydrostatic pressure, $\sigma_{q,Sd}$, can be taken as $0.5\sigma_{p,Sd}$. This implies that, the tubular member takes the entire capped-end force arising from external hydrostatic pressure. In reality, the stresses in the member due to this force depend on the restraint provided by the rest of the structure on the member. The stress computed from a more rigorous analysis may be substituted for $0.5\sigma_{p,Sd}$.

(a) For $\sigma_{a,Sd} \ge \sigma_{q,Sd}$ (net axial tension condition)

$$\frac{\sigma_{a,Sd} - \sigma_{q,Sd}}{f_{th,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \le 1.0 \qquad \textbf{(6.34)}$$

where

- $\sigma_{a,Sd}$ = design axial stress that excludes the effect of capped-end axial compression arising from external hydrostatic pressure (tension positive)
- $\sigma_{q,Sd}$ = capped-end design axial compression stress due to external hydrostatic pressure $(= 0.5\sigma_{p,Sd})$ (compression positive)
- $\sigma_{my,Sd}$ = design in plane bending stress
- $\sigma_{mz,Sd}$ = design out of plane bending stress
- $f_{th,Rd}$ = design axial tensile resistance in the presence of external hydrostatic pressure which is given by the following formula:

$$f_{th,Rd} = \frac{f_y}{\gamma_M}\left[\sqrt{1 + 0.09B^2 - B^{2\eta}} - 0.3B\right] \qquad \textbf{(6.35)}$$

- $f_{mh,Rd}$ = design bending resistance in the presence of external hydrostatic pressure which is given by the following formula:

$$f_{mh,Rd} = \frac{f_m}{\gamma_M}\left[\sqrt{1 + 0.09B^2 - B^{2\eta}} - 0.3B\right] \qquad \textbf{(6.36)}$$

- γ_M = see section 6.3.7

and

$$B = \frac{\sigma_{p,Sd}}{f_{h,Rd}}, \quad B \le 1.0 \qquad \textbf{(6.37)}$$

$$\eta = 5 - 4\frac{f_h}{f_y} \qquad \textbf{(6.38)}$$

(b) For $\sigma_{a,Sd} < \sigma_{q,Sd}$ (net axial compression condition)

$$\frac{\left|\sigma_{a,Sd} - \sigma_{q,Sd}\right|}{f_{cl,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \le 1.0 \qquad \textbf{(6.39)}$$

$$f_{cl,Rd} = \frac{f_{cl}}{\gamma_m} \qquad \textbf{(6.40)}$$

where f_{cl} is found from equation (6.6) and (6.7).

when $\sigma_{c,Sd} > 0.5\frac{f_{cle}}{\gamma_M}$ and $f_{cle} > 0.5f_{he}$, the following equation should also be satisfied:

$$\frac{\sigma_{c,Sd} - 0.5\dfrac{f_{he}}{\gamma_M}}{\dfrac{f_{cle}}{\gamma_M} - 0.5\dfrac{f_{he}}{\gamma_M}} + \left[\frac{\sigma_{p,Sd}}{\dfrac{f_{he}}{\gamma_M}}\right]^2 \le 1.0 \qquad \textbf{(6.41)}$$

in which $\sigma_{c,Sd} = \sigma_{m,Sd} + \sigma_{q,Sd} - \sigma_{a,Sd}$; $\sigma_{c,Sd}$ should reflect the maximum combined compressive stress.

$$\sigma_{m,Sd} = \frac{\sqrt{M_{z,Sd}^2 + M_{y,Sd}^2}}{W}$$

γ_M = see section 6.3.7

Method B ($\sigma_{ac,Sd}$ is in tension)

In this method, the calculated member axial stress, $\sigma_{ac,Sd}$, includes the effect of the hydrostatic capped-end axial stress. Only the following equation needs to be satisfied:

$$\frac{\sigma_{ac,Sd}}{f_{th,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \le 1.0 \qquad \textbf{(6.42)}$$

where

$\sigma_{ac,Sd}$ = design axial stress that includes the effect of the capped-end compression arising from external hydrostatic pressure (tension positive)

6.3.9.2 Axial compression, bending, and hydrostatic pressure

Tubular members subjected to combined compression, bending, and hydrostatic pressure should be proportioned to satisfy the following requirements at all cross sections along their length.

Method A ($\sigma_{a,Sd}$ is in compression)

$$\frac{\sigma_{a,Sd}}{f_{ch,Rd}} + \frac{1}{f_{mh,Rd}}\left[\left(\frac{C_{my}\sigma_{my,Sd}}{1-\dfrac{\sigma_{a,Sd}}{f_{Ey}}}\right)^2 + \left(\frac{C_{mz}\sigma_{mz,Sd}}{1-\dfrac{\sigma_{a,Sd}}{f_{Ez}}}\right)^2\right]^{0.5} \le 1.0 \qquad \textbf{(6.43)}$$

$$\frac{\sigma_{a,Sd} + \sigma_{q,Sd}}{f_{cl,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \le 1.0 \qquad \textbf{(6.44)}$$

where

$\sigma_{a,Sd}$ = design axial stress that excludes the effect of capped-end axial compression arising from external hydrostatic pressure (compression positive)

$$f_{Ey} = \frac{\pi^2 E}{\left[\frac{kl}{i}\right]_y^2} \quad \textbf{(6.45)}$$

$$f_{Ez} = \frac{\pi^2 E}{\left[\frac{kl}{i}\right]_z^2} \quad \textbf{(6.46)}$$

$f_{ch,Rd}$ = design axial compression strength in the presence of external hydrostatic pressure which is given by the following formulas:

$$f_{ch,Rd} = \frac{1}{2}\frac{f_{cl}}{\gamma_M}[\xi - \frac{2\sigma_{q,Sd}}{f_{cl}} + \sqrt{\xi^2 + 1.12\bar{\lambda}^2 \frac{\sigma_{q,Sd}}{f_{cl}}}] \qquad \text{for } \bar{\lambda} < 1.34\sqrt{\left[(1-\frac{2\sigma_{q,Sd}}{f_{cl}})\right]^{-1}} \quad \textbf{(6.47)}$$

and

$$f_{ch,Rd} = \frac{0.9}{\bar{\lambda}^2}\frac{f_{cl}}{\gamma_M}, \qquad \text{for } \bar{\lambda} \geq 1.34\sqrt{\left[(1-\frac{2\sigma_{q,Sd}}{f_{cl}})\right]^{-1}} \quad \textbf{(6.48)}$$

where

$$\xi = 1 - 0.28\bar{\lambda}^2 \quad \textbf{(6.49)}$$

When $\sigma_{c,Sd} > 0.5\frac{f_{he}}{\gamma_M}$ and $f_{cle} > 0.5 f_{he}$, equation (6.41), in which $\sigma_{c,Sd} = \sigma_{m,Sd} + \sigma_{q,Sd} + \sigma_{a,Sd}$, should also be satisfied.

γ_M = see section 6.3.7

Method B ($\sigma_{ac,Sd}$ is in compression)

(a) for $\sigma_{ac,Sd} > \sigma_{q,Sd}$

$$\frac{\sigma_{ac,Sd} - \sigma_{q,Sd}}{f_{ch,Rd}} + \frac{1}{f_{mh,Rd}}\left[\left(\frac{C_{my}\sigma_{my,Sd}}{1 - \frac{\sigma_{ac,Sd} - \sigma_{q,Sd}}{f_{Ey}}}\right)^2 + \left(\frac{C_{mz}\sigma_{mz,Sd}}{1 - \frac{\sigma_{ac,Sd} - \sigma_{q,Sd}}{f_{Ez}}}\right)^2\right]^{0.5} \leq 1.0 \quad \textbf{(6.50)}$$

$$\frac{\sigma_{ac,Sd}}{f_{cl,Rd}} + \frac{\sqrt{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}}{f_{mh,Rd}} \leq 1.0 \quad \textbf{(6.51)}$$

$\sigma_{ac,Sd}$ = design axial stress that includes the effect of capped-end axial compression arising from external hydrostatic pressure (compression positive)

When $\sigma_{c,Sd} > 0.5\frac{f_{he}}{\gamma_M}$ and $\frac{f_{cle}}{\gamma_M} > 0.5\frac{f_{he}}{\gamma_M}$, equation (6.41), in which $\sigma_{c,Sd} = \sigma_{m,Sd} + \sigma_{ac,Sd}$, should also be satisfied.

(b) for $\sigma_{ac,Sd} \le \sigma_{q,Sd}$

For $\sigma_{ac,Sd} \le \sigma_{q,Sd}$, equation (6.51) should be satisfied.

When $\sigma_{c,Sd} > 0.5\frac{f_{he}}{\gamma_M}$ and $\frac{f_{cle}}{\gamma_M} > 0.5\frac{f_{he}}{\gamma_M}$, equation (6.41), in which $\sigma_{c,Sd} = \sigma_{m,Sd} + \sigma_{ac,Sd}$, should also be satisfied.

γ_M = see section 6.3.7

6.4 Tubular joints

6.4.1 General

The following provisions apply to the design of tubular joints formed by the connection of two or more members. Terminology for simple joints is defined in Figure 6-1. Figure 6-1 also gives some design requirements with respect to joint geometry. The gap for simple K-joints should be larger than 50 mm and less than D.

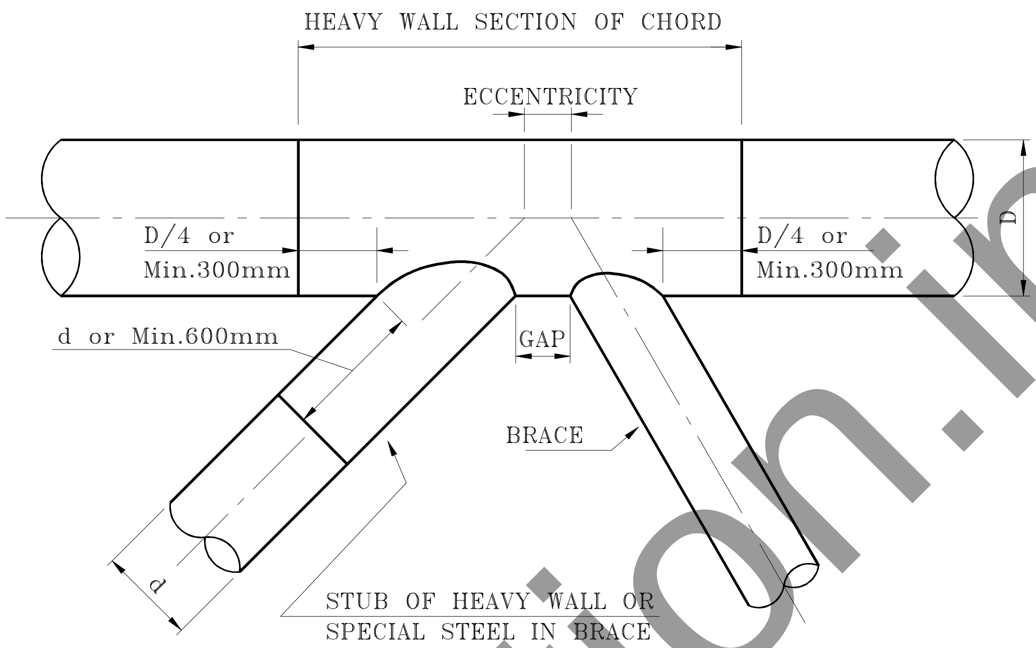


Figure 6-1 Detail of simple joint

Reductions in secondary (deflection induced) bending moments or inelastic relaxation through use of joint elastic stiffness may be considered. In certain instances, hydrostatic pressure effects may be significant.

6.4.2 Joint classification

Joint classification is the process whereby the axial force in a given brace is subdivided into K, X and Y components of actions, corresponding to the three joint types for which resistance equations exist. Such subdivision normally considers all of the members in one plane at a joint. For purposes of this provision, brace planes within ±15º of each other may be considered as being in a common plane. Each brace in the plane can have a unique classification that could vary with action condition. The classification can be a mixture between the above three joint types. Once the breakdown into axial components is established, the resistance of the joint can be estimated using the procedures in 6.4.3.

Figure 6-2 provides some simple examples of joint classification. For a brace to be considered as K-joint classification, the axial force in the brace should be balanced to within 10% by forces in other braces in the same plane and on the same side of the joint. For Y-joint classification, the axial force in the brace is reacted as beam shear in the chord. For X-joint classification, the axial force in the brace is carried through the chord to braces on the opposite side.

Additional explanation of joint-classification is found in the Commentary

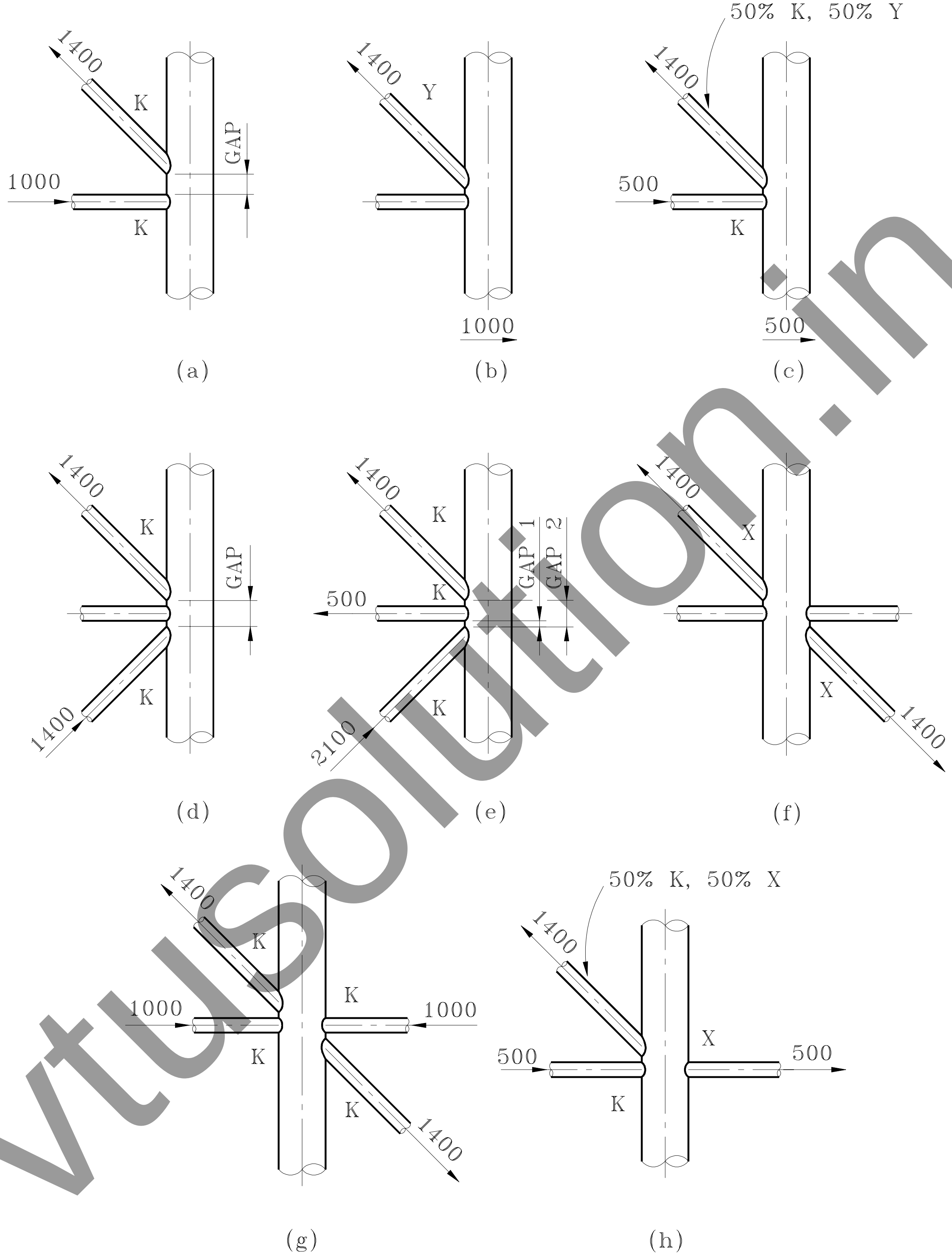


Figure 6-2 Classification of simple joints

6.4.3 Strength of simple joints

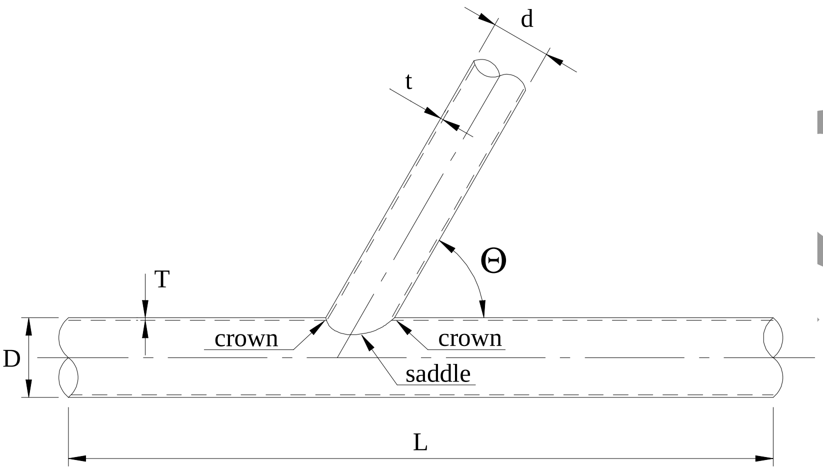
6.4.3.1 General

The validity range for application of the practice defined in 6.4.3 is as follows:

$0.2 \le \beta \le 1.0$
$10 \le \gamma \le 50$
$30^\circ \le \theta \le 90^\circ$

$$\frac{g}{D} \ge -0.6 \text{ (for K joints)}$$

The above geometry parameters are defined in Figure 6-3 to Figure 6-6.



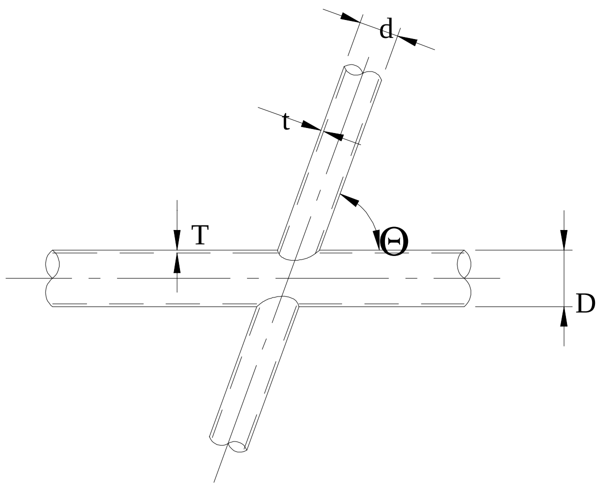
$$\beta = \frac{d}{D}$$

$$\gamma = \frac{D}{2T}$$

$$\tau = \frac{t}{T}$$

$$\alpha = \frac{2L}{D}$$

Figure 6-3 Definition of geometrical parameters for T- or Y-joints



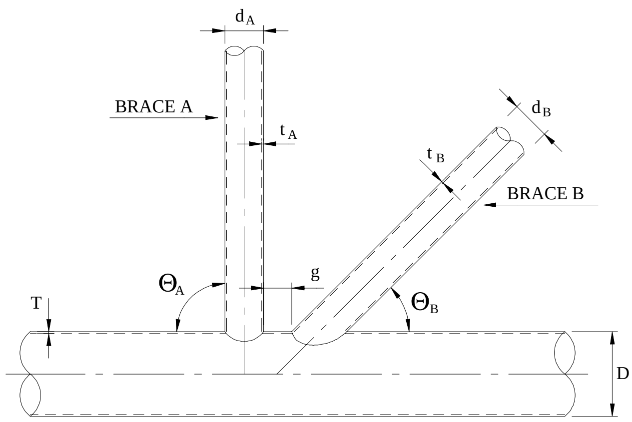
$$\beta = \frac{d}{D}$$

$$\gamma = \frac{D}{2T}$$

$$\tau = \frac{t}{T}$$

$$\alpha = \frac{2L}{D}$$

Figure 6-4 Definition of geometrical parameters for X-joints

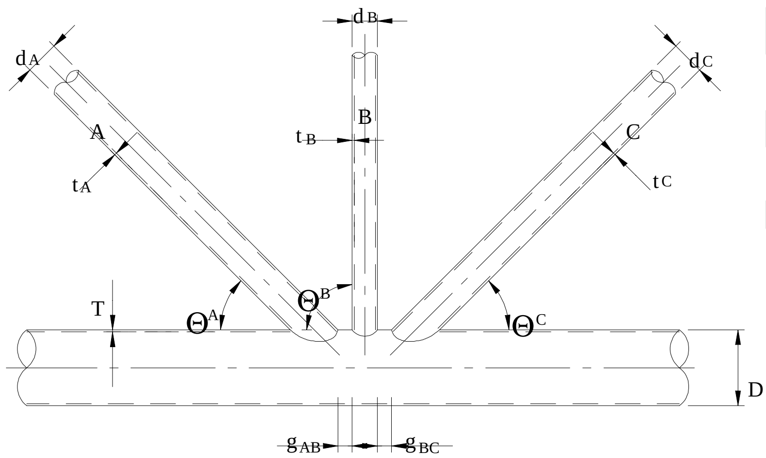


$$\beta_A = \frac{d_A}{D} \qquad \beta_B = \frac{d_B}{D}$$

$$\tau_A = \frac{t_A}{T} \qquad \tau_B = \frac{t_B}{T}$$

$$\gamma = \frac{D}{2T} \qquad \zeta = \frac{g}{D}$$

Figure 6-5 Definition of geometrical parameters for K-joints



$$\beta_A = \frac{d_A}{D} \qquad \beta_B = \frac{d_B}{D} \qquad \beta_C = \frac{d_C}{D}$$

$$\tau_A = \frac{t_A}{T} \qquad \tau_B = \frac{t_B}{T} \qquad \tau_C = \frac{t_C}{T}$$

$$\zeta_{AB} = \frac{g_{AB}}{D} \qquad \zeta_{BC} = \frac{g_{BC}}{D} \qquad \gamma = \frac{D}{2T}$$

Figure 6-6 Definition of geometrical parameters for KT-joints

6.4.3.2 Basic resistance

Tubular joints without overlap of principal braces and having no gussets, diaphragms, grout, or stiffeners should be designed using the following guidelines.

The characteristic resistances for simple tubular joints are defined as follows:

$$N_{Rd} = \frac{f_y T^2}{\gamma_M \sin\theta} Q_u Q_f \qquad \textbf{(6.52)}$$

$$M_{Rd} = \frac{f_y T^2 d}{\gamma_M \sin\theta} Q_u Q_f \qquad \textbf{(6.53)}$$

Where

N_{Rd} = the joint design axial resistance
M_{Rd} = the joint design bending moment resistance
f_y = the yield strength of the chord member at the joint
γ_M = 1.15

For joints with joint cans, N_{Rd} shall not exceed the resistance limits defined in 6.4.3.5

For braces with axial forces with a classification that is a mixture of K, Y and X joints, a weighted average of N_{Rd} based on the portion of each in the total action is used to calculate the resistance.

6.4.3.3 Strength factor Q_u

Q_u varies with the joint and action type, as given in Table 6-3.

Table 6-3 Values for Q_u

	Brace action			
Joint Classification	**Axial Tension**	**Axial Compression**	**In-plane Bending**	**Out-of-plane bending**
K	$(1.9+19\beta)Q_\beta^{0.5}Q_g Q_{yy}$	$(1.9+19\beta)Q_\beta^{0.5}Q_g Q_{yy}$	$4.5\beta\,\gamma^{0.5}$	$3.2\gamma^{0.5\beta^2}$
Y	30β	$(1.9+19\beta)Q_\beta^{0.5}$	$4.5\beta\,\gamma^{0.5}$	$3.2\gamma^{0.5\beta^2}$
X	23β for $\beta \le 0.9$ $21+(\beta\text{-}0.9)(17\gamma-220)$ for $\beta > 0.9$	$(2.8+14\beta)Q_\beta$	$4.5\beta\,\gamma^{0.5}$	$3.2\gamma^{0.5\beta^2}$

The following notes apply to Table 6-3:

(a) Q_β is a geometric factor defined by:

$$Q_\beta = \frac{0.3}{\beta(1-0.833\beta)} \qquad \text{for } \beta > 0.6$$

$$Q_\beta = 1.0 \qquad \text{for } \beta \le 0.6$$

(b) Q_g is a gap factor defined by:

$$Q_g = 1.9 - \left(\frac{g}{D}\right)^{0.5} \qquad \text{for } \frac{g}{T} \ge 2.0, \text{but } Q_g \ge 1.0$$

$$Q_g = 0.13 + 0.65\phi\gamma^{0.5} \qquad \text{for } \frac{g}{T} \le -2.0$$

$$\text{where } \phi = \frac{t\, f_{y,b}}{T\, f_{y,c}}$$

$f_{y,b}$ = yield strength of brace

$f_{y,c}$ = yield strength of chord

Q_g = linear interpolated value between the limiting values of the above expressions for $-2.0 \le \frac{g}{T} \le 2.0$

(b) Q_{yy} is an angle correction factor defined by:

$$Q_{yy} = 1.0 \qquad \text{when } \theta_t \le 4\theta_c - 90°$$

$$Q_{yy} = \frac{110° + 4\theta_c - \theta_t}{200°} \qquad \text{when } \theta_t > 4\theta_c - 90°$$

with θ_c and θ_t referring to the included angle for the compression and tension brace, respectively.

6.4.3.4 Chord action factor Q_f

Q_f is a design factor to account for the presence of factored actions in the chord.

$$Q_f = 1.0 - \lambda c A^2 \qquad \textbf{(6.54)}$$

Where:

λ	=	0.030 for brace axial force in equation (6.52)
	=	0.045 for brace in-plane bending moment in equation (6.53)
	=	0.021 for brace out-of-plane bending moment in equation (6.53)
c	=	14 for Y and K joints
	=	25 for X joints

The parameter A is defined as follows:

$$A^2 = \left(\frac{\sigma_{a,Sd}}{f_y}\right)^2 + \left(\frac{\sigma_{my,Sd}^2 + \sigma_{mz,Sd}^2}{f_m^2}\right) \qquad \textbf{(6.55)}$$

where:

$\sigma_{a,Sd}$	=	design axial stress in chord
$\sigma_{my,Sd}$	=	design in-plane bending stress in chord
$\sigma_{mz,Sd}$	=	design out-of-plane bending stress in chord
f_y	=	yield strength
f_m	=	characteristic bending strength for the chord determined from equation (6.10)-(6.12)

The chord thickness at the joint should be used in the above calculations. The highest value of A for the chord on either side of the brace intersection should be used in Equation (6.54).

Apart from X joints with $\beta > 0.9$, Q_f may be set to unity if the magnitude of the chord axial tension stress is greater than the maximum combined stress due to chord moments.

6.4.3.5 Design axial resistance for X and Y joints with joint cans

For Y and X joints with axial force and where a joint can is specified, the joint design resistance should be calculated as follows:

$$N_{Rd} = \left(r + (1 - r)\left(\frac{T_n}{T_c}\right)^2 \right) N_{can,Rd} \qquad \textbf{(6.56)}$$

where

$N_{can,Rd}$ = N_{Rd} from Equation (6.52) based on chord can geometric and material properties
T_n = nominal chord member thickness
T_c = chord can thickness
r = L/D for joints with $\beta \le 0.9$
= $\left[\beta + (1-\beta)\left(10\frac{L}{D} - 9\right)\right]$ for joints with $\beta > 0.9$
L = the least distance between crown and edge of chord can, excluding taper

In no case shall r be taken as greater than unity.

6.4.3.6 Strength check

Joint resistance shall satisfy the following interaction equation for axial force and/or bending moments in the brace:

$$\frac{N_{Sd}}{N_{Rd}} + \left(\frac{M_{y,Sd}}{M_{y,Rd}}\right)^2 + \frac{M_{z,Sd}}{M_{z,Rd}} \le 1 \qquad \textbf{(6.57)}$$

where:

N_{Sd} = design axial force in the brace member
N_{Rd} = the joint design axial resistance
$M_{y,Sd}$ = design in-plane bending moment in the brace member
$M_{z,Sd}$ = design out-of-plane bending moment in the brace member
$M_{y,Rd}$ = design in-plane bending resistance
$M_{z,Rd}$ = design out-of-plane bending resistance

6.4.4 Overlap joints

Braces that overlap in- or out-of-plane at the chord member form overlap joints.

Joints that have in-plane overlap involving two or more braces may be designed on the following basis:

a) For brace axial force conditions that are essentially balanced (within 10%), the individual brace intersection resistance may be calculated using the guidelines in 6.4.3. However, for the overlapping brace, the resistance should be limited to that of an Y joint with the controlling through brace properties (effective yield and geometry) assumed to represent the chord and the included angle between the braces representing θ. In instances of extreme overlap, shear parallel to the chord face is a potential failure mode and should be checked.
b) The above assumption of K joint behaviour applies only to the portion of axial force that is balanced. Any residual portion of an individual brace force that is not balanced should be treated as Y or X forces (see 6.4.2).
c) If all or some of the axial forces in the relevant braces have the same sign, the combined force representing the portion of force that has the same sign should be used to check the through brace intersection resistance.
d) For in-plane bending moments that are essentially balanced (equal and opposite), the resistance can be taken as the simple joint resistance for an individual brace (Equation (6.53)). If the brace moments have the same orientation, the combined moment should be used to check the through brace intersection resistance. As for axial forces, the overlapping brace should also be checked on the basis of the chord having through brace properties.
e) Out-of-plane moments generally have the same orientation. The combined moment should be used to check the through brace intersection resistance. The moment in the overlapping brace should also be checked on the basis of the chord having through brace properties.
f) The relevant joint axial or moment resistance shall be limited to the individual brace axial and moment resistance, respectively (see 6.4.3.2).
g) The Q_f and brace action interaction expressions associated with simple joints may be used for overlapping joints. The simple joint guidelines covering mixed classification may be followed in assessing utilisation values.

Joints with out-of-plane overlap may be assessed on the same general basis as in-plane overlapping joints, except that axial force resistance should normally revert to that for Y joints.

6.4.5 Ringstiffened joints

Design resistance of ringstiffened joints shall be determined by use of recognised engineering methods. Such methods can be elastic analyses, plastic analyses or linear or non-linear finite element analyses. See Commentary.

6.4.6 Cast joints

Cast joints are defined as joints formed using a casting process. They can be of any geometry and of variable wall thickness.

The design of a cast joint requires calibrated FE analyses. An acceptable design approach for strength is to limit the stresses found by linear elastic analyses to the design strength $\frac{f_y}{\gamma_M}$ using appropriate yield criteria. Elastic peak stresses may be reduced following similar design principles as described in Commentary to 6.4.5.

6.5 Strength of conical transitions

6.5.1 General

The provisions given in this section are for the design of concentric cone frusta between tubular sections. They may also be applied to conical transitions at brace ends, where the junction provisions apply only to the brace-end transition away from the joint.

6.5.2 Design stresses

6.5.2.1 Equivalent design axial stress in the cone section.

The equivalent design axial stress (meridional stress) at any section within the conical transition can be determined by the following equation:

$$\sigma_{equ,Sd} = \frac{\sigma_{ac,Sd} + \sigma_{mc,Sd}}{\cos\alpha} \tag{6.58}$$

$$\sigma_{ac,Sd} = \frac{N_{Sd}}{\pi(D_s - t_c \cos\alpha)t_c} \tag{6.59}$$

$$\sigma_{mc,Sd} = \frac{M_{Sd}}{\frac{\pi}{4}(D_s - t_c \cos\alpha)^2 t_c} \tag{6.60}$$

where

$\sigma_{equ,Sd}$	=	equivalent design axial stress within the conical transition
$\sigma_{ac,Sd}$	=	design axial stress at the section within the cone due to global actions
$\sigma_{mc,Sd}$	=	design bending stress at the section within the cone due to global actions
D_s	=	outer cone diameter at the section under consideration
t_c	=	cone thickness
α	=	the slope angle of the cone (see Figure 6-7)

N_{Sd} and M_{Sd} are the design axial force and design bending moment at the section under consideration. $\sigma_{ac,Sd}$ and $\sigma_{mc,Sd}$ should be calculated at the junctions of the cone sides.

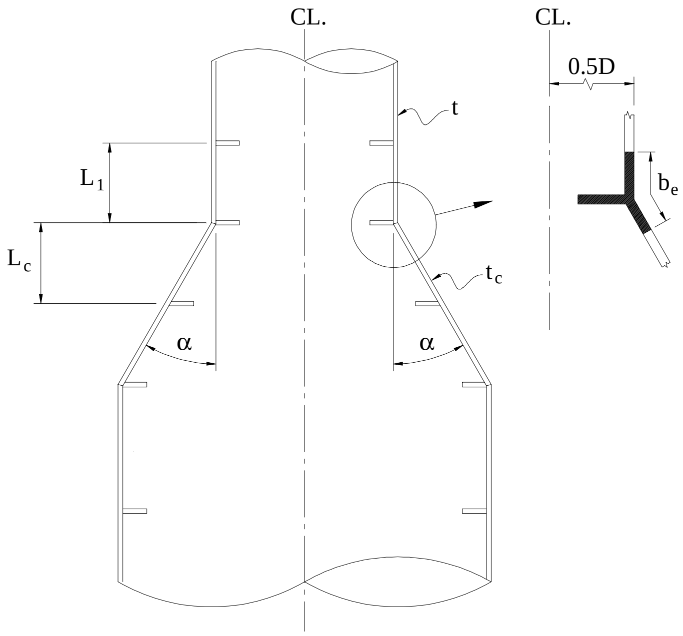


Figure 6-7 Cone geometry

6.5.2.2 Local bending stress at unstiffened junctions

In lieu of a detailed analysis, the local bending stress at each side of unstiffened tubular-cone junctions can be estimated by the following equations:

$$\sigma_{mlt,Sd} = \frac{0.6t\sqrt{D_j(t+t_c)}}{t^2}(\sigma_{at,Sd} + \sigma_{mt,Sd})\tan\alpha \tag{6.61}$$

$$\sigma_{mlc,Sd} = \frac{0.6t\sqrt{D_j(t+t_c)}}{t_c^2}(\sigma_{at,Sd} + \sigma_{mt,Sd})\tan\alpha \tag{6.62}$$

where

$\sigma_{mlt,Sd}$ = local design bending stress at the tubular side of unstiffened tubular-cone junction
$\sigma_{mlc,Sd}$ = local design bending stress at the cone side of unstiffened tubular-cone junction
D_j = cylinder diameter at junction
t = tubular member wall thickness
$\sigma_{at,Sd}$ = design axial stress in tubular section at junction due to global actions
$\sigma_{mt,Sd}$ = design bending stress in tubular section at junction due to global actions

6.5.2.3 Hoop stress at unstiffened junctions

The design hoop stress at unstiffened tubular-cone junctions due to unbalanced radial line forces may be estimated from:

$$\sigma_{hc,Sd} = 0.45\sqrt{\frac{D_j}{t}}(\sigma_{at,Sd} + \sigma_{mt,Sd})\tan\alpha \quad \textbf{(6.63)}$$

At the smaller-diameter junction, the hoop stress is tensile (or compressive) when ($\sigma_{at,Sd}$ + $\sigma_{mt,Sd}$) is tensile (or compressive). Similarly, the hoop stress at the larger-diameter junction is tensile (or compressive) when ($\sigma_{at,Sd}$ + $\sigma_{mt,Sd}$) is compressive (or tensile).

6.5.3 Strength requirements without external hydrostatic pressure

6.5.3.1 Local buckling under axial compression

For local buckling under combined axial compression and bending, the following equation should be satisfied at all sections within the conical transition.

$$\sigma_{equ,Sd} \le \frac{f_{clc}}{\gamma_M} \quad \textbf{(6.64)}$$

where

f_{clc} = local buckling strength of conical transition

For conical transitions with slope angle $\alpha < 30°$, f_{clc} can be determined using (6.6) to (6.8) with an equivalent diameter, D_e, at the section under consideration.

$$D_e = \frac{D_s}{\cos\alpha} \quad \textbf{(6.65)}$$

For conical transitions of constant wall thickness, it would be conservative to use the diameter at the larger end of the cone as D_s in Eq. (6.65).

6.5.3.2 Junction yielding

Yielding at a junction of a cone should be checked on both tubular and cone sides. This section only applies when the hoop stress, $\sigma_{hc,Sd}$, is tensile.

For net axial tension, that is, when $\sigma_{tot,Sd}$ is tensile,

$$\sqrt{\sigma_{tot,Sd}^2 + \sigma_{hc,Sd}^2 - \sigma_{hc,Sd}\sigma_{tot,Sd}} \le \frac{f_y}{\gamma_M} \quad \textbf{(6.66)}$$

For net axial compression, that is, when $\sigma_{tot,Sd}$ is compression,

$$\sqrt{\sigma_{tot,Sd}^2 + \sigma_{hc,Sd}^2 + \sigma_{hc,Sd}\,|\sigma_{tot,Sd}|} \le \frac{f_y}{\gamma_M} \quad \textbf{(6.67)}$$

where

$\sigma_{tot,Sd}$ = $\sigma_{at,Sd} + \sigma_{mt,Sd} + \sigma_{mlt,Sd}$ for checking stresses on the tubular side of the junction

= $\dfrac{\sigma_{ac,Sd} + \sigma_{mc,Sd}}{\cos\alpha} + \sigma_{mlc,Sd}$ for checking stresses on the cone side of the junction

f_y = corresponding tubular or cone yield strength.

6.5.3.3 Junction buckling.

This section only applies when the hoop stress, $\sigma_{hc,Sd}$, is compressive. In the equations, $\sigma_{hc,Sd}$ denotes the positive absolute value of the hoop compression.

For net axial tension, that is, when $\sigma_{tot,Sd}$ is tensile,

$$a^2 + b^{2\eta} + 2\nu\, a\, b \le 1.0 \qquad \textbf{(6.68)}$$

where

$$a = \frac{\sigma_{tot,Sd}}{\dfrac{f_y}{\gamma_M}} \qquad \textbf{(6.69)}$$

$$b = \frac{\sigma_{hc,Sd}}{\dfrac{f_h}{\gamma_M}} \qquad \textbf{(6.70)}$$

and ν is Poisson’s ratio of 0.3 and η is defined in Equation (6.38)

For net axial compression, that is, when $\sigma_{tot,Sd}$ is compressive,

$$\sigma_{tot,Sd} \le \frac{f_{cj}}{\gamma_M} \qquad \textbf{(6.71)}$$

and

$$\sigma_{hc,Sd} \le \frac{f_h}{\gamma_M} \qquad \textbf{(6.72)}$$

where

f_{cj} = corresponding tubular or cone characteristic axial compressive strength

f_h can be determined using equations (6.17) to (6.19) with $f_{he} = 0.80E(t/D_j)^2$ and corresponding f_y.

6.5.4 Strength requirements with external hydrostatic pressure

6.5.4.1 Hoop buckling

Unstiffened conical transitions, or cone segments between stiffening rings with slope angle α < 30°, may be designed for hoop collapse by consideration of an equivalent tubular using the equations in

Sect. 6.3.6.1. The effective diameter is D/cosα, where D is the diameter at the larger end of the cone segment. The equivalent design axial stress should be used to represent the axial stress in the design. The length of the cone should be the slant height of the cone or the distance between the adjacent rings for ring stiffened cone transition.

6.5.4.2 Junction yielding and buckling

The net design hoop stress at a tubular-cone junction is given by the algebraic sum of $\sigma_{hc,Sd}$ and $\sigma_{h,Sd}$, that is

$$\sigma_{hj,Sd} = \sigma_{hc,Sd} + \sigma_{h,Sd} \tag{6.73}$$

where

$\sigma_{h,Sd}$ = design hoop stress due to the external hydrostatic pressure, see eq. (6.16)

When $\sigma_{hj,Sd}$ is tensile, the equations in Sect. 6.5.3.2 should be satisfied by using $\sigma_{hj,Sd}$ instead of $\sigma_{hc,Sd}$. When $\sigma_{hj,Sd}$ is compressive, the equations in Sect. 6.5.3.3 should be satisfied by using $\sigma_{hj,Sd}$ instead of $\sigma_{hc,Sd}$.

6.5.5 Ring design

6.5.5.1 General

A tubular-cone junction that does not satisfy the above criteria may be strengthened either by increasing the tubular and cone thicknesses at the junction, or by providing a stiffening ring at the junction.

6.5.5.2 Junction rings without external hydrostatic pressure

If stiffening rings are required, the section properties should be chosen to satisfy both the following requirements:

$$A_c = \frac{tD_j}{f_y}(\sigma_{a,Sd} + \sigma_{m,Sd})\tan\alpha \tag{6.74}$$

$$I_c = \frac{tD_j D_c^2}{8E}(\sigma_{a,Sd} + \sigma_{m,Sd})\tan\alpha \tag{6.75}$$

where

$\sigma_{a,Sd}$ = larger of $\sigma_{at,Sd}$ and $\sigma_{ac,Sd}$.
$\sigma_{m,Sd}$ = larger of $\sigma_{mt,Sd}$ and $\sigma_{mc,Sd}$.
D_c = diameter to centroid of composite ring section. See Note 4 in section 6.3.6.2
A_c = cross-sectional area of composite ring section
I_c = moment of inertia of composite ring section

In computing A_c and I_c, the effective width of shell wall acting as a flange for the composite ring section may be computed from:

$$b_e = 0.55(\sqrt{D_j t} + \sqrt{D_j t_c}) \qquad \textbf{(6.76)}$$

Notes:

1. *For internal rings, D_j should be used instead of D_c in Equation (6.75).*
2. *For external rings , D_j in equations (6.73) and (6.74) should be taken to the centroid of the composite ring*

6.5.5.3 Junction rings with external hydrostatic pressure

Circumferential stiffening rings required at the tubular-cone junctions should be designed such that the moment of inertia of the composite ring section is equal to or greater than the sum of Equations (6.75) and (6.78):

$$I_{cT} \geq I_c + I_{ch} \qquad \textbf{(6.77)}$$

where

$$I_{ch} = \frac{D_j^2}{16E}\left\{tL_1 f_{he} + \frac{t_c L_c f_{hec}}{\cos^2\alpha}\right\} \qquad \textbf{(6.78)}$$

where

I_{cT}	=	moment of inertia of composite ring section with external hydrostatic pressure and with effective width of flange computed from Equation (6.76)
D_j	=	diameter of tubular at junction. See Note 4 of section 6.3.6.2
L_c	=	distance to first stiffening ring in cone section along cone axis $\leq 1.13\sqrt{\frac{D_e^3}{t}}$
L_1	=	distance to first stiffening ring in tubular section $\leq 1.13\sqrt{\frac{D_j^3}{t}}$
f_{he}	=	elastic hoop buckling strength for tubular
f_{hec}	=	f_{he} for cone section treated as an equivalent tubular
D_e	=	larger of equivalent diameters at the junctions

Notes:

1. *A junction ring is not required for hydrostatic collapse if Equation (6.15) is satisfied with f_{he} computed using C_h equal to 0.44 $(\cos\alpha)\cdot(t/D_j)$ in Equation (6.20), where D_j is the tubular diameter at the junction.*
2. *For external rings, D_j in Equation (6.77) should be taken to the centroid of the composite ring, except in the calculation of L_1.*

6.5.5.4 Intermediate stiffening rings

If required, circumferential stiffening rings within cone transitions may be designed using Equation (6.21) with an equivalent diameter equal to $D_s/\cos\alpha$, where D_s is the cone diameter at the section under consideration, t is the cone thickness, L is the average distance to adjacent rings along the cone axis and f_{he} is the average of the elastic hoop buckling strength values computed for the two adjacent bays.

6.6 Design of plated structures

6.6.1 General

6.6.1.1 Failure modes

Section 6.6 addresses failure modes for unstiffened and stiffened plates, which are not covered by member design checks. The failure modes are:

- Yielding of plates in bending due to lateral load
- Buckling of slender plates (high span to thickness ratio) due to in-plane compressive stresses
- Buckling of plates due to concentrated loads (patch loads)
- Buckling of stiffened plates with biaxial in-plane membrane stress and lateral load

Material factor γ_M=1.15 for design of plates.

6.6.1.2 Definitions

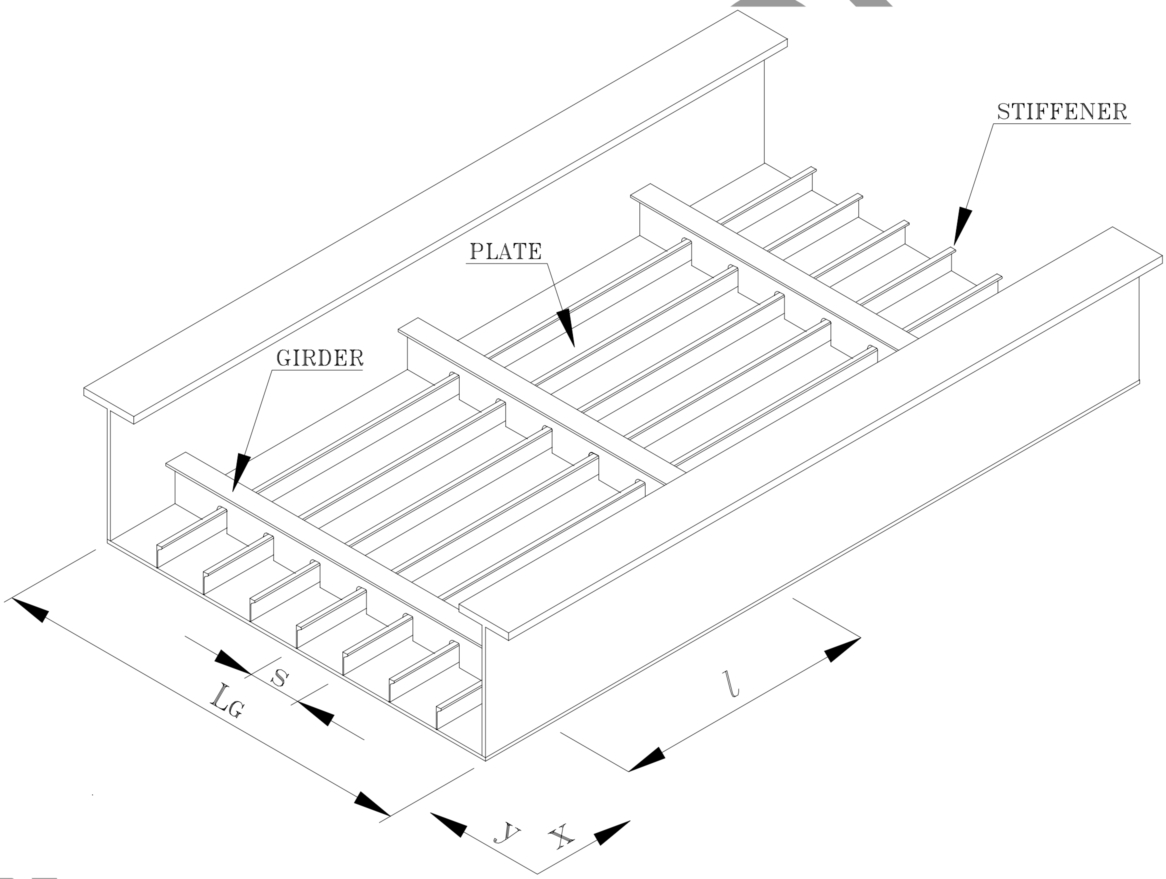


Figure 6-8 Stiffened plate panel

The terms used in this standard for checking of plates are shown in Figure 6-8. The plate panel may be web or flange of a beam, or a part of a box girder, a pontoon, a hull or an integrated plated deck.

6.6.1.3 Buckling of plates

Buckling checks of unstiffened plates in compression shall be made according to the effective width method. The reduction in plate resistance for in-plane compressive forces is expressed by a reduced (the effective) width of the plate which is multiplied with the design yield stress to obtain the resistance. (See Figure 6-9).

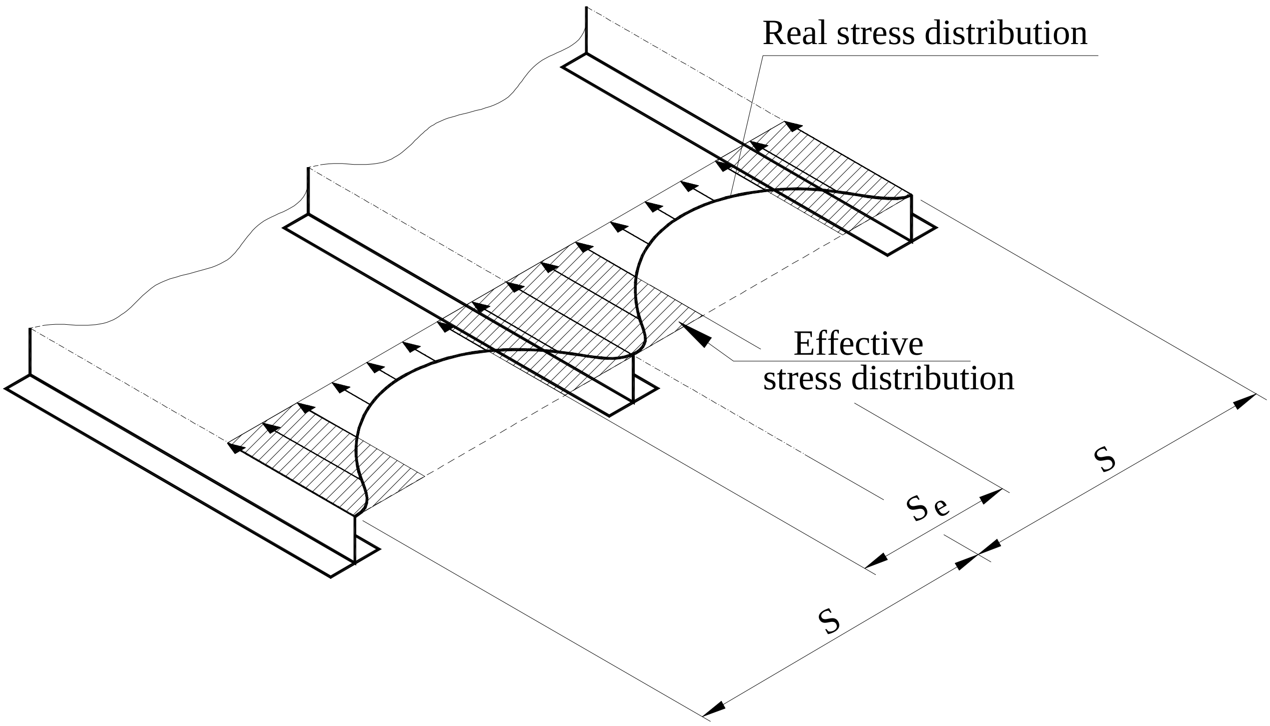
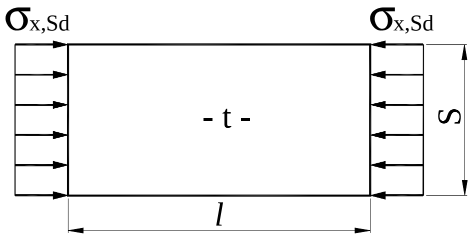
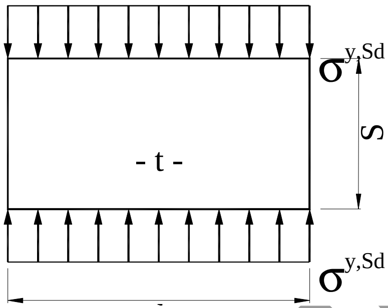
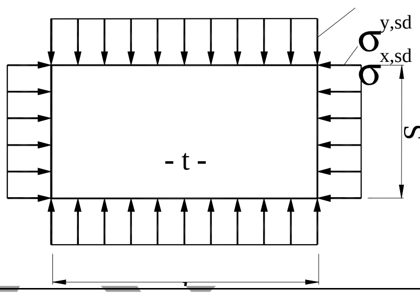
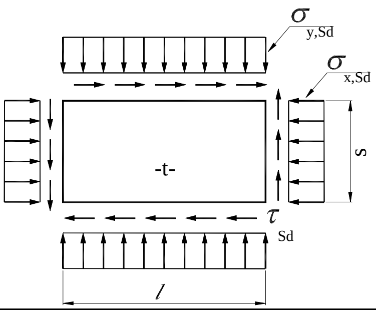
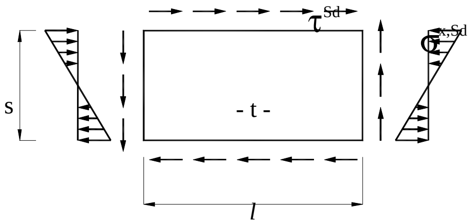


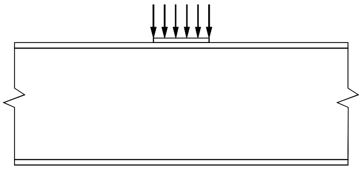
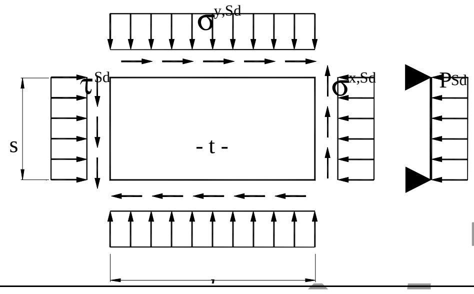
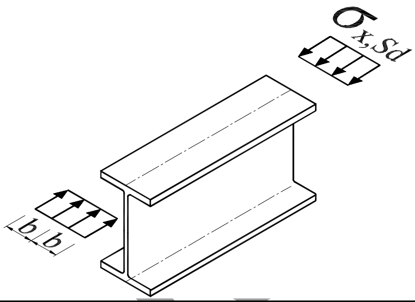
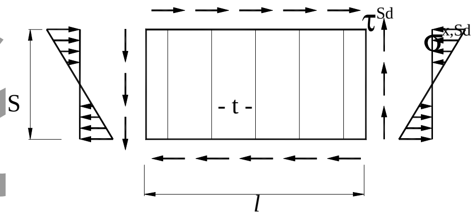
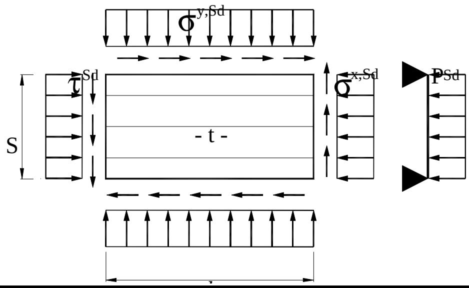
Figure 6-9 Effective width concept

Buckling of plates should be checked according to the requirements given in either Eurocode 3 or NS 3472 or the requirements given in this standard. Guidance on which standard to be used is given in Table 6-4.

Table 6-4 Reference table for buckling checks of plate panels

Description	**Load**	**Sketch**	**Code reference**	**Limiting value**
Unstiffened plate	Longitudinal compression		6.6.3.1	$s < l$ Buckling check not necessary if $\frac{s}{t} \le 42\,\varepsilon$
Unstiffened plate	Transverse compression		6.6.3.3	$s < l$ Buckling check not necessary if $\frac{s}{t} \le 5.4\varepsilon$
Unstiffened plate	Combined longitudinal and transverse compression		6.6.3.4	$s < l$ Buckling check not necessary if $\frac{s}{t} \le 5.4\varepsilon$
Unstiffened plate	Combined longitudinal and transverse compression and shear		6.6.3.4	$s < l$ Buckling check not necessary if $\frac{s}{t} \le 5.4\varepsilon$
Unstiffened plate	Pure bending and shear		NS 3472 or Eurocode 3	$s < l$ Buckling check not necessary if $\frac{s}{t} \le 69\varepsilon$

$\varepsilon = \sqrt{235/f_y}$

Description	Load	Sketch	Code reference	Limiting value
Unstiffened plate	Concentrated loads		NS 3472 or Eurocode 3	
Unstiffened plate	Uniform lateral load and in-plane normal and shear stresses		6.6.2 and 6.6.3.4	$s < l$ Buckling check not necessary if $\frac{s}{t} \leq 5.4\varepsilon$
Flange outstand	Longitudinal compression		NS 3472 or Eurocode 3	Buckling check of flange outstand not necessary if $\frac{b}{t_f} \leq 15\varepsilon$
Transverse stiffened plate panel	Bending moment and shear		NS 3472 or Eurocode 3	
Longitudinal stiffened plate panel	Longitudinal and transverse compression combined with shear and lateral load		6.6.4	

$\varepsilon = \sqrt{235/f_y}$

6.6.2 Lateral loaded plates.

For plates subjected to lateral pressure, either alone or in combination with in-plane stresses, the stresses may be checked by the following formula:

$$p_{Sd} \le 4.0 \frac{f_y}{\gamma_M} \left(\frac{t}{s}\right)^2 \left[\psi_y + \left(\frac{s}{l}\right)^2 \psi_x\right] \qquad \textbf{(6.79)}$$

where

p_{Sd} = design lateral pressure

$$\psi_y = \frac{1 - \left(\frac{\sigma_{j,Sd}}{f_y}\right)^2}{\sqrt{1 - \frac{3}{4}\left(\frac{\sigma_{x,Sd}}{f_y}\right)^2 - 3\left(\frac{\tau_{Sd}}{f_y}\right)^2}} \qquad \textbf{(6.80)}$$

$$\psi_x = \frac{1 - \left(\frac{\sigma_{j,Sd}}{f_y}\right)^2}{\sqrt{1 - \frac{3}{4}\left(\frac{\sigma_{y,Sd}}{f_y}\right)^2 - 3\left(\frac{\tau_{Sd}}{f_y}\right)^2}} \qquad \textbf{(6.81)}$$

$$\sigma_{j,Sd} = \sqrt{\sigma_{x,Sd}^2 + \sigma_{y,Sd}^2 - \sigma_{x,Sd} \cdot \sigma_{y,Sd} + 3\tau_{Sd}^2}$$

This formula for the design of a plate subjected to lateral pressure is based on yield-line theory, and accounts for the reduction of the moment resistance along the yield-line due to applied in-plane stresses. The reduced resistance is calculated based on von Mises' equivalent stress. It is emphasized that the formulation is based on a yield pattern assuming yield lines along all four edges, and will give uncertain results for cases where yield-lines can not be developed along all edges. Furthermore, since the formula does not take account of second-order effects, plates subjected to compressive stresses shall also fulfill the requirements of 6.6.3.4.

6.6.3 Buckling of unstiffened plates

6.6.3.1 Buckling of unstiffened plates under longitudinally uniform compression

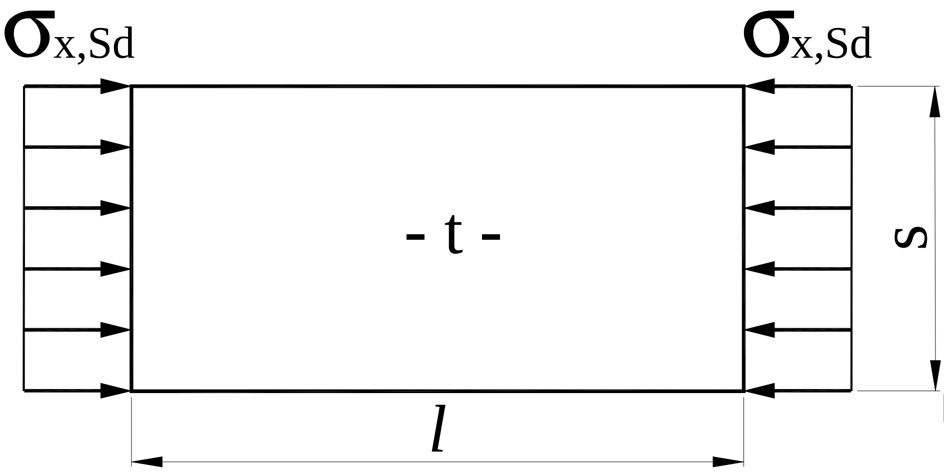


Figure 6-10 Plate with longitudinal compression

The buckling resistance of an unstiffened plate under longitudinal compression stress may be found from:

$$N_{x,Rd} = t \cdot C_x \cdot s \cdot \frac{f_y}{\gamma_M} \qquad \textbf{(6.82)}$$

where

$$C_x = 1 \qquad \text{when } \bar{\lambda}_p \leq 0.673 \qquad \textbf{(6.83)}$$

$$C_x = \frac{\left(\bar{\lambda}_p - 0.22\right)}{\bar{\lambda}_p^2} \qquad \text{when } \bar{\lambda}_p > 0.673 \qquad \textbf{(6.84)}$$

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

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{f_{cr}}} = 0.525 \frac{s}{t} \sqrt{\frac{f_y}{E}} \qquad \textbf{(6.85)}$$

In which

s	=	plate width
t	=	plate thickness
f_{cr}	=	critical plate buckling strength

The resistance of the plate is satisfactory when:

$$N_{x,Sd} = t \cdot s \cdot \sigma_{x,Sd} \leq N_{x,Rd} \qquad \textbf{(6.86)}$$

6.6.3.2 Buckling of unstiffened plates with variable longitudinal stress

The buckling resistance of an unstiffened plate with variable longitudinal stress may be found from:

$$N_{x,Rd} = t \cdot C_x \cdot s \cdot \frac{f_y}{\gamma_M} \qquad \textbf{(6.87)}$$

where

$$C_x = 1 \qquad \text{when } \bar{\lambda}_p \leq 0.673 \qquad \textbf{(6.88)}$$

$$C_x = \frac{\left(\bar{\lambda}_p - 0.22\right)}{\bar{\lambda}_p^2} \qquad \text{when } \bar{\lambda}_p > 0.673 \qquad \textbf{(6.89)}$$

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

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{f_{cr}}} = \frac{s}{t} \cdot \frac{1}{28.4\varepsilon\sqrt{k_\sigma}} \qquad \textbf{(6.90)}$$

In which

s	=	plate width	
t	=	plate thickness	
f_{cr}	=	critical plate buckling strength	
ε	=	$\sqrt{\frac{235}{f_y}}$	
k_σ	=	$\frac{8.2}{1.05+\psi}$	for $0 \leq \psi \leq 1$
		$7.81-6.29\psi+9.78\psi^2$	for $-1 \leq \psi < 0$
		$5.98(1-\psi)^2$	for $-2 \leq \psi < -1$

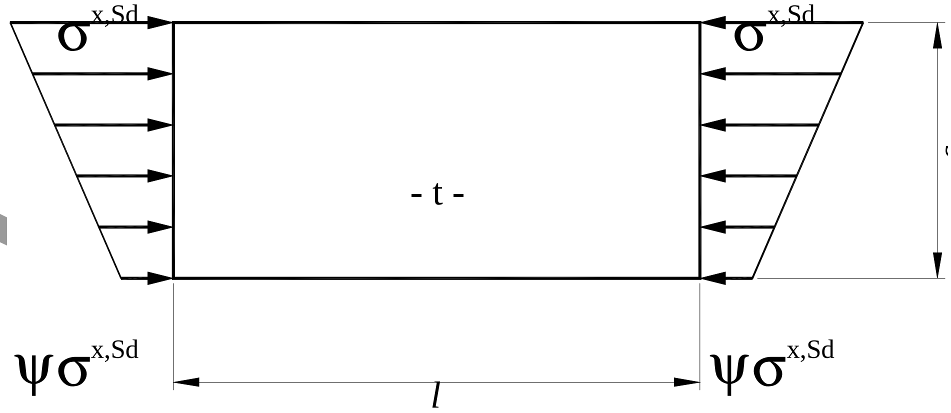


Figure 6-11 Plate with variable longitudinal stress

The resistance of the plate is satisfactory when:

$$N_{x,Sd} = t\int \sigma_{c,Sd} ds \le N_{x,Rd} \tag{6.91}$$

where

$\sigma_{c,Sd}$ = longitudinal compressive stress (tensile stresses to be set equal to zero)

6.6.3.3 Buckling of unstiffened plates with transverse compression

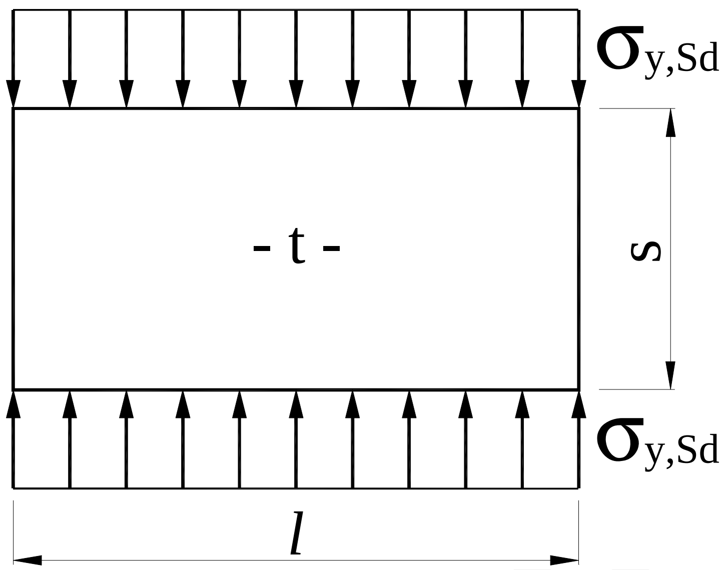


Figure 6-12 Plate with transverse compression

The buckling resistance of a plate under transverse compression force may be found from:

$$N_{y,Rd} = t \cdot C_y \cdot l \cdot \frac{f_y}{\gamma_M} \tag{6.92}$$

where

$$C_y = \frac{C_x s}{l} + 0.1\left(1 - \frac{s}{l}\right)\left(1 + \frac{0.276}{\bar{\lambda}_p^2}\right)^2, \qquad \text{but } C_y \le 1.0 \tag{6.93}$$

In which

C_x is found in (6.83) or (6.84)
$\bar{\lambda}_p$ is found in (6.85)
l = plate length
s = plate width

The resistance of the plate is satisfactory when:

$$N_{y,Sd} = \sigma_{y,Sd} \cdot t \cdot l \le N_{y,Rd} \tag{6.94}$$

In case of linear variable stress the check can be done by use of an uniformly distributed stress equal the design stress value at a distance 0.5s from the most stressed end of the plate.

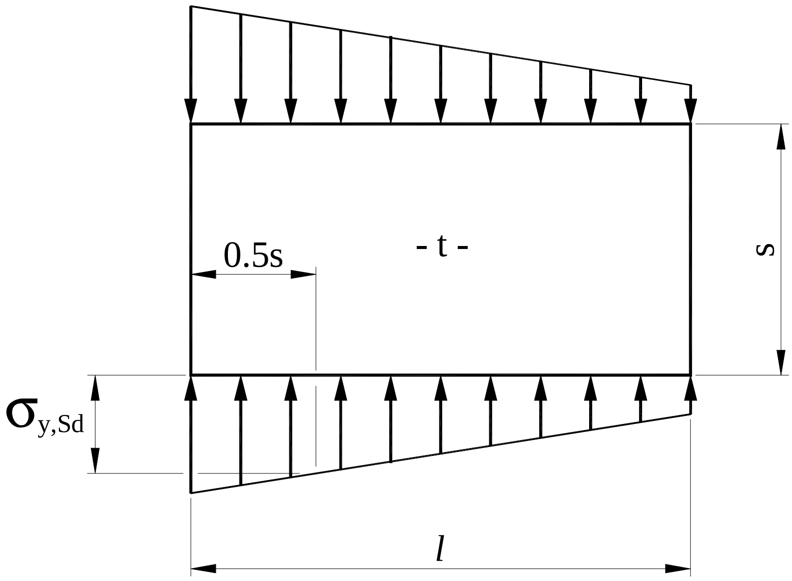


Figure 6-13 Linear variable stress in the transverse direction

6.6.3.4 Buckling of unstiffened biaxially loaded plates with shear

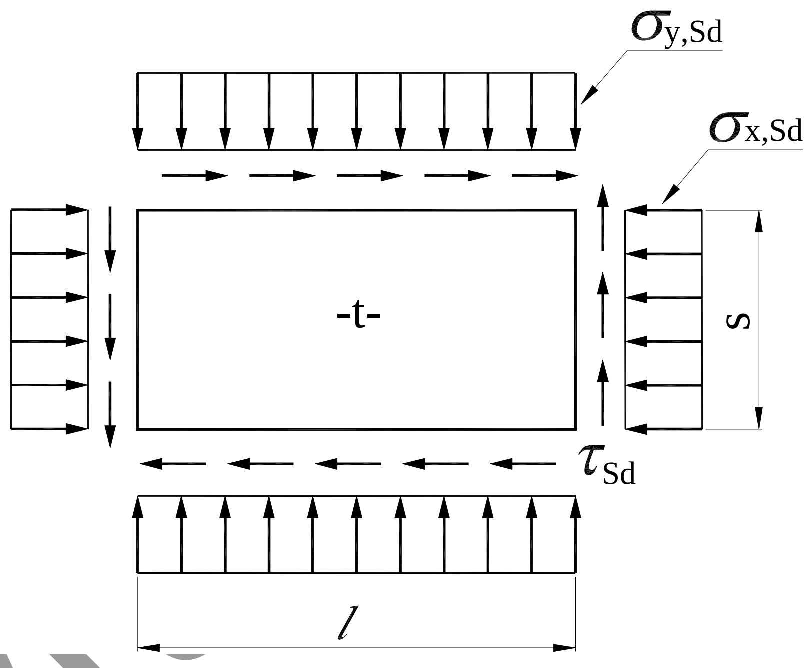


Figure 6-14 Biaxially loaded plate with shear

a. Biaxially loaded plate with shear

The buckling resistance of an unstiffened plate subjected to longitudinal compression, transverse compression or tension and shear may be found from:

$$N_{x,Rd} = t \cdot C_x \cdot C_y \cdot C_\tau \cdot s \cdot \frac{f_y}{\gamma_M} \qquad \textbf{(6.95)}$$

where

C_x is found from (6.83) or (6.84)

If the plate is subjected to transverse compression, C_y may be found from:

$$C_y = \sqrt{1 - \left(\frac{\sigma_{y,Sd}}{f_{ky}}\right)^2} \tag{6.96}$$

where

$$f_{ky} = f_y\left(\frac{s}{l}C_x + 0.1\left(1 - \frac{s}{l}\right)\left(1 + \frac{0.276}{\bar{\lambda}_p^2}\right)^2\right), \quad \text{but } f_{ky} \le f_y \tag{6.97}$$

If the plate is subjected to transverse tension, may C_y be found from:

$$C_y = \frac{1}{2}\left(\sqrt{4 - 3\left(\frac{\sigma_{y,Sd}}{f_y}\right)^2} + \frac{\sigma_{y,Sd}}{f_y}\right), \qquad C_y \le 1.0 \tag{6.98}$$

Tensile stresses are defined as negative.

C_τ is found from (6.119)

The resistance of the plate is satisfactory if:

$$N_{x,Sd} = t \cdot s \cdot \sigma_{x,Sd} \le N_{x,Rd} \tag{6.99}$$

b. plates with transverse compression and shear:

The buckling resistance of an unstiffened plate under transverse compression and shear may be found from:

$$N_{y,Rd} = t \cdot C_y \cdot C_\tau \cdot l \cdot \frac{f_y}{\gamma_M} \tag{6.100}$$

where

C_y is found from (6.93)
C_τ is found from (6.119)

The resistance of the plate is satisfactory if:

$$N_{y,Sd} = \sigma_{y,Sd} \cdot t \cdot l \le N_{y,Rd} \tag{6.101}$$

6.6.4 Stiffened plates

6.6.4.1 General

This chapter deals with stiffeners in plate fields subjected to axial stress in two directions, shear stress and lateral load.

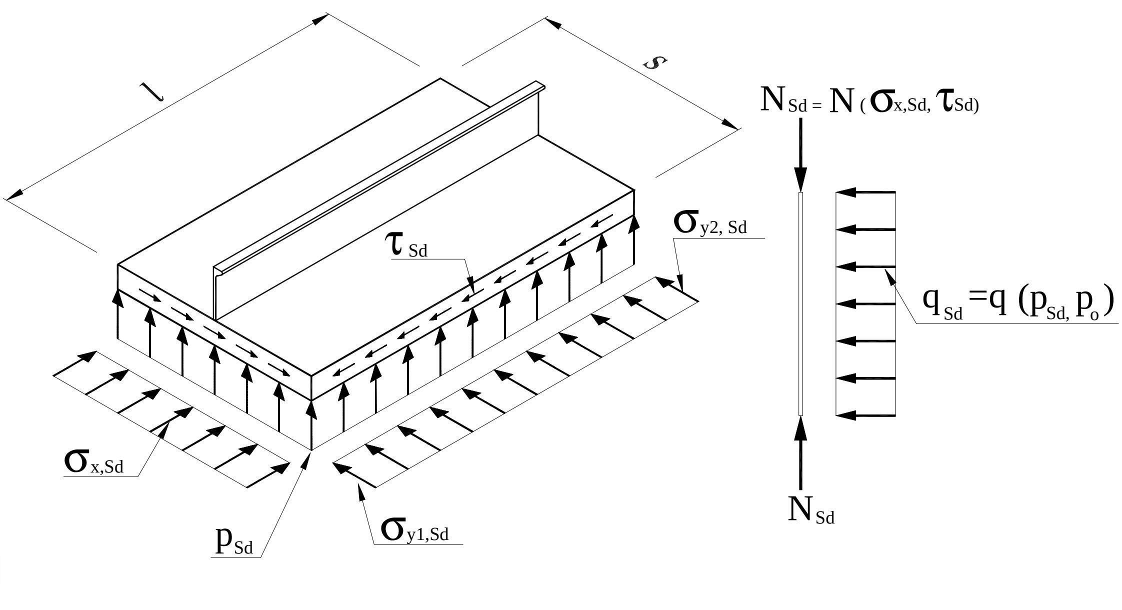
There are different formulaes for stiffeners being continuous or connected to frames with their full moment strengths and simple supported (sniped) stiffeners.

Examples of stiffened plates are shown in Figure 6-8.

The stiffener cross section need to fulfill class 3 requirement according to Eurocode 3 or NS 3472. For shear leg effects see Commentary.

The plate between stiffeners will normally be checked implicit by the check of the stiffener since plate buckling is accounted for by the effective width method. However, in case of small or zero $\sigma_{x,Sd}$ stresses it is necessary to check that $\sigma_{y,Sd}$ is less than plate resistance according to equation (6.94).

For slender stiffened plates it may be considered to neglect the load carrying resistance in the direction transverse to the stiffener and assum $\sigma_{y,Sd}$ stresses carried solely by the girder. In this case effective girder flange is determined by disregarding stiffeners and the stiffener with plate can be checked with neglecting $\sigma_{y,Sd}$ stresses. (Method 2 in 6.6.5.4).



STIFFENED PLATE → BEAM COLUMN

Figure 6-15 Strut model

6.6.4.2 Forces in the idealised stiffened plate

Stiffened plates subjected to combined forces, see Figure 6-15, are to be designed to resist a equivalent axial force defined by:

$$N_{Sd} = \sigma_{x,Sd}(A_s + st) + C\tau_{Sd}st \tag{6.102}$$

where

A_s = cross sectional area of stiffener
s = distance between stiffeners
t = plate thickness
$\sigma_{x,Sd}$ = axial stress in plate and stiffener with compressive stresses as positive

$$C = Q\left(7 - 5\left(\frac{s}{l}\right)^2\right)\left(\frac{\tau_{Sd} - \tau_{crg}}{\tau_{crl}}\right)^2 \quad \text{for } \tau_{Sd} > \tau_{crg} \tag{6.103}$$

$$C = 0 \quad \text{for } \tau_{Sd} < \tau_{crg} \tag{6.104}$$

Q = $\bar{\lambda} - 0.2$, but not less than 0 and not greater that 1.0

$\bar{\lambda} = \sqrt{\frac{f_y}{f_E}}$, where f_E is taken from (6.123)

τ_{crg} = critical shear stress for the plate with the stiffeners removed, according to (6.105)
τ_{crl} = critical shear stress for the plate panel between two stiffeners, according to (6.105)

$$\tau_{cr} = 0.6f_y, \quad \text{for } \bar{\lambda}_\tau \le 1 \tag{6.105}$$

$$= \frac{0.6}{\bar{\lambda}_\tau^2} f_y, \quad \text{for } \bar{\lambda}_\tau > 1$$

where

$$\bar{\lambda}_\tau = 0.035\frac{s}{t}\frac{1}{\sqrt{k}}\sqrt{\frac{f_y}{235}} \tag{6.106}$$

where $k = k_g$ when calculating τ_{crg} and $k = k_l$ when calculating τ_{crl} and

$$k_g = 5.34 + 4\left(\frac{l}{L_G}\right)^2, \text{for } l \ge L_G \tag{6.107}$$

$$= 5.34\left(\frac{l}{L_G}\right)^2 + 4, \text{for } l < L_G$$

$$k_l = 5.34 + 4\left(\frac{l}{s}\right)^2, \text{for } l \ge s \tag{6.108}$$

$$= 5.34\left(\frac{l}{s}\right)^2 + 4, \text{for } l < s$$

L_G = Girder length
s = stiffener spacing, see Figure 6-8

The lateral line load should be taken as:

$$q_{Sd} = (p_{Sd} + p_0)\, s \tag{6.109}$$

p_0 shall be applied in the direction of the external pressure p_{Sd}. For situations where p_{Sd} is less than p_0 the stiffener need to be checked for p_0 applied in both directions (i.e. at plate side and stiffener side).

p_{Sd} = design lateral pressure
s = stiffener spacing

$$p_0 = (0.6 + 0.4\psi) C_0 \sigma_{y1,Sd} \qquad \text{if } \psi > -1.5 \tag{6.110}$$

$$p_0 = 0 \qquad \text{if } \psi \le -1.5 \tag{6.111}$$

where

$$C_0 = \left(0.4 - \frac{0.8}{(1+n)^2}\right)\frac{f_y}{E}\left(\frac{l}{s}\right)^2 \frac{t}{h}, \quad \text{but not less than } 0.02\frac{t}{s} \tag{6.112}$$

$\psi = \dfrac{\sigma_{y2,Sd}}{\sigma_{y1,Sd}}$

$\sigma_{y1,Sd}$ = larger design stress in the transverse direction, with tensile stresses taken as negative
$\sigma_{y2,Sd}$ = smaller design transverse stress, with tensile stresses taken as negative
n = numbers of stiffeners in the plate
h = web height of the stiffener

6.6.4.3 Effective plate width

The effective plate width for a continuous stiffener subjected to longitudinal and transverse stress and shear is found from:

$$\frac{s_e}{s} = C_{xs} C_{ys} C_{\tau s} \tag{6.113}$$

The reduction factor in the longitudinal direction, C_{xs}, is found from:

$$C_{xs} = \frac{\bar{\lambda}_p - 0.22}{\bar{\lambda}_p^2}, \quad \text{if } \bar{\lambda}_p > 0.673 \tag{6.114}$$

$$= 1.0, \qquad \text{if } \bar{\lambda}_p \le 0.673$$

where

$$\bar{\lambda}_p = 0.525 \frac{s}{t} \sqrt{\frac{f_y}{E}} \tag{6.115}$$

and the reduction factor for compression stresses in the transverse direction, C_{ys}, is found from:

$$C_{ys} = \sqrt{1 - \left(\frac{\sigma_{y,Sd}}{f_{ky}} \right)^2} \tag{6.116}$$

where

$$f_{ky} = f_y \left(\frac{s}{l} C_{xs} + 0.1 \left(1 - \frac{s}{l}\right) \left(1 + \frac{0.276}{\bar{\lambda}_p^2}\right)^2 \right), \quad \text{but } f_{ky} \le f_y \tag{6.117}$$

In case of linear variable stress, $\sigma_{y,Sd}$ may be found at a distance 0.5s from the most stressed end of the plate.

The reduction factor for tension stresses in the transverse direction, C_{ys}, is found from:

$$C_{ys} = \frac{1}{2} \left(\sqrt{4 - 3 \left(\frac{\sigma_{y,Sd}}{f_y} \right)^2} + \frac{\sigma_{y,Sd}}{f_y} \right), \qquad C_{ys} \le 1.0 \tag{6.118}$$

Tensile stresses are defined as negative.

The reduction factor for shear is found from:

$$C_{\tau s} = \sqrt{1 - 3 \left(\frac{\tau_{Sd}}{f_y} \right)^2} \tag{6.119}$$

In the case of varying stiffener spacing, s, or unequal thickness in the adjoining plate panels, the effective widths, s_e, and the applied force, $N_{x,Sd}$, should be calculated individually for both side of the stiffener using the geometry of the adjacent plates. See also Figure 6-16.

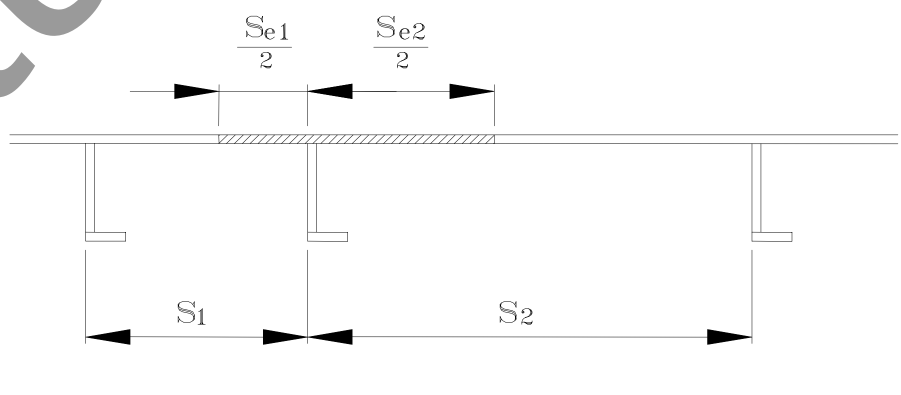


Figure 6-16 Effective widths for varying stiffener spacing

6.6.4.4 Characteristic buckling strength of stiffeners

The characteristic buckling strength for stiffeners may be found from:

$$\frac{f_k}{f_r} = 1 \qquad \text{if } \bar{\lambda} \le 0.2 \tag{6.120}$$

$$\frac{f_k}{f_r} = \frac{1+\mu+\bar{\lambda}^2 - \sqrt{\left(1+\mu+\bar{\lambda}^2\right)^2 - 4\bar{\lambda}^2}}{2\bar{\lambda}^2} \qquad \text{if } \bar{\lambda} > 0.2 \tag{6.121}$$

In which

$$\bar{\lambda} = \sqrt{\frac{f_r}{f_E}} \tag{6.122}$$

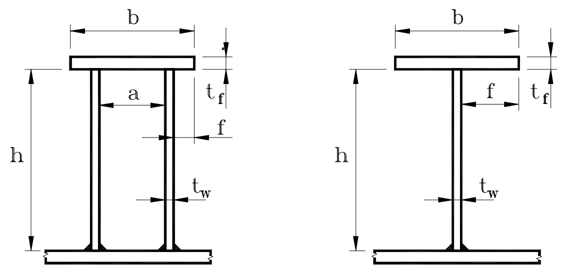
$$f_E = \pi^2 E \left(\frac{i_e}{l_k}\right)^2 \tag{6.123}$$

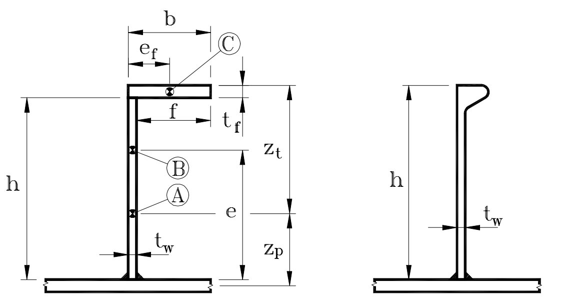
$$\mu = \left(0.34 + 0.08\frac{z_p}{i_e}\right)\left(\bar{\lambda} - 0.2\right) \qquad \text{for check at plate side} \tag{6.124}$$

$$\mu = \left(0.34 + 0.08\frac{z_t}{i_e}\right)\left(\bar{\lambda} - 0.2\right) \qquad \text{for check at stiffener side} \tag{6.125}$$

where

f_r	=	f_y	for check at plate side
f_r	=	f_y	for check at stiffener side if $\bar{\lambda}_T \le 0.6$
f_r	=	f_T	for check at stiffener side if $\bar{\lambda}_T > 0.6$, f_T may be found in section 6.6.4.5
$\bar{\lambda}_T$	=		see equation (6.129)
l_k			may be found from (6.162)
i_e	=	$\sqrt{\frac{I_e}{A_e}}$	effective radius of gyration
I_e			effective moment of inertia
A_e			effective area
z_p, z_t			is defined in Figure 6-17





A = centroid of stiffener with effective plate flange.
B = centroid of stiffener exclusive of any plate flange.
C = centroid of flange.

Figure 6-17 Definition of cross-section parameters for stiffeners or girders

6.6.4.5 Torsional buckling of stiffeners

The torsional buckling strength may be found from:

$$\frac{f_T}{f_y} = 1.0 \qquad \text{if } \bar{\lambda}_T \le 0.6 \tag{6.126}$$

$$\frac{f_T}{f_y} = \frac{1+\mu+\bar{\lambda}_T^2 - \sqrt{\left(1+\mu+\bar{\lambda}_T^2\right)^2 - 4\bar{\lambda}_T^2}}{2\bar{\lambda}_T^2} \qquad \text{if } \bar{\lambda}_T > 0.6 \tag{6.127}$$

where

$$\mu = 0.35\left(\bar{\lambda}_T - 0.6\right) \tag{6.128}$$

$$\bar{\lambda}_T = \sqrt{\frac{f_y}{f_{ET}}} \tag{6.129}$$

Generally f_{ET} may be found from:

$$f_{ET} = \beta\frac{GI_t}{I_{po}} + \pi^2\frac{Eh_s^2 I_z}{I_{po} l_T^2} \tag{6.130}$$

For T- and L-stiffeners f_{ET} may be found from:

$$f_{ET} = \beta \frac{A_W + \left(\frac{t_f}{t_W}\right)^2 A_f}{A_W + 3A_f} G\left(\frac{t_W}{h}\right)^2 + \frac{\pi^2 E I_z}{\left(\frac{A_W}{3} + A_f\right) l_T^2} \quad (6.131)$$

$$I_z = \frac{1}{12} A_f b^2 + e_f^2 \frac{A_f}{1 + \frac{A_f}{A_W}} \quad (6.132)$$

β	=	1.0, or may alternatively for stocky plates be calculated as per equation (6.133) for $s \leq l$
A_f	=	cross sectional area of flange
A_W	=	cross sectional area of web
G	=	shear modulus
I_{po}	=	polar moment of inertia = $\int r^2 dA$ where r is measured from the connection between the stiffener and the plate.
I_t	=	stiffener torsional moment of inertia (St. Venant torsion).
I_z	=	moment of inertia of the stiffeners neutral axis normal to the plane of the plate.
b	=	flange width
e_f	=	flange eccentricity, see fig Figure 6-17
h	=	web height
h_s	=	distance from stiffener toe (connection between stiffener and plate) to the shear centre of the stiffener.
l_T	=	distance between sideways supports of stiffener, distance between tripping brackets (torsional buckling length).
t	=	plate thickness
t_f	=	thickness of flange
t_W	=	thickness of web

$$\beta = \frac{3C + 0.2}{C + 0.2} \quad (6.133)$$

$$C = \frac{h}{s}\left(\frac{t}{t_w}\right)^3 \sqrt{(1-\eta)} \quad (6.134)$$

where

$$\eta = \frac{\sigma_{j,Sd}}{f_{kp}} \qquad \eta \leq 1.0 \quad (6.135)$$

$$\sigma_{j,Sd} = \sqrt{\sigma_{x,Sd}^2 + \sigma_{y,Sd}^2 - \sigma_{x,Sd}\sigma_{y,Sd} + 3\tau_{Sd}^2} \quad (6.136)$$

$$f_{kp} = \frac{f_y}{\sqrt{1 + \bar{\lambda}_e^4}} \quad (6.137)$$

$$\bar{\lambda}_e^2 = \frac{f_y}{\sigma_{j,Sd}}\left(\left(\frac{\sigma_{x,Sd}}{f_{Epx}}\right)^c + \left(\frac{\sigma_{y,Sd}}{f_{Epy}}\right)^c + \left(\frac{\tau_{Sd}}{f_{Ep\tau}}\right)^c\right)^{\frac{1}{c}} \tag{6.138}$$

$$c = 2 - \frac{s}{l} \tag{6.139}$$

$$f_{Epx} = 3.62E\left(\frac{t}{s}\right)^2 \tag{6.140}$$

$$f_{Epy} = 0.9E\left(\frac{t}{s}\right)^2 \tag{6.141}$$

$$f_{Ep\tau} = 5.0E\left(\frac{t}{s}\right)^2 \tag{6.142}$$

$\sigma_{x,Sd}$ and $\sigma_{y,Sd}$ should be set to zero if in tension

6.6.4.6 Interaction formulas for axial compression and lateral pressure

a. Continuous stiffeners

For continuous stiffeners the following four interaction equations need to be fullfilled in case of lateral pressure on plate side:

$$\frac{N_{Sd}}{N_{ks,Rd}} + \frac{M_{1,Sd} - N_{Sd} \cdot z^*}{M_{s,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \le 1 \tag{6.143}$$

$$\frac{N_{Sd}}{N_{kp,Rd}} - \frac{N_{Sd}}{N_{Rd}} + \frac{M_{1,Sd} - N_{Sd} \cdot z^*}{M_{p,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \le 1 \tag{6.144}$$

$$\frac{N_{Sd}}{N_{ks,Rd}} - \frac{N_{Sd}}{N_{Rd}} + \frac{M_{2,Sd} + N_{Sd} \cdot z^*}{M_{s,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \le 1 \tag{6.145}$$

$$\frac{N_{Sd}}{N_{kp,Rd}} + \frac{M_{2,Sd} + N_{Sd} \cdot z^*}{M_{p,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \le 1 \tag{6.146}$$

The following four interaction equations need to be fulfilled in case of lateral pressure on stiffener side:

$$\frac{N_{Sd}}{N_{ks,Rd}} - \frac{N_{Sd}}{N_{Rd}} + \frac{M_{1,Sd} + N_{Sd} \cdot z^*}{M_{s,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \leq 1 \tag{6.147}$$

$$\frac{N_{Sd}}{N_{kp,Rd}} + \frac{M_{1,Sd} + N_{Sd} \cdot z^*}{M_{p,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \leq 1 \tag{6.148}$$

$$\frac{N_{Sd}}{N_{ks,Rd}} + \frac{M_{2,Sd} - N_{Sd} \cdot z^*}{M_{s,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \leq 1 \tag{6.149}$$

$$\frac{N_{Sd}}{N_{kp,Rd}} - \frac{N_{Sd}}{N_{Rd}} + \frac{M_{2,Sd} - N_{Sd} \cdot z^*}{M_{p,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \leq 1 \tag{6.150}$$

Resistance parameters see 6.6.4.7 for stiffener and 6.6.5.3 for girders.

$M_{1,Sd}$ = $\left|\frac{q_{Sd} l^2}{12}\right|$ for continuous stiffeners with equal spans and equal lateral pressure in all spans

= absolute value of the actual largest support moment for continuous stiffeners with unequal spans and/or unequal lateral pressure in adjacent spans

$M_{2,Sd}$ = $\left|\frac{q_{Sd} l^2}{24}\right|$ for continuous stiffeners with equal spans and equal lateral pressure in all spans

= absolute value of the actual largest field moment for continuous stiffeners with unequal spans and/or unequal lateral pressure in adjacent spans

q_{Sd} is given in equation (6.109)

l = span length

z^* is the distance from the neutral axis of the effective section to the working point of the axial force. z^* may be optimised so that the resistance from equations (6.143) to (6.146) or (6.147) to (6.150) is maximised. The value of z^* is taken positive towards the plate. The simplification $z^* = 0$ is always allowed.

b. Simple supported stiffener (sniped stiffener):

Lateral pressure on plate side:

$$\frac{N_{Sd}}{N_{ks,Rd}} - \frac{N_{Sd}}{N_{Rd}} + \frac{\left|\frac{q_{Sd} l^2}{8}\right| + N_{Sd} \cdot z^*}{M_{s,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \le 1 \tag{6.151}$$

$$\frac{N_{Sd}}{N_{kp,Rd}} + \frac{\left|\frac{q_{Sd} l^2}{8}\right| + N_{Sd} \cdot z^*}{M_{p,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \le 1 \tag{6.152}$$

Lateral pressure on stiffener side:

$$\frac{N_{Sd}}{N_{ks,Rd}} + \frac{\left|\frac{q_{Sd} l^2}{8}\right| - N_{Sd} \cdot z^*}{M_{s,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \le 1 \tag{6.153}$$

$$\frac{N_{Sd}}{N_{kp,Rd}} - \frac{N_{Sd}}{N_{Rd}} + \frac{\left|\frac{q_{Sd} l^2}{8}\right| - N_{Sd} \cdot z^*}{M_{p,Rd}\left(1 - \frac{N_{Sd}}{N_E}\right)} \le 1 \tag{6.154}$$

q_{Sd} is given in equation (6.109)
l = span length

z^* is the distance from the neutral axis of the effective section to the working point of the axial force, which for a sniped stiffener will be in the centre of the plate. The value of z^* is taken positive towards the plate.

6.6.4.7 Resistance parameters for stiffeners

The following resistance parameters are used in the interaction equations for stiffeners.

$$N_{Rd} = A_e \frac{f_y}{\gamma_M} \tag{6.155}$$

A_e = $(A_s + s_e t)$, effective area of stiffener and plate

$$N_{ks,Rd} = A_e \frac{f_k}{\gamma_M} \tag{6.156}$$

where f_k is calculated from section 6.6.4.4 using (6.125).

$$N_{kp,Rd} = A_e \frac{f_k}{\gamma_M} \tag{6.157}$$

where f_k is calculated from section 6.6.4.4 using (6.124).

A_s = cross sectional area of stiffener

s_e = effective width, see 6.6.4.3

$$M_{s,Rd} = W_{es} \frac{f_r}{\gamma_M} \tag{6.158}$$

where f_r is found from section 6.6.4.4.

$$M_{p,Rd} = W_{ep} \frac{f_y}{\gamma_M} \tag{6.159}$$

W_{ep} = $\frac{I_e}{z_p}$, effective elastic section modulus on plate side, see Figure 6-17

W_{es} = $\frac{I_e}{z_t}$, effective elastic section modulus on stiffener side, see Figure 6-17

$$N_E = \frac{\pi^2 E A_e}{\left(\frac{l_k}{i_e}\right)^2} \tag{6.160}$$

where

$$i_e = \sqrt{\frac{I_e}{A_e}} \tag{6.161}$$

For a continuous stiffener the buckling length may be calculated from the following equation:

$$l_k = l\left(1 - 0.5\left|\frac{p_{Sd}}{p_f}\right|\right) \tag{6.162}$$

where p_{Sd} is design lateral pressure and p_f is the lateral pressure giving yield in extreme fibre at support.

$$p_f = \frac{12W}{l^2} \frac{f_y}{\gamma_M} \frac{1}{s} \tag{6.163}$$

W = the smaller of W_{ep} and W_{es}
l = span length

In case of varying lateral pressure, p_{Sd} in equation (6.162) is to be taken as the minimum of the value in the adjoining spans.

For simple supported stiffener $l_k = l$

6.6.4.8 Resistance of stiffeners with predominantly bending

If the stiffener is within section class 1 and the axial force N_{Sd} is less than the smaller of 0.1 $N_{ks,Rd}$ and 0.1 $N_{kp,Rd}$ the resistance may be found from:

$$\frac{M_{Sd}}{M_{pl,Rd}} \leq 1 \qquad \textbf{(6.164)}$$

where

$M_{pl,Rd}$ = design plastic moment resistance

6.6.5 Buckling of girders

6.6.5.1 General

Checking of girders is similar to the check for stiffeners of stiffened plates in equation (6.143) to (6.150) or (6.151) to (6.154) for continuous or sniped girders, respectively. Forces shall be calculated according to section 6.6.5.2 and cross section properties according to 6.6.5.4. Girder resistances should be found from 6.6.5.3. Torsional buckling of girders may be assessed according to 6.6.5.5.

6.6.5.2 Girder forces

The axial force should be taken as:

$$N_{y,Sd} = \sigma_{y,Sd}(lt + A_G) \qquad \textbf{(6.165)}$$

The lateral lineload should be taken as:

$$q_{Sd} = (p_{Sd} + p_0)l \qquad \textbf{(6.166)}$$

where

p_{Sd} = design lateral pressure
p_0 = equivalent lateral pressure
A_G = cross sectional area of girder

The calculation of the additional equivalent lateral pressure due to longitudinal compression stresses and shear shall be calculated as follows:

- For compression in the x-direction:

$$p_0 = \frac{0.4\left(t + \frac{A_s}{s}\right)}{h\left(1 - \frac{s}{L_G}\right)} \frac{f_y}{E}\left(\frac{L_G}{l}\right)^2 \left(\sigma_{x,Sd} + C\tau_{Sd}\right), \qquad \textbf{(6.167)}$$

But not less than $0.02 \frac{t + \frac{A_s}{s}}{l}\left(\sigma_{x,Sd} + C\tau_{Sd}\right)$

where

C		is found from equation (6.103) or (6.104) with
τ_{crg}	=	critical shear stress of panel with girders removed, calculated from eq. (6.105) with $\bar{\lambda}_\tau$ taken as below
τ_{crl}	=	critical shear stress of panel between girders calculated from eq. (6.105) with $\bar{\lambda}_\tau$ taken as below
$\bar{\lambda}_\tau$	=	$\sqrt{\frac{0.6 f_y}{\tau_{ce}}}$
with		
τ_{cel}	=	$\frac{18 \cdot E}{t l^2}\left(\frac{t \cdot I_s}{s}\right)^{0.75}$
τ_{ceg}	=	$\frac{\tau_{cel}}{4(n_G + 1)^2}$
n_G		the largest number of stiffeners of equally loaded girders to each side of the girder under consideration
h	=	web height of girder
A_s	=	cross sectional area of stiffener
L_G	=	girder span
s	=	stiffener spacing

For linear variation of $\sigma_{x,Sd}$, the maximum value within $0.25L_G$ to each side of the midpoint of the span may be used.

τ_{Sd} should correspond to the average shear flow over the panel.

- For tension in the x-direction:

$$p_0 = \frac{0.4\left(t + \frac{A_s}{s}\right)}{h\left(1 - \frac{s}{L_G}\right)} \frac{f_y}{E}\left(\frac{L_G}{l}\right)^2 C\tau_{Sd} \qquad \textbf{(6.168)}$$

6.6.5.3 Resistance parameters for girders

The resistance of girders may be determined by the interaction formulas in section 6.6.4.6 using the below resistances.

$$N_{Rd} = (A_G + l_e t)\frac{f_y}{\gamma_M} \quad \textbf{(6.169)}$$

$$N_{ks,Rd} = (A_G + l_e t)\frac{f_k}{\gamma_M} \quad \textbf{(6.170)}$$

Where f_k is calculated from section 6.6.4.4 using μ according to (6.125).

$$N_{kp,Rd} = (A_G + l_e t)\frac{f_k}{\gamma_M} \quad \textbf{(6.171)}$$

Where f_k is calculated from section 6.6.4.4 using μ according to (6.124).

where:

f_r = f_y for check at plate side
f_r = f_y for check at girder flange side if $\bar{\lambda}_{TG} \le 0.6$
f_r = f_{TG} for check at girder flange side if $\bar{\lambda}_{TG} > 0.6$

$$f_E = \pi^2 E \left(\frac{i_e}{L_{Gk}}\right)^2 \quad \textbf{(6.172)}$$

L_{Gk} = buckling length of girder equal L_G unless further evaluations are made
f_{TG} may be found in equation (6.187) and $\bar{\lambda}_{TG}$ may be found in equation (6.188).
A_G = cross sectional area of girder
l_e = effective width of girder plate, see 6.6.5.4

$$M_{s,Rd} = W_{eG}\frac{f_r}{\gamma_M} \quad \textbf{(6.173)}$$

where f_r is found from above for check at girder flange side.

$$M_{p,Rd} = W_{ep}\frac{f_y}{\gamma_M} \quad \textbf{(6.174)}$$

W_{ep} = $\frac{I_e}{z_p}$, effective elastic section modulus on plate side, see Figure 6-17

W_{eG} = $\frac{I_e}{z_t}$, effective elastic section modulus on girder flange side, see Figure 6-17

$$N_E = \frac{\pi^2 E A_e}{\left(\frac{L_{Gk}}{i_e}\right)^2} \quad \textbf{(6.175)}$$

where

$$i_e = \sqrt{\frac{I_e}{A_e}} \qquad \textbf{(6.176)}$$

6.6.5.4 Effective widths of girder plates

The effective width for the plate of the girder is taken equal to:

$$\frac{l_e}{l} = C_{xG} \cdot C_{yG} \cdot C_{\tau G} \qquad \textbf{(6.177)}$$

For the determination of the effective width the designer is given two options denoted method 1 and method 2. These methods are described in the following:

Method 1. Calculation of the girder by assuming the stiffened plate is effective against transverse compression (σ_y) stresses.

In this method the effective width may be found from:

$$C_{xG} = \sqrt{1 - \left(\frac{\sigma_{x,Sd}}{f_{kx}}\right)^2} \qquad \textbf{(6.178)}$$

where:

$$f_{kx} = C_{xs} f_y \qquad \textbf{(6.179)}$$

C_{xs} is found from (6.114).

If the σ_y stress in the girder is in tension due to the combined girder axial force and bending moment over the total span of the girder, C_{yG} may be found from:

$$C_{yG} = \frac{L_G}{l \cdot \sqrt{4 - \left(\frac{L_G}{l}\right)^2}}, \qquad C_{yG} \leq 1 \qquad \textbf{(6.180)}$$

If the σ_y stress in the plate is partly or complete in compression C_{yG} may be found from:

$$C_{yG} = \frac{s}{l} C_{xs} + 0.1\left(1 - \frac{s}{l}\right)\left(1 + \frac{0.276}{\bar{\lambda}_p^2}\right)^2, \qquad C_{yG} \leq 1 \qquad \textbf{(6.181)}$$

$$C_{\tau G} = \sqrt{1 - 3\left(\frac{\tau_{Sd}}{f_y}\right)^2} \qquad \textbf{(6.182)}$$

$\bar{\lambda}_p$ may be found from equation (6.115).

Method 2. Calculation of the girder by assuming the stiffened plate is not effective against transverse compression σ_y stresses.

In this case the plate and stiffener can be checked with σ_y stresses equal to zero.

In method 2 the effective width for the girder has to be calculated as if the stiffener was removed that means:

$$C_{xG} = \sqrt{1 - \left(\frac{\sigma_{x,Sd}}{f_y}\right)^2} \tag{6.183}$$

Where $\sigma_{x,Sd}$ is based on total plate and stiffener area in x-direction.

$$\begin{aligned} C_{yG} &= \frac{\bar{\lambda}_G - 0.22}{\bar{\lambda}_G^2}, \qquad \text{if } \bar{\lambda}_G > 0.673 \\ &= 1.0, \qquad\qquad\; \text{if } \bar{\lambda}_G \le 0.673 \end{aligned} \tag{6.184}$$

where

$$\bar{\lambda}_G = 0.525 \frac{l}{t} \sqrt{\frac{f_y}{E}} \tag{6.185}$$

$$C_{\tau G} = \sqrt{1 - 3\left(\frac{\tau_{Sd}}{f_y}\right)^2} \tag{6.186}$$

6.6.5.5 Torsional buckling of girders

The torsional buckling strength for girders may be calculated according to section 6.6.4.5 taking:

$$f_{TG} = \frac{\pi^2 E I_z}{\left(A_f + \frac{A_w}{3}\right) L_{GT}^2} \tag{6.187}$$

$$\bar{\lambda}_{TG} = \sqrt{\frac{f_y}{f_{TG}}} \tag{6.188}$$

where

L_{GT} = distance between lateral supports
A_f, A_w = cross sectional area of flange and web of girder

I_z = moment of inertia of girder (exclusive of plate flange) about the neutral axis perpendicular to the plate

Torsional buckling need not to be considered if tripping brackets are provided so that the laterally unsupported length L_{GT}, does not exceed the value L_{GT0} defined by:

$$\frac{L_{GT0}}{b} = C\sqrt{\frac{EA_f}{f_y\left(A_f + \frac{A_w}{3}\right)}} \qquad \textbf{(6.189)}$$

where

b = flange width
C = 0.55 for symmetric flanges
1.10 for one sided flanges

Tripping brackets are to be designed for a lateral force P_{Sd}, which may be taken equal to (see Figure 6-18):

$$P_{Sd} = 0.02\sigma_{y,Sd}\left(A_f + \frac{A_w}{3}\right) \qquad \textbf{(6.190)}$$

$\sigma_{y,Sd}$ = compressive stress in the free flange

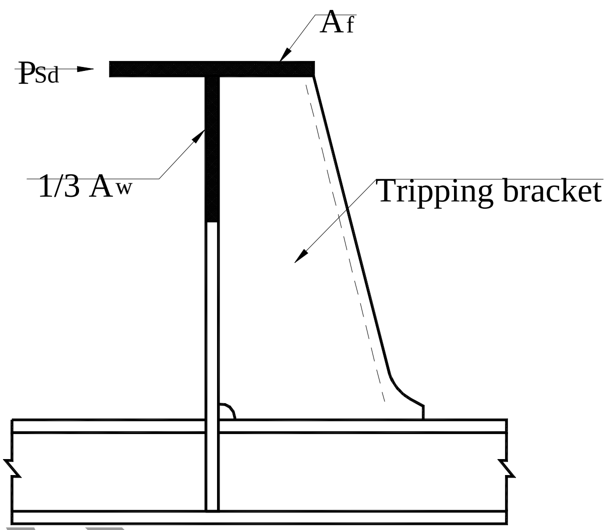


Figure 6-18 Definitions for tripping brackets

6.6.6 Local buckling of stiffeners, girders and brackets

6.6.6.1 Local buckling of stiffeners and girders

The methodology given in 6.6.4 is only valid for webs and flanges that satisfy the requirements to class 3 in Eurocode 3 or NS 3472.

6.6.6.2 Requirements to web stiffeners

In lieu of more refined analysis such as in 6.6.4, web stiffeners should satisfy the following requirements:

- Transverse web stiffeners:

$$I_s > 0.3 l_t s^2 t_W \left(2.5\frac{l_t}{s} - 2\frac{s}{l_t}\right)\frac{f_y}{E} \quad \textbf{(6.191)}$$

I_s	=	moment of inertia of web stiffener with full web plate flange s
l_t	=	length of transverse web stiffener
s	=	distance between transverse web stiffeners

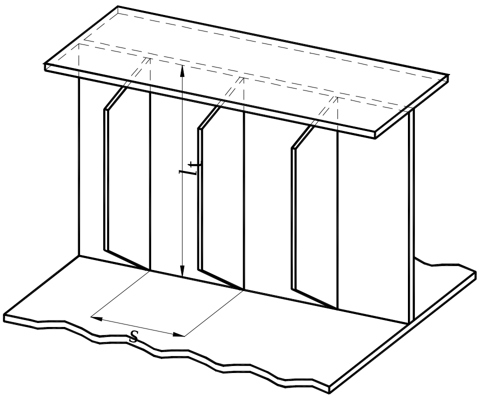


Figure 6-19 Definitions for transverse web stiffeners

- Longitudinal web stiffener:

$$I_s > 0.25 l_l^2 \left(A_s + s t_W\right)\frac{f_y}{E} \quad \textbf{(6.192)}$$

I_s	=	moment of inertia of web stiffener with full web plate flange s.
A_s	=	cross sectional area of web stiffener exclusive web plating.
l_l	=	length of longitudinal web stiffener
s	=	distance between longitudinal web stiffeners

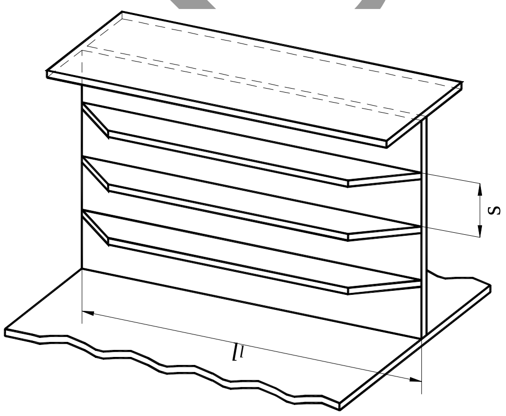


Figure 6-20 Definitions for longitudinal web stiffeners

6.6.6.3 Buckling of brackets

Brackets are to be stiffened in such a way that:

$$d_0 \leq 0.7 t_b \sqrt{\frac{E}{f_y}} \tag{6.193}$$

$$d_1 \leq 1.65 t_b \sqrt{\frac{E}{f_y}} \tag{6.194}$$

$$d_2 \leq 1.35 t_b \sqrt{\frac{E}{f_y}} \tag{6.195}$$

t_b = plate thickness of bracket.

Stiffeners as required in (6.194) to (6.195) may be designed in accordance with 6.6.4. See Figure 6-21 for definitions.

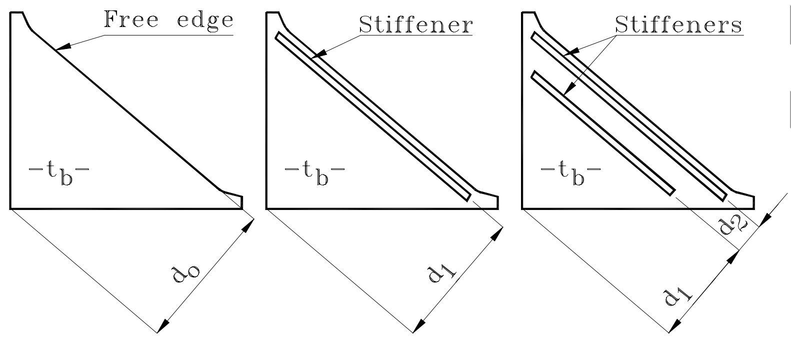


Figure 6-21 Definitions for brackets

6.7 Design of cylindrical shells

Unstiffened and ringstiffened cylindrical shells subjected to axial force, bending moment and hydrostatic pressure may be designed according to 6.3. For more refined analysis of cylindrical shells or cylindrical shells with other stiffening geometry or loading, Annex B should be used.

The susceptibility to less favourable post-critical behaviour associated with more slender geometry and more and /or larger stress components may be expressed through the reduced slenderness parameter, $\bar{\lambda}$, which is defined in Annex B.

The resulting material factor for design of shell structures is given as

$$\begin{aligned} \gamma_M &= 1.15 && \text{for } \bar{\lambda}_s < 0.5 \\ \gamma_M &= 0.85 + 0.60\bar{\lambda}_s && \text{for } 0.5 \leq \bar{\lambda}_s \leq 1.0 \\ \gamma_M &= 1.45 && \text{for } \bar{\lambda}_s > 1.0 \end{aligned} \tag{6.196}$$

$\bar{\lambda}_s$ is defined in Annex B.

6.8 Design against unstable fracture

6.8.1 General

Normally brittle fracture in offshore structures is avoided by selecting materials according to section 5 and with only acceptable defects present in the structure after fabrication.

Unstable fracture may occur under unfavourable combinations of geometry, fracture toughness, welding defects and stress levels. The risk of unstable fracture is generally greatest with large material thickness where the state of deformation is plane strain. For normal steel qualities, this typically implies a material thickness in excess of 40-50 mm, but this is dependent on the factors: geometry, fracture toughness, weld defects and stress level.

See reference /8/ for guidance on the use of fracture mechanics. If relevant fracture toughness data is lacking, material testing should be performed.

6.8.2 Determination of maximum allowable defect size

The design stress shall be determined with load coefficients given in the Regulations. The maximum applied tensile stress, accounting also for possible stress concentrations, shall be considered when calculating the tolerable defect size. Relevant residual stresses shall be included in the evaluation. Normally, a structure is designed based on the principle that plastic hinges may develop without giving rise to unstable fracture. In such case, the design nominal stress for unstable fracture shall not be less than the yield stress of the member.

The characteristic fracture toughness, K_{Ic} ,(or, alternatively K_c , J_c , J_{Ic} , $CTOD_c$), shall be determined as the lower 5% fractile of the test results. The design fracture toughness shall be calculated from the following formula:

$$K_{Icd} = \frac{K_{Ic}}{\gamma_M} \qquad \textbf{(6.197)}$$

where

γ_M = 1.15 for members where failure will be without substantial consequences
γ_M = 1.4 for members with substantial consequences

Notes:

1. *"Substantial consequences" in this context means that failure of the joint will entail:*
   Danger of loss of human life;
   Significant polution;
   Major financial consequences.
2. *"Without substantial consequences" is understood failure where it can be demonstrated that the structure satisfy the requirement to damaged condition according to the Accidental Limit States with failure in the actual joint as the defined damage.*

The design values of the J-integral and CTOD shall be determined with a safety level corresponding to that used to determine the design fracture toughness. For example, the design CTOD is found from the following formula:

$$CTOD_{cd} = \frac{CTOD_c}{\gamma_M^2} \qquad \textbf{(6.198)}$$

The maximum defect size likely to remain undetected (a_i) shall be established, based on an evaluation of the inspection method, access for inspection during fabrication, fabrication method, and the thickness and geometry of the structure,. When determining the value of a_i consideration shall be given to the capabilities of the inspection method to detect, localise, and size the defect.

The maximum allowable defect size (a_m), shall be calculated on the basis of the total stress (or corresponding strains) and the design fracture toughness. It shall be shown that $a_i < a_m$.

For a structure subjected to fatigue loading, the crack growth may be calculated by fracture mechanics. The initial defect size shall be taken as a_i . The final crack size, a_u , shall be determined with the fatigue load applied over the expected life time. It shall be verified that $a_u < a_m$.

7 SERVICEABILITY LIMIT STATES

7.1 General

General requirements for the serviceability limit states are given in NORSOK N-001.

Shear lag effects need to be considered for beams with wide flanges. Normally reduced effective width of slender plates due to buckling need not to be included.

7.2 Out of plane deflection of plates

Check of serviceability limit states for slender plates related to out of plane deflection may normally be omitted if the smallest span of the plate is less than 150 times the plate thickness. See Commentary to 6.6.1.3.

8 FATIGUE LIMIT STATES

8.1 General

In this standard, requirements are given in relation to fatigue analyses based on fatigue tests and fracture mechanics. Reference is made to Annex C for more details with respect to fatigue design.

The aim of fatigue design is to ensure that the structure has an adequate fatigue life. Calculated fatigue lives can also form the basis for efficient inspection programmes during fabrication and the operational life of the structure.

The design fatigue life for the structure components should be based on the structure service life specified by the operator. If no structure service life is specified by the operator, a service life of 15 years shall be used. A short design fatigue life will imply shorter inspection intervals.

To ensure that the structure will fulfil the intended function, a fatigue assessment, supported where appropriate by a detailed fatigue analysis should be carried out for each individual member which is subjected to fatigue loading. It should be noted that any element or member of the structure, every welded joint and attachment, or other form of stress concentration, is potentially a source of fatigue cracking and should be individually considered.

The number of load cycles shall be multiplied with the appropriate factor in Table 8-1 before the fatigue analysis is performed.

Table 8-1 Design fatigue factors

Classification of structural components based on damage consequence	Access for inspection and repair		
	No access or in the splash zone	Accessible	
		Below splash zone	Above splash zone
Substantial consequences	10	3	2
Without substantial consequences	3	2	1

"Substantial consequences" in this context means that failure of the joint will entail:

- Danger of loss of human life;
- Significant pollution;
- Major financial consequences.

"Without substantial consequences" is understood failure where it can be demonstrated that the structure satisfy the requirement to damaged condition according to the Accidental Limit States with failure in the actual joint as the defined damage.

Welds in joints below 150 m waterdepth should be assumed inaccessible for in-service inspection.

In project phases where it is possible to increase fatigue life by modification of structural details, grinding of welds should not be assumed to provide a measurable increase in the fatigue life.

8.2 Methods for fatigue analysis

The fatigue analysis should be based on S-N data, determined by fatigue testing of the considered welded detail, and the linear damage hypothesis. When appropriate, the fatigue analysis may alternatively be based on fracture mechanics. If the fatigue life estimate based on fatigue tests is short for a component where a failure may lead to substantial consequences, a more accurate investigation considering a larger portion of the structure, or a fracture mechanics analysis, should be performed. For calculations based on fracture mechanics, it should be documented that the in-service inspection accommodate a sufficient time interval between time of crack detection and the time of unstable fracture. See also chapter 6.8. Reference is made to Annex C for more details.

All significant stress ranges, which contribute to fatigue damage in the structure, should be considered. The long term distribution of stress ranges may be found by deterministic or spectral analysis. Dynamic effects shall be duly accounted for when establishing the stress history.

9 ACCIDENTAL DAMAGE LIMIT STATES

The structure shall be checked for all Accidental Damage Limit States (ALS) for the design accidental actions defined in the risk analysis.

The structure shall according to NORSOK N-001 be checked in two steps:

a) Resistance of the structure against design accidental actions

b) Post accident resistance of the structure against environmental actions. Should only be checked if the resistance is reduced by structural damage caused by the design accidental actions

- The overall objective of design against accidental actions is to achieve a system where the main safety functions are not impaired by the design accidental actions. In general the failure criteria to be considered, should also be defined in the risk analyses. (See Commentary)

The design against accidental actions may be done by direct calculation of the effects imposed by the actions on the structure, or indirectly, by design of the structure as tolerable to accidents. Examples of the latter is compartmentation of floating units which provides sufficient integrity to survive certain collision scenarios without further calculations.

The inherent uncertainty of the frequency and magnitude of the accidental loads, as well as the approximate nature of the methods for determination of accidental action effects, shall be recognised. It is therefore essential to apply sound engineering judgement and pragmatic evaluations in the design.

If non-linear, dynamic finite elements analysis is applied, it shall be verified that all behavioural effects and local failure modes (e.g. strain rate, local buckling, joint overloading, joint fracture) are accounted for implicitly by the modelling adopted, or else subjected to explicit evaluation.

Typical accidental actions are:

- Impact from ship collisions
- Impact from dropped objects
- Fire
- Explosions

The different types of accidental actions require different methods and analyses to assess the structural resistance. Design recommendations for the most common types of accidental actions are given in Annex A.

The material factor for accidental limit state $\gamma_M = 1.0$

10 REASSESSMENT OF STRUCTURES

10.1 General

An existing structure shall undergo an integrity assessment to demonstrate fitness for purpose if one or more of the following conditions exists:

- Extension of service life beyond the originally calculated design life
- Damage or deterioration to a primary structural component
- Change of use that violates the original design or previous integrity assessment basis
- Departure form the original basis of design:
  - Increased loading on the structure
  - Inadequate deck height

Assessment of existing structures should be based on the most recent information of the structure. Material properties may be revised from design values to 'as built' values. Foundation data may be updated according to findings during installation and data from adjacent structures (if applicable). Load data should be revised according to latest met-ocean data and the current layout of the structure. The water depth should be revised according to measured installation data and later scour and settlements.

The integrity of structural elements and systems should be evaluated by using a rational, defensible engineering approach. The consequence of minor events or minor structural changes may in most case be sufficiently evaluated by comparison with existing engineering documentation for the structure in question. If this fails to provide the required documentation, additional analyses will be required using either conventional design-level checks or non-linear second-order analysis methods.

10.2 Extended fatigue life

An extended life is considered to be acceptable and within normal design criteria if the calculated fatigue life is longer than the total design life times the Fatigue Design Factor.

Otherwise an extended life may be based on results from performed inspections throughout the prior service life. Such an evaluation should be based on:

- calculated crack growth
- crack growth characteristics; i.e. crack length/depth as function of time/number of cycles (this depends on type of joint, type of loading, stress range, and possibility for redistribution of stress)
- reliability of inspection method used
- elapsed time from last inspection performed.

It is recommended to use Eddy current or Magnetic Particle Inspection for inspection of surface cracks starting at hot spots.

Even if cracks are not found it might in some situations be recommended to perform a light grinding at the hot spot area to remove undercuts and increase the reliability of the inspection.

Detected cracks may be ground and inspected again to document that they are removed. The remaining life of such a repair should be assessed in each case; but provided that less than one third the thickness is removed locally by grinding a considerable fatigue life may still be documented depending on type of joint and loading conditions.

10.3 Material properties

The yield strength should be taken as the minimum guaranteed yield strength given in material certificates for the steel used in the structure, provided such certificates exist.

Alternatively, material tests may be used to establish the characteristic 'as built' yield strength. Due consideration must be given to the inherent variability in the data. The determination of a characteristic value shall be in accordance with the evaluation procedure given in NS – ENV 1993 1-1, Annex Z.

The material factor γ_M is 1.15 unless otherwise determined by separate structural reliability studies.

10.4 Corrosion allowance

The presence of local or overall corrosion should be taken into account in determining the properties of corroded members of structural components.

Strength assessment shall be based on net sections, reduced for corrosion allowance.

10.5 Foundations

Foundation data should be updated according to information gained during installation of the structure and from supplementary information from adjacent installations.

Pile resistance should primarily be estimated on the basis of static design procedures. If sufficient data are available it may be appropriate to adjust the design soil strength due to loading rate effects and cyclic load effects.

If pile-driving records are available, the design information should be updated according to additional information with respect to location and thickness of soil layers. One dimensional wave equation based methods may be used to estimate soil resistance to driving and infer a revised estimate of as-installed resistance.

The load carrying effect of mud-mats and horizontal mudline members should not be considered in the integrity assessment.

10.6 Damaged and corroded members

10.6.1 General

The ultimate strength of damaged members should be evaluated by using a rational, justifiable engineering approach. Alternatively, the strength of damaged members may be determined by refined analyses.

Corroded or damaged members and joints should be modelled to represent the actual corroded or damaged properties. Strengthened or repaired members and joints should be modelled to represent the actual strengthened or repaired properties.

10.6.2 Dented tubular members

10.6.2.1 Axial tension

Dented tubular members subjected to axial tension loads should be assessed to satisfy the following condition

$$N_{Sd} \leq N_{dent,t,Rd} = \frac{f_y \cdot A_0}{\gamma_M} \qquad \textbf{(10.1)}$$

where

- N_{Sd} = design axial force
- $N_{dent,t,Rd}$ = design axial tension capacity of the dented section
- A_0 = cross-sectional area of the undamaged section
- f_y = characteristic yield strength of steel
- γ_M = resistance factor

10.6.2.2 Axial compression

Dented tubular members subjected to axial compressive loads should be assessed to satisfy the following condition

$$N_{Sd} \leq N_{dent,c,Rd} = \frac{N_{dent,c}}{\gamma_M} \qquad \textbf{(10.2)}$$

$$N_{dent,c} = \begin{cases} (1.0 - 0.28\,\bar{\lambda}_d^2) \cdot \xi_C \cdot f_y A_0 & , \text{ for } \bar{\lambda}_d \leq 1.34 \\ \dfrac{0.9}{\bar{\lambda}_d^2} \cdot \xi_C \cdot f_y A_0 & , \text{ for } \bar{\lambda}_d > 1.34 \end{cases} \qquad \textbf{(10.3)}$$

where

- $N_{dent,c,Rd}$ = design axial compressive capacity of the dented section
- $N_{dent,c}$ = characteristic axial compressive capacity of dented member
- $\bar{\lambda}_d$ = reduced slenderness of dented member, which may be calculated as

$$= \sqrt{\frac{N_{dent,c}}{N_{E,dent}}} = \sqrt{\frac{\xi_C}{\xi_M}} \cdot \bar{\lambda}_0$$

- $\bar{\lambda}_0$ = reduced slenderness of undamaged member
- ξ_C = $\exp(-0.08\,\frac{\delta}{t})$ for $\frac{\delta}{t} < 10$ **(10.4)**
- ξ_M = $\exp(-0.06\,\frac{\delta}{t})$ for $\frac{\delta}{t} < 10$ **(10.5)**
- δ = dent depth
- t = wall thickness
- γ_M = resistance factor

10.6.2.3 Bending

Dented tubular members subjected to bending loads should be assessed to satisfy the following condition

$$M_{Sd} \le M_{dent,Rd} = \begin{cases} \xi_M \cdot M_{Rd} & \text{if the dented area acts in compression} \\ M_{Rd} & \text{otherwise} \end{cases} \quad \textbf{(10.6)}$$

where

M_{Sd} = design bending moment
$M_{dent,Rd}$ = design bending capacity of dented section
M_{Rd} = design bending capacity of undamaged sections, as given in Section **6.3**.

10.6.2.4 Combined loading

Dented tubular members under combined loading should be assessed to satisfy the following condition:

$$\frac{N_{Sd}}{N_{dent,c,Rd}} + \sqrt{\left(\frac{N_{Sd}\,\Delta y_2 + C_{m1} M_{1,Sd}}{(1-\frac{N_{Sd}}{N_{E,dent}})\,M_{dent,Rd}}\right)^{\alpha} + \left(\frac{N_{Sd}\,\Delta y_1 + C_{m2} M_{2,Sd}}{(1-\frac{N_{Sd}}{N_E})\,M_{Rd}}\right)^{2}} \le 1, \quad \text{N in compression}$$

$$\frac{N_{Sd}}{N_{dent,t,Rd}} + \sqrt{\left(\frac{M_{1,Sd}}{M_{dent,Rd}}\right)^{\alpha} + \left(\frac{M_{2,Sd}}{M_{Rd}}\right)^{2}} \le 1, \quad \text{N in tension} \quad \textbf{(10.7)}$$

where

$$\alpha = \begin{cases} 2 - 3\dfrac{\delta}{D} & \text{if the dented area acts in compression} \\ 2 & \text{otherwise} \end{cases} \quad \textbf{(10.8)}$$

N_{Sd} = design axial force on the dented section
$M_{1,Sd}$ = design bending moment about an axis parallel to the dent
$M_{2,Sd}$ = design bending moment about an axis perpendicular to the dent
$N_{E,dent}$ = Euler buckling strength of the dented section, for buckling in-line with the dent
= $\pi^2 \dfrac{E\,I_{dent}}{(k\,l)^2}$
k = effective length factor, as defined in Table 6-2
I_{dent} = moment of inertia of the dented cross-section, which may be calculated as
= $\xi_M\ I$
I = moment of inertia of undamaged section
Δy_1 = member out-of-straightness perpendicular to the dent
Δy_2 = member out-of-straightness in-line with the dent
C_{m1}, C_{m2} = moment reduction factor, as defined in Table 6-2

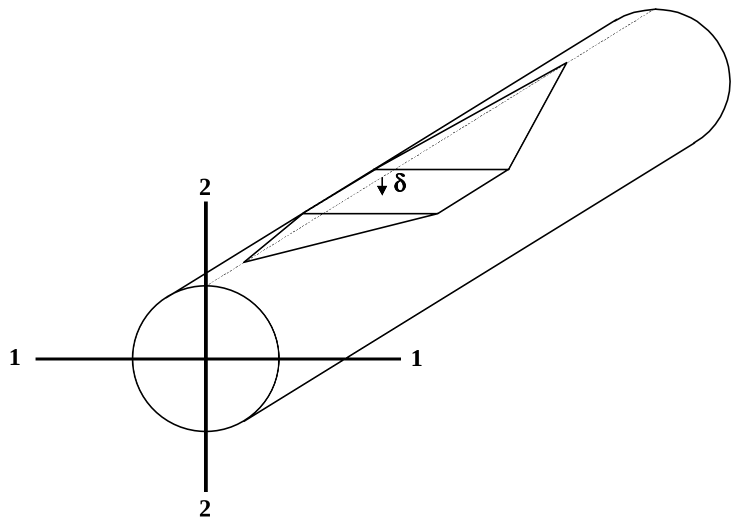


Figure 10-1 Definition of axes for dented section

10.6.3 Corroded members

In lieu of refined analyses, the strength of uniformly corroded members can be assessed by assuming a uniform thickness loss for the entire member. The reduced thickness should be consistent with the average material loss due to corrosion. The member with the reduced thickness can then be evaluated as an undamaged member.

In lieu of refined analyses, the strength of members with severe localised corrosion can be assessed by treating the corroded part of the cross-section as non-effective, and using the provisions given for dented tubulars (Section 10.6.2). An equivalent dent depth can be estimated from (10.9), and the resulting resistance calculated from (10.7).

$$\frac{\bar{\delta}}{D} = \frac{1}{2}\left(1 - \cos\pi\frac{A_{Corr}}{A}\right) \qquad \textbf{(10.9)}$$

where

$\bar{\delta}$ = equivalent dent depth
D = tube diameter
A_{Corr} = corroded part of the cross-section
A = full cross section area

For fatigue sensitive conditions, a fatigue evaluation of the corroded member should also be considered.

10.7 Cracked members and joints

10.7.1 General

Welded connections containing cracks may be assessed by fracture mechanics such as given by /8/.

10.7.2 Partially cracked tubular members

In lieu of refined analyses, partially cracked members with the cracked area loaded in compression can be treated in a similar manner to the one discussed for dented tubulars (Section 10.6.2). An equivalent dent depth can be estimated from (10.10), and the resulting resistance calculated from (10.7).

$$\frac{\bar{\delta}}{D} = \frac{1}{2}(1 - \cos\pi \frac{A_{Crack}}{A}) \qquad \textbf{(10.10)}$$

where

$\bar{\delta}$ = equivalent dent depth
D = tube diameter
A_{Crack} = crack area
A = full cross section area

Partially cracked members with the cracked area loaded in tension should be subject to a fracture mechanics assessment considering tearing mode of failure and ductile crack growth.

For fatigue sensitive conditions, a fatigue evaluation of the cracked member should also be considered.

10.7.3 Tubular joints with cracks

The static strength of a cracked tubular joint can be calculated by reducing the joint resistances for a corresponding uncracked geometry taken from Section 6.4.3, with an appropriate reduction factor accounting for the reduced ligament area /12/. The reduced strength is given by

$$N_{crack,Rd} = F_{AR} \cdot N_{Rd}$$
$$M_{crack,Rd} = F_{AR} \cdot M_{Rd} \qquad \textbf{(10.11)}$$

where

$N_{crack,Rd}$ = axial resistance of the cracked joint
$M_{crack,Rd}$ = bending resistance of cracked joint
N_{Rd} and M_{Rd} are given in Section 6.4.

$$F_{AR} = (1 - \frac{A_C}{A}) \cdot \left(\frac{1}{Q_\beta}\right)^{m_q} \qquad \textbf{(10.12)}$$

A_C = cracked area of the brace / chord intersection
A = full area of the brace /chord intersection
Q_β = tubular joint geometry factor, given in Section 6.4.3.3
m_q = 0 for part-thickness cracks

For tubular joints containing through-thickness cracks, validated correction factors are limited. Guidance on available data are listed in the Commentary.

Post-peak behaviour and ductility of cracked tubular joints should be subjected to separate assessment in each case.

10.8 Repaired and strengthened members and joints

10.8.1 General

The ultimate strength of repaired members and joints should be evaluated by using a rational, and justifiable engineering approach. Alternatively, the strength of repaired members and joints may be determined by refined analyses.

10.8.2 Grouted tubular members

The resistance of grouted tubular members, with and without dents, may be determined according to the following paragraphs.

10.8.2.1 Axial tension

The resistance of grouted tubular members under axial tension should be assessed to satisfy the following condition:

$$N_{Sd} \le N_{tg,Rd} = \frac{f_y \cdot A_S}{\gamma_M} \qquad \textbf{(10.13)}$$

where

N_{Sd} = design axial force on the grouted section
$N_{tg,Rd}$ = design axial tension resistance of the grouted, composite section
A_S = gross steel area = πDt
f_y = characteristic yield strength of steel
γ_M = material factor

10.8.2.2 Axial compression

The resistance of grouted tubular members under axial compression should be assessed to satisfy the following condition:

$$N_{Sd} \le N_{cg,Rd} = \frac{N_{cg}}{\gamma_M} \qquad \textbf{(10.14)}$$

$$N_{cg} = \begin{cases} (1.0 - 0.28\,\bar{\lambda}_g^2) \cdot N_{ug} & , \text{ for } \bar{\lambda}_g \le 1.34 \\ \dfrac{0.9}{\bar{\lambda}_g^2} \cdot N_{ug} & , \text{ for } \bar{\lambda}_g > 1.34 \end{cases} \qquad \textbf{(10.15)}$$

$$\bar{\lambda}_g = \sqrt{\frac{N_{ug}}{N_{Eg}}} \qquad \textbf{(10.16)}$$

where
N_{Sd} = design axial force on the grouted section
$N_{cg,Rd}$ = design axial compression resistance of the grouted member

N_{ug} = axial yield resistance of the composite cross-section

$$= A_S f_y + 0.67 A_G f_{cg} \qquad \textbf{(10.17)}$$

N_{Eg} = elastic Euler buckling load of the grouted member

$$= \pi^2 \frac{E_S I_S + 0.8 E_G I_G}{(k\,l)^2} \qquad \textbf{(10.18)}$$

f_{cg} = characteristic cube strength of grout
A_S = cross-sectional area of the steel

$$= \begin{cases} \pi Dt & \text{, for intact sections} \\ \pi Dt\,(1 - \dfrac{\alpha - \sin\alpha}{\pi}) & \text{, for dented sections} \end{cases} \qquad \textbf{(10.19)}$$

A_G = cross-sectional area of the grout

$$= \begin{cases} \pi \dfrac{D^2}{4} & \text{, for intact sections} \\ \pi \dfrac{D^2}{4}\,(1 - \dfrac{\alpha}{\pi} + \dfrac{1}{2}\dfrac{\sin 2\alpha}{\pi}) & \text{, for dented sections} \end{cases} \qquad \textbf{(10.20)}$$

I_S = effective moment of inertia of steel cross section

$$= \pi \frac{D^3 t}{8}\left(1 - \frac{\alpha}{\pi} - \frac{\sin 2\alpha}{2\pi} + \frac{2\sin\alpha \cdot \cos^2\alpha}{\pi}\right) - A_S e_S^2 \qquad \textbf{(10.21)}$$

I_G = effective moment of inertia of grout cross section

$$= \pi \frac{D^4}{64}\left(1 - \frac{\alpha}{\pi} + \frac{\sin 4\alpha}{4\pi}\right) - A_G e_G^2 \qquad \textbf{(10.22)}$$

E_S = modulus of elasticity of the steel
E_G = modulus of elasticity of the grout
m = modular ration of E_S/E_G
≈ 18, in lieu of actual data
e_S = distance from centroid of dented steel section to the centroid of the intact steel section

$$= \frac{D^2 t \sin\alpha\,(1 - \cos\alpha)}{2A_S} \qquad \textbf{(10.23)}$$

e_G = distance from centroid of dented grout section to the centroid of the intact grout section

$$= \frac{(D \sin\alpha i^3}{12 A_G} \qquad \textbf{(10.24)}$$

α = $\cos^{-1}(1 - 2\dfrac{\delta}{D})$
δ = dent depth
D = tube diameter

10.8.2.3 Bending

The resistance of fully grouted tubular members subject to bending loads should be assessed to satisfy the following condition:

$$M_{Sd} \leq M_{g,Rd} = \frac{W_{tr} \cdot f_{bg}}{\gamma_M} \qquad \textbf{(10.25)}$$

where

M_{Sd} = design bending moment for the grouted section
$M_{g,Rd}$ = design bending resistance of the grouted member
W_{tr} = elastic section modulus of the transformed, composite section

$$\approx \frac{2}{D}\left(I_S + \frac{I_G}{m}\right) \qquad \textbf{(10.26)}$$

m = modular ration of E_S/E_G
$\approx$ 18, in lieu of actual data
f_{bg} = characteristic bending strength of grouted member

$$= \frac{4}{\pi} f_y \cdot \xi_\delta \left(1 + \frac{\xi_m}{100}\right) \qquad \textbf{(10.27)}$$

$$\xi_\delta = 1 - 0.5\frac{\delta}{D} - 1.6\left(\frac{\delta}{D}\right)^2 \qquad \textbf{(10.28)}$$

$$\xi_m = 5.5 \cdot \xi_\delta \left(0.6 \frac{f_{cg}}{f_y} \frac{D}{t}\right)^{0.66} \qquad \textbf{(10.29)}$$

f_{cg} = characteristic cube strength of grout

10.8.2.4 Combined axial tension and bending

In lieu of refined analyses, the resistance of fully grouted tubulars in combined tension and bending may be assessed by Equation (6.26) by neglecting the effect of the grout, or by Equation (10.25) if the maximum stress due to tension is small compared to the that of the bending component.

10.8.2.5 Combined axial compression and bending

The resistance of fully grouted tubular members under combined axial compression and bending should be assessed to satisfy the following condition:

$$\frac{N_{Sd}}{N_{cg,Rd}} + T_1 \frac{M_{Sd}}{M_{g,Rd}} + T_2\left(\frac{M_{Sd}}{M_{g,Rd}}\right)^2 \leq 1 \quad , \text{for } \frac{N_{Sd}}{N_{cg,Rd}} \geq \frac{K_2}{K_1} \qquad \textbf{(10.30)}$$

$$\frac{M_{Sd}}{M_{g,Rd}} \leq 1 \quad , \text{for } \frac{N_{Sd}}{N_{cg,Rd}} < \frac{K_2}{K_1}$$

where

N_{Sd} = design axial force on the grouted section, compression negative

$N_{cg,Rd}$ = design axial compression resistance of the grouted member
M_{Sd} = design bending moment for the grouted section
$M_{g,Rd}$ = design bending resistance of the grouted member

$T_2 = 4\frac{K_3}{K_2}$

$T_1 = 1-\frac{K_2}{K_1}-T_2$

$$K_1 = \frac{N_{cg}}{N_{ug}} = \begin{cases} 1.0-0.28\,\bar{\lambda}_g^2 & \text{, for } \bar{\lambda}_g \le 1.34 \\ \dfrac{0.9}{\bar{\lambda}_g^2} & \text{, for } \bar{\lambda}_g > 1.34 \end{cases}$$

$$K_2 = K_{20}\frac{115-30\,(2\beta-1)\,(1.8-\gamma)-100\bar{\lambda}_g}{50\,(2.1-\beta)} \quad ,0 \le K_2 \le K_{20}$$

$$K_3 = K_{30}+\bar{\lambda}_g\frac{(0.5\beta+0.4)\,(\gamma^2-0.5)+0.15}{1+\bar{\lambda}_g^3}$$

$K_{20} = 0.9\gamma^2+0.2 \quad \le 0.75$

$K_{30} = 0.04-\gamma/15 \quad \ge 0$

$$\gamma = \frac{0.67\,A_g\,(f_{cg}+C_1 f_y\frac{t}{D})}{N_{ug}}$$

β = 1, provided no end moments apply, otherwise it is the ratio of the smaller to the larger end moment

$$C_1 = \frac{4\,\varphi\varepsilon}{\sqrt{1+\varphi+\varphi^2}}$$

$\phi = 0.02\,(25-\frac{k\,l}{D}) \quad \ge 0$

$\gamma = 0.25\,(25-\frac{k\,l}{D}) \quad \ge 0$

k = effective length factor, as defined in Table 6-2
l = length of member

10.8.3 Grouted joints

For grouted joints that are otherwise simple in configuration, the simple joint provisions defined in Section 6.4.3 may be used with the following additions:

a) The Q_U values in Table 6-3 should be replaced with values pertinent for grouted sections. See Commentary. Classification and joint can derating may be disregarded for fully grouted joints. The adopted Q_U values should not be less than those for simple joints.

b) For brace compression loading, the joint resistance will normally be limited by the squash or local buckling resistance of the brace. This brace resistance should therefore be used in the joint interaction expressions given by Equation (6.57).

c) For brace tension loading, the Q_U value can be established from consideration of the shear pullout resistance of the chord plug.

d) For brace moment loading, the Q_U value section can be established from consideration of the shear pullout resistance of the portion of the chord plug subjected to tension, taking into account the shift in the axis of rotation of the brace.

e) Tension and moment resistances shall not exceed the brace resistances.

f) For double-skin joints, the failure may also occur by chord ovalisation. The ovalisation resistance can estimated by substituting the following effective thickness into the simple joint equations:

$$t_{eff} = \sqrt{t^2 + t_p^2} \qquad \textbf{(10.31)}$$

where

t_{eff} = effective thickness of chord and internal pipe
t = wall thickness of chord
t_p = wall thickness of internal member

t_{eff} should be used in place of t in the simple joint equations, including the γ term. The Q_f calculation, however, should be based on t. It is presumed that calculation of Q_f has already accounted for load sharing between the chord and the inner member, such that further consideration of the effect of grout on this term is unnecessary.

10.9 Plates and cylindrical shells with dents and permanent deflections

10.9.1 Plates with permanent deflections

The residual strength of moderately distorted plates loaded by longitudinal compression can be assessed by an effective width approach /9/ where the effective width is given by

$$\frac{s_{ered}}{s} = C_{xs}\left(1 - \frac{0.39}{\bar{\lambda}_p}\frac{\delta}{t}\right), \quad \frac{\delta}{\bar{\lambda}_p t} < 0.525 \qquad \textbf{(10.32)}$$

in which

C_{xs} is found from (6.83) or (6.84)
$\bar{\lambda}_p$ is found from (6.85)
s = plate width between stiffeners
s_{ered} = effective width of plate with permanent deflections
t = plate thickness
δ = maximum permanent distortion

The resistance of the plate is given by (6.82)

s_{ered} can also be used to check stiffened plates loaded by longitudinal compression by substituting C_{xs} with $\frac{s_{ered}}{s}$ in (6.113).

10.9.2 Longitudinally stiffened cylindrical shells with dents

The compressive strength of longitudinally stiffened cylindrical shells with dents may be assessed by an effective section approach, assuming the dented area ineffective /10/. The stress becomes critical when the total stress in the damaged zone becomes equal to the stress for an undamaged cylinder

$$\frac{\sigma_{c,Sd}}{f_{cl,Rd}} \cdot \left\{ 1 + \frac{A_R\ e}{W_R} \right\} \frac{A}{A_R} \quad \leq\ 1 \qquad \textbf{(10.33)}$$

where

$\sigma_{c,Sd}$ = design axial stress in the damaged cylinder

$f_{cl,Rd}$ = design local buckling resistance of undamaged cylinder

A_R = net area of circular part of the cylinder (total minus dented area)

= $A \cdot (1 - \frac{\alpha}{\pi})$

A = gross area of the undamaged cylinder

α = $\cos^{-1}(1 - 2\frac{\delta}{D})$

e = distance from neutral axis of the cylindrical part of the dented cylinder to the neutral axis of the original, intact cylinder

= $\frac{D}{2}\frac{\sin\alpha}{\pi - \alpha}$

W_R = elastic section modulus of the circular part of the dented cylinder

= $W \cdot \dfrac{\pi - \alpha - \frac{1}{2}\sin 2\alpha - 2\dfrac{\sin^2\alpha}{\pi - \alpha}}{\pi\,(1 - 2\frac{\delta}{D} + 2\frac{|e|}{D})}$

W = elastic section modulus of the intact cylinder

11 REFERENCES

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12 COMMENTARY

This Commentary provides additional guidance and background to selected clauses of the standard. The Commentary is an informative part of this standard.

Comm. 1 Scope

In general the provisions of this standard are developed, tested and calibrated for steel with traditional stress/strain relations. Design of structures made from steel material with higher yield strength may require additional or different design checks due to, among other the following effects:

- usually a higher yield to tensile strength ratio implying less strain hardening,
- larger elastic deflection before reaching resistance limit, which is important where second order effects play a role,
- usually reduced weld overmatch leading to increased risk of failure in weld material, and
- reduced maximum elongation.

Comm. 4 General Provisions

The notation R_k and S_k must be read as indicative for a resulting characteristic resistance and action effect respectively. In the general case the design resistance, R_d, will be a function of several parameters where the material parameter f_y should be divided by γ_M, and S_d will be derived as a summation of different characteristic actions multiplied with different partial coefficients.

The groups of limit states, applied in this standard, are defined according to ISO 13819 part 1. In contrast, Eurocode 3 and most national building codes are treating limit states associated with failure due to accidental loads or fatigue as ultimate limit states. Whilst fatigue and failure from accidental loads are characteristically similar to ultimate limit states, it is convenient to distinguish between them due to different load factors for ULS, FLS and ALS. FLS and ALS may therefore be regarded as subgroups of ULS.

Comm. 5.1 Design class

The check for damaged condition according to the Accidental Damage Limit States (ALS) imply that the structure with damage is checked for all characteristic actions, but with all safety factors set to unity. With regard to material selection the associated damage is brittle failure or lamellar tearing. If the structure subjected to one such failure is still capable of resisting the characteristic loads, the joint or component should be designated DC 3 or DC 4. As an example one can consider a system of 4 cantilever beams linked by a transverse beam at their free ends. A material failure at the supports in one of the beams will only reduce the resistance with 25%. (Assuming sufficient strength in the transverse beam). Since the structure without damage need to be checked with partial safety factors according to Ultimate Limit States, such a system will prove to satisfy ALS, and design class DC 3 or DC 4 dependent upon complexity will apply. A similar system of only two cantilever beams will normally not fullfill the ALS criterion to damage condition and DC 1 or DC 2 will apply.

Comm. 5.2 Steel quality level

Steel materials selected in accordance with the tabled steel quality level (SQL) and as per Material Data Sheet (MDS) in NORSOK M-120 are assured to be of the same weldability within each SQL group, irrelevant of the actual material thickness or yield strength chosen. Requirements for SQL I are more severe than those for SQL II.

This is achieved through the different optional requirements stated in each MDS for each SQL. The achievement of balanced weldability is reached mainly by two means:

- Requirement to higher energy absorption at toughness testing for yield strength above 400 MPa.
- Lowering the test temperature for qualification testing, reflecting the differences between SQL II and I and stepwise increases in material thickness.

For high strength material of 420 MPa and above, the MDS's assume the same minimum yield strength irrelevant of material thickness of plates. This is normally achieved through adjustments in chemical composition, but without jeopardising the required weldability.

Improved properties in the trough thickness direction (SQL I) should be specified for steel materials where failure due to lamellar tearing will mean significant loss of resistance. Alternatively the plate can be tested after welding to check that lamellar tearing has not taken place.

Comm. 6 Ultimate Limit States

Comm. 6.1 General

Eurocode 3 uses the following definition of Ultimate Limit States: "Ultimate limit states are those associated with collapse, or with other forms of structural failure which may endanger the safety of people. States prior to structural collapse which, for simplicity, are considered in place of the collapse itself are also classified and treated as ultimate limit states". For structures designed according to this standard structural failures which will imply significant pollution or major financial concequences should be considered in addition to human safety.

All steel structures behave more or less non-linear when loaded to their ultimate limit. The formulae for design resistance in this standard or similar codes and standards are therefore developed on the basis that permanent deformation may take place.

Traditionally, offshore structures are analysed by linear methods to determine the internal distribution of forces and moments, and the resistances of the cross-sections are checked according to design resistances found in design codes. The design formulae often require deformations well into the in-elastic range in order to mobilise the prescribed resistances. However, no further checks are considered necessary as long as the internal forces and moments are determined by linear methods.

When non-linear analysis methods are used, additional checks of ductility and repeated yielding, are to be performed. The check for repeated yielding is only necessary in case of cyclic loading e.g. wave loads.

The check for ductility requires that all sections subjected to deformation into the in-elastic range should deform without loss of the assumed load-bearing resistance. Such loss of resistance can be due to fracture, instability of cross-sectional parts or member buckling. The design codes give little guidance on this issue, with exception for stability of cross-sectional parts in yield hinges, which will normally be covered by requirement to cross-sectional class 1. See e.g. Eurocode 3.

The check against repeated yielding is often referred to as the check that the structure can achieve a stable state called shakedown. In the general case it is necessary to define a characteristic cycling load and to use this load with appropriate partial safety factors. It should be checked that yielding only take place in the first few loading cycles and that later load repetitions only cause responses in the linear range. Alternatively a low cycle fatigue check can be performed and substitute the check for shakedown.

If non-linear analyses are applied, it shall be checked that the analysis tool and the modelling adopted represent the non-linear behaviour of all structural elements that may contribute to the failure mechanism with sufficient accuracy (See Ultiguide /11/). Stiffness, resistance and post ultimate behaviour (if applicable) should be represented, including local failure modes such as local buckling, joint overload, joint fracture etc.

Use of non-linear analysis methods may result in more structural elements being governed by the requirements to the Serviceability Limit State and additional SLS requirements may be needed compared with design using linear methods.

The following simplifications are valid for design with respect to the ultimate limit states, provided that ductile failure modes can be assumed:

- built in stresses from fabrication and erection may be neglected when fabrication requirements according to NORSOK M-101 are met,
- stresses from differential temperature under normal conditions e.g. sun heating may be neglected, and
- the analytical static model need not in detail represent the real structure e.g. secondary (deflection induced) moments may be neglected for trusses.

Comm. 6.2 Ductility

Steel structures behave generally ductile when loaded to their limits. The established design practise is based on this behaviour, which is beneficial both with respect to the design and performance of the structure. For a ductile structure, significant deflections may occur before failure and thus give a collapse warning. Ductile structures also have larger energy absorption capabilities against impact loads. The possibility for the structure to redistribute stresses lessens the need for an accurate stress calculation during design as the structure may redistribute forces and moments to be in accordance with the assumed static model. This is the basis for use of linear analyses for ULS checks even for structures, which behave significantly non-linear when approaching their ultimate limit states.

Comm. 6.3.1 General

This section has been developed specifically for circular tubular shapes that are typical of offshore platform construction. The types of tubulars covered include fabricated roll-bent tubulars with a longitudinal weld seam, hot-finished seamless pipes, and ERW pipes that have undergone some types of post-weld heat treatment or normalisation to relieve residual stresses. Relief of the residual stresses is necessary to remove the "rounded" stress-strain characteristic that frequently arises from the ERW form of manufacture.

The recommendations in this section are basically in accordance with draft for ISO 13819-2, /13/.

Comm. 6.3.3 Axial compression

Tubular members subjected to axial compression are subject to failure due to either material yielding, overall column buckling, local buckling, or a combination of these failure modes.

The characteristic equation for column buckling is a function of $\bar{\lambda}$, a normalised form of column slenderness parameter given by $(f_{cl}/f_E)^{0.5}$; where f_{cl} is the local buckling strength of the cross-section and f_E is the Euler buckling strength for a perfect column.

For members with two or more different cross sections, the following steps can be used to determined the resistance:

- Determine the elastic buckling load, N_E, for the complete member, taking into account the end restraints and variable cross-section properties. In most cases, the effective length factor of the member needs to be determined.
- The design compressive resistance, $N_{cr,Rd}$, is given by

$$N_{c,Rd} = \frac{N_{cl}}{\gamma_M}\left(1 - 0.28\frac{N_{cl}}{N_E}\right) \quad \text{for } \frac{N_{cl}}{N_E} < 1.34 \tag{12.1}$$

$$N_{c,Rd} = \frac{0.9\,N_E}{\gamma_M} \quad \text{for } \frac{N_{cl}}{N_E} \geq 1.34 \tag{12.2}$$

in which

N_{cl}	=	smallest characteristic local axial compressive strength of all the cross sections
	=	f_{cl} A
f_{cl}	=	as given by Equation (6.6) or (6.7)
A	=	cross-sectional area

In design analysis, a member with variable cross sections can be modelled with several prismatic elements. For each prismatic element, added length and/or input effective length factor are used to ensure that the design compressive resistance is correctly determined.

The theoretical value of C_x for an ideal tubular is 0.6. However, a reduced value of C_x = 0.3 is recommended for use in Equation (6.8) to account for the effect of initial geometric imperfections within tolerance limits given in NORSOK M-101. A reduced value of C_x = 0.3 is also implicit in the limits for f_y/f_{cle} given in Equations (6.6) or (6.7).

Short tubular members subjected to axial compression will fail either by material yielding or local buckling, depending on the diameter-to-thickness (D/t) ratio. Tubular members with low D/t ratios are generally not subject to local buckling under axial compression and can be designed on the basis of material yielding, i.e., the local buckling stress may be considered equal to the yield strength. However, as the D/t ratio increases, the elastic local buckling strength decreases, and the tubular should be checked for local buckling.

Unstiffened thin-walled tubular subjected to axial compression and bending are prone to sudden failures at loads well below the theoretical buckling loads predicted by classical small-deflection shell theory. There is a sudden drop in load-carrying resistance upon buckling of such members. The post-buckling reserve strength of tubular members is small, in contrast to the post-buckling behaviour of flat plates in compression, which usually continue to carry substantial load after local buckling. For this reason, there is a need for more conservatism in the definition of buckling load for tubulars than for most other structural elements. The large scatter in test data also necessitates a relatively conservative design procedure. The large scatter in test data is partly caused by initial imperfections generated by fabrication. Other factors of influence are boundary conditions and built-in residual stresses (Ref. /3/ and /4/).

Some experimental evidence indicates that inelastic local buckling may be less sensitive to initial imperfections and residual stresses than elastic local buckling (Ref. /3/). Therefore, in order to achieve a robust design, it is recommended to select member geometry such that local buckling due to axial forces is avoided.

The characteristic equations are developed by screening test data and establishing the curve at 95% success at the 50% confidence level, which satisfies the following conditions:

1) it has a plateau of material characteristic yield strength over the range $0 \le f_y/f_{cle} \le 0.17$,
2) it has the general form of Equation (6.7),
3) it converges to the elastic critical buckling curve with increasing member slenderness ratio, and
4) the difference between the mean minus 1.645 standard deviations of test data and the developed equations is minimum.

The local buckling data base contains 38 acceptable tests performed by several different investigators (Ref. /3/).

A comparison between test data and the characteristic local buckling strength equation, Equations (6.6) to (6.8) was made. The developed equations have the bias of 1.065, the standard deviation of 0.073, and the coefficient of variation of 0.068.

The elastic local buckling stress formula recommended in Equation (6.8) represent one-half of the theoretical local buckling stress computed using classical small-deflection theory. This reduction accounts for the detrimental effect of geometric imperfections. Based on the test data shown in /3/, this reduction is considered to be conservative for tubulars with $t \ge 6$ mm and $D/t < 120$. Offshore platform members typically fall within these dimensional limits. For thinner tubulars and tubulars with higher D/t ratios, larger imperfection reduction factors may be required, ref. Annex B.

The local buckling database limits the applicability of the nominal strength equations to $D/t<120$ and $t \ge 6$mm. Annex B provides guidance for the design of tubular members beyond these dimensional limits.

Comm. 6.3.6.1 Hoop buckling

Unstiffened tubular members under external hydrostatic pressure are subject to elastic or inelastic local buckling of the shell wall between the restraints. Once initiated, the collapse will tend to flatten the member from one end to the other. Ring-stiffened members are subject to local buckling of the shell wall between rings. The shell buckles between the rings, while the rings remain essentially circular. However, the rings may rotate or warp out of their plane. Ring-stiffened tubular members are also subject to general instability, which occurs when the rings and shell wall buckle simultaneously at the critical load. It is desirable to provide rings with sufficient residual strength to prevent general instability. Reference is made to Annex B, buckling strength of shells.

For tubular members satisfying the maximum out-of-roundness tolerance of 1 percent, the hoop buckling strength is given by Eqs. (6.17) to (6.19). For ring-stiffened members, Eqs. (6.17) to (6.19) gives the hoop buckling strength of the shell wall between the rings. To account for the possible 1 percent out-of roundness, the elastic hoop buckling stress is taken as 0.8 of the theoretical value calculated using classical small deflection theory. That is, C_h=0.44t/D, whereas the theoretical C_h=0.55t/D. In addition, the remaining C_h values are lower bound estimates.

For members with out-of-roundness greater than 1 %, but less than 3 %, a reduced elastic hoop buckling strength, f_{he}, should be determined (Ref. /3/). The characteristic hoop buckling strength, f_h, is then determined using the reduced f_{he}.

Reduced $f_{he} = \alpha \dfrac{f_{he}}{0.8}$

in which

α = geometric imperfection factor

$= 1 - 0.2\sqrt{\dfrac{D_{max} - D_{min}}{0.01\ D_{nom}}}$

$\dfrac{D_{max} - D_{min}}{0.01\ D_{nom}}$ = out-of-roundness (%)

where D_{max} and D_{min} are the maximum and minimum of any measured diameter at a cross section and D_{nom} the nominal diameter.

Comm. 6.3.6.2 Ring stiffener design

The formula recommended for determining the moment of inertia of stiffening rings, Equation (6.21), provides sufficient strength to resist buckling of the ring and shell even after the shell has buckled between stiffeners. It is assumed that the shell offers no support after buckling and transfers all its forces to the effective stiffener section. The stiffener ring is designed as an isolated ring that buckles into two waves (n=2) at a collapse pressure 20 percent higher than the strength of the shell.

The effect of ring stiffeners on increased axial resistance for large diameter/thickness ratios is not accounted for in this section. Reference is made to Annex B for guidance on this.

Comm. 6.3.8 Tubular members subjected to combined loads without hydrostatic pressure

This section describes the background of the design requirements in Section 6.3.8, which covers unstiffened and ring-stiffened cylindrical shell instability mode interactions when subjected to combined axial and bending loads without hydrostatic pressure

In this section and Section 6.3.9, the designer should include the second order frame moment or P-Δ effect in the bending stresses, when it is significant. The P-Δ effect may be significant in the design of unbraced deck legs, piles, and laterally flexible structures.

Comm. 6.3.8.1 Axial tension and bending

This section provides a resistance check for components under combined axial tensile load and bending. The interaction equation is modified compared with the present draft to ISO /13/ as the term for axial load is rised in the power of 1.75.

Comm. 6.3.8.2 Axial compression and bending

This section provides an overall beam-column stability check, Equation (6.27), and strength check, Equation (6.28), for components under combined axial compression force and bending.

Use of ϕ-functions (Livesley Ref. /5/) implies that exact solutions are obtained for the considered buckling problem. General effective buckling lengths have been derived using the ϕ-functions incorporating end flexibility of the members. The results for an X-brace with four equal-length members are shown in Figure 12-1 to Figure 12-3 as a function of the load distribution in the system Q/P and of the end rotational stiffness. P is the maximum compression force. The non-dimensional parameter ρ is given as:

$$\rho = \frac{CL}{EI}$$

where C is the local rotational stiffness at a node (accounting for local joint stiffness and stiffness of the other members going into the joint) and I is the moment of inertia of the tubular member. Normally L refers to center-line-to-center-line distances between nodes.

The results in Figure 12-2 indicates for realistic end conditions ($\rho > 3$) for single braces, i.e., when Q/P = 1, a k-factor less than 0.8 is acceptable provided end joint flexibilities are not lost as the braces become fully loaded. Experimental results relating to frames seem to confirm that 0.7 is acceptable. For X-braces where the magnitude of the tension brace load is at least 50% of the magnitude of the compression brace load, i.e., when $Q/P < -0.5$, and the joints remain effective, k = 0.45 times the length L is supported by these results, whilst 0.4 seems justified based on experimental evidence.

Figure 12-2 and Figure 12-3 provide effective length factors for X-braces when the longer segment is equal to 0.6 times the brace length and 0.7 times the brace length, respectively.

To estimate the effective length of a unbraced column, such as superstructure legs, the use of the alignment chart in Figure 12-4 provides a fairly rapid method for determining adequate k-values.

For cantilever tubular members, an independent and rational analysis is required in the determination of appropriate effective length factors. Such analysis shall take full account of all large deflection (P-Δ) effects. For a cantilever tubular member, C_m=1.0.

The use of the moment reduction factor (C_m) in the combined interaction equations, such as Eq. (6.27), is to obtain an equivalent moment that is less conservative. The C_m values recommended in Table 6-2 are similar to those recommended in Ref. /6/.

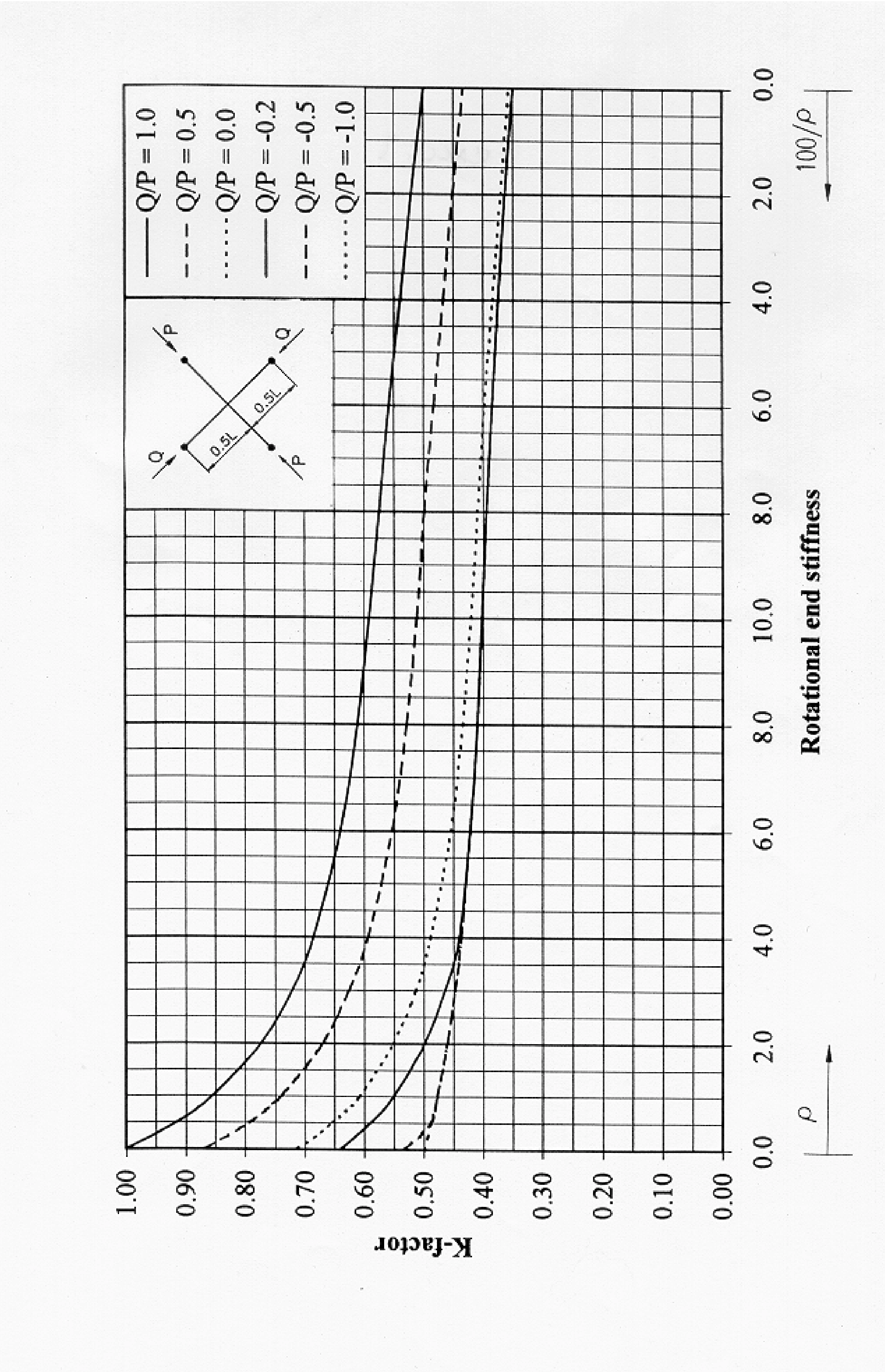

Figure 12-1 Effective length factors for a X-brace with equal brace lengths

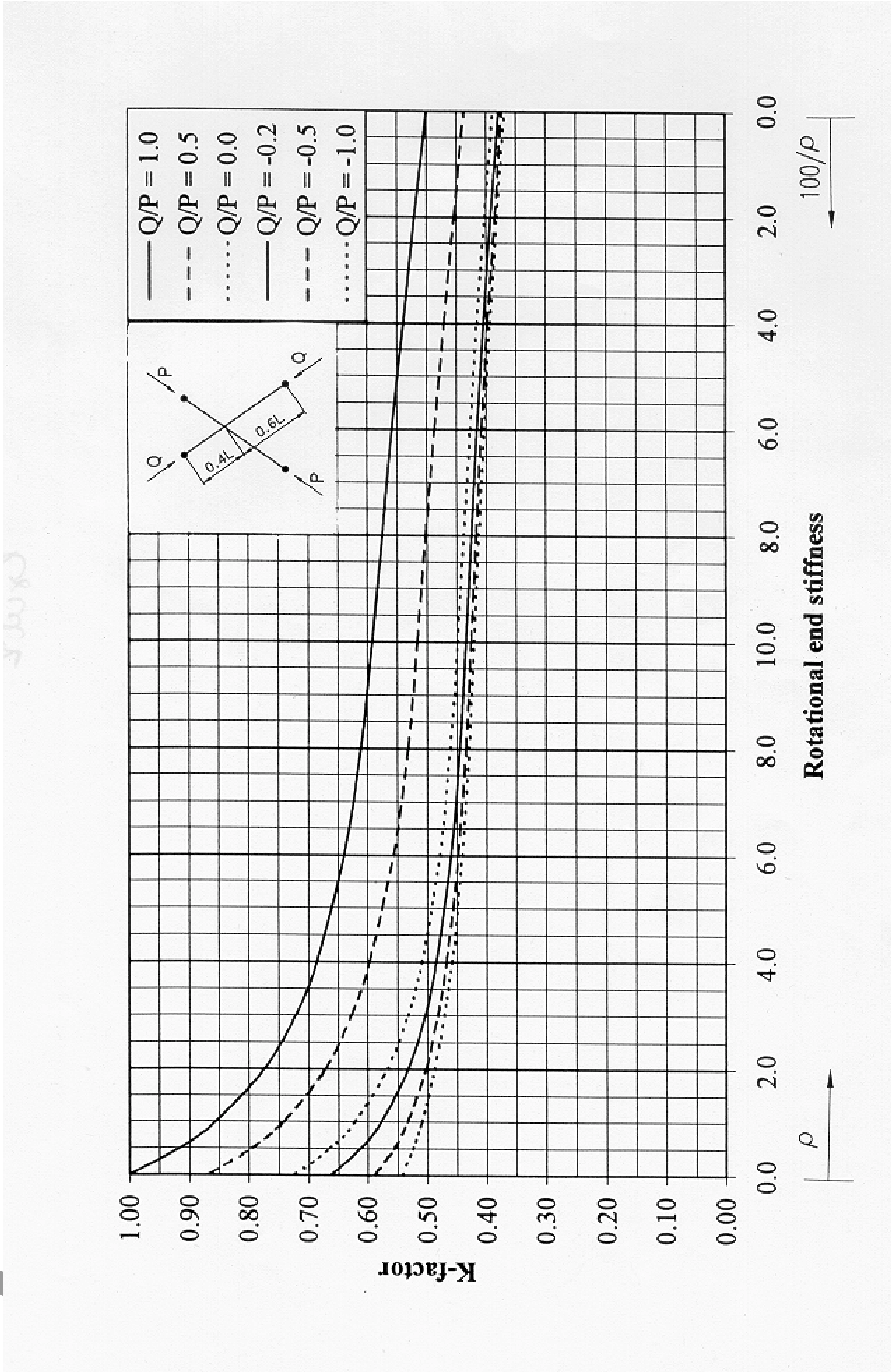

Figure 12-2 Effective length factors for a X-brace with the shorter segment equal to 0.4 times the brace length

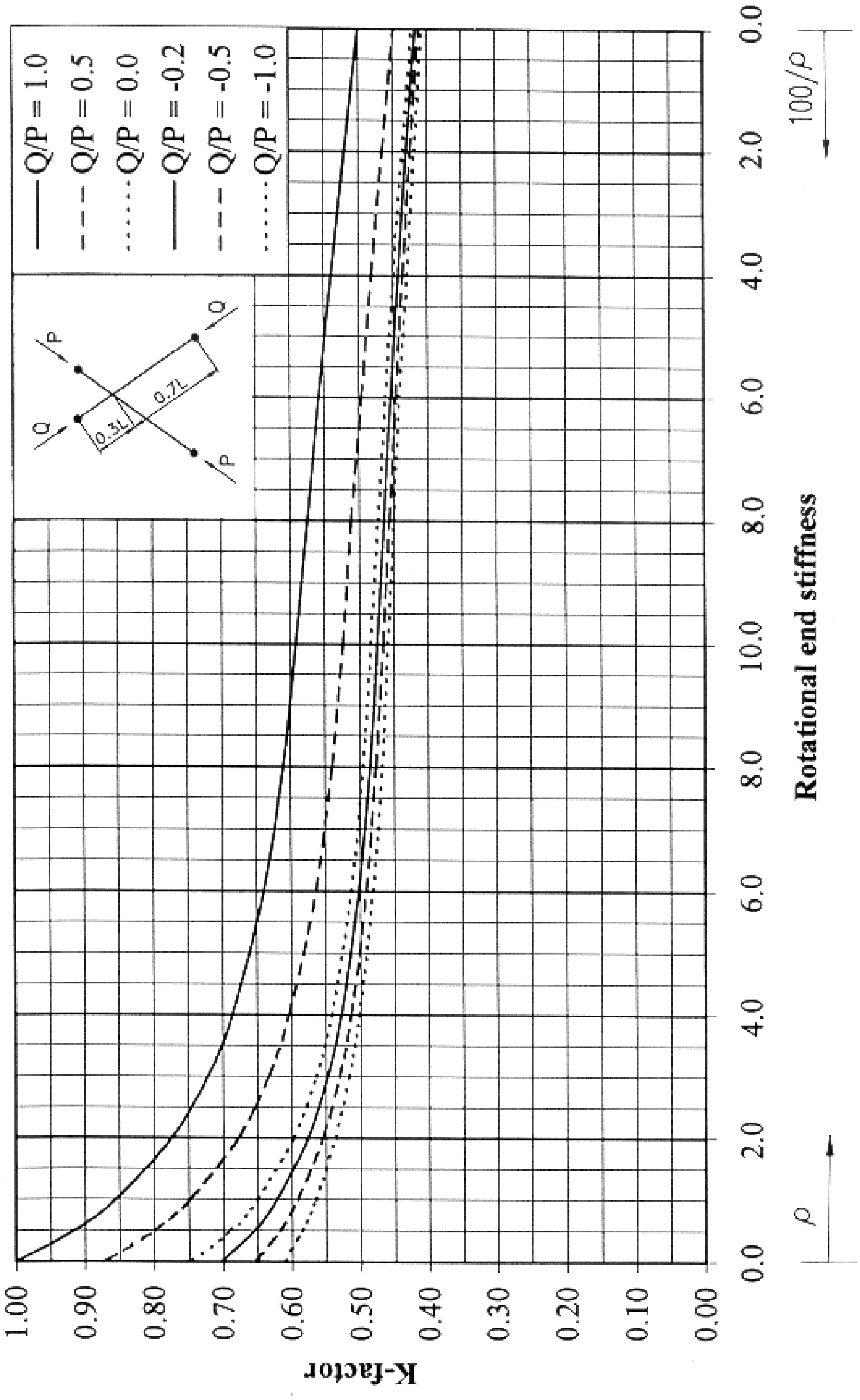

Figure 12-3 Effective length factors for a X-brace with the shorter segment equal to 0.3 times the brace length

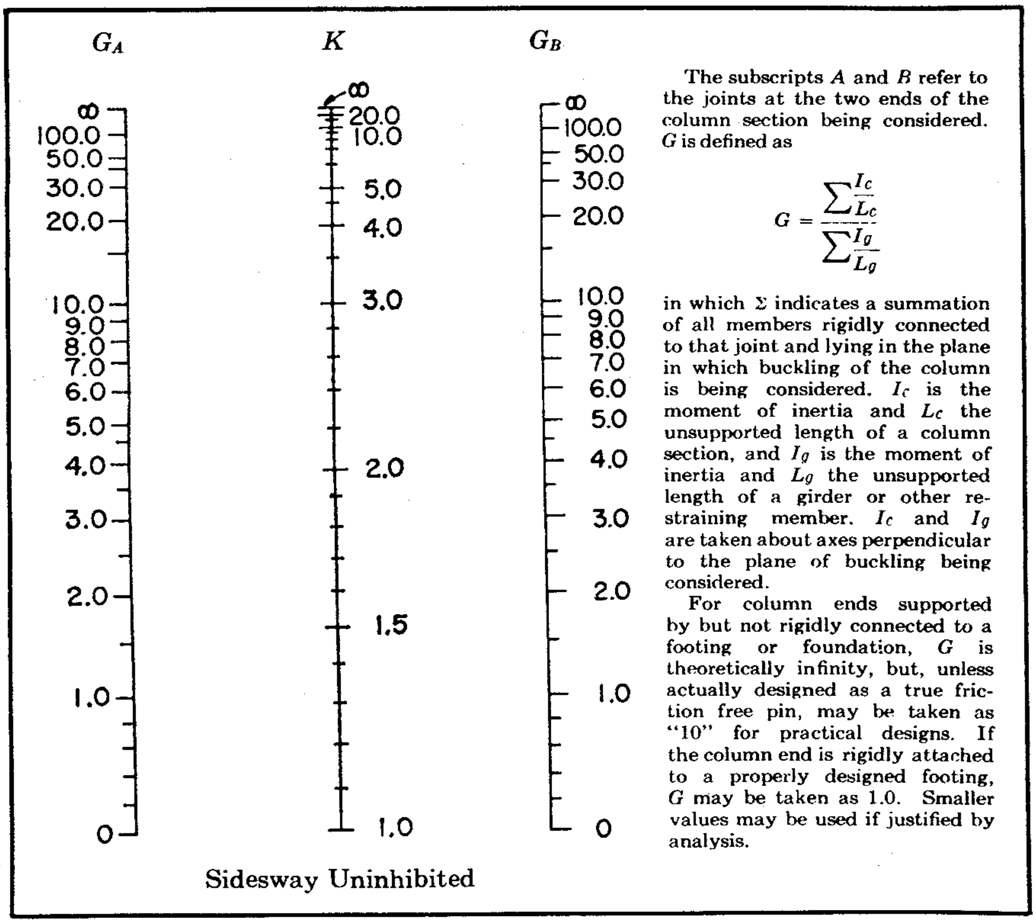

Figure 12-4 Alignment chart for effective length of columns in continuous frames

Comm. 6.3.9 Tubular members subjected to combined loads with hydrostatic pressure

This section provides strength design interaction equations for the cases in which a tubular member is subjected to axial tension or compression, and/or bending combined with external hydrostatic pressure.

Some guidance on significance of hydrostatic pressure may be found from Figure 12-5 for a given water depth and diameter/thickness ratio.

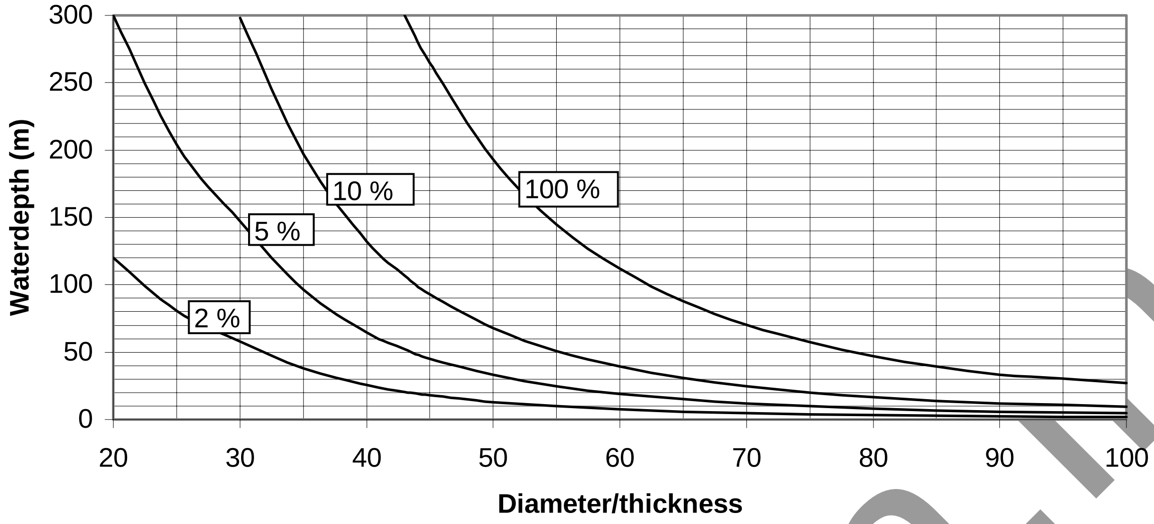

Figure 12-5 Reduction in bending resistance of members due to external pressure, ($\gamma_M \cdot \gamma_f$ = 1.5, f_y= 350 MPa)

The design equations are categorised into two design approaches, Methods A and B. The main purpose of providing two methods is to facilitate tubular member design by the two common analyses used by designers. Either design method is acceptable. In the limit when the hydrostatic pressure is zero, the design equations in this section reduce to those given in Sect. 6.3.8.

In both methods the hoop compression is not explicitly included in the analysis, but its effect on member design is considered within the design interaction equations. For both design methods, the hoop collapse design check stipulated in Sect. 6.3.6 must be satisfied first.

Method A should be used when the capped-end axial compression due to external hydrostatic pressure is not explicitly included in the analysis, but its effect is accounted for while computing the member utilisation ratio.

Method B should be used when the capped-end axial compression due to external hydrostatic pressure is included explicitly in the analysis as nodal loads. The explicit application of the capped-end axial compression in the analysis allows for a more precise redistribution of the capped-end load based on the relative stiffnesses of the braces at a node.

The two methods are not identical. However, since redistribution of the capped-end axial compression in Method B is minimal because of similar brace sizes at a node, the difference between the two methods should be small.

Comm. 6.3.9.1 Axial tension, bending, and hydrostatic pressure

Method A ($\sigma_{a,Sd}$ is in tension)

The actual member axial stress, or the net axial stress, is estimated by subtracting the full capped-end axial compression from the calculated axial tension, $\sigma_{a,Sd}$. The net axial tensile stress, ($\sigma_{a,Sd}$-$\sigma_{q,Sd}$), is then used in Eq. (6.34), which is a linear tension-bending interaction equation. There are mainly three effects due to the presence of external hydrostatic pressure: 1) a reduction of the axial tension due to the presence of a capped-end axial compression, 2) a reduction of the axial tensile strength, $f_{th,Rd}$, caused by the hoop compression, and 3) a reduction of the bending strength, $f_{mh,Rd}$, caused by the hoop compression.

As demonstrated in Ref. /3/, the axial tension-hydrostatic pressure interaction is similar to the bending-hydrostatic pressure interaction. The reduced axial tensile and bending strengths, as given by Eqs. (6.35) and (6.36), were derived from the following ultimate-strength interaction equations:

combined axial tension and hydrostatic pressure:

$$\left[\frac{\sigma_{a,Sd}}{f_d}\right]^2 + \left[\frac{\sigma_{p,Sd}}{f_{h,Rd}}\right]^{2\eta} + 2\nu\left[\frac{\sigma_{p,Sd}}{f_{h,Rd}}\right]\left[\frac{\sigma_{a,Sd}}{f_d}\right] = 1.0 \qquad f_d = \frac{f_y}{\gamma_M} \tag{12.3}$$

$$\left[\frac{\sigma_{m,Sd}}{f_{m,Rd}}\right]^2 + \left[\frac{\sigma_{p,Sd}}{f_{h,Rd}}\right]^{2\eta} + 2\nu\left[\frac{\sigma_{p,Sd}}{f_{h,Rd}}\right]\left[\frac{\sigma_{m,Sd}}{f_{m,Rd}}\right] = 1.0 \qquad f_{m,Rd} = \frac{f_m}{\gamma_M} \tag{12.4}$$

To obtain the axial tensile and bending strengths from the above two equations, the $\sigma_{a,Sd}$ and $\sigma_{m,Sd}$ terms are represented by f_{th} and f_{mh}, respectively, which are given by the positive roots of the quadratic equations.

When the calculated axial tensile stress is greater than or equal to the capped-end axial compression, that is, $\sigma_{a,Sd} \geq \sigma_{q,Sd}$, the member is subjected to net axial tension. For this case, the member yield strength, f_y, is not replaced by a local buckling axial strength.

When the calculated axial tensile stress is less than the capped-end axial compression, that is, $\sigma_{a,Sd} < \sigma_{q,Sd}$, the member is subjected to net axial compression and to a quasi-hydrostatic pressure condition. (A member is subjected to a pure hydrostatic pressure condition when the net axial compressive stress is equal to the capped-end axial stress, that is, $\sigma_{a,Sd} = 0$.) Under this condition there is no member instability. Hence for this case, in which $\sigma_{a,Sd} \leq \sigma_{q,Sd}$, the cross-sectional yield criterion (Eq. (6.39)) and the cross-sectional elastic buckling criterion (Eq. (6.41)) need to be satisfied.

Method B ($\sigma_{ac,Sd}$ is in tension)

In this method the member net axial stress is the calculated value, $\sigma_{ac,Sd}$, since the effect of the capped-end axial compression is explicitly included in the design analysis. Therefore, the calculated axial tensile stress, $\sigma_{ac,Sd}$, can be used directly in the cross-sectional strength check, as given in Eq. (6.42).

Comm. 6.3.9.2 Axial compression, bending, and hydrostatic pressure

Method A ($\sigma_{a,Sd}$ is in compression)

The capped-end axial compression due to hydrostatic pressure does not cause buckling of a member under combined external compression and hydrostatic pressure. The major contribution of the capped-end axial compression is earlier yielding of the member in the presence of residual stresses and additional external axial compression. The earlier yielding in turn results in a lower column buckling strength for the member, as given in Eq. (6.47). When there is no hydrostatic pressure, that is, $\sigma_{q,Sd} = 0$, Eq. (6.47) reduces to the in-air case, Eq. (6.3).

It is incorrect to estimate the reduced column buckling strength by subtracting the capped-end axial compression from the in-air buckling strength calculated by Eq. (6.3). This approach assumes that the capped-end axial compression can cause buckling and actually the reduced strength can be negative for cases where the capped-end axial compression is greater than the in-air buckling strength.

For the stability check (Eq. (6.43)), the calculated axial compression, $\sigma_{a,Sd}$, which is the additional external axial compression, is used. The effect of the capped-end axial compression is captured in the buckling strength, $f_{ch,Rd}$, which is derived for hydrostatic conditions. For strength or cross-sectional yield check (Eq. (6.44)), the net axial compression of the member is used. In addition, the cross-section elastic buckling criterion (Eq. (6.41)) need to be satisfied.

Method B ($\sigma_{ac,Sd}$ is in compression)

In this method the calculated axial stress, $\sigma_{ac,Sd}$, is the net axial compressive stress of the member since the capped-end axial compression is included in the design analysis. For the stability check (Eq. (6.50)), the axial compression to be used with the equation is the component that is in additional to the pure hydrostatic pressure condition. Therefore, the capped-end axial compression is subtracted from the net axial compressive stress in Eq. (6.50). For the strength check (Eq. (6.51)), the net axial compressive stress is used.

When the calculated axial compressive stress is less than the capped-end axial compression, the member is under a quasi-hydrostatic pressure condition. That is, the net axial compression is less than the capped-end axial compression due to pure hydrostatic pressure. Under this loading, the member can not buckle as a beam-column. Of course, hoop collapse is still a limit state. For this case, in which $\sigma_{ac,Sd} \leq \sigma_{q,Sd}$, only the yield criterion of Eq. (6.51) needs to be satisfied.

Comm 6.4 Tubular joints

Reasonable alternative methods to the requirements in this standard may be used for the design of joints. Test data and analytical techniques may be used as a basis for design, provided that it is demonstrated that the resistance of such joints can be reliably estimated. The recommendations presented here have been derived from a consideration of the characteristic strength of tubular joints. Characteristic strength is comparable to lower bound strength. Care should therefore be taken in using the results of limited tests programs or analytical investigations to provide an estimate of joint resistance. Consideration shall be given to the imposition of a reduction factor on the calculation of joint resistance to account for a small amount of data or a poor basis for the calculation. Analytical or numerical techniques should be calibrated and benchmarked to suitable test data.

The formulas in this section are based upon draft to ISO 13819-2, /13/.

Comm. 6.4.1 General

Detailing practice

Joint detailing is an essential element of joint design. For simple tubular joints, the recommended detailing nomenclature and dimensioning are shown in Figure 6-1. This practice indicates that if an increased wall thickness of chord or special steel, is required, it should extend past the outside edge of incoming bracing a minimum of one quarter of the chord diameter or 300 mm, whichever is greater. Short chord can lengths can lead to a downgrading of joint resistance. The designer should consider specifying an increase of such chord can length to remove the need for resistance downgrading (see 6.4.3.5). An increased wall thickness of brace or special steel, if required, should extend a minimum of one brace diameter or 600 mm, whichever is greater. Neither the cited chord can nor brace stub dimension includes the length over which thickness taper occurs.

The minimum nominal gap between adjacent braces, whether in- or out-of-plane, is normally 50 mm. Care should be taken to ensure that overlap of welds at the toes of the joint is avoided. When overlapping braces occur, the amount of overlap should preferably be at least d/4 (where d is diameter of the through brace) or 150 mm, whichever is greater. This dimension is measured along the axis of the through member.

Where overlapping of braces is necessary or preferred, the brace with the larger wall thickness should be the through brace and fully welded to the chord. Further, where substantial overlap occurs, the larger diameter brace should be specified as the through member. This brace require an end stub to ensure that the thickness is at least equal to that of the overlapping brace.

Comm. 6.4.2 Joint classification

Case (h) in Figure 6-2 is a good example of the actions and classification hierarchy that should be adopted in the classification of joints. Replacement of brace actions by a combination of tension and compression force to give the same net action is not permitted. For example, replacing the force in the horizontal brace on the left hand side of the joint by a compression force of 1000 and tension force of 500 is not permitted, as this may result in an inappropriate X classification for this horizontal brace and a K classification for the diagonal brace.

Special consideration should be given to establish the proper gap if a portion of the action is related to K-joint behaviour. The most obvious case in Figure 6-2 is (a), for which the appropriate gap is between adjacent braces. However, if an intermediate brace exists, as in case (d), the appropriate gap is between the outer loaded braces. In this case, since the gap is often large, the K-joint resistance could revert to that of a Y-joint. Case (e) is instructive in that the appropriate gap for the middle brace is gap 1, whereas for the top brace it is gap 2. Although the bottom brace is treated as 100% K classification, a weighted average in resistance is required, depending on how much of the acting axial force in this brace is balanced by the middle brace (gap 1) and how much is balanced by the top brace (gap 2).

Comm. 6.4.3.3 Strength factor Q_u

The Q_u term for tension forces is based on limiting the resistance to first crack.

Comm. 6.4.5 Ringstiffened joints

For ringstiffened joints, the load effects determined by elastic theory will, in general, include local stress peaks. In the ULS check, such peaks may be reduced to mean values within limited areas. The extent of these areas shall be evaluated for the actual geometry. An assumed redistribution of stress should not lead to significant change in the equilibrium of the different parts of the joint. E. g. if an action in a brace is resisted by a shear force over a ringstiffener with an associated moment, a removal of local stress peaks should not imply significant reduction in this moment required for equilibrium.

Ringstiffened joints may be designed according to plastic theory provided that all parts of the joint belongs to cross section class 1 or 2 of NS 3472. Load effects may be determined by assuming relevant plastic collapse mechanisms. The characteristic resistance shall be determined by recognised methods of plastic theory. The design resistance is determined by dividing the characteristic resistance by $\gamma_M = 1.15$.
The resistance may also be determined based on non-linear analysis. The computer programme used for such analysis should be validated as providing reliable results in comparison with other analysis and tests. Also the type of input to the analysis should be calibrated to provide reliable results. This includes type of element used, element mesh and material description in terms of stress strain relationship.

Comm. 6.5.3.1 Local buckling under axial compression

Platforms generally have a very small number of cones. Thus, it might be more expeditious to design the cones with a geometry such that the axial resistance is equal to that of yield, see Section 6.3.3.

Comm. 6.5.3.2 Junction yielding

The resistance of the junction is checked according to von Mises yield criterion when the hoop stress is tensile.

Comm. 6.5.4.1 Hoop buckling

Hoop buckling is analysed similarly to that of a tubular subjected to external pressure using equivalent geometry properties.

Comm. 6.5.4.2 Junction yielding and buckling

The load effect from external pressure is directly added to the existing stress at the junction for utilisation check with respect to yielding and buckling.

Comm. 6.5.5.2 Junction rings without external hydrostatic pressure

The resistance of stiffeners at a junction is checked as a ring where the effective area of the tubular and the cone is added to that of the ring section.

Comm. 6.5.5.3 Junction rings with external hydrostatic pressure

The required moment of inertia is derived as the sum of that required for the junction itself and that due to external pressure.

Comm. 6.5.5.4 Intermediate stiffening rings

Design of intermediate ring stiffeners within a cone is performed along the same principles as used for design of ring stiffeners in tubulars.

Comm. 6.6.1.1 Failure modes

Stocky plate elements need only to be checked against excessive yielding. With stocky plates are understood plates with sufficient low slenderness so the resistance against compressive stresses is not reduced due to buckling. I.e. the plate elements satisfy the requirements for sections class 3 in NS3472 or Eurocode 3. By excessive yielding is meant yielding which is associated with failure or collapse of the structure or structural component. Methods for resistance checks of plates against yielding include:

- elastic analysis with check of von Mises yield criteria at all points (a conservative check against excessive yielding),
- yield line theory, and
- non-linear finite element analysis.

Comm. 6.6.1.3 Buckling of plates

Slender plates designed according to the effective width formula utilise the plates in the post critical range. This means that higher plate stresses than the buckling stress according to linear theory or the so-called critical buckling stress are allowed. Very slender plates, i.e. span to thickness ratio greater than 150, may need to be checked for serviceability limit states or fatigue limit states. Failure modes in the serviceability limit states are reduced aesthetic appearance due to out of plane distortions or snap through if the plate is suddenly changing its out of plane deformation pattern. As the main source for the distortions will be due to welding during fabrication, the most effective way to prevent these phenomena is to limit the slenderness of the plate. The likelihood of fatigue cracking at the weld along the edges of the plate may increase for very slender plates if the in plane loading is dynamic. This stems from bending stresses in the plate created by out of plane deflection in a deflected plate with in plane loading. For plates with slenderness less than 150, ordinary fatigue checks where out of plane deflections of plate are disregarded will be sufficient.

Comm. 6.6.4.1 General

For wide flanges the stresses in the longitudinal direction will vary due to shear deformations. (Shear lag). For buckling check of flanges with longitudinal stiffeners shear lag effects may be neglected as long as the flange width is less than 0.2L to each side of the web (bulkhead). L being length between points of counterflexure.

Comm. 6.6.4.6 Interaction formulas for axial compression and lateral pressure

The equations (6.143) and (6.144) may be seen as interaction formulas for the stiffener and plate side respectively for a section at the support. Equations (6.145) and (6.146) are likewise interaction checks at the mid-span of the stiffener. See also Figure 12-6.

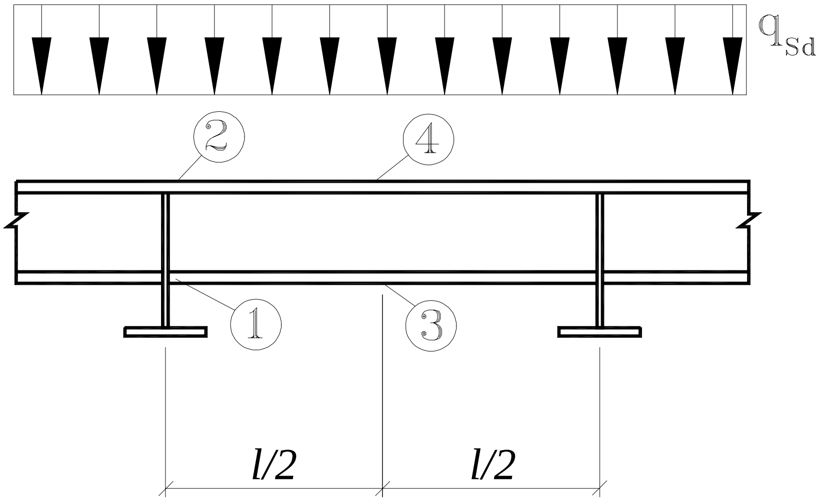

Figure 12-6 Check points for interaction equations

With the lateral load on the stiffener side, the stresses change sign and the equations (6.147) to (6.150) shall be used. The sections to be checked remain the same.

The eccentricity z* is introduced in the equations to find the maximum resistance of the stiffened panel. In the ultimate limit state a continuos stiffened panel will carry the load in the axis giving the maximum load. For calculation of the forces and moments in the total structure, of which the stiffened panel is a part, the working point for the stiffened panel should correspond to the assumed value of z*. In most cases the influence of variations in z* on global forces and moments will be negligible. See also Figure 12-7.

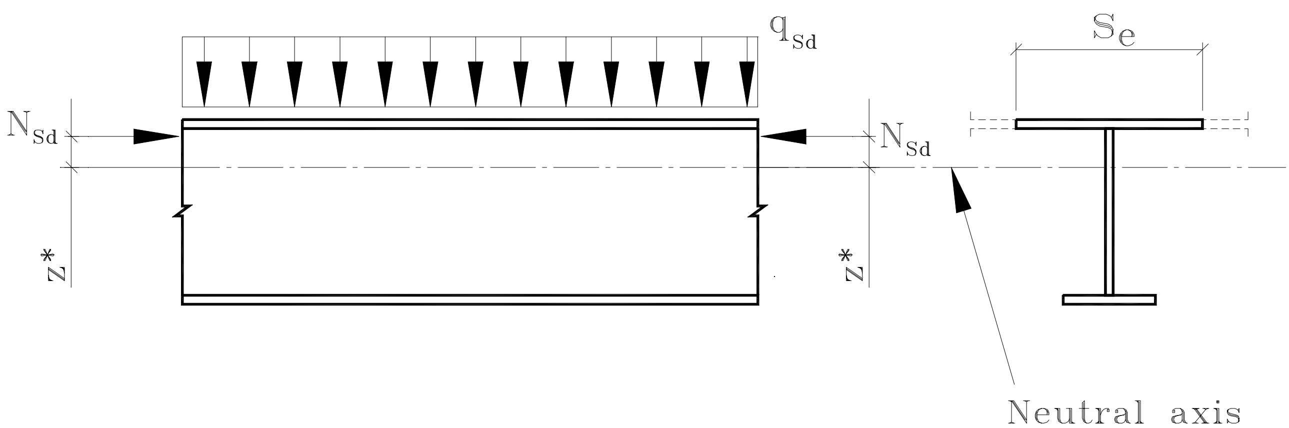

Figure 12-7 Definition of z^*. Positive value shown

Comm. 6.6.5 Buckling of girders

When a stiffened panel supported by girders is subjected to lateral loads the moments from this load should be included in the check of the girder. If the girder is checked according to method 1, the stiffener and plate should also be checked for the σ_y stresses imposed by the bending of the girder.

In method 2, the σ_y stresses imposed by the bending of the girder can be neglected when checking plate and stiffener.

Comm. 6.7 Design of cylindrical shells

A tubular section in air with diameter/thickness ratio larger than 60 is likely to fail by local buckling at an axial stress less than the material yield strength. Based on Eurocode 3 the upper limit of section class 3, where an axial stress equal yield strength can be achieved, is a D/t ratio of $21150/f_y$ where f_y is material yield strength in MPa. The resistance of members failing due to local buckling is more sensitive to geometric imperfections than members that can sustain yielding over the thickness and allow some redistribution of local stresses due to yielding. The failure of such members is normally associated with a descending post-critical behaviour that may be compared more with that of a brittle structure, i. e. the redistribution of load can not be expected. Structures with this behaviour are denoted as shells. A definition of a shell structure should not only include geometry and material resistance, but also loading as the axial resistance is reduced by e. g. increasing pressure. Design equations have been developed to account for different loading conditions, see Annex B. The background for these design equations is given by Odland /2/.

Comm. 9 Accidental Damage Limit States

Examples of failure criteria are:

- Critical deformation criteria defined by integrity of passive fire protection. To be considered for walls resisting explosion pressure and shall serve as fire barrier after the explosion.
- Critical deflection for structures to avoid damage to process equipment (Riser, gas pipe, etc). To be considered for structures or part of structures exposed to impact loads as ship collision, dropped object etc.
- Critical deformation to avoid leakage of compartments. To be considered in case of impact against floating structures where the acceptable collision damage is defined by the minimum number of undamaged compartments to remain stable.

Comm. 10.8.3 Grouted joints

This clause is based on draft ISO 13819-2, /13/.

Recommended Q_U values are given in Table 12-1.

Table 12-1 Values for Q_u for grouted joints

	Brace action		
	Axial Tension	**In-plane Bending**	**Out-of-plane bending**
K / Y	$2.5\,\beta\,\gamma\,K_a$	$1.5\,\beta\,\gamma$	$1.5\,\beta\,\gamma$
X	$2.5\,\beta\,\gamma\,K_a$	$1.5\,\beta\,\gamma$	$1.5\,\beta\,\gamma \big/ \sqrt{Q_\beta}$

$$K_a = \frac{1}{2}\left(1+\frac{1}{\sin\theta}\right)$$

No Q_U term is given for axial compression, since the compression resistance of grouted joints in most cases are limited by that of the brace.

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DESIGN OF STEEL STRUCTURES

ANNEX A
DESIGN AGAINST ACCIDENTAL ACTIONS

CONTENTS

A.1 SYMBOLS

A	Cross-sectional area
A_e	Effective area of stiffener and effective plate flange
A_s	Area of stiffener
A_p	Projected cross-sectional area
B	Width of contact area
C_D	Hydrodynamic drag coefficient
D	Diameter of circular sections, plate stiffness
E	Young's Modulus of elasticity, $2.1 \cdot 10^5$
E_p	Plastic modulus
E_{kin}	Kinetic energy
E_s	Strain energy
F	Lateral load, total load
H	Non-dimensional plastic stiffness
I	Moment of inertia, impuls
J	Mass moment of inertia
K	Stiffness, characteristic stiffness, plate stiffness
K_{LM}	Load-mass transformation factor
L	Beam length
M	Total mass, cross-sectional moment
M_P	Plastic bending moment resistance
N_P	Plastic axial resistance
T	Fundamental period of vibration
N	Axial force
N_{Sd}	Design axial compressive force
N_{Rd}	Design axial compressive capacity
R	Resistance
R_0	Plastic collapse resistance in bending
V	Volume, displacement
W_P	Plastic section modulus
W	Elastic section modulus
a	Added mass
a_s	Added mass for ship
a_i	Added mass for installation
b	Width of collision contact zone
b_f	Flange width
c	Factor

c_f	Axial flexibility factor
c_{lp}	Plastic zone length factor
c_w	Displacement factor for strain calculation
d	Smaller diameter of threaded end of drill collar
d_c	Characteristic dimension for strain calculation
$\bar{f}$	Generalised load
f_y	Characteristic yield strength
g	Acceleration of gravity, 9.81 m/s^2
h_w	Web height for stiffener/girder
i	Radius of gyration
$\bar{k}$	Generalised stiffness
k_l	Load transformation factor
k_m	Mass transformation factor
k_{lm}	Load-mass transformation factor
l	Plate length, beam length
m	Distributed mass
m_s	Ship mass
m_i	Installation mass
m_{eq}	Equivalent mass
$\bar{m}$	Generalised mass
p	Explosion pressure
p_c	Plastic collapse pressure in bending for plate
r	Radius of deformed area
s	Distance, stiffener spacing
s_c	Characteristic distance
s_e	Effective width of plate
t	Thickness, time
t_d	Duration of explosion
t_f	Flange thickness
t_w	Web thickness
v_s	Velocity of ship
v_i	Velocity of installation
v_t	Terminal velocity
w	Deformation, displacement
w_c	Characteristic deformation
w_d	dent depth
$\bar{w}$	Non-dimensional deformation
x	Axial coordinate
y	Generalised displacement, displacement amplitude
y_{el}	Generalised displacement at elastic limit
z	Distance from pivot point to collision point

α	Plate aspect parameter
β	Cross-sectional slenderness factor
ε	Yield strength factor, strain
ε_{cr}	Critical strain for rupture
ε_y	Yield strain
η	Plate eigenperiod parameter
ϕ	Displacement shape function
$\bar{\lambda}$	Reduced slenderness ratio
μ	Ductility ratio
ν	Poisson's ratio, 0.3
θ	Angle
ρ	Density of steel, 7860 kg/m^3
ρ_w	Density of sea water, 1025 kg/m^3
τ	Shear stress
τ_{cr}	Critical shear stress for plate plugging
ξ	Interpolation factor
ψ	Plate stiffness parameter

A.2 GENERAL

This Annex deals with the design to maintain the load-bearing function of the structures during accidental events. The overall goal of the design against accidental actions is to achieve a system where the main safety functions of the installation are not impaired.

Design Accidental Actions and associated performance criteria are determined by Quantified Risk Assessment (QRA), see NORSOK N-003 /1/.

In conjunction with design against accidental actions, performance criteria may need to be formulated such that the structure or components or sub-assemblies thereof - during the accident or within a certain time period after the accident - shall not impair the main safety functions such as:

- usability of escapeways,
- integrity of shelter areas,
- global load bearing capacity.

The performance criteria derived will typically be related to:

- energy dissipation
- local strength
- resistance to deformation (e.g braces in contact with risers/caissons, use of escape ways)
- endurance of fire protection
- ductility (allowable strains) - to avoid cracks in components, fire walls, passive fire protection etc.

The inherent uncertainty of the frequency and magnitude of the accidental loads as well as the approximate nature of the methods for determination analysis of accidental load effects shall be recognised. It is therefore essential to apply sound engineering judgement and pragmatic evaluations in the design.

The material factor to be used for checks of accidental limit states is $\gamma_M = 1.0$

A.3 SHIP COLLISIONS

A.3.1 General

The ship collision action is characterised by a kinetic energy, governed by the mass of the ship, including hydrodynamic added mass and the speed of the ship at the instant of impact. Depending upon the impact conditions, a part of the kinetic energy may remain as kinetic energy after the impact. The remainder of the kinetic energy has to be dissipated as strain energy in the installation and, possibly, in the vessel. Generally this involves large plastic strains and significant structural damage to either the installation or the ship or both. The strain energy dissipation is estimated from force-deformation relationships for the installation and the ship, where the deformations in the installation shall comply with ductiliy and stability requirements.

The load bearing function of the installation shall remain intact with the damages imposed by the ship collision action. In addition, the residual strength requirements given in Section A.7 shall be complied with.

The structural effects from ship collision may either be determined by non-linear dynamic finite element analyses or by energy considerations combined with simple elastic-plastic methods.

If non-linear dynamic finite element analysis is applied all effects described in the following paragraphs shall either be implicitly covered by the modelling adopted or subjected to special considerations, whenever relevant.

Often the integrity of the installation can be verified by means of simple calculation models.

If simple calculation models are used the part of the collision energy that needs to be dissipated as strain energy can be calculated by means of the principles of conservation of momentum and conservation of energy, refer Section A.3.3.

It is convenient to consider the strain energy dissipation in the installation to take part on three different levels:

- local cross-section
- component/sub-structure
- total system

Interaction between the three levels of energy dissipation shall be considered.

Plastic modes of energy dissipation shall be considered for cross-sections and component/substructures in direct contact with the ship. Elastic strain energy can in most cases be disregarded, but elastic axial flexibility may have a substantial effect on the load-deformation relationships for components/sub-structures. Elastic energy may contribute significantly on a global level.

A.3.2 Design principles

With respect to the distribution of strain energy dissipation there may be distinguished between, see FigureA.3-1:

- strength design
- ductility design
- shared-energy design

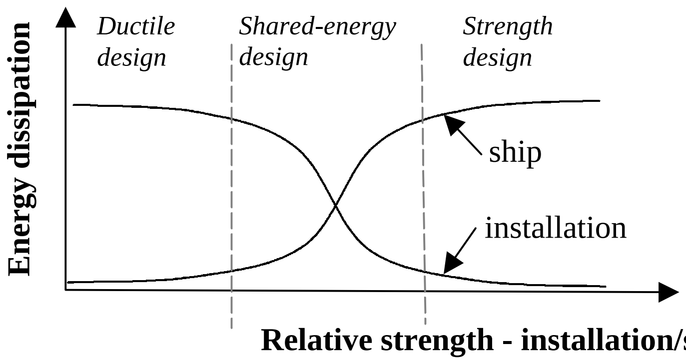


Figure A.3-1 Energy dissipation for strength, ductile and shared-energy design

Strength design implies that the installation is strong enough to resist the collision force with minor deformation, so that the ship is forced to deform and dissipate the major part of the energy.

Ductility design implies that the installation undergoes large, plastic deformations and dissipates the major part of the collision energy.

Shared energy design implies that both the installation and ship contribute significantly to the energy dissipation.

From calculation point of view strength design or ductility design is favourable. In this case the response of the «soft» structure can be calculated on the basis of simple considerations of the geometry of the «rigid» structure. In shared energy design both the magnitude and distribution of the collision force depends upon the deformation of both structures. This interaction makes the analysis more complex.

In most cases ductility or shared energy design is used. However, strength design may in some cases be achievable with little increase in steel weight.

A.3.3 Collision mechanics

A.3.3.1 Strain energy dissipation

The collision energy to be dissipated as strain energy may - depending on the type of installation and the purpose of the analysis - be taken as:

Compliant installations

$$E_s = \frac{1}{2}(m_s + a_s)v_s^2 \frac{\left(1 - \frac{v_i}{v_s}\right)^2}{1 + \frac{m_s + a_s}{m_i + a_i}} \qquad \textbf{(A.3.1)}$$

Fixed installations

$$E_s = \frac{1}{2}(m_s + a_s)v_s^2 \qquad \textbf{(A.3.2)}$$

Articulated columns

$$E_s = \frac{1}{2}(m_s + a_s)\frac{\left(1-\frac{v_i}{v_s}\right)^2}{1+\frac{m_s z^2}{J}} \qquad \textbf{(A.3.3)}$$

m_s = ship mass
a_s = ship added mass
v_s = impact speed
m_i = mass of installation
a_i = added mass of installation
v_i = velocity of installation
J = mass moment of inertia of installation (including added mass) with respect to effective pivot point
z = distance from pivot point to point of contact

In most cases the velocity of the installation can be disregarded, i.e. $v_i = 0$.

The installation can be assumed compliant if the duration of impact is small compared to the fundamental period of vibration of the installation. If the duration of impact is comparatively long, the installation can be assumed fixed.

Jacket structures can normally be considered as fixed. Floating platforms (semi-submersibles, TLP's, production vessels) can normally be considered as compliant. Jack-ups may be classified as fixed or compliant.

A.3.3.2 Reaction force to deck

In the acceleration phase the inertia of the topside structure generates large reaction forces. An upper bound of the maximum force between the collision zone and the deck for bottom supported installations may be obtained by considering the platform compliant for the assessment of total strain energy dissipation and assume the platform fixed at deck level when the collision response is evaluated.

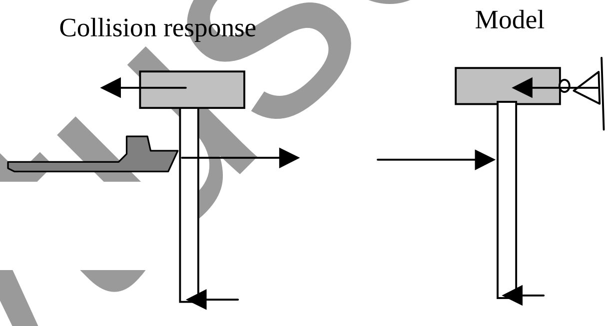


Figure A.3-2 Model for assessment of reaction force to deck

A.3.4 Dissipation of strain energy

The structural response of the ship and installation can formally be represented as load-deformation relationships as illustrated in Figure A.3-3. The strain energy dissipated by the ship and installation equals the total area under the load-deformation curves.

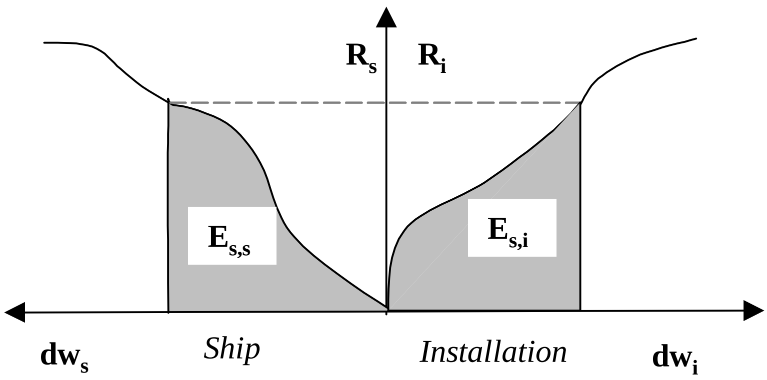


Figure A.3-3 Dissipation of strain energy in ship and platform

$$E_s = E_{s,s} + E_{s,i} = \int_0^{w_{s,max}} R_s dw_s + \int_0^{w_{i,max}} R_i dw_i \quad \textbf{(A.3.4)}$$

As the load level is not known a priori an incremental procedure is generally needed.

The load-deformation relationships for the ship and the installation are often established independently of each other assuming the other object infinitely rigid. This method may have, however, severe limitations; both structures will dissipate some energy regardless of the relative strength.

Often the stronger of the ship and platform will experience less damage and the softer more damage than what is predicted with the approach described above. As the softer structure deforms the impact force is distributed over a larger contact area. Accordingly, the resistance of the strong structure increases. This may be interpreted as an "upward" shift of the resistance curve for the stronger structure (refer Figure A.3-3).

Care should be exercised that the load-deformation curves calculated are representative for the true, interactive nature of the contact between the two structures.

A.3.5 Ship collision forces

A.3.5.1 Recommended force-deformation relationships

Force-deformation relationships for a supply vessels with a displacement of 5000 tons are given in Figure A.3-4 for broad side -, bow-, stern end and stern corner impact for a vessel with stern roller.

The curves for broad side and stern end impacts are based upon penetration of an infinitely rigid, vertical cylinder with a given diameter and may be used for impacts against jacket legs (D = 1.5 m) and large diameter columns (D = 10m).

The curve for stern corner impact is based upon penetration of an infinitely rigid cylinder and may be used for large diameter column impacts.

In lieu of more accurate calculations the curves in Figure A.3-4 may be used for square-rounded columns.

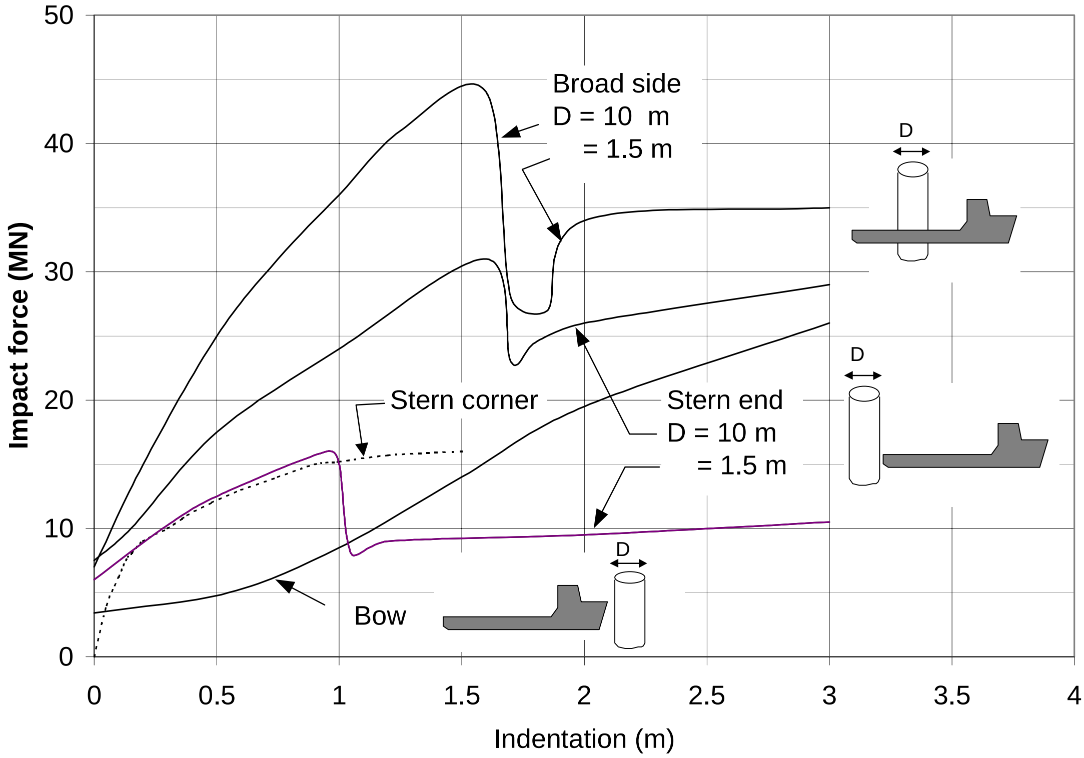


Figure A.3-4 Recommended-deformation curve for beam, bow and stern impact

The curve for bow impact is based upon collision with an infinitely rigid, plane wall and may be used for large diameter column impacts, but should not be used for significantly different collision events, e.g. impact against tubular braces.

For beam -, stern end – and stern corner impacts against jacket braces all energy shall normally be assumed dissipated by the brace, refer Comm. A.3.5.2.

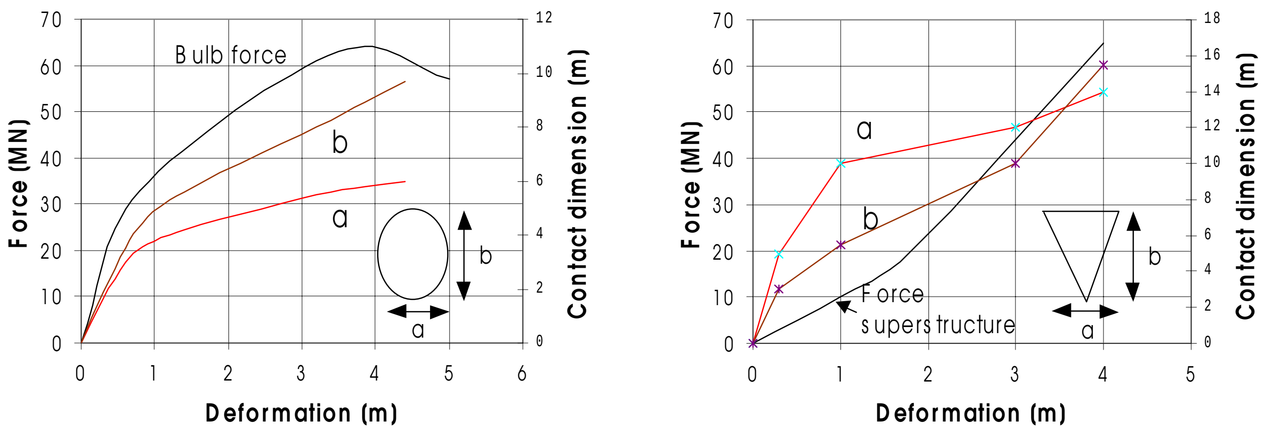


Figure A.3-5 Force -deformation relationship for tanker bow impact (~ 125.000 dwt)

Force-deformation relationships for tanker bow impact is given in Figure A.3-5 for the bulbous part and the superstructure, respectively. The curves may be used provided that the impacted structure (e.g. stern of floating production vessels) does not undergo substantial deformation i.e. strength design requirements are complied with. If this condition is not met interaction between the bow and the impacted structure shall be taken into consideration. Non-linear finite element methods or simplified plastic analysis techniques of members subjected to axial crushing shall be employed /3/, /5/.

A.3.5.2 Force contact area for strength design of large diameter columns.

The basis for the curves in Figure A.3-4 is strength design, i.e. limited local deformations of the installation at the point of contact. In addition to resisting the total collision force, large diameter columns have to resist local concentrations (subsets) of the collision force, given for stern corner impact in Table A.3-1 and stern end impact in Table A.3-2.

Table A.3-1 Local concentrated collision force -evenly distributed over a rectangular area. Stern corner impact

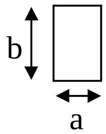
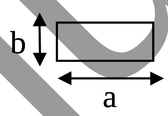
Contact area a(m)	Contact area b (m)	Force (MN)	
0.35	0.65	3.0	
0.35	1.65	6.4	
0.20	1.15	5.4	

Table A.3-2 Local concentrated collision force -evenly distributed over a rectangular area. Stern end impact

Contact area a (m)	Contact area b(m)	Force (MN)	
0.6	0.3	5.6	
0.9	0.5	7.5	
2.0	1.1	10	

If strength design is not aimed for - and in lieu of more accurate assessment (e.g. nonlinear finite element analysis) - all strain energy has to be assumed dissipated by the column, corresponding to indentation by an infinitely rigid stern corner.

A.3.6 Force-deformation relationships for denting of tubular members

The contribution from local denting to energy dissipation is small for brace members in typical jackets and should be neglected.

The resistance to indentation of unstiffened tubes may be taken from Figure A.3-6. Alternatively, the resistance may be calculated from Equation (A.3.5):

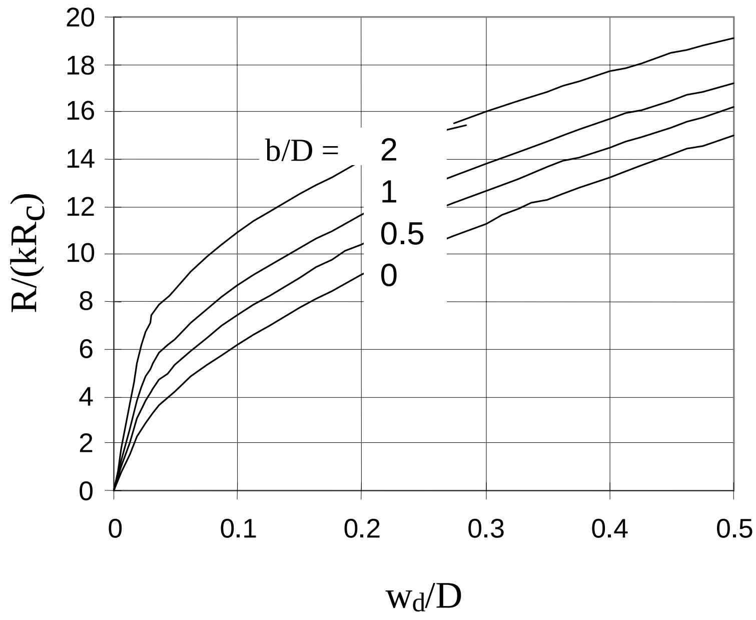


Figure A.3-6 Resistance curve for local denting

$$\frac{R}{R_c} = kc_1\left(\frac{w_d}{D}\right)^{c_2} \qquad \textbf{(A.3.5)}$$

$$R_c = f_y \frac{t^2}{4}\sqrt{\frac{D}{t}}$$

$$c_1 = 22 + 1.2\frac{B}{D}$$

$$c_2 = \frac{1.925}{3.5 + \frac{B}{D}}$$

$$k = 1.0 \qquad \frac{N_{Sd}}{N_{Rd}} \le 0.2$$

$$k = 1.0 - 2\left(\frac{N_{Sd}}{N_{Rd}} - 0.2\right) \qquad 0.2 < \frac{N_{Sd}}{N_{Rd}} < 0.6$$

$$k = 0 \qquad 0.6 \le \frac{N_{Sd}}{N_{Rd}}$$

N_{Sd} = design axial compressive force
N_{Rd} = design axial compressive resistance
B = width of contact area
w_d = dent depth

The curves are inaccurate for small indentation, and they should not be used to verify a design where the dent damage is required to be less than $\frac{w_d}{D} < 0.05$

A.3.7 Force-deformation relationships for beams

A.3.7.1 General

The response of a beam subjected to a collision load is initially governed by bending, which is affected by and interacts with local denting under the load. The bending capacity is also reduced if local buckling takes place on the compression side. As the beam undergoes finite deformations, the load carrying capacity may increase considerably due to the development of membrane tension forces. This depends upon the ability of adjacent structure to restrain the connections at the member ends to inward displacements. Provided that the connections do not fail, the energy dissipation capacity is either limited by tension failure of the member or rupture of the connection.

Simple plastic methods of analysis are generally applicable. Special considerations shall be given to the effect of :

- elastic flexibility of member/adjacent structure
- local deformation of cross-section
- local buckling
- strength of connections
- strength of adjacent structure
- fracture

A.3.7.2 Plastic force-deformation relationships including elastic, axial flexibility

Relatively small axial displacements have a significant influence on the development of tensile forces in members undergoing large lateral deformations. An equivalent elastic, axial stiffness may be defined as

$$\frac{1}{K} = \frac{1}{K_{node}} + \frac{\ell}{2EA} \qquad \textbf{(A.3.6)}$$

K_{node} = axial stiffness of the node with the considered member removed. This may be determined by introducing unit loads in member axis direction at the end nodes with the member removed.

Plastic force-deformation relationship for a central collision (midway between nodes) may be obtained from :

- Figure A.3-7 for tubular members
- Figure A.3-8 for stiffened plates in lieu of more accurate analysis.

The following notation applies:

$R_0 = \frac{4c_1 M_P}{\ell}$	plastic collapse resistance in bending for the member, for the case that contact point is at mid span
$\overline{w} = \frac{w}{c_1 w_c}$	non-dimensional deformation
$c = \frac{4c_1 K w_c^{\,2}}{f_y A \ell}$	non-dimensional spring stiffness
$c_1 = 2$	for clamped beams
$c_1 = 1$	for pinned beams

$w_c = \frac{D}{2}$ characteristic deformation for tubular beams

$w_c = \frac{1.2W_P}{A}$ characteristic deformation for stiffened plating

W_P = plastic section modulus

ℓ = member length

For non-central collisions the force-deformation relationship may be taken as the mean value of the force-deformation curves for central collision with member half length equal to the smaller and the larger portion of the member length, respectively.

For members where the plastic moment capacity of adjacent members is smaller than the moment capacity of the impacted member the force-deformation relationship may be interpolated from the curves for pinned ends and clamped ends:

$$R = \xi R_{clamped} + (1-\xi) R_{pinned} \tag{A.3.7}$$

where

$$0 \le \xi = \frac{R_0^{actual}}{4\frac{M_P}{\ell}} - 1 \le 1 \tag{A.3.8}$$

R_0^{actual} = Plastic resistance by bending action of beam accounting for actual bending resistance of adjacent members

$$R_0^{actual} = \frac{4M_P + 2M_{P1} + 2M_{P2}}{\ell} \tag{A.3.9}$$

$$M_{Pj} = \sum_i M_{Pj,i} \le M_P \quad \text{i = adjacent member no i,} \quad \text{j = end number \{1,2\}} \tag{A.3.10}$$

$M_{Pj,i}$ = Plastic bending resistance for member no. i.

Elastic, rotational flexibility of the node is normally of moderate significance

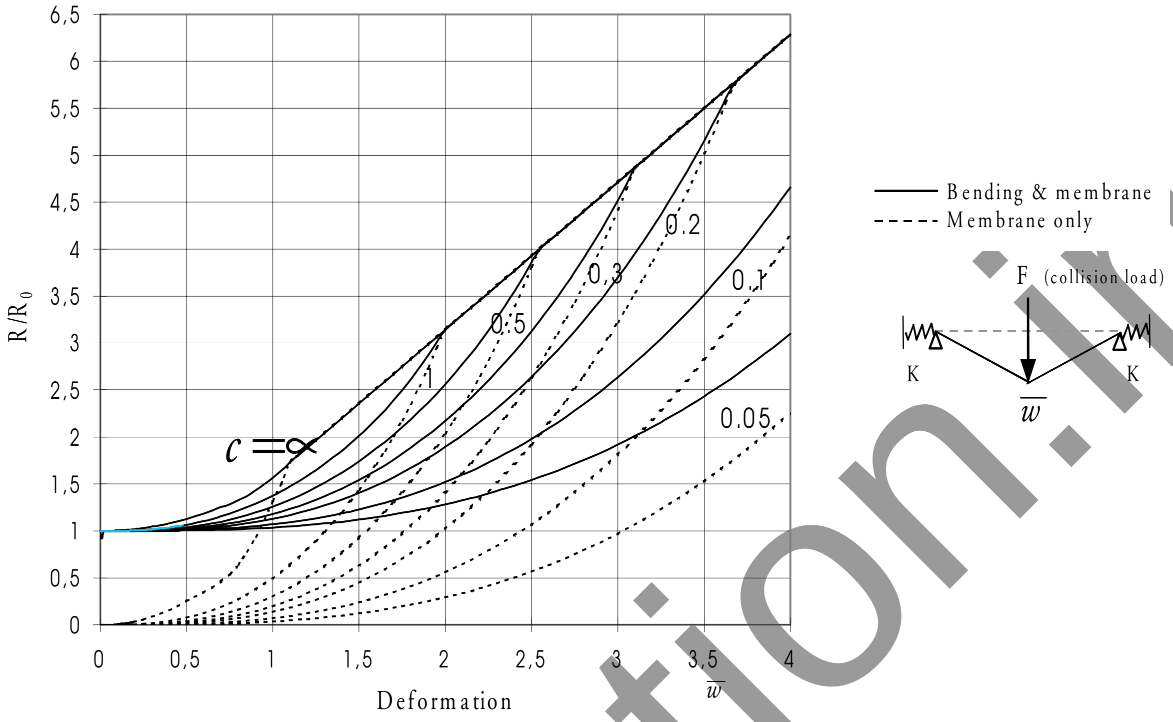


Figure A.3-7 Force-deformation relationship for tubular beam with axial flexibility

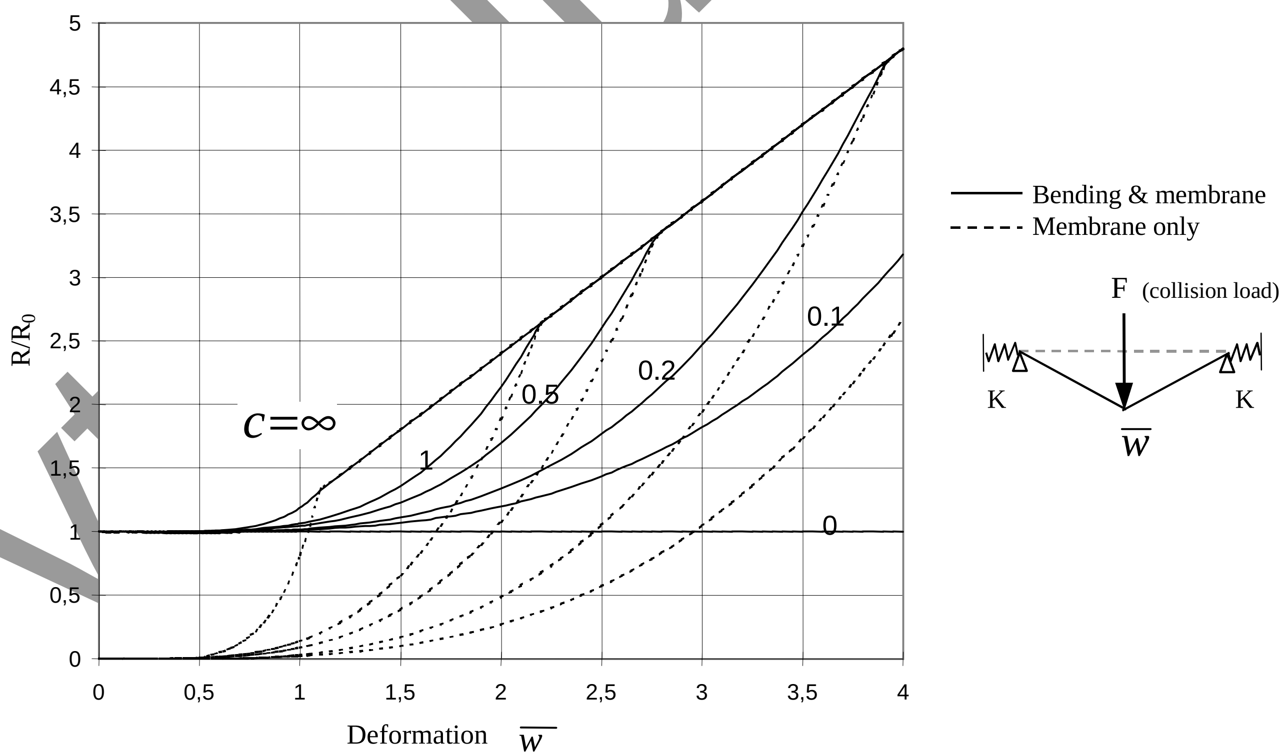


Figure A.3-8 Force-deformation relationship for stiffened plate with axial flexibility.

A.3.7.3 Bending capacity of dented tubular members

The reduction in plastic moment capacity due to local denting shall be considered for members in compression or moderate tension, but can be neglected for members entering the fully plastic membrane state.

Conservatively, the flat part of the dented section according to the model shown in Figure A.3-9 may be assumed non-effective. This gives:

$$\frac{M_{red}}{M_P} = \cos\frac{\theta}{2} - \frac{1}{2}\sin\theta \qquad \textbf{(A.3.11)}$$

$$M_P = f_y D^2 t$$

$$\theta = \arccos\left(1 - \frac{2w_d}{D}\right)$$

w_d = dent depth as defined in Figure A.3-9.

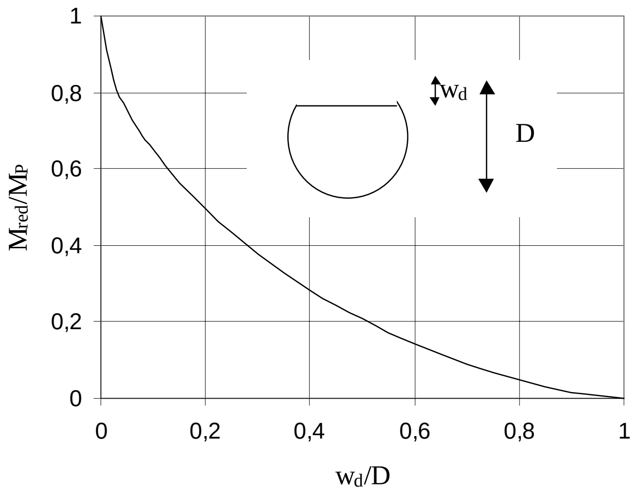


Figure A.3-9 Reduction of moment capacity due to local dent

A.3.8 Strength of connections

Provided that large plastic strains can develop in the impacted member, the strength of the connections that the member frames into has to be checked.

The resistance of connections should be taken from ULS requirements in this standard for tubular joints and Eurocode 3 or NS3472 for other joints.

For braces reaching the fully plastic tension state, the connection shall be checked for a load equal to the axial resistance of the member. The design axial stress shall be assumed equal to the ultimate tensile strength of the material.

If the axial force in a tension member becomes equal to the axial capacity of the connection, the connection has to undergo gross deformations. The energy dissipation will be limited and rupture has to be considered at a given deformation. A safe approach is to assume disconnection of the member once the axial force in the member reaches the axial capacity of the connection.

If the capacity of the connection is exceeded in compression and bending, this does not necessarily mean failure of the member. The post-collapse strength of the connection may be taken into account provided that such information is available.

A.3.9 Strength of adjacent structure

The strength of structural members adjacent to the impacted member/sub-structure must be checked to see whether they can provide the support required by the assumed collapse mechanism. If the adjacent structure fails, the collapse mechanism must be modified accordingly. Since, the physical behaviour becomes more complex with mechanisms consisting of an increasing number of members it is recommended to consider a design which involves as few members as possible for each collision scenario.

A.3.10 Ductility limits

A.3.10.1 General

The maximum energy that the impacted member can dissipate will – ultimately - be limited by local buckling on the compressive side or fracture on the tensile side of cross-sections undergoing finite rotation.

If the member is restrained against inward axial displacement, any local buckling must take place before the tensile strain due to membrane elongation overrides the effect of rotation induced compressive strain.

If local buckling does not take place, fracture is assumed to occur when the tensile strain due to the combined effect of rotation and membrane elongation exceeds a critical value.

To ensure that members with small axial restraint maintain moment capacity during significant plastic rotation it is recommended that cross-sections be proportioned to Class 1 requirements, defined in Eurocode 3 or NS3472.

Initiation of local buckling does, however, not necessarily imply that the capacity with respect to energy dissipation is exhausted, particularly for Class 1 and Class 2 cross-sections. The degradation of the cross-sectional resistance in the post-buckling range may be taken into account provided that such information is available, refer Comm. A.3.10.1

For members undergoing membrane stretching a lower bound to the post-buckling load-carrying capacity may be obtained by using the load-deformation curve for pure membrane action.

A.3.10.2 Local buckling

Circular cross-sections:

Buckling does not need to be considered for a beam with axial restraints if the following condition is fulfilled:

(A.3.12)

$$\beta \leq \left(\frac{14 c_f f_y}{c_1} \left(\frac{\kappa \ell}{d_c} \right)^2 \right)^{\frac{1}{3}}$$

where

$$\beta = \frac{D/t}{235/f_y} \tag{A.3.13}$$

axial flexibility factor

$$c_f = \left(\frac{\sqrt{c}}{1+\sqrt{c}}\right)^2 \tag{A.3.14}$$

d_c	=		characteristic dimension
	=	D	for circular cross-sections
c_1	=	2	for clamped ends
	=	1	for pinned ends
c	=		non-dimensional spring stiffness, refer Section A.3.7.2
$\kappa \ell \leq 0.5 \ell$	=		the smaller distance from location of collision load to adjacent joint

If this condition is not met, buckling may be assumed to occur when the lateral deformation exceeds

$$\frac{w}{d_c} = \frac{1}{2c_f}\left(1 - \sqrt{1 - \frac{14 c_f f_y}{c_1 \beta^3}\left(\frac{\kappa\ell}{d_c}\right)^2}\right) \tag{A.3.15}$$

For small axial restraint (c < 0.05) the critical deformation may be taken as

$$\frac{w}{d_c} = \frac{3.5 f_y}{c_1 \beta^3}\left(\frac{\kappa\ell}{d_c}\right)^2 \tag{A.3.16}$$

Stiffened plates/ I/H-profiles:

In lieu of more accurate calculations the expressions given for circular profiles in Eq. (A.3.15) and (A.3.16) may be used with

d_c	=		characteristic dimension for local buckling, equal to twice the distance from the plastic neutral axis in bending to the extreme fibre of the cross-section
	=	h	height of cross-section for symmetric I –profiles
	=	$2h_w$	for stiffened plating (for simplicity)

For flanges subjected to compression;

$$\beta = 2.5 \frac{b_f/t_f}{\sqrt{235/f_y}} \quad \text{class 1 cross-sections} \tag{A.3.17}$$

$$\beta = 3 \frac{b_f/t_f}{\sqrt{235/f_y}} \quad \text{class 2 and class 3 cross-sections} \tag{A.3.18}$$

For webs subjected to bending

$$\beta = 0.7 \frac{h_w / t_w}{\sqrt{235/f_y}} \qquad \text{class 1 cross-sections} \qquad \textbf{(A.3.19)}$$

$$\beta = 0.8 \frac{h_w / t_w}{\sqrt{235/f_y}} \qquad \text{class 2 and class 3 cross-sections} \qquad \textbf{(A.3.20)}$$

b_f = flange width
t_f = flange thickness
h_w = web height
t_w = web thickness

A.3.10.3 Tensile Fracture

The degree of plastic deformation or critical strain at fracture will show a significant scatter and depends upon the following factors:

- material toughness
- presence of defects
- strain rate
- presence of strain concentrations

The critical strain for plastic deformations of sections containing defects need to be determined based on fracture mechanics methods. (See chapter 6.5.) Welds normally contain defects and welded joints are likely to achieve lower toughness than the parent material. For these reasons structures that need to undergo large plastic deformations should be designed in such a way that the plastic straining takes place away outside the weld. In ordinary full penetration welds, the overmatching weld material will ensure that minimal plastic straining occurs in the welded joints even in cases with yielding of the gross cross section of the member. In such situations, the critical strain will be in the parent material and will be dependent upon the following parameters:

- stress gradients
- dimensions of the cross section
- presence of strain concentrations
- material yield to tensile strength ratio
- material ductility

Simple plastic theory does not provide information on strains as such. Therefor, strain levels should be assessed by means of adequate analytic models of the strain distributions in the plastic zones or by non-linear finite element analysis with a sufficiently detailed mesh in the plastic zones.

When structures are designed so that yielding take place in the parent material, the following value for the critical average strain in axially loaded plate material may be used in conjunction with nonlinear finite element analysis or simple plastic analysis

$$\varepsilon_{cr} = 0.02 + 0.65 \frac{t}{\ell} \qquad \textbf{(A.3.21)}$$

where:

t = plate thickness
ℓ = length of plastic zone. Minimum 5t

A.3.10.4 Tensile fracture in yield hinges

When the force deformation relationships for beams given in Section A.3.7.2 are used rupture may be assumed to occur when the deformation exceeds a value given by

$$\frac{w}{d_c} = \frac{1}{2c_f}\left(\sqrt{1+4c_w c_f \varepsilon_{cr}} - 1\right) \qquad \textbf{(A.3.22)}$$

where the following factors are defined;

Displacement factor

$$c_w = \frac{1}{c_1}\left(c_{lp}\left(1-\frac{2}{3}c_{lp}\right)+4\left(1-\frac{W}{W_P}\right)\frac{\varepsilon_{cr}}{\varepsilon_y}\right)\left(\frac{\kappa\ell}{D}\right)^2 \qquad \textbf{(A.3.23)}$$

plastic zone length factor

$$c_{lp} = \frac{\left(\frac{\varepsilon_{cr}}{\varepsilon_y}-1\right)\frac{W}{W_P}H}{\left(\frac{\varepsilon_{cr}}{\varepsilon_y}-1\right)\frac{W}{W_P}H+1} \qquad \textbf{(A.3.24)}$$

axial flexibility factor

$$c_f = \left(\frac{\sqrt{c}}{1+\sqrt{c}}\right)^2 \qquad \textbf{(A.3.25)}$$

non-dimensional plastic stiffness

$$H = \frac{E_p}{E} = \frac{1}{E}\left(\frac{f_{cr}-f_y}{\varepsilon_{cr}-\varepsilon_y}\right) \qquad \textbf{(A.3.26)}$$

c_1	=	2	for clamped ends
	=	1	for pinned ends
c	=		non-dimensional spring stiffness, refer Section A.3.7.2
κl	≤	$0.5l$	the smaller distance from location of collision load to adjacent joint
W	=		elastic section modulus
W_P	=		plastic section modulus
ε_{cr}	=		critical strain for rupture
$\varepsilon_y = \frac{f_y}{E}$			yield strain
f_y	=		yield strength
f_{cr}	=		strength corresponding to ε_{cr}

The characteristic dimension shall be taken as:

d_c	=	D	diameter of tubular beams
	=	$2h_w$	twice the web height for stiffened plates
	=	h	height of cross-section for symmetric I-profiles

For small axial restraint (c < 0.05) the critical deformation may be taken as

$$\frac{w}{d_c} = c_w \varepsilon_{cr} \qquad \textbf{(A.3.27)}$$

The critical strain ε_{cr} and corresponding strength f_{cr} should be selected so that idealised bi-linear stress-strain relation gives reasonable results. See Commentary. For typical steel material grades the following values are proposed:

Table A.3-3 Proposed values for e_{cr} and H for different steel grades

Steel grade	ε_{cr}	H
S 235	20 %	0.0022
S 355	15 %	0.0034
S 460	10 %	0.0034

A.3.11 Resistance of large diameter, stiffened columns

A.3.11.1 General

Impact on a ring stiffener as well as midway between ring stiffeners shall be considered.

Plastic methods of analysis are generally applicable.

A.3.11.2 Longitudinal stiffeners

For ductile design the resistance of longitudinal stiffeners in the beam mode of deformation can be calculated using the procedure described for stiffened plating, section A.3.7.

For strength design against stern corner impact, the plastic bending moment capacity of the longitudinal stiffeners has to comply with the requirement given in Figure A.3-10, on the assumption that the entire load given in Table A.3-2 is taken by one stiffener.

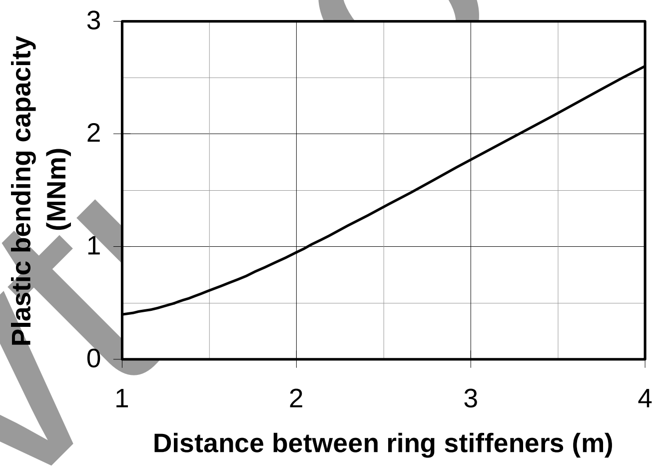


Figure A.3-10 Required bending capacity of longitudinal stiffeners

A.3.11.3 Ring stiffeners

In lieu of more accurate analysis the plastic collapse load of a ring-stiffener can be estimated from:

$$F_0 = \frac{4\sqrt{2}M_P}{\sqrt{w_c D}} \qquad \textbf{(A.3.28)}$$

where

$w_c = \frac{W_P}{A_e}$ = characteristic deformation of ring stiffener

D = column radius

M_P = plastic bending resistance of ring-stiffener including effective shell flange

W_P = plastic section modulus of ring stiffener including effective shell flange

A_e = area of ring stiffener including effective shell flange

Effective flange of shell plating: Use effective flange of stiffened plates, see Chapter 6.

For ductile design it can be assumed that the resistance of the ring stiffener is constant and equal to the plastic collapse load, provided that requirements for stability of cross-sections are complied with, refer Section A.3.10.2.

A.3.11.4 Decks and bulkheads

Calculation of energy dissipation in decks and bulkheads has to be based upon recognised methods for plastic analysis of deep, axial crushing. It shall be documented that the collapse mechanisms assumed yield a realistic representation of the true deformation field.

A.3.12 Energy dissipation in floating production vessels

For strength design the side or stern shall resist crushing force of the bow of the off-take tanker. In lieu of more accurate calculations the force-deformation curve given in Section A.3.5.2 may be applied.

For ductile design the resistance of stiffened plating in the beam mode of deformation can be calculated using the procedure described in section A.3.7.2.

Calculation of energy dissipation in stringers, decks and bulkheads subjected to gross, axial crushing shall be based upon recognised methods for plastic analysis, eg. /3/ and /5/. It shall be documented that the folding mechanisms assumed yield a realistic representation of the true deformation field.

A.3.13 Global integrity during impact

Normally, it is unlikely that the installation will turn into a global collapse mechanism under direct collision load, because the collision load is typically an order of magnitude smaller than the resultant design wave force.

Linear analysis often suffices to check that global integrity is maintained.

The installation should be checked for the maximum collision force.

For installations responding predominantly statically the maximum collision force occurs at maximum deformation.

For structures responding predominantly impulsively the maximum collision force occurs at small global deformation of the platform. An upper bound to the collision force is to assume that the installation is fixed with respect to global displacement. (e.g. jack-up fixed with respect to deck displacement)

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A.4 DROPPED OBJECTS

A.4.1 General

The dropped object action is characterised by a kinetic energy, governed by the mass of the object, including any hydrodynamic added mass, and the velocity of the object at the instant of impact. In most cases the major part of the kinetic energy has to be dissipated as strain energy in the impacted component and, possibly, in the dropped object. Generally, this involves large plastic strains and significant structural damage to the impacted component. The strain energy dissipation is estimated from force-deformation relationships for the component and the object, where the deformations in the component shall comply with ductiliy and stability requirements.

The load bearing function of the installation shall remain intact with the damages imposed by the dropped object action. In addition, the residual strength requirements given in Section A.7 shall be complied with.

Dropped objects are rarely critical to the global integrity of the installation and will mostly cause local damages. The major threat to global integrity is probably puncturing of buoyancy tanks, which could impair the hydrostatic stability of floating installations. Puncturing of a single tank is normally covered by the general requirements to compartmentation and watertight integrity given in ISO 131819-1 and NORSOK N-001.

The structural effects from dropped objects may either be determined by non-linear dynamic finite element analyses or by energy considerations combined with simple elastic-plastic methods as given in Sections A.4.2 - A.4.5.

If non-linear dynamic finite element analysis is applied all effects described in the following paragraphs shall either be implicitly covered by the modelling adopted or subjected to special considerations, whenever relevant.

A.4.2 Impact velocity

The kinetic energy of a falling object is given by

$$E_{kin} = \frac{1}{2}mv^2 \qquad \textbf{(A.4.1)}$$

for objects falling in air and,

$$E_{kin} = \frac{1}{2}(m+a)v^2 \qquad \textbf{(A.4.2)}$$

for objects falling in water.

a = hydrodynamic added mass for considered motion

For impacts in air the velocity is given by

$$v = \sqrt{2gs} \tag{A.4.3}$$

s = travelled distance from drop point
v = v_0 at sea surface

For objects falling rectilinearly in water the velocity depends upon the reduction of speed during impact with water and the falling distance relative to the characteristic distance for the object.

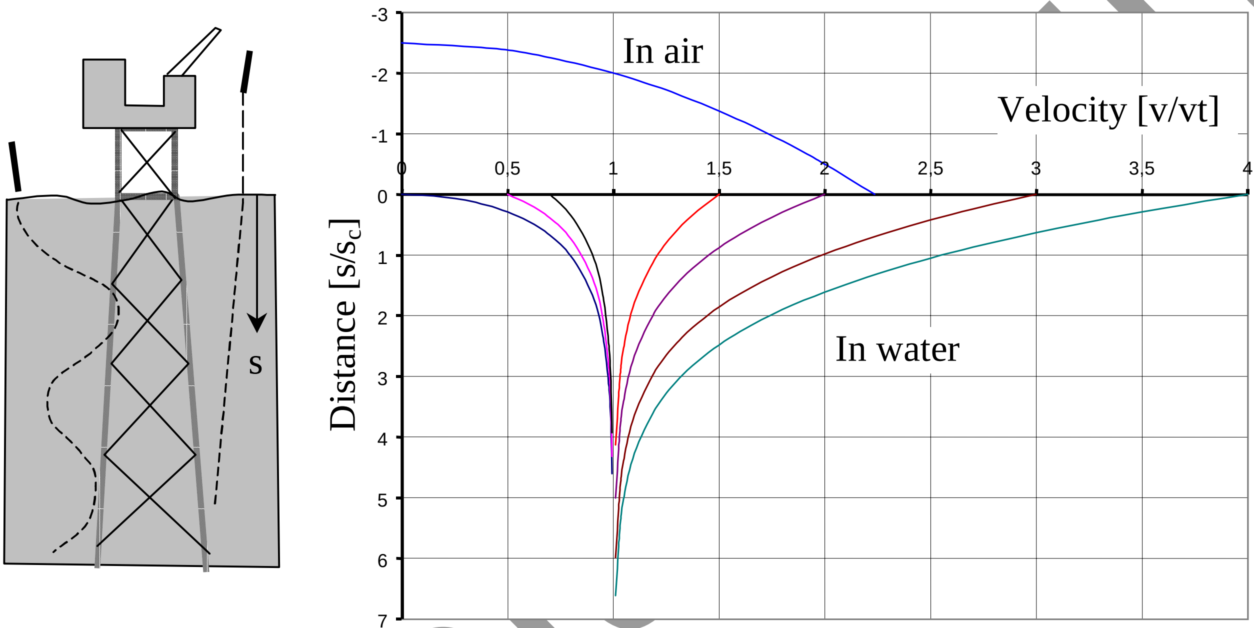


Figure A.4-1 Velocity profile for objects falling in water

The loss of momentum during impact with water is given by

$$m\Delta v = \int_0^{t_d} F(t)dt \tag{A.4.4}$$

F(t) = force during impact with sea surface

After the impact with water the object proceeds with the speed

$$v = v_0 - \Delta v$$

Assuming that the hydrodynamic resistance during fall in water is of drag type the velocity in water can be taken from Figure A.4-1 where

$$v_t = \sqrt{\frac{2g(m - \rho_w V)}{\rho_w C_d A}}$$ = terminal velocity for the object

$$s_c = \frac{m + a}{\rho_w C_d A_p} = \frac{v_t^{\,2}\left(1 + \frac{a}{m}\right)}{2g\left(1 - \frac{\rho_w V}{m}\right)}$$ = characteristic distance

ρ_w = density of sea water
C_d = hydrodynamic drag coefficient for the object in the considered motion
m = mass of object
A_p = projected cross-sectional area of the object
V = object displacement

The major uncertainty is associated with calculating the loss of momentum during impact with sea surface, given by Equation (A.4.4). However, if the travelled distance is such that the velocity is close to the terminal velocity, the impact with sea surface is of little significance.

Typical terminal velocities for some typical objects are given in Table A.4-1

Table A.4-1 Terminal velocities for objects falling in water.

Item	**Mass [kN]**	**Terminal velocity [m/s]**
Drill collar Winch, Riser pump	28 250 100	23-24
BOP annular preventer	50	16
Mud pump	330	7

Rectilinear motion is likely for blunt objects and objects which do not rotate about their longitudinal axis. Bar-like objects (e.g. pipes) which do not rotate about their longitudinal axis may execute lateral, damped oscillatory motions as illustrated in Figure A.4-1.

A.4.3 Dissipation of strain energy

The structural response of the dropped object and the impacted component can formally be represented as load-deformation relationships as illustrated in Figure A.4-2. The part of the impact energy dissipated as strain energy equals the total area under the load-deformation curves.

$$E_s = E_{s,s} + E_{s,i} = \int_0^{w_{s,max}} R_s dw_s + \int_0^{w_{i,max}} R_i dw_i \qquad \textbf{(A.4.5)}$$

As the load level is not known a priori an incremental approach is generally required.

Often the object can be assumed to be infinitely rigid (e.g axial impact from pipes and massive objects) so that all energy is to be dissipated by the impacted component..

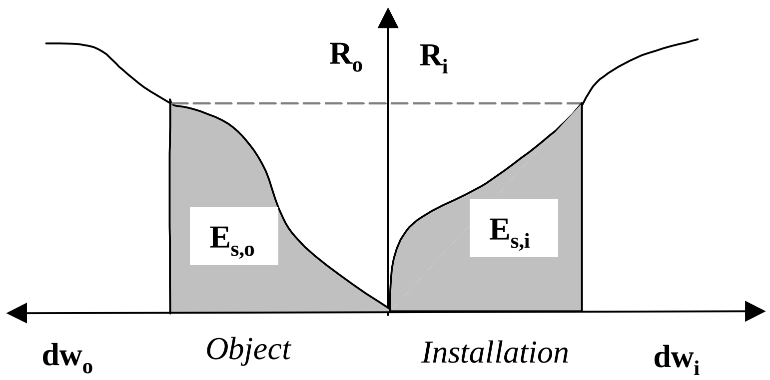


Figure A.4-2 Dissipation of strain energy in dropped object and installation

If the object is assumed to be deformable, the interactive nature of the deformation of the two structures should be recognised.

A.4.4 Resistance/energy dissipation

A.4.4.1 Stiffened plates subjected to drill collar impact

The energy dissipated in the plating subjected to drill collar impact is given by

$$E_{sp} = \frac{R^2}{K}\left(1 + 0.48\frac{m_i}{m}\right)^2 \qquad \textbf{(A.4.6)}$$

$$K = \frac{1}{2}\pi f_y t\left(\frac{1 + 5\frac{d}{r} - 6c^2 + 6.25\left(\frac{d}{r}\right)^2}{(1+c)^2}\right)$$: stiffness of plate enclosed by hinge circle

f_y = characteristic yield strength

$c = -e^{-2.5\left(1-\frac{d}{r}\right)}$

$R = \pi d t \tau$ = contact force for $\tau \le \tau_{cr}$ refer Section A.4.5.1 for τ_{cr}

$m_i = \rho_p \pi r^2 t$ = mass of plate enclosed by hinge circle

m = mass of dropped object

ρp = mass density of steel plate

d = smaller diameter at threaded end of drill collar

r = smaller distance from the point of impact to the plate boundary defined by adjacent stiffeners/girders, refer Figure A.4-3.

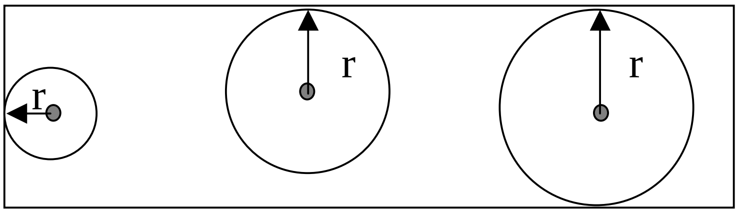


Figure A.4-3 Definition of distance to plate boundary

A.4.4.2 Stiffeners/girders

In lieu of more accurate calculations stiffeners and girders subjected to impact with blunt objects may be analysed with resistance models given in Section A.6.9.

A.4.4.3 Dropped object

Calculation of energy dissipation in deformable dropped objects shall be based upon recognised methods for plastic analysis. It shall be documented that the collapse mechanisms assumed yield a realistic representation of the true deformation field.

A.4.5 Limits for energy dissipation

A.4.5.1 Pipes on plated structures

The maximum shear stress for plugging of plates due to drill collar impacts may be taken as

$$\tau_{cr} = f_y\left(0.42 + 0.41\frac{t}{d}\right) \qquad \textbf{(A.4.7)}$$

A.4.5.2 Blunt objects

For stability of cross-sections and tensile fracture, refer Section A.3.10.

A.5 FIRE

A.5.1 General

The characteristic fire structural action is temperature rise in exposed members. The temporal and spatial variation of temperature depends on the fire intensity, whether or not the structural members are fully or partly engulfed by the flame and to what extent the members are insulated.

Structural steel expands at elevated temperatures and internal stresses are developed in redundant structures. These stresses are most often of moderate significance with respect to global integrity. The heating causes also progressive loss of strength and stiffness and is, in redundant structures, accompanied by redistribution of forces from members with low strength to members that retain their load bearing capacity. A substantial loss of load-bearing capacity of individual members and subassemblies may take place, but the load bearing function of the installation shall remain intact during exposure to the fire action.

In addition, the residual strength requirements given in Section A.7 shall be complied with.

Structural analysis may be performed on either

- individual members
- subassemblies
- entire system

The assessment of fire load effect and mechanical response shall be based on either

- simple calculation methods applied to individual members,
- general calculation methods,

or a combination.

Simple calculation methods may give overly conservative results. General calculation methods are methods in which engineering principles are applied in a realistic manner to specific applications.

Assessment of individual members by means of simple calculation methods should be based upon the provisions given in Eurocode 3 Part 1.2. /2/.

Assessment by means of general calculation methods shall satisfy the provisions given in Eurocode 3 Part1.2 , Section 4.3.

In addition, the requirements given in this section for mechanical response analysis with nonlinear finite element methods shall be complied with.

Assessment of ultimate strength is not needed if the maximum steel temperature is below

400 ^{o}C, but deformation criteria may have to be checked for impairment of main safety functions.

A.5.2 General calculation methods

Structural analysis methods for non-linear, ultimate strength assessment may be classified as

- stress-strain based methods
- stress-resultants based (yield/plastic hinge) methods

Stress-strain based methods are methods where non-linear material behaviour is accounted for on fibre level.

Stress-resultants based methods are methods where non-linear material behaviour is accounted for on stress-resultants level based upon closed form solutions/interaction equation for cross-sectional forces and moments.

A.5.3 Material modelling

In stress-strain based analysis temperature dependent stress-strain relationships given in Eurocode 3, Part 1.2 , Section 3.2 may be used.

For stress resultants based design the temperature reduction of the elastic modulus may be taken as $k_{E,\theta}$ according to Eurocode 3 . The yield stress may be taken equal to the effective yield stress, $f_{y,\theta}$, . The temperature reduction of the effective yield stress may be taken as $k_{y,\theta}$.

Provided that above the above requirements are complied with creep does need explicit consideration.

A.5.4 Equivalent imperfections

To account for the effect of residual stresses and lateral distortions compressive members shall be modelled with an initial, sinusoidal imperfection with amplitude given by

Elasto-plastic material models

$$\frac{e^*}{\ell} = \frac{1}{\pi}\sqrt{\frac{\sigma_Y}{E}}\frac{i}{z_0}\alpha$$

Elastic-plastic material models:

$$\frac{e^*}{\ell} = \frac{W_p}{W}\frac{1}{\pi}\sqrt{\frac{\sigma_Y}{E}}\frac{i}{z_0}\alpha \qquad = \frac{1}{\pi}\sqrt{\frac{\sigma_Y}{E}}\frac{W_P}{\sqrt{AI}}\alpha$$

α = 0.5 for fire exposed members according to column curve c, Eurocode 3
i = radius of gyration
z_0 = distance from neutral axis to extreme fibre of cross-section
W_P = plastic section modulus
W = elastic section modulus
A = cross-sectional area
I = moment of inertia
e^* = amplitude of initial distortion
ℓ = member length

The initial out-of-straightness should be applied on each physical member. If the member is modelled by several finite elements the initial out-of-straightness should be applied as displaced nodes.

The initial out-of-straightness shall be applied in the same direction as the deformations caused by the temperature gradients.

A.5.5 Empirical correction factor

The empirical correction factor of 1.2 has to be accounted for in calculating the critical strength in compression and bending for design according to Eurocode 3, refer Comm. A.5.5.

A.5.6 Local cross sectional buckling

If shell modelling is used, it shall be verified that the software and the modelling is capable of predicting local buckling with sufficient accuracy. If necessary, local shell imperfections have to be introduced in a similar manner to the approach adopted for lateral distortion of beams

If beam modelling is used local cross-sectional buckling shall be given explicit consideration.

In lieu of more accurate analysis cross-sections subjected to plastic deformations shall satisfy compactness requirements given in Eurocode 3 :

- class 1 : Locations with plastic hinges (approximately full plastic utilization)
- class 2 : Locations with yield hinges (partial plastification)

If this criterion is not complied with explicit considerations shall be performed. The load-bearing capacity will be reduced significantly after the onset of buckling, but may still be significant. A conservative approach is to remove the member from further analysis.

Compactness requirements for class 1 and class 2 cross-sections may be disregarded provided that the member is capable of developing significant membrane forces.

A.5.7 Ductility limits

A.5.7.1 General

The ductility of beams and connections increase at elevated temperatures compared to normal conditions. Little information exists.

A.5.7.2 Beams in bending

In lieu of more accurate analysis requirements given in Section A.3.10 are to be complied with.

A.5.7.3 Beams in tension

In lieu of more accurate analysis an average elongation of 3% of the member length (ISO) with a reasonably uniform temperature can be assumed.

Local temperature peaks may localise plastic strains. The maximum local strain shall therefor not exceed ε_a = 15%.

A.5.8 Capacity of connections

In lieu of more accurate calculations the capacity of the connection can be taken as:

$$R_\theta = k_{y,\theta} R_0$$

where

R_0 = capacity of connection at normal temperature

$k_{y,\theta}$ = temperature reduction of effective yield stress for maximum temperature in connection

A.6 EXPLOSIONS

A.6.1 General

Explosion loads are characterised by temporal and spatial pressure distribution. The most important temporal parameters are rise time, maximum pressure and pulse duration.

For components and sub-structures the explosion pressure shall normally be considered uniformly distributed. On global level the spatial distribution is normally nonuniform both with respect to pressure and duration.

The response to explosion loads may either be determined by non-linear dynamic finite element analysis or by simple calculation models based on Single Degree Of Freedom (SDOF) analogies and elastic-plastic methods of analysis.

If non-linear dynamic finite element analysis is applied all effects described in the following paragraphs shall either be implicitly covered by the modelling adopted or subjected to special considerations, whenever relevant

In the simple calculation models the component is transformed to a single spring-mass system exposed to an equivalent load pulse by means of suitable shape functions for the displacements in the elastic and elastic-plastic range. The shape functions allow calculation of the characteristic resistance curve and equivalent mass in the elastic and elastic-plastic range as well as the fundamental period of vibration for the SDOF system in the elastic range.

Provided that the temporal variation of the pressure can be assumed to be triangular, the maximum displacement of the component can be calculated from design charts for the SDOF system as a function of pressure duration versus fundamental period of vibration and equivalent load amplitude versus maximum resistance in the elastic range. The maximum displacement must comply with ductiliy and stability requirements for the component.

The load bearing function of the installation shall remain intact with the damages imposed by the explosion loads. In addition, the residual strength requirements given in Section A.7 shall be complied with.

A.6.2 Classification of response

The response of structural components can conveniently be classified into three categories according to the duration of the explosion pressure pulse, t_d, relative to the fundamental period of vibration of the component, T:

- Impulsive domain - $t_d/T < 0.3$
- Dynamic domain - $0.3 < t_d/T < 3$
- Quasi-static domain - $3 < t_d/T$

Impulsive domain:

The response is governed by the impulse defined by

$$I = \int_0^{t_d} F(t)\,dt \qquad \textbf{(A.6.1)}$$

Hence, the structure may resist a very high peak pressure provided that the duration is sufficiently small. The maximum deformation, w_{max}, of the component can be calculated iteratively from the equation

$$I = \sqrt{2m_{eq}\int_0^{w_{max}} R(w)dw} \qquad \textbf{(A.6.2)}$$

where

$R(w)$ = force-deformation relationship for the component
m_{eq} = equivalent mass for the component

Quasi-static-domain:

The response is governed by the peak pressure and the rise time of the pressure relative to the fundamental period of vibration. If the rise time is small the maximum deformation of the component can be solved iteratively from the equation:

$$w_{max} = \frac{1}{F_{max}}\int_0^{w_{max}} R(w)dw \qquad \textbf{(A.6.3)}$$

If the rise time is large the maximum deformation can be solved from the static condition

$$F_{max} = R(w_{max}) \qquad \textbf{(A.6.4)}$$

Dynamic domain:

The response has to be solved from numerical integration of the dynamic equations of equilibrium.

A.6.3 Failure modes for stiffened panels

Various failure modes for a stiffened panel are illustrated in Figure A.6-1. Suggested analysis model and reference to applicable resistance functions are listed in Table A.6-1. Application of a Single Degree of Freedom (SDOF) model in the analysis of stiffeners/girders with effective flange is implicitly based on the assumption that dynamic inter action between the plate flange and the profile can be neglected.

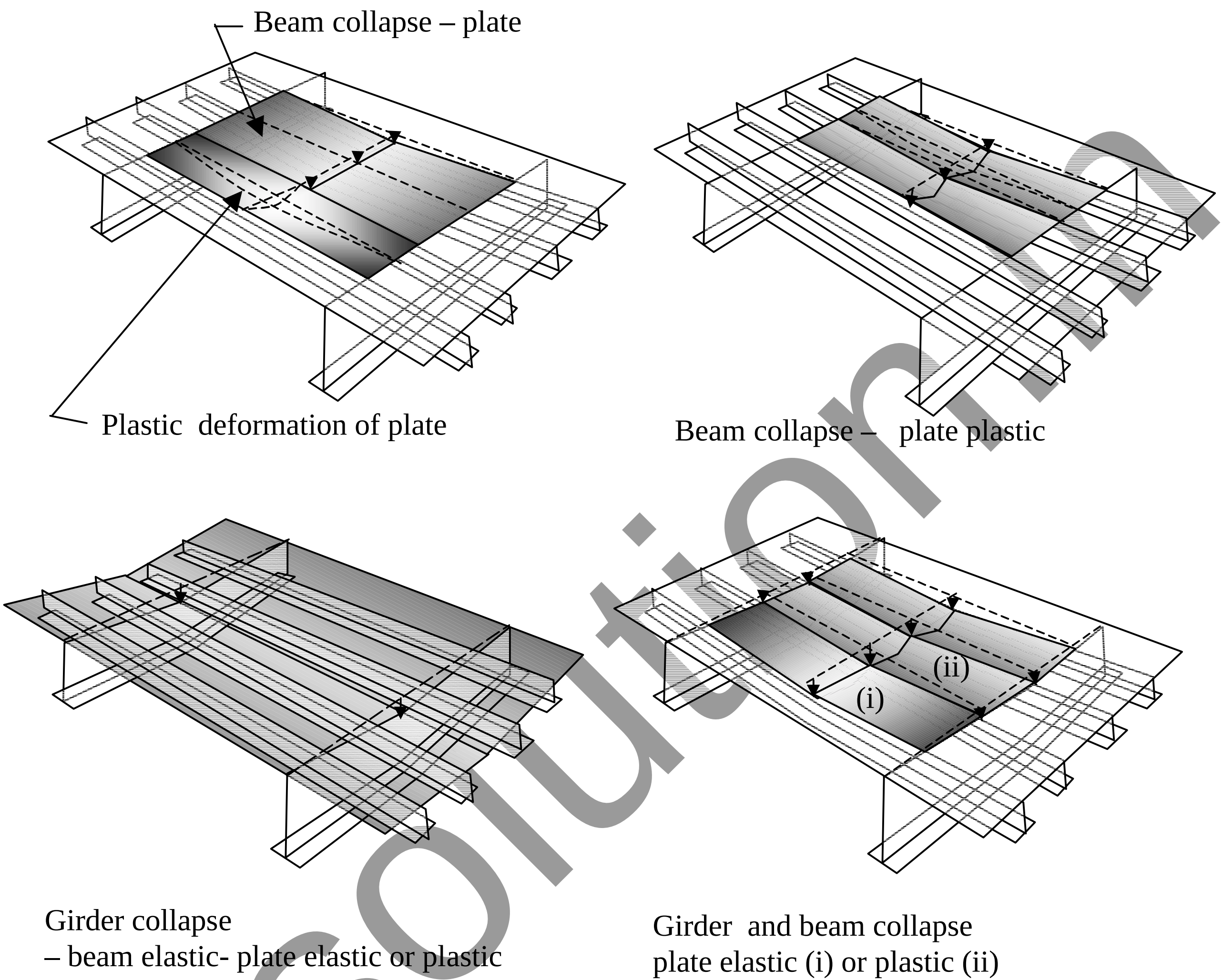


Figure A.6-1 Failure modes for two-way stiffened panel

Table A.6-1 Analysis models

Failure mode	Simplified analysis model	Resistance models	Comment
Elastic-plastic deformation of plate	SDOF	Section A.6.8	
Stiffener plastic – plate elastic	SDOF	Stiffener: Section A.6.9.1-2. Plate: Section A.6.8.1	Elastic, effective flange of plate
Stiffener plastic – plate plastic	SDOF	Stiffener: Section A.6.9.1-2. Plate: Section A.6.8	Effective width of plate at mid span. Elastic, effective flange of plate at ends.
Girder plastic – stiffener and plating elastic	SDOF	Girder: Section .A.6.9.1-2 Plate: Section A.6.8	Elastic, effective flange of plate with concentrated loads (stiffener reactions). Stiffener mass included.
Girder plastic – stiffener elastic – plate plastic	SDOF	Girder: Section A.6.9.1-2 Plate: Section A.6.8	Effective width of plate at girder mid span and ends. Stiffener mass included
Girder and stiffener plastic – plate elastic	MDOF	Girder and stiffener: Section A.6.9.1-2 Plate: Section A.6.8	Dynamic reactions of stiffeners → loading for girders
Girder and stiffener plastic – plate plastic	MDOF	Girder and stiffener: Section A.6.9.1-2 Plate: Section A.6.8	Dynamic reactions of stiffeners → loading for girders

By girder/stiffener ***plastic*** is understood that the maximum displacement w_{max} exceeds the elastic limit w_{el}

A.6.4 SDOF system analogy

Biggs method:

For many practical design problems it is convenient to assume that the structure - exposed to the dynamic pressure pulse - ultimately attains a deformed configuration comparable to the static deformation pattern. Using the static deformation pattern as displacement shape function, i.e.

$$w(x,t) = \phi(x)y(t)$$

the dynamic equations of equilibrium can be transformed to an equivalent single degree of freedom system:

$$\bar{m}\ddot{y} + \bar{k}y = \bar{f}(t) \qquad \textbf{(A.6.5)}$$

$\phi(x)$ = displacement shape function

$y(t)$ = displacement amplitude

$\bar{m} = \int_\ell m\phi(x)^2 dx + \sum_i M_i \phi_i^{\,2}$ = generalized mass

$\bar{f}(t) = \int_\ell p(t)\phi(x)dx + \sum_i F_i \phi_i$ = generalized load

$\bar{k} = \int_\ell EI\phi_{,xx}(x)^2 dx$ = generalized elastic bending stiffness

$\bar{k} = 0$ = generalized plastic bending stiffness (fully developed mechanism)

$\bar{k} = \int_\ell N\phi_{,x}(x)^2 dx$ = generalized membrane stiffness (fully plastic: $N = N_P$)

m = distributed mass

M_i = concentrated mass
p = explosion pressure
F_i = concentrated load (e.g. support reactions)
x_i = position of concentrated mass/load
$\varphi_i = \varphi(x = x_i)$

The equilibrium equation can alternatively be expressed as:

$$k_{lm} M\ddot{y} + Ky = F(t) \qquad \textbf{(A.6.6)}$$

where

$k_{lm} = \dfrac{k_m}{k_l}$ = load-mass transformation factor

$k_m = \dfrac{\overline{m}}{M}$ = mass transformation factor

$k_l = \dfrac{\bar{f}}{F}$ = load transformation factor

$M = \int_\ell m dx + \sum_i M_i$ = total mass

$F = \int_\ell p dx + \sum_i F_i$ = total load

$K = \dfrac{\bar{k}}{k_m}$ = characteristic stiffness

The natural period of vibration for the equivalent system is given by

$$T = 2\pi\sqrt{\frac{\overline{m}}{\bar{k}}} = 2\pi\sqrt{\frac{k_{lm}M}{K}}$$

The response , y(t), is - in addition to the load history - entirely governed by the total mass, load-mass factor and the characteristic stiffness.

For a *linear system*, the load mass factor and the characteristic stiffness are constant. The response is then alternatively governed by the eigenperiod and the characteristic stiffness.

For a *non-linear system*, the load-mass factor and the characteristic stiffness depend on the response (deformations). Non-linear systems may often conveniently be approximated by equivalent bi-linear or tri-linear systems, see Section A.6.7. In such cases the response can be expressed in terms of (see Figure A.6-10 for definitions):

K_1 = characteristic stiffness in the initial, linear resistance domain
y_{el} = displacement at the end of the initial, linear resistance domain
T = eigenperiod in the initial, linear resistance domain

and, if relevant,

K_3 = normalised characteristic resistance in the third linear resistance domain.

Characteristic stiffnesses are given explicitly or can be derived from load-deformation relationships given in Section A.6.9. If the response is governed by different mechanical behaviour relevant characteristic stiffnesses must be calculated.

For a given explosion load history the maximum displacement, y_{max}, is found by analytical or numerical integration of equation (A.6.6).

For standard load histories and standard resistance curves maximum displacements can be presented in design charts. Figure A.6-2 shows the normalised maximum displacement of a SDOF system with a bi- (K_3=0) or trilinear (K_3 > 0) resistance function, exposed to a triangular pressure pulse with zero rise time. When the duration of the pressure pulse relative to the eigenperiod in the initial, linear resistance range is known, the maximum displacement can be determined directly from the diagram as illustrated in Figure A.6-2.

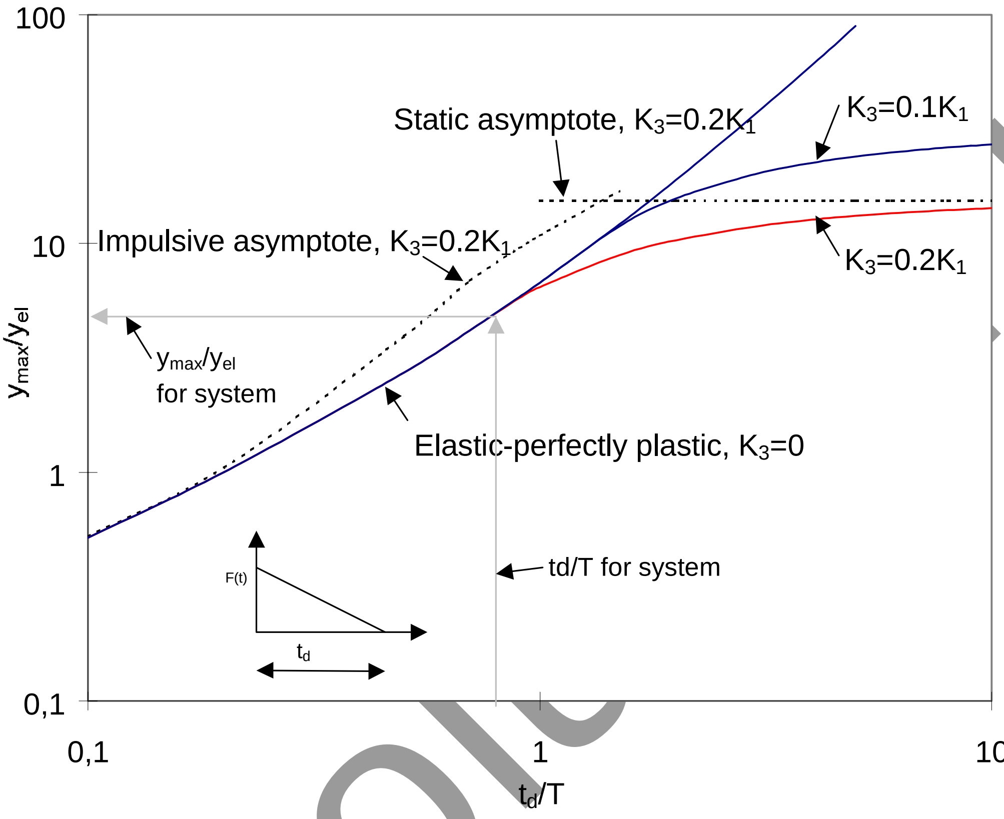


Figure A.6-2 Maximum response a SDOF system to a triangular pressure pulse with zero rise time. F_{max}/R_{el}= 2

Design charts for systems with bi- or tri-linear resistance curves subjected to a triangular pressure pulse with different rise time are given in Figures A.6-4 - A6-7.

Baker's method

The governing equations (A.6.1) and (A.6.2) for the maximum response in the impulsive domain and the quasi-static domain may also be used along with response charts for maximum displacement for different F_{max}/R_{el} ratios to produce pressure-impulse (F_{max},I) diagrams - iso-damage curves - provided that the maximum pressure is known. Figure A.6-3 shows such a relationship obtained for an elastic-perfectly plastic system when the maximum dynamic response is y_{max}/y_{el}=10.

Pressure-impulse combinations to the left and below of the iso-damage curve represent admissible events, to the right and above inadmissible events.

The advantage of using iso-damage diagrams is that "back-ward" calculations are possible:
The diagram is established on the basis of the resistance curve. The information may be used to

screen explosion pressure histories and eliminate those that obviously lie in the admissible domain. This will reduce the need for large complex simulation of explosion scenaries.

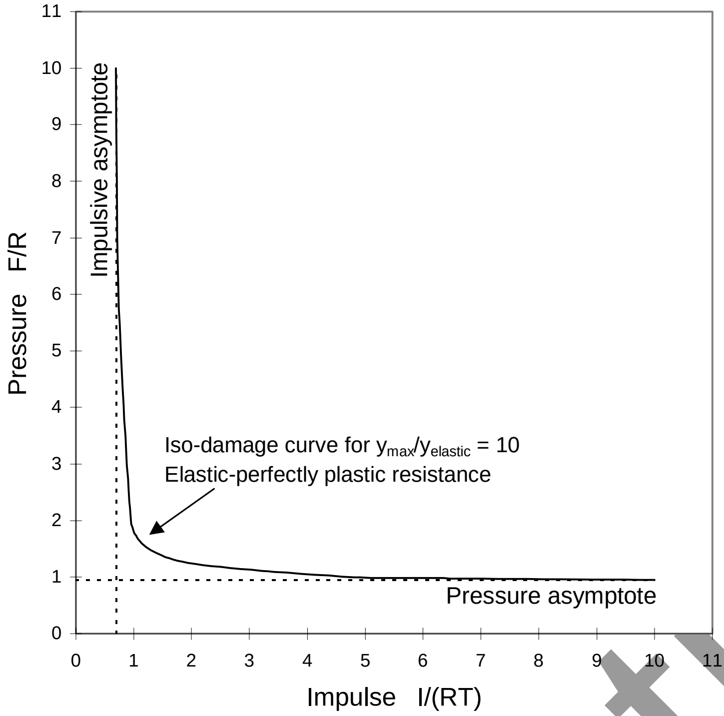


Figure A.6-3 Iso-damage curve for ymax/yel = 10. Triangular pressure pulse.

A.6.4.1 Dynamic response charts for SDOF system

Transformation factors for elastic–plastic-membrane deformation of beams and one-way slabs with different boundary conditions are given in Table A.6-2

Maximum displacement for a SDOF system exposed to different pressure histories are displayed in Figures A.6-4 - A.6-7.

The characteristic response of the system is based upon the resistance in the linear range, $K=K_{1,}$ where the equivalent stiffness is determined from the elastic solution to the actual system.

If the displacement shape function changes as a non-linear structure undergoes deformation the transformation factors change. In lieu of accurate analysis an average value of the combined load-mass transformation factor can be used:.

$$k_{lm}^{average} = \frac{k_{lm}^{elastic} + (\mu - 1)k_{lm}^{plastic}}{\mu} \qquad \textbf{(A.6.7)}$$

$\mu \quad = \quad y_{max}/y_{el} \quad$ ductility ratio

Since μ is not known a priori iterative calculations may be necessary.

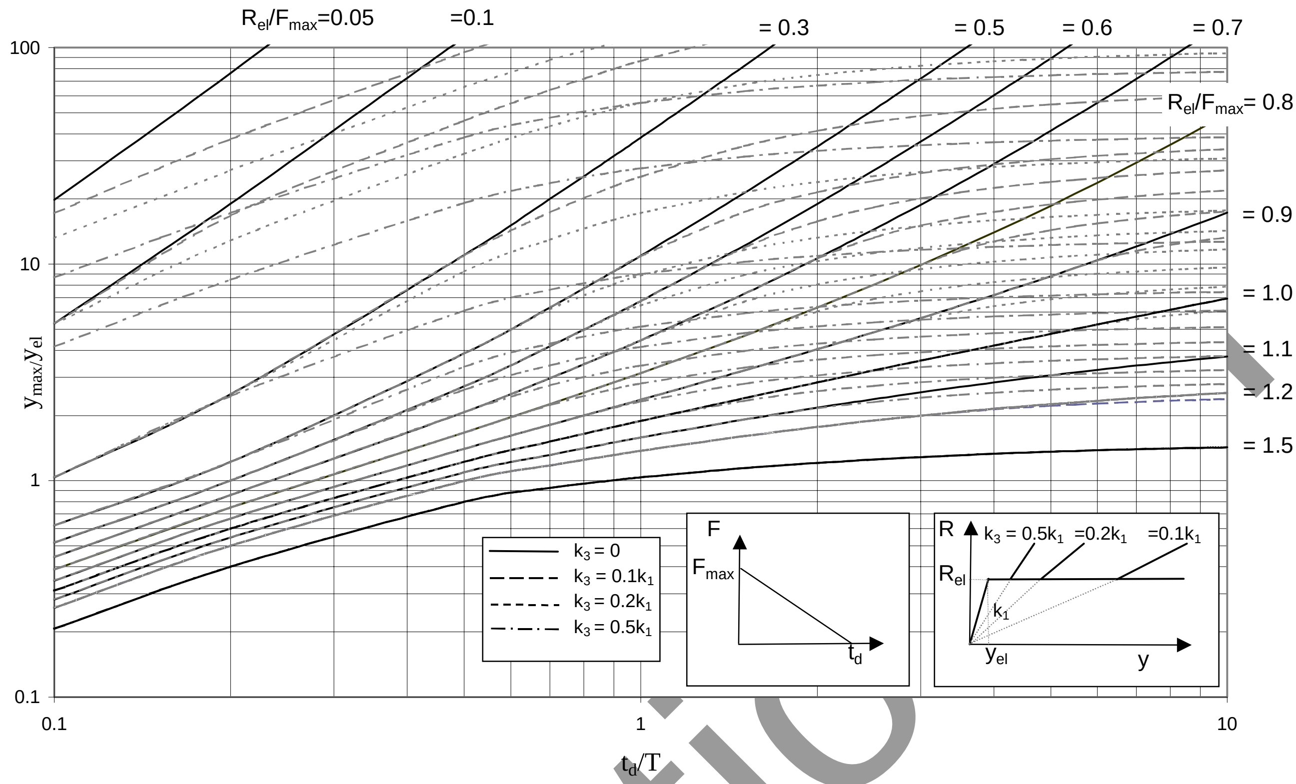


Figure A.6-4 Dynamic response of a SDOF system to a triangular load (rise time=0)

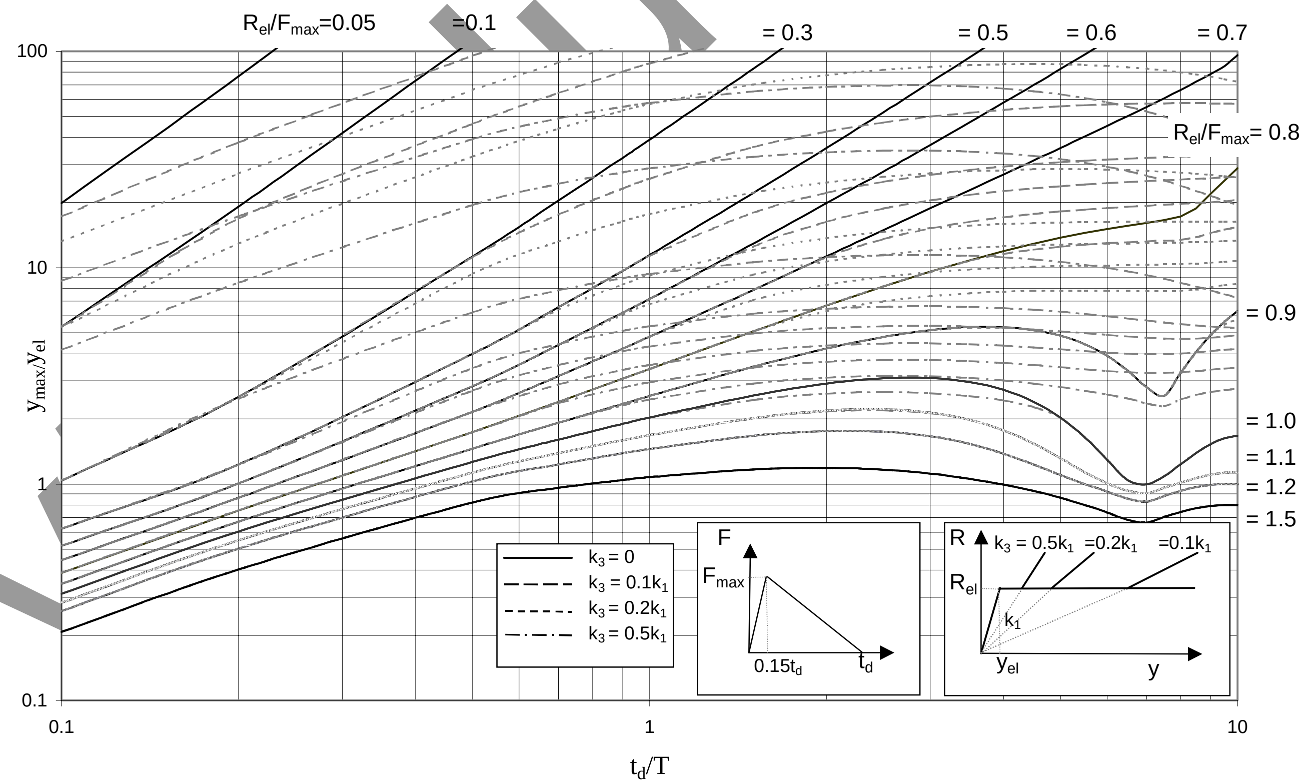


Figure A.6-5 Dynamic response of a SDOF system to a triangular load (rise time=0.15t_d)

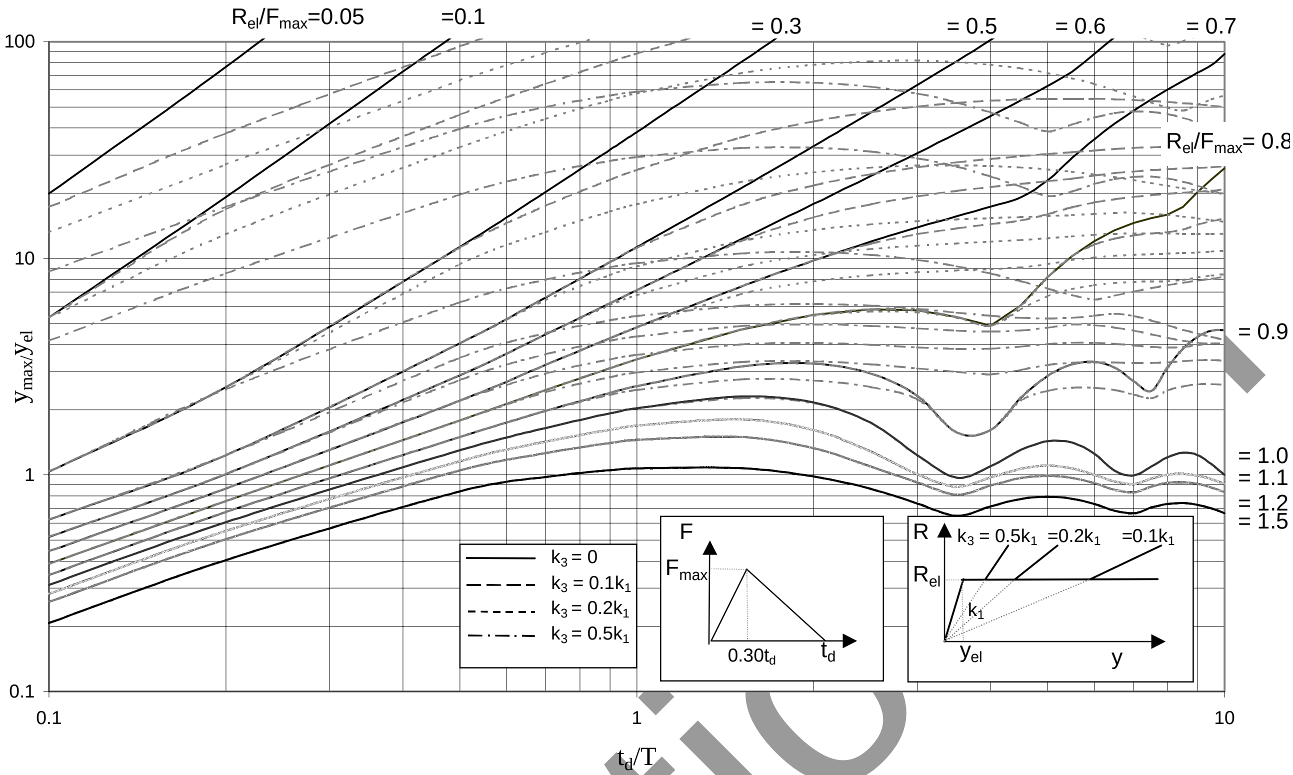


Figure A.6-6 Dynamic response of a SDOF system to a triangular load (rise time=$0.30t_d$)

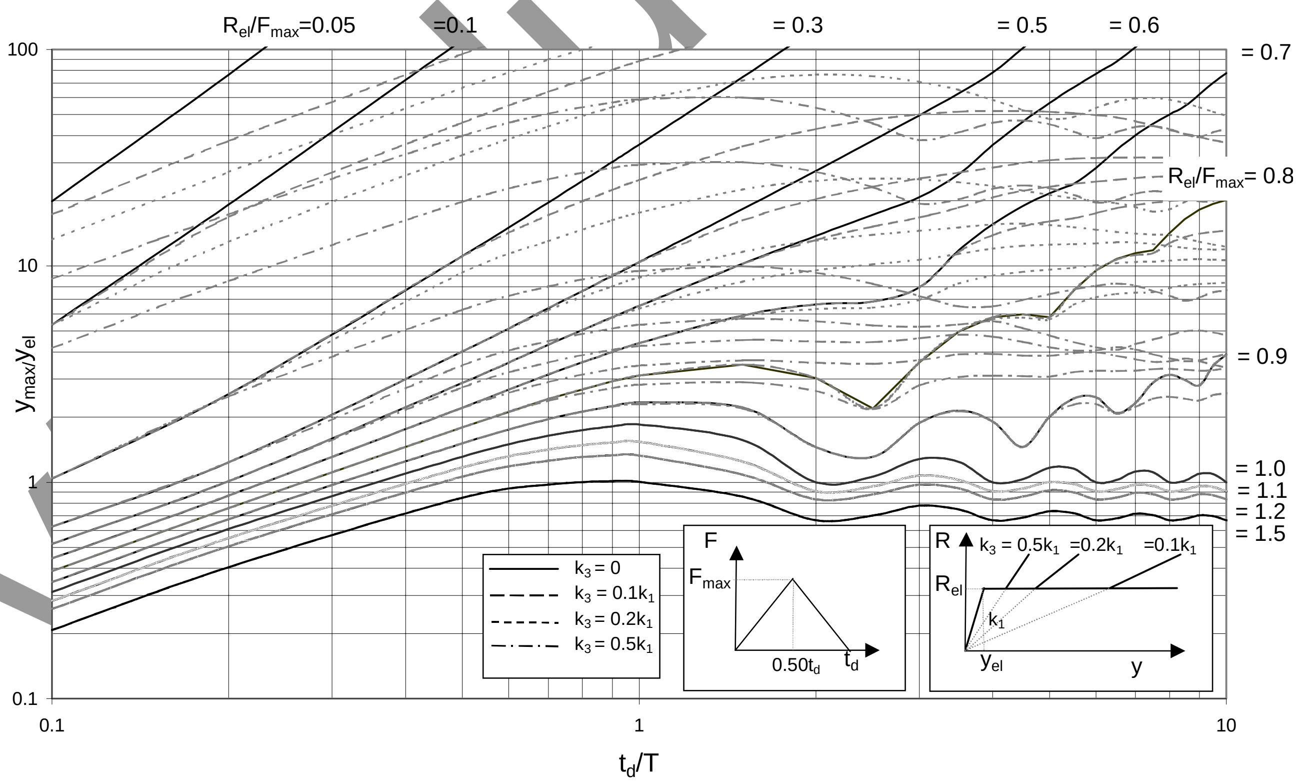


Figure A.6-7 Dynamic response of a SDOF system to a triangular load (rise time=$0.50t_d$)

A.6.5 MDOF analysis

SDOF analysis of built-up structures (e.g. stiffeners supported by girders) is admissible if

- the fundamental periods of elastic vibration are sufficiently separated
- the response of the component with the smallest eigenperiod does not enter the elastic-plastic domain so that the period is drastically increased

If these conditions are not met, then significant interaction between the different vibration modes is anticipated and a multi degree of freedom analysis is required with simultaneous time integration of the coupled system.

A.6.6 Classification of resistance properties

A.6.6.1 Cross-sectional behaviour

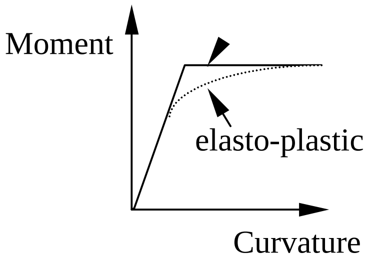


Figure A.6-8 Bending moment-curvature relationships

Elasto-plastic : The effect of partial yielding on bending moment accounted for

Elastic-perfectly plastic: Linear elastic up to fully plastic bending moment

The simple models described herein assume elastic-perfectly plastic material behaviour. Any strain hardening may be accounted for by equivalent (increased) yield stress.

A.6.6.2 Component behaviour

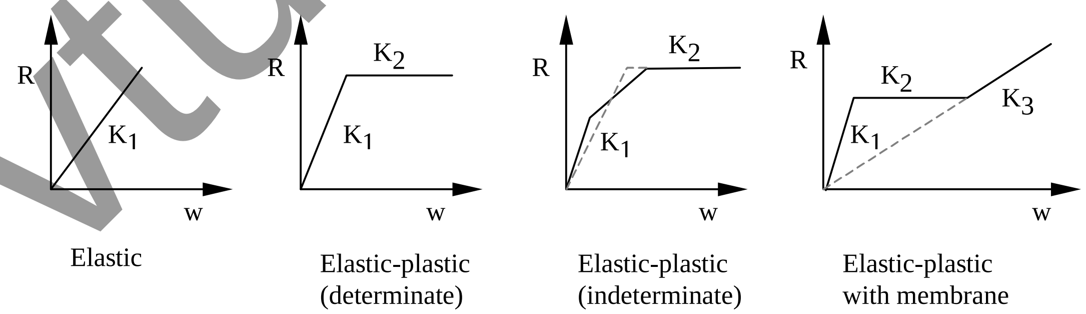


Figure A.6-9 Resistance curves

Elastic: Elastic material, small deformations

Elastic-plastic (determinate): Elastic-perfectly plastic material. Statically determinate system. Bending mechanism fully developed with occurrence of first plastic hinge(s)/yield lines. No axial restraint

Elastic-plastic (indeterminate): Elastic perfectly plastic material. Statically indeterminate system. Bending mechanism develops with sequential formation of plastic hinges/yield lines. No axial restraint. For simplified analysis this system may be modelled as an elastic-plastic (determinate) system with equivalent initial stiffness.

Elastic-plastic with membrane: Elastic-perfectly plastic material. Statically determinate (or indeterminate). Ends restrained against axial displacement. Increase in load-carrying capacity caused by development of membrane forces .

A.6.7 Idealisation of resistance curves

The resistance curves in clause A.6.6 are idealised. For simplified SDOF analysis the resistance characteristics of a real non-linear system may be approximately modelled. An example with a tri-linear approximation is illustrated in Figure A.6-10.

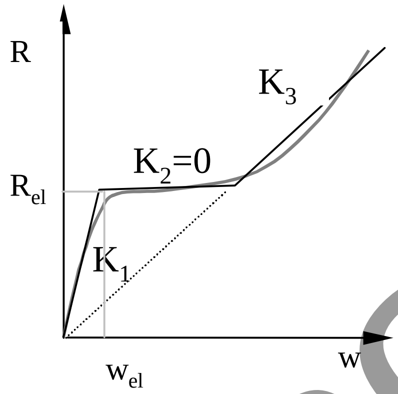


Figure A.6-10 Representation of a non-linear resistance by an equivalent tri-linear system.

In lieu of more accurate analysis the resistance curve of elastic-plastic systems may be composed by an elastic resistance and a rigid-plastic resistance as illustrated in Figure A.6-11.

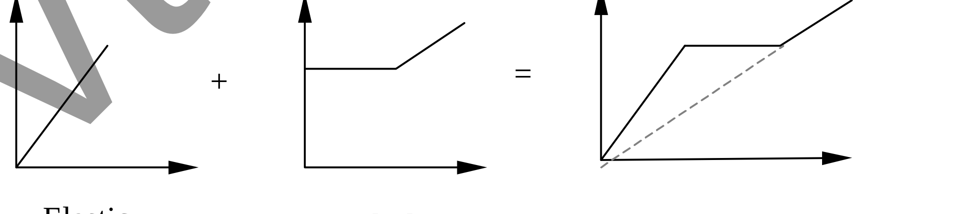


Figure A.6-11 Construction of elastic-plastic resistance curve

A.6.8 Resistance curves and transformation factors for plates

A.6.8.1 Elastic - rigid plastic relationships

In lieu of more accurate calculations rigid plastic theory combined with elastic theory may be used.

In the elastic range the stiffness and fundamental period of vibration of a clamped plate under uniform lateral pressure can be expressed as:

$p = Kw$ = pressure-displacement relationship for plate center

$K = \psi \frac{D}{s^4}$ = plate stiffness

$T = \frac{2\pi}{\eta}\sqrt{\frac{\rho t s^4}{D}}$ = natural period of vibration

$D = E\frac{t^3}{12(1-\nu^2)}$ = plate stiffness

The factors ψ and η are given in Figure A.6-12

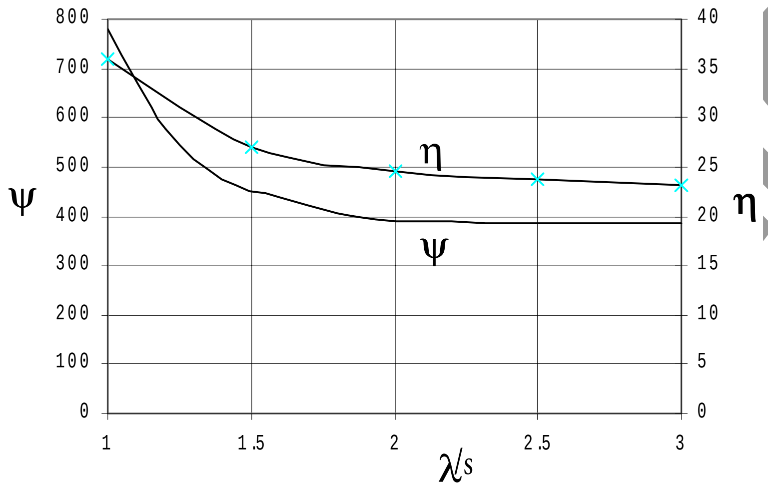


Figure A.6-12 Coefficients ψ and η.

The plastic load-carrying capacity of plates subjected to uniform pressure can be taken as:

$$\frac{p}{p_c} = 1 + \overline{w}^2\left(\frac{\alpha + (3-2\alpha)^2}{9-3\alpha}\right) \qquad \overline{w} \le 1 \tag{A.6.8}$$

$$\frac{p}{p_c} = 2\overline{w}\left(1 + \frac{\alpha(2-\alpha)}{3-\alpha}\left(\frac{1}{3\overline{w}^2} - 1\right)\right) \qquad \overline{w} \ge 1$$

Pinned ends :

$$\overline{w} = 2\frac{w}{t} \qquad p_c = \frac{6 f_y t^2}{\ell^2 \alpha^2}$$

Clamped ends :

$$\overline{w} = \frac{w}{t} \qquad p_c = \frac{12 f_y t^2}{\ell^2 \alpha^2}$$

$$\alpha = \frac{s}{\ell}\left(\sqrt{3 + (\frac{s}{\ell})^2} - \frac{s}{\ell}\right) = \quad \text{plate aspect parameter}$$

$\ell\,(>s)$ = plate length
s = plate width
t = plate thickness
p_c = plastic collapse pressure in bending for plates with no axial restraint

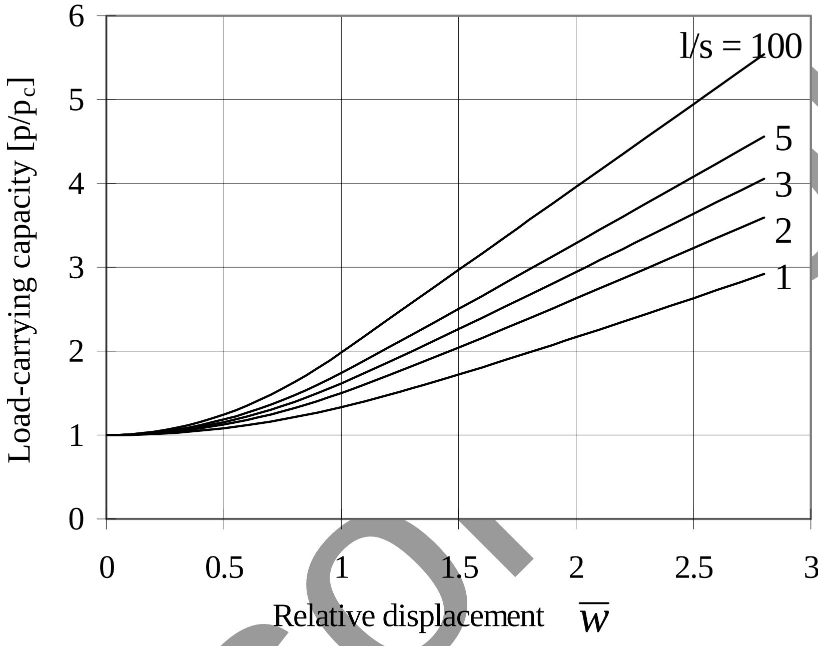


Figure A.6-13 Plastic load-carrying capacities of plates as a function of lateral displacement

A.6.8.2 Axial restraint

Unlike stiffeners no simple method with a clear physical interpretation exists to quantify the effect of flexible boundaries for a continuous plate field under uniform pressure. Full axial restraint may probably be assumed.

At the panel boundaries assumption of full axial restraint is non-conservative. In lieu of more accurate calculation the axial restraint may be estimated by removing the plate and apply a uniformly distributed unit in-plane load normal to the plate edges. The axial stiffness should be taken as the inverse of the maximum in-plane displacement of the long edge. The relative reduction of the plastic load-carrying capacity can calculated according to the procedure described in Section A.6.9.2 for a beam with rectangular cross-section (plate thickness x unit width) and length equal to stiffener spacing, using the diagram for α = 2.

For a plate in the middle of a continuous, uniformly loaded panel a high degree of axial restraint is likely. A non-dimensional spring factor c= 1.0 is suggested.

If membrane forces are likely to develop it has to be verified that the adjacent structure is strong enough to anchor fully plastic membrane forces.

A.6.8.3 Tensile fracture of yield hinges

In lieu of more accurate calculations the procedure described in Section A.3.10.4 may be used for a beam with rectangular cross-section (plate thickness x unit width) and length equal to stiffener spacing.

A.6.9 Resistance curves and transformation factors for beams

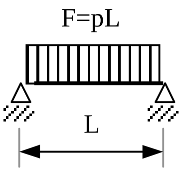
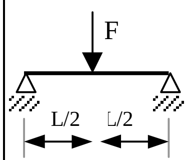
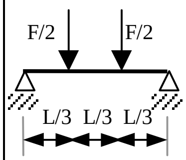
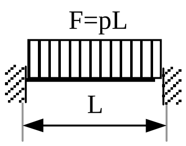
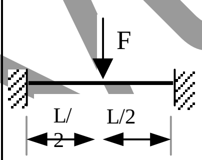
Provided that the stiffeners/girders remain stable against local buckling, tripping or lateral torsional buckling stiffened plates/girders may be treated as beams. Simple elastic-plastic methods of analysis are generally applicable. Special considerations shall be given to the effect of:

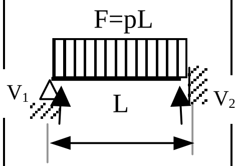
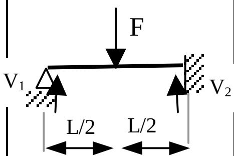
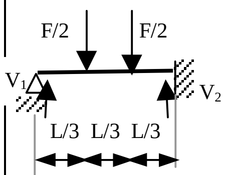
- Elastic flexibility of member/adjacent structure
- Local deformation of cross-section
- Local buckling
- Strength of connections
- Strength of adjacent structure
- Fracture

A.6.9.1 Beams with no- or full axial restraint

Equivalent springs and transformation factors for load and mass for various idealised elasto-plastic systems are shown in Table A.6-2.

Table A.6-2 Transformation factors for beams with various boundary and load conditions.

Load case	Resistance domain	Load Factor k_l	Mass factor k_m		Load-mass factor k_{lm}		Maximum resistance R_{el}	Characteristic stiffness K	Dynamic reaction V
			Concen-trated mass	Uniform mass	Concen-trated mass	Uniform mass			
	Elastic	0.64		0.50		0.78	$\frac{8M_p}{L}$	$\frac{384EI}{5L^3}$	$0.39R+0.11F$
	Plastic bending	0.50		0.33		0.66	$\frac{8M_p}{L}$	0	$0.38R_{el}+0.12F$
	Plastic membrane	0.50		0.33		0.66		$\frac{4N_P}{L}$	$\frac{2N_P y_{max}}{L}$
	Elastic	1.0	1.0	0.49	1.0	0.49	$\frac{4M_p}{L}$	$\frac{48EI}{L^3}$	$0.78R-0.28F$
	Plastic bending	1.0	1.0	0.33	1.0	0.33	$\frac{4M_p}{L}$	0	$0.75R_{el}-0.25F$
	Plastic membrane	1.0	1.0	0.33	1.0	0.33		$\frac{4N_P}{L}$	$\frac{2N_P y_{max}}{L}$
	Elastic	0.87	0.76	0.52	0.87	0.60	$\frac{6M_p}{L}$	$\frac{56.4EI}{L^3}$	$0.525R-0.025F$
	Plastic bending	1.0	1.0	0.56	1.0	0.56	$\frac{6M_p}{L}$	0	$0.52R_{el}-0.02F$
	Plastic membrane	1.0	1.0	0.56	1.0	0.56		$\frac{6N_P}{L}$	$\frac{3N_P y_{max}}{L}$
	Elastic	0.53		0.41		0.77	$\frac{12M_{ps}}{L}$	$\frac{384EI}{L^3}$	$0.36R+0.14F$
	Elasto-plastic bending	0.64		0.50		0.78	$\frac{8(M_{ps}+M_{Pm})}{L}$	$\frac{384EI}{5L^3}$ $\left(\frac{307EI}{L^3}\right)$	$0.39R_{el}+0.11F$
	Plastic bending	0.50		0.33		0.66	$\frac{8(M_{ps}+M_{Pm})}{L}$	0	$0.38R_{el}+0.12F$
	Plastic membrane	0.50		0.33		0.66		$\frac{4N_P}{L}$	$\frac{2N_p y_{max}}{L}$
	Elastic	1.0	1.0	0.37	1.0	0.37	$\frac{4(M_{ps}+M_{Pm})}{L}$	$\frac{192EI}{L^3}$	$0.71R-0.21F$
	Plastic bending	1.0	1.0	0.33	1.0	0.33	$\frac{4(M_{ps}+M_{Pm})}{L}$	0	$0.75R_{el}-0.25F$
	Plastic membrane	1.0	1.0	0.33	1.0	0.33		$\frac{4N_P}{L}$	$\frac{2N_P y_{max}}{L}$

Load case	Resistance domain	Load Factor k_l	Mass factor k_m: Concentrated mass	Mass factor k_m: Uniform mass	Load-mass factor k_{lm}: Concentrated mass	Load-mass factor k_{lm}: Uniform mass	Maximum resistance R_{el}	Characteristic stiffness K	Dynamic reaction V
	Elastic	0.58		0.45		0.78	$\frac{8M_{ps}}{L}$	$\frac{185EI}{L^3}$	$V_1 = 0.26R + 0.12F$ $V_2 = 0.43R + 0.19F$
	Elasto-plastic Bending	0.64		0.50		0.78	$\frac{4(M_{ps} + 2M_{Pm})}{L}$	$\frac{384EI}{5L^3}$ $\left(\frac{160EI}{L^3}\right)$	$0.39R + 0.11F \pm M_{Ps}/L$
	Plastic bending	0.50		0.33		0.66	$\frac{4(M_{ps} + 2M_{Pm})}{L}$	0	$0.38R + 0.12F \pm M_{Ps}/L$
	Plastic membrane	0.50		0.33		0.66		$\frac{4N_P}{L}$	$\frac{2N_P y_{max}}{L}$
	Elastic	1.0	1.0	0.43	1.0	0.43	$\frac{16M_{Ps}}{3L}$	$\frac{107EI}{L^3}$	$V_1 = 0.25R + 0.07F$ $V_2 = 0.54R + 0.14F$
	Elasto-plastic Bending	1.0	1.0	0.49	1.0	0.49	$\frac{2(M_{ps} + 2M_{Pm})}{L}$	$\frac{48EI}{L^3}$ $\left(\frac{106EI}{L^3}\right)$	$0.78R - 0.28F \pm M_{Ps}/L$
	Plastic bending	1.0	1.0	0.33	1.0	0.33	$\frac{2(M_{ps} + 2M_{Pm})}{L}$	0	$0.75R - 0.25F \pm M_{Ps}/L$
	Plastic membrane	1.0	1.0	0.33	1.0	0.33		$\frac{4N_P}{L}$	$\frac{2N_P y_{max}}{L}$
	Elastic	0.81	0.67	0.45	0.83	0.55	$\frac{6M_{Ps}}{L}$	$\frac{132EI}{L^3}$	$V_1 = 0.17R + 0.17F$ $V_2 = 0.33R + 0.33F$
	Elasto-plastic Bending	0.87	0.76	0.52	0.87	0.60	$\frac{2(M_{ps} + 3M_{Pm})}{L}$	$\frac{56EI}{L^3}$ $\left(\frac{122EI}{L^3}\right)$	$0.525R - 0.025F \pm M_{Ps}/L$
	Plastic bending	1.0	1.0	0.56	1.0	0.56	$\frac{2(M_{ps} + 3M_{Pm})}{L}$	0	$0.52R_{el} - 0.02F \pm M_{Ps}/L$
	Plastic membrane	1.0	1.0	0.56	1.0	0.56		$\frac{6N_P}{L}$	$\frac{3N_P y_{max}}{L}$

A.6.9.2 Beams with partial end restraint.

Relatively small axial displacements have a significant influence on the development of tensile forces in members undergoing large lateral deformations. An equivalent elastic, axial stiffness may be defined as

$$\frac{1}{K} = \frac{1}{K_{node}} + \frac{\ell}{2EA} \qquad \textbf{(A.6.9)}$$

k_{node} = axial stiffness of the node with the considered member removed. This may be determined by introducing unit loads in member axis direction at the end nodes with the member removed.

Plastic force-deformation relationship for a beam under uniform pressure may obtained from Figure A.6-14, Figure A.6-15 or Figure A.6-16 if the plastic interaction between axial force and bending moment can be approximated by the following equation:

$$\frac{M}{M_P} + \left(\frac{N}{N_P}\right)^{\alpha} = 1 \qquad 1 < \alpha < 2 \qquad \textbf{(A.6.10)}$$

In lieu of more accurate analysis α = 1.2 can be assumed for stiffened plates.

$R_0 = p\ell = \frac{8c_1M_P}{\ell}$ = plastic collapse load in bending for the member.

ℓ = member length

$\overline{w} = \frac{w}{c_1 w_c}$ = non-dimensional deformation

$w_c = \frac{\alpha W_P}{A}$ = characteristic beam height for beams described by plastic interaction equation (A.6.10).

$W_P = z_g A_e$ = plastic section modulus for stiffened plate

$A = A_s + st$ = total area of stiffener and plate flange

$A_e = A_s + s_e t$ = effective cross-sectional area of stiffener and plate flange,

z_g = distance form plate flange to stiffener centre of gravity.

A_s = stiffener area

s = stiffener spacing

s_e = effective width of plate flange see A.6.9.3

z_g = distance from stiffener toe to centre of gravity of effective cross-section.

$c = \frac{4c_1 k w_c^2}{f_y A \ell}$ = non-dimensional spring stiffness

c_1 = 2 for clamped beams

c_1 = 1 for pinned beams

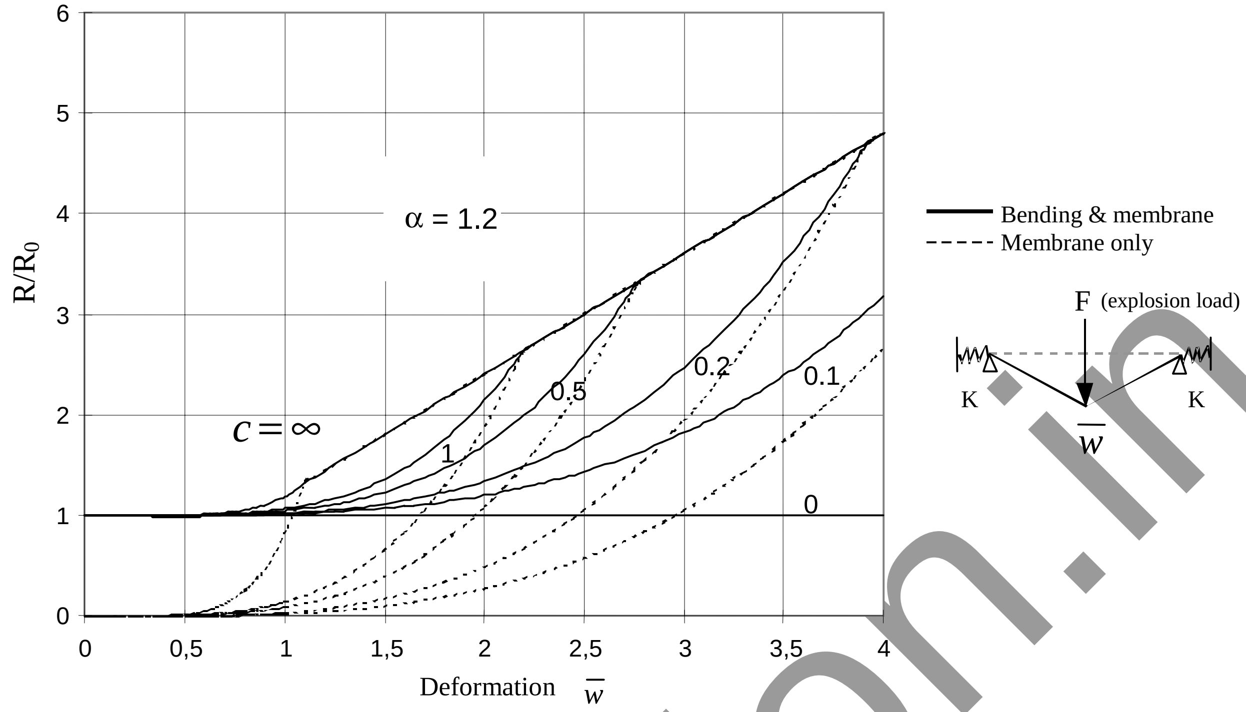


Figure A.6-14 Plastic load-deformation relationship for beam with axial flexibility (α=1.2)

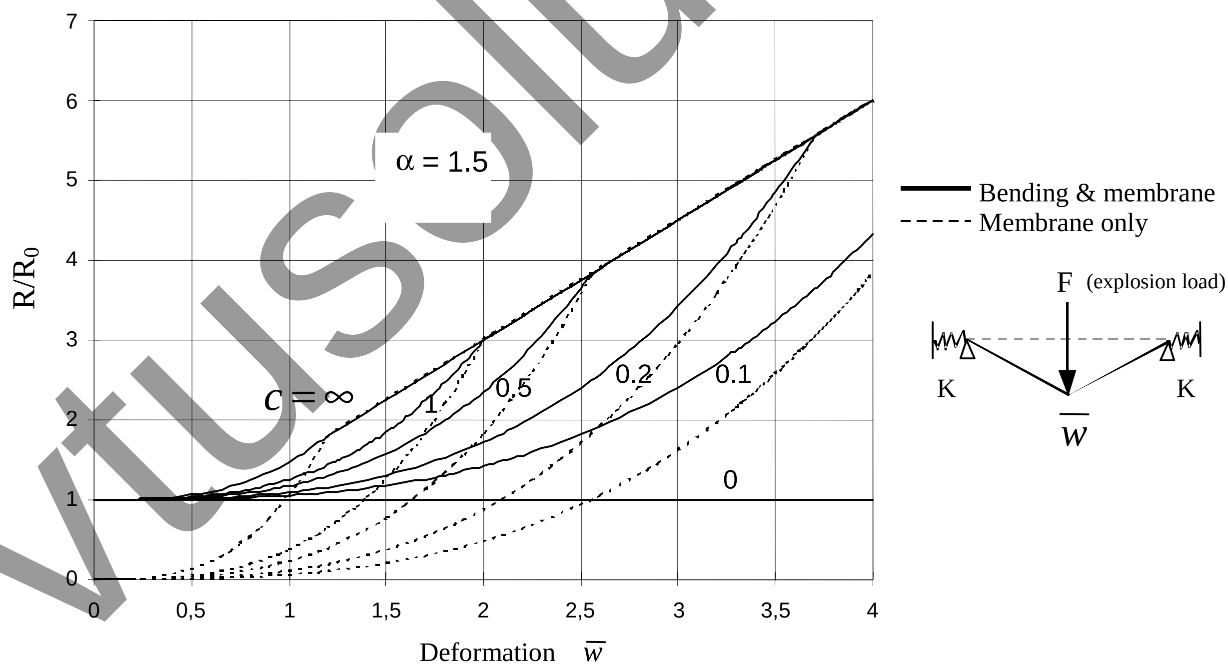


Figure A.6-15 Plastic load-deformation relationship for beam with axial flexibility (α=1.5)

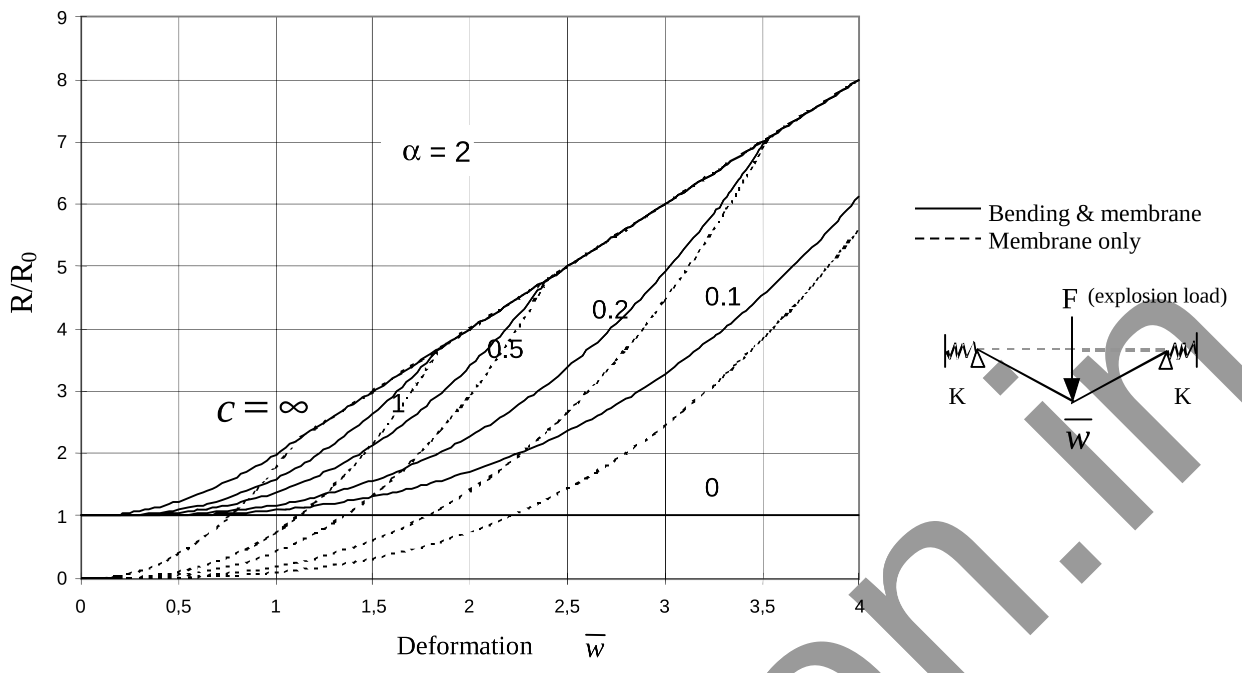


Figure A.6-16 Plastic load-deformation relationship for beam with axial flexibility (α=2)

For members where the plastic moment capacity of adjacent members is smaller than the moment capacity of the exposed member the force-deformation relationship may be interpolated from the curves for pinned ends and clamped ends:

$$R = \xi R_{clamped} + (1-\xi) R_{pinned} \quad \textbf{(A.6.11)}$$

where

$$0 \le \xi = \frac{R_0^{actual}}{8\frac{M_P}{\ell}} - 1 \le 1 \quad \textbf{(A.6.12)}$$

R_0^{actual} = Collapse load in bending for beam accounting for actual bending resistance of adjacent members

$$R_{actual} = \frac{8M_P + 4M_{P1} + 4M_{P2}}{\ell} \quad \textbf{(A.6.13)}$$

$$M_{Pj} = \sum_i M_{Pj,i} \le M_P \quad ; \text{ i = adjacent member no i, j = end number \{1,2\}} \quad \textbf{(A.6.14)}$$

$M_{Pj,i}$ = Plastic bending moment for member no. i.

Elastic, rotational flexibility of the node is normally of moderate significance

A.6.9.3 Effective flange

For deformations in the elastic range the effective width (shear lag effect) of the plate flange, s_e, of simply supported or clamped stiffeners/girders may be taken from Figure A.6-17

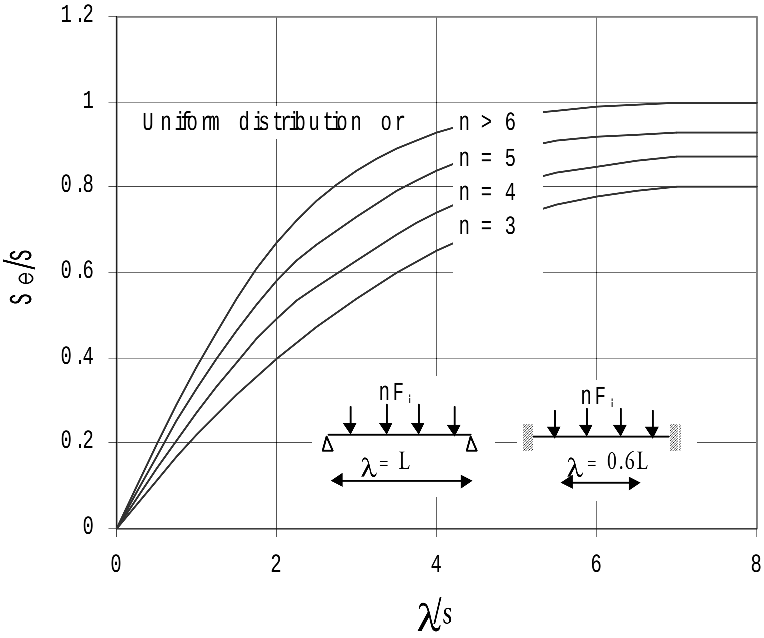


Figure A.6-17 Effective flange for stiffeners and girders in the elastic range

In the elasto-plastic and plastic range the effective width is to be reduced if the plate flange is on the compression side due to plate buckling:

- For stiffeners it is only necessary to reduce the plate flange for buckling at stiffener midsection.
- For girders with transversely stiffened plate flange reduction due to plate buckling is also to be considered at ends.

The effective plate flange is affected by explosion induced permanent deformations. Effective width of unstiffened plates with conventional imperfections is given in Section 6.6.4.3. In lieu of accurate calculations the effective width may be taken as

$$\frac{s_e}{s} = \frac{s_{ef}}{s}\left(1 - \frac{0.4}{\bar{\lambda}_p}\frac{w_{max}}{t}\right)$$

s_{ef} = effective width according to Section 6.6.4.3
$\bar{\lambda}_p$ = plate slenderness ratio, see Section 6.6.4.3
w_{max} = maximum lateral displacement of plate

The effective width for elastic deformations may be used when the plate flange is on the tension side.

A.6.9.4 Strength of adjacent structure

The adjacent structure must be checked to see whether it can provide the support required by the assumed collapse mechanism for the member/sub-structure

A.6.9.5 Strength of connections

The capacity of connections can be taken from recognised codes

The connection shall be checked for the dynamic reaction force given in Table A.6-2.

For beams with axial restraint the connection should also be checked for lateral - and axial reaction in the membrane phase:

If the axial force in a tension member exceeds the axial capacity of the connection the member must be assumed disconnected.

If the capacity of the connection is exceeded in compression and bending, this does not necessarily mean failure of the member. The post-collapse strength of the connection may be taken into account provided that such information is available.

A.6.9.6 Ductility limits

Reference is made to Section A.3.10

The local buckling criterion in Section A.3.10.2 and tensile fracture criterion given in Section A.3.10.3 may be used with:

d_c = characteristic dimension equal to twice the distance from the plastic neutral axis in bending to the extreme fibre of the cross-section

c = non-dimensional axial spring stiffness calculated in Section A.6.9.2

Alternatively, the ductility ratios $\mu = \frac{y_{max}}{y_{el}}$ in Table A.6-3 may be used.

Table A.6-3 Ductility ratios μ – beams with no axial restraint

Boundary conditions	**Load**	**Cross-section category**		
		Class 1	**Class 2**	**Class 3**
Cantilevered	Concentrated Distributed	6 7	4 5	2 2
Pinned	Concentrated Distributed	6 12	4 8	2 3
Fixed	Concentrated Distributed	6 4	4 3	2 2

A.7 RESIDUAL STRENGTH

A.7.1 General

The second step in the accidental limit state check is to verify the residual strength of the installation with damage caused by the accidental load.

The check shall be carried out for functional loads and design environmental loads.

The partial safety factor for loads and material can be taken equal to unity.

After the action of the accidental load an internal, residual stress/force field remains in the structure. The resultant of this field is zero. In most cases the residual stresses/forces have a minor influence on the residual capacity and may be neglected. Any detrimental effect of the residual stress on the ultimate capacity should, however, be subject to explicit consideration. If necessary, the residual stresses should be included in the analysis.

If non-linear FE analysis is used, the residual stress field can be conveniently included by performing integrated analysis:

1) Application of design accidental loading
2) Removal of accidental load by unloading
3) Application of design environmental load

A.7.2 Modelling of damaged members

A.7.2.1 General

Compressive members with large lateral deformations will often contribute little to load-carrying and can be omitted from analysis.

A.7.2.2 Members with dents, holes, out-of-straightness

Tubular members with dents, holes and out-of-straightness, reference is made to Chapter 10.

Stiffened plates/Beams/Girders with deformed plate flange and out-of-straightness, reference is made to Chapter 10.

A.8 REFERENCES

/1/ NORSOK Standard N-003 Action and Action Effect

/2/ NS-ENV 1993-1 Eurocode 3: Design of Steel structures Part 1-2. General rules - Structural fire design

/3/ Amdahl, J.: "Energy Absorption in Ship-Platform Impacts", UR-83-34, Dept. Marine Structures, Norwegian Institute of Technology, Trondheim, 1983.

/4/ SCI 1993: Interim Guidance Notes for the Design and Protection of Topside Structures against Explosion and Fire

/5/ Amdahl, J.: "Mechanics of Ship-Ship Collisions- Basic Crushing Mechanics".West Europene Graduate Shcool of Marine Technology, WEGEMT , Copenhagen, 1995

A.9 COMMENTARY

Comm. A.3.1 General

For typical installations, the contribution to energy dissipation from elastic deformation of component/substructures in direct contact with the ship is very small and can normally be neglected. Consequently, plastic methods of analysis apply.

However, elastic elongation of the hit member as well as axial flexibility of the nodes to which the member is connected, have a significant impact on the development of membrane forces in the member. This effect has to be taken into account in the analysis, which is otherwise based on plastic methods. The diagrams in Section A.3.7.2 are based on such an approach.

Depending on the structure size/configuration as well as the location of impact elastic strain energy over the entire structure may contribute significantly.

Comm. A.3.2 Design principles

The transition from essentially strength behaviour to ductile response can be very abrupt and sensitive to minor changes in scantlings. E.g. integrated analyses of impact between the stern of a supply vessel and a large diameter column have shown that with moderate change of (ring - and longitudinal) stiffener size and/or spacing, the energy dissipation may shift from predominantly platform based to predominantly vessel based. Due attention should be paid to this sensitivity when the calculation procedure described in Section A.3.5 is applied.

Comm. A.3.5.1 Recommended

The curve for bow impact in Figure A.3-4 has been derived on the assumption of impacts against an infinitely rigid wall. Sometimes the curve has been used erroneously to assess the energy dissipation in bow-brace impacts.

Experience from small-scale tests /3/ indicate that the bow undergoes very little deformation until the brace becomes strong enough to crush the bow. Hence, the brace absorbs most of the energy. When the brace is strong enough to crush the bow the situation is reversed; the brace remains virtually undamaged.

On the basis of the tests results and simple plastic methods of analysis, force-deformation curves for bows subjected to (strong) brace impact were established in /3/ as a function of impact location and brace diameter. These curves are reproduced in Figure A.9-1. In order to fulfil a strength design requirement the brace should at least resist the load level indicated by the broken line (recommended design curve). For braces with a diameter to thickness ratio < 40 it should be sufficient to verify that the plastic collapse load in bending for the brace is larger than the required level. For larger diameter to thickness ratios local denting must probably be taken into account.

Normally sized jacket braces are not strong enough to resist the likely bow forces given in Figure A.9-1, and therefore it has to be assumed to absorb the entire strain energy. For the same reasons it has also to be assumed that the brace has to absorb all energy for stern and beam impact with supply vessels.

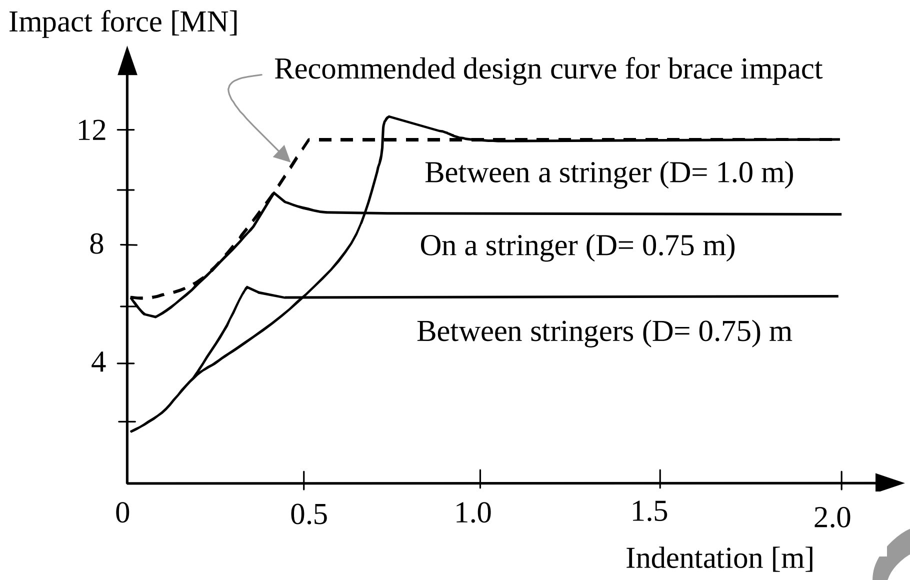


Figure A.9-1 Load-deformation curves for bow-bracing impact /3/

Comm. A.3.5.2 Force contact area for strength design of large diameter columns.

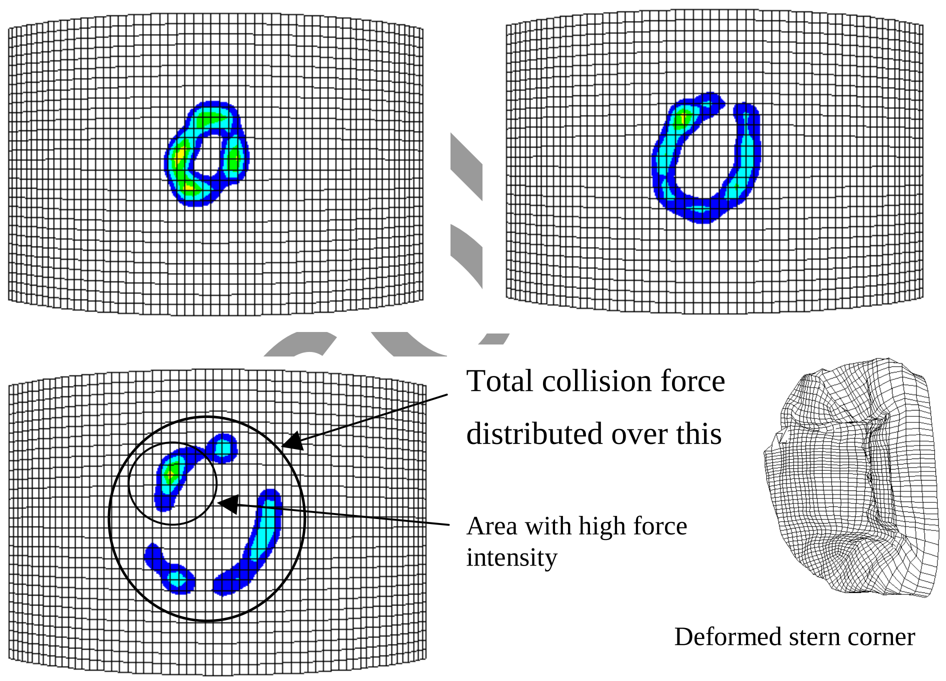


Figure A.9-2 Distribution of contact force for stern corner/large diameter column impact

Figure A.9-2 shows an example of the evolution of contact force intensity during a collision between the stern corner of a supply vessel and a stiffened column. In the beginning the contact is concentrated at the extreme end of the corner, but as the corner deforms it undergoes inversion and the contact ceases in the central part. The contact area is then, roughly speaking, bounded by two concentric circles, but the distribution is uneven.

The force-deformation curves given in Figure A.3-4 relate to total collision force for stern end - and stern corner impact , respectively. Table A.3-1 and Table A.3-2 give the anticipated maximum force intensities (local force and local contact areas, i.e. subsets of the total force and total area) at various stages of deformation.

The basis for the design curves is integrated, non-linear finite element analysis of stern/column impacts.

The information given in A.3.5.2 may be used to perform strength design. If strength design is not achieved numerical analyses have shown that the column is likely to undergo severe deformations and absorb a major part of the strain energy. In lieu of more accurate calculations (e.g. non-linear FEM) it has to be assumed that the column absorbs all strain energy.

Comm. A.3.10.1 General

If the degradation of bending capacity of the beam cross-section after buckling is known the load-carrying capacity may be interpolated from the curves with full bending capacity and no bending capacity according to the expression:

$$R(\overline{w}) = R_{M_p=1}(\overline{w})\xi + R_{M_p=0}(\overline{w})(1-\xi) \qquad \textbf{(A.9.1)}$$

$R_{M_p=1}(\overline{w})$ = Collapse load with full bending contribution

$R_{M_p=0}(\overline{w})$ = Collapse load with no bending contribution

$$\xi = \frac{R_{M_p,red}}{R_{M_p=1}(\overline{w}=0)}$$

$R_{M_{P,red}}$ = Plastic collapse load in bending with reduced cross-sectional capacities. This has to be updated along with the degradation of cross-sectional bending capacity.

Comm. A.3.10.4 Tensile fracture in yield hinges

The rupture criterion is calculated using conventional beam theory. A linear strain hardening model is adopted. For a cantilever beam subjected to a concentrated load at the end, the strain distribution along the beam can be determined from the bending moment variation. In Figure A.9-3 the strain variation, $\varepsilon = \varepsilon_{cr}/\varepsilon_Y$, is shown for a circular cross-section for a given hardening parameter. The extreme importance of strain hardening is evident; with no strain hardening the high strains are very localised close to the support (x = 0), with strain hardening the plastic zone expands dramatically.

On the basis of the strain distribution the rotation in the plastic zone and the corresponding lateral deformation can be determined.

If the beam response is affected by development of membrane forces it is assumed that the membrane strain follows the same relative distribution as the bending strain. By introducing the kinematic relationships for beam elongation, the maximum membrane strain can be calculated for a given displacement.

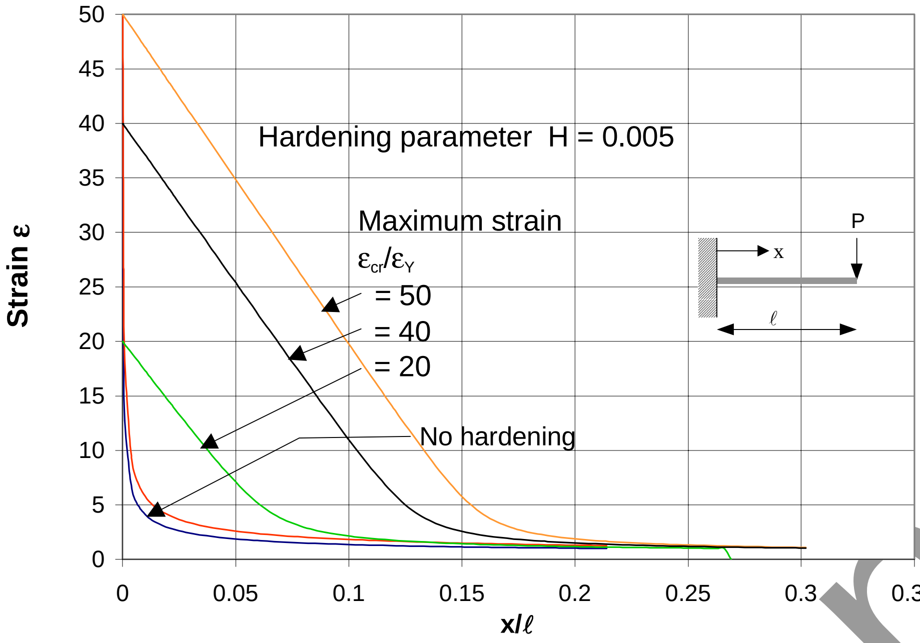


Figure A.9-3 Axial variation of maximum strain for a cantilever beam with circular cross-section

Adding the bending strain and the membrane strain allows determination of the critical displacement as a function of the total critical strain.

Figure A.9-4 shows deformation at rupture for a fully clamped beam as a function of the axial flexibility factor c.

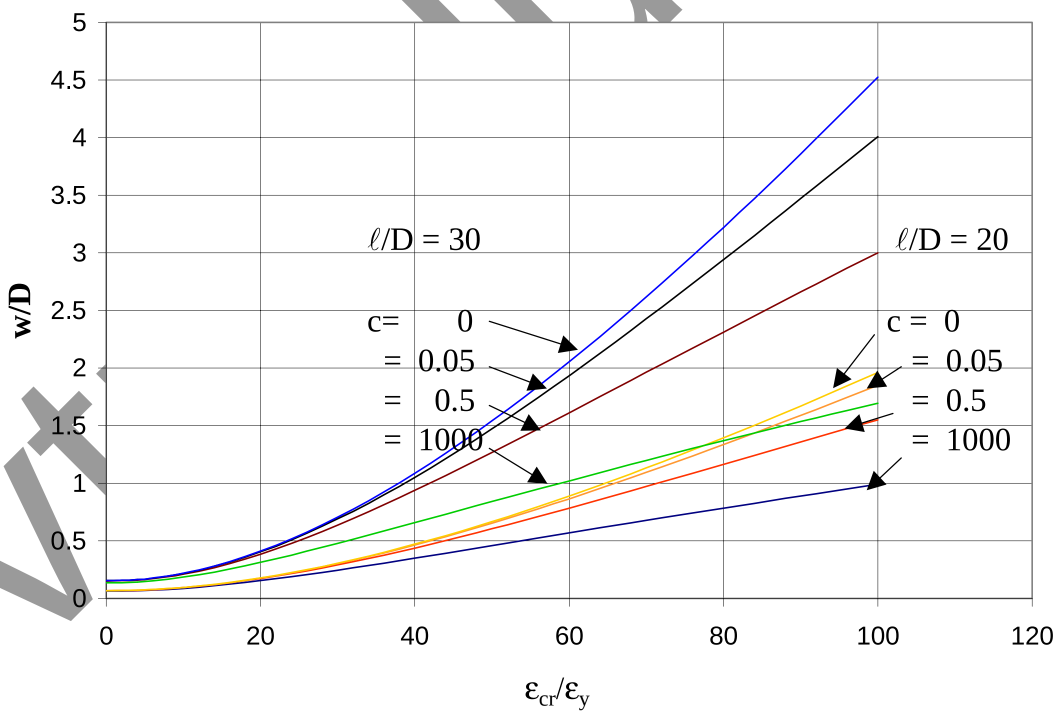


Figure A.9-4 Maximum deformation for a tubular fully clamped beam. (H=0.005)

The plastic stiffness factor H is determined from the stress-strain relationship for the material.. The equivalent linear stiffness shall be determined such that the total area under the stress-strain curve up to the critical strain is preserved (The two portions of the shaded area shall be equal), refer Figure A.9-5. It is unconservative and not allowable to use a reduced effective yield stress and a plastic stiffness factor as illustrated in Figure A.9-6.

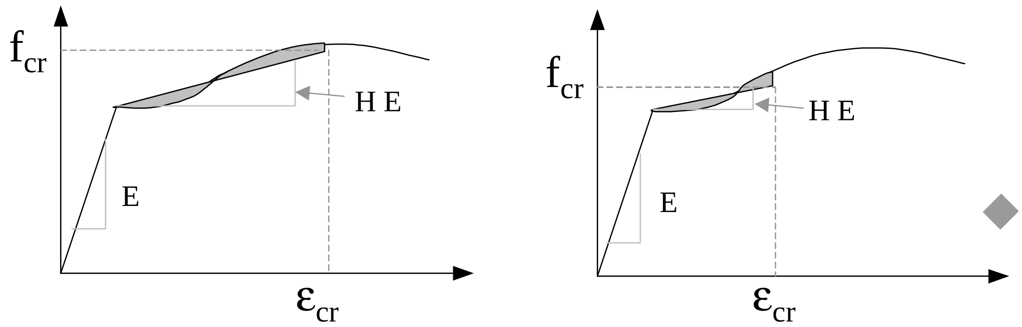


Figure A.9-5 Determination of plastic stiffness

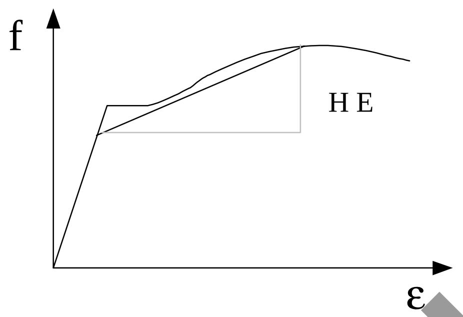


Figure A.9-6 Erroneous determination of plastic stiffness

Comm. A.5.1 General

For redundant structures thermal expansion may cause buckling of members below 400^0 C. Forces due to thermal expansion are, however, purely internal and will be released once the member buckles. The net effect of thermal expansion is therefore often to create lateral distortions in heated members. In most cases these lateral distortions will have a moderate influence on the ultimate strength of the system.

As thermal expansion is the source of considerable inconvenience in conjunction with numerical analysis it would tempting to replace its effect by equivalent, initial lateral member distortions. There is however, not sufficient information to support such a procedure at present.

Comm. A.5.5 Empirical correction factor

In Eurocode 3 an empirical reduction factor of 1.2 is applied in order to obtain better fit between test results and column curve c for fire exposed compressive members. In the design check this is performed by multiplying the design axial load by 1.2. In non-linear analysis such a procedure is, impractical. In non-linear space frame, stress resultants based analysis the correction factor can be included by dividing the yield compressive load and the Euler buckling load by a factor of 1.2. (The influence of axial force on members stiffness is accounted for by the so-called Livesly's stability multipliers, which are functions of the Euler buckling load.) In this way the reduction factor is applied consistently to both elastic and elasto-plastic buckling.

The above correction factor comes in addition to the reduction caused by yield stress and elastic modulus degradation at elevated temperature if the reduced slenderness is' larger than 0.2.

Comm. A.6.2 Classification of response

Equation (A.6.2) is derived using the principle of conservation of momentum to determine the kinetic energy of the component at the end of the explosion pulse. The entire kinetic energy is then assumed dissipated as strain energy.

Equation (A.6.3) is based on the assumption that the explosion pressure has remained at its peak value during the entire deformation and equates the external work with the total strain energy. In general, the explosion pressure is not balanced by resistance, giving rise to inertia forces. Eventually, these inertia forces will be dissipated as strain energy.

Equation (A.6.4) is based on the assumption that the pressure increases slowly so that the static condition (pressure balanced by resistance) applies during the entire deformation.

Comm. A.6.4 SDOF system analogy

The displacement at the end of the initial, linear resistance domain y_{el} will generally *not* coincide with the displacement at first yield. Typically, y_{el} represents the displacement at the initiation of a plastic collapse mechanism. Hence, y_{el} is larger than the displacement at first yield for two reasons:

i. Change from elastic to plastic stress distribution over beam cross-section

ii. Bending moment redistribution over the beam (redundant beams) as plastic hinges form

Comm. A.6.4.1 Dynamic response charts for SDOF system

Figure A.6-3 is derived from the dynamic response chart for a SDOF system subjected to a triangular load with zero rise time given in Figure A.6-4.

In the example it is assumed that from ductility considerations for the assumed mode of deformation a maximum displacement of ten times elastic limit is acceptable. Hence the line

$\frac{y_{allow}}{y_{el}} = \frac{y_{max}}{y_{el}} = 10$ represents the upper limit for the displacement of the component. From the diagram it is seen that several combinations of pulses characterised by F_{max} and t_d may produce this displacement limit. Each intersection with a response curve (e.g. $k_3 = 0$) yields a normalized pressure

$$\frac{F}{R} = \frac{F_{max}}{R_{el}}$$

and a normalised impulse

$$\frac{I}{RT} = \frac{\frac{1}{2}F_{max}t_d}{R_{el}T} = \frac{1}{2\frac{R_{el}}{F_{max}}} \cdot \frac{t_d}{T}$$

By plotting corresponding values of normalised impulse and normalised pressure the iso-damage curve given in Figure A.6-3 is obtained.

Comm. A.6.9.6 Ductility limits

The table is taken from Reference /4/. The values are based upon a limiting strain, elasto.plastic material and cross-sectional shape factor 1.12 for beams and 1.5 for plates. Strain hardening and any membrane effect will increase the effective ductility ratio. The values are likely to be conservative.

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DESIGN OF STEEL STRUCTURES

ANNEX B
BUCKLING STRENGTH OF SHELLS

CONTENTS

B.1 INTRODUCTION

This chapter treats the buckling of circular cylindrical shells, see Figure B.1-1. The shell may be stiffened by longitudinal stiffeners and/or ring frames.

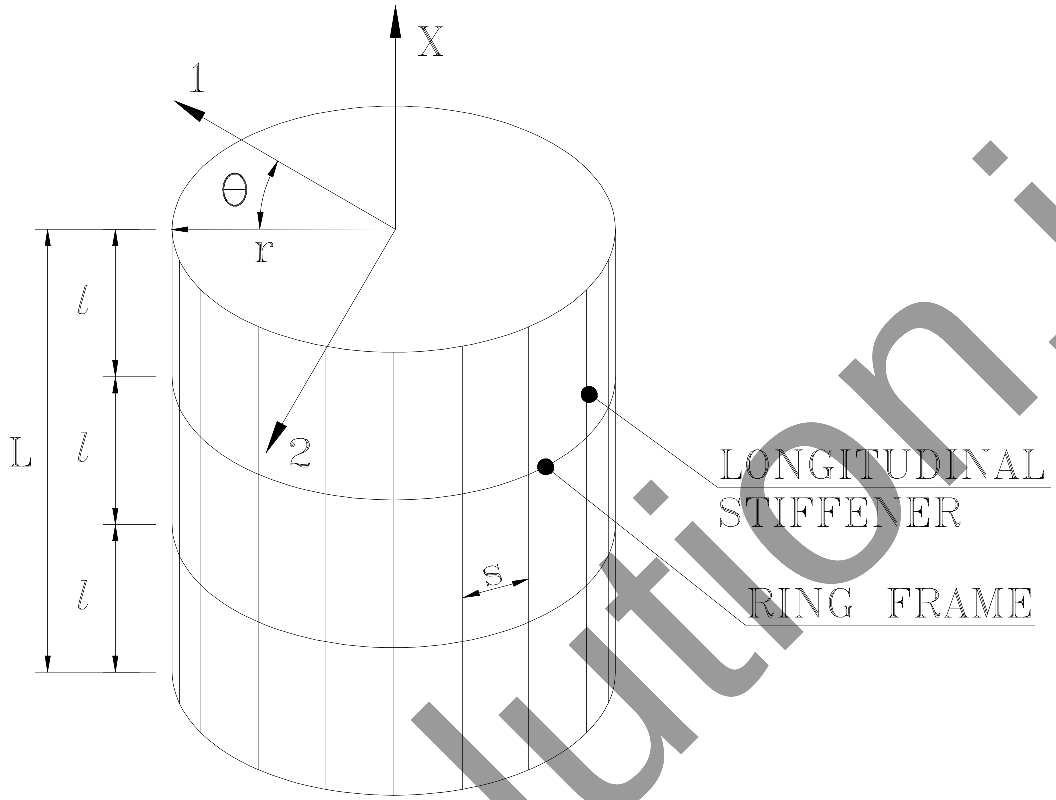

Figure B.1-1 Stiffened cylindrical shell

It is assumed that the edges are effectively supported by ring frames, bulkheads or end closures.

Stiffened circular cylindrical shells have to be dimensioned against several buckling failure modes. The relevant modes are defined in B.3. To exclude local buckling of longitudinal stiffeners and rings, explicit requirements are given in B.5.10.

In Table B.3-1 reference is made to recommended methods for buckling analysis with respect to different buckling modes. The methods are to be considered as semi-empirical. The reason for basing the design on semi-empirical methods is that the agreement between theoretical and experimental buckling loads for some cases has been found to be non-existent. This discrepancy is due to the effect of geometric imperfections and residual stresses in fabricated structures. Actual geometric imperfections and residual stresses do in general not appear as explicit parameters in the expressions for buckling resistance. This means that the methods for buckling analysis are based on an assumed level of imperfections. This level is reflected by the tolerance requirements given in NORSOK M-101.

The recommended methods for buckling analyses may be substituted by more refined analyses or model tests taking into account the real boundary conditions, the pre-buckling edge disturbances, the actual geometric imperfections, the non-linear material behaviour and the residual welding stresses.

B.2 SYMBOLS AND DEFINITIONS

The following symbols are used without a specific definition in the text where they appear:

A	cross-sectional area of a longitudinal stiffener (exclusive of shell flange)
A_f	cross sectional area of flange ($=bt_f$)
A_R	cross-sectional area of a ring frame (exclusive of shell flange)
A_{Req}	required cross sectional area (exclusive of effective plate flange) of ring frame to avoid panel ring buckling
A_w	cross sectional area of web ($=ht_w$)
C	reduced buckling coefficient
C_1	coefficient
C_2	coefficient
E	Young's modulus = $2.1\cdot10^5$ N/mm^2
G	shear modulus, $G = \frac{E}{2(1+\nu)}$
I	moment of inertia of a longitudinal stiffener (exclusive of shell flange)
I_{po}	polar moment of inertia
I_R	effective moment of inertia of a ring frame
I_{sef}	moment of inertia of stiffener including effective shell width s_e
I_t	stiffener torsional moment of inertia (St. Venant torsion).
I_z	moment of inertia of a stiffeners neutral axis normal to the plane of the plate
I_h	minimum required moment of inertia of ringframes inclusive effective shell flange in a cylindrical shell subjected to external lateral or hydrostatic pressure
I_x	minimum required moment of inertia of ringframes inclusive effective shell flange in a cylindrical shell subjected to axial compression and/or bending
I_{xh}	minimum required moment of inertia of ringframes inclusive effective shell flange in a cylindrical shell subjected to torsion and/or shear
L	cylinder length
L_H	equivalent cylinder length
M_{Sd}	design bending moment
N_{Sd}	design axial force
Q_{Sd}	design shear force
T_{Sd}	design torsional moment

$$Z_L = \frac{L^2}{rt}\sqrt{1-\nu^2}$$

$$Z_l = \frac{l^2}{rt}\sqrt{1-\nu^2}$$

$$Z_s = \frac{s^2}{rt}\sqrt{1-\nu^2}$$

a factor

b flange width, factor

c factor

e distance from shell to centeroid of ring frame exclusive of any shell flange

e_f flange eccentricity

f_{ak} reduced characteristic buckling strength

f_{akd} design local buckling strength

f_E Euler buckling strength

f_E elastic buckling strength

f_{Ea} elastic buckling strength for axial force.

f_{Eh} elastic buckling strength for hydrostatic pressure, lateral pressure and circumferential compression.

f_{Em} elastic buckling strength for bending moment.

f_{ET} elastic buckling strength for torsion

$f_{E\tau}$ elastic buckling strength for torsion and shear force.

f_k characteristic buckling strength

f_{kc} characteristic column buckling strength

f_{kcd} design column buckling strength

f_{ks} characteristic buckling strength of a shell

f_{ksd} design buckling strength of a shell

f_r characteristic material strength

f_T torsional buckling strength

f_y yield strength of the material

h web height

h_s distance from stiffener toe (connection between stiffener and plate) to the shear centre of the stiffener.

i radius of gyration

i_h effective radius of gyration of ring frame inclusive affective shell flange

k effective length factor

l distance between ring frames

l_e equivalent length

l_{ef} effective width of shell plating

l_{eo}	equivalent length
l_T	torsional buckling length
p_{Sd}	design lateral pressure
r	shell radius
r_e	equivalent radius
r_f	radius of the shell measured to the ring flange
r_0	radius of the shell measured to the neutral axis of ring frame with effective shell flange, l_{eo}
s	distance between longitudinal stiffeners
s_e	effective shell width
t	shell thickness
t_b	thickness of bulkhead
t_e	equivalent thickness
t_f	thickness of flange
t_w	thickness of web
w	initial-out-of roundness
z_t	distance from outer edge of ring flange to centroid of stiffener inclusive effective shell plating
α, α_A	coefficient
β	coefficient
γ_M	material factor
$\bar{\lambda}$	reduced slenderness
$\bar{\lambda}_s$	reduced shell slenderness
$\bar{\lambda}_T$	reduced torsional slenderness
μ	coefficient
θ	circumferential co-ordinate measured from axis 1
ρ	coefficient
ν	Poisson's ratio = 0.3
$\sigma_{a,Sd}$	design membrane stress in the longitudinal direction due to uniform axial compression
$\sigma_{h,Sd}$	design membrane stress in the circumferential direction
$\sigma_{hR,Sd}$	design membrane stress in a ring frame
$\sigma_{hm,Sd}$	design circumferential bending stress in a shell at a bulkhead or a ringframe
$\sigma_{j,Sd}$	design equivalent von Mieses stress
$\sigma_{m,Sd}$	design membrane stress in the longitudinal direction due to global bending
$\sigma_{x,Sd}$	design membrane stress in the longitudinal direction
$\sigma_{xm,Sd}$	design longitudinal bending stress in a shell at a bulkhead or a ringframe

τ_{Sd} design shear stress tangential to the shell surface (in sections x = constant and θ = constant)

$\tau_{T,Sd}$ design shear stress tangential to the shell surface due to torsional moment

$\tau_{Q,Sd}$ design shear stress tangential to the shell surface due to overall shear forces

ξ coefficient

ψ coefficient

ζ coefficient

A general ring frame cross section is shown in Figure B.2-1, where:

- A Centeroid of ring frame with effective shell flange, l_{eo}
- B Centeroid of ring frame exclusive any shell flange
- C Centeroid of free flange

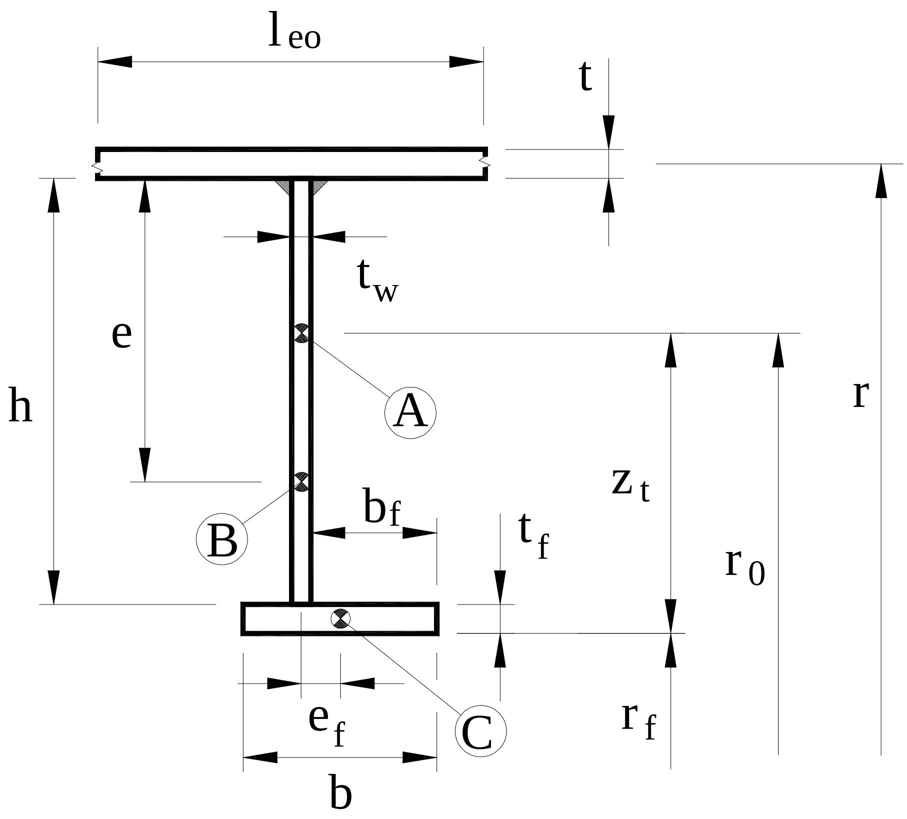

Figure B.2-1 Cross sectional parameters for a ring frame

B.3 BUCKLING MODES

The buckling modes for stiffened cylindrical shells are categorized as follows:

a) Shell buckling: Buckling of shell plating between rings/ longitudinal stiffeners

b) Panel stiffener buckling: Buckling of shell plating including longitudinal stiffeners. Rings are nodal lines

c) Panel ring buckling: Buckling of shell plating including rings. Longitudinal stiffeners act as nodal lines

d) General buckling: Buckling of shell plating including longitudinal stiffeners and rings

e) Column buckling: Buckling of the cylinder as a column

   For long cylindrical shells it is possible that interaction between local buckling and overall column buckling may occur because second order effects of axial compression alter the stress distribution calculated from linear theory. It is then necessary to take this effect into account in the column buckling analysis. This is done by basing the column buckling on a reduced yield strength, f_{kc}, as given for the relevant type of structure.

f) Local buckling of longitudinal stiffeners and rings. See B.5.10

The buckling modes and their relevance for the different cylinder geometries are illustrated in Table B.3-1.

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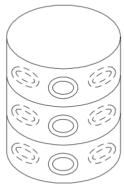
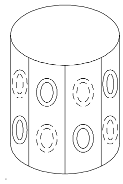
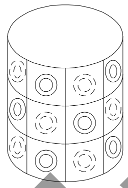
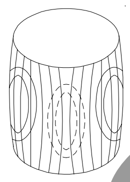
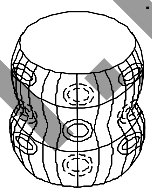
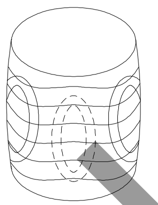
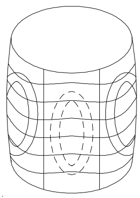
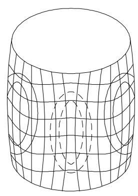
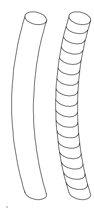
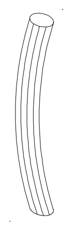
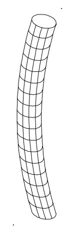
Table B.3-1 Buckling modes for different types of cylinders

Buckling mode	Type of structure geometry		
	Ring stiffened (unstiffened circular)	Longitudinal stiffened	Orthogonally stiffened
a) Shell buckling	 B.5.5	 B.5.6	 B.5.7
b) Panel stiffener buckling		 B.5.6	 B.5.7
Panel ring buckling	 B.5.5		 B.5.7
General buckling			 B.5.7
Column buckling	 B.5.8	 B.5.8	 B.5.8

B.4 STRESSES IN CLOSED CYLINDERS

B.4.1 General

The stress resultants governing the stresses in a cylindrical shell is normally defined by the following quantities:

N_{Sd} = Design axial force
M_{Sd} = Design bending moments
T_{Sd} = Design torsional moment
Q_{Sd} = Design shear force
p_{Sd} = Design lateral pressure

Any of the above quantities may be a function of the axial co-ordinate x. In addition p_{Sd} may be a function of the circumferential co-ordinate θ, measured from axis 1. p_{Sd} is always to be taken as the difference between internal and external pressures, i.e. p_{Sd} is taken positive outwards.

Actual combinations of the above actions are to be considered in the buckling strength assessments.

B.4.2 Stresses

B.4.2.1 General

The membrane stresses at an arbitrary point of the shell plating, due to any or all of the above five actions, are completely defined by the following three stress components:

$\sigma_{x,Sd}$ = design membrane stress in the longitudinal direction
$\sigma_{h,Sd}$ = design membrane stress in the circumferential direction
τ_{Sd} = design shear stress tangential to the shell surface (in sections x = constant and θ = constant)

B.4.2.2 Longitudinal membrane stress

If the simple beam theory is applicable, the design longitudinal membrane stress may be taken as:

$$\sigma_{x,Sd} = \sigma_{a,Sd} + \sigma_{m,Sd} \quad \textbf{(B.4.1)}$$

where $\sigma_{a,Sd}$ is due to uniform axial compression and $\sigma_{m,Sd}$ is due to bending.

For a cylindrical shell without longitudinal stiffeners:

$$\sigma_{a,Sd} = \frac{N_{Sd}}{2\pi r t} \quad \textbf{(B.4.2)}$$

$$\sigma_{m,Sd} = \frac{M_{1,Sd}}{\pi r^2 t}\sin\theta + \frac{M_{2,Sd}}{\pi r^2 t}\cos\theta \quad \textbf{(B.4.3)}$$

For a cylindrical shell with longitudinal stiffeners it is usually permissible to replace the shell thickness by the equivalent thickness:

$$t_e = t + \frac{A}{s} \quad \textbf{(B.4.4)}$$

B.4.2.3 Shear stresses

If simple beam theory is applicable, the membrane shear stress may be taken as:

$$\tau_{Sd} = \tau_{T,Sd} + \tau_{Q,Sd} \tag{B.4.5}$$

where $\tau_{T,Sd}$ is due to the torsional moment and $\tau_{Q,Sd}$ is due to the overall shear forces.

$$\tau_{T,Sd} = \frac{T_{Sd}}{2\pi r^2 t} \tag{B.4.6}$$

$$\tau_{Q,Sd} = \frac{Q_{1,Sd}}{\pi r t}\cos\theta + \frac{Q_{2,Sd}}{\pi r t}\sin\theta \tag{B.4.7}$$

where the signs of the torsional moment and the shear forces must be reflected. Circumferential and longitudinal stiffeners are normally not considered to affect τ_{Sd}.

B.4.2.4 Circumferential membrane stress

For an unstiffened cylinder the circumferential membrane stress may be taken as:

$$\sigma_{h,Sd} = \frac{p_{Sd}\, r}{t} \tag{B.4.8}$$

provided p_{Sd} is constant (gas pressure) or a sine or cosine function of θ (liquid pressure).

For a ringstiffened cylinder (without longitudinal stiffeners) the circumferential membrane stress midway between two ring frames may be taken as:

$$\sigma_{h,Sd} = \frac{p_{Sd}\, r}{t} - \frac{\alpha\zeta}{\alpha+1}\left(\frac{p_{Sd}\, r}{t} - \nu\sigma_{x,Sd}\right) \tag{B.4.9}$$

where

$$\zeta = 2\frac{\sinh\beta\cos\beta + \cosh\beta\sin\beta}{\sinh 2\beta + \sin 2\beta}\text{, but } \zeta \geq 0 \tag{B.4.10}$$

$$\beta = \frac{l}{1.56\sqrt{r\,t}} \tag{B.4.11}$$

$$\alpha = \frac{A_R}{l_{eo}\, t} \tag{B.4.12}$$

$$l_{eo} = \frac{l}{\beta}\left(\frac{\cosh 2\beta - \cos 2\beta}{\sinh 2\beta + \sin 2\beta}\right) \tag{B.4.13}$$

ζ and l_{eo} may also be obtained from Figure B.4-1.

For simplification of the analysis the following approximation may be made:

$l_{eo} = l$ or $l_{eo} = 1.56\sqrt{r\,t}$ whichever is the smaller.

For the particular case when p_{Sd} is constant and $\sigma_{x,Sd}$ is due to the end pressure alone, the above formula may be written as:

$$\sigma_{h,Sd} = \frac{p_{Sd}\, r}{t}\left(1 - \frac{\alpha\left(1-\frac{\nu}{2}\right)\zeta}{\alpha+1}\right) \quad \textbf{(B.4.14)}$$

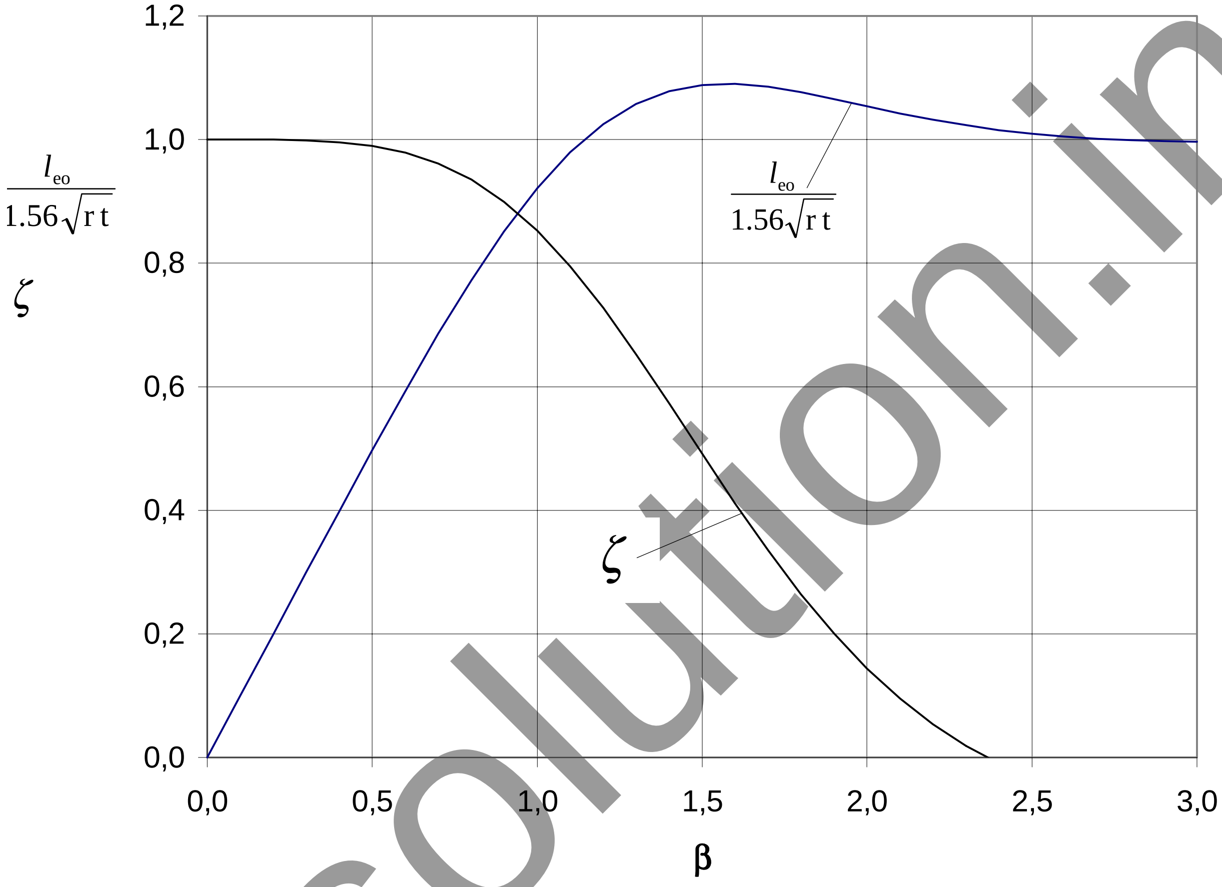


Figure B.4-1 The parameters l_{eo} and ζ

B.4.2.5 Circumferential stress in a ring frame

For ring stiffened shells the circumferential stress in a ring frame at the distance r_r ($r_r = r_f$ at ring flange position) from the cylinder axis may be taken as:

$$\sigma_{hR,Sd} = \left(\frac{p_{Sd}\, r}{t} - \nu\sigma_{x,Sd}\right)\frac{1}{1+\alpha}\left(\frac{r}{r_r}\right) \quad \textbf{(B.4.15)}$$

For the particular case when p_{Sd} is constant and $\sigma_{x,Sd}$ is due to the end pressure alone, the above formula can be written as:

$$\sigma_{hR,Sd} = \frac{p_{Sd}\, r}{t}\left(\frac{1-\frac{\nu}{2}}{1+\alpha}\right)\frac{r}{r_r} \quad \textbf{(B.4.16)}$$

For longitudinally stiffened shells α should be replaced by $\frac{A_R}{l\,t}$ in eq. (B.4.15) and (B.4.16).

B.4.2.6 Stresses in shells at bulkheads and ring stiffeners

The below stresses may be applied in a check for local yielding in the material based on a von Mieses' equivalent stress criterion. These bending stresses should also be accounted for in the fatigue check, but may be neglected in the evaluation of buckling stability.

The circumferential membrane stress at a ring frame for a ringstiffened cylinder (without longitudinal stiffeners) may be taken as:

$$\sigma_{h,Sd} = \left(\frac{p_{Sd}\, r}{t} - \nu\sigma_{x,Sd}\right)\frac{1}{1+\alpha} + \nu\sigma_{x,Sd} \qquad \textbf{(B.4.17)}$$

In the case of a bulkhead instead of a ring, A_R is taken as $\frac{r\,t_b}{(1-\nu)}$, where t_b is the thickness of the bulkhead. For the particular case when p_{Sd} is constant and $\sigma_{x,Sd}$ is due to the end pressure alone, the above formula can be written as:

$$\sigma_{h,Sd} = \frac{p_{Sd}\, r}{t}\left(\frac{1-\frac{\nu}{2}}{1+\alpha} + \frac{\nu}{2}\right) \qquad \textbf{(B.4.18)}$$

Bending stresses and associated shear stresses will occur in the vicinity of "discontinuities" such as bulkheads and frames. These stresses need normally not be considered in detail, except the maximum bending stresses. The longitudinal bending stress in the shell at a bulkhead or a ring frame may be taken as:

$$\sigma_{xm,Sd} = \left(\frac{p_{Sd}\, r}{t} - \sigma_{h,Sd}\right)\sqrt{\frac{3}{1-\nu^2}} \qquad \textbf{(B.4.19)}$$

where $\sigma_{h,Sd}$ is given in (B.4.17) or (B.4.18).

The circumferential bending stress in the shell at a bulkhead or a ring frame is:

$$\sigma_{hm,Sd} = \nu\sigma_{xm,Sd} \qquad \textbf{(B.4.20)}$$

B.5 BUCKLING RESISTANCE OF CYLINDRICAL SHELLS

B.5.1 Stability requirement

The stability requirement for shells subjected to one or more of the following components:

- axial compression or tension
- bending
- circumferential compression or tension
- torsion
- shear

is given by:

$$\sigma_{j,Sd} \leq f_{ksd} \qquad \textbf{(B.5.1)}$$

$\sigma_{j,Sd}$ is defined in (B.5.6) and the design shell buckling strength is defined as:

$$f_{ksd} = \frac{f_{ks}}{\gamma_M} \qquad \textbf{(B.5.2)}$$

The characteristic buckling strength, f_{ks}, is calculated in accordance with Section B.5.2.

γ_M = material factor given as:

$$\begin{aligned} \gamma_M &= 1.15 && \text{for } \bar{\lambda}_s < 0.5 \\ \gamma_M &= 0.85 + 0.60\bar{\lambda}_s && \text{for } 0.5 \leq \bar{\lambda}_s \leq 1.0 \\ \gamma_M &= 1.45 && \text{for } \bar{\lambda}_s > 1.0 \end{aligned} \qquad \textbf{(B.5.3)}$$

Slender shell structures may be subjected to global column buckling. Evaluation of global column buckling is found in B.5.8.

B.5.2 Characteristic buckling strength of shells

The characteristic buckling strength of shells is defined as:

$$f_{ks} = \frac{f_y}{\sqrt{1 + \bar{\lambda}_s^4}} \qquad \textbf{(B.5.4)}$$

where

$$\bar{\lambda}_s^2 = \frac{f_y}{\sigma_{j,Sd}}\left[\frac{\sigma_{a0,Sd}}{f_{Ea}} + \frac{\sigma_{m0,Sd}}{f_{Em}} + \frac{\sigma_{h0,Sd}}{f_{Eh}} + \frac{\tau_{Sd}}{f_{E\tau}}\right] \qquad \textbf{(B.5.5)}$$

$$\sigma_{j,Sd} = \sqrt{(\sigma_{a,Sd} + \sigma_{m,Sd})^2 - (\sigma_{a,Sd} + \sigma_{m,Sd})\sigma_{h,Sd} + \sigma_{h,Sd}^{\ 2} + 3\tau_{Sd}^2} \qquad \textbf{(B.5.6)}$$

$$\sigma_{a0,Sd} = \begin{cases} 0 & \text{if } \sigma_{a,Sd} \geq 0 \\ -\sigma_{a,Sd} & \text{if } \sigma_{a,Sd} < 0 \end{cases} \qquad \textbf{(B.5.7)}$$

$$\sigma_{m0,Sd} = \begin{cases} 0 & \text{if } \sigma_{m,Sd} \geq 0 \\ -\sigma_{m,Sd} & \text{if } \sigma_{m,Sd} < 0 \end{cases} \quad \textbf{(B.5.8)}$$

$$\sigma_{h0,Sd} = \begin{cases} 0 & \text{if } \sigma_{h,Sd} \geq 0, \text{internal net pressure} \\ -\sigma_{h,Sd} & \text{if } \sigma_{h,Sd} < 0 \end{cases} \quad \textbf{(B.5.9)}$$

$\sigma_{a,Sd}$ = design axial stress in the shell due to axial forces (tension positive), see eq. (B.4.2)
$\sigma_{m,Sd}$ = design bending stress in the shell due to global bending moment (tension positive), see eq. (B.4.3)
$\sigma_{h,Sd}$ = design circumferential stress in the shell due to external pressure (tension positive), see eq. (B.4.8), (B.4.9) or (B.4.14). For ring stiffened cylinders shall only stresses midway between rings be used
τ_{Sd} = design shear stress in the shell due to torsional moments and shear force, see eq. (B.4.5)

f_{Ea}, f_{Em}, f_{Eh} and $f_{E\tau}$ are the elastic buckling strengths of curved panels or circular cylindrical shells subjected to axial compression forces, global bending moments, lateral pressure, and torsional moments and/or shear forces respectively, where:

f_{Ea} = elastic buckling strength for axial force
f_{Em} = elastic buckling strength for bending moment
f_{Eh} = elastic buckling strength for hydrostatic pressure, lateral pressure and circumferential compression
$f_{E\tau}$ = elastic buckling strength for torsion and shear force

These may be calculated in accordance with section B.5.3 to B.5.7 taking the appropriate buckling coefficients into account.

B.5.3 Elastic buckling strength of unstiffened curved panels

B.5.3.1 General

The buckling mode to be checked is:

a) Shell buckling, see B.5.3.2

B.5.3.2 Shell buckling

The characteristic buckling strength is calculated from B.5.2.

The elastic buckling strength of curved panels with aspect ratio *l*/s > 1 is given by:

$$f_E = C \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{s}\right)^2 \quad \textbf{(B.5.10)}$$

A curved panel with aspect ratio *l*/s < 1 may be considered as an unstiffened circular cylindrical shell with length equal to *l*, see B.5.4.2.

The reduced buckling coefficient may be calculated as:

$$C = \psi \sqrt{1 + \left(\frac{\rho \xi}{\psi}\right)^2} \quad \textbf{(B.5.11)}$$

The values for ψ, ξ and ρ are given in Table B.5-1 for the most important load cases.

Table B.5-1 Buckling coefficient for unstiffened curved panels, mode a) Shell buckling

	ψ	ξ	ρ
Axial stress	4	$0.702\,Z_s$	$0.5\left(1+\frac{r}{150t}\right)^{-0.5}$
Shear stress	$5.34+4\left(\frac{s}{l}\right)^2$	$0.856\sqrt{\frac{s}{l}}\,Z_s^{3/4}$	0.6
Circumferential compression	$\left[1+\left(\frac{s}{l}\right)^2\right]^2$	$1.04\frac{s}{l}\sqrt{Z_s}$	0.6

The curvature parameter Z_s is defined as:

$$Z_s = \frac{s^2}{rt}\sqrt{1-\nu^2} \tag{B.5.12}$$

B.5.4 Elastic buckling strength of unstiffened circular cylinders

B.5.4.1 General

The buckling modes to be checked are:

a) Shell buckling, see B.5.4.2

e) Column buckling, see B.5.8

B.5.4.2 Shell buckling

The characteristic buckling strength of unstiffened circular cylinders is calculated from B.5.2.

The elastic buckling strength of unstiffened circular cylindrical shells with aspect ratio l/s < 1 is given by:

$$f_E = C\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{l}\right)^2 \tag{B.5.13}$$

A cylindrical shell with aspect ratio l/s > 1 may be considered as a curved panel, see B.5.3.2.

The reduced buckling coefficient may be calculated as:

$$C = \psi\sqrt{1+\left(\frac{\rho\xi}{\psi}\right)^2} \tag{B.5.14}$$

The values for ψ, ξ and ρ are given in Table B.5-2 for the most important load cases.

The curvature parameter Z is defined as:

$$Z_l = \frac{l^2}{rt}\sqrt{1-\nu^2} \tag{B.5.15}$$

For long cylinders the solutions in Table B.5-2 will be pessimistic. Alternative solutions are:

- Torsion and shear force

If $\frac{l}{r} > 3{,}85\sqrt{\frac{r}{t}}$ then the elastic buckling strength may be calculated as:

$$f_{E\tau} = 0{,}25\,E\left(\frac{t}{r}\right)^{3/2} \tag{B.5.16}$$

- Lateral/hydrostatic pressure

If $\frac{l}{r} > 2{,}25\sqrt{\frac{r}{t}}$ then the elastic buckling strength may be calculated as:

$$f_{Eh} = 0{,}25\,E\left(\frac{t}{r}\right)^{2} \tag{B.5.17}$$

Table B.5-2 Buckling coefficients for unstiffened cylindrical shells, mode a) Shell buckling

	ψ	ξ	ρ
Axial stress	1	$0.702\,Z_l$	$0.5\left(1+\frac{r}{150t}\right)^{-0.5}$
Bending	1	$0.702\,Z_l$	$0.5\left(1+\frac{r}{300t}\right)^{-0.5}$
Torsion and shear force	5.34	$0.856\,Z_l^{3/4}$	0.6
Lateral pressure[1)]	4	$1.04\sqrt{Z_l}$	0.6
Hydrostatic pressure[2)]	2	$1.04\sqrt{Z_l}$	0.6

Note 1: Lateral pressure is used when the capped end axial force due to hydrostatic pressure is not included in the axial force.
Note 2: Hydrostatic pressure is used when the capped end axial force due to hydrostatic pressure is included in the axial force.

B.5.5 Ring stiffened shells

B.5.5.1 General

The buckling modes to be checked are:

a) Shell buckling, see B.5.4.2

c) Panel ring buckling, see B.5.5.2

e) Column buckling, see B.5.8

B.5.5.2 Panel ring buckling

The rings will normally be proportioned to avoid the panel ring buckling mode. This is ensured if the following requirements are satisfied.

B.5.5.2.1 Cross sectional area

The cross sectional area of a ring frame (exclusive of effective shell plate flange) should not be less than A_{Req}, which is defined by:

$$A_{Req} \geq \left(\frac{2}{Z_l^2} + 0.06 \right) l \, t \qquad \textbf{(B.5.18)}$$

B.5.5.2.2 Moment of inertia

The effective moment of inertia of a ring frame (inclusive effective shell plate flange) should not be less than I_R, which is defined by:

$$I_R = I_x + I_{xh} + I_h \qquad \textbf{(B.5.19)}$$

I_x, I_{xh} and I_h are defined in eq. (B.5.22), (B.5.24)and (B.5.25), (see also section B.5.5.2.7), the effective width of the shell plate flange is defined in B.5.5.2.3.

B.5.5.2.3 Effective width

The effective width of the shell plating to be included in the actual moment of inertia of a ring frame shall be taken as the smaller of:

$$l_{ef} = \frac{1.56\sqrt{rt}}{1 + 12\frac{t}{r}} \qquad \textbf{(B.5.20)}$$

and

$$l_{ef} = l \qquad \textbf{(B.5.21)}$$

B.5.5.2.4 Calculation of I_x

The moment of inertia of ring frames inclusive effective width of shell plate in a cylindrical shell subjected to axial compression and/or bending should not be less than I_x, which is defined by:

$$I_x = \frac{\sigma_{x,Sd}\, t (1 + \alpha_A) r_0{}^4}{500\, E\, l} \qquad \textbf{(B.5.22)}$$

where

$$\alpha_A = \frac{A}{s\,t} \qquad \textbf{(B.5.23)}$$

A = cross sectional area of a longitudinal stiffener.

B.5.5.2.5 Calculation of I_{xh}

The moment of inertia of ring frames inclusive effective width of shell plate in a cylindrical shell subjected to torsion and/or shear should not be less than I_{xh}, which is defined by:

$$I_{xh} = \left(\frac{\tau_{Sd}}{E}\right)^{8/5}\left(\frac{r_0}{L}\right)^{1/5} L r_0 t l \qquad \textbf{(B.5.24)}$$

B.5.5.2.6 Simplified calculation of I_h for external pressure

The moment of inertia of ring frames inclusive effective width of shell plate in a cylindrical shell subjected to external lateral pressure should not be less than I_h, which is conservatively defined by:

$$I_h = \frac{|p_{Sd}| r r_0^2 l}{3E}\left[2 + \frac{3 E z_t \delta_0}{r_0^2\left(\frac{f_r}{2} - \sigma_{hR,Sd}\right)}\right] \qquad \textbf{(B.5.25)}$$

The characteristic material resistance f_r shall be taken as:

- For fabricated ring frames:
  $f_r = f_T$
- For cold-formed ring frames:
  $f_r = 0.9 f_T$

The torsional buckling strength, f_T, may be taken equal to the yield strength, f_y, if the following requirements are satisfied:

- Flat bar ring frames:

$$h \le 0.4 t_W \sqrt{\frac{E}{f_y}} \qquad \textbf{(B.5.26)}$$

- Flanged ring frames:

$$h \le 1.35 t_W \sqrt{\frac{E}{f_y}} \qquad \textbf{(B.5.27)}$$

$$b \ge \frac{7h}{\sqrt{10 + \frac{E}{f_y}\frac{h}{r}}} \qquad \textbf{(B.5.28)}$$

Otherwise f_T may be obtained from B.5.9.

z_t is defined in Figure B.2-1.

The assumed mode of deformation of the ring frame corresponds to ovalization, and the initial out-of-roundness is defined by:

$$w = \delta_0 \cos 2\theta \qquad \textbf{(B.5.29)}$$

$$\delta_0 = 0.005 r \qquad \textbf{(B.5.30)}$$

Alternatively the capacity of the ring frame may be assessed from B.5.5.2.7.

B.5.5.2.7 Refined calculation of I_h for external pressure

If a ring stiffened cylinder, or a part of a ring stiffened cylinder, is effectively supported at the ends, the following procedure may be used to calculate required moment of inertia I_h. For design it might be recommended to start with equation (B.5.25) to arrive at an initial geometry. (The reason for is that I_h is implicit in the present procedure in equations (B.5.40) and (B.5.44). Thus it is simpler to start with the explicit equation (B.5.25)).

- External pressure

When a ring stiffened cylinder is subjected to external pressure the ring stiffeners should satisfy

$$|p_{Sd}| \le 0.75 \frac{f_k}{\gamma_m} \frac{t\, r_f \left(1 + \frac{A_R}{l_{eo} t}\right)}{r^2 \left(1 - \frac{\nu}{2}\right)} \quad \textbf{(B.5.31)}$$

where

p_{Sd}	=	design pressure
t	=	shell thickness
r_f	=	radius of the shell measured to the ring flange, see Figure B.2-1
r	=	shell radius
l_{eo}	=	smaller of $1.56\sqrt{rt}$ and l
A_R	=	cross sectional area of ring stiffener (exclusive shell flange)

f_k is the characteristic buckling strength found from:

$$\frac{f_k}{f_r} = \frac{1 + \mu + \bar{\lambda}^2 - \sqrt{\left(1 + \mu + \bar{\lambda}^2\right)^2 - 4\bar{\lambda}^2}}{2\bar{\lambda}^2} \quad \textbf{(B.5.32)}$$

where

$$\bar{\lambda} = \sqrt{\frac{f_r}{f_E}} \quad \textbf{(B.5.33)}$$

The values for the parameters f_r, f_E and μ may be taken as:

The characteristic material strength, f_r, may be taken equal to the yield strength, f_y, if the following requirements are satisfied:

- Flat bar ring frames:

$$h \le 0.4\, t_W \sqrt{\frac{E}{f_y}} \quad \textbf{(B.5.34)}$$

- Flanged ring frames:

$$h \le 1.35\, t_W \sqrt{\frac{E}{f_y}} \quad \textbf{(B.5.35)}$$

$$b \ge \frac{7h}{\sqrt{10 + \frac{E}{f_y}\frac{h}{r}}} \quad \textbf{(B.5.36)}$$

Otherwise f_r should be set to f_T. f_T may be obtained from B.5.9.

$$f_E = C_1 \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{L}\right)^2 \quad \textbf{(B.5.37)}$$

where

$$C_1 = \frac{2(1+\alpha)}{1+\beta} \left(\sqrt{1 + \frac{0.27\, Z_L}{\sqrt{1+\alpha}}} - \frac{\alpha}{1+\alpha} \right) \quad \textbf{(B.5.38)}$$

$$Z_L = \frac{L^2}{r\,t} \sqrt{1-\nu^2} \quad \textbf{(B.5.39)}$$

$$\alpha = \frac{12(1-\nu^2)\, I_h}{l\, t^3} \quad \textbf{(B.5.40)}$$

$$\beta = \frac{A_R}{l_{eo} t} \quad \textbf{(B.5.41)}$$

$$\mu = \frac{z_t \delta}{i_h^2} \frac{r_r}{r} \frac{l}{l_{eo}} \left(1 - \frac{C_2}{C_1}\right) \frac{1}{1-\nu/2} \quad \textbf{(B.5.42)}$$

$$\delta = 0.005 r \quad \textbf{(B.5.43)}$$

$$i_h^2 = \frac{I_h}{A_R + l_{eo} t} \quad \textbf{(B.5.44)}$$

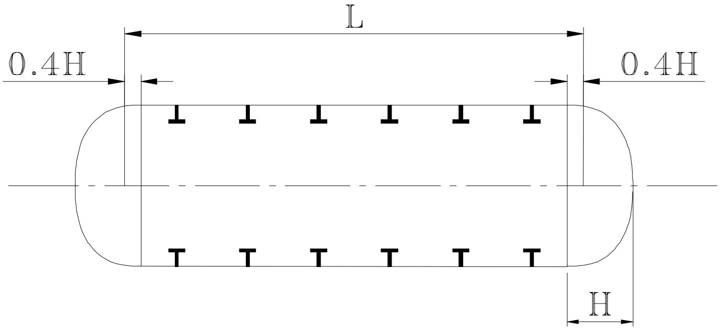
z_t = distance from outer edge of ring flange to centroid of stiffener inclusive effective shell plating, see Figure B.2-1

$$C_2 = 2\sqrt{1 + 0.27\, Z_L} \quad \textbf{(B.5.45)}$$

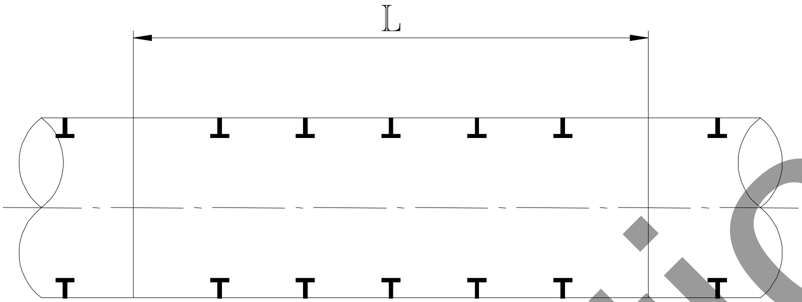
L = distance between effective supports of the ring stiffened cylinder. Effective supports may be:

- End closures, see Figure B.5-1a
- Bulkheads, see Figure B.5-1b
- Heavy ring frames, see Figure B.5-1c

The moment of inertia of a heavy ring frame has to comply with the requirement given in eq. (B.5.19) with *l* substituted by L_H, which is defined in Figure B.5-1d.



a.



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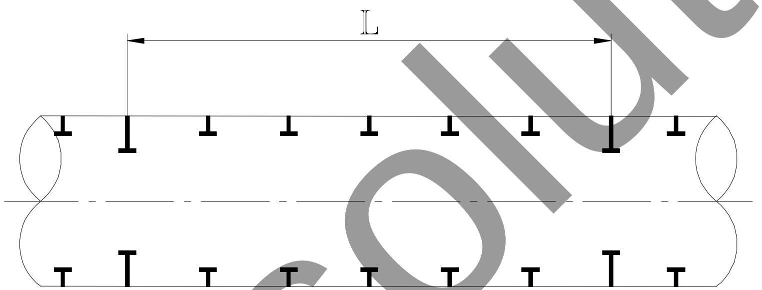

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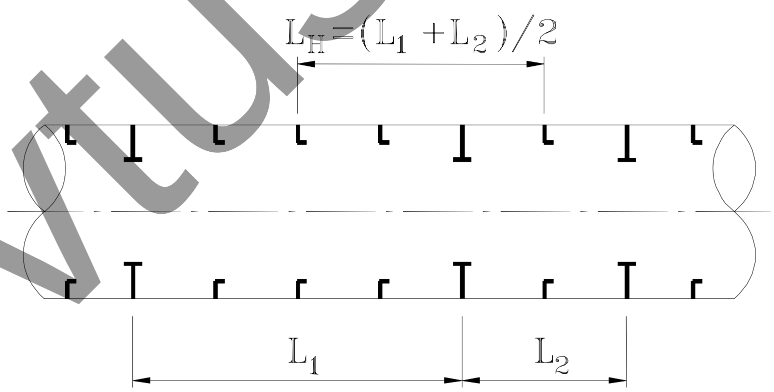

d.

Figure B.5-1 Definition of parameters L and L_H

B.5.6 Longitudinally stiffened shells

B.5.6.1 General

Lightly stiffened shells where $\frac{s}{t} > 3\sqrt{\frac{r}{t}}$ will behave basically as an unstiffened shell and shall be calculated as an unstiffened shell according to the requirements in B.5.3.2 or B.5.4.2.

Shells with a greater number of stiffeners such that $\frac{s}{t} \le 3\sqrt{\frac{r}{t}}$ may be designed according to the requirements given below or as an equivalent flat plate according to NORSOK N-004 section 6.5 taking into account a design transverse stress equal to $\frac{p_{Sd}\, r}{t}$.

The buckling modes to be checked are:

a) Shell buckling, see B.5.6.2

b) Panel stiffener buckling, see B.5.6.3

e) Column buckling, see B.5.8

B.5.6.2 Shell buckling

The characteristic buckling strength is found from B.5.2 and the elastic buckling strengths are given in B.5.3.2 or B.5.4.2

B.5.6.3 Panel stiffener buckling

B.5.6.3.1 General

The characteristic buckling strength is found from B.5.2. It is necessary to base the strength assessment on effective shell area. The axial stress $\sigma_{a,Sd}$ and bending stress $\sigma_{m,Sd}$ are per effective shell width, s_e is calculated from B.5.6.3.3.

Torsional buckling of longitudinal stiffeners may be excluded as a possible failure mode if the following requirements are fulfilled:

- Flat bar longitudinal stiffeners:

$$h \le 0.4\, t_W \sqrt{\frac{E}{f_y}} \qquad \textbf{(B.5.46)}$$

- Flanged longitudinal stiffeners:

$$\bar{\lambda}_T \le 0.6 \qquad \textbf{(B.5.47)}$$

If the above requirements are not fulfilled for the longitudinal stiffeners, an alternative design procedure is to replace the yield strength, f_y, with the torsional buckling strength, f_T, in all equations.

$\bar{\lambda}_T$ and f_T may be found in section B.5.9.

B.5.6.3.2 Elastic buckling strength

The elastic buckling strength of longitudinally stiffened cylindrical shells is given by

$$f_E = C \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{l}\right)^2 \quad \textbf{(B.5.48)}$$

The reduced buckling coefficient may be calculated as

$$C = \psi \sqrt{1 + \left(\frac{\rho \xi}{\psi}\right)^2} \quad \textbf{(B.5.49)}$$

The values for ψ, ξ and ρ are given in table Table B.5-3 for the most important load cases.

Table B.5-3 Buckling coefficients for stiffened cylindrical shells, mode b) Panel stiffener buckling

	ψ	ξ	ρ
Axial stress	$\frac{1+\alpha}{1+\frac{A}{s_e t}}$	$0.702\, Z_l$	0.5
Torsion and shear stress	$5.34 + 1.82\left(\frac{l}{s}\right)^{4/3} \alpha^{1/3}$	$0.856\, Z_l^{3/4}$	0.6
Lateral Pressure	$2\left(1+\sqrt{1+\alpha}\right)$	$1.04\sqrt{Z_l}$	0.6

where

$$Z_l = \frac{l^2}{r\,t}\sqrt{1-\nu^2} \quad \textbf{(B.5.50)}$$

$$\alpha = \frac{12\left(1-\nu^2\right) I_{sef}}{s\,t^3} \quad \textbf{(B.5.51)}$$

A = area of one stiffener, exclusive shell plate
I_{sef} = moment of inertia of stiffener including effective shell width s_e, see eq. (B.5.52).

B.5.6.3.3 Effective shell width

The effective shell width, s_e, may be calculated from:

$$\frac{s_e}{s} = \frac{f_{ks}}{\sigma_{j,Sd}} \frac{\sigma_{x,Sd}}{f_y} \quad \textbf{(B.5.52)}$$

where

f_{ks} = characteristic buckling strength from B.5.3.2/B.5.4.2
$\sigma_{j,sd}$ = design equivalent von Mises stress, see eq. (B.5.6)
$\sigma_{x,Sd}$ = design membrane stress from axial force and bending moment, see eq. (B.4.1)
f_y = yield strength

B.5.7 Orthogonally stiffened shells

B.5.7.1 General

The buckling modes to be checked are:

a) Shell buckling (unstiffened curved panels), see B.5.7.2

b) Panel stiffener buckling, see B.5.7.3

c) Panel ring buckling, see B.5.7.3

d) General buckling, see B.5.7.4

e) Column buckling, see B.5.8

B.5.7.2 Shell buckling

The characteristic buckling strength is found from B.5.2 and the elastic buckling strengths are given in B.5.3.2 or B.5.4.2.

B.5.7.3 Panel ring buckling

Conservative strength assessment following B.5.5.2.

B.5.7.4 General buckling

The rings will normally be proportioned to avoid the general buckling mode. Applicable criteria are given in B.5.5.

B.5.8 Column buckling

B.5.8.1 Stability requirement

The column buckling strength should be assessed if

$$\left(\frac{kL}{i}\right)^2 \geq 2{,}5\frac{E}{f_y} \qquad \textbf{(B.5.53)}$$

where:

k = effective length factor, given in table 4 in N-004
= 2 for cantilevers
L = total cylinder length
i = $\sqrt{\frac{I}{A}}$ = radius of gyration of cylinder section
I = moment of inertia of the complete cylinder section (about weakest axis), including longitudinal stiffeners/internal bulkheads if any
A = cross sectional area of complete cylinder section; including longitudinal stiffeners/internal bulkheads if any.

The stability requirement for a shell-column subjected to axial compression, bending, circumferential compression is given by:

$$\frac{\sigma_{a,Sd}}{f_{kcd}} + \frac{1}{f_{akd}}\left[\left(\frac{\sigma_{m1,Sd}}{1-\frac{\sigma_{a,Sd}}{f_{E1}}}\right)^2 + \left(\frac{\sigma_{m2,Sd}}{1-\frac{\sigma_{a,Sd}}{f_{E2}}}\right)^2\right]^{0.5} \le 1.0 \qquad \textbf{(B.5.54)}$$

where

$\sigma_{a,Sd}$ = design axial compression stress, see eq. (B.4.2)
$\sigma_{m,Sd}$ = maximum design bending stress about given axis, see eq. (B.4.3)
f_{akd} = design local buckling strength, see B.5.8.2
f_{kcd} = design column buckling strength, see eq. (B.5.56)
f_{E1},f_{E2} = Euler buckling strength found from eq. (B.5.55)

$$f_{Ei} = \frac{\pi^2 EI_i}{(k_i L_i)^2 A}, \; i = 1,2 \qquad \textbf{(B.5.55)}$$

$$f_{kcd} = \frac{f_{kc}}{\gamma_M} \qquad \textbf{(B.5.56)}$$

γ_M = material factor, see eq. (B.5.3)
f_{kc} = characteristic column buckling strength, see eq. (B.5.57) or (B.5.58)

B.5.8.2 Column buckling strength

The characteristic buckling strength, f_{kc}, for column buckling may be defined as:

$$f_{kc} = [1.0 - 0.28\bar{\lambda}^2] f_{ak} \qquad \text{for } \bar{\lambda} \le 1.34 \qquad \textbf{(B.5.57)}$$

$$f_{kc} = \frac{0.9}{\bar{\lambda}^2} f_{ak} \qquad \text{for } \bar{\lambda} > 1.34 \qquad \textbf{(B.5.58)}$$

where

$$\bar{\lambda} = \sqrt{\frac{f_{ak}}{f_E}} = \frac{kl}{\pi i}\sqrt{\frac{f_{ak}}{E}} \qquad \textbf{(B.5.59)}$$

In the general case shall f_{ak} be set equal to $\sigma_{a0,Sd}$ derived from eq. (B.5.1) where the actual values for $\sigma_{m,Sd}$, $\sigma_{h,Sd}$ and τ_{Sd} have been applied. For the special case when the shell is an unstiffened shell the following method may be used to calculate f_{ak}.

$$f_{ak} = \frac{b + \sqrt{b^2 - 4ac}}{2a} \qquad \textbf{(B.5.60)}$$

$$a = 1 + \frac{f_y^2}{f_{Ea}^2} \qquad \textbf{(B.5.61)}$$

$$b = \left(\frac{2f_y^2}{f_{Ea} f_{Eh}} - 1 \right) \sigma_{h,Sd} \qquad \textbf{(B.5.62)}$$

$$c = \sigma_{h,Sd}^2 + \frac{f_y^2 \sigma_{h,sd}^2}{f_{Eh}^2} - f_y^2 \qquad \textbf{(B.5.63)}$$

$$f_{akd} = \frac{f_{ak}}{\gamma_M} \qquad \textbf{(B.5.64)}$$

$\sigma_{h,Sd}$ = design circumferential membrane stress, see eq. (B.4.8) or (B.4.9), tension positive
f_y = yield strength
γ_M = material factor, see eq. (B.5.3)
f_{Ea}, f_{Eh} = elastic buckling strengths, see B.5.2

B.5.9 Torsional buckling

The torsional buckling strength may be found from:

$$\frac{f_T}{f_y} = 1.0 \qquad \text{if } \bar{\lambda}_T \le 0.6 \qquad \textbf{(B.5.65)}$$

$$\frac{f_T}{f_y} = \frac{1 + \mu + \bar{\lambda}_T^2 - \sqrt{\left(1 + \mu + \bar{\lambda}_T^2\right)^2 - 4\bar{\lambda}_T^2}}{2\bar{\lambda}_T^2} \qquad \text{if } \bar{\lambda}_T > 0.6 \qquad \textbf{(B.5.66)}$$

where

$$\mu = 0.35\left(\bar{\lambda}_T - 0.6\right) \qquad \textbf{(B.5.67)}$$

$$\bar{\lambda}_T = \sqrt{\frac{f_y}{f_{ET}}} \qquad \textbf{(B.5.68)}$$

Generally f_{ET} may be found from:

$$f_{ET} = \beta \frac{GI_T}{I_{po}} + \pi^2 \frac{Eh_s^2 I_z}{I_{po} l_T^2} \qquad \textbf{(B.5.69)}$$

For T- and L-stiffeners f_{ET} may be found from:

$$f_{ET} = \beta \frac{A_W + \left(\frac{t_f}{t_W}\right)^2 A_f}{A_W + 3A_f} G \left(\frac{t_W}{h}\right)^2 + \frac{\pi^2 E I_z}{\left(\frac{A_W}{3} + A_f\right) l_T^2} \qquad \textbf{(B.5.70)}$$

For flat bar ring stiffeners f_{ET} may be found from:

$$f_{ET} = \left[\beta + 0.2\frac{h}{r}\right] G \left(\frac{t_w}{h}\right)^2 \tag{B.5.71}$$

$$I_z = \frac{1}{12} A_f b^2 + e_f^2 \frac{A_f}{1 + \frac{A_f}{A_W}} \tag{B.5.72}$$

β	=	1.0, or may alternatively for stocky shells be calculated as per equation (B.5.73)
A_f	=	cross sectional area of flange
A_W	=	cross sectional area of web
G	=	shear modulus
I_{po}	=	polar moment of inertia = $\int r^2 dA$ where r is measured from the connection between the stiffener and the plate
I_t	=	stiffener torsional moment of inertia (St. Venant torsion)
I_z	=	moment of inertia of the stiffeners neutral axis normal to the plane of the plate
l_T	=	for ring stiffeners: distance (arc length) between tripping brackets. l_T need not be taken greater than $\pi\sqrt{rt}$ for the analysis for longitudinal stiffeners: distance between ring frames
b	=	flange width
e_f	=	flange eccentricity, see fig Figure B.2-1
h	=	web height
h_s	=	distance from stiffener toe (connection between stiffener and plate) to the shear centre of the stiffener
t	=	shell thickness
t_f	=	thickness of flange
t_W	=	thickness of web

$$\beta = \frac{3C + 0.2}{C + 0.2} \tag{B.5.73}$$

$$C = \frac{h}{s}\left(\frac{t}{t_w}\right)^3 \sqrt{(1-\eta)} \tag{B.5.74}$$

where

$$\eta = \frac{\sigma_{j,Sd}}{f_{ks}} \tag{B.5.75}$$

$\sigma_{j,Sd}$ may be found from eq.(B.5.6) and f_{ks} may be calculated from eq. (B.5.4) using the elastic buckling strengths from B.5.3.2 or B.5.4.2.

Ring frames in a cylindrical shell which is not designed for external lateral pressure shall be so proportioned that the reduced slenderness with respect to torsional buckling, $\overline{\lambda}_T$, is not greater than 0.6.

B.5.10 Local buckling of longitudinal stiffeners and ring stiffeners

B.5.10.1 Ring stiffeners

The geometric proportions of ring frame cross sections should comply with the requirements given below (see Figure B.2-1 for definitions):

- Flat bar ring frames:

$$h \le 0.4\, t_W \sqrt{\frac{E}{f_y}} \qquad \textbf{(B.5.76)}$$

- Flanged ring frames:

$$h \le 1.35\, t_W \sqrt{\frac{E}{f_y}} \qquad \textbf{(B.5.77)}$$

$$b_f \ge 0.4 t_f \sqrt{\frac{E}{f_y}} \qquad \textbf{(B.5.78)}$$

$$\frac{h}{t_w} \le \frac{2}{3} \sqrt{\frac{r_f A_w E}{h A_f f_y}} \qquad \textbf{(B.5.79)}$$

$$\frac{e_f}{t_w} \le \frac{1}{3} \frac{r_f}{h} \frac{A_w}{A_f} \qquad \textbf{(B.5.80)}$$

B.5.10.2 Longitudinal stiffeners

The geometric proportions of longitudinal stiffeners should comply with the requirements given below (see Figure B.2-1 for definitions):

- Flat bar longitudinal stiffeners:

$$h \le 0.4\, t_W \sqrt{\frac{E}{f_y}} \qquad \textbf{(B.5.81)}$$

- Flanged longitudinal stiffeners:

$$h \le 1.35\, t_W \sqrt{\frac{E}{f_y}} \qquad \textbf{(B.5.82)}$$

$$b_f \ge 0.4 t_f \sqrt{\frac{E}{f_y}} \qquad \textbf{(B.5.83)}$$

B.6 UNSTIFFENED CONICAL SHELLS

B.6.1 Introduction

This chapter treates the buckling of unstiffened conical shells, see Figure B.6-1.

Buckling of conical shells is treated like buckling of an equivalent circular cylindrical shell.

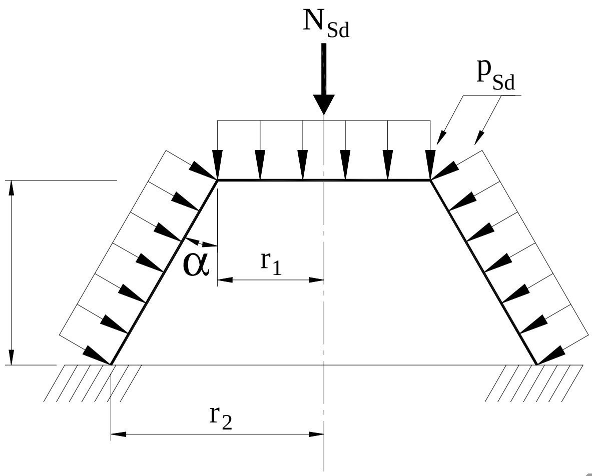

Figure B.6-1 Conical shell

B.6.2 Stresses in conical shells

B.6.2.1 General

The loading condition governing the stresses in a truncated conical shell, Figure B.6-1, is normally defined by the following quantities:

N_{Sd} = design overall axial force exclusive of end pressure
$M_{1,Sd}$ = design overall bending moment acting about principal axis 1
$M_{2,Sd}$ = design overall bending moment acting about principal axis 2
T_{Sd} = design overall torsional moment
$Q_{1,Sd}$ = design overall shear force acting normal to principal axis 1
$Q_{2,Sd}$ = design overall shear force acting normal to principal axis 2
p_{Sd} = design lateral pressure

Any of the above quantities may be a function of the co-ordinate x along the shell generator. In addition p_{Sd} may be a function of the circumferential co-ordinate θ, measured from axis 1.

p_{Sd} is always to be taken as the difference between internal and external pressures, i.e. p_{Sd} is taken positive outwards.

The membrane stresses at an arbitrary point of the shell plating, due to any or all of the above seven actions, are completely defined by the following three stress components:

$\sigma_{x,Sd}$ = normal stress in the longitudinal direction
$\sigma_{h,Sd}$ = normal stress in the circumferential direction
τ_{Sd} = shear stress tangential to the shell surface (in sections x = constant and θ = constant)

B.6.2.2 Longitudinal membrane stress

If simple beam theory is applicable, the longitudinal membrane stress may be taken as:

$$\sigma_{x,Sd} = \sigma_{a,Sd} + \sigma_{m,Sd} \quad \textbf{(B.6.1)}$$

where $\sigma_{a,Sd}$ is due to uniform axial compression and $\sigma_{m,Sd}$ is due to bending.

For a conical shell without stiffeners along the generator:

$$\sigma_{a,Sd} = \frac{p_{Sd}\, r}{2\, t_e} + \frac{N_{Sd}}{2\pi\, r\, t_e} \quad \textbf{(B.6.2)}$$

$$\sigma_{m,Sd} = \frac{M_{1,Sd}}{\pi r^2 t_e}\sin\theta + \frac{M_{2,Sd}}{\pi r^2 t_e}\cos\theta \quad \textbf{(B.6.3)}$$

where

t_e = $t \cos\alpha$

B.6.2.3 Circumferential membrane stress

The circumferential membrane stress may be taken as:

$$\sigma_{h,Sd} = \frac{p_{Sd}\, r}{t_e} \quad \textbf{(B.6.4)}$$

where

t_e = $t \cos\alpha$

B.6.2.4 Shear stress

If simple beam theory is applicable, the membrane shear stress may be taken as:

$$\tau_{Sd} = \tau_{T,Sd} + \tau_{Q,Sd} \quad \textbf{(B.6.5)}$$

where $\tau_{T,Sd}$ is due to the torsional moment and $\tau_{Q,Sd}$ is due to the overall shear forces

$$\tau_{T,Sd} = \frac{T_{Sd}}{2\pi r^2 t} \quad \textbf{(B.6.6)}$$

$$\tau_{Q,Sd} = \frac{Q_{1,Sd}}{\pi r t}\cos\theta + \frac{Q_{2,Sd}}{\pi r t}\sin\theta \quad \textbf{(B.6.7)}$$

where the signs of the torsional moment and the shear forces must be reflected.

B.6.3 Shell buckling

The characteristic buckling strength of a conical shell may be determined according to the procedure given for unstiffened cylindrical shells, B.5.4.

The elastic buckling strength of a conical shell may be taken equal to the elastic buckling resistance of an equivalent unstiffened cylindrical shell defined by:

$$r_e = \frac{r_1 + r_2}{2\cos\alpha} \tag{B.6.8}$$

$$l_e = \frac{l}{\cos\alpha} \tag{B.6.9}$$

The buckling strength of conical shells has to comply with the requirements given in B.5.4 for cylindrical shells. In lieu of more accurate analyses, the requirements are to be satisfied at any point of the conical shell, based on a membrane stress distribution according to section B.6.2.

B.7 REFERENCES

/1/ Odland, J.: “Improvements in design Methodology for Stiffened and Unstiffened Cylindrical Structures”. BOSS’88. Proceedings of the International Conference on behaviour of Offshore Structures. Trondheim, June 1988.

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DESIGN OF STEEL STRUCTURES

ANNEX C
FATIGUE STRENGTH ANALYSIS

CONTENTS

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C.1 INTRODUCTION

C.1.1 General

In this Annex to NORSOK Standard N-004, recommendations are given in relation to fatigue analyses based on fatigue tests and fracture mechanics.

The aim of fatigue design is to ensure that the structure has an adequate fatigue life. Calculated fatigue lives also form the basis for efficient inspection programmes during fabrication and the operational life of the structure.

To ensure that the structure will fulfil its intended function, a fatigue assessment, supported where appropriate by a detailed fatigue analysis, should be carried out for each individual member which is subjected to fatigue loading. See also section C.1.4. It should be noted that any element or member of the structure, every welded joint and attachment or other form of stress concentration, is potentially a source of fatigue cracking and should be individually considered.

C.1.2 Validity of standard

This standard is valid for steel materials with yield strength less than 500 MPa.

This standard is also valid for bolts in air environment or with protection corresponding to that condition of grades up to 10.9.

C.1.3 Methods for fatigue analysis

The fatigue analysis should be based on S-N data, determined by fatigue testing of the considered welded detail, and the linear damage hypothesis. When appropriate, the fatigue analysis may alternatively be based on fracture mechanics. If the fatigue life estimate based on fatigue tests is short for a component where a failure may lead to severe consequences, a more accurate investigation considering a larger portion of the structure, or a fracture mechanics analysis, should be performed. For calculations based on fracture mechanics, it should be documented that there is a sufficient time interval between time of crack detection during in-service inspection and the time of unstable fracture.

All significant stress ranges, which contribute to fatigue damage in the structure, should be considered. The long term distribution of stress ranges may be found by deterministic or spectral analysis. Dynamic effects shall be duly accounted for when establishing the stress history.

A fatigue analysis may be based on an expected stress history, which can be defined as expected number of cycles at each stress range level during the predicted life span. A practical application of this is to establish a long term stress range history that are to the safe side. The part of the stress range history contributing most significantly to the fatigue damage should be most carefully evaluated. See also commentary for some guidance. It should be noted that the shape parameter h in the Weibull distribution has a significant impact on calculated fatigue damage. For effect of the shape parameter on fatigue damage see also design charts in Figure C.2-23 and Figure C.2-24. Thus, when the fatigue damage is calculated based on closed form solutions with an assumption of a Weibull long term stress range distribution, a shape parameter to the safe side should be used.

C.1.4 Guidance to when a detailed fatigue analysis can be omitted.

A detailed fatigue analysis can be omitted if the largest nominal stress range is less than 21 MPa. Reference is made to constant amplitude fatigue limit in Table C.2-1and Table C.2-2. For Design Fatigue Factors (DFF) larger than 1 the allowable fatigue limit should be reduced by a factor $(DFF)^{-0.33}$. A detailed fatigue analysis can also be omitted if the largest nominal stress range for actual details defined in Appendix 1 is less than the constant amplitude fatigue limit at 10^7 cycles in Table C.2-1 for air and Table C.2-2 for seawater with cathodic protection. For Design Fatigue Factors larger than 1 the allowable fatigue limit should also here be reduced by a factor $(DFF)^{-0.33}$.

Requirements to detailed fatigue analysis may also be assessed based on the fatigue assessment charts in Figure C.2-23 and Figure C.2-24.

C.1.5 Symbols

C	material parameter
D	accumulated fatigue damage, diameter of chord
DFF	Design Fatigue Factor
D_j	cylinder diameter at junction
F	fatigue life
I	moment of inertia of tubulars
K_{max}, K_{min}	maximum and minimum stress intensity factors respectively
ΔK	$K_{max} - K_{min}$
L	length of chord, length of thickness transition
N	number of cycles to failure
N_i	number of cycles to failure at constant stress range $\Delta\sigma_i$
N_{Sd}	axial force in tubulars
R	outer radius of considered chord, reduction factor on fatigue life
SCF	stress concentration factor
SCF_{AS}	stress concentration factor at the saddle for axial load
SCF_{AC}	stress concentration factor at the crown for axial load
SCF_{MIP}	stress concentration factor for in plane moment
SCF_{MOP}	stress concentration factor for out of plane moment
T	thickness of chord
T_e	equivalent thickness of chord
T_d	design life in seconds
Q	probability for exceedance of the stress range $\Delta\sigma$
a	crack depth
a_i	half crack depth for internal cracks
$\bar{a}$	intercept of the design S-N curve with the log N axis
$e^{-\alpha}$	$= \exp(-\alpha)$
g	gap = a/D; factor depending on the geometry of the member and the crack
h	Weibull shape parameter, weld size

k	number of stress blocks, exponent on thickness
l	segment lengths of the tubulars
m	negative inverse slope of the S-N curve; crack growth parameter
n_i	number of stress cycles in stress block *i*
n_o	is the number of cycles over the time period for which the stress range level $\Delta\sigma_o$ is defined
t_{ref}	reference thickness
t	plate thickness, thickness of brace member
t_c	cone thickness
t_p	plate thickness
q	Weibull scale parameter
Γ	gamma function
η	usage factor
α	the slope angle of the cone; α = L/D
β	d/D
δ	eccentricity
γ	R/T
ν_o	average zero-crossing frequency
$\sigma_{nominal}$	nominal stress
$\sigma_{hot\ spot}$	hot spot stress or geometric stress
σ_x	maximum nominal stresses due to axial force
σ_{my} and σ_{mz}	maximum nominal stresses due to bending about the y-axis and the z-axis
$\Delta\sigma$	stress range
$\Delta\sigma_o$	Stress range exceeded once out of n_0 cycles
τ	t/T

C.2 FATIGUE ANALYSIS BASED ON FATIGUE TESTS

C.2.1 General

The fatigue life may be calculated based on the S-N fatigue approach under the assumption of linear cumulative damage (Palmgren-Miner rule).

When the long-term stress range distribution is expressed by a stress histogram, consisting of a convenient number of constant amplitude stress range blocks $\Delta\sigma_i$ each with a number of stress repetitions n_i the fatigue criterion reads

$$D = \sum_{i=1}^{k} \frac{n_i}{N_i} = \frac{1}{\bar{a}} \sum_{i=1}^{k} n_i \cdot \left(\Delta\sigma_i\right)^m \leq \eta \qquad \textbf{(C.2.1)}$$

where

D	=	accumulated fatigue damage
$\bar{a}$	=	intercept of the design S-N curve with the log N axis
m	=	negative inverse slope of the S-N curve
k	=	number of stress blocks
n_i	=	number of stress cycles in stress block i
N_i	=	number of cycles to failure at constant stress range $\Delta\sigma_i$
η	=	usage factor (=1/Design Fatigue Factor from NORSOK Standard N-004)

Applying a histogram to express the stress distribution, the number of stress blocks, k, should be large enough to ensure reasonable numerical accuracy, and should not be less than 20. Due consideration should be given to selection of integration method as the position of the integration points may have a significant influence on the calculated fatigue life dependent on integration method.

See also section C.2.11 for calculation of fatigue damage.

C.2.2 Stresses to be considered

The procedure for the fatigue analysis is based on the assumption that it is only necessary to consider the ranges of cyclic principal stresses in determining the fatigue endurance (i. e. mean stresses are neglected for fatigue assessment of welded connections).

When the potential fatigue crack is located in the parent material at the weld toe, the relevant hot spot stress is the range of maximum principal stress adjacent to the potential crack location with stress concentrations being taken into account.

For joints other than tubular joints, the joint classification and corresponding S-N curves takes into account the local stress concentrations created by the joints themselves and by the weld profile. The design stress can therefore be regarded as the nominal stress, adjacent to the weld under consideration. However, if the joint is situated in a region of stress concentration resulting from the gross shape of the structure, this must be taken into account in calculating the nominal stress. As an example, for the weld shown in Figure C.2-2a), the relevant hot spot stress for fatigue design would be the tensile stress, σ. For the weld shown in Figure C.2-2 b), the stress concentration factor for the global geometry must in addition be accounted for, giving the relevant hot spot stress equal to SCF·σ, where SCF is the stress concentration factor due to the hole.

The maximum principal stress range within 45° of the normal to the weld toe should be used for the analysis.

The relevant stress range for potential cracks in the weld throat of load-carrying fillet-welded joints and partial penetration welded joints may be found as:

$$\Delta\sigma_w = \sqrt{\Delta\sigma_{\perp}^2 + \Delta\tau_{\perp}^2 + 0.2\,\Delta\tau_{//}^2} \tag{C.2.2}$$

See Figure C.2-1 for explanation of stress components.

The total stress fluctuation (i.e. maximum compression and maximum tension) should be considered to be transmitted through the welds for fatigue assessments.

For detailed finite element analysis of welded connections other than tubular joints it may also be convenient to use the alternative hot spot stress for fatigue life assessment, see C.2.10.3 for further guidance.

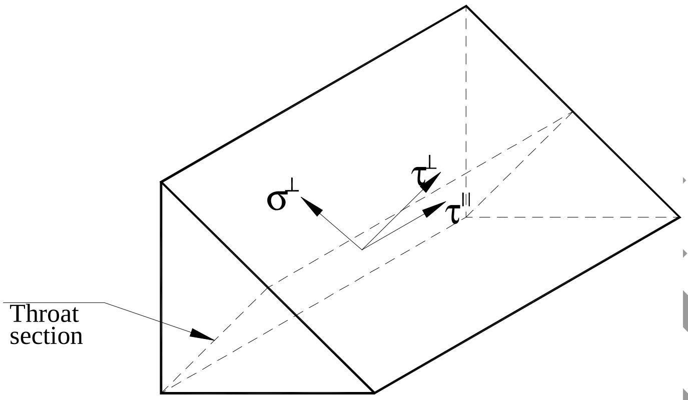


Figure C.2-1 Explanation of stresses on the throat section of a fillet weld

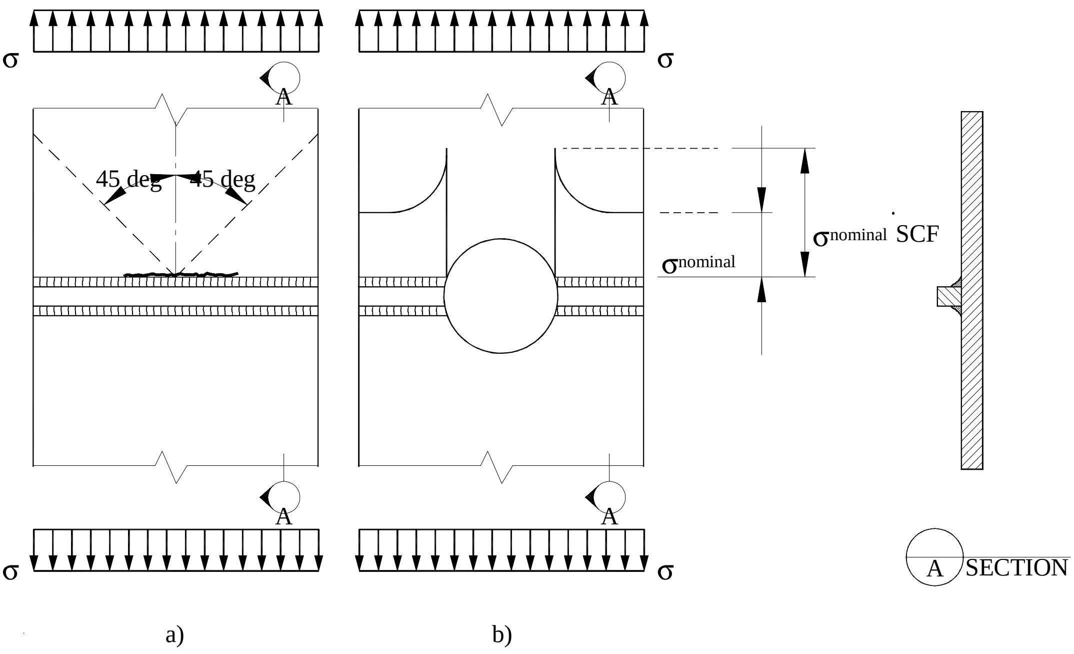


Figure C.2-2 Explanation of local stresses

For a tubular joint, i. e. chord to brace connection, the stress to be used for design purpose is the range of idealised hot spot stress defined by: the greatest value of the extrapolation to the maximum principal stress distribution immediately outside the region effected by the geometry of the weld. The hot spot stress to be used in combination with the T-curve is thus calculated as

$$\sigma_{\text{hot spot stress}} = \text{SCF}\,\sigma_{\text{nominal}} \quad \textbf{(C.2.3)}$$

where

SCF = stress concentration factor as given in section C.2.6.3.

Where support plating below bearings are designed with fillet welded connection, it should be verified that fatigue cracking of the weld will not occur. Even though the joint may be required to carry wholly compressive stresses and the plate surfaces may be machined to fit, for fatigue purposes, the total stress fluctuation should be considered to be transmitted through the welds.

If it is assumed that compressive loading is transferred through contact, it should be verified that contact will not be lost during the welding. The actual installation condition including maximum construction tolerances should be accounted for.

C.2.3 S-N curves

C.2.3.1 S-N curves and joint classification

For practical fatigue design, welded joints are divided into several classes, each with a corresponding design S-N curve. All tubular joints are assumed to be class T. Other types of joint, including tube to plate, may fall in one of the 14 classes specified in Table C.2-1, Table C.2-2, Table C.2-3 depending upon:

- The geometrical arrangement of the detail;
- The direction of the fluctuating stress relative to the detail;
- The method of fabrication and inspection of the detail.

Each construction detail at which fatigue cracks may potentially develop should, where possible, be placed in its relevant joint class in accordance with criteria given in Appendix 1. It should be noted that, in any welded joint, there are several locations at which fatigue cracks may develop, e. g. at the weld toe in each of the parts joined, at the weld ends, and in the weld itself. Each location should be classified separately.

The fatigue design is based on use of S-N curves, which are obtained from fatigue tests. The design S-N curves which follows are based on the mean-minus-two-standard-deviation curves for relevant experimental data. The S-N curves are thus associated with a 97.6% probability of survival.

The basic design S-N curve is given as

$$\log N = \log \bar{a} - m \log \Delta\sigma \quad \textbf{(C.2.4)}$$

where

N = predicted number of cycles to failure for stress range $\Delta\sigma$
$\Delta\sigma$ = stress range
m = negative inverse slope of S-N curve
$\log \bar{a}$ = intercept of log N-axis by S-N curve

$$\log \bar{a} = \log a - 2s \quad \textbf{(C.2.5)}$$

where

a = constant relating to mean S-N curve
s = standard deviation of log N.

The fatigue strength of welded joints is to some extent dependent on plate thickness. This effect is due to the local geometry of the weld toe in relation to thickness of the adjoining plates. See also effect of profiling on thickness effect in C.4.2. It is also dependent on the stress gradient over the thickness. The thickness effect is accounted for by a modification on stress such that the design S-N curve for thickness larger than the reference thickness reads:

$$\log N = \log \bar{a} - m \log\left(\Delta\sigma\left(\frac{t}{t_{ref}}\right)^{k}\right) \qquad \textbf{(C.2.6)}$$

where

m = negative inverse slope of the S - N curve
log a = intercept of log N axis
tref = reference thickness equal 25 mm for welded connections other than tubular joints. For tubular joints the reference thickness is 32 mm. For bolts tref = 25 mm.
t = thickness through which a crack will most likely grow. t = tref is used for thickness less than tref.
k = thickness exponent on fatigue strength as given in Table C.2-1, Table C.2-2, and Table C.2-3.
k = 0.10 for tubular butt welds made from one side.
k = 0.40 for threaded bolts subjected to stress variation in the axial direction.

In general the thickness exponent is included in the design equation to account for a situation that the actual size of the structural component considered is different in geometry from that the S-N data are based on. The thickness exponent is considered to account for different size of plate through which a crack will most likely grow. To some extent it also accounts for size of weld and attachment. However, it does not account for weld length or length of component different from that tested such as e. g. design of mooring systems with a significant larger number of chain links in the actual mooring line than that the test data are based on. Then the size effect should be carefully considered using probabilistic theory to achieve a reliable design, see commentary.

C.2.3.2 S-N curves in air

S-N curves for air environment are given in Table C.2-1 and Figure C.2-3.

Table C.2-1 S-N curves in air

S-N curve	$\log \bar{a}_1$ **N ≤ 10^7 cycles $m_1 = 3.0$**	$\log \bar{a}_2$ **N > 10^7 cycles $m_2 = 5.0$**	**Constant amplitude fatigue limit at 10^7 cycles *)**	**Thickness exponent k**	**Stress concentration in the S-N detail as derived by the hot spot method**
B1	12.913	16.856	93.57	0	
B2	12.739	16.566	81.87	0	
C	12.592	16.320	73.10	0.15	
C1	12.449	16.081	65.50	0.15	
C2	12.301	15.835	58.48	0.15	
D	12.164	15.606	52.63	0.20	1.00
E	12.010	15.350	46.78	0.20	1.13
F	11.855	15.091	41.52	0.25	1.27
F1	11.699	14.832	36.84	0.25	1.43
F3	11.546	14.576	32.75	0.25	1.61
G	11.398	14.330	29.24	0.25	1.80
W1	11.261	14.101	26.32	0.25	2.00
W2	11.107	13.845	23.39	0.25	2.25
W3	10.970	13.617	21.05	0.25	2.50
T	12.164	15.606	52.63	0.25 for SCF ≤ 10.0 0.30 for SCF >10.0	1.00

*) See also C.1.4.

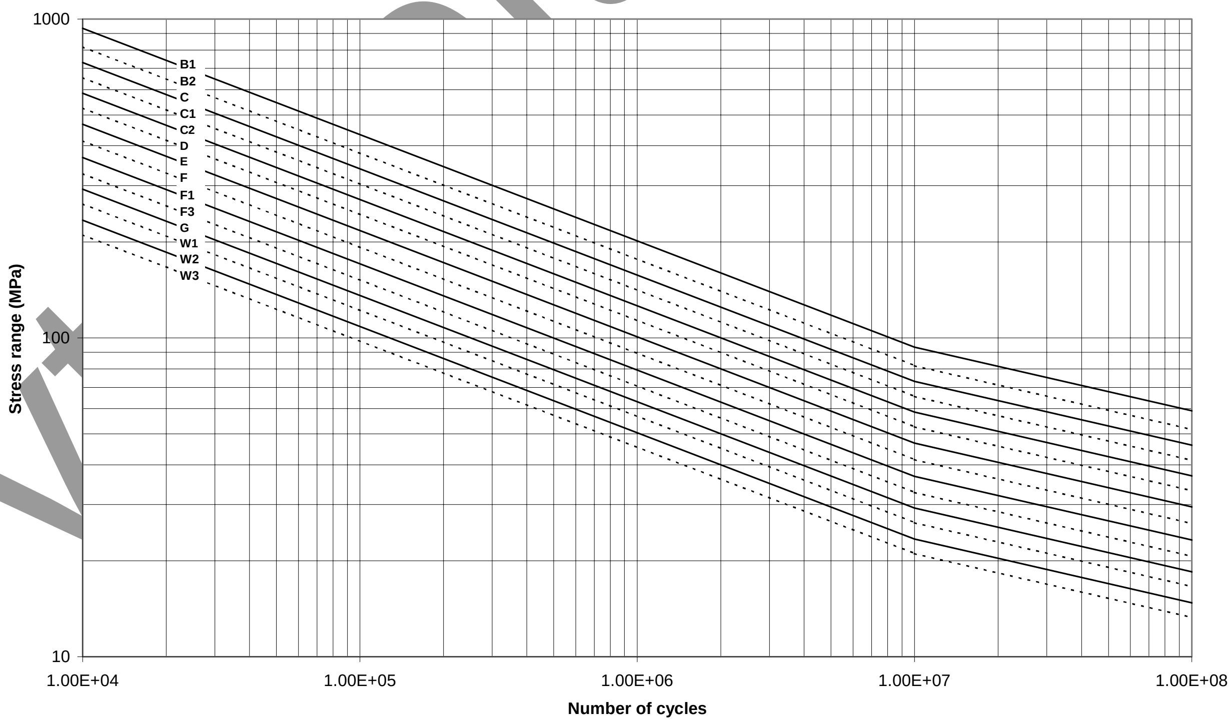


Figure C.2-3 S-N curves in air

C.2.3.3 S-N curves in seawater with cathodic protection

S-N curves for seawater environment with cathodic protection are given in Table C.2-2 and Figure C.2-4.

Table C.2-2 S-N curves in seawater with cathodic protection

S-N curve	$\log \bar{a}_1$ $N \le 10^6$ cycles $m_1 = 3.0$	$\log \bar{a}_2$ $N > 10^6$ cycles $m_2 = 5.0$	Constant amplitude fatigue limit at 10^7 cycles *)	Thickness exponent k	Stress concentration in the S-N detail as derived by the hot spot method
B1	12.513	16.856	93.57	0	
B2	12.339	16.566	81.87	0	
C	12.192	16.320	73.10	0.15	
C1	12.049	16.081	65.50	0.15	
C2	11.901	15.835	58.48	0.15	
D	11.764	15.606	52.63	0.20	1.00
E	11.610	15.350	46.78	0.20	1.13
F	11.455	15.091	41.52	0.25	1.27
F1	11.299	14.832	36.84	0.25	1.43
F3	11.146	14.576	32.75	0.25	1.61
G	10.998	14.330	29.24	0.25	1.80
W1	10.861	14.101	26.32	0.25	2.00
W2	10.707	13.845	23.39	0.25	2.25
W3	10.570	13.617	21.05	0.25	2.50
T	11.764	15.606	52.63	0.25 for SCF ≤ 10.0 0.30 for SCF >10.0	1.00

*) See also C.1.4.

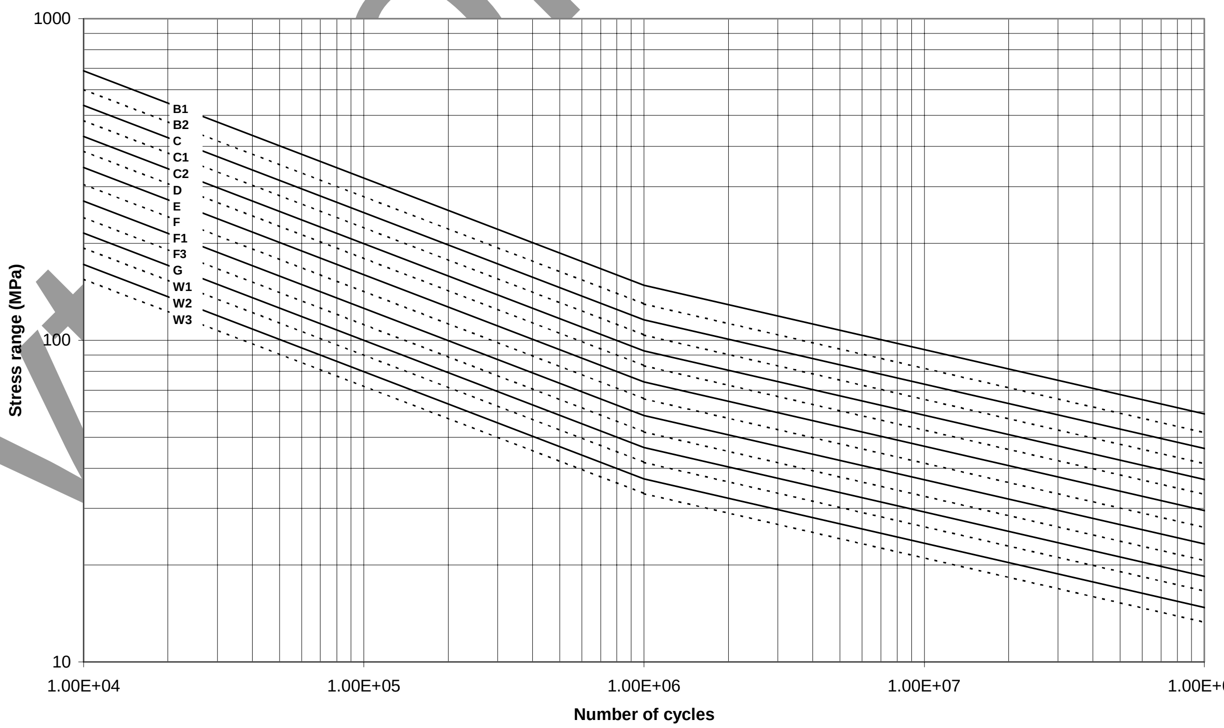


Figure C.2-4 S-N curves in seawater with cathodic protection

C.2.3.4 S-N curves for tubular joints

S-N curves for tubular joints in air environment and in seawater with cathodic protection are given in Table C.2-1, Table C.2-2 and Figure C.2-5.

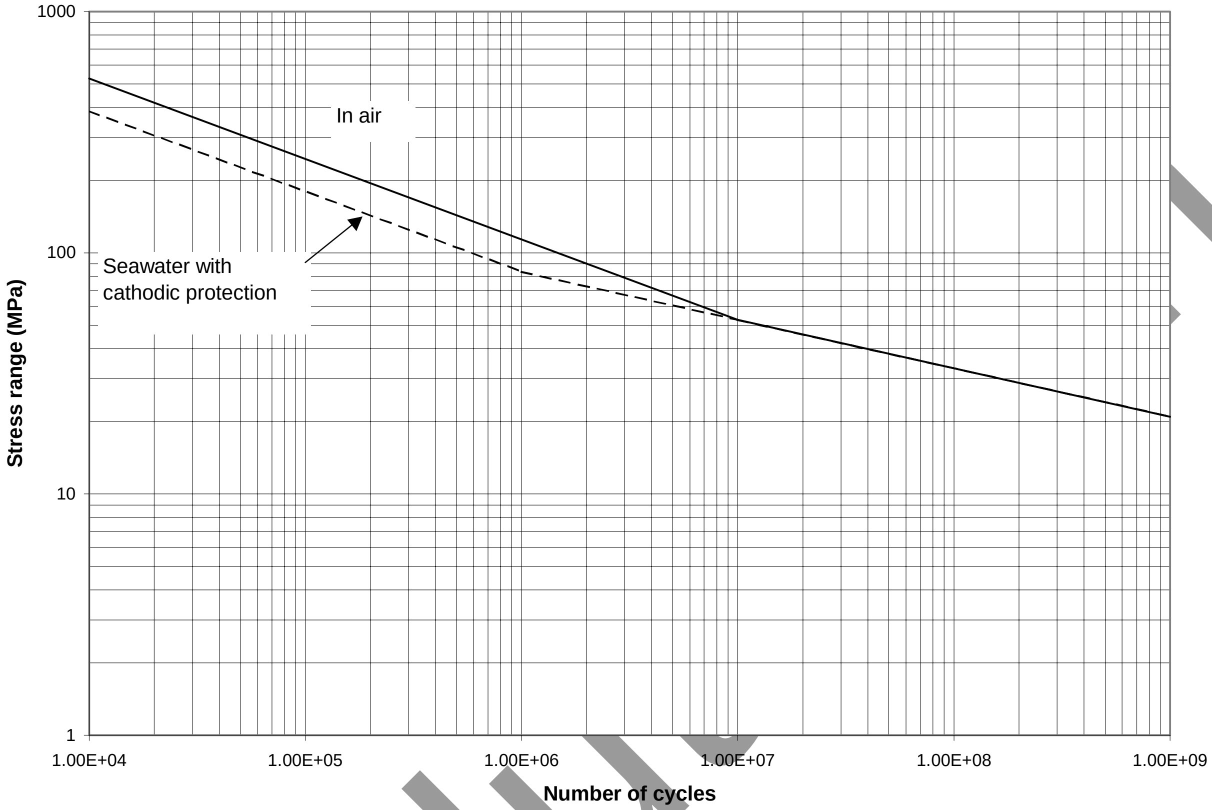


Figure C.2-5 S-N curves for tubular joints in air and in seawater with cathodic protection

C.2.3.5 S-N curves for cast nodes

It is recommended to use the C curve for cast nodes. Based on tests a more optimistic curve might be used. However, the C curve is recommended used in order to allow for weld repairs after possible defects after casting and possible fatigue cracks after some service life.

For cast nodes a reference thickness t_{ref} = 38 mm may be used provided that any possible repair welds have been ground to a smooth surface.

C.2.3.6 S-N curves for forged nodes

For forged nodes the B1 curve may be used for nodes designed with a Design Fatigue Factor equal to 10. For designs with DFF less than 10 it is recommended to use the C-curve to allow for weld repair if fatigue cracks should occur during service life.

C.2.3.7 S-N curves for free corrosion

S-N curves for free corrosion are given in Table C.2-3.

Table C.2-3 S-N curves in seawater for free corrosion

S-N curve	$\log \bar{a}$ For all cycles m = 3.0	Thickness exponent k
B1	12.436	0
B2	12.262	0
C	12.115	0.15
C1	11.972	0.15
C2	11.824	0.15
D	11.687	0.20
E	11.533	0.20
F	11.378	0.25
F1	11.222	0.25
F3	11.068	0.25
G	10.921	0.25
W1	10.784	0.25
W2	10.630	0.25
W3	10.493	0.25
T	11.687	0.25 for SCF ≤ 10.0 0.30 for SCF >10.0

C.2.4 Mean stress influence for non welded structures

For fatigue analysis of regions in the base material not significantly effected by residual stresses due to welding, the stress range may be reduced dependent on whether mean cycling stress is tension or compression. This reduction may e.g. be carried out for cut-outs in the base material. The calculated stress range obtained may be multiplied by the reduction factor f_m as obtained from Figure C.2-6 before entering the S-N curve.

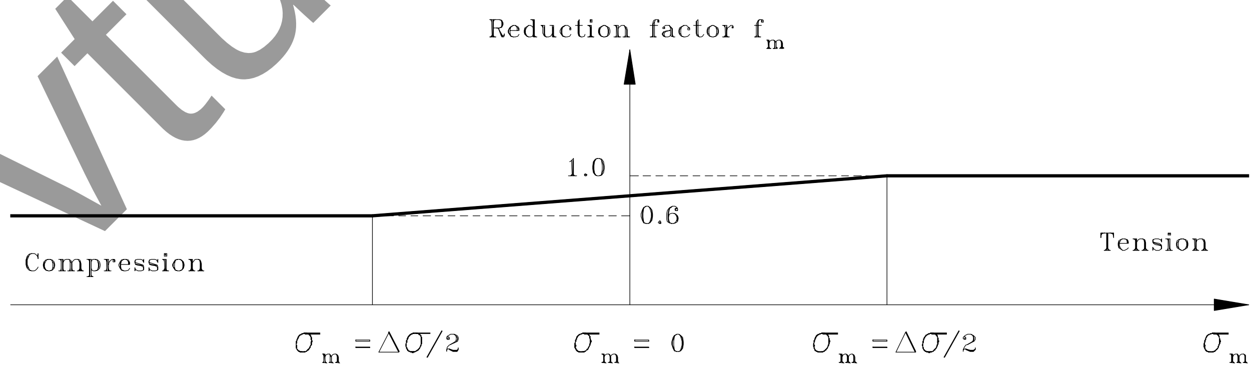


Figure C.2-6 Stress range reduction factor to be used with the S-N curve for base material

C.2.5 Effect of fabrication tolerances

Normally larger fabrication tolerances are allowed in real structures than that accounted for in the test specimens used to derive S-N data, ref. NORSOK standard M-101. Therefore, additional stresses resulting from normal fabrication tolerances should be considered included in the fatigue design. Special attention should be given to the fabrication tolerances for simple butt welds in plates and tubulars as these are giving the most significant increase in additional stress. Stress concentration factors for butt welds are given in C.2.6.1 and at tubular circumferential welds in C.2.6.3.

A stress concentration factor derived from C.2.6 should be used for connections welded from both sides. For connections welded from one side the SCF due to fabrication tolerances can be put equal one if the weld is symmetric about the root. A SCF from C.2.6 should be used if the weld is made from one side and is not symmetric about the root.

C.2.6 Stress concentration factors

C.2.6.1 Stress concentration factors for plated structures

C.2.6.1.1 General

A stress concentration factor may be defined as the ratio of hot spot stress range over nominal stress range.

The eccentricity between welded plates may be accounted for in the calculation of stress concentration factor. The following formula applies for a butt weld in an unstiffened plate or for a pipe butt weld with a large radius:

$$SCF = 1 + \frac{3\delta}{t} \tag{C.2.7}$$

where δ is eccentricity and t is plate thickness.

C.2.6.1.2 Stress concentration factors for rounded rectangular holes

Stress concentration factors for rounded rectangular holes are given in Figure C.2-7.

Where there is one stress raiser close to another detail being evaluated with respect to fatigue, the interaction of stress between these should be considered. An example of this is a welded connection in a vicinity of a hole. Then the increase in stress at the considered detail due to the hole can be evaluated from Figure C.2-8.

Some guidelines on effect of interaction of different holes may be found in Peterson's Stress Concentration Factors, Pilkey (1997)/19/.

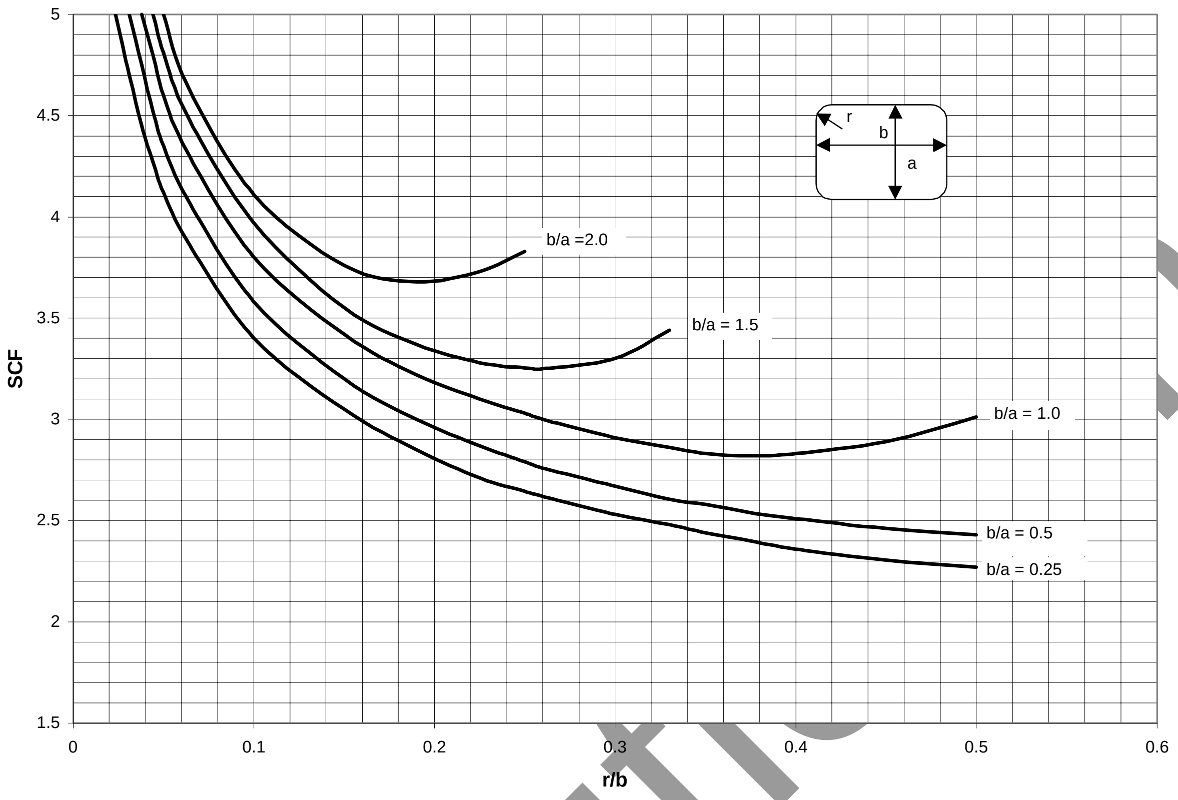


Figure C.2-7 Stress concentration factors for rounded rectangular holes

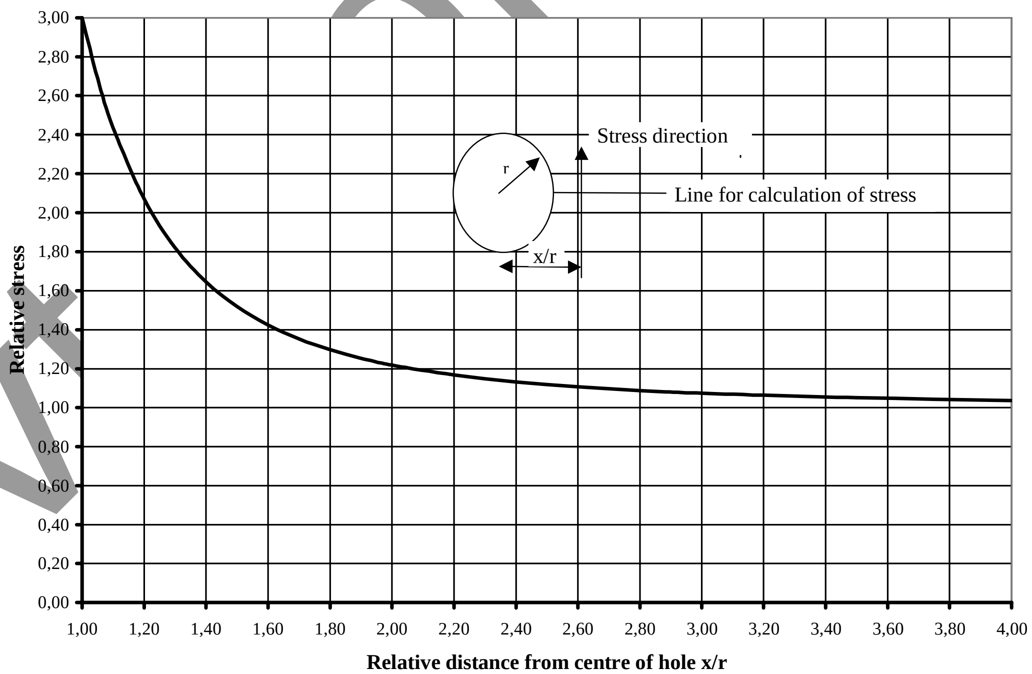


Figure C.2-8 Stress distribution at a hole

C.2.6.1.3 Stress concentration factors for holes with edge reinforcement

Stress concentration factors for holes with reinforcement are given in Appendix 3 to this Annex. The calculated hot spot stress should be used together with S-N curves assessed from welded built-up sections in Appendix 1 for stresses parallel with the weld. For stresses normal to the weld the resulting stress should be combined with the D-curve. Reference is made to Appendix 1 for selection of S-N curve to be used for evaluation of the shear stress in the weld. See also section C.2.2

C.2.6.2 Stress concentration factors for ship details

Stress concentration factors for ship details may be found in ref. /1/. S-N curve C from this annex may be used if the procedure of ref. /1/ is used to determine the notch stress. S-N curve D from this annex may be used if the procedure of ref. /1/ is used to determine the hot spot stress (Excluding the notch factor K_w from the analysis, as this factor is accounted for in the D-curve).

C.2.6.3 Stress concentration factors for tubular joints and members

C.2.6.3.1 Stress concentration factors for simple tubular joints

Stress concentration factors for simple tubular joints are given in Appendix 2 of this Annex.

C.2.6.3.2 Superposition of stresses in tubular joints

The stresses are calculated at the crown and the saddle points, see Figure C.2-9. Then the hot spot stress at these points is derived by summation of the single stress components from axial, in-plane and out of plane action. The hot spot stress may be higher for the intermediate points between the saddle and the crown. The hot spot stress at these points is derived by a linear interpolation of the stress due to the axial action at the crown and saddle and a sinusoidal variation of the bending stress resulting from in-plane and out of plane bending. Thus the hot spot stress should be evaluated at 8 spots around the circumference of the intersection, ref. Figure C.2-10.

$$\sigma_1 = SCF_{AC}\sigma_x - SCF_{MIP}\sigma_{my} \qquad \textbf{(C.2.8)}$$

$$\sigma_2 = \frac{1}{2}(SCF_{AC} + SCF_{AS})\sigma_x - \frac{1}{2}\sqrt{2}\ SCF_{MIP}\sigma_{my} + \frac{1}{2}\sqrt{2}\ SCF_{MOP}\sigma_{mz}$$

$$\sigma_3 = SCF_{AS}\sigma_x + SCF_{MOP}\sigma_{mx}$$

$$\sigma_4 = \frac{1}{2}(SCF_{AC} + SCF_{AS})\sigma_x + \frac{1}{2}\sqrt{2}\ SCF_{MIP}\sigma_{my} + \frac{1}{2}\sqrt{2}\ SCF_{MOP}\sigma_{mz}$$

$$\sigma_5 = SCF_{AC}\sigma_x + SCF_{MIP}\sigma_{my}$$

$$\sigma_6 = \frac{1}{2}(SCF_{AC} + SCF_{AS})\sigma_x + \frac{1}{2}\sqrt{2}\ SCF_{MIP}\sigma_{my} - \frac{1}{2}\sqrt{2}\ SCF_{MOP}\sigma_{mz}$$

$$\sigma_7 = SCF_{AS}\sigma_x - SCF_{MOP}\sigma_{mx}$$

$$\sigma_8 = \frac{1}{2}(SCF_{AC} + SCF_{AS})\sigma_x - \frac{1}{2}\sqrt{2}\ SCF_{MIP}\sigma_{my} - \frac{1}{2}\sqrt{2}\ SCF_{MOP}\sigma_{mz}$$

Here σ_x, σ_{my} and σ_{mz} are the maximum nominal stresses due to axial load and bending in-plane and out-of-plane respectively. SCF_{AS} is the stress concentration factor at the saddle for axial load and the SCF_{AC} is the stress concentration factor at the crown. SCF_{MIP} is the stress concentration factor for in plane moment and SCF_{MOP} is the stress concentration factor for out of plane moment.

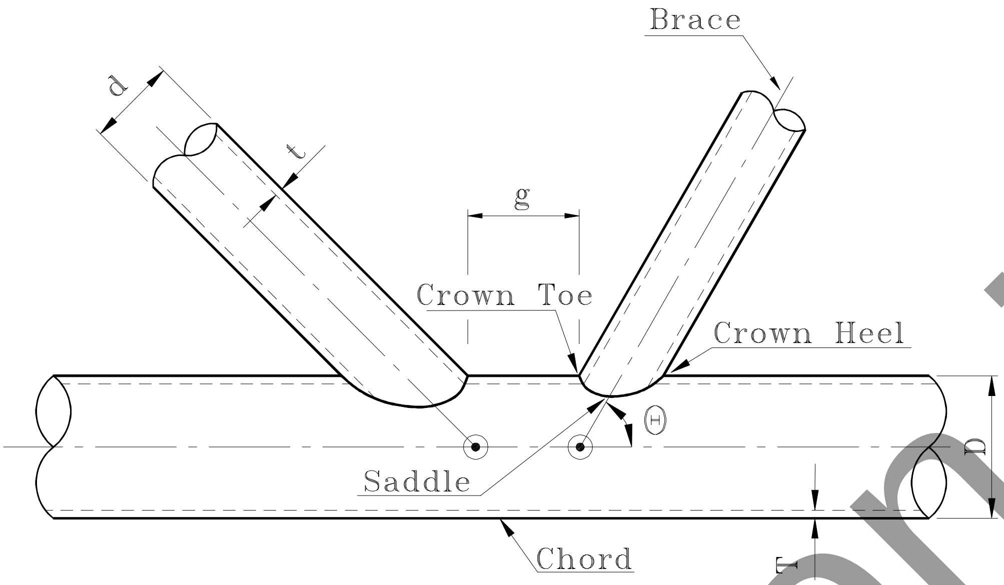
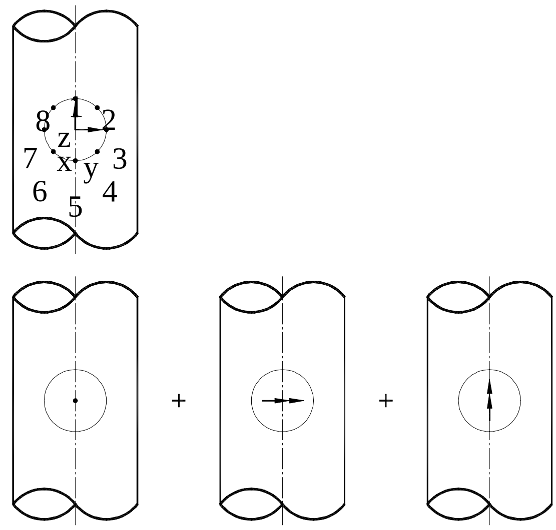


Figure C.2-9 Geometrical definitions for tubular joints



Axial load In-plane Out-of-plane

Figure C.2-10 Superposition of stresses

Influence functions may be used as an alternative to the procedure given here to calculate hot spot stress. See e.g. ref. /24/ and ref./2/.

C.2.6.3.3 Tubular joints welded from one side

The root area of single-sided welded tubular joints may be more critical with respect to fatigue cracks than the outside region connecting the brace to the chord. In such cases, it is recommended that stubs are provided for tubular joints where high fatigue strength is required, such that welding from the backside can be performed.

Failure from the root has been observed at the saddle position of tubular joints where the brace diameter is equal the chord diameter, both in laboratory tests and in service. It is likely that fatigue cracking from the root might occur for rather low stress concentrations. Thus, special attention should be given to joints other than simple joints, such as ring-stiffened joints and joints where weld profiling or grinding on the surface is required to achieve sufficient fatigue life. It should be remembered that surface improvement does not increase the fatigue life at the root.

Based on experience it is not likely that fatigue cracking from the inside will occur earlier than from the outside for simple T and Y joints and K type tubular joints. The same consideration may be made for X-joints with $\beta \leq 0.90$. For other joints and for simple tubular X-joints with $\beta > 0.90$ it is recommended that a fatigue assessment of the root area is performed. Some guidance on such an assessment can be found in the commentary.

Due to limited accessibility for in service inspection a higher design fatigue factor should be considered used for the weld root than for the outside weld toe hot spot.

Reference is also made to the commentary.

C.2.6.3.4 Stress concentration factors for stiffened tubular joints

Equations for joints for ring stiffened joints are given in ref./3/. The following points should be noted regarding the equations:

- The derived SCF ratios for the brace/chord intersection and the SCFs for the ring edge are mean values, although the degree of scatter and proposed design factors are given.
- Short chord effects shall be taken into account where relevant
- For joints with diameter ratio $\beta \geq 0.8$, the effect of stiffening is uncertain. It may even increase the SCF.
- The maximum of the saddle and crown values should be applied around the whole brace/chord intersection.

The following points can be made about the use of ring stiffeners in general:

- Thin shell FE analysis should be avoided for calculating the SCF if the maximum stress is expected to be near the brace-ring crossing point in the fatigue analysis.
- Ring stiffeners have a marked effect on the circumferential stress in the chord, but have little or no effect on the longitudinal stress.
- Ring stiffeners outside the brace footprint have little effect on the SCF, but may be of help for the static strength.
- Failures in the ring inner edge or brace ring interface occur internally, and will probably only be detected after through thickness cracking, at which the majority of the fatigue life will have been expired. These areas should therefore be considered as non-inspectable unless more sophisticated inspection methods are used.

C.2.6.3.5 Grouted tubular joints

Grouted joints have either the chord completely filled with grout (single skin grouted joints) or the annulus between the chord and an inner member filled with grout (double skin grouted joints). The SCF of a grouted joint depends on load history. The SCF is less if the bond between the chord and the grout is unbroken. For model testing of grouted joints the bond should be broken prior to SCF measurements. Due to the grout the tensile and compressive SCF may be different. The tensile value only should be used both in testing and in fatigue analysis.

The grouted joints shall be treated as simple joints, except that the chord thickness in the γ term for saddle SCF calculation for brace and chord shall be substituted with an equivalent chord wall thickness given by

$$T_e = (5D + 134T)/144 \qquad \textbf{(C.2.9)}$$

where D and T are chord diameter and thickness respectively.

Joints with high β or low γ ratios have little effect of grouting. The benefits of grouting should be neglected for joints with $\beta > 0.9$ or $\gamma \leq 12.0$ unless documented otherwise.

C.2.6.3.6 Cast nodes

It is recommended that finite element analysis should be used to determine the magnitude and location of the maximum stress range in castings sensitive to fatigue. The finite element model should use volume elements at the critical areas and properly model the shape of the joint. Consideration should be given to the inside of the castings. The brace to casting circumferential butt weld (which is designed to an appropriate S-N curve for such connections) may be the most critical location for fatigue.

C.2.6.3.7 Stress Concentration Factors for Tubular Butt Weld Connections

Due to less severe S-N curve for the outside than the inside, it is strongly recommended that tubular butt weld connections are designed such that any thickness transitions are placed on the outside (see Figure C.2-11). For this geometry, the SCF for the transition applies to the outside. On the inside it is then conservative to use SCF = 1.0. Thickness transitions are normally to be fabricated with slope 1:4.

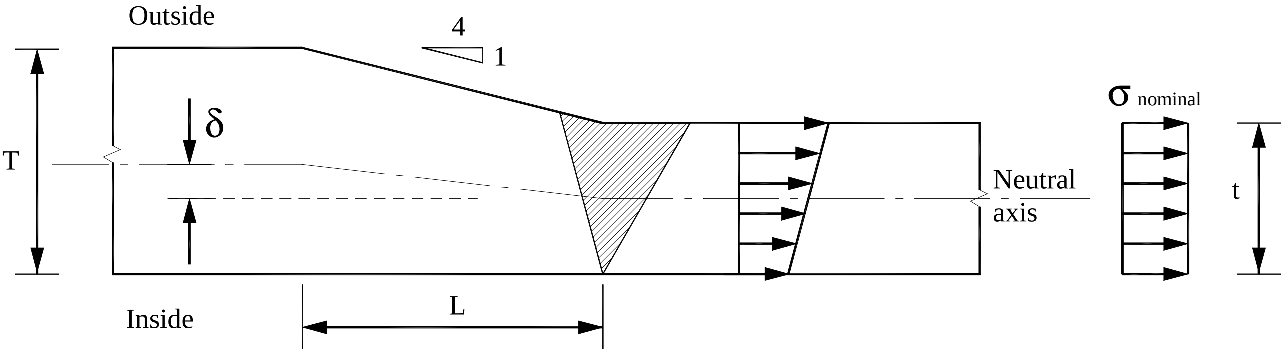


Figure C.2-11 Preferred transition in thickness is on outside of tubular butt weld

Stress concentrations at tubular butt weld connections are due to eccentricities resulting from different sources. These may be classified as concentricity (difference in tubular diameters), differences in thickness of joined tubulars, out of roundness and centre eccentricity, see Figure C.2-13 and Figure C.2-14. The resulting eccentricity may be conservatively evaluated by a direct summation of the contribution from the different sources. The eccentricity due to out of roundness normally gives the largest contribution to the resulting eccentricity δ.

It is conservative to use the formula for plate eccentricities for calculation of SCF at tubular butt welds. The effect of the diameter in relation to thickness may be included by use of the following formula:

$$SCF = 1 + \frac{6\delta}{t} \frac{1}{1+\left(\frac{T}{t}\right)^{2.5}} e^{-\alpha} \qquad \textbf{(C.2.10)}$$

where

$$\alpha = \frac{1.82L}{\sqrt{D\,t}} \frac{1}{1+\left(\frac{T}{t}\right)^{2.5}}$$

This formula also takes into account the length over which the eccentricity is distributed: L, ref. Figure C.2-11 and Figure C.2-12. The stress concentration is reduced as L is increased and/or D is reduced. It is noted that for small L and large D the last formula provides stress concentration factors that are not much lower than that of the simpler formula for plates.

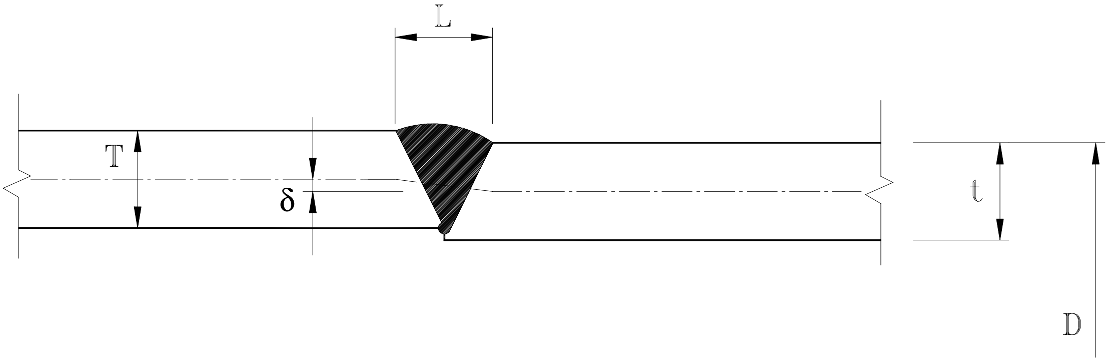


Figure C.2-12 Section through weld

The transition of the weld to base material on the outside of the tubular can normally be classified as E. If welding is performed in a horizontal position it can be classified as D.

In tubulars, the root side of welds made from one side is normally classified as F3. This requires good workmanship during construction, in order to ensure full penetration welds, and that work is checked by non-destructive examination. It may be difficult to document a full penetration weld in most cases due to limitations in the non-destructive examination technique to detect defects in the root area. The F3 curve can be considered to account for some lack of penetration, but it should be noted that a major part of the fatigue life is associated with the initial crack growth while the defects are small. This may be evaluated by fracture mechanics such as described in PD 6493. Therefore, if a fabrication method is used where lack of penetration is to be expected, the design S-N curves should be adjusted to account for this by use of fracture mechanics.

For global bending moments over the tubular section it is the nominal stress derived at the neutral axis of Figure C.2-11 that should be used together with an SCF from equation (C.2.10) for calculation of hot spot stress.

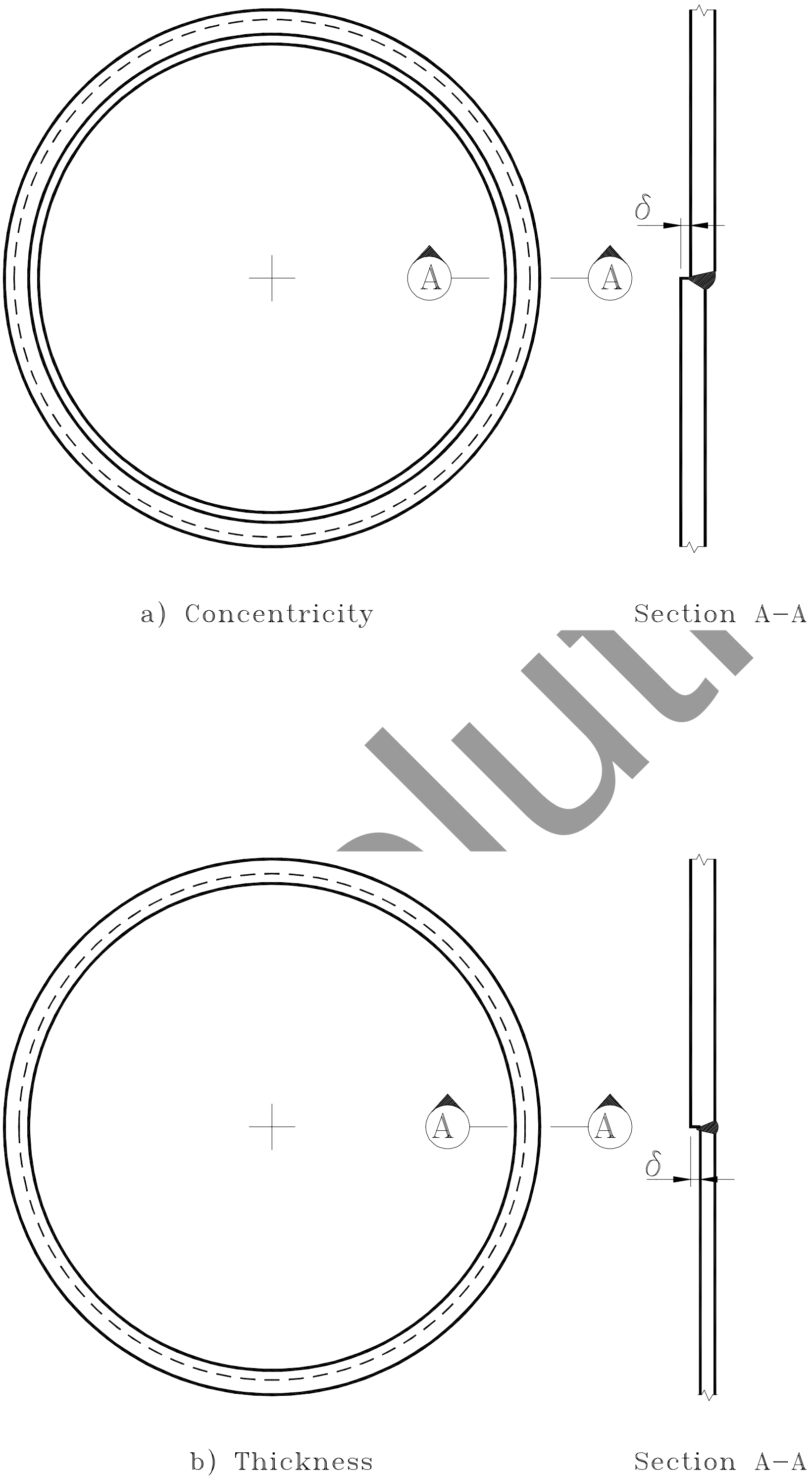


Figure C.2-13 Geometric sources of local stress concentrations in tubular butt welds

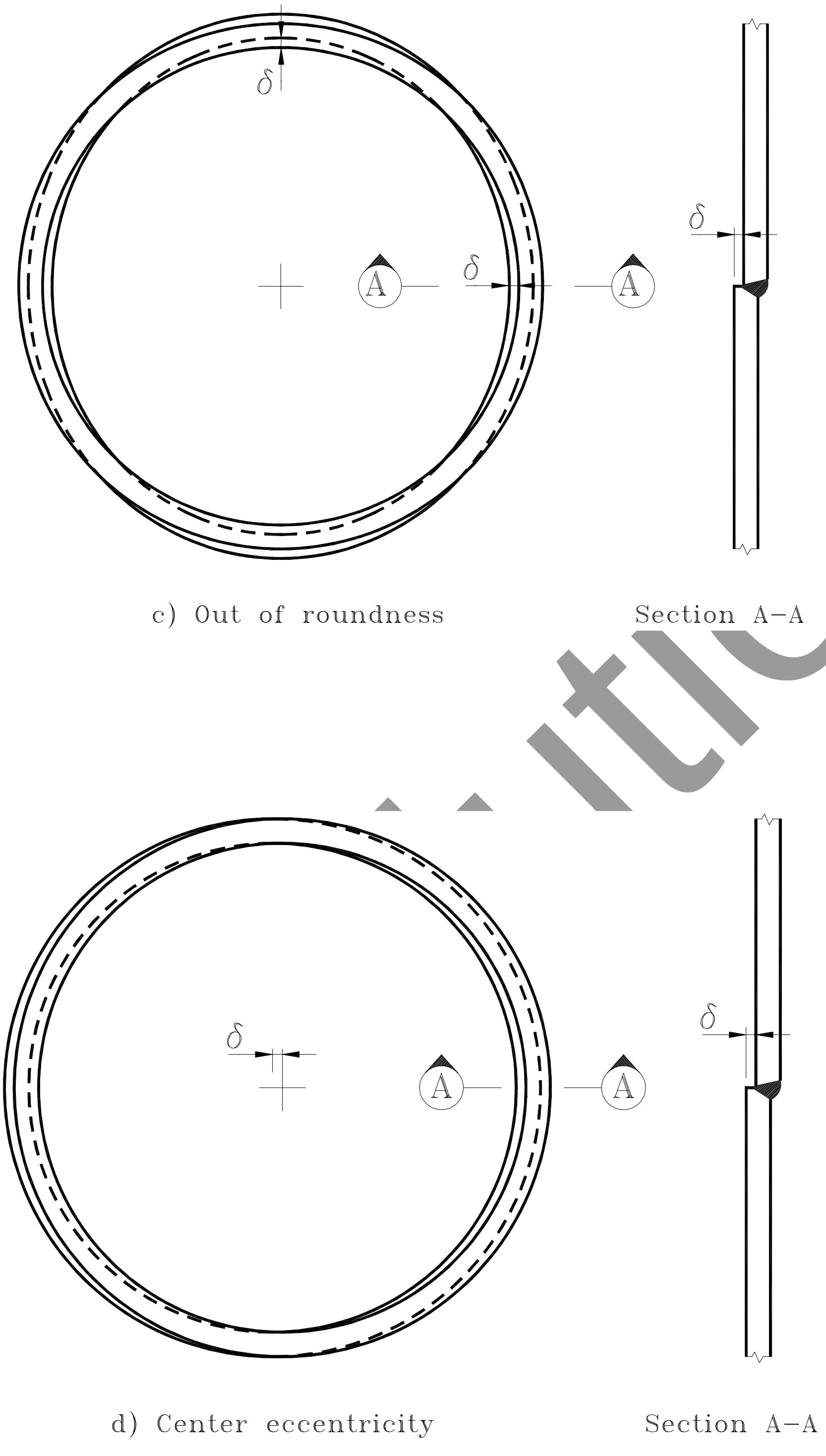


Figure C.2-14 Geometric sources of local stress concentrations in tubular butt welds

C.2.6.3.8 Stiffened shells

The stress concentration at a ring stiffener can be calculated as

$$SCF = 1 + \frac{0.54}{\alpha} \text{ for the outside}$$ **(C.2.11)**

$$SCF = 1 - \frac{0.54}{\alpha} \text{ for the inside}$$

$$\alpha = 1 + \frac{1.56t\sqrt{rt}}{A_r}$$

where

A_r	=	area of ring stiffener without effective shell.
r	=	radius of shell measured from centre to mean shell thickness
t	=	thickness of shell plating.

It can thus be noted that it is more efficient to place ring stiffeners on the inside of shell, as compared with the outside. In addition, if the shell comprises longitudinal stiffeners that is ended, it is recommended to end the longitudinal stiffeners against ring stiffeners for the inside. The corresponding combination on the outside gives a considerably larger stress concentration.

The SCF = 1.0 if continuous longitudinal stiffeners are used.

In the case of a bulkhead instead of a ring, A_r is taken as $\frac{rt_b}{(1-\nu)}$, where t_b is the thickness of the bulkhead.

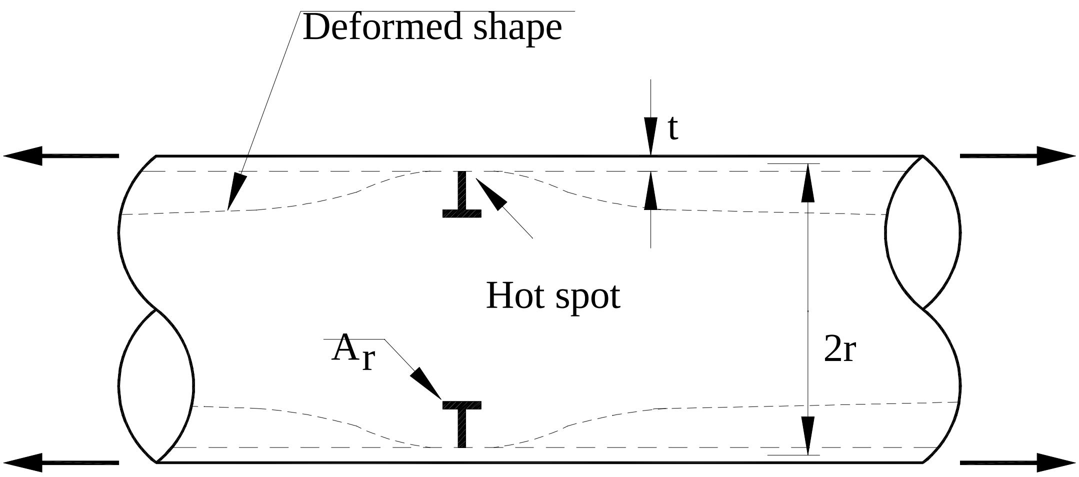


Figure C.2-15 Ring stiffened shell

C.2.6.3.9 Conical transitions

The stress concentration at each side of unstiffened tubular-cone junctions can be estimated by the following equations:

$$SCF = 1 + \frac{0.6\,t\sqrt{D_j(t + t_c)}}{t^2}\tan\alpha \qquad \textbf{(C.2.12)}$$

$$SCF = 1 + \frac{0.6\,t\sqrt{D_j(t + t_c)}}{t_c^2}\tan\alpha \qquad \textbf{(C.2.13)}$$

where

Dj	=	cylinder diameter at junction
t	=	tubular member wall thickness
tc	=	cone thickness
α	=	the slope angle of the cone (see Figure C.2-16)

The stress concentration at a junction with ring stiffener can be calculated as

$$SCF = 1 + \left(0.54 + \frac{0.91 D_j t}{A_r}\tan\alpha\right)\frac{1}{\beta} \text{ at the outside smaller diameter junction} \qquad \textbf{(C.2.14)}$$

$$SCF = 1 - \left(0.54 + \frac{0.91 D_j t}{A_r}\tan\alpha\right)\frac{1}{\beta} \text{ at the inside smaller diameter junction}$$

$$SCF = 1 + \left(0.54 - \frac{0.91 D_j t}{A_r}\tan\alpha\right)\frac{1}{\beta} \text{ at the outside larger diameter junction}$$

$$SCF = 1 - \left(0.54 - \frac{0.91 D_j t}{A_r}\tan\alpha\right)\frac{1}{\beta} \text{ at the inside larger diameter junction}$$

$$\beta = 1 + \frac{1.10\,t\sqrt{D_j t}}{A_r}$$

where

A_r = area of ring stiffener without effective shell.

If a ring stiffener is placed a distance δ away from the intersection lines, ref. Figure C.2-16, an additional stress concentration should be included to account for this eccentricity:

$$SCF = 1 + 3\frac{\delta}{t}\tan\alpha \qquad \textbf{(C.2.15)}$$

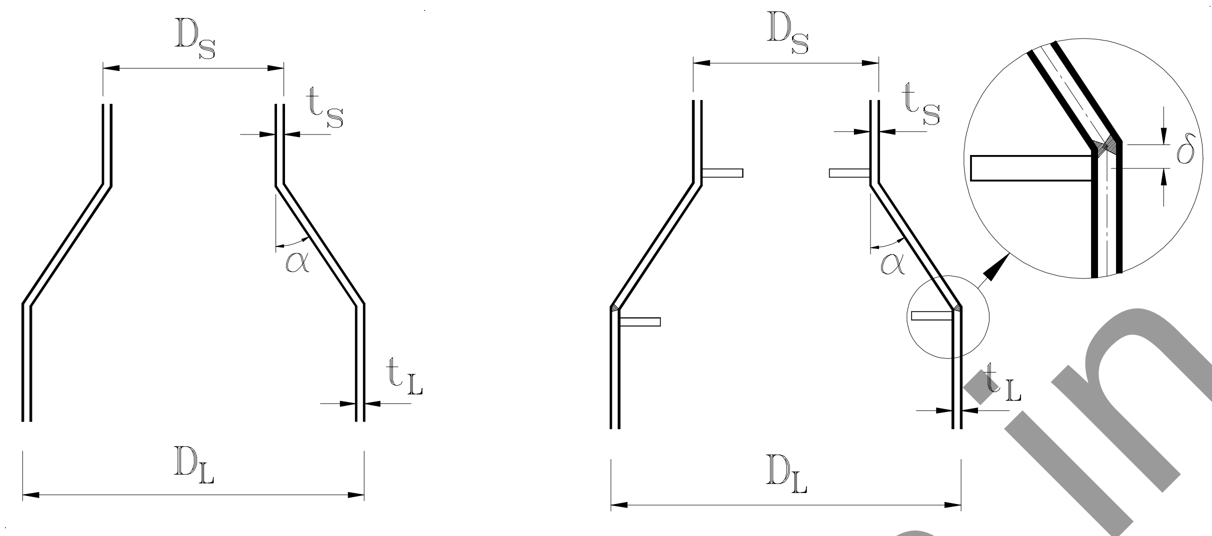


Figure C.2-16 Cone geometry

C.2.3.6.10 Stress concentration factors for tubulars subjected to axial force

This section applies to tubular sections welded together to long strings and subjected to axial tension. Tethers and risers of a TLP are examples of such structures.

The colinearity with small angle deviation between consecutive fabricated tubular segments results in increased stress due to a resulting global bending moment, see Figure C.2-17. The eccentricity due to colinearity is a function of axial tension in the tubular and is significantly reduced as the axial force is increased by tension. Assuming that the moment M results from an eccentricity δ_N where pretension is accounted for in the analysis, the following derivation of a stress concentration factor is performed:

$$\sigma = \frac{N}{\pi(D-t)t}\,SCF \qquad \textbf{(C.2.16)}$$

where the stress concentration factor is

$$SCF = 1 + \frac{4\delta_N}{D-t} \qquad \textbf{(C.2.17)}$$

where δ_N is eccentricity as function of the axial force N_{Sd} and D is outer diameter. The eccentricity for two elements is indicated in Figure C.2-18. With zero tension the eccentricity is δ. With an axial tension force N_{Sd} the eccentricity becomes:

$$\delta_N = \delta \frac{\tanh k\mathit{l}}{k\mathit{l}} \qquad \textbf{(C.2.18)}$$

where

$k = \sqrt{\frac{N_{Sd}}{EI}}$
l = segment lengths of the tubulars
N_{Sd} = axial force in tubulars

I = moment of inertia of tubulars

The formula for reduction in eccentricity due to increased axial force can be deduced from differential equation for the deflected shape of the model shown in Figure C.2-18. Thus the non-linearity in terms of geometry is included in the formula for the stress concentration factor.

Judgement should be used to evaluate the number of elements to be considered, and whether deviation from a straight line is systematic or random, ref. Figure C.2-17. In the first case, the errors must be added linearly, in the second case it may be added quadratically.

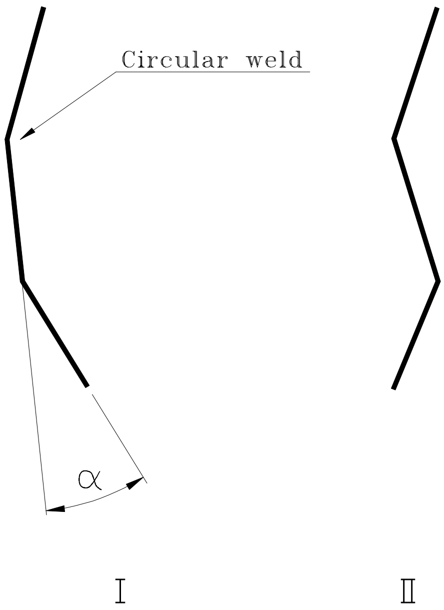


Figure C.2-17 Colinearity or angle deviation in pipe segment fabrication, I = Systematic deviation, II = random deviation

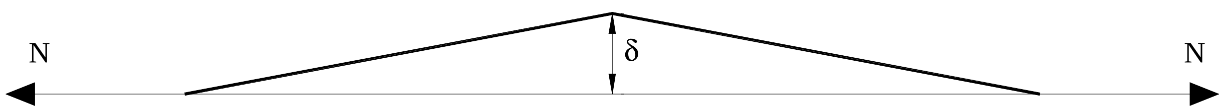


Figure C.2-18 Eccentricity due to colinearity

C.2.6.4 Stress concentration factors for joints with square sections

Stress concentration factors for T- and X- square to square joints may be found in Van Wingerde et al. (1993) /27/.

Stress concentration factors for Y- and K square to square joints for d/D_w less or equal 0.75 may be found from Soh and Soh (1992) /8/; d = depth and width of brace; D_w = depth and width of chord. These stress concentration factors may be used together with the D-curve.

The following stress concentration factors may be used for $d/D_w = 1.0$, in lieu of a more detailed analysis for calculation of hot spot stress:

Axial: 1.9
In-plane bending: 4.0
Out-of plane bending: 1.35

These stress concentration factors should be used together with the F-curve.

C.2.7 Fillet and partial penetration welds

Design should be performed such that fatigue cracking from the root is less likely than from the toe region. The reason for this is that a fatigue crack at the toe can be found by in-service inspection while a fatigue crack starting at the root can not be discovered before the crack has grown through the weld. Thus the design of the weld geometry should be performed such that the fatigue life for cracks starting at the root is longer than the fatigue life of the toe. Figure C.2-20 can be used for evaluation of required penetration. The lack of penetration, ($2a_i$), obtained from this figure may be further reduced by a factor of 0.80 in order to obtain a recommended design value for avoidance of fatigue cracking from the root. The notation used is explained by Figure C.2-19.

It should be added that it is difficult to detect internal defects by NDE in fillet/partial penetration welds. Such connections should therefore not be used in Design Class 1 and 2.

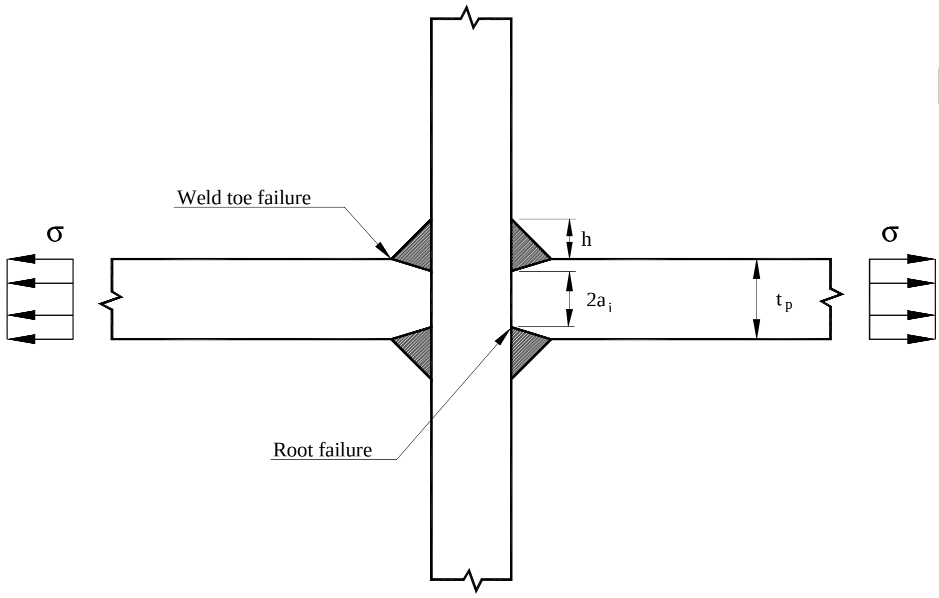


Figure C.2-19 Welded connection with partial penetration weld

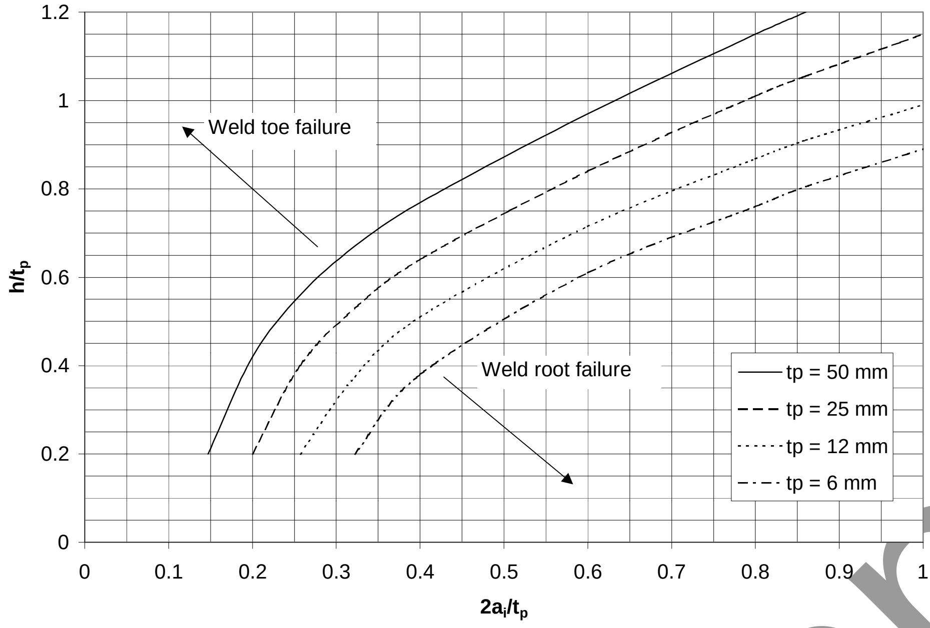


Figure C.2-20 Weld geometry with probability of root failure equal toe failure

C.2.8 Bolts

A bolted joint connection subjected to dynamic loading should be designed with pretensioned bolts. The pretension should be high enough to avoid slipping after relevant loss of pretension during service life. Connections where the pretensioned bolts are subjected to dynamic axial forces should be designed with respect to fatigue taking into account the stress range in the bolts resulting from tension and compression range. The stress range in the bolts may be assessed based on e.g. Waløen /23/ or VDI 2230 /26/.

C.2.9 Pipelines

Welds in pipelines are normally made with a symmetric weld groove with welding from the outside only. The tolerances are rather strict compared with other structural elements with eccentricity less than 0.1*t or maximum 3 mm. (t = wall thickness) The fabrication of pipelines also implies a systematic and standardised NDE of the root area where defects are most critical. Provided that the same acceptance criteria are used for pipelines with larger wall thickness as for that used as reference thickness (25 mm), a thickness exponent k = 0 may be used for hot spot at the root and k = 0.15 for the weld toe. Provided that these requirements are fulfilled, the detail at the root side may be classified as F1 with SCF = 1.0, ref. Table C.2-4. The F-curve and SCF = 1.0 may be used for welding on temporary backing, ref. Table C.2-4.

Reference is made to Table C.2-4 for other tolerances and welding from both sides.

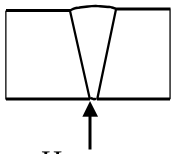
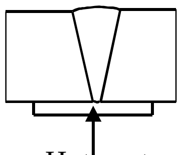
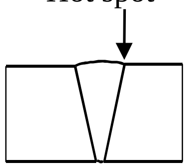
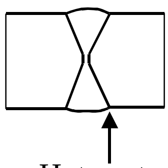
For weld grooves that are not symmetrical in shape a stress concentration due to maximum allowable eccentricity should be included. This stress concentration factor can be assessed based on the following analytical expression:

(C.2.19)

$$SCF = 1 + \frac{3\delta}{t}\exp\left(-\left(\frac{D}{t}\right)^{-0.5}\right)$$

This stress concentration factor can also be used for fatigue assessments of the weld toes, ref. also Table C.2-4

Table C.2-4 Classification of welds in pipelines

Description: Welding	Description: Geometry and hot spot	Tolerance requirement	S-N curve	Thickness exponent k	SCF
Single side	 Hot spot	$\delta \leq \min(0.1t, 3\ mm)$	F1	0.00	1.0
		$\delta > \min(0.1t, 3\ mm)$	F3	0.00	1.0
Single side on backing	 Hot spot	$\delta \leq \min(0.1t, 2\ mm)$	F	0.00	1.0
		$\delta > \min(0.1t, 2\ mm)$	F1	0.00	1.0
Single side	Hot spot 		D	0.15	Eq. (C.2.19)
Double side	 Hot spot		D	0.15	Eq. (C.2.19)

C.2.10 Calculation of hot spot stress by finite element analysis

C.2.10.1 General

From detailed finite element analysis of structures it may be difficult to evaluate what is "nominal stress" to be used together with the S-N curves, as some of the local stress due to a detail is accounted for in the S-N curve.

In most cases it may therefore be more convenient to use an alternative approach for calculation of fatigue damage when local stresses are obtained from finite element analysis.

It is realised that it is difficult to calculate the notch stress at a weld due to a significant scatter in local weld geometry and different types of imperfections. This scatter is normally more efficiently accounted for by use of an appropriate S-N curve. In this respect it should also be mentioned that the weld toe region has to be modelled with a radius in order to obtain reliable results for the notch stress.

If a corner detail with zero radius is modelled, the calculated stress will approach infinity as the element size is decreased to zero. The modelling of a relevant radius requires a very fine element mesh, increasing the size of the computer model. In addition, a proper radius to be used for the analysis will likely be a matter for discussion.

Hence, for design analysis a simplified numerical procedure is used in order to reduce the demand for large fine mesh models for the calculation of SCF factors:

- The stress concentration or the notch factor due to the weld itself is included in the S-N curve to be used, the D-curve.
- The stress concentration due to the geometry effect of the actual detail is calculated by means of a fine mesh model using shell elements (or solid elements), resulting in a geometric SCF factor.

This procedure is denoted the hot spot method.

It is important to have a continuous and not too steep, change in the density of the element mesh in the areas where the hot spot stresses are to be analysed.

The geometry of the elements should be evaluated carefully in order to avoid errors due to deformed elements (for example corner angles between 60° and 120° and length/breadth ratio less than 5).

The size of the model should be so large that the calculated results are not significantly affected by assumptions made for boundary conditions and application of loads.

C.2.10.2 Tubular joints

The stress range at the hot spot of tubular joints should be combined with the T-curve.

Analysis based on thick shell elements may be used. In this case, the weld is not included in the model. The hot spot stress may be determined as for welded connections.

More reliable results are obtained by including the weld in the model. This implies use of three-dimensional elements. Here the Gaussian points may be placed $0.1\sqrt{rt}$ from the weld toe (r = radius of considered tubular and t = thickness). The stress at this point may be used directly in the fatigue assessment.

C.2.10.3 Welded connections other than tubular joints

The stress range at the hot spot of welded connections should be combined with S-N curve D. The C-curve may be used if machining of the weld surface to the base material is performed. Then the machining has to be performed such that the local stress concentration due to the weld is removed.

The aim of the finite element analysis is not normally to calculate directly the notch stress at a detail, but to calculate the geometric stress distribution in the region at the hot spot such that these stresses can be used as a basis for derivation of stress concentration factors. Reference is made to Figure C.2-21 as an example showing the stress distribution in front of an attachment (A-B) welded to a plate with thickness *t*. The notch stress is due to the presence of the attachment and the weld. The aim of the finite element analysis is to calculate the stress at the weld toe (hot spot) due to the presence of the attachment, denoted geometric stress, $\sigma_{hot\ spot}$. The stress concentration factor due to this geometry effect is defined as,

$$SCF = \frac{\sigma_{hot\ spot}}{\sigma_{nominal}} \qquad \textbf{(C.2.20)}$$

Thus the main emphasis of the finite element analysis is to make a model that will give stresses with sufficient accuracy at a region outside that effected by the weld. The model should have a fine mesh for extrapolation of stresses back to the weld toe in order to ensure a sufficiently accurate calculation of SCF.

FEM stress concentration models are generally very sensitive to element type and mesh size. By decreasing the element size the FEM stresses at discontinuities will approach infinity. It is therefore necessary to set a lower bound for element size and use an extrapolation procedure to the hot spot to have a uniform basis for comparison of results from different computer programs and users. On the other hand, in order to pick up the geometric stress increase properly, it is important that the stress reference points in t/2 and 3t/2 (see Figure C.2-21) are not inside the same element. This implies that element sizes of the order of the plate thickness are to be used for the modelling. If solid modelling is used, the element size in way of the hot spot may have to be reduced to half the plate thickness in case the overall geometry of the weld is included in the model representation.

Element stresses are normally derived at the gaussian integration points. Depending on element type it may be necessary to perform several extrapolations in order to determine the stress at the location representing the weld toe. In order to preserve the information of the direction of principal stresses at the hot spot, component stresses are to be used for the extrapolation. When shell elements are used for the modelling and the overall geometry of the weld is not included in the model, the extrapolation shall be performed to the element intersection lines. If the (overall) weld geometry is included in the model, the extrapolation is related to the weld toe as shown in Figure C.2-21. The stresses are first extrapolated from the gaussian integration points to the plate surface. A further extrapolation to the line A - B is then conducted. The final extrapolation of component stresses is carried out as a linear extrapolation of surface stresses along line A - B at a distance t/2 and 3t/2 from either the weld toe, or alternatively the element intersection line (where t denotes the plate thickness). Having determined the extrapolated stress components at the hot spot, the principal stresses are to be calculated and used for the fatigue evaluation.

Some comments on element size is given in the Commentary.

It is recommended to perform a verification of the procedure on a detail that is S-N classified and that is similar in geometry and loading to that is going to be analysed. If the verification analysis comes out with a different SCF ($SCF_{Verification}$) than that inherent the S-N detail, ref. e.g. Table C.2-1, a resulting stress concentration factor can be calculated as

$$SCF = SCF_{Analysis} \cdot \frac{SCF_{S-N\ Table\ C.2-1}}{SCF_{Verification}}$$

where

$SCF_{S-N\ Table\ C.2-1}$	=	Stress concentration in the S-N detail as derived by the hot spot method, see Table C.2-1.
$SCF_{Analysis}$	=	Stress concentration factor for the analysed detail.

It should be noted that the hot spot concept can not be used for fatigue cracks starting from the weld root of fillet/partial penetration welds.

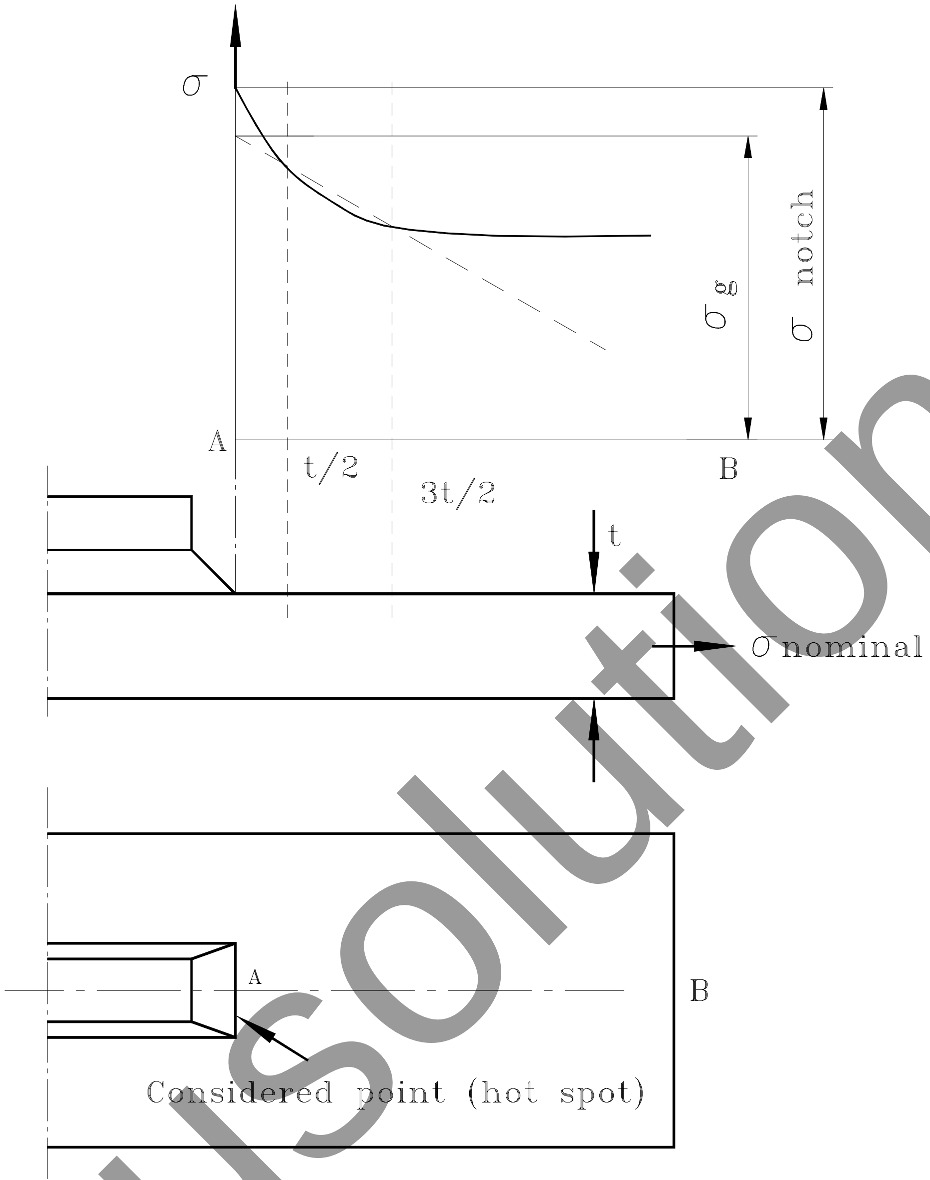


Figure C.2-21 Stress distribution at an attachment and extrapolation of stresses

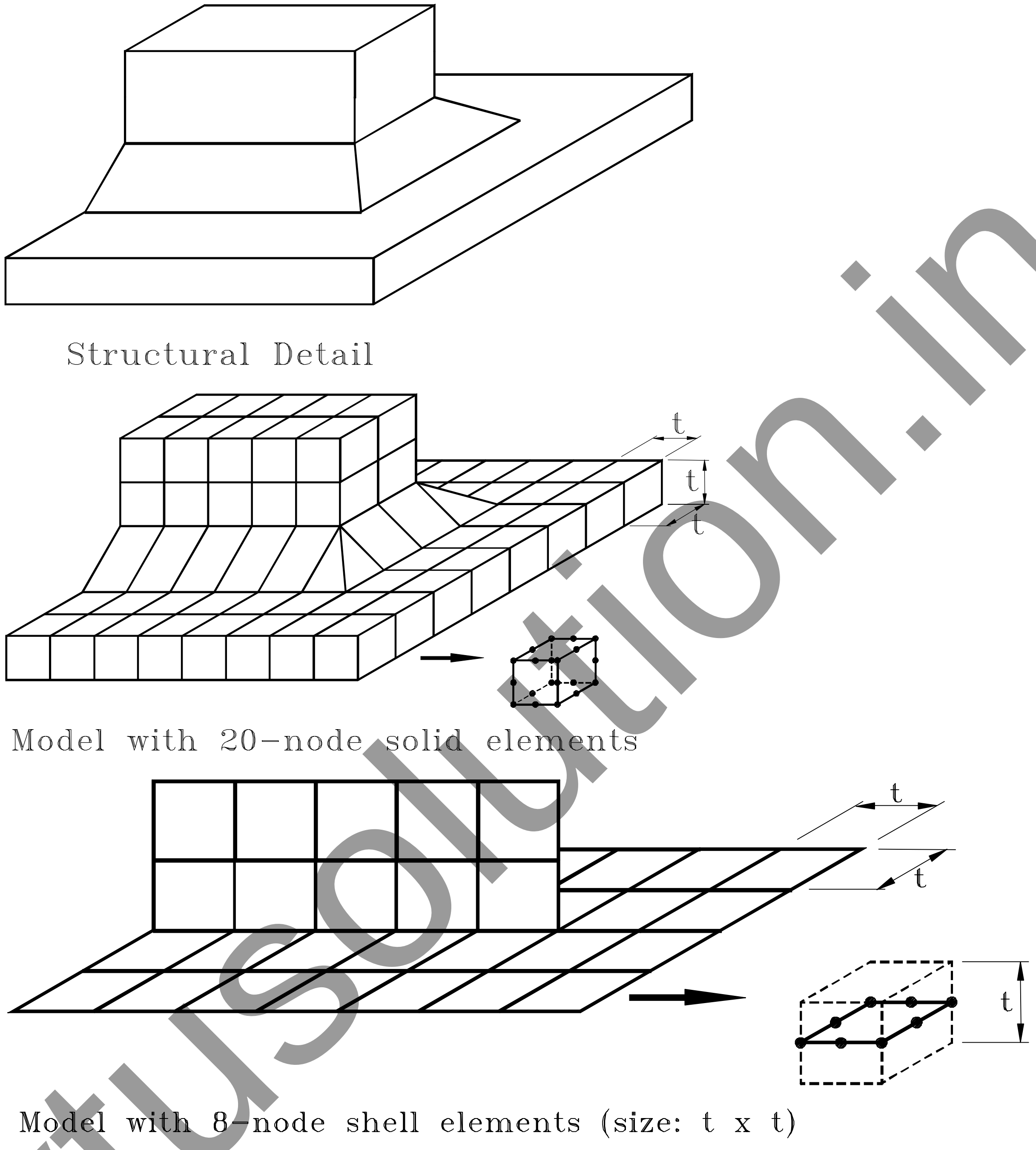


Figure C.2-22 Examples of modelling

C.2.11 Simplified fatigue analysis

C.2.11.1 General

The long term stress range distribution may be presented as a two-parameter Weibull distribution

$$Q(\Delta\sigma) = \exp\left[-\left(\frac{\Delta\sigma}{q}\right)^h\right] \qquad \textbf{(C.2.21)}$$

where

Q = probability for exceedance of the stress range $\Delta\sigma$
h = Weibull shape parameter
q = Weibull scale parameter is defined from the stress range level, $\Delta\sigma_o$, as

$$q = \frac{\Delta\sigma_0}{(\ln n_0)^{1/h}} \qquad \textbf{(C.2.22)}$$

where n_o is the total number of cycles over the time period for which the stress range level $\Delta\sigma_o$ is defined.

When the long-term stress range distribution is defined applying Weibull distributions for the different load conditions, and a one-slope S-N curve is used, the fatigue damage is given by

$$D = \frac{\nu_0 T_d}{\bar{a}} q^m \Gamma(1+\frac{m}{h}) \le \eta \qquad \textbf{(C.2.23)}$$

where

T_d = design life in seconds
h = Weibull stress range shape distribution parameter
q = Weibull stress range scale distribution parameter
ν_o = average zero-crossing frequency

$\Gamma(1+\frac{m}{h})$ = gamma function. Values of the gamma function are listed in Table C.2-5

Use of one slope S-N curves leads to results on the safe side for calculated fatigue lives (with slope of curve N < 10^6-10^7 cycles).

For other expressions for fatigue damage see commentary.

Table C.2-5 Numerical values for Γ(1+m/h)

h	m = 3.0	h	m = 3.0	h	m = 3.0
0.60	120.000	0.77	20.548	0.94	7.671
0.61	104.403	0.78	19.087	0.95	7.342
0.62	91.350	0.79	17.772	0.96	7.035
0,63	80.358	0.80	16.586	0.97	6.750
0,64	71.048	0.81	15.514	0.98	6.483
0,65	63.119	0.82	14.542	0.99	6.234
0,66	56.331	0.83	13.658	1.00	6.000
0,67	50.491	0.84	12.853	1.01	5.781
0,68	45.442	0.85	12.118	1.02	5.575
0,69	41.058	0.86	11.446	1.03	5.382
0,70	37.234	0.87	10.829	1.04	5.200
0,71	33.886	0.88	10.263	1.05	5.029
0,72	30.942	0.89	9.741	1.06	4.868
0,73	28.344	0.90	9.261	1.07	4.715
0,74	26.044	0.91	8.816	1.08	4.571
0.75	24.000	0.92	8.405	1.09	4.435
0.76	22.178	0.93	8.024	1.10	4.306

C.2.11.2 Fatigue design charts

Design charts for components in air and in seawater with cathodic protection are shown in Figure C.2-23 and Figure C.2-24 respectively. These charts have been derived based on the two slopes S-N curves given in this annex. The corresponding numerical values are given in Table C.2-6 and Table C.2-7.

These design charts have been derived based on an assumption of an allowable fatigue damage η = 1.0 during 10^8 cycles (20 years service life). For design with other allowable fatigue damages, η, the allowable stress from the design charts should be reduced by factors derived from

Table C.2-8 and Table C.2-9 for conditions in air and in seawater with cathodic protection respectively.

The stresses derived here correspond to the reference thickness. For thickness larger than the reference thickness, an allowable extreme stress range during 10^8 cycles may be obtained as

$$\sigma_{0,t} = \sigma_{0,tref}\left(\frac{t_{ref}}{t}\right)^k \tag{C.2.24}$$

where

k = thickness exponent, see C.2.3.1 and Table C.2-1.
$\sigma_{0,tref}$ = allowable stress as derived from Table C.2-6 - Table C.2-9.

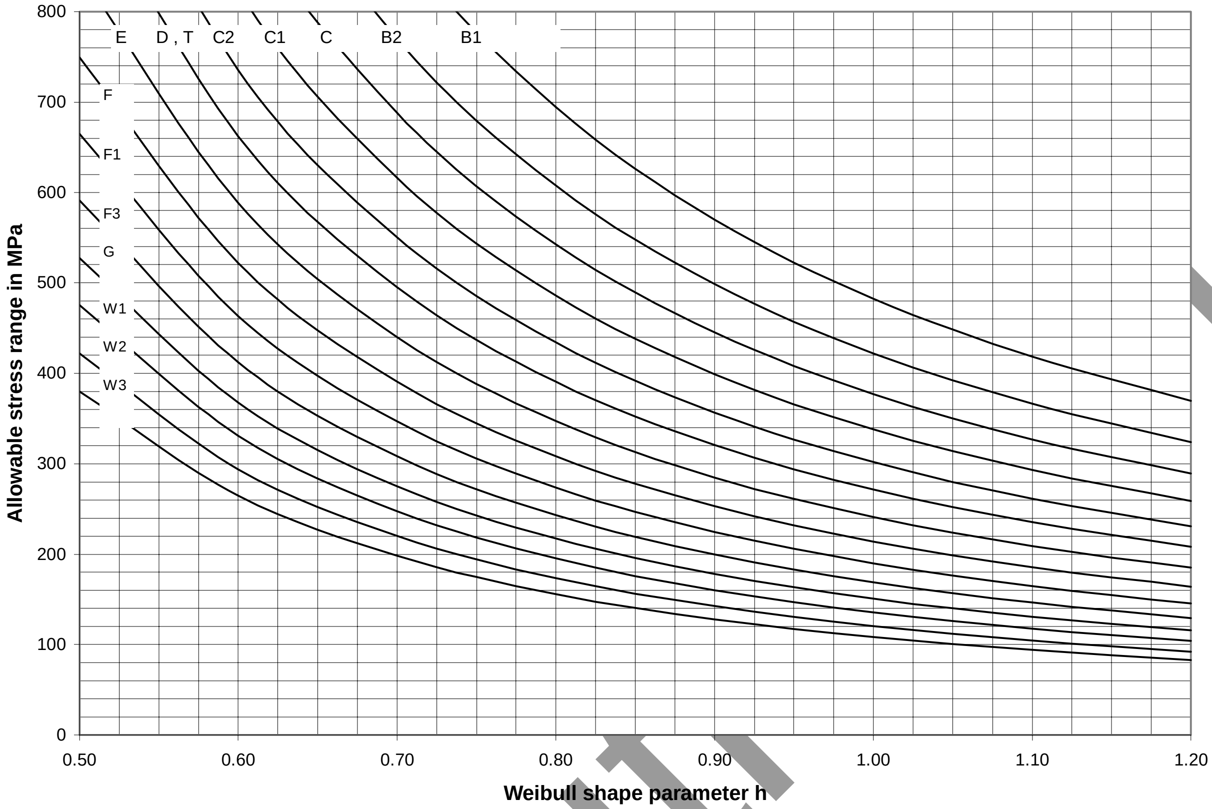


Figure C.2-23 Allowable extreme stress range during 10^8 cycles for components in air

Table C.2-6 Allowable extreme stress range in MPa during 10^8 cycles for components in air

S-N curves	**Weibull shape parameter h**							
	0.50	**0.60**	**0.70**	**0.80**	**0.90**	**1.00**	**1.10**	**1.20**
B1	1687.9	1176.7	880.5	694.5	570.1	482.5	418.4	369.8
B2	1476.9	1029.5	770.4	607.7	498.8	422.2	366.1	323.6
C	1319.3	919.6	688.1	542.8	445.5	377.2	326.9	289.0
C1	1182.0	824.0	616.5	486.2	399.2	337.8	292.9	258.9
C2	1055.3	735.6	550.3	434.1	356.3	301.6	261.5	231.1
D and T	949.9	662.1	495.4	390.7	320.8	271.5	235.4	208.1
E	843.9	588.3	440.2	347.2	284.9	241.2	209.2	184.9
F	749.2	522.3	390.8	308.2	253.0	214.1	185.6	164.1
F1	664.8	463.4	346.7	273.5	224.5	190.0	164.7	145.6
F3	591.1	412.0	308.3	243.2	199.6	169.0	146.5	129.4
G	527.6	367.8	275.2	217.1	178.2	150.8	130.8	115.6
W1	475.0	331.0	247.8	195.4	160.4	135.8	117.7	104.0
W2	422.1	294.1	220.1	173.6	142.5	120.6	104.6	92.5
W3	379.9	264.8	198.2	156.0	128.2	108.6	94.2	83.2

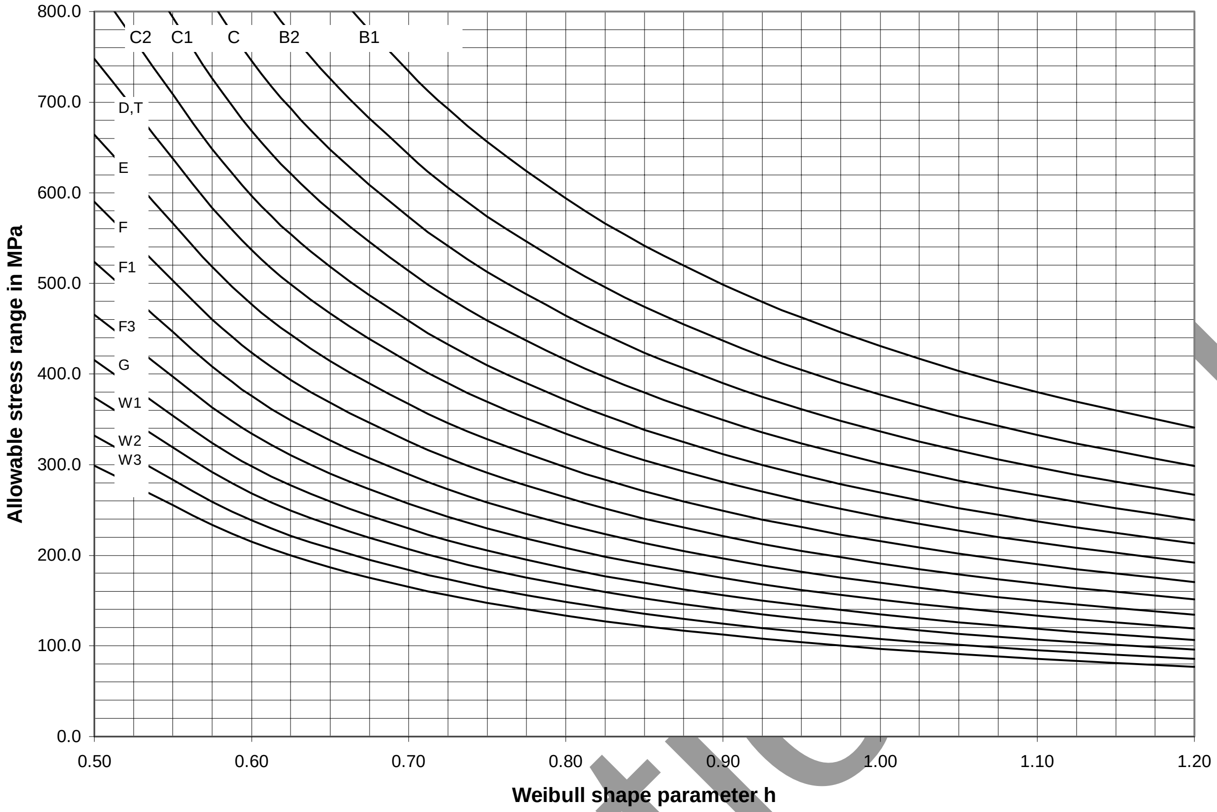


Figure C.2-24 Allowable extreme stress range during 10^8 cycles for components in seawater with cathodic protection

Table C.2-7 Allowable extreme stress range in MPa during 10^8 cycles for components in seawater with cathodic protection

S-N curves	Weibull shape parameter h							
	0.50	**0.60**	**0.70**	**0.80**	**0.90**	**1.00**	**1.10**	**1.20**
B1	1328.7	953.9	734.0	594.0	498.8	430.7	380.1	341.0
B2	1162.6	834.6	642.2	519.8	436.4	376.9	332.6	298.4
C	1038.5	745.5	573.6	464.3	389.8	336.7	297.0	266.5
C1	930.5	668.0	513.9	415.8	349.3	301.5	266.1	238.7
C2	830.7	596.3	458.7	371.3	311.7	269.2	237.6	213.1
D and T	747.8	536.7	413.0	334.2	280.7	242.4	213.9	191.9
E	664.3	476.9	367.0	297.0	249.3	215.3	190.1	170.5
F	589.8	423.4	325.8	263.6	221.4	191.1	168.6	151.3
F1	523.3	375.7	289.0	233.9	196.4	169.6	149.6	134.3
F3	465.3	334.0	257.0	208.0	174.6	150.9	133.1	119.3
G	415.3	298.2	229.4	185.7	155.9	134.6	118.8	106.6
W1	373.9	268.3	206.6	167.1	140.3	121.2	106.9	95.9
W2	332.3	238.4	183.5	148.5	124.7	107.7	95.0	85.3
W3	299.1	214.7	165.2	133.4	112.2	96.9	85.6	76.7

Table C.2-8 Reduction factor on stress for air to reduce fatigue damage utilisation to η

Fatigue damage utilisation η	Weibull shape parameter h							
	0.50	**0.60**	**0.70**	**0.80**	**0.90**	**1.00**	**1.10**	**1.20**
0.50	0.805	0.810	0.816	0.821	0.826	0.831	0.835	0.839
0.30	0.688	0.697	0.706	0.715	0.723	0.730	0.737	0.742
0.10	0.497	0.512	0.526	0.540	0.552	0.563	0.573	0.581

Table C.2-9 Reduction factor on stress for seawater with cathodic protection to reduce fatigue damage utilisation to η

Fatigue damage utilisation η	Weibull shape parameter h							
	0.50	**0.60**	**0.70**	**0.80**	**0.90**	**1.00**	**1.10**	**1.20**
0.50	0.821	0.831	0.840	0.847	0.853	0.857	0.861	0.864
0.30	0.713	0.729	0.743	0.753	0.762	0.769	0.773	0.778
0.10	0.535	0.558	0.577	0.592	0.604	0.613	0.619	0.623

C.3 FATIGUE ANALYSIS BASED ON FRACTURE MECHANICS

Fracture mechanics may be used for fatigue analyses as supplement to S-N data.

Fracture mechanics is recommended for use in assessment of acceptable defects, evaluation of acceptance criteria for fabrication and for planning in-service inspection.

The purpose of such analysis is to document, by means of calculations, that fatigue cracks, which might occur during service life, will not exceed the crack size corresponding to unstable fracture. The calculations should be performed such that the structural reliability by use of fracture mechanics will not be less than by use of S-N data. This can be achieved by performing the analysis according to the following procedure:

- Crack growth parameter C determined as mean plus 2 standard deviation.
- A careful evaluation of initial defects that might be present in the structure when taking into account the actual NDE inspection method used to detect cracks during fabrication.
- Use of geometry functions that are on the safe side.
- Use utilisation factors similar to those used when the fatigue analysis is based on S-N data.

As crack initiation is not included in the fracture mechanics approach, shorter fatigue life is normally derived from fracture mechanics than by S-N data.

In a case that the results from fracture mechanics analyses cannot directly be compared with S-N data it might be recommended to perform a comparison for a detail where S-N data are available, in order to verify the assumptions made for the fracture mechanics analyses.

The initial crack size to be used in the calculation should be considered in each case, taking account of experienced imperfection or defect sizes for various weldments, geometries, access and reliability of the inspection method. For surface cracks starting from transitions between weld/base material, a crack depth of 0.5 mm (e.g. due to undercuts and microcracks at bottom of the undercuts) may be assumed if other documented information about crack depth is not available.

It is normally, assumed that compressive stresses do not contribute to crack propagation. However, for welded connections containing residual stresses, the whole stress range should be applied. Only stress components normal to the propagation plane need to be considered.

The Paris' equation may be used to predict the crack propagation or the fatigue life:

$$\frac{da}{dN} = C(\Delta K)^m \qquad \textbf{(C.3.1)}$$

where

ΔK = $K_{max} - K_{min}$
N = Number of cycles to failure
a = crack depth. It is here assumed that the crack depth/length ratio is low (less than 1:5).
C, m = material parameters, see PD 6493 (1998)/7/.

The stress intensity factor K may be expressed as

$$K = \sigma g \sqrt{\pi a} \qquad \textbf{(C.3.2)}$$

where

σ = nominal stress in the member normal to the crack
g = factor depending on the geometry of the member and the crack.

See PD 6493 (1998) for further guidelines related to fatigue assessment based on fracture mechanics.

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C.4 IMPROVEMENT OF FATIGUE LIFE BY FABRICATION

C.4.1 General

It should be noted that improvement of the toe will not improve the fatigue life if fatigue cracking from the root is the most likely failure mode. The considerations made in the following are for conditions where the root is not considered to be a critical initiation point. The effect from different improvement methods as given in the following can not be added.

C.4.2 Weld profiling by machining or grinding

By weld profiling in this section is understood profiling by machining or grinding as profiling by welding only is not concidered to be an efficient mean to improve fatigue lives.

In design calculations, the thickness effect may be reduced to an exponent 0.15 provided that the weld is profiled by either machining or grinding to a radius of approximately half the plate thickness, (T/2 with stress direction as shown in Figure C.4-1).

Where weld profiling is used, the fatigue life can be increased by a factor of 2.

As an alternative to including a factor 2 increase to fatigue life one may take account of a reduced local stress concentration factor achieved by weld profiling. Some guidance on such evaluation may be found in ref. /15/.

C.4.3 Grinding

Where local grinding of the weld toes below any visible undercuts is performed the fatigue life may be increased by a factor of 2. In addition the thickness effect may be reduced to an exponent k = 0.20. Reference is made to Figure C.4-1. Grinding a weld toe tangentially to the plate surface, as at A, will produce only little improvement in strength. To be efficient, grinding should extend below the plate surface, as at B, in order to remove toe defects. Grinding is normally carried out by a rotary burr. The treatment should produce a smooth concave profile at the weld toe with the depth of the depression penetrating into the plate surface to at leaset 0.5 mm below the bottom of any visible undercut (see Figure C.4-1). The undercut should not be larger than that the maximum required grinding depth should not exceed 2 mm or 10% of the plate thickness, whichever is smaller.

In general grinding has been used as an efficient method for reliable fatigue life improvement after fabrication. Grinding also improves the reliability of inspection after fabrication and during service life. However, experience indicate that it may be a good design practice to exclude this factor at the design stage. The designer is advised to improve the details locally by other means, or to reduce the stress range through design and keep the possibility of fatigue life improvement as a reserve to allow for possible increase in fatigue loading during the design and fabrication process, see also section 8.1.

It should also be noted that if grinding is required to achieve a specified fatigue life, the hot spot stress is rather high. Due to grinding a larger fraction of the fatigue life is spent during the initiation of fatigue cracks, and the crack grows faster after initiation. This implies use of shorter inspection intervals during service life in order to detect the cracks before they become dangerous for the integrity of the structure.

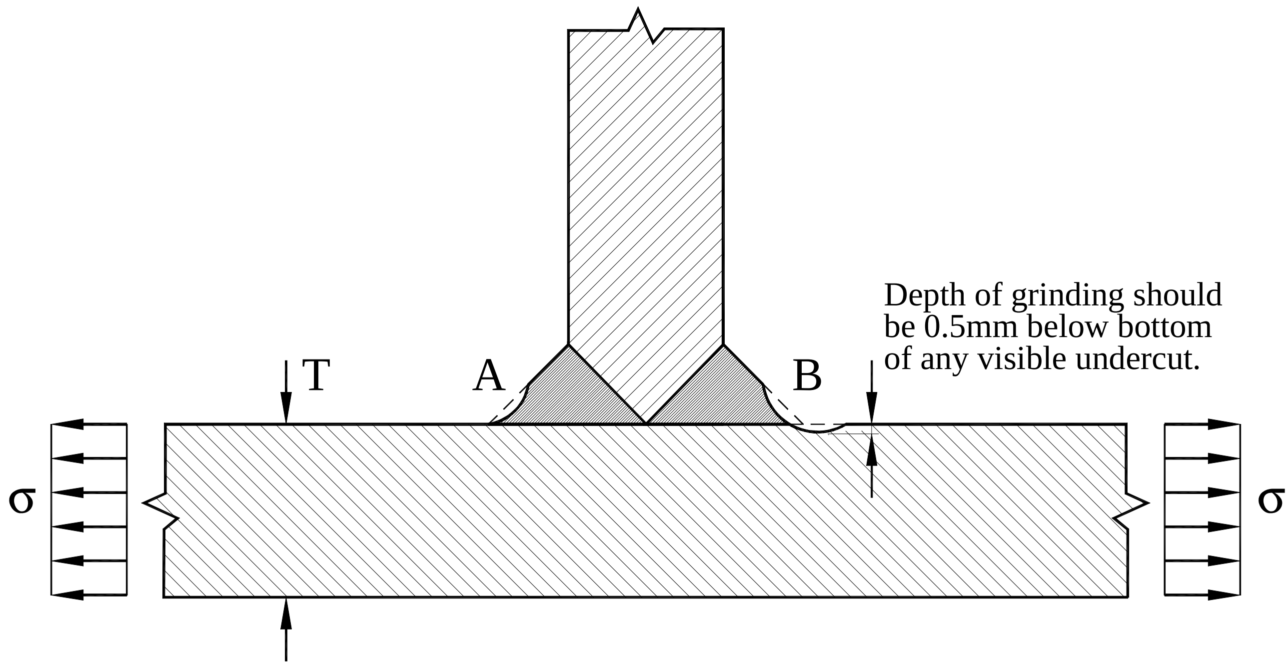

Figure C.4-1 Grinding of welds

C.4.4 TIG dressing

The fatigue life may be improved by a factor 2 by TIG dressing.

Due to uncertainties regarding quality assurance of the welding process, this method may not be recommendable for general use at the design stage.

C.4.5 Hammer peening

The fatigue life may be improved by a factor of 4 by means of hammer peening.

However, the following limitations apply:

- Hammer peening should only be used on members where failure will be without substantial consequences, ref. Chapter 10 of this standard.
- Hammer peening may only be used when minimum load of predominant load ranges is compressive or zero.
- Small numbers of tensile overloads should not be considered damaging, since tensile overloads themselves introduce compressive residual stresses at cracks.
- Overload in compression must be avoided, because the residual stress set up by hammer peening will be destroyed.
- Peening must include 4 passes at weld toe.
- Peening tip must be small enough to reach weld toe.

Due to uncertainties regarding quality assurance of the process, this method may not be recommendable for general use at the design stage.

C.5 UNCERTAINTIES IN FATIGUE LIFE PREDICTION

Large uncertainties are normally associated with fatigue life assessments. Reliability methods may be used to illustrate the effect of uncertainties on probability of a fatigue failure. An example of this is shown in Figure C.5-1 based on mean expected uncertainties for a jacket design from /17/. As the calculated probability of failure is sensitive to assumptions made for the analysis it may be recommendable to evaluate the calculated reliability values in a relative sense. Using Figure C.5-1 in this way, it might be concluded that a design modification to achieve a longer calculated fatigue life is an efficient means to reduce probability of a fatigue failure.

The effect of scatter in S-N data may be illustrated by Figure C.5-2 where the difference between calculated life is shown for mean S-N data and design S-N data (which is determined as mean minus 2 standard deviations).

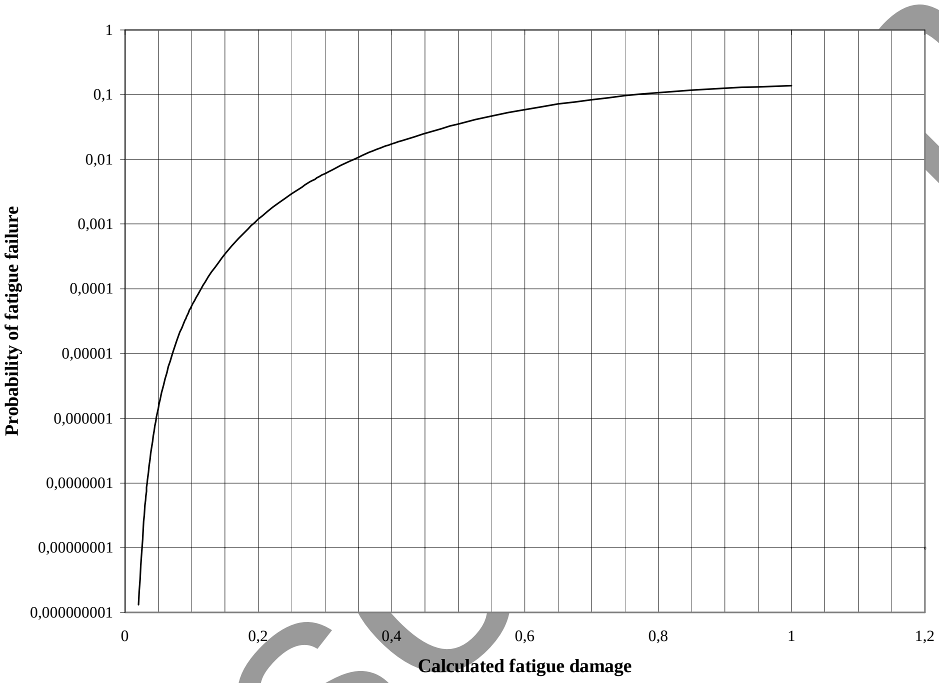

Figure C.5-1 Calculated probability of fatigue failure as function of calculated damage

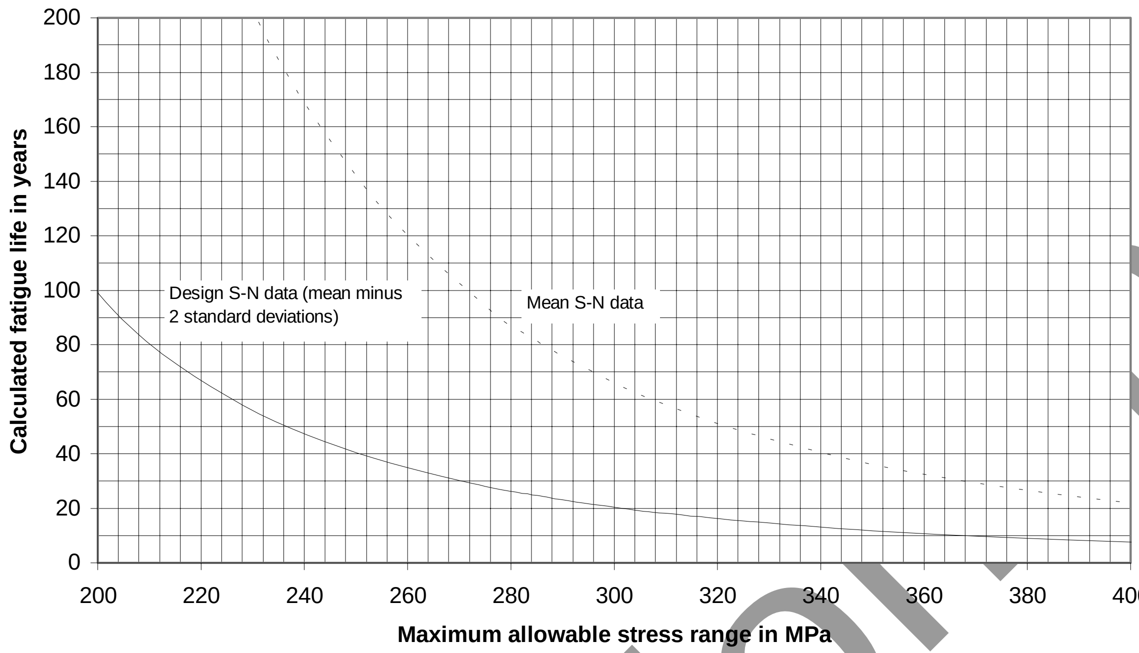

Figure C.5-2 Effect of scatter in S-N data on calculated fatigue life

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C.7 COMMENTARY

Comm. C.1.3 Methods for fatigue analysis

Important part of action history

The contribution to fatigue damage for different regions of an Weibull distribution is shown in Figure C.7-1 for a fatigue damage equal 1.0 (and 0.5) for a 20-year period. The calculation is based on a Weibull long term stress range distribution with h = 1.0 (in the range that is typical for a semisubmersible and an S-N curve with slope m = 3.0 for N <10^7 for and m = 5.0 for N > 10^7 cycles (Typical S-N curve for air condition).

It is noted that the most important part of the long-term stress range is for actions having a probability of exceedance in the range 10^{-3} to 10^{-1}. This corresponds to logn = 5-7 in Figure C.7-1.

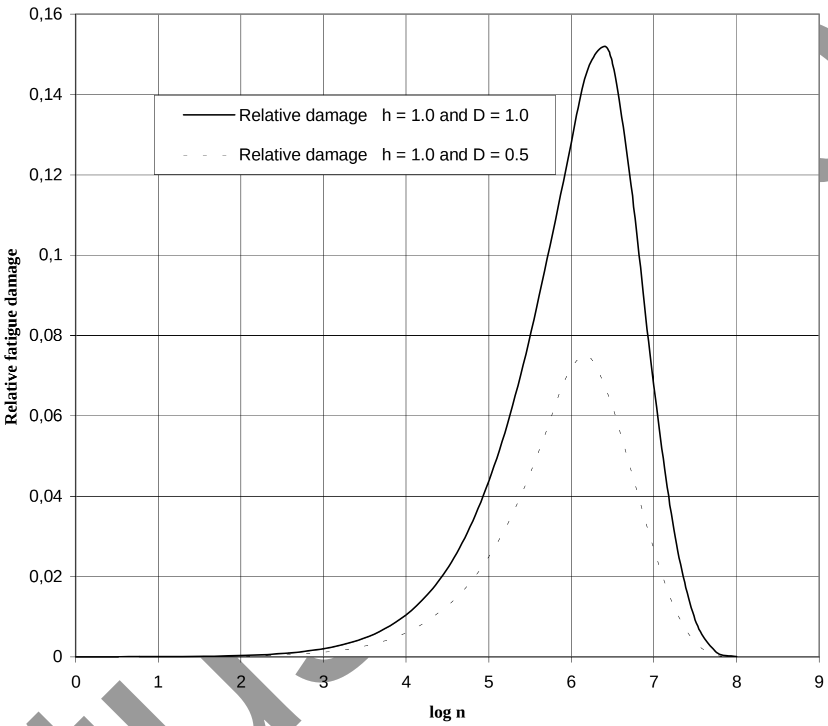

Figure C.7-1 Relative fatigue damage in Webull distribution of stress ranges

Comm. C.2.11 Simplified fatigue analysis

Weibull distributed Stress Range and Bilinear S-N curves

When a bi-linear or two-slope S-N curve is used, the fatigue damage expression is given by

$$D = \nu_0 T_d \left[\frac{q^{m_1}}{\bar{a}_1} \Gamma\left(1+\frac{m_1}{h}; \left(\frac{S_1}{q}\right)^h\right) + \frac{q^{m_2}}{\bar{a}_2} \gamma\left(1+\frac{m_2}{h}; \left(\frac{S_1}{q}\right)^h\right)\right] \le \eta \quad \textbf{(C.7.1)}$$

where

S_1 = Stress range for which change of slope of S-N curve occur

$\bar{a}_1, m_1$ = S-N fatigue parameters for N < 10^7 cycles (air condition)

$\bar{a}_2, m_2$ = S-N fatigue parameters for N > 10^7 cycles (air condition)

$\gamma(\ )$ = Incomplete Gamma function, to be found in standard tables

$\Gamma(\ ;\ )$ = Complementary Incomplete Gamma function, to be found in standard tables

Alternatively the damage may be calculated by a direct integration of damage below each part of the bilinear S-N curves:

$$D = \int_0^{S_1} \frac{n_0 f(S, \Delta\sigma_0, h)}{N_2(S)} dS + \int_{S_1}^{\Delta\sigma_0} \frac{n_0 f(S, \Delta\sigma_0, h)}{N_1(S)} dS \quad \textbf{(C.7.2)}$$

Where the density Weibull function is given by

$$f(S, \Delta\sigma_0, h) = h \frac{S^{h-1}}{q(\Delta\sigma_0, h)^h} \exp\left(-\left(\frac{S}{q(\Delta\sigma_0, h)}\right)^h\right) \quad \textbf{(C.7.3)}$$

S-N curves for air condition is assumed such that the crossing point of S-N curves is here at 10^7 cycles. The stress range corresponding to this number of cycles is

$$S_1 = \left(\frac{\bar{a}_1}{10^7}\right)^{\frac{1}{m_1}} \quad \textbf{(C.7.4)}$$

The left part of S-N curve is described by notation 1, while the right part is described by notation 2.

Short term Rayleigh distribution and linear S-N curve

When the long term stress range distribution is defined through a short term Rayleigh distribution within each short term period for the different loading conditions, and a one-slope S-N curve is used, the fatigue criterion reads,

$$D = \frac{\nu_0 T_d}{\bar{a}} \Gamma\left(1+\frac{m}{2}\right) \cdot \sum_{i=1, j=1}^{\substack{\text{all seastates} \\ \text{all headings}}} r_{ij} \left(2\sqrt{2 m_{0ij}}\right)^m \le \eta \quad \textbf{(C.7.5)}$$

where

r_{ij} = the relative number of stress cycles in short-term condition i, j

ν_o = long-term average zero-crossing-frequency (H_Z)
m_{oij} = zero spectral moment of stress response process

The Gamma function, $\Gamma(1+\frac{m}{2})$ is equal to 1.33 for m = 3.0.

When a bi-linear or two-slope S-N curve is applied, the fatigue damage expression is given as,

$$D = \nu_0 T_d \sum_{i=1,\, j=1}^{\substack{\text{all seastates} \\ \text{all headings}}} r_{ij} \left[\frac{(2\sqrt{2m_{0ij}})^{m_1}}{\bar{a}_1} \Gamma\left(1+\frac{m_1}{2}; \left(\frac{S_0}{2\sqrt{2m_{0ij}}}\right)^2\right) + \frac{(2\sqrt{2m_{0ij}})^{m_2}}{\bar{a}_2} \gamma\left(1+\frac{m_2}{2}; \left(\frac{S_0}{2\sqrt{2m_{0ij}}}\right)^2\right) \right]$$

Comm. C.2.3 S-N curves

Size effect

The size effect may be explained by a number of different parameters:

- Thickness of plate – which is explained by a more severe notch with increasing plate thickness at the region where the fatigue cracks are normally initiated.
- Attachment length – which is explained by a more severe notch stress due to more flow of stress into a thick attachment than a short.
- Volume effect – which for surface defects can be explained by increased weld length and therefore increased possibility for imperfections that can be initiated into fatigue cracks.

It might be added that some authors group all these 3 effects into one group of “thickness effect” or size effect. In this annex, the thickness exponent is assumed to cover the first item in the list above and partly the second, although also an increased attachment length reduces the S-N class as shown in Appendix 1 of this Annex. Examples of the third effect and how it can be accounted for in an actual design is explained in more detail in the following. Reference may also be made to/12//19/ and /18/ for more background and explanation of the thickness effect.

Test specimens used for fatigue testing are normally smaller than actual structural components used in structures. The correspondence in S-N data depends on the stress distribution at the hot spot region. For traditional tubular joints there is one local hot spot region, while at e. g. circumferential welds of TLP tethers there is a length significantly longer than in the test specimens having the similar order of stress range. Crack growth is normally initiated from small defects at the transition zone from weld to base material. The longer the weld, the larger is the probability of a large defect. Thus, a specimen having a long weld region is expected to have a shorter fatigue life than a short weld. This can be accounted for in an actual design by probabilistic analysis of a series system, ref. e. g. Madsen et al. (1986). Weld length in a tether system is one example where such analysis should be considered used to achieve a reliable fatigue design. A mooring line consisting of chains is another example where reliability methods may be used to properly account for the size effect in addition to that prescribed in this document.

For threaded bolts, the stress concentration at the root of the threads is increasing with increasing diameter. Based on fatigue tests, it is recommended to use k = 0.40 which can be assumed to include size effects both due to the notch itself, and due to increased length of notch around circumference with increased diameter. The thickness exponent may be less for rolled threads. Thus for purpose made bolts with large diameters, it may be recommendable to perform testing of some bolts to verify a fatigue capacity to be used for design. It should be remembered that the design S-N data is obtained as mean minus 2 standard deviation in a log S-log N diagram.

S-N curves

The relationship between S-N curves in this document and those given by Eurocode 3 for air environment is given in Table C.7-1. It should be noted that the correspondence between S-N curves in this document and Eurocode 3 relates only to number of cycles less than $5*10^6$.

Table C.7-1 NORSOK notation in relation to Eurocode 3

NORSOK notation	Eurocode 3 notation
B1	160
B2	140
C	125
C1	112
C2	100
D	90
E	80
F	71
F1	63
F3	56
G	50
W1	45
W2	40
W3	36
T	

Comm. C.2.6 Stress concentration factors

Reference is made to /9/ for further background on this procedure of calculating a resulting hot spot stress from superposition of stress components.

The formula for SCF at a tubular butt weld can be outlined based on theory for thin walled structures, /25/.

Comm. C.2.6.3.3 Tubular joints welded from one side

The fatigue design of the root area of tubular joints welded from one side may be considered as follows:

- Lack of penetration is hard to control by non-destructive examination and it is considered more difficult to detect possible defects at a root area of a tubular joint welded from one side, than for a butt weld welded from one side.
- For butt welds welded from one side the joint may be classified as F3. The defect size inherent in this curve is less than 1-2 mm (This defect size may be evaluated by fracture mechanics calculations and the calculated value will depend on plate thickness. A long defect should be considered here with the defect size measured in the thickness direction of the tubular). Defect sizes up to 5 mm may be present without being detected even with a detailed examination of the root of a tubular joint. A factor for reduction of fatigue life due to a possible large root defect in a tubular joint compared to a butt weld may be evaluated based on fracture mechanics analysis.

- The stress field at the root may be derived from a finite element analysis. The crack growth may be assumed to be normal to the direction of the maximum principal stress. The fatigue life is first calculated for an initial defect size corresponding to that of the F3 curve: F(Life a_i = 1 mm). Then the fatigue life is calculated for an initial defect size corresponding to that of a tubular joint welded from one side: F(Life a_i = 5 mm). The fatigue life reduction factor, R, is obtained from eq. (C.7.7).
- A modified S-N curve below F3 is calculated from eq. (C.7.8). An S-N curve corresponding to this log a value (or below) may now be used for fatigue life analysis based on nominal stress at the root as calculated by a detailed finite element analysis.
- Fatigue cracking from the root is harder to discover by in service inspection than crack growth from the toe. Therefore, an additional factor on fatigue life should be considered for crack growth from the root, see Chapter 10 of this standard.

$$R = \frac{F(\text{Life } a_i = 5\,\text{mm})}{F(\text{Life } a_i = 1\,\text{mm})} \qquad \textbf{(C.7.6)}$$

$$\log a = 11.546 + \log(R) \qquad \textbf{(C.7.7)}$$

The following simplified approach for fatigue life assessment of the weld root may be performed as an alternative procedure:

- As above an additional factor on fatigue life should be considered used for crack growth from the root.
- Normally the stress on the outside of the brace at the hot spot is larger than at the root area. Hence it is considered to be conservative to use the brace SCF for evaluation of fatigue life at the root. As an approximation the SCF for the inside can be calculated as

$$SCF_{inside} = SCF_{brace} - 2.0 \qquad \textbf{(C.7.8)}$$

- The fatigue life for the root may now be calculated using the W3 curve.

This procedure is applicalbe for simple tubular joints only.

Comm. C.2.10 Calculation of hot spot stress by finite element analysis

This procedure has been tested against known target values for a typical F detail, and a cut-out in a web for longitudinal transfer, ref./4/. Acceptable results were achieved using 8-node shell elements and 20 node volume elements with size equal the thickness. However, this methodology is experience based, and caution is advisable when using it for details which differ from those for which it was calibrated.

Based on experience with use of different finite element programs, it is, however, considered difficult to arrive at a firm calculation procedure that can be applied in general for all type of details. It is therefore recommended that the analysis method is tested against a well known detail, or that the fatigue detail should be modelled by a very fine element mesh including the weld notch to determine a target for the analysis, prior to use for fatigue assessment.

Comm. C.4 Improvement of fatigue life by fabrication

Reference is made to /16/ for effect of weld improvements on fatigue life. Reference is also made to /22/ for effect of weld profiling on thickness effect. Reference is made to /13/ for fatigue of welded joints peened underwater.

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DESIGN OF STEEL STRUCTURES

ANNEX C, APPENDIX 1
CLASSIFICATION OF STRUCTURAL DETAILS

Annex C, Appendix 1
Classification of structural details

Table 1 Non-welded details

Notes on potential modes of failure

In plain steel, fatigue cracks will initiate at the surface, usually either at surface irregularities or at corners of the cross-section. In welded construction, fatigue failure will rarely occur in a region of plain material since the fatigue strength of the welded joints will usually be much lower. In steel with boltholes or other stress concentrations arising from the shape of the member, failure will usually initiate at the stress concentration. The applied stress range shall include applicable stress concentration factors arising from the shape of the member.

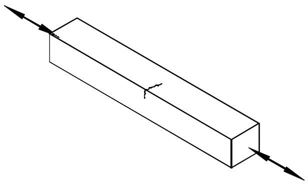
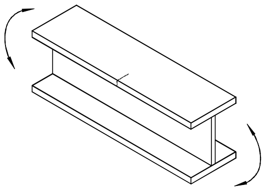
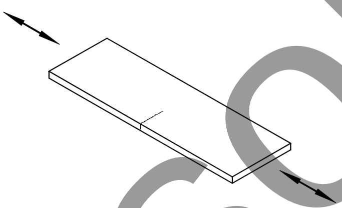
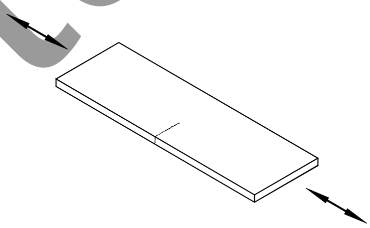
Detail category	Constructional details	Description	Requirement
B1	1.  2. 	1. Rolled or extruded plates and flats 2. Rolled sections	1. to 2.: - Sharp edges, surface and rolling flaws to be improved by grinding. - For members who can acquire stress concentrations due to rust pitting etc. is curve C required.
B2	3. 	3. Machine gas cut or sheared material with no drag lines	3. - All visible signs of edge discontinuities should be removed. - No repair by weld refill. - Re-entrant corners (slope <1:4) or aperture should be improved by grinding for any visible defects. - At apertures the design stress area should be taken as the net cross-section area.
C	4. 	4. Manually gas cut material or material with machine gas cut edges with shallow and regular draglines.	4. - Subsequently dressed to remove all edge discontinuities - No repair by weld refill. - Re-entrant corners (slope <1:4) or aperture should be improved by grinding for any visible defects. - At apertures the design stress area should be taken as the net cross-section area.

Table 2 Bolted connections

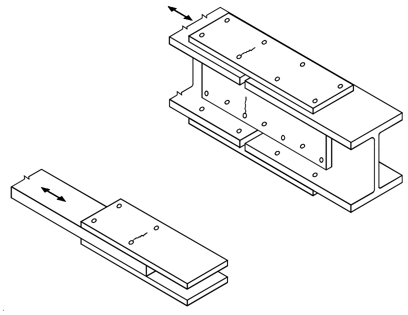
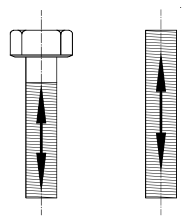
Detail category	Constructional details	Description	Requirement
C1	1., 2. 	1. Unsupported one-sided connections shall be avoided or else effects eccentricities taken into account when calculating stresses. 2. Beam splices or bolted cover plates.	1. and 2.: - Stresses to be calculated in the gross section. - Bolts subjected to reversal forces in shear shall be designed as a slip resistant connection and only the members need to be checked for fatigue.
	3. 	3. Bolts and threaded rods in tension.	3.: - Tensile stresses to be calculated using the tensile stress area of the bolt. - For preloaded bolts, the stress-range in the bolt depends upon the level of preload and the geometry of the connection, see e.g. Waløen; Maskindeler 2.
F1		Cold rolled threads with no following heat treatment like hot galvanising.	
W3		Cut threads.	

Table 3 Continuous welds essentially parallel to the direction of applied stress

Notes on potential modes of failure.

With the excess weld material dressed flush, fatigue cracks would be expected to initiate at weld defect locations. In the as welded condition, cracks may initiate at start-stop positions or, if these are not present, at weld surface ripples.

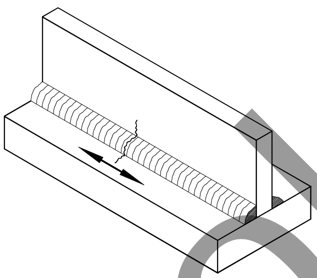
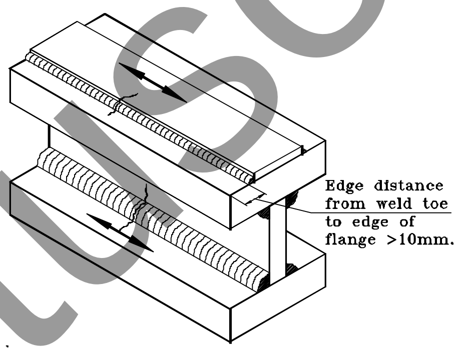
General comments

a) Backing strips

If backing strips are used in these joints, they must be continuous. If they are attached by welding, such welds must also comply with the relevant joint classification requirements (note particularly that tack welds, unless subsequently ground out or covered by a continuous weld, would reduce the joint to class F)

b) Edge distance

An edge distance criterion exists to limit the possibility of local stress concentrations occurring at unwelded edges as a result, for example, of undercut, weld spatter, or accidental overweave in manual fillet welding (see also notes in Table 7). Although an edge distance can be specified only for the “width” direction of an element, it is equally important to ensure that no accidental undercutting occurs on the unwelded corners of, for example cover plates or box girder flanges. If undercutting occurs it should subsequently be ground smooth.

Detail category	Constructional details	Description	Requirement
C	1.  2. 	1. Automatic butt welds carried out from both sides. If a specialist inspection demonstrates that longitudinal welds are free from significant flaws, category B2 may be used. 2. Automatic fillet welds. Cover plate ends shall be verified using detail 5. in Table 8.	1. and 2.: - No start-stop position is permitted except when the repair is performed by a specialist and inspection carried out to verify the proper execution of the repair.

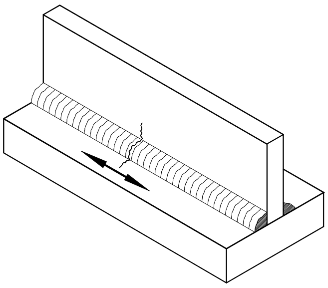
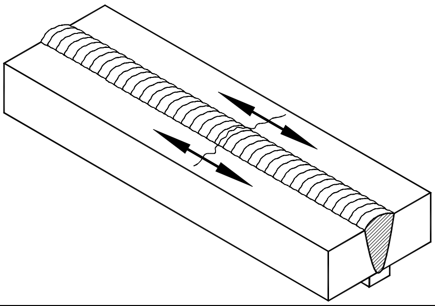
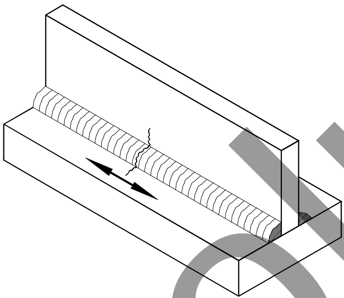
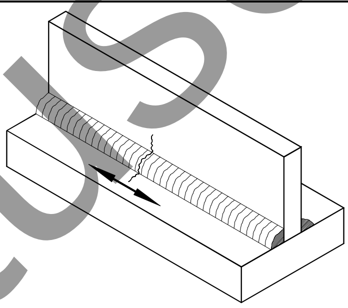
Detail category	**Constructional details**	**Description**	**Requirement**
C1	3.  4. 	3. Automatic fillet or butt welds carried out from both sides but containing stop-start positions. 4. Automatic butt welds made from one side only, with a backing bar, but without start-stop positions.	4.: - When the detail contains start-stop positions use category C2
C2		5. Manual fillet or butt welds. 6. Manual or automatic butt welds carried out from one side only, particularly for box girders.	6.: - A very good fit between the flange and web plates is essential. Prepare the web edge such that the root face is adequate for the achievement of regular root penetration with out brake-out.
C2		7. Repaired automatic or manual fillet or butt welds.	7.: - Improvement methods that are adequately verified may restore the original category.

Table 4 Intermittent welds and welds at cope holes

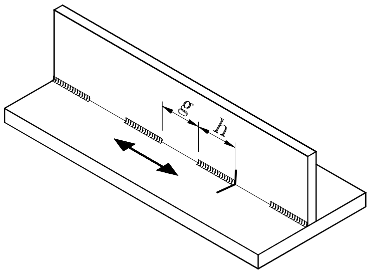
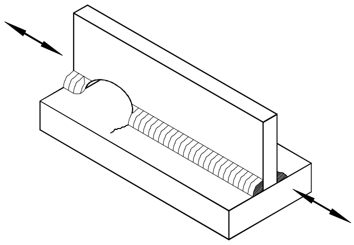
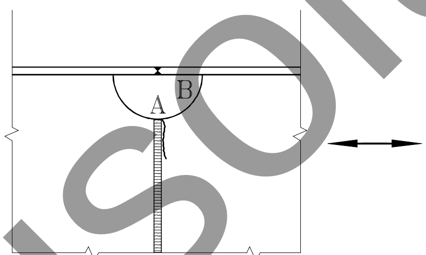
Detail category	Constructional details	Description	Requirement
E	1. 	1. Stitch or tack welds not subsequently covered by a continuous weld.	1.: - Intermittent fillet weld with gap ratio g/h ≤ 2.5.
F	2. 	2. Ends of continuous welds at cope holes.	2.: - Cope hole not to be filled with weld material.
	3. 	3. Cope hole and transverse butt weld.	3.: - For butt weld in material with cope hole may advice on fatigue assessment be found in ref /1/. - The SCF (or K-factor) from ref. /1/ may be used together with the C curve.

Table 5 Transverse butt welds, welded from both sides

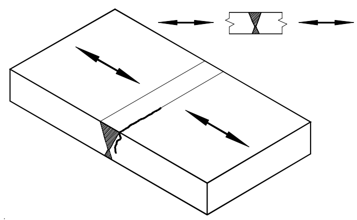
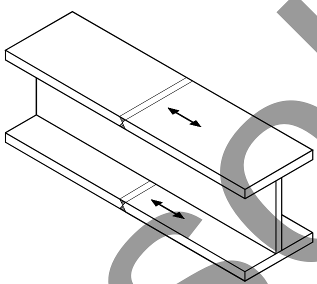
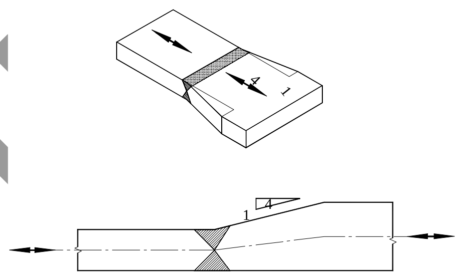
Notes on potential modes of failure

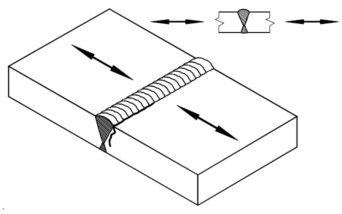
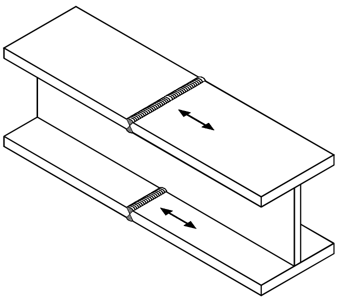
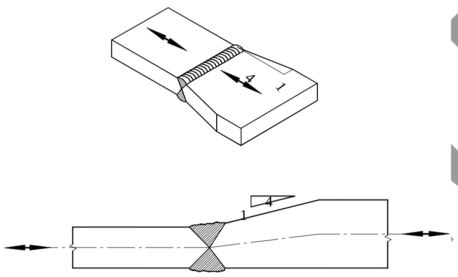
With the weld ends machined flush with the plate edges, fatigue cracks in the as-welded condition normally initiate at the weld toe, so that the fatigue strength depends largely upon the shape of the weld overfill. If the overfill is dressed flush, the stress concentration caused by it is removed, and failure is then associated with weld defects.

Design stresses

In the design of butt welds that are not symmetric about the root and not are aligned, the stresses must include the effect of any eccentricity (see section C2.5 and C2.6 in Annex C).

With connections that are supported laterally, e.g. flanges of a beam that are supported by the web, eccentricity may be neglected.

Detail category	Constructional details	Description	Requirement
C1	1.  2.  3. 	1. Transverse splices in plates flats and rolled sections. 2. Flange splices in plate girders. 3. Transverse splices in plates or flats tapered in width or in thickness where the slope is not greater than 1:4.	1. and 2.: - Details 1. and 2. may be increased to Category C when high quality welding is achieved and the weld is proved free from significant defects by non-destructive examination (it is assumed that this is fulfilled by inspection category A). 1., 2. and 3.: - All welds ground flush to plate surface parallel to direction of the arrow. - Weld run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - All welds welded in horizontal position in shop.

Detail category	Constructional details	Description	Requirement
D	4.  5.  6.  	4. Transverse splices in plates and flats. 5. Transverse splices in rolled sections or welded plate girders. 6. Transverse splices in plates or flats tapered in width or in thickness where the slope is not greater than 1:4.	4., 5. and 6.: - The height of the weld convexity to be not greater than 10% of the weld width, with smooth transitions to the plate surface. - Welds made in flat position in shop. - Weld run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress.

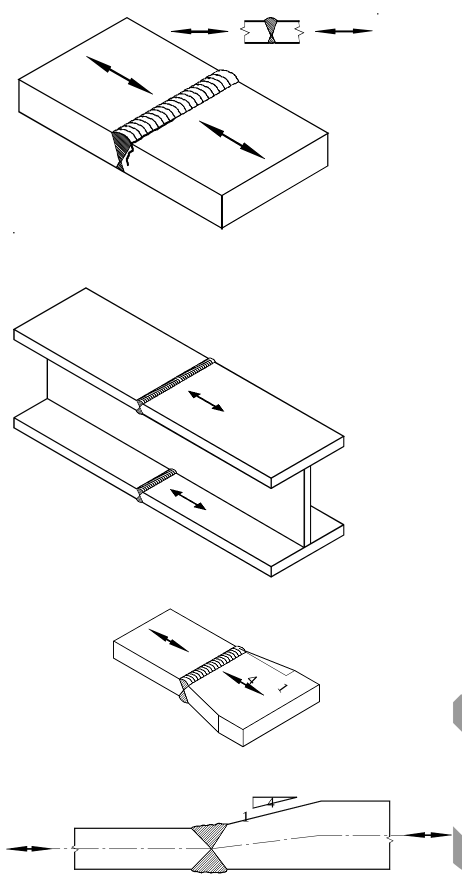
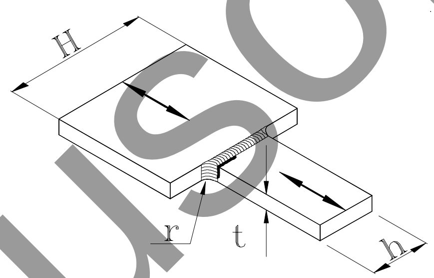
Detail category	Constructional details	Description	Requirement
E	7. 	7. Transverse splices in plates, flats, rolled sections or plate girders made <u>at site</u>.	7.: - The height of the weld convexity to be not greater than 20% of the weld width. - Weld run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress.
	8. 	8. Transverse splice between plates of unequal width, with the weld ends ground to a radius.	8.: - The stress concentration has been accounted for in the joint classification. - The width ratio H/h should be less than 2.
F1	$\frac{r}{h} \geq 0.16$		
F3	$\frac{r}{h} \geq 0.11$		

Table 6 Transverse butt welds, welded from one side

Notes on potential modes of failure

With the weld ends machined flush with the plate edges, fatigue cracks in the as-welded condition normally initiate at the weld toe, so that the fatigue strength depends largely upon the shape of the weld overfill. If the overfill is dressed flush, the stress concentration caused by it is removed, and failure is then associated with weld defects. In welds made on permanent backing strip, fatigue cracks most likely initiate at the weld metal/strip junction.

Design stresses

In the design of butt welds that are not symmetric about the root and not are aligned, the stresses must include the effect of any eccentricity (see section C2.5 and C2.6 in Annex C).

With connections that are supported laterally, e.g. flanges of a beam that are supported by the web, eccentricity may be neglected.

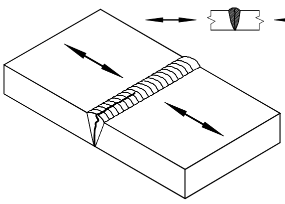
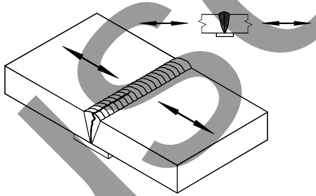
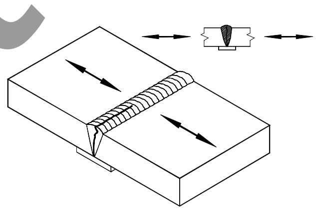
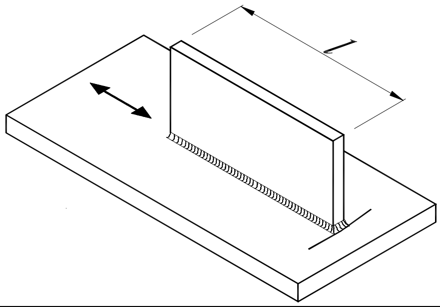
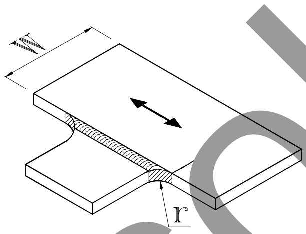
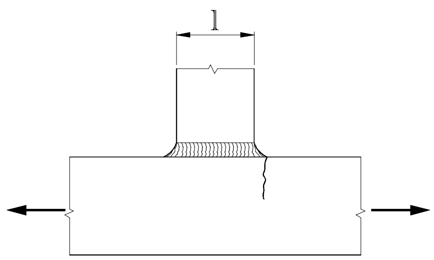
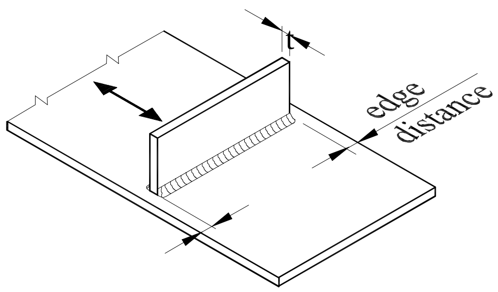
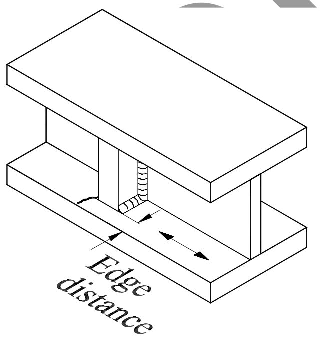
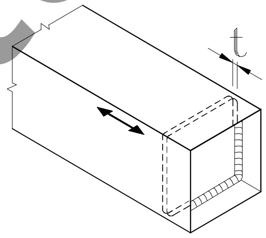
Detail category	**Constructional details**	**Description**	**Requirement**
W3	1. 	1. Butt weld made from one side only and without backing strip.	1.: With the root proved free from defects larger than 1-2mm (in the thickness direction) by non-destructive testing, detail 1 may be recategorised to F3 (it is assumed that this is fulfilled by inspection category A). If it is likely that larger defects may be present after the inspection the detail may be downgraded from F3 based on fatigue life calculation using fracture mechanics. The analysis should then be based on a relevant defect size.
F	2. 	2. Transverse butt weld on a permanent backing strip without fillet welds.	
G	3. 	3. Transverse butt weld on a backing strip fillet welded to the plate.	

Table 7 Welded attachments on the surface or the edge of a stressed member

Notes on potential modes of failure

When the weld is parallel to the direction of the applied stress, fatigue cracks normally initiate at the weld ends. When the weld is transverse to direction of stressing, cracks usually initiate at the weld toe; for attachments involving a single, as opposed to a double, weld cracks may also initiate at the weld root. The cracks then propagate into the stressed member. When the welds are on or adjacent to the edge of the stressed member the stress concentration is increased and the fatigue strength is reduced; this is the reason for specifying an "edge distance" in some of this joints (see also note on edge distance in table Table 3).

Detail category	Constructional details	Description	Requirement
	1. 	1. Welded longitudinal attachment.	1. The detail category is given for: - Edge distance ≥ 10mm - For edge distance < 10mm the detail category shall be downgraded with one SN-curve.
E	$l \le 50$mm		
F	$50 < l \le 120$mm		
F1	$120 < l \le 300$mm		
F3	$l > 300$mm		
	2. 	2. Gusset plate with a radius welded to the edge of a plate or beam flange.	
D	$\frac{1}{3} \le \frac{r}{W}$, $r \ge 150$mm		
F	$\frac{1}{6} \le \frac{r}{W} < \frac{1}{3}$		
F1	$\frac{1}{10} \le \frac{r}{W} < \frac{1}{6}$		
F3	$\frac{1}{16} \le \frac{r}{W} < \frac{1}{10}$		
G	$\frac{1}{25} \le \frac{r}{W} < \frac{1}{16}$		

Detail category	Constructional details	Description	Requirement
	3. 	3. Gusset plate welded to the edge of a plate or beam flange.	
G	$l \leq 150mm$		
W1	$150 < l \leq 300mm$		
W2	$l > 300mm$		
	4.  5.  6. 	4. Transverse attachments with edge distance ≥ 10mm. 5. Vertical stiffener welded to a beam or a plate girder. 6. Diaphragms of box girders welded to the flange or web.	5.: - The stress range should be calculated using principal stresses if the stiffener terminates in the web. 5. and 6.: The detail category is given for: - Edge distance ≥ 10mm - For edge distance < 10mm the detail category shall be downgraded with one SN-curve.
E	t ≤ 12mm		
F	t > 12mm		

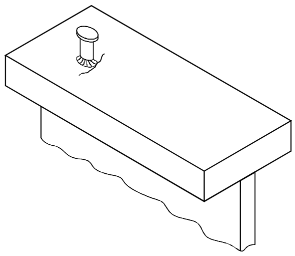
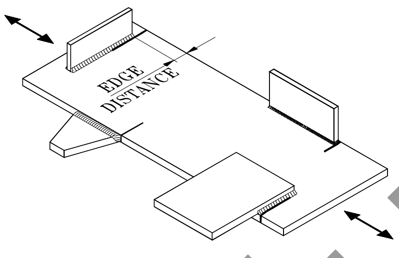
Detail category	**Constructional details**	**Description**	**Requirement**
	7. 	7. Welded shear connector to base material.	
E	Edge distance ≥ 10mm		
G	Edge distance < 10mm		
G	8. 	8. Welded attachment with edge distance < 10mm.	

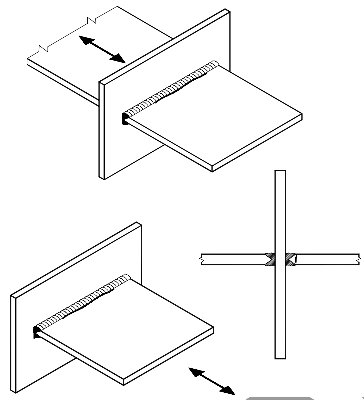
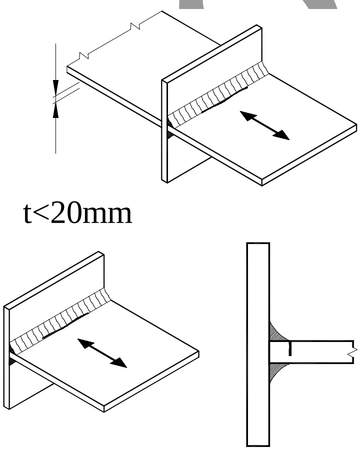
Table 8 Welded joints with load carrying welds

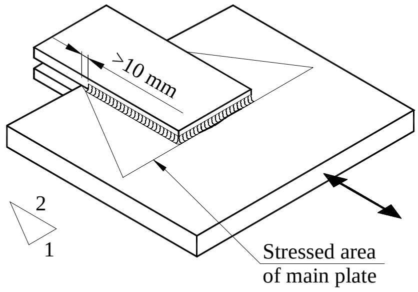
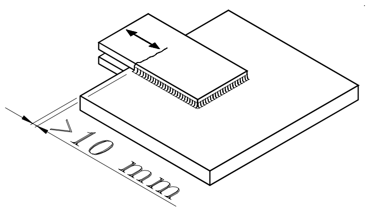
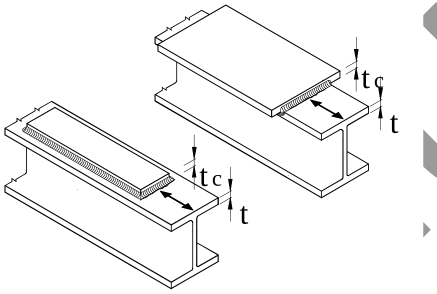
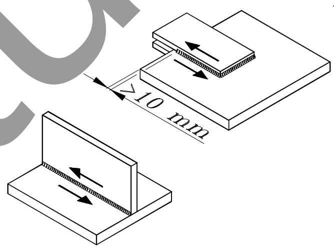
Notes on potential modes of failure

Failure in cruciform or T joints with full penetration welds will normally initiate at the weld toe. In joints made with load-carrying fillet or partial penetration butt welds, cracking may initiate either at the weld toe and propagate into the plate, or at the weld root and propagate through the weld. In welds parallel to the direction of the applied stress, however, weld failure is uncommon. In this case, cracks normally initiate at the weld end and propagate into the plate perpendicular to the direction of applied stress. The stress concentration is increased, and the fatigue strength is therefore reduced, if the weld end is located on or adjacent to the edge of a stressed member rather than on its surface.

Design stresses

In the design of cruciform joints, which are not aligned the stresses, must include the effect of any eccentricity. The maximum value of the eccentricity may normally be taken from the fabrication tolerances. The design stress may be obtained as the nominal stress multiplied by the stress concentration factor due to the eccentricity.

Detail category	**Constructional details**	**Description**	**Requirement**
F	1. 	1. Full penetration butt welded cruciform joint	1.: - Inspected and found free from significant defects. The detail category is given for: - Edge distance ≥ 10mm - For edge distance < 10mm the detail category shall be downgraded with one SN-curve.
W3	2. 	2. Partial penetration tee-butt joint or fillet welded joint and effective full penetration in tee-butt joint. See also section C2.7.	2.: - Two fatigue assessments are required. Firstly, root cracking is evaluated taking Category W3 for σ_w. σ_w is defined in C.2.2. Secondly, toe cracking is evaluated by determining the stress range in the load-carrying plates and use Category G. - If the requirements in C2.7 are fulfilled and the edge distance ≥ 10mm, Category F1 may be used for partial penetration welds and F3 for fillet welds.

Detail category	Constructional details	Description	Requirement
F1	3. 	3. Fillet welded overlap joint. Crack in main plate.	3.: - Stress in the main plate to be calculated on the basis of area shown in the sketch. - Weld termination more than 10 mm from plate edge. - Shear cracking in the weld should be verified using detail 7.
W1	4. 	4. Fillet welded overlap joint. Crack in overlapping plate.	4.: - Stress to be calculated in the overlapping plate elements - Weld termination more than 10 mm from plate edge. - Shear cracking in the weld should be verified using detail 7.
	5. 	5. End zones of single or multiple welded cover plates in beams and plate girders. Cover plates with or without frontal weld.	5.: - When the cover plate is wider than the flange, a frontal weld, carefully ground to remove undercut, is necessary.
G	t and $t_c \le 20$ mm		
W3	t and $t_c > 20$ mm		
E	6. and 7. 	6. Continuous fillet welds transmitting a shear flow, such as web to flange welds in plate girders. For continuous full penetration butt weld in shear use Category C2. 7. Fillet welded lap joint.	6.: - Stress range to be calculated from the weld throat area. 7.: - Stress range to be calculated from the weld throat area considering the total length of the weld. - Weld terminations more than 10 mm from the plate edge.

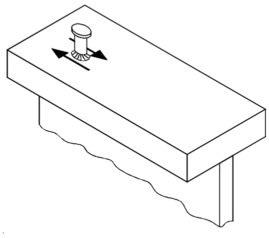
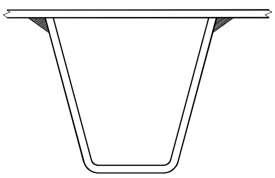
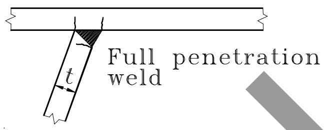
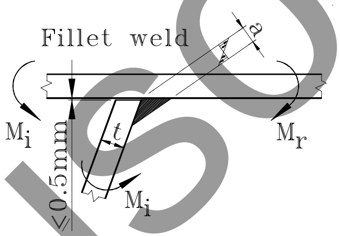
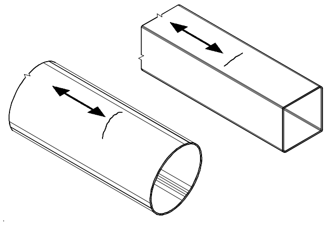
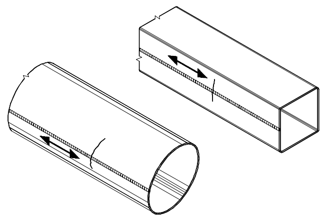
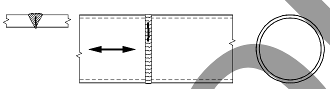
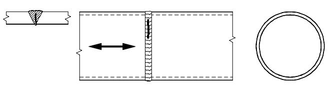
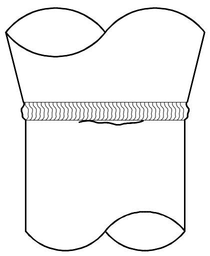
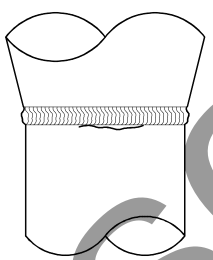
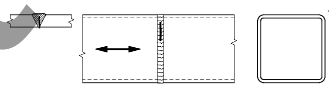
Detail category	Constructional details	Description	Requirement
E	8. 	8. Stud connectors (failure in the weld or heat affected zone).	8.: - The shear stress to be calculated on the nominal cross section of the stud.
	9. 	9. Trapezoidal stiffener welded to deck plate with fillet weld or full or partial penetration butt weld.	9.: - For a full penetration butt weld, the bending stress range shall be calculated on the basis of the thickness of the stiffener. - For a fillet weld or a partial penetration butt weld, the bending stress range shall be calculated on the basis of the throat thickness of the weld, or the thickness of the stiffener if smaller.
F			
G			

Table 9 Hollow sections

Detail category	Constructional details	Description	Requirement
B1	1. 	1. Non-welded sections.	1.: Sharp edges and surface flaws to be improved by grinding.
B2	2. 	2. Automatic longitudinal seam welds (for all other cases, see Table 3).	2.: - No stop /start positions, and free from defects outside the tolerances of NORSOK M-101.
C1		3. Circumferential butt weld made from both sides dressed flush.	3., 4., 5. And 6.: - The applied stress must include the stress concentration factor to allow for any thickness change and for fabrication tolerances, ref C.2.6.3.7. - The requirements to the corresponding detail category in Table 5 apply.
D		4. Circumferential butt weld made from both sides.	
E		5. Circumferential butt weld made from both sides made at site.	
F		6. Circumferential butt weld made from one side on a backing bar.	
F3	7. 	7. Circumferential butt weld made from one side without a backing bar.	7.: - The applied stress should include the stress concentration factor to allow for any thickness change and for fabrication tolerances, ref. C.2.6.3.7. - The weld root proved free from defects larger than 1-2mm.

Detail category	**Constructional details**	**Description**	**Requirement**
C1	8., 9., 10 and 11. 	8. Circumferential butt welds between tubular and conical sections, weld made from both sides dressed flush.	8, 9., 10., and 11.: - The applied stress must also include the stress concentration factor due to the overall form of the joint, ref. C.2.6.3.9. - The requirements to the corresponding detail category in Table 5 apply.
D		9. Circumferential butt welds between tubular and conical sections, weld made from both sides.	
E		10. Circumferential butt welds between tubular and conical sections, weld made from both sides made at site.	
F		11. Circumferential butt welds between tubular and conical sections, weld made from one side on a backing bar.	
F3	12. 	12. Circumferential butt welds between tubular and conical sections, weld made from one side without a backing bar.	12.: - The applied stress must also include the stress concentration factor due to the overall form of the joint - The weld root proved free from defects larger than 1-2mm.
F3	13. 	13. Butt welded end to end connection of rectangular hollow sections.	13.: - With the weld root proved free from defects larger than 1-2 mm.

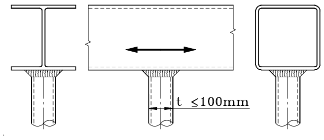
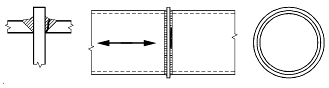
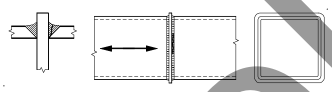
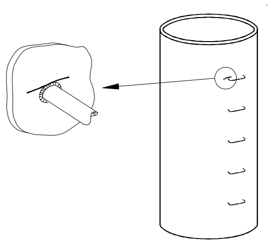
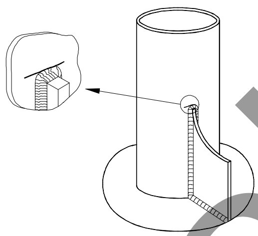
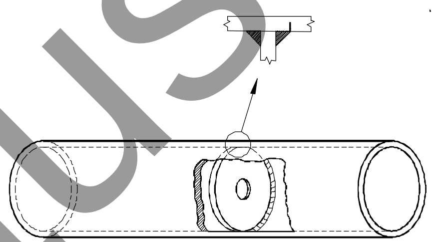
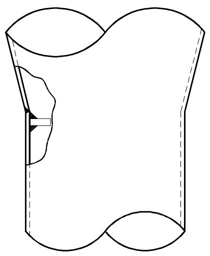
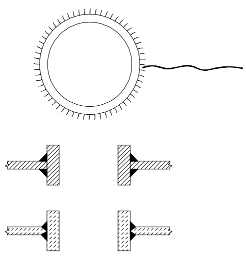
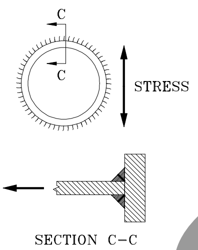
Detail category	Constructional details	Description	Requirement
F	14. 	14. Circular or rectangular hollow section, fillet welded to another section.	14.: - Non load carrying welds. - Section width parallel to stress direction ≤ 100mm. - All other cases, see Table 7.
G	15. 	15. Circular hollow section butt welded end to end with an intermediate plate.	15.: - Load carrying welds. - Welds inspected and found free from defects outside the tolerances of NORSOK M-101 - Details with wall thickness greater than 8mm may be classified Category F3.
W1	16. 	16. Rectangular hollow section butt welded end to end with an intermediate plate.	16.: - Load carrying welds. - Welds inspected and found free from defects outside the tolerances of NORSOK M-101 - Details with wall thickness greater than 8mm may be classified as Category G.

Table 10 Details relating to tubular members

Detail category	Constructional details	Description	Requirement
T		1. Parent material adjacent to the toes of full penetration welded tubular joints.	1.: - The design should be based on the hot spot stress.
F1	2. 	2. Welded rungs.	
F1	3. and 4. 	3. Gusseted connections made with full penetration welds.	3.: - The design stress must include the stress concentration factor due to the overall form of the joint.
F3		4. Gusseted connections made with fillet welds.	4.: - The design stress must include the stress concentration factor due to the overall form of the joint.
F	5. 	5. Parent material at the toe of a weld attaching a diaphragm to a tubular member.	The nominal design stress for the inside may be determined from C.2.6.3.8.

Detail category	**Constructional details**	**Description**	**Requirement**
E to G, see Table 7	6. 	6.Parent material (of the stressed member) adjacent to the toes of a bevel butt or fillet welded attachments in region of stress concentration.	6.: -Class depends on attachment length (see Table 7) but stress must include the stress concentration factor due to the overall shape of adjoining structure.
D	7. 	7.Parent material to, or weld metal in welds around a penetration through a wall of a member (on a plane essentially perpendicular to the direction of stress)	7.: In this situation the relevant stress must include the stress concentration factor due to the overall geometry of the detail.
W1		8.Weld metal in partial penetration or fillet welded joints around a penetration through the wall of a member (on a plane essentially parallel to the plane of stress).	8.: - The stress in the weld should include an appropriate stress concentration factor to allow for the overall joint geometry. Reference is also made to Appendix 3.

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DESIGN OF STEEL STRUCTURES

ANNEX C, APPENDIX 2
SCFS FOR TUBULAR JOINTS

Annex C, Appendix 2
SCFS for tubular joints

Stress concentration factors for simple tubular joints and overlap joints

Stress concentration factors for tubular joints for joint types T/Y are given in Table 1, for joint types X in Table 2, for joint types K in Table 3 and

Table 4 and for joint types KT in Table 5.

Joint classification is the process whereby the axial force in a given brace is subdivided into K, X and Y components of actions corresponding to the three joint types for which stress concentration equations exists. Such subdivision normally considers all of the members in one plane at a joint. For purposes of this provision, brace planes within ±15º of each other may be considered as being in a common plane. Each brace in the plane can have a unique classification that could vary with action condition. The classification can be a mixture between the above three joint types.

Figure 1 provides some simple examples of joint classification. For a brace to be considered as K-joint classification, the axial force in the brace should be balanced to within 10% by forces in other braces in the same plane and on the same side of the joint. For Y-joint classification, the axial force in the brace is reacted as beam shear in the chord. For X-joint classification, the axial force in the brace is carried through the chord to braces on the opposite side. Figure 1 c), e) and h) shows joints with a combination of classifications. In c) 50% of the diagonal force is balanced with a force in the vertical in a K-joint and 50% of the diagonal force is balanced with a beam shear force in the chord in an Y-joint. In e) 33% of the incoming diagonal force is balanced with a force in the vertical in a K-joint with gap 1 and 67% of the incoming diagonal force is balanced with a force in the other diagonal in a K-joint with gap 2. In h) 50% of the diagonal force is balanced with a force in the vertical on the same side of the chord in a K-joint and 50% of the diagonal force is balanced with a force in the vertical on the opposite side of the chord in a X-joint.

Definitions of geometrical parameters can be found in Figure 2.

A classification of joints can be based on a deterministic analysis using a wave height corresponding to that with the largest contribution to fatigue damage. A conservative classification may be used keeping in mind that

$SCF_X > SCF_Y > SCF_K$

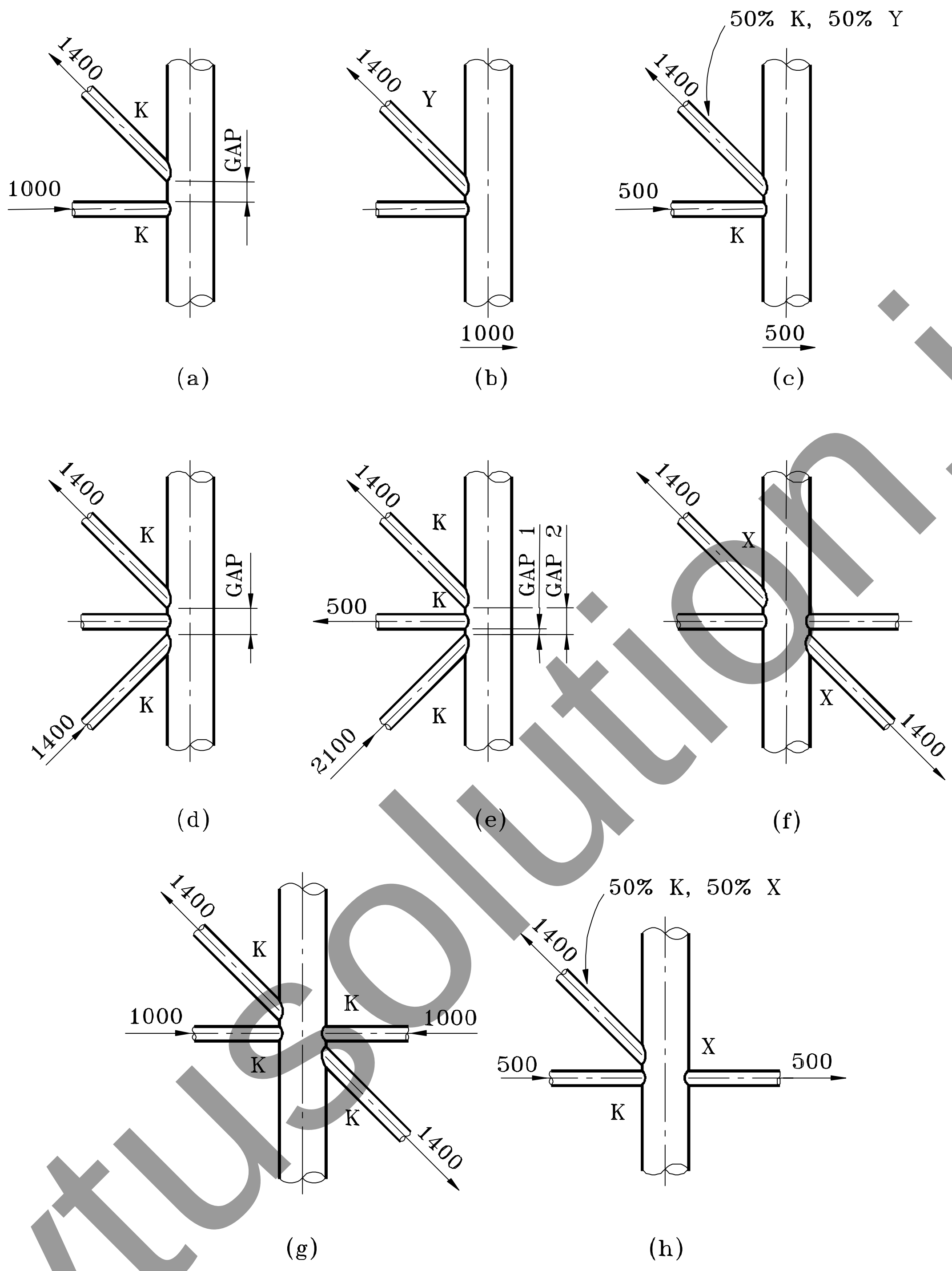
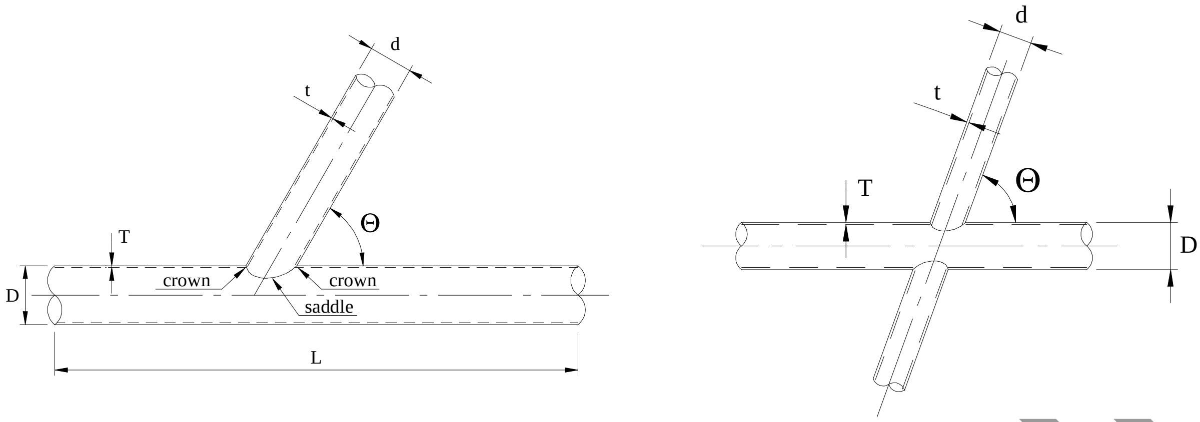
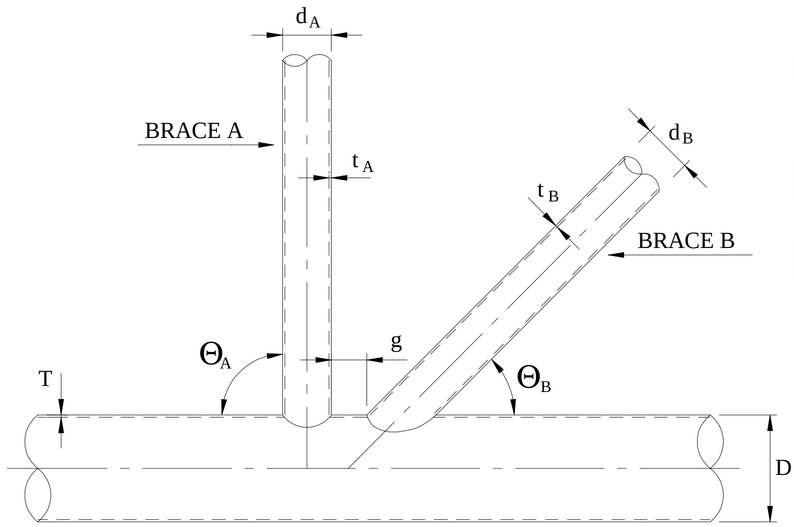


Figure 1 Classification of simple joints.



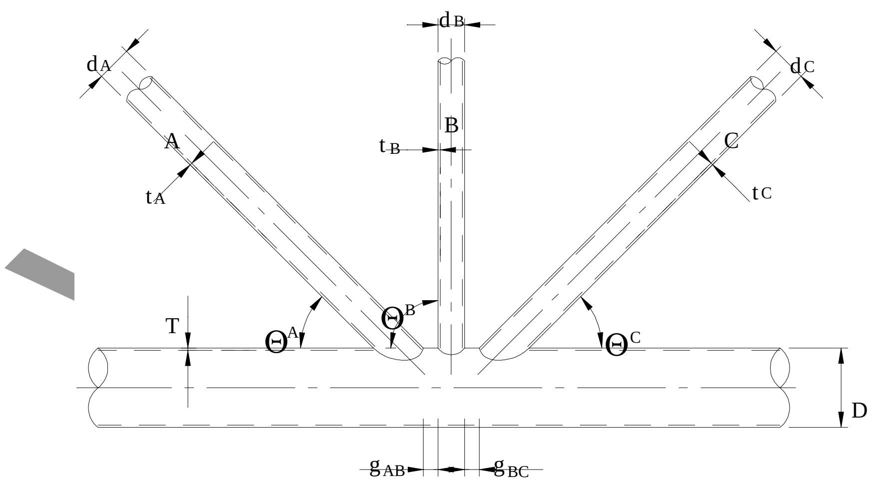
$$\beta = \frac{d}{D} \qquad \alpha = \frac{2L}{D} \qquad \gamma = \frac{D}{2T} \qquad \tau = \frac{t}{T}$$



$$\beta_A = \frac{d_A}{D} \qquad \beta_B = \frac{d_B}{D}$$

$$\tau_A = \frac{t_A}{T} \qquad \tau_B = \frac{t_B}{T}$$

$$\gamma = \frac{D}{2T} \qquad \zeta = \frac{g}{D}$$



$$\beta_A = \frac{d_A}{D} \quad \beta_B = \frac{d_B}{D} \quad \beta_C = \frac{d_C}{D}$$

$$\tau_A = \frac{t_A}{T} \quad \tau_B = \frac{t_B}{T} \quad \tau_C = \frac{t_C}{T}$$

$$\gamma = \frac{D}{2T} \quad \zeta_{AB} = \frac{g_{AB}}{D} \quad \zeta_{BC} = \frac{g_{BC}}{D}$$

Figure 2 Definition of geometrical parameters.

The validity range for the equations in Table 1 to Table 5 are as follows:

$$0.2 \le \beta \le 1.0$$

$$0.2 \le \tau \le 1.0$$

$$8 \le \gamma \le 32$$

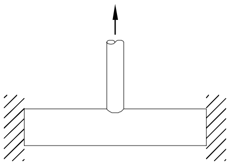
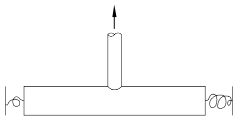
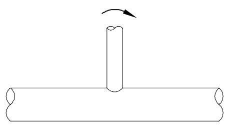
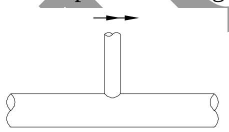
$$4 \le \alpha \le 40$$

$$20^\circ \le \theta \le 90^\circ$$

$$\frac{-0.6\beta}{\sin\theta} \le \zeta \le 1.0$$

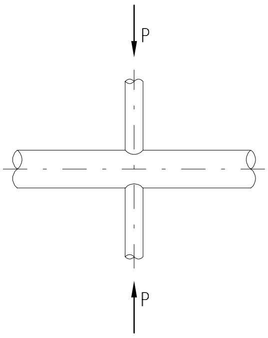
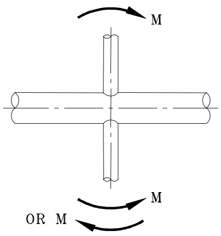
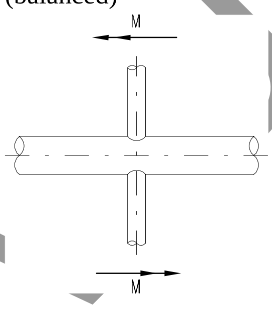
Reference is made to C.2.10.2 if actual geometry is outside validity range.

Table 1 Stress Concentration Factors for Simple Tubular T/Y Joints

Load type and fixity conditions	SCF equations	Eqn. No.	Short chord correction
Axial load- Chord ends fixed 	Chord saddle: $\gamma\,\tau^{1.1}\left(1.11-3(\beta-0.52)^2\right)(\sin\theta)^{1.6}$	(1)	F1
	Chord crown: $\gamma^{0.2}\tau\left(2.65+5(\beta-0.65)^2\right)+\tau\,\beta\,(0.25\,\alpha-3)\sin\theta$	(2)	None
	Brace saddle: $1.3+\gamma\,\tau^{0.52}\alpha^{0.1}\left(0.187-1.25\beta^{1.1}(\beta-0.96)\right)(\sin\theta)^{(2.7\text{-}0.01\alpha)}$	(3)	F1
	Brace crown: $3+\gamma^{1.2}\left(0.12\exp(-4\beta)+0.011\beta^2-0.045\right)+\beta\,\tau(0.1\,\alpha-1.2)$	(4)	None
Axial load- General fixity conditions 	Chord saddle: $(\text{Eqn.}(1))+C_1(0.8\,\alpha-6)\,\tau\,\beta^2\left(1-\beta^2\right)^{0.5}(\sin 2\theta)^2$	(5)	F2
	Chord crown: $\gamma^{0.2}\tau\left(2.65+5(\beta-0.65)^2\right)+\tau\,\beta\,(C_2\alpha-3)\,\sin\theta$	(6)	None
	Brace saddle: (Eqn.(3))		F2
	Brace crown: $3+\gamma^{1.2}\left(0.12\exp(-4\beta)+0.011\beta^2-0.045\right)+\beta\,\tau\,(C_3\,\alpha-1.2)$	(7)	None
In-plane bending 	Chord crown: $1.45\beta\,\tau^{0.85}\gamma^{(1-0.68\beta)}(\sin\theta)^{0.7}$	(8)	None
	Brace crown: $1+0.65\beta\,\tau^{0.4}\gamma^{(1.09-0.77\beta)}(\sin\theta)^{(0.06\gamma\text{-}1.16)}$	(9)	None
Out-of-plane bending 	Chord saddle: $\gamma\,\tau\,\beta\left(1.7-1.05\beta^3\right)(\sin\theta)^{1.6}$	(10)	F3
	Brace saddle: $\tau^{-0.54}\gamma^{-0.05}\left(0.99-0.47\,\beta+0.08\,\beta^4\right)\cdot(\text{Eqn.}(10))$	(11)	F3

Short chord correction factors ($\alpha < 12$)	Chord-end fixity parameter
$F1 = 1 - \left(0.83\beta - 0.56\beta^2 - 0.02\right)\gamma^{0.23}\exp\left(-0.21\,\gamma^{-1.16}\alpha^{2.5}\right)$	$C_1 = 2(C-0.5)$
$F2 = 1 - \left(1.43\beta - 0.97\beta^2 - 0.03\right)\gamma^{0.04}\exp\left(-0.71\,\gamma^{-1.38}\alpha^{2.5}\right)$	$C_2 = C/2$
$F3 = 1 - 0.55\beta^{1.8}\gamma^{0.16}\exp\left(-0.49\,\gamma^{-0.89}\alpha^{1.8}\right)$	$C_3 = C/5$
where $\exp(x) = e^x$	C = chord end fixity parameter
	$0.5 \le C \le 1.0$, Typically C = 0.7

Table 2 Stress Concentration Factors for Simple X Tubular Joints

Load type and fixity conditions	**SCF equation**	**Eqn. no.**
Axial load (balanced)  	Chord saddle: $3.87\,\gamma\,\tau\,\beta\left(1.10-\beta^{1.8}\right)(\sin\theta)^{1.7}$ Chord crown: $\gamma^{0.2}\tau\left(2.65+5(\beta-0.65)^2\right)-3\,\tau\,\beta\sin\theta$ Brace saddle: $1+1.9\,\gamma\,\tau^{0.5}\beta^{0.9}\left(1.09-\beta^{1.7}\right)(\sin\theta)^{2.5}$ Brace crown: $3+\gamma^{1.2}\left(0.12\exp(-4\beta)+0.011\beta^2-0.045\right)$ In joints with short cords ($\alpha < 12$) the saddle SCF can be reduced by the factor F1 (fixed chord ends) or F2 (pinned chord ends) where $F1 = 1 - \left(0.83\beta - 0.56\beta^2 - 0.02\right)\gamma^{0.23}\exp\left(-0.21\gamma^{-1.16}\alpha^{2.5}\right)$ $F2 = 1 - \left(1.43\beta - 0.97\beta^2 - 0.03\right)\gamma^{0.04}\exp\left(-0.71\gamma^{-1.38}\alpha^{2.5}\right)$	(12) (13) (14) (15)
In plane bending  	Chord crown: (Eqn.(8)) Brace crown: (Eqn. (9))	
Out of plane bending (balanced)  	Chord saddle: $\gamma\,\tau\,\beta\left(1.56-1.34\beta^4\right)(\sin\theta)^{1.6}$ Brace saddle: $\tau^{-0.54}\gamma^{-0.05}\left(0.99-0.47\beta+0.08\beta^4\right)\cdot$ (Eqn.(16)) In joints with short chords ($\alpha < 12$) eqns. (16) and (17) can be reduced by the factor F3 where: $F3 = 1 - 0.55\beta^{1.8}\gamma^{0.16}\exp\left(-0.49\gamma^{-0.89}\alpha^{1.8}\right)$	(16) (17)

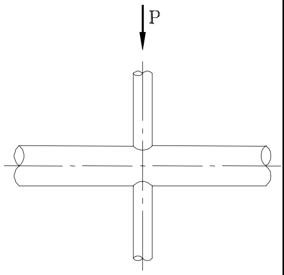
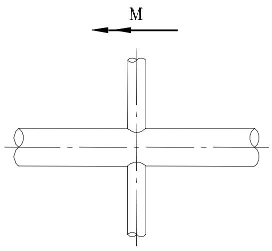
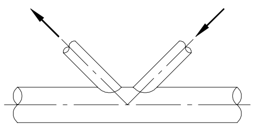
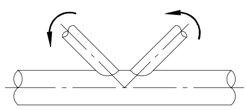
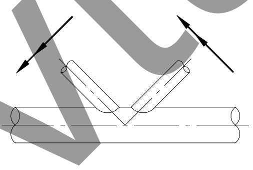
Load type and fixity conditions	**SCF equation**	**Eqn. no.**
Axial load in one brace only 	Chord saddle: $\left(1 - 0.26\,\beta^3\right) \cdot$ (Eqn. (5)) Chord crown: (Eqn. (6)) Brace saddle $\left(1 - 0.26\,\beta^3\right) \cdot$ (Eqn. (3)) Brace crown: (Eqn.(7)) In joints with short chords ($\alpha < 12$) the saddle SCFs can be reduced by the factor F1 (fixed chord ends) or F2 (pinned chord ends) where: $F1 = 1 - \left(0.83\,\beta - 0.56\,\beta^2 - 0.02\right)\gamma^{0.23}\exp\left(-0.21\,\gamma^{-1.16}\alpha^{2.5}\right)$ $F2 = 1 - \left(1.43\,\beta - 0.97\,\beta^2 - 0.03\right)\gamma^{0.04}\exp\left(-0.71\,\gamma^{-1.38}\alpha^{2.5}\right)$	(18) (19)
Out-of-plane bending on one brace only: 	Chord saddle: (Eqn. (10)) Brace saddle: (Eqn. (11)) In joints with short chords ($\alpha < 12$) eqns. (10) and (11) can be reduced by the factor F3 where: $F3 = 1 - 0.55\,\beta^{1.8}\gamma^{0.16}\exp\left(-0.49\,\gamma^{-0.89}\alpha^{1.8}\right)$	

Table 3 Stress Concentration Factors for Simple Tubular K Joints and Overlap K Joints

Load type and fixity conditions	**SCF equation**	**Eqn. no.**	**Short chord correction**
Balanced axial load 	Chord: $\tau^{0.9}\gamma^{0.5}\left(0.67-\beta^{2}+1.16\beta\right)\sin\theta\left(\frac{\sin\theta_{max}}{\sin\theta_{min}}\right)^{0.30}\cdot$ $\left(\frac{\beta_{max}}{\beta_{min}}\right)^{0.30}\left(1.64+0.29\beta^{-0.38}ATAN\left(8\zeta\right)\right)$ Brace: $1+\left(1.97-1.57\beta^{0.25}\right)\tau^{-0.14}(\sin\theta)^{0.7}\cdot$ (Eqn. (20))+ $\sin^{1.8}(\theta_{max}+\theta_{min})\cdot\left(0.131-0.084\,ATAN\left(14\zeta+4.2\beta\right)\right)\cdot$ $C\beta^{1.5}\gamma^{0.5}\tau^{-1.22}$ Where: C = 0 for gap joints C = 1 for the through brace C = 0.5 for the overlapping brace Note that τ, β, θ and the nominal stress relate to the brace under consideration ATAN is arctangent evaluated in radians	(20) (21)	None None
Unbalanced in plane bending 	Chord crown: (Eqn. (8)) (for overlaps exceeding 30% of contact length use 1.2·(Eqn. (8))) Gap joint brace crown: (Eqn. (9)) Overlap joint brace crown: (Eqn. (9))·$(0.9+0.4\beta)$	 (22)	
Unbalanced out-of-plane bending 	Chord saddle SCF adjacent to brace A: $(\text{Eqn. }(10))_A\left(1-0.08(\beta_B\gamma)^{0.5}\exp(-0.8x)\right)+(\text{Eqn. }(10))_B$ $\left(1-0.08(\beta_A\gamma)^{0.5}\exp(-0.8x)\right)\left(2.05\beta_{max}^{0.5}\exp(-1.3x)\right)$ where $x=1+\frac{\zeta\sin\theta_A}{\beta_A}$ Brace A saddle SCF $\tau^{-0.54}\gamma^{-0.05}\left(0.99-0.47\beta+0.08\beta^{4}\right)\cdot$ (Eqn. (23))	(23) (24)	F4 F4

$F4=1-1.07\beta^{1.88}\exp\left(-0.16\gamma^{-1.06}\alpha^{2.4}\right)$

$(\text{Eqn. }(10))_A$ is the chord SCF adjacent to brace A as estimated from Eqn.(10).

Note that the designation of braces A and B is not geometry dependent. It is nominated by the user.

Table 4 Stress Concentration Factors for Simple Tubular K Joints and Overlap K Joints

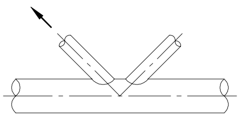
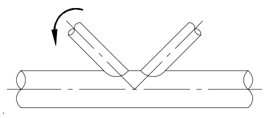
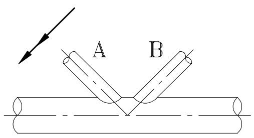
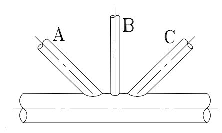
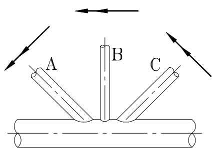
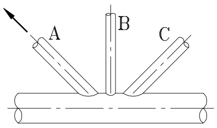
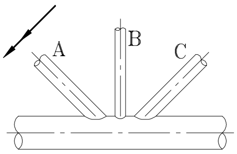
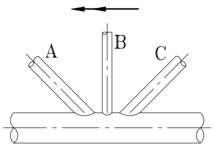
Axial load on one brace only 	Chord saddle: (Eqn. (5)) Chord crown: (Eqn. (6)) Brace saddle: (Eqn.(3)) Brace crown: (Eqn. (7)) Note that all geometric parameters and the resulting SCF's relate to the loaded brace.		F1 - F1 -
In-plane-bending on one brace only 	Chord crown: (Eqn. (8)) Brace crown: (Eqn. (9)) Note that all geometric parameters and the resulting SCF's relate to the loaded brace.		
Out-of-plane bending on one brace only 	Chord saddle: $(\text{Eqn. }(10))_A \cdot \left(1 - 0.08\left(\beta_B \gamma\right)^{0.5} \exp\left(-0.8\,x\right)\right)$ where $x = 1 + \frac{\zeta \sin\theta_A}{\beta_A}$	(25)	F3
	Brace saddle: $\tau^{-0.54} \gamma^{-0.05} \left(0.99 - 0.47\beta + 0.08\beta^4\right) \cdot$ (Eqn. (25))	(26)	F3
Short chord correction factors: $F1 = 1 - \left(0.83\beta - 0.56\beta^2 - 0.02\right)\gamma^{0.23} \exp\left(-0.21\gamma^{-1.16}\alpha^{2.5}\right)$ $F2 = 1 - \left(1.43\beta - 0.97\beta^2 - 0.03\right)\gamma^{0.04} \exp\left(-0.71\gamma^{-1.38}\alpha^{2.5}\right)$ $F3 = 1 - 0.55\beta^{1.8}\gamma^{0.16} \exp\left(-0.49\gamma^{-0.89}\alpha^{1.8}\right)$			

Table 5 Stress Concentration Factors for Simple KT Tubular Joints and Overlap KT Joints

Load type	**SCF equation**	**Eqn. no.**
Balanced axial load  	Chord: (Eqn. (20)) Brace: (Eqn. (21)) For the diagonal braces A & C use $\zeta = \zeta_{AB} + \zeta_{BC} + \beta_B$ For the central brace, B, use ζ = maximum of ζ_{AB}, ζ_{BC}	
In-plane bending	Chord crown: (Eqn. (8)) Brace crown: (Eqn. (9))	
Unbalanced out-of-plane bending  	Chord saddle SCF adjacent to diagonal brace A: (Eqn. (10))$_A$ $\left(1-0.08(\beta_B\gamma)^{0.5}\exp(-0.8\,x_{AB})\right)\left(1-0.08(\beta_C\gamma)^{0.5}\exp(-0.8\,x_{AC})\right)+$ (Eqn (10))$_B \cdot \left(1-0.08\,(\beta_A\gamma)^{0.5}\exp(-0.8x_{AB})\right)\left(2.05\beta_{max}^{0.5}\exp(-1.3x_{AB})\right)+$ (Eqn (10))$_C \cdot \left(1-0.08(\beta_A\gamma)^{0.5}\exp(-0.8x_{AC})\right)\left(2.05\beta_{max}^{0.5}\exp(-1.3x_{AC})\right)$ where $x_{AB} = 1+\frac{\zeta_{AB}\sin\theta_A}{\beta_A}$ $x_{AC} = 1+\frac{(\zeta_{AB}+\zeta_{BC}+\beta_B)\sin\theta_A}{\beta_A}$ Chord saddle SCF adjacent to central brace B: (Eqn. (10))$_B \cdot \left(1-0.08(\beta_A\gamma)^{0.5}\exp(-0.8\,x_{AB})\right)^{P_1} \cdot$ $\left(1-0.08(\beta_C\gamma)^{0.5}\exp(-0.8\,x_{BC})\right)^{P_2} +$ (Eqn. (10))$_A \cdot \left(1-0.08(\beta_B\gamma)^{0.5}\exp(-0.8\,x_{AB})\right)\left(2.05\beta_{max}^{0.5}\exp(-1.3\,x_{AB})\right)+$ (Eqn. (10))$_C \cdot \left(1-0.08(\beta_B\gamma)^{0.5}\exp(-0.8\,x_{BC})\right)\left(2.05\beta_{max}^{0.5}\exp(-1.3\,x_{BC})\right)$ where $x_{AB} = 1+\frac{\zeta_{AB}\sin\theta_B}{\beta_B}$ $x_{BC} = 1+\frac{\zeta_{BC}\sin\theta_B}{\beta_B}$ $P_1 = \left(\frac{\beta_A}{\beta_B}\right)^2$ $P_2 = \left(\frac{\beta_C}{\beta_B}\right)^2$	(27) (28)
Out-of-plane bending brace SCFs	Out-of-plane bending brace SCFs are obtained directly from the adjacent chord SCFs using: $\tau^{-0.54}\gamma^{-0.05}\left(0.99-0.47\beta+0.08\beta^4\right)\cdot SCF_{chord}$ where SCF_{chord} = (Eqn. (27)) or (Eqn. (28))	(29)

Load type	**SCF equation**	**Eqn. no.**
Axial load on one brace only  	Chord saddle: (Eqn. (5)) Chord crown: (Eqn. (6)) Brace saddle: (Eqn. (3)) Brace crown: (Eqn. (7))	
Out-of-plane bending on one brace only  	Chord SCF adjacent to diagonal brace A: $(\text{Eqn. (10)})_A \cdot \left(1-0.08(\beta_B\gamma)^{0.5}\exp(-0.8x_{AB})\right)\left(1-0.08(\beta_C\gamma)^{0.5}\exp(-0.8x_{AC})\right)$ where $x_{AB} = 1+\frac{\zeta_{AB}\sin\theta_A}{\beta_A}$ $x_{AC} = 1+\frac{(\zeta_{AB}+\zeta_{BC}+\beta_B)\sin\theta_A}{\beta_A}$	(30)
 	Chord SCF adjacent to central brace B: $(\text{Eqn. (10)})_B \cdot \left(1-0.08(\beta_A\gamma)^{0.5}\exp(-0.8x_{AB})\right)^{P_1} \cdot$ $\left(1-0.08(\beta_C\gamma)^{0.5}\exp(-0.8x_{BC})\right)^{P_2}$ where $x_{AB} = 1+\frac{\zeta_{AB}\sin\theta_B}{\beta_B}$ $x_{BC} = 1+\frac{\zeta_{BC}\sin\theta_B}{\beta_B}$ $P_1 = \left(\frac{\beta_A}{\beta_B}\right)^2$ $P_2 = \left(\frac{\beta_C}{\beta_B}\right)^2$	(31)
Out-of-plane brace SCFs	Out-of-plane brace SCFs are obtained directly from the adjacent chord SCFs using: $\tau^{-0.54}\gamma^{-0.05}\left(0.99-0.47\beta+0.08\beta^4\right)\cdot \text{SCF}_{chord}$	(32)

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DESIGN OF STEEL STRUCTURES

ANNEX C, APPENDIX 3
SCFS FOR CUT-OUTS

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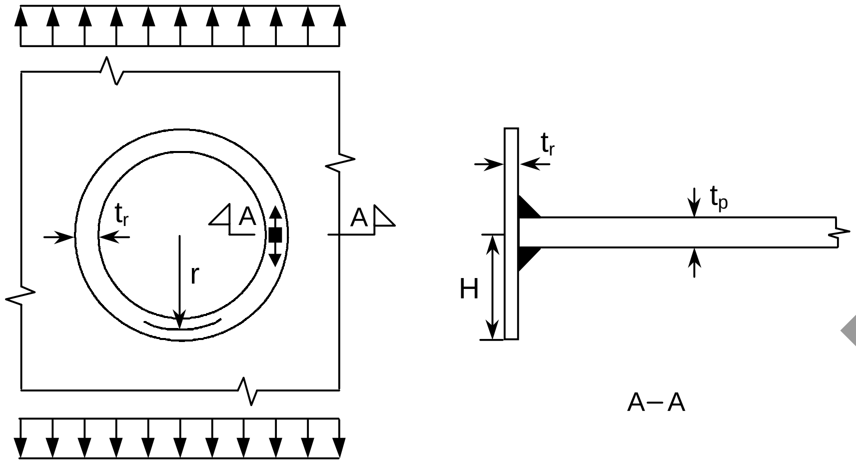
Annex C, Appendix 3
SCFS for cut-outs

Stress concentration factors at holes in plates with inserted tubulars are given in Figure 1-Figure 14.

Stress concentration factors at holes in plates with ring reinforcement are given in Figure 15-Figure 18.

Stress concentration factors at holes in plates with double ring reinforcement given in Figure 19-Figure 22.

The SCFs in these figures may also be used for fatigue assessments of the welds. $\Delta\sigma_{\perp}$ and $\Delta\tau_{\perp}$ in the weld throat in equation C.2.2 may be derived from the force in the plate acting normal to the weld. $\tau_{//}$ may be derived from the assumption that the shear force in the plate is transferred through the weld to the inserted tubular or the ring reinforcements. The total stress range from equation C.2.2 is then used together with the W3 curve to evaluate number of cycles until failure.



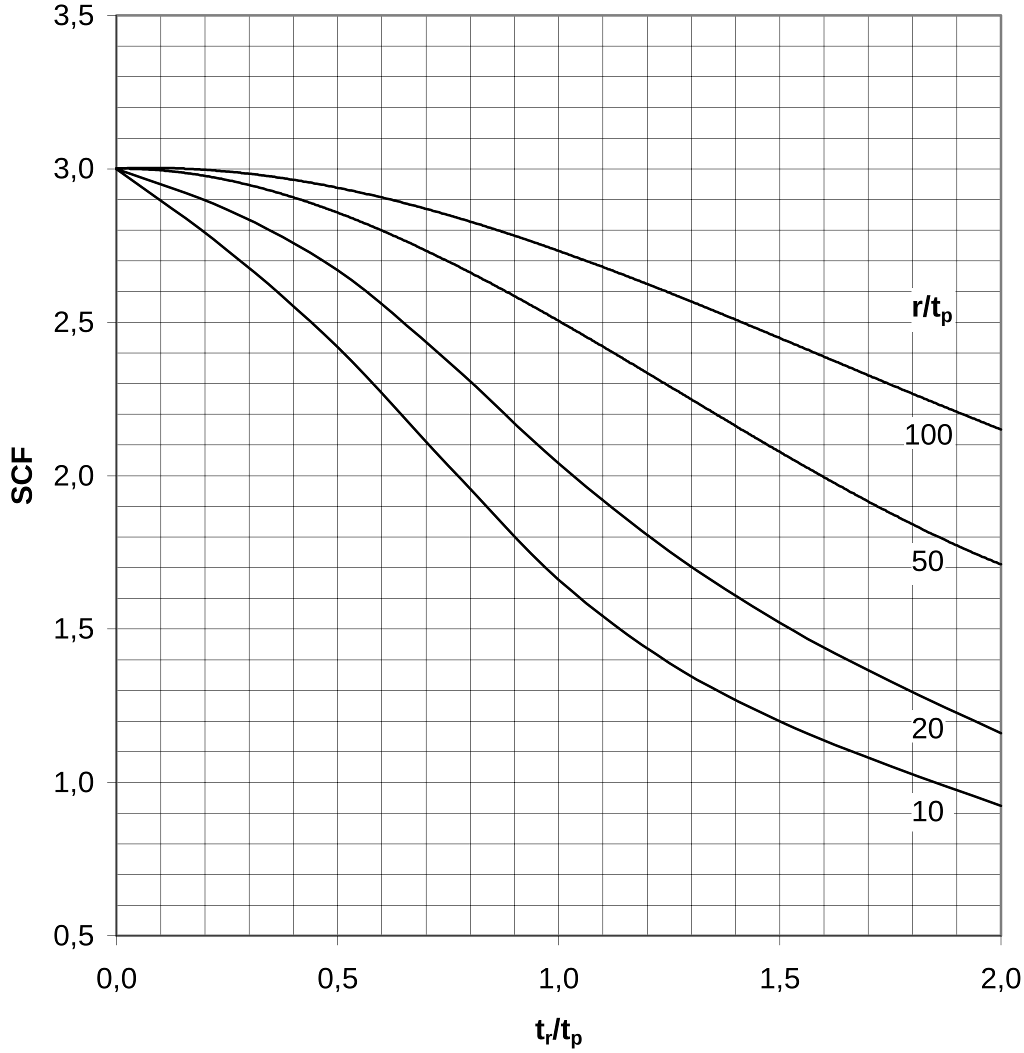


Figure 1 SCF at hole with inserted tubular. Stress in tubular, parallel with weld, at middle of tubular thickness. $H/t_r = 2$.

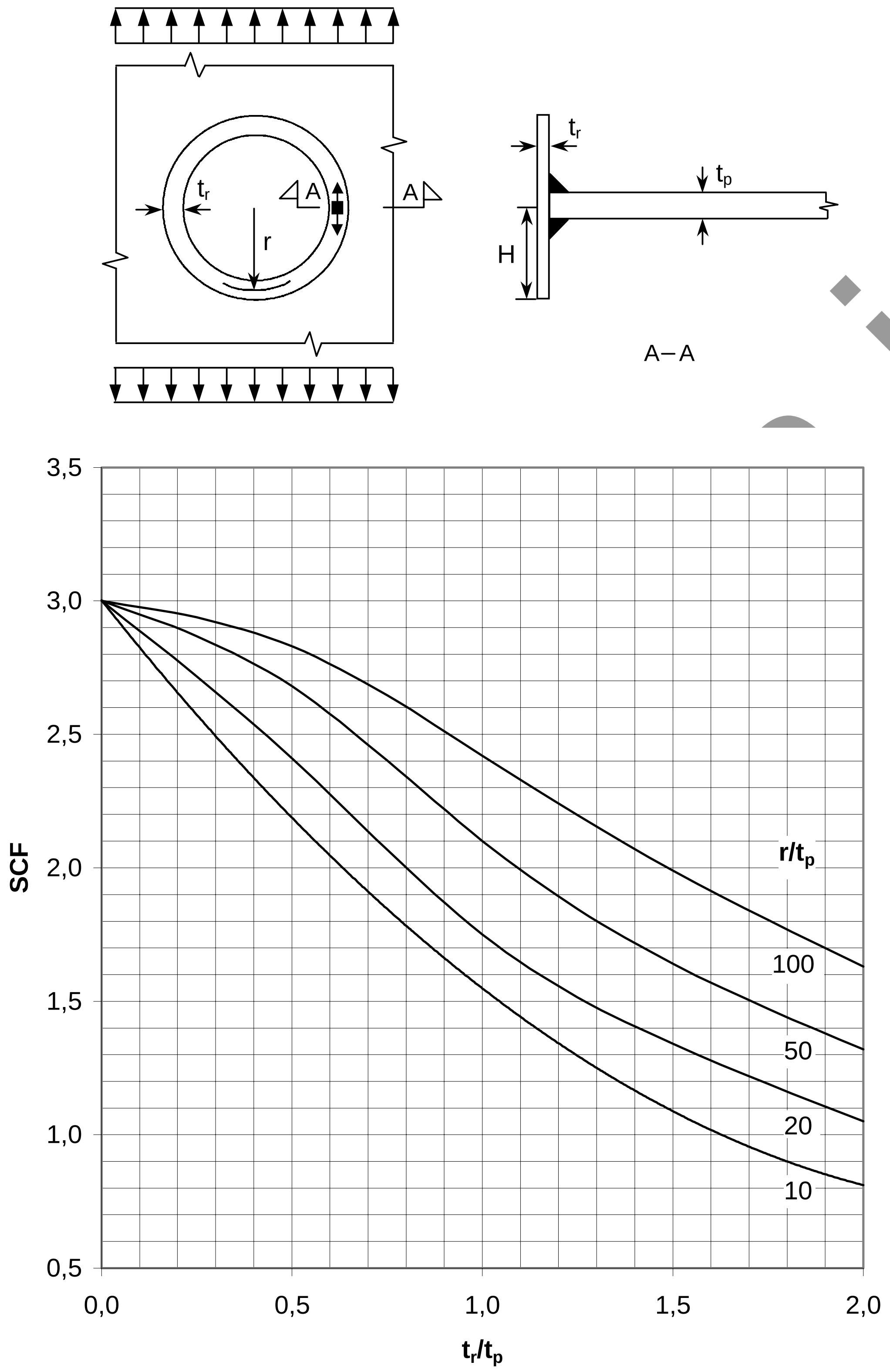
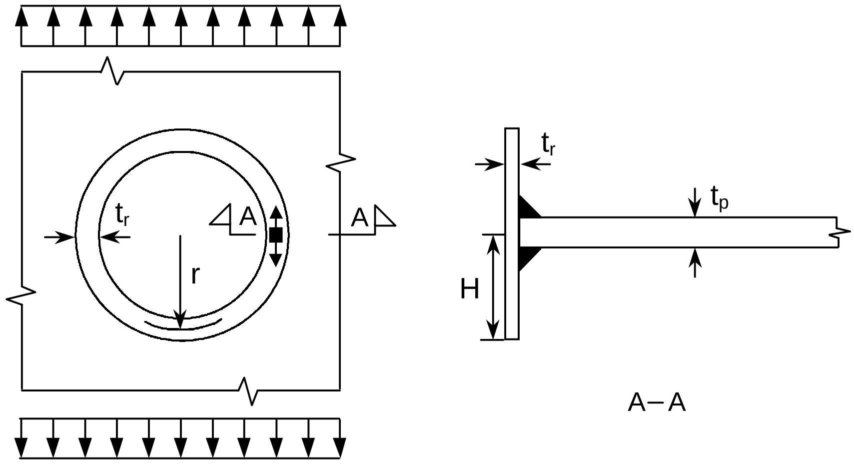


Figure 2 SCF at hole with inserted tubular. Stress in tubular, parallel with weld, at middle of tubular thickness. $H/t_r = 5$.



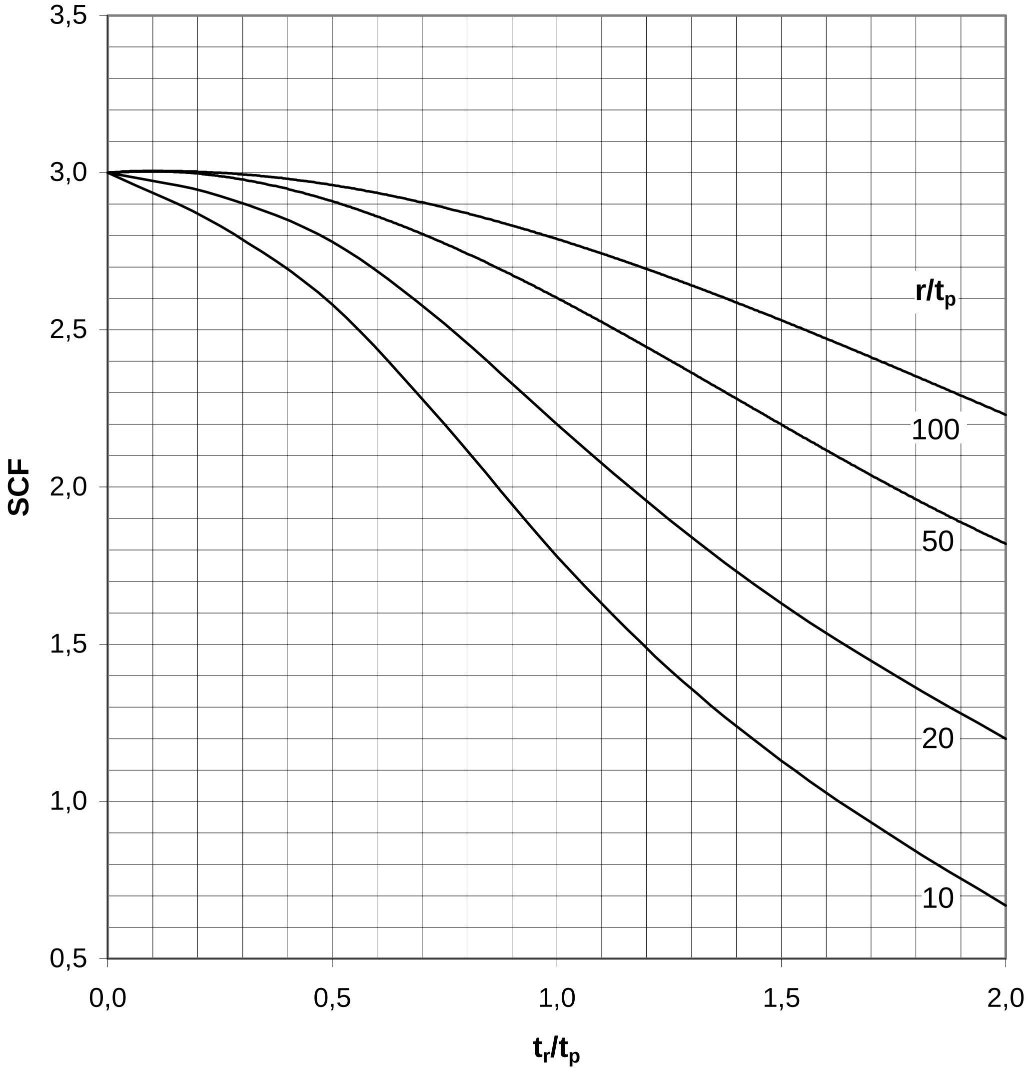


Figure 3 SCF at hole with inserted tubular. Stress at outer surface of tubular, parallel with weld. $H/t_r = 2$.

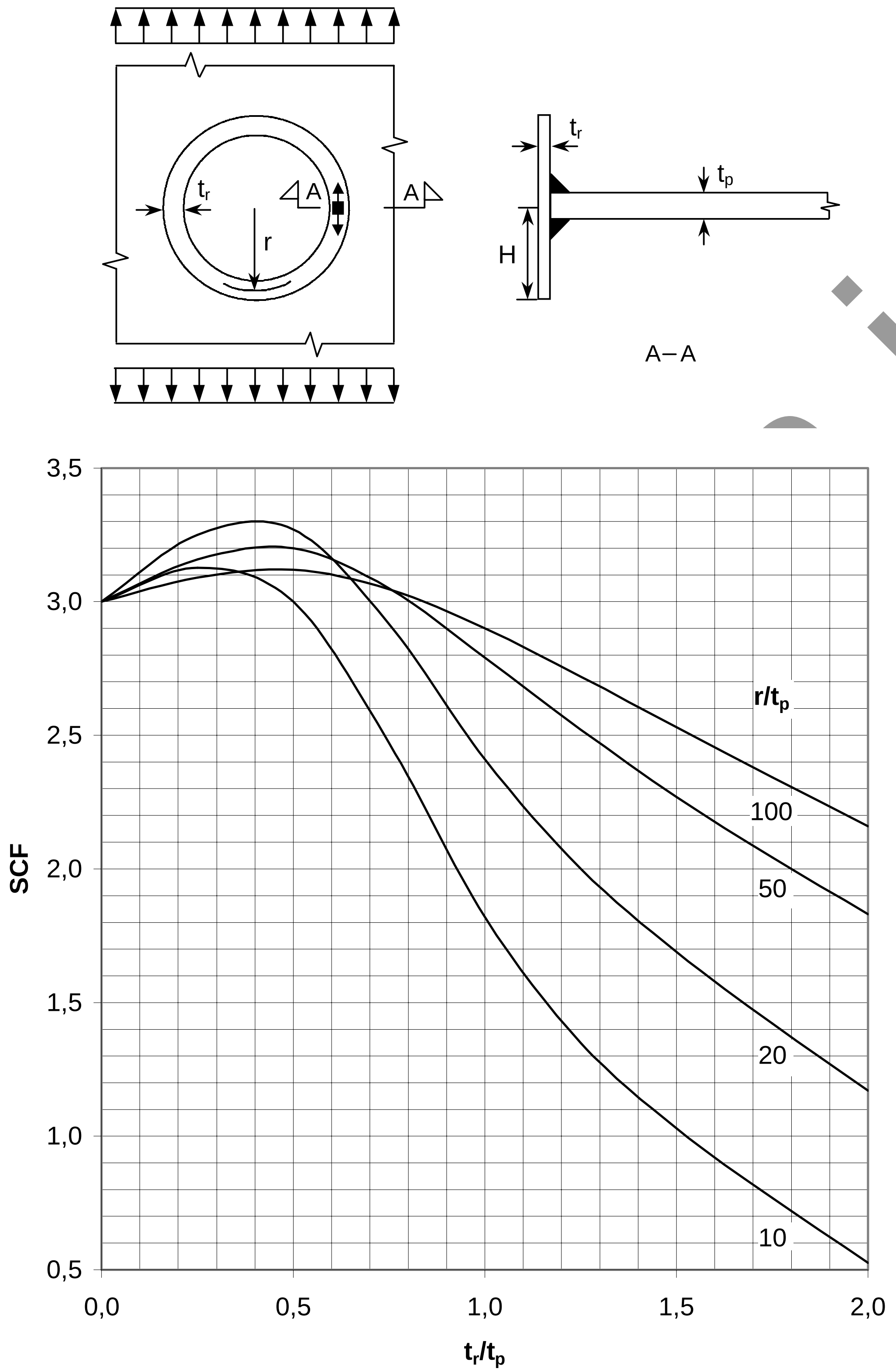


Figure 4 SCF at hole with inserted tubular. Stress at outer surface of tubular, parallel with weld. $H/t_r = 5$.

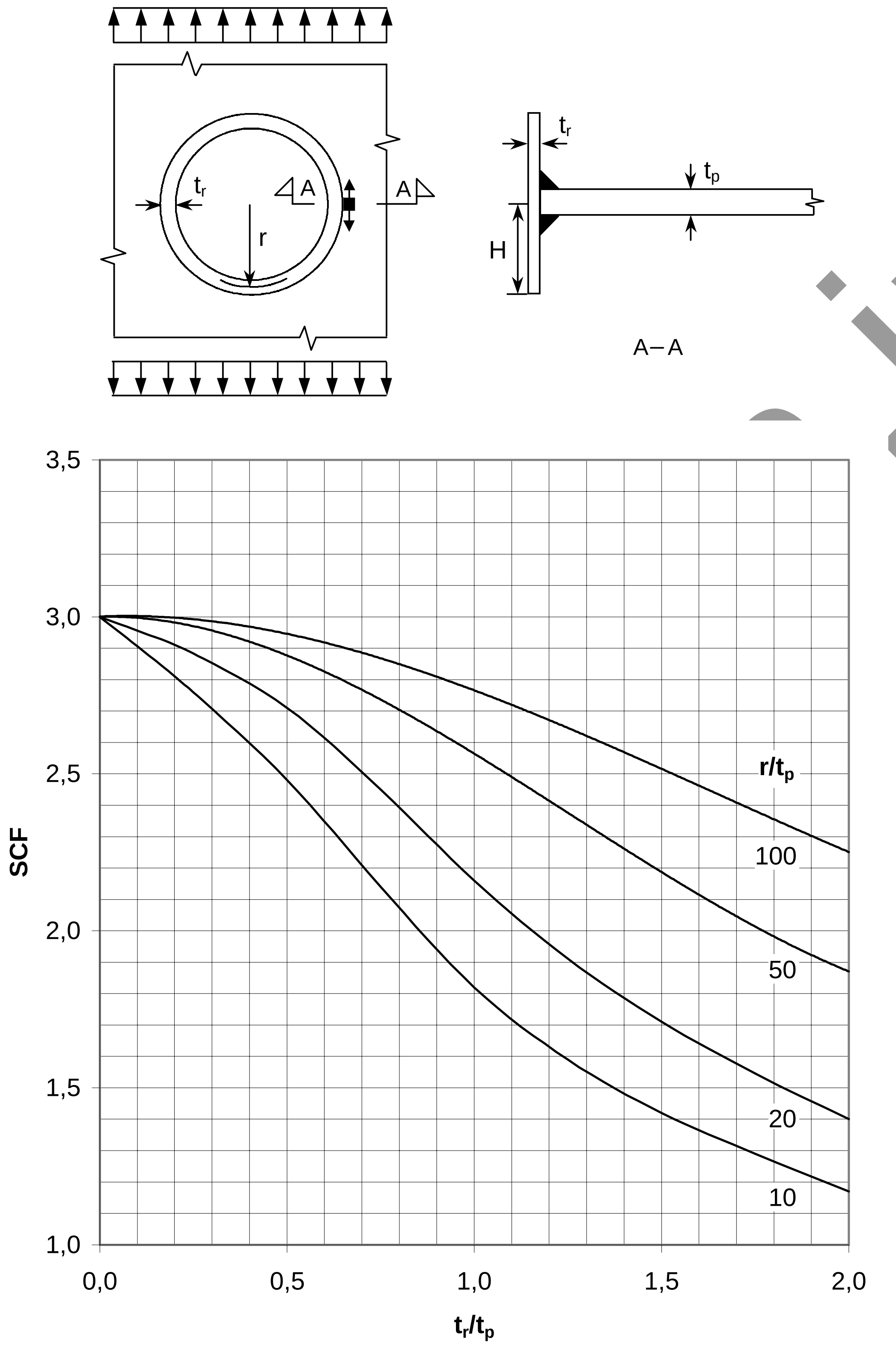


Figure 5 SCF at hole with inserted tubular. Stress in plate, parallel with weld. $H/t_r = 2$.

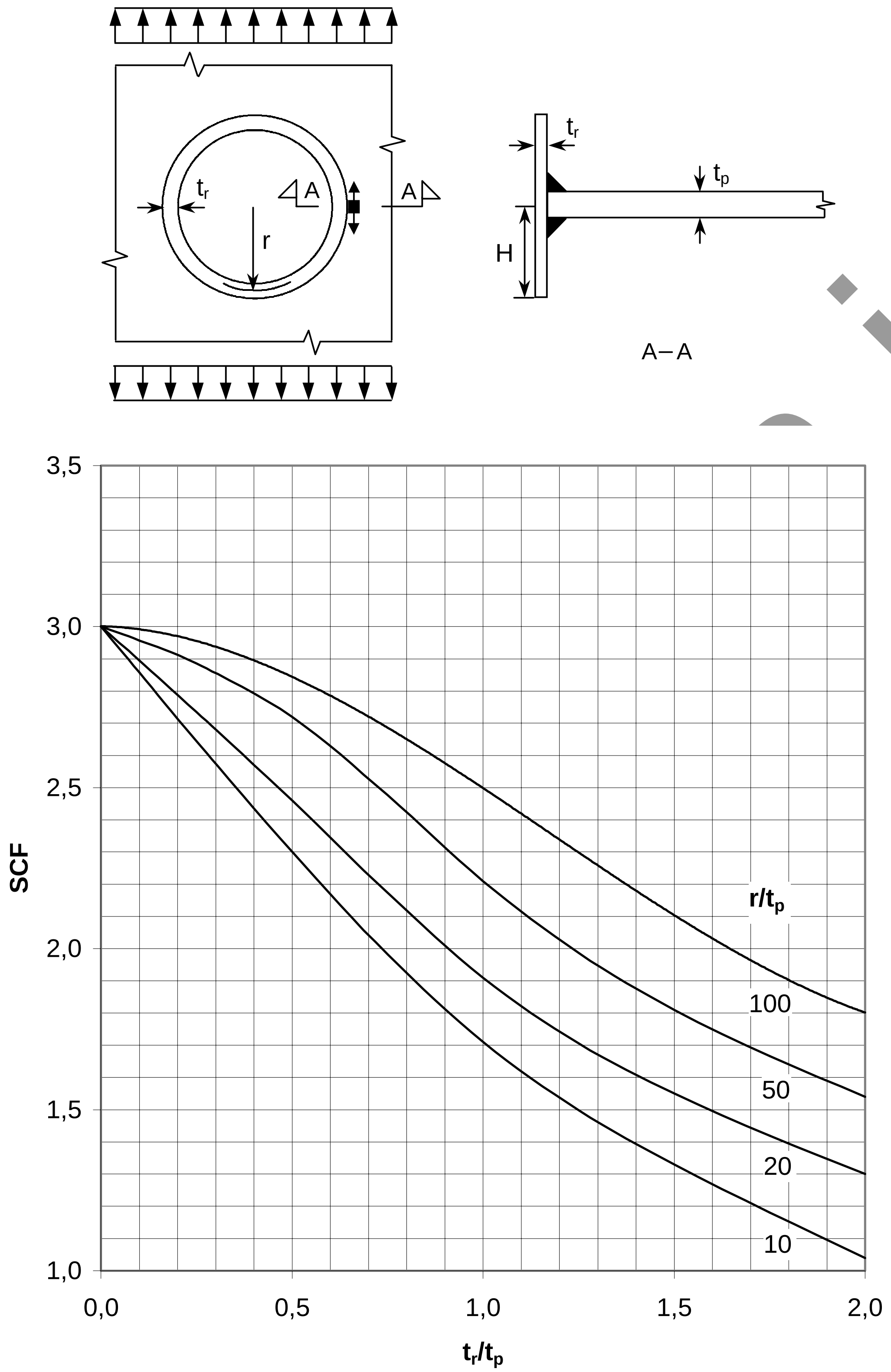


Figure 6 SCF at hole with inserted tubular. Stress in plate, parallel with weld. $H/t_r = 5$.

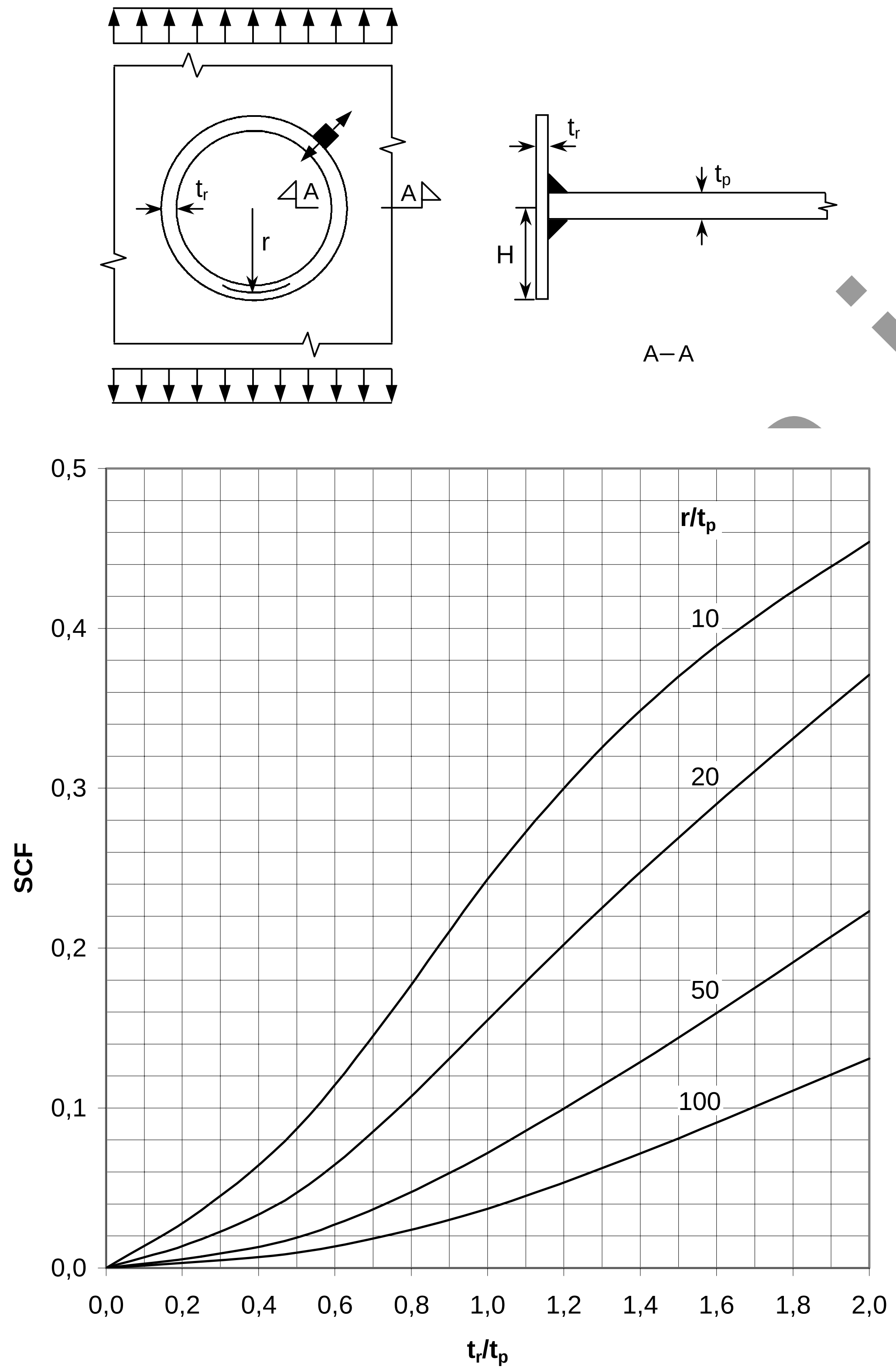


Figure 7 SCF at hole with inserted tubular. Stress in plate, normal to weld. $H/t_r = 2$

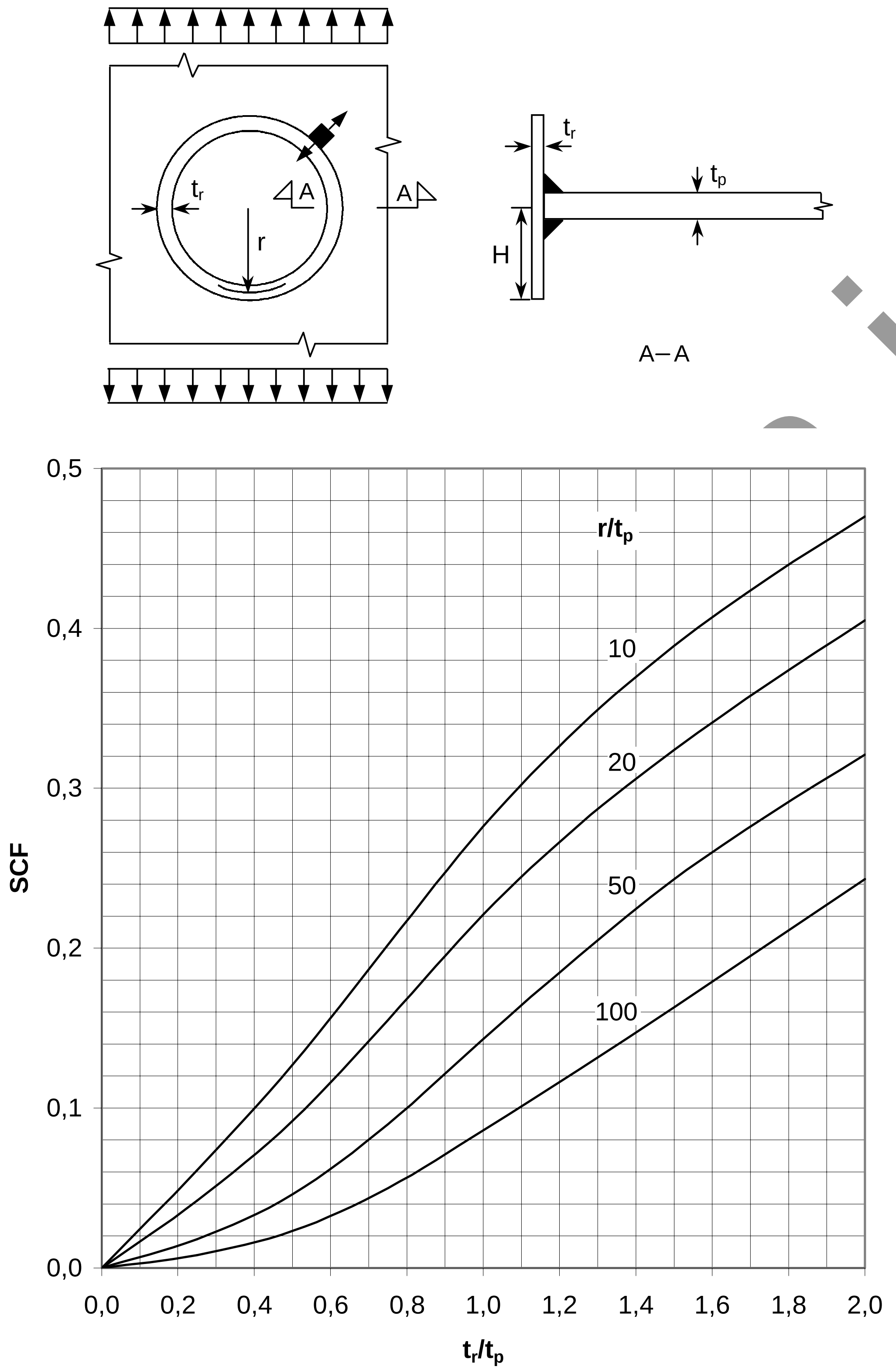


Figure 8 SCF at hole with inserted tubular. Stress in plate, normal to weld. $H/t_r = 5$

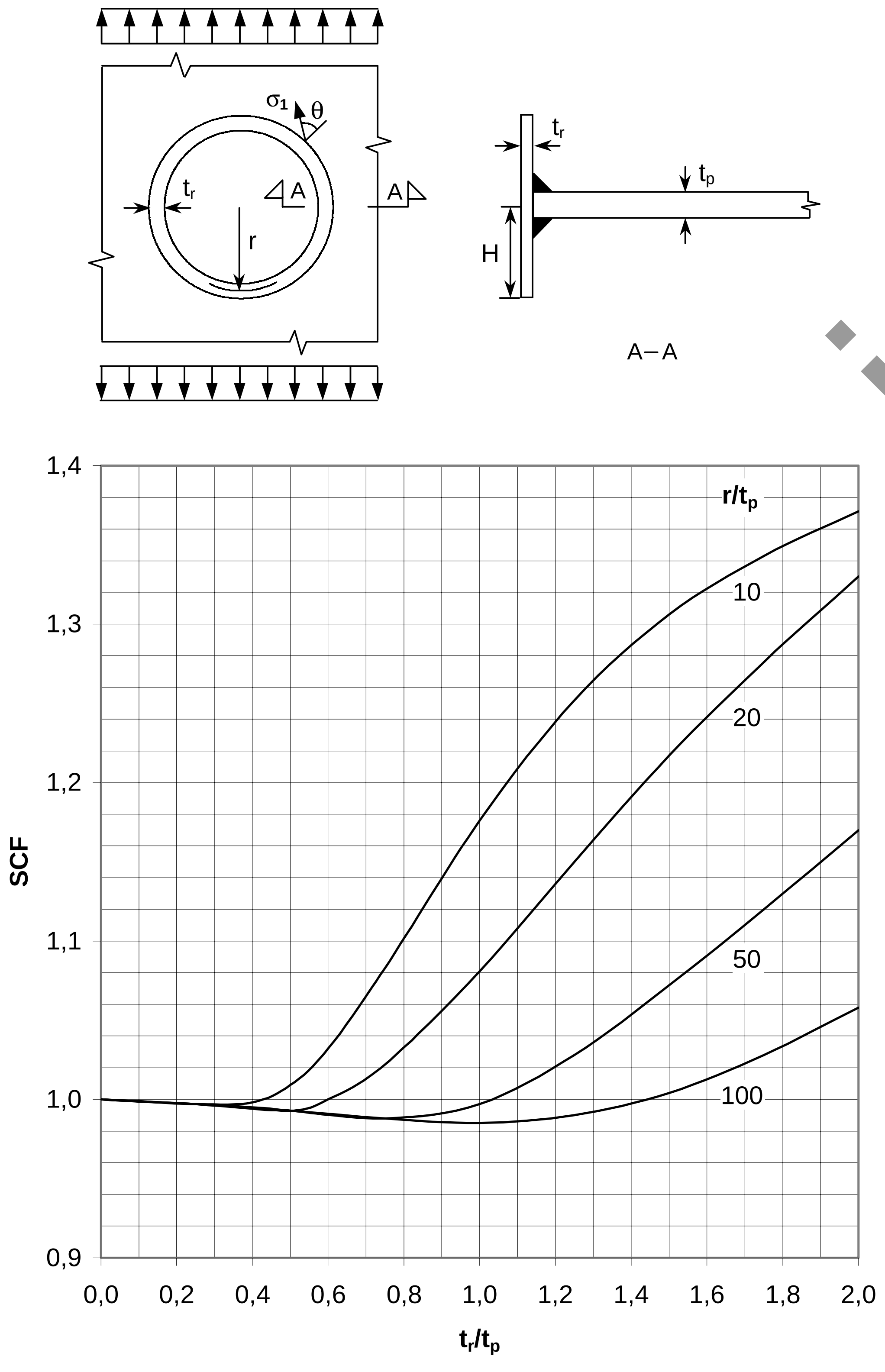


Figure 9 SCF at hole with inserted tubular. Principal stress in plate. $H/t_r = 2$

Table 1 θ = angle to principal stress. $H/t_r = 2$

t_r/t_p	$r/t_p=10$	$r/t_p=20$	$r/t_p=50$	$r/t_p=100$
0.0	90	90	90	90
0.5	72	80	86	88
1.0	56	63	75	82
1.5	50	54	64	73
2.0	46	50	57	66

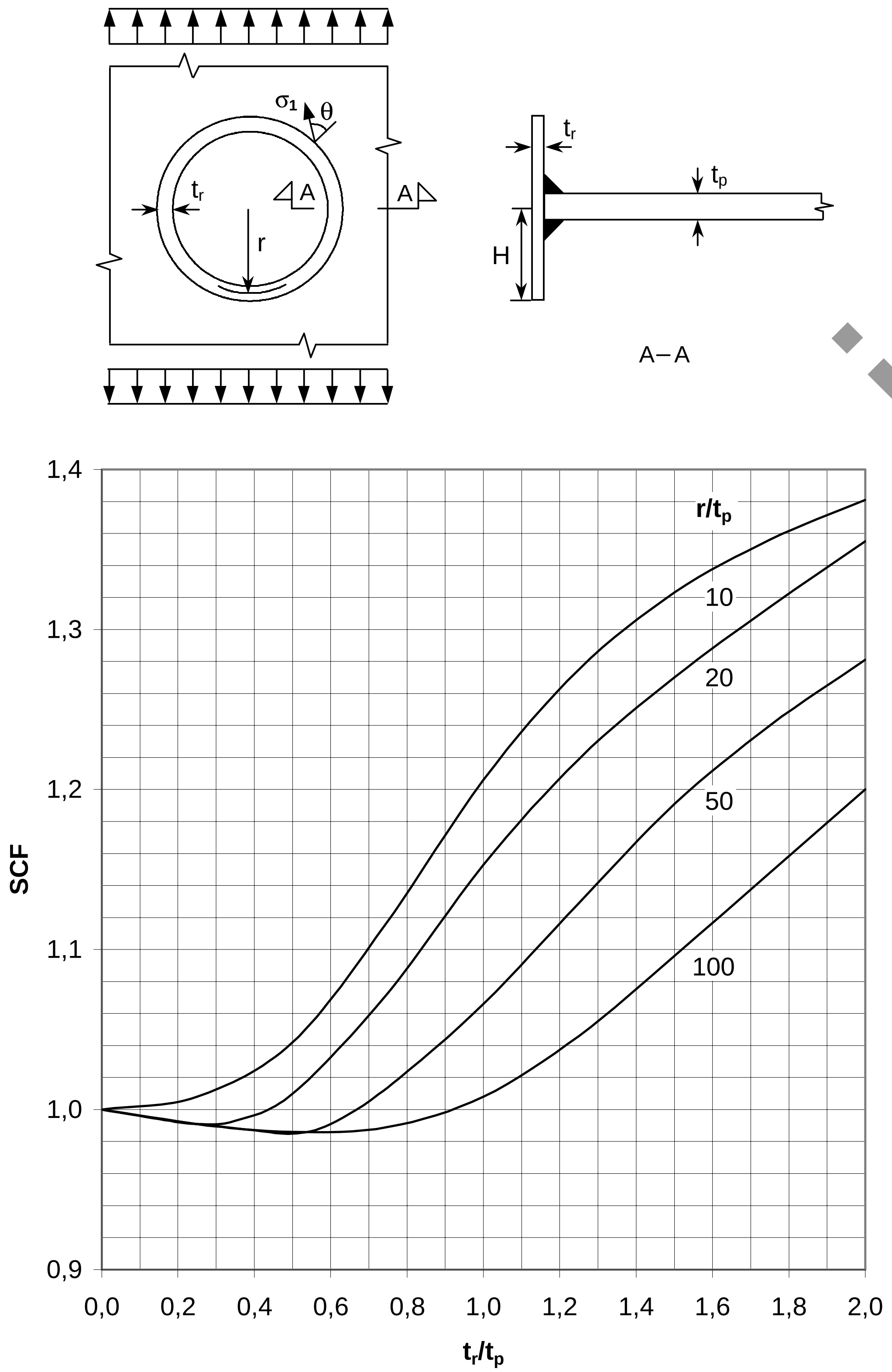
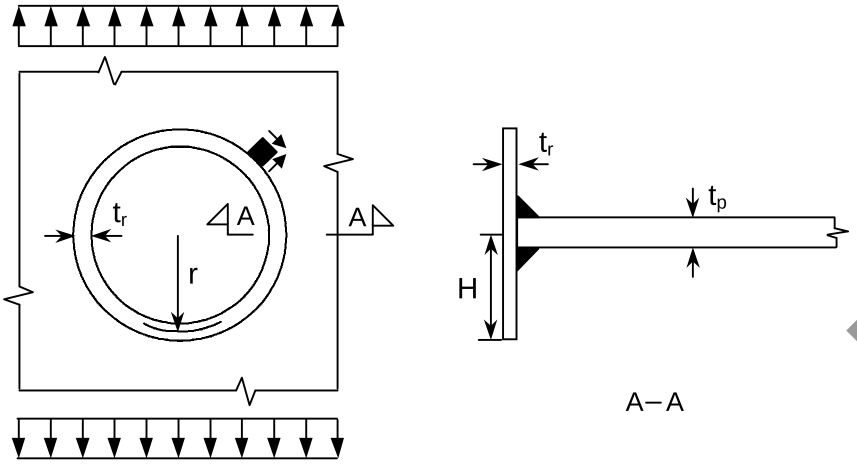


Figure 10 SCF at hole with inserted tubular. Principal stress in plate. $H/t_r = 5$

Table 2 θ = angle to principal stress. $H/t_r = 5$

t_r/t_p	r/t_p=10	r/t_p=20	r/t_p=50	r/t_p=100
0.0	90	90	90	90
0.5	66	72	80	85
1.0	54	58	65	72
1.5	49	52	56	62
2.0	46	48	52	56



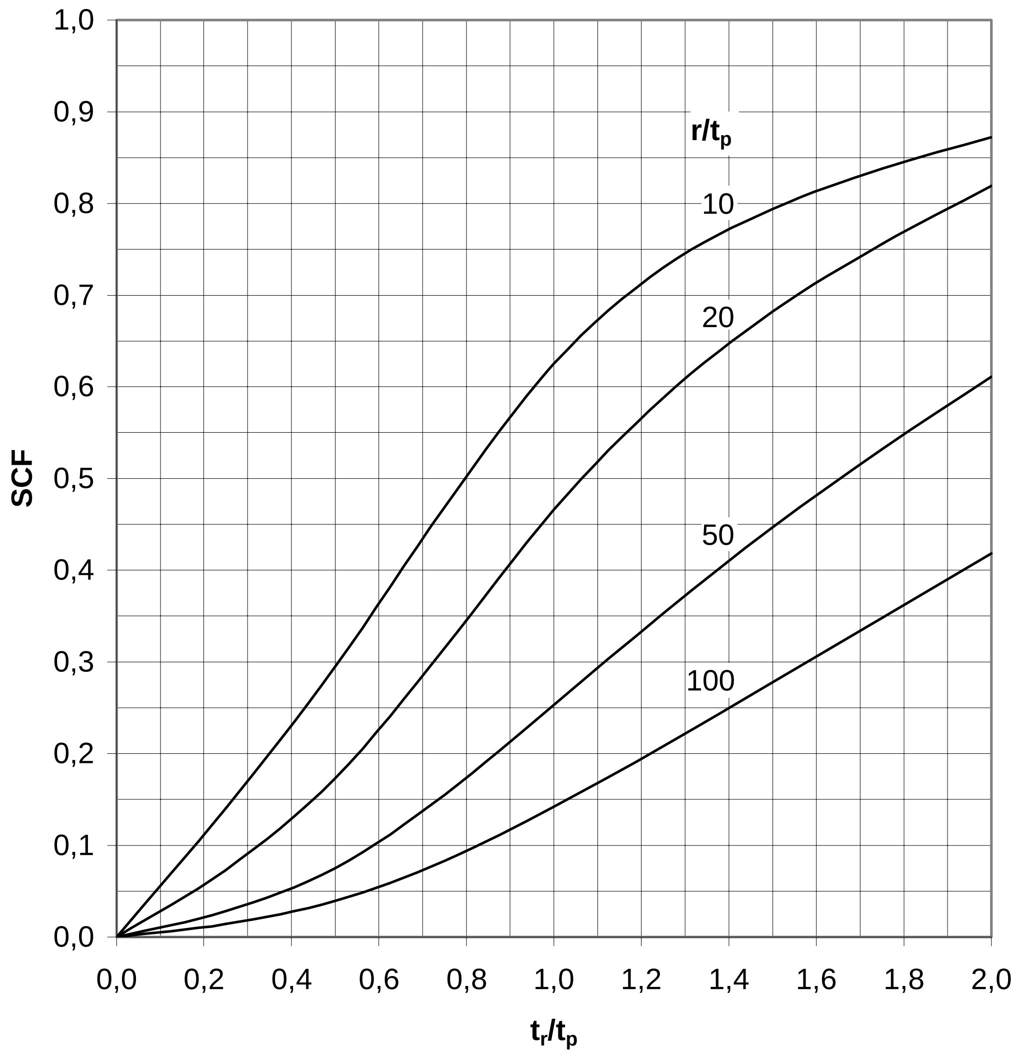
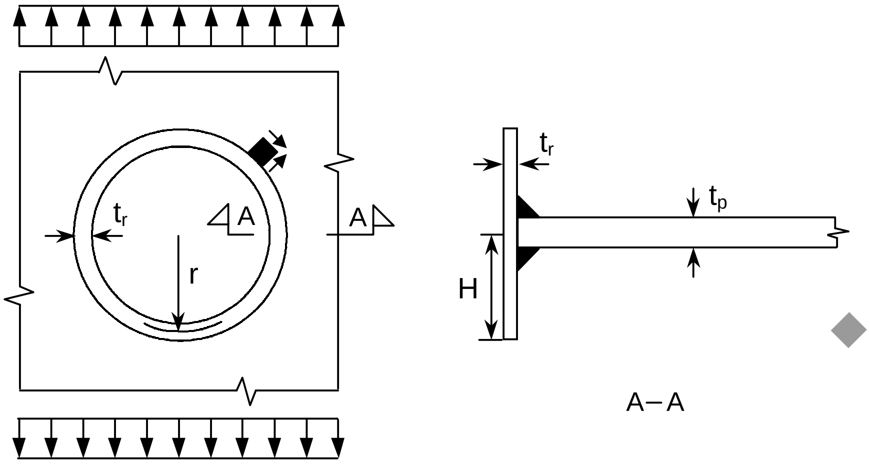


Figure 11 SCF at hole with inserted tubular. Shear stress in plate. $H/t_r = 2$



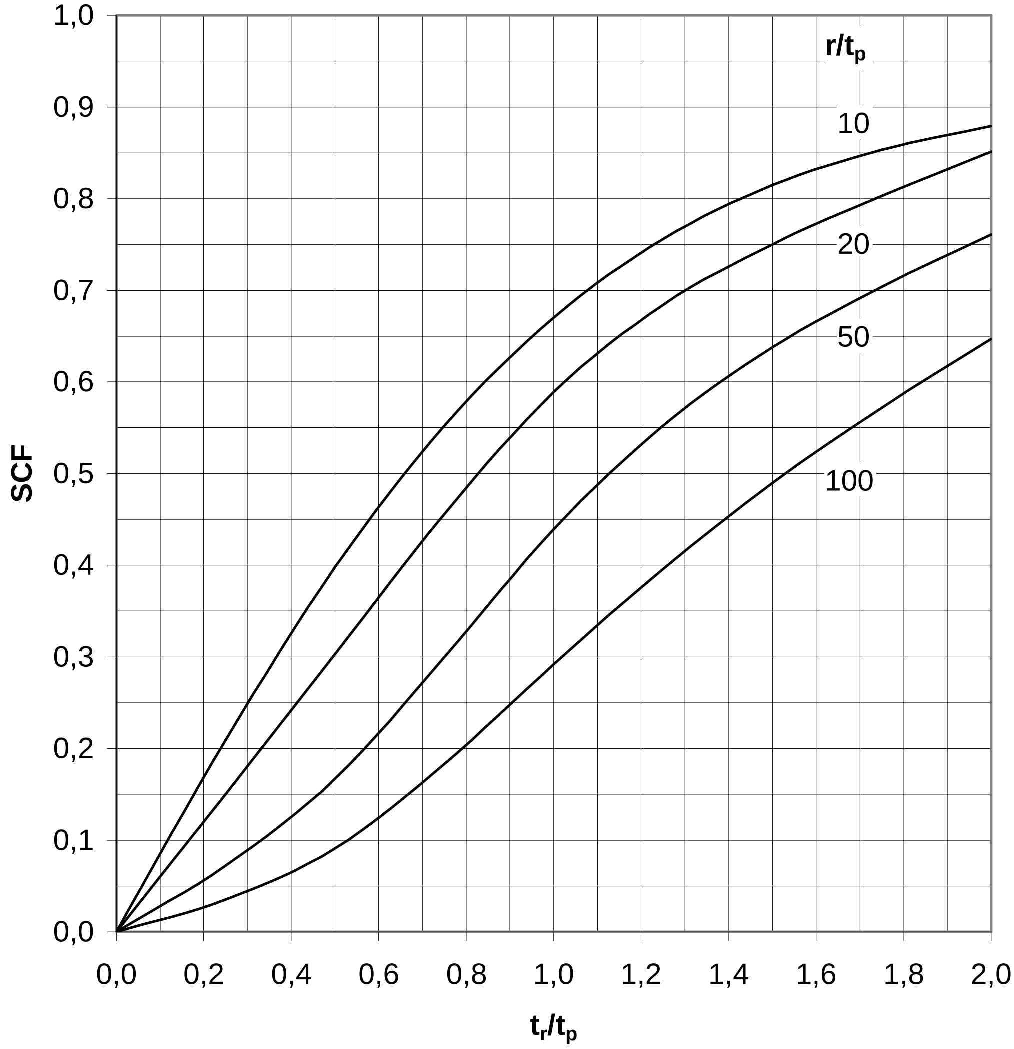


Figure 12 SCF at hole with inserted tubular. Shear stress in plate. $H/t_r = 5$

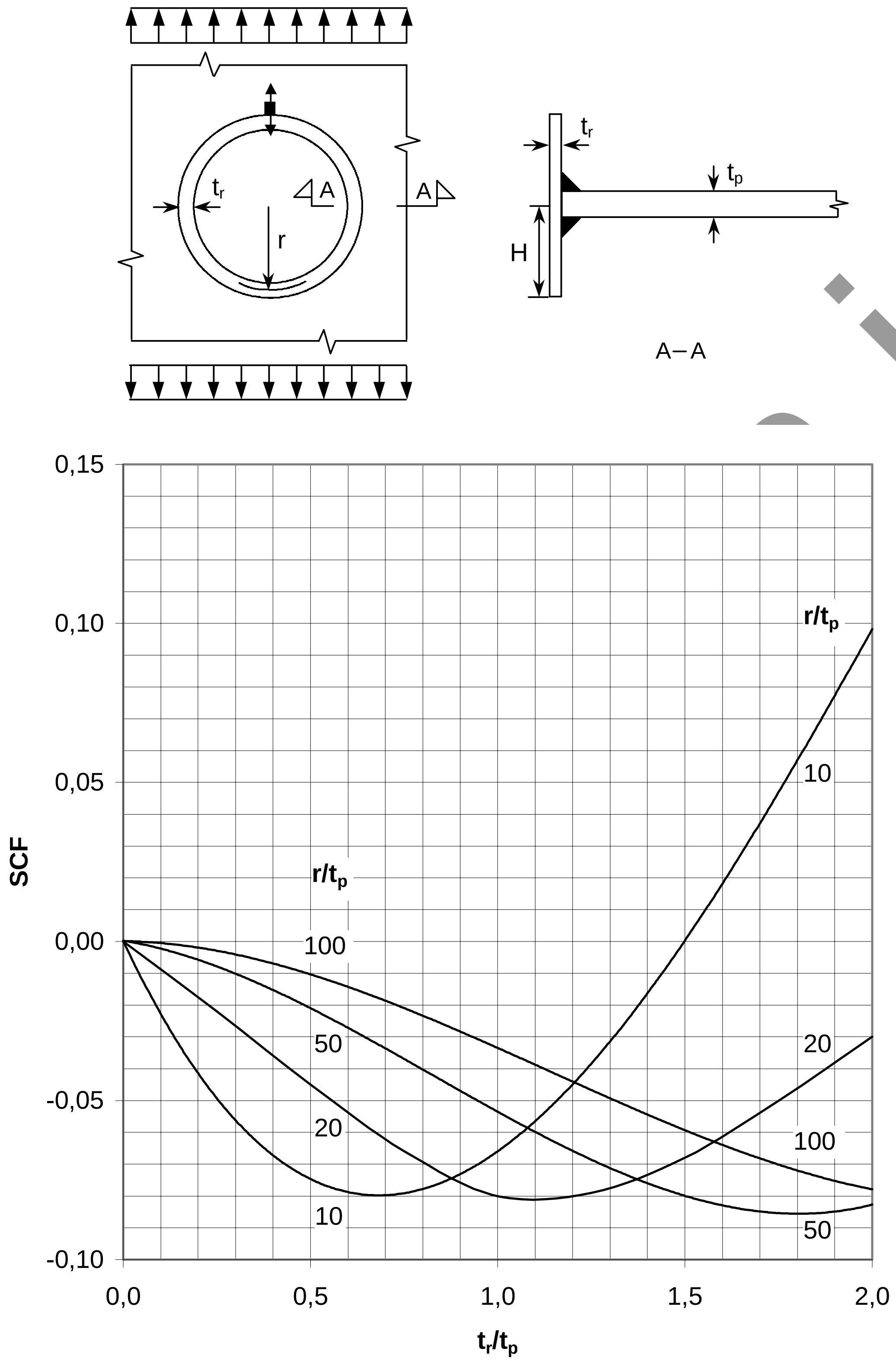
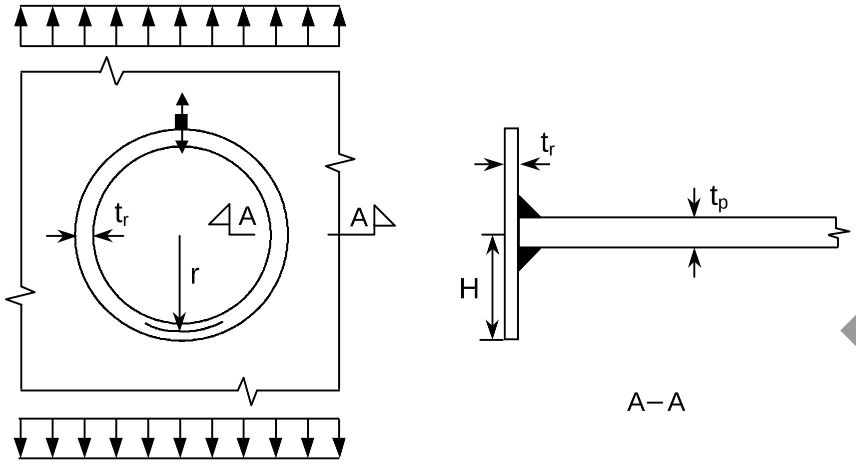


Figure 13 SCF at hole with inserted tubular. Stress in plate, normal to weld. $H/t_r = 2$.



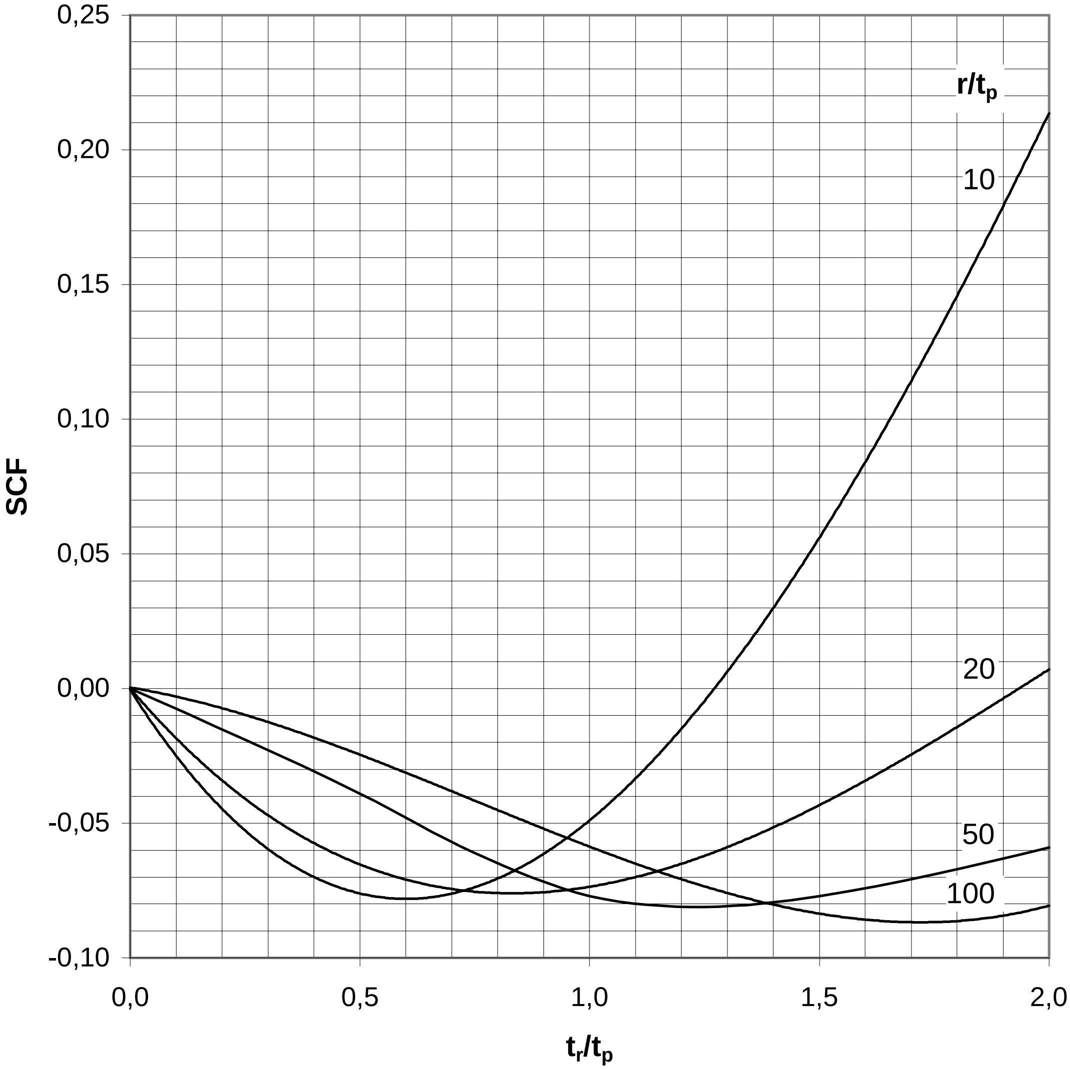


Figure 14 SCF at hole with inserted tubular. Stress in plate, normal to weld. $H/t_r = 5$

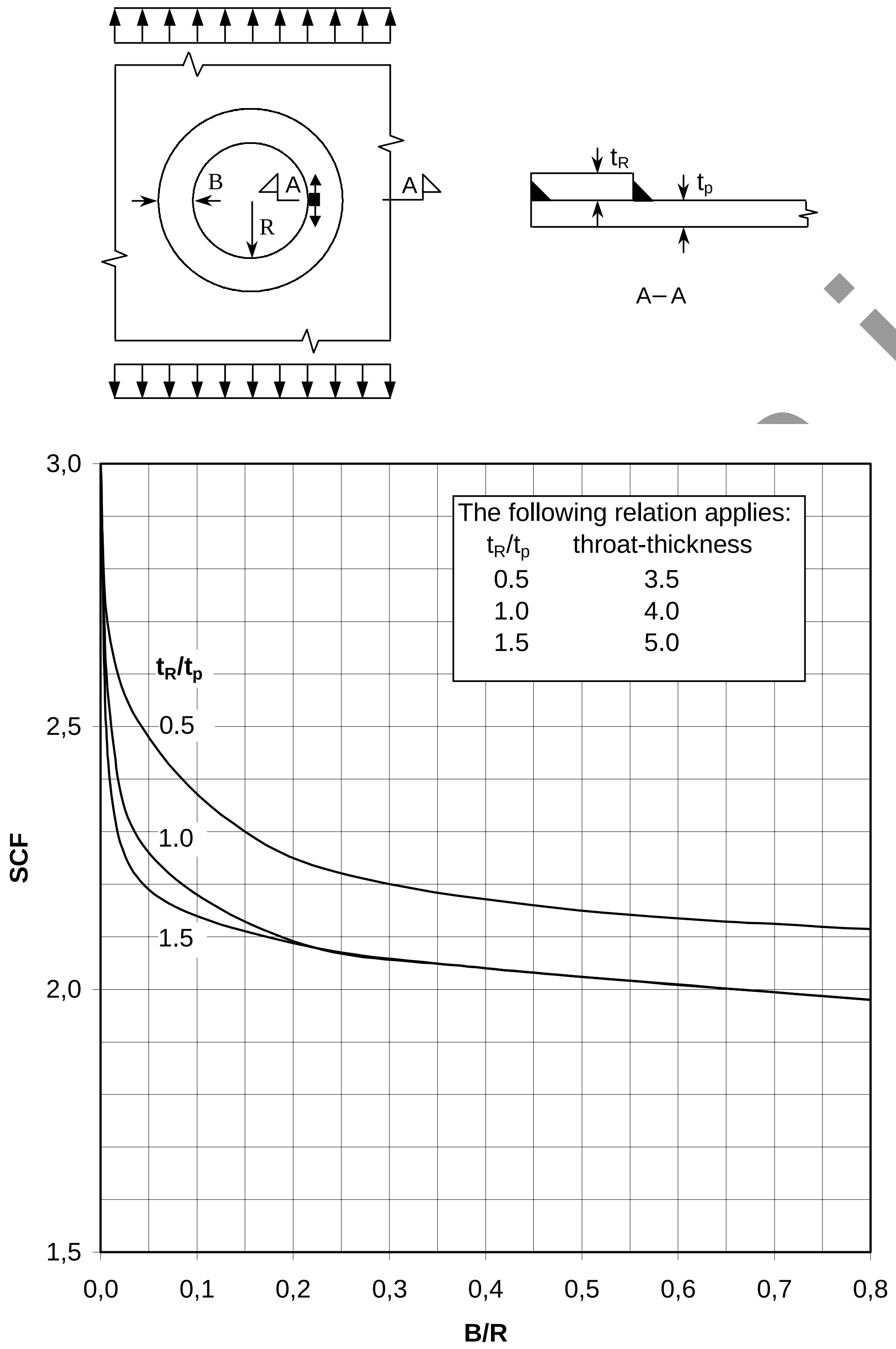


Figure 15 SCF at hole with ring reinforcement. Stress at inner edge of ring.

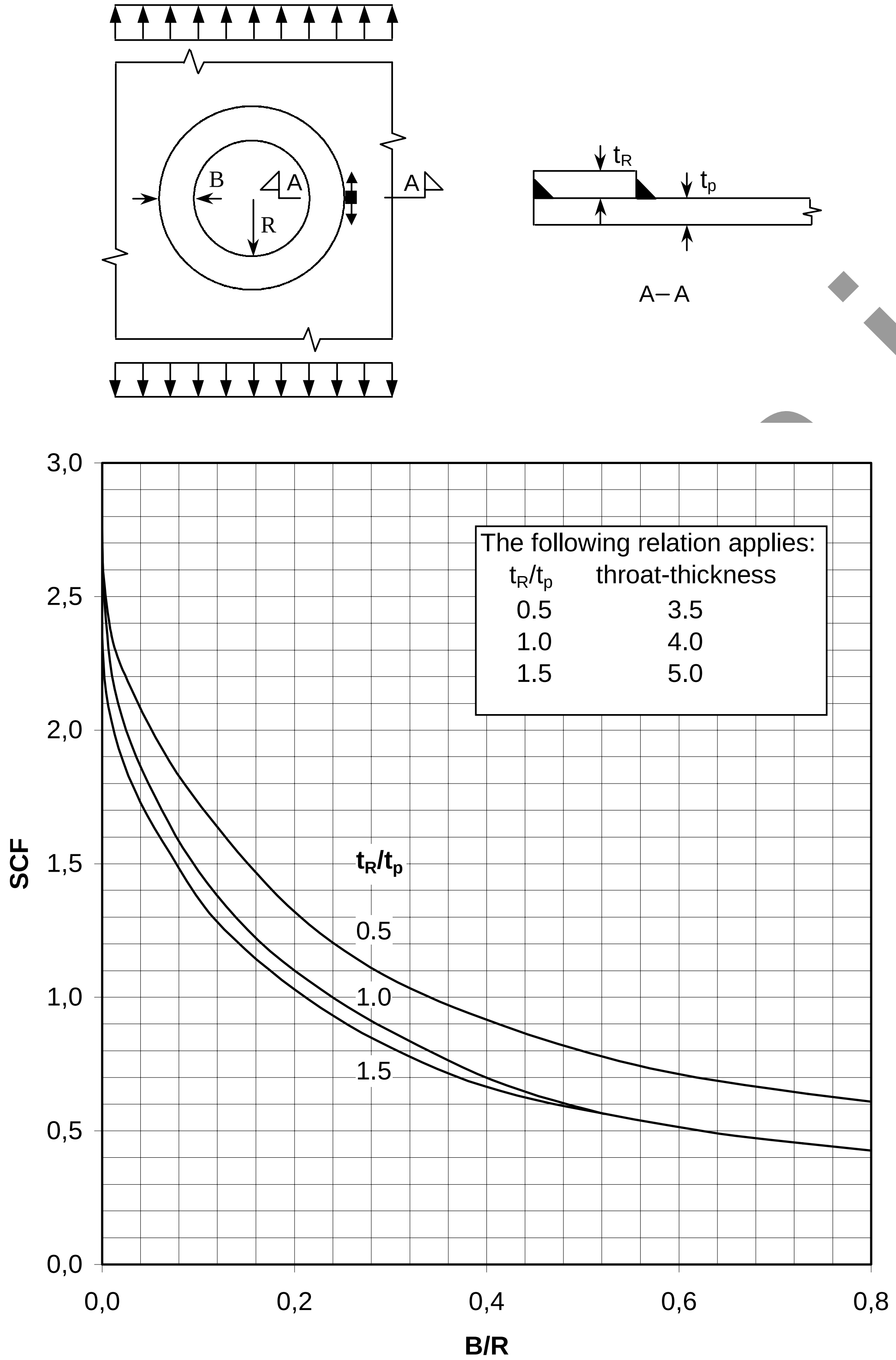


Figure 16 SCF at hole with ring reinforcement. Stress in plate, parallel with weld.

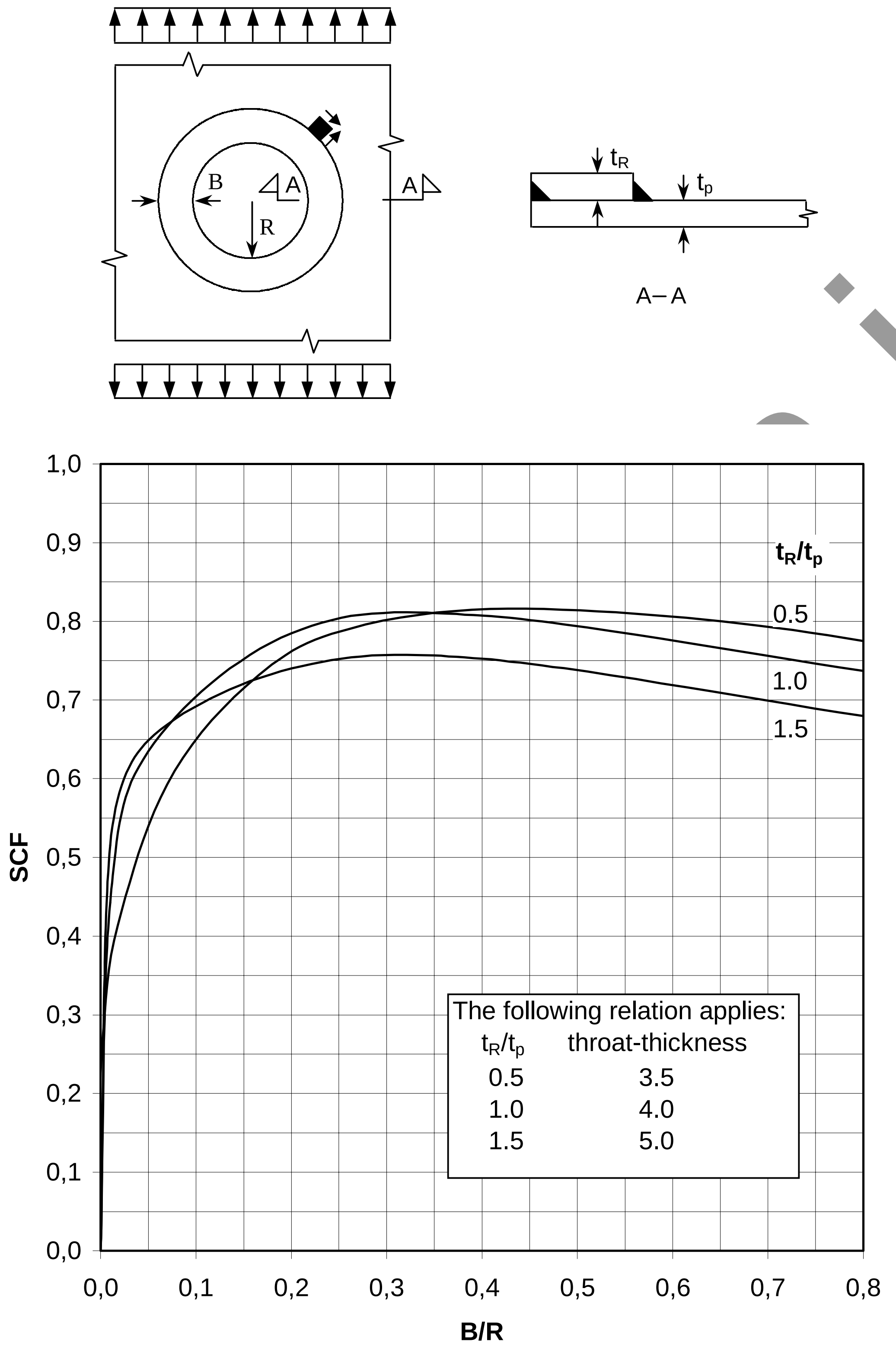


Figure 17 SCF at hole with ring reinforcement. Shear stress in weld.

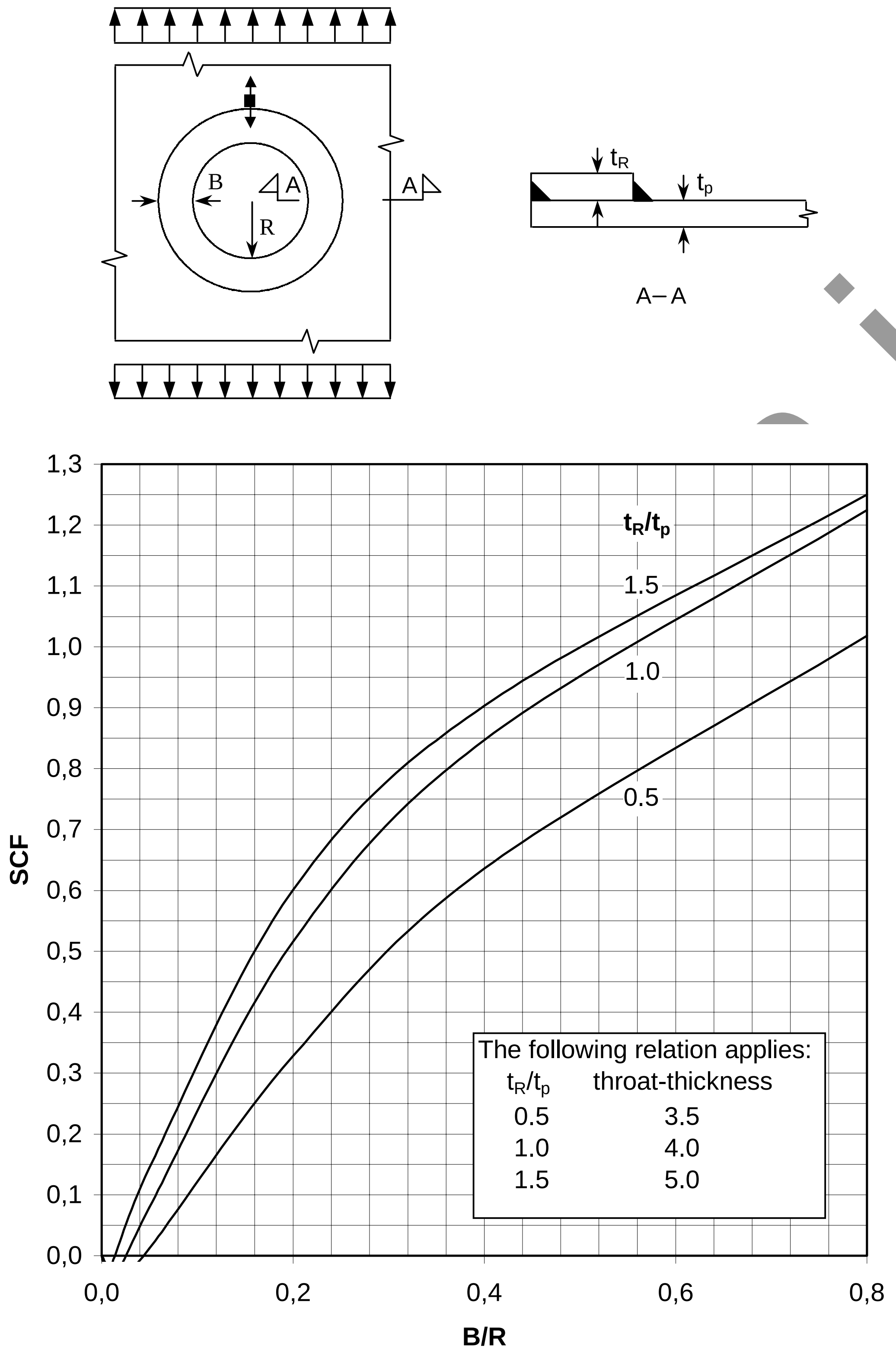
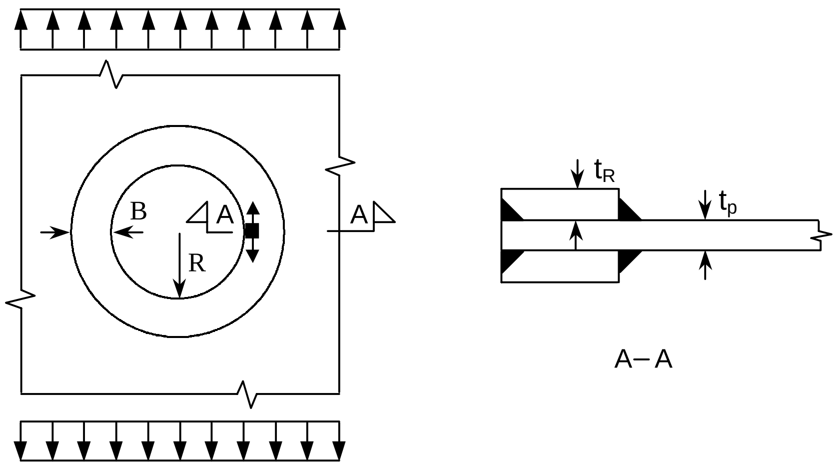


Figure 18 SCF at hole with ring reinforcement. Stress in plate, normal to weld.



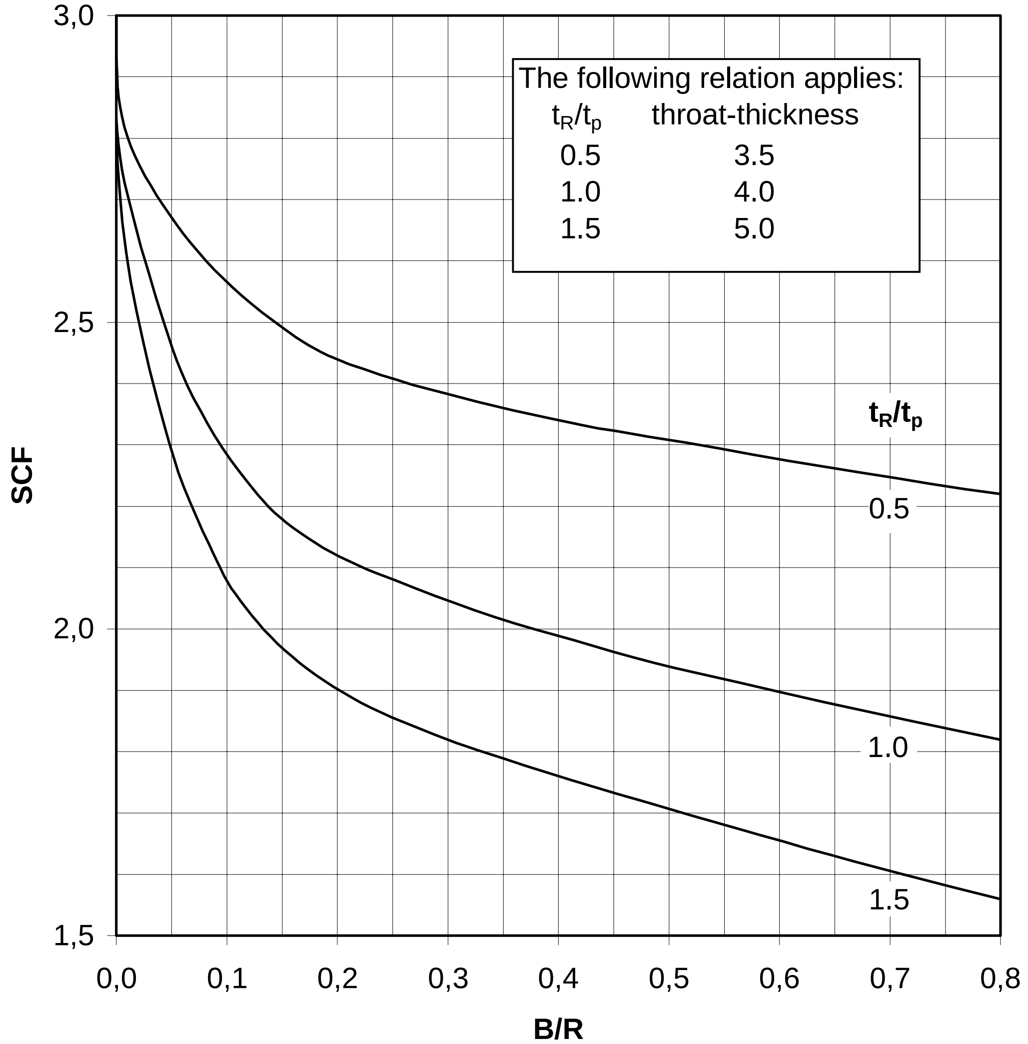


Figure 19 SCF at hole with double ring reinforcement. Stress at inner edge of ring.

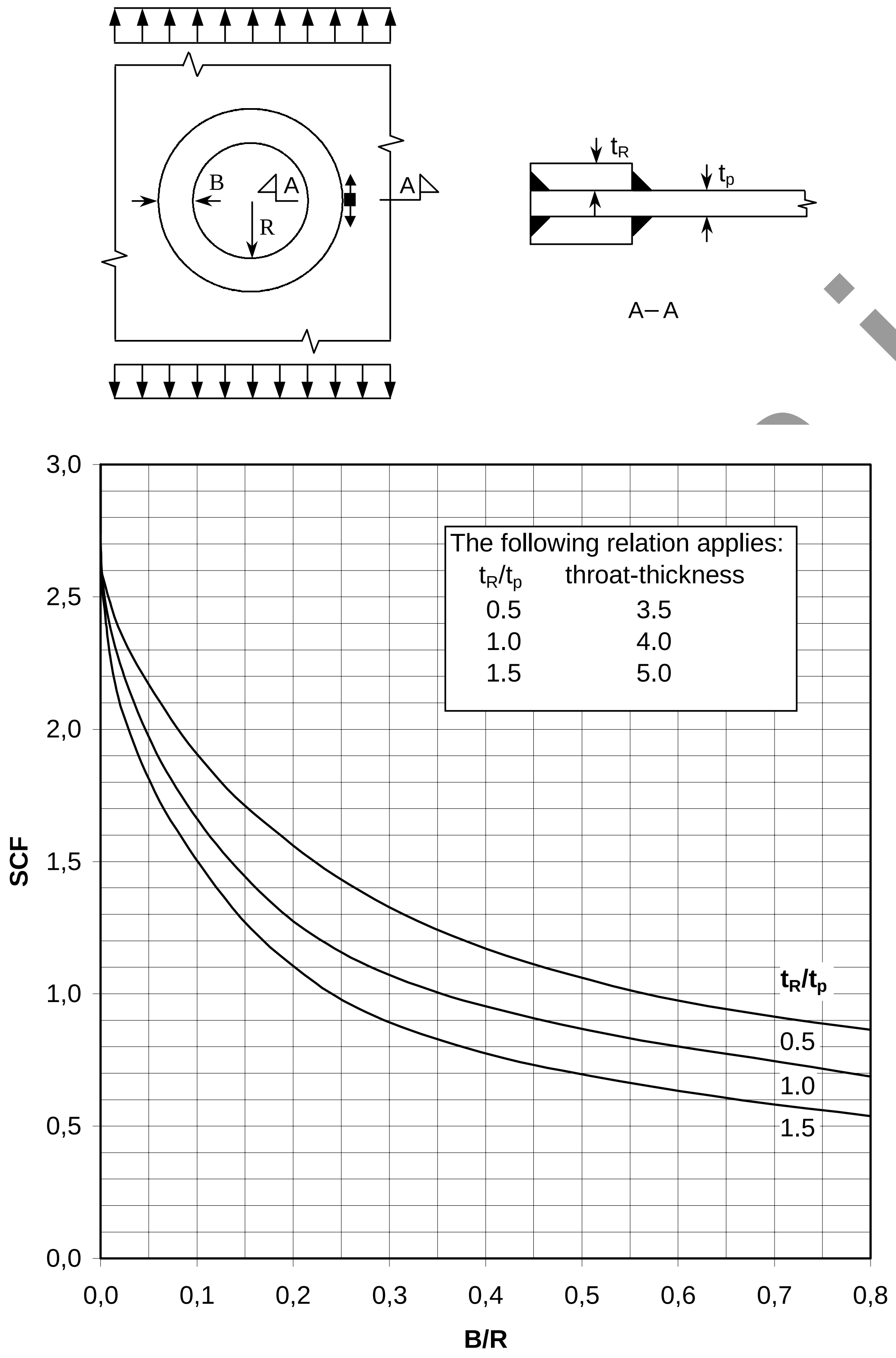


Figure 20 SCF at hole with double ring reinforcement. Stress in plate, parallel with weld.

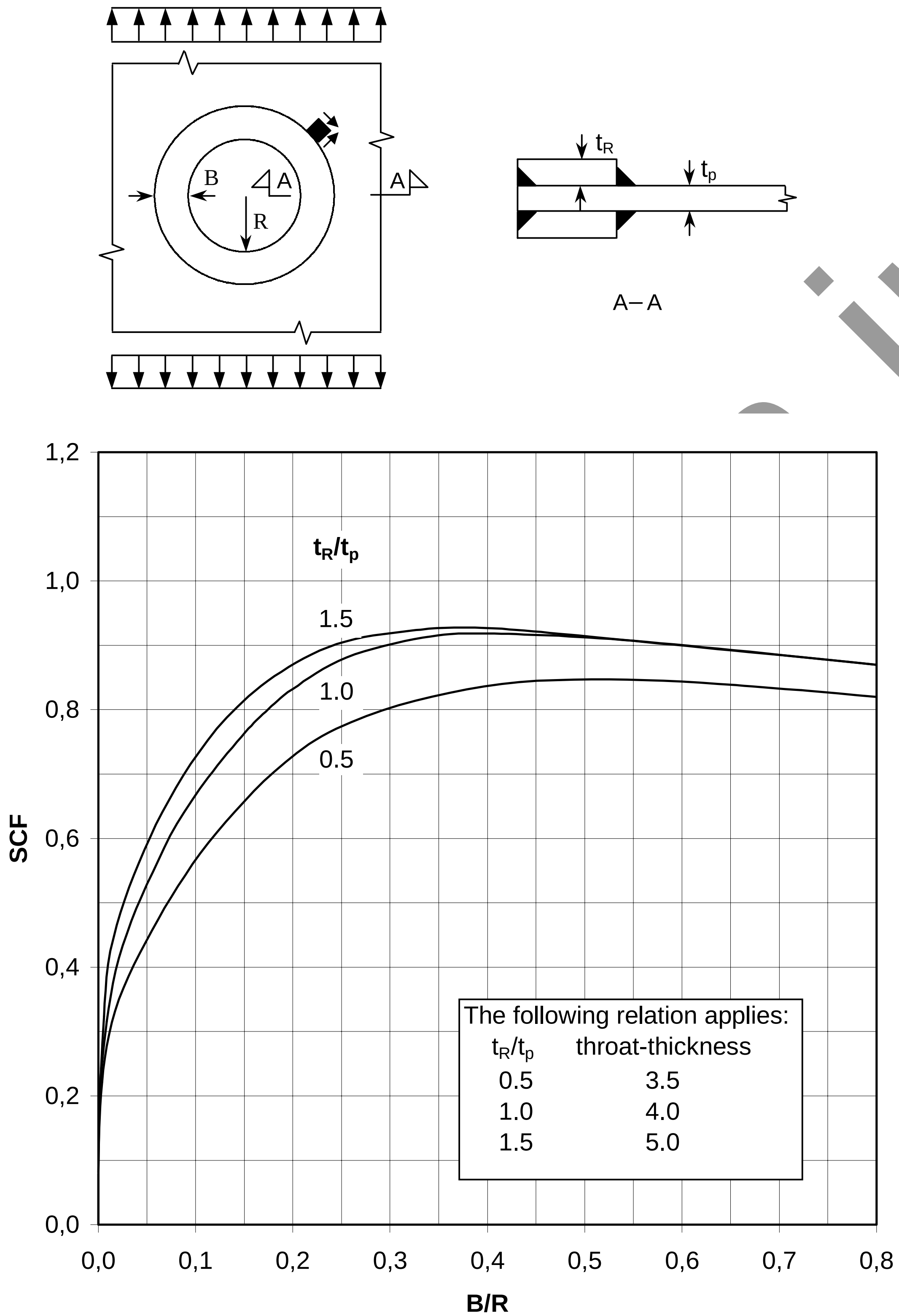
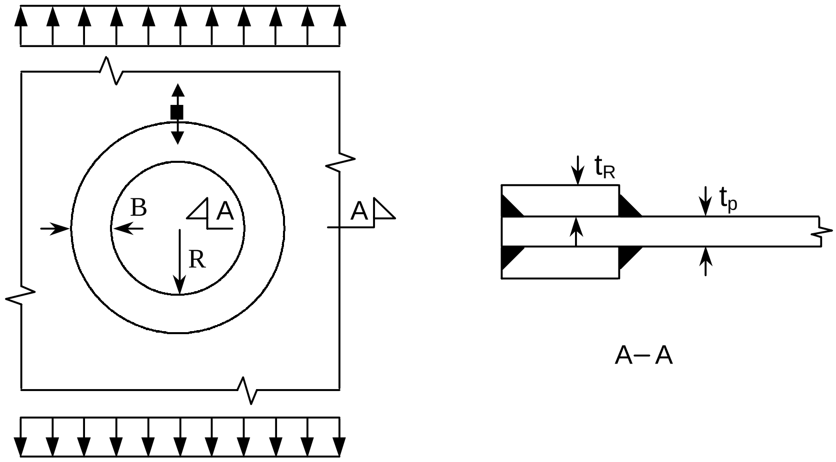


Figure 21 SCF at hole with double ring reinforcement. Shear stress in weld.



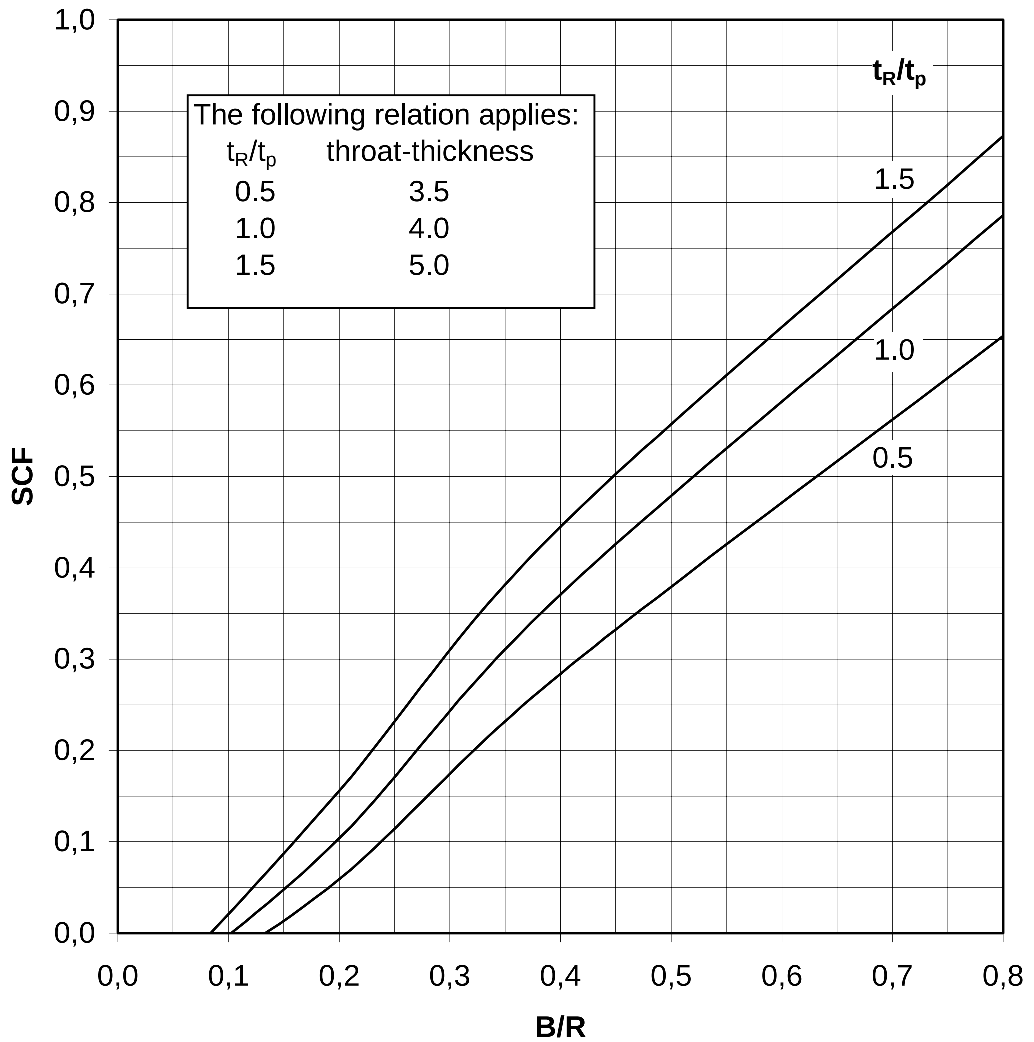


Figure 22 SCF at hole with double ring reinforcement. Stress in plate, normal to weld.

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DESIGN OF STEEL STRUCTURES

ANNEX K
SPECIAL DESIGN PROVISIONS FOR JACKETS

CONTENTS

K.1 GENERAL

K.1.1 Introduction

This Annex to NORSOK N-004 is intended to give guidance on how jacket steel structures should be designed according to the provisions of relevant NORSOK standards. It is intended to give guidance to how the various standards should be applied for jacket structures, how important parameters should be selected and to give additional requirements specially relevant for jacket structures.

K.1.2 Definitions

Jacket:	A welded tubular space frame structure consisting of vertical or battered legs supported by a lateral bracing system. The jacket is designed to support the topside facilities, provide supports for conductors, risers, and other appurtenances and serve as a template for the foundation system.
Bottle leg:	Leg section with larger diameter to thickness ratio used as buoyancy compartment during installation and to facilitate effective pile cluster design.
Foundation pile:	Steel tubular driven into the soil and fixed to the jacket structure for transfere of global actions.
Pile cluster:	Pile sleeves for foundation piles arranged in groups with shear connections to jacket legs. Basis for mudmats and skirts.
Bucket foundation:	Steel plate construction integrated to bottom of jacket leg penetrating into soil for fixing platform to ground.
Multilegged jacket:	A jacket with more than 4 legs.

K.1.3 Design for non-operational phases

The jacket shall be designed to resist actions associated with conditions that will occur during stages from fabrication yard to the stage where the platform is ready for operation at final location. Such stages are transportation at the fabrication yard and load-out, sea transportation, installation, mating and hook-up.

Abandonment of the jacket shall be planned for in design.

K.1.4 Design for operational phases

The jacket shall be designed to resist actions caused by gravity, wind, waves and current, earthquake and accidental actions that may occur during its service life.

Each mode of operation of the platform, such as drilling, production, work over or combinations thereof should be considered.

The fatigue requirements as given in Section 8 shall be fulfilled.

K.2 STRUCTURAL CLASSIFICATION

K.2.1 Structural classification

Selection of steel quality level and requirements for inspection of welds shall be based on a systematic classification of welded joints according to the structural significance and complexity of joints.

The main criterion for decision of design class (DC) of welded joints is the significance with respect to joint complexity and the consequences of failure of the joint. In addition the stress predictability (complexity) will influence the DC selection. Classification of DC for a typical 4-legged jacket is given in Table K.2-1 and Figure K.2-1.

A practical limit for water depths for which inservice inspection can be performed by use of diver technology may be set to 150 m.

Table K.2-1 Typical design classes in jackets

Joint/ Component	*Design Class* *DC*	*Steel Quality Level S.Q.L*	*Inspection Category* *Note a)*	*Comments*
Legs and main bracing system				
Leg nodes & cones / Butt welds Legs nodes & cones / Longitud. welds	2	(I) or II	A or B B	*Note c), Note d)*
Leg strakes / Butt welds Leg strakes / Longitudinal welds	2	II	A or B B	*Note c), Note d)*
Lift-nodes / complex Lift-nodes / simple	1 2	I (I) or II	A or B A or B	
Nodes in vertical bracing / Butt welds Nodes in vertical bracing / Longitudinal welds	4	II	B C	
Vertical bracing / Butt welds Vertical bracing / Longitud. welds	4	II	B C	
Bottle leg / Butt welds Bottle leg / Longitudinal welds	2	II	A or B B	
Horizontal bracing / All welds Nodes horizontal bracings / All welds	4	III	B or C B or C	
Watertight diaphragms / All welds Ring stiffeners, main nodes / All welds Ring stiffeners bottle leg / All welds	2	II (I) or II (I) or II	A or B A A	
Other stiffening / All welds	5	III	D or E	
Foundation system				
Mudmat & yoke plate incl. stiffening / All welds	4	III	C	*Note e)*
Skirts & bucket foundation plates incl. stiffening / All welds	4	III	C	
Shear plates / All welds Pile sleeves / All welds	4 4	III III	C C	*Note e)* *Note e)*
Pile sleeve catcher, cone & spacers	5	IV	D or E	
Piles, top part / Butt welds Piles, top part / Longitud. welds Piles, remaining / All welds	4	III	A	*Note e)*
Appurtenances and outfitting steel				
Riser guides / All welds J-tubes & supports / All welds Conductor support / All welds Caissons & support / All welds	4 4 4 5	III III III IV	B or C	*Note b)*
Outfitting / Butt welds Outfitting / Part-Pen. & Fillets	4 or 5	III or IV	D or E	*Note b)*

Notes:

a) Local areas of welds with high utilisation shall be marked with frames showing areas for
b) mandatory NDT when partial NDT are selected. Inspection categories depending on access for in-service inspection and repair.
c) Outfitting structures are normally of minor importance for the structural safety and integrity. However, in certain cases the operational safety is directly influenced by the outfitting and special assessment is required in design and fabrication.
A typical example is guides and supports for gas risers.
d) If multilegged jacket with corner legs supporting a foundation systems;
e) Upper part of corner legs and inner legs: DC 4, III, B or C
f) Lower part of corner legs: DC 2, II, A or B
g) If multilegged jacket where each leg supporting a foundation system: DC 4, III, B or C
h) If one or two pile(s) per leg: DC2, II.

When the design class is defined the material shall be selected according to Section 5.

The drawings shall indicate the inspection category for all welds according to Section 5.

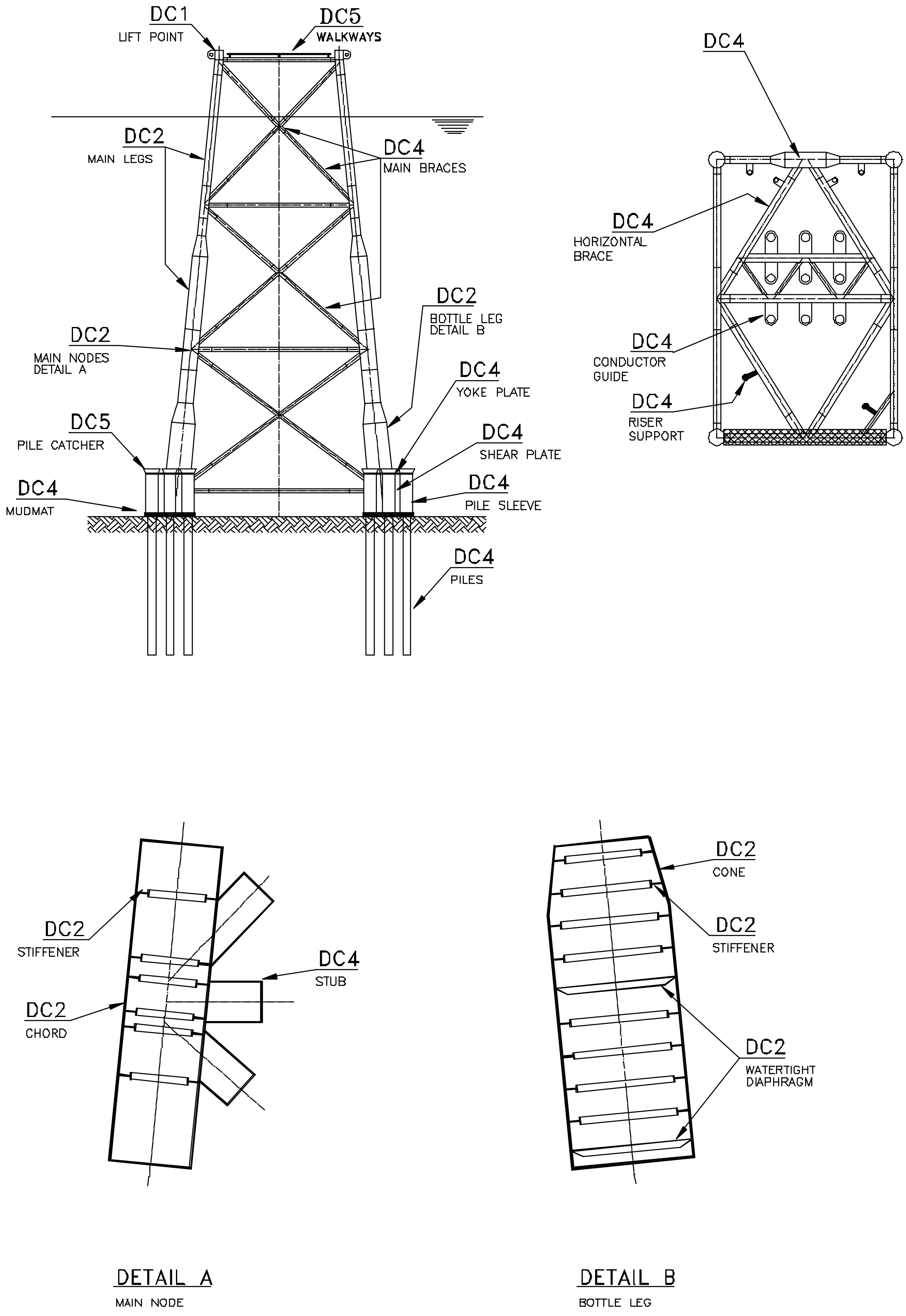

Figure K.2-1 Typical design classes. 4-legged jacket with pileclusters.

K.3 DESIGN ACTIONS

K.3.1 General

Characteristic actions shall be used as reference actions in the partial coefficient method.

Design actions are in general defined in NORSOK N-003.

Applicable action categories and combinations of actions relevant for jacket design are given in this Section.

K.3.2 Permanent action

Permanent actions are actions that will not vary in magnitude, position or direction during the period considered. Permanent actions relevant for jacket design are:

- mass of jacket incl. piles above mudline
- mass of appurtenances supported by the jacket and/or topside
- mass of permanently installed topside such as accommodation, drilling and production equipment
- buoyancy and hydrostatic pressure from sea water
- mass of marine growth

For design of a jacket, the characteristic value of permanent actions representing the mass of steel to be designed shall include contingency factors. The contingency plan shall reflect the uncertainties in the weight and material estimates as expressed by the stage in the calculations process.

At the detail design stage, the characteristic values of permanent actions are to be verified by accurate measurements (weight control) or calculated based upon accurate data.

K.3.3 Variable action

Variable actions on deck areas are actions that may vary in magnitude, position and direction during the period under consideration.

Normally, a jacket design is based upon budget weight of topside and a specified envelope of centre of gravity to account for variable actions on deck areas.

K.3.4 Deformation action

Deformation actions are actions due to deformations applied to the structure.

Deformation actions relevant for jacket design are:

- pre-stressing
- temperature
- barge deflections
- differential settlements
- uneven seabed
- field subsidence effects

K.3.5 Environmental action

K.3.5.1 General

The characteristic value of an environmental action is the maximum or minimum value (whichever is the most unfavourable) corresponding to an action effect with a prescribed probability of exceedance.

The environmental conditions should be described using relevant data for the relevant period and areas in which the jacket is to be fabricated, transported, installed and operated.

The long-term variation of environmental phenomena such as wind, waves and current should be described by recognised statistical distributions relevant to the environmental parameter considered. Information on the joint probability of the various environmental actions may be taken into account if such information is available and can be adequately documented.

Details of environmental actions are found in NORSOK N-003.

K.3.5.2 Wave and current action

Wave actions may be described either by deterministic or statistical methods. Normally, a deterministic analysis method is used for jacket design.

In deterministic design analysis based on regular wave formulation, the wave is described by wave period, wave height and water depth. Stokes 5^{th} order wave kinematic theory shall be used.

In stochastic wave description, the short-term irregular sea states are described by means of wave energy spectra which are normally characterised by significant wave height (H_s), and average zero-up-crossing period (T_z), or spectral peak period (T_p).

Wave and current actions on the jacket structure are normally calculated by use of Morison's equation. Member drag (C_D) and inertia (C_M) coeffisients shall be established according to NORSOK N-003.

Shielding effects may be taken into account where the presence of such effects can be adequately documented, (NORSOK N-003).

Where elements are closely grouped (e.g. conductors), solidification effects shall be considered.

The design current velocity and profile is to be selected using site specific statistics. The current applied in design shall include both tidal and wind induced effects. The current profile is normally defined up to still water level. In combination with waves, the profile shall be adjusted such that the flux of the current is kept constant.

K.3.5.3 Wind action

Extreme values of wind speeds are to be expressed in terms of most probable largest values with their corresponding recurrence periods.

Design of jacket and foundation shall be based upon the 1 hour extreme mean wind defined at a reference height of 10m above still water level. The 1 minute wind gust parameter shall be used when wind action is applied in combination with the wave, and the wave action being the dominating action.

Relevant scaling methods to obtain averaged wind speeds and wind height profiles is given in NORSOK N-003.

K.3.5.4 Earthquake action

For earthquake analyses, ground motions may be defined either as response spectra or time histories.

The selection of method depends on the actual problem being considered.

For deep-water jacket structures with long fundamental periods and significant soil-structure interaction effects, time-history analysis is normally recommended.

If inelastic material behaviour is considered, time-history methods are normally necessary.

When performing time-history earthquake analysis, the response of the structure/foundation system shall be analysed for a representative set of time histories.

In areas with small or moderate earthquake activity, the ULS check may be omitted and the earthquake analysis may be limited to the ALS check only.

K.3.5.5 Ice

If the structure is to be located in an area where snow or icing may accumulate, icebergs, or sea-ice may develop or drift, actions from such phenomena shall be taken into account where relevant, ref. NORSOK N-003.

K.3.6 Accidental action

Accidental actions are actions related to abnormal operation or technical failure.

Details of accidental actions are found in NORSOK N-003.

Accidental actions relevant for jacket design are:

- impact from vessel
- dropped object from crane handling
- pool fire at sea
- extreme environmental actions

The determination of accidental actions is normally to be based on a risk analysis.

K.3.7 Fatigue actions

Repetitive actions, which may lead to possible significant fatigue damage, shall be evaluated. The following listed sources of fatigue actions shall, where relevant, be considered :

- waves (including those actions caused by slamming and variable (dynamic) pressures).
- wind (especially when vortex induced vibrations may occur), e.g. during fabrication, installation and operation.
- currents (especially when vortex induced vibrations may occur).

The effects of both local and global dynamic response shall be properly accounted for when determining response distributions related to fatigue actions.

K.3.8 Combination of actions

Action coefficients and combinations of actions for the different limit states are in general given in NORSOK N-003.

The Serviceability Limit State (SLS) need not normally to be considered in jacket design if not special functional requirements are specified. Such functional requirements could be distance to other facilities (bridge connections, drilling rig) etc.

K.3.9 Vortex shedding

Vortex-induced action and vibrations with regard to wind, current and waves shall be investigated and taken into account where relevant.

Any contribution to fatigue damage from vortex shedding shall be added to damage from drag and inertia actions from the wave, (NORSOK N-003). Relevant hot-spot locations may be considered.

K.3.10 Wave slamming

Horizontal or near-horizontal members that may be subjected to wave slamming shall be designed to resist impact forces caused by such effects.

Any contribution to fatigue damage from wave slamming shall be added to damage from drag and inertia actions from the wave, (NORSOK N-003). Relevant hot-spot locations may be considered.

K.4 GLOBAL RESPONSE ANALYSES

K.4.1 General

The selected method of response analysis is dependent on the design condition; ULS, FLS or ALS. Due attention shall be paid to the dynamic behaviour of the structure including possible non-linearities in action and response.

K.4.2 Dynamic effects

Dynamic effects are present in all jacket structures. However, the significance of the dynamic effects is dependent upon stiffness, soil-structure interaction, mass and mass distribution and the structural configuration. E.g. a complex or wide configurated space frame structure will experince different inertia actions than a narrow or column type structure due to influence on the natural period.

For traditional jacket platforms with natural periods less than 3 seconds, the inertial actions may be neglected and a quasi-static analysis will suffice provided that the provisions in K4.4 are adhered to.

The inertial action is established by a global dynamic analysis. The magnitude of the inertial action can be a direct result of the global dynamic analysis, or can be derived from the quasi-static analysis and the Dynamic Amplification Factor (DAF) from the global dynamic analysis.

Wave and other time varying actions should be given realistic representations of the frequency content of the action. Time history methods using random waves are preferred. Frequency domain methods may be used for the global dynamic analysis both for ULS and FLS, provided the linearization of the drag force can be justified.

The random waves should originate from one or more wave spectra that are plausible conditions for producing the design wave defined shape.

K.4.3 Analysis modelling

Internal forces in members should be determined by three-dimensional structural analysis.

Analytical models utilised in the global analyses of a platform should adequately describe the properties of the actual structure including the foundation.

The non-linear behaviour of axial and lateral soil-foundation system should be modelled explicitly to ensure action-deflection compatibility between the structure and the soil-foundation system.

The structural members of the framework may be modelled using one or more beam elements for each span between nodes. The number of beam elements depends on the element formulation, distribution of actions and potential for local dynamic response. Major eccentricities of action carrying members should be assessed and incorporated in the model. Face-to-face length may be modelled by stiff-end formulation of the chord.

For analyses of the operational design phases (static strength), any corrosion allowance on members should be subtracted in the stiffness and stress calculations, but included in the wave action calculations. For fatigue analysis, half the corrosion allowance thickness should be included both for action, stiffness and stress calculations.

Determination of fatigue lives in pile cluster connections should be determined based on stresses derived from a local finite element analysis.

Appurtenances such as conductors, J-tubes and risers, caisson, ladders and stairs, launch box, boat landing, guides and anodes should be considered for inclusion in the hydrodynamic model of the structure. Depending upon the type and number, appurtenances can significantly increase the global wave forces. In addition, forces on some appurtenances may be important for local member design. Appurtenances not welded to main structure are generally modelled by non-structural members that only contribute as equivalent wave forces.

The increase in hydrodynamic- and gravity actions caused by marine growth shall be accounted for.

K.4.4 Design conditions

K.4.4.1 General

The platform shall be designed to resist gravity actions, wave, current and wind actions, earthquake and accidental actions that may occur during its service life.

K.4.4.2 In-place ULS analysis

Requirements concerning in-place ULS capacity checks are given in Section 6.

Each mode of operation of the platform, such as drilling, production, work-over or combination thereof shall be considered. The stiffness of the deck structure shall be modelled in sufficient detail to ensure compatibility between the deck design and the jacket design.

Studies should be performed to establish maximum base shear of wave and current actions for dimensioning of jacket bracings. Maximum overturning moments shall be established for dimensioning of jacket legs and foundation system. Diagonal approach directions normal to an axis through adjacent legs should also be considered in search for maximum leg and foundation reactions. The studies shall be undertaken by varying wave approach directions, wave periods and water depths. Detail design analysis should be based on minimum eight wave approach directions. A reduced number of approach directions may be considered for early stage design analysis for jackets which are symmetric about the two vertical axes.

Maximum local member actions may occur for wave positions other than that causing the maximum global force. Horizontal members close to still water level shall be checked both for maximum vertical and horizontal wave particle velocities. The effect of varying buoyancy shall also be included.

The choice of an appropriate design wave theory is to be based on particular considerations for the problem in question. Stokes' 5^{th} order wave theory is normally used for jacket design. However, for low water depth jacket locations, shallow water effects shall be assessed.

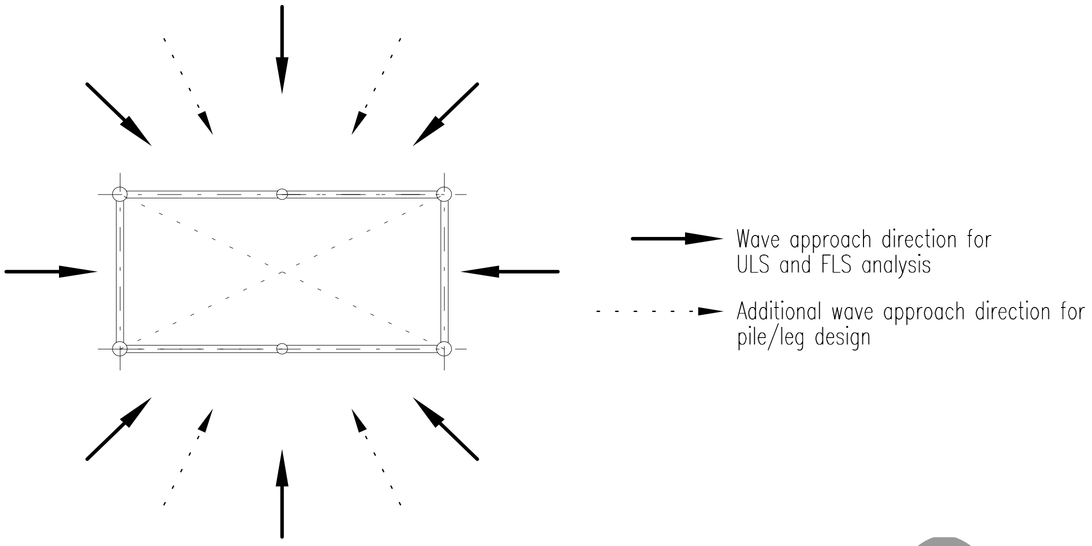

Figure K.4-1 Recommended wave approach directions for ULS and FLS analysis

K.4.4.3 Fatigue analysis

General requirements concerning fatigue analysis of steel structures are given in Section 8. Special considerations and details of fatigue strength analysis are given in Annex C.

Fatigue analyses should include all relevant actions contributing to the fatigue damage both in non-operational and operational design conditions. Local action effects due to wave slamming, vortex shedding are to be included when calculating fatigue damage if relevant.

Whilst jackets in low to moderate water depths are not normally sensitive to dynamic effects, non-linearities associated with wave theory and free-surface effects may be important. A deterministic analysis is normally recommended for such jackets. In the detail design phase, the deterministic analysis should include eight wave approach directions and at least four wave heights from each direction. Wave forces should be calculated for at least ten positions in each wave. In performing a deterministic wave analysis, due attention should be paid to the choice of wave periods if such data are not explicitly specified, or can be determined based upon wave scatter diagrams relevant for the location.

In lack of site specific data, the wave periods shall be determined based on a wave steepness of 1/20 (NORSOK N-003).

For deep water jackets and jackets where the dynamic effects are important, a fatigue analysis in the frequency domain (dynamic stochastic analysis) is recommended.

Frequency domain action calculations are carried out in order to determine hydrodynamic transfer functions for member force intensities. The transfer functions are expressed as complex numbers in order to describe the phase lags between the action variable and the incomming wave

Studies are to be performed to investigate wave actions for a range of periods in order to ensure a sufficient accuracy of the response. For a stochastic fatigue analysis, it is important to select periods such that response amplifications and cancellations are included. Also selection of wave periods in relation to the platform fundamental period of vibration is important. The number of periods included in the analysis should not be less than 30, and be in the range from T = 2 sec. to at least T = 20 sec.

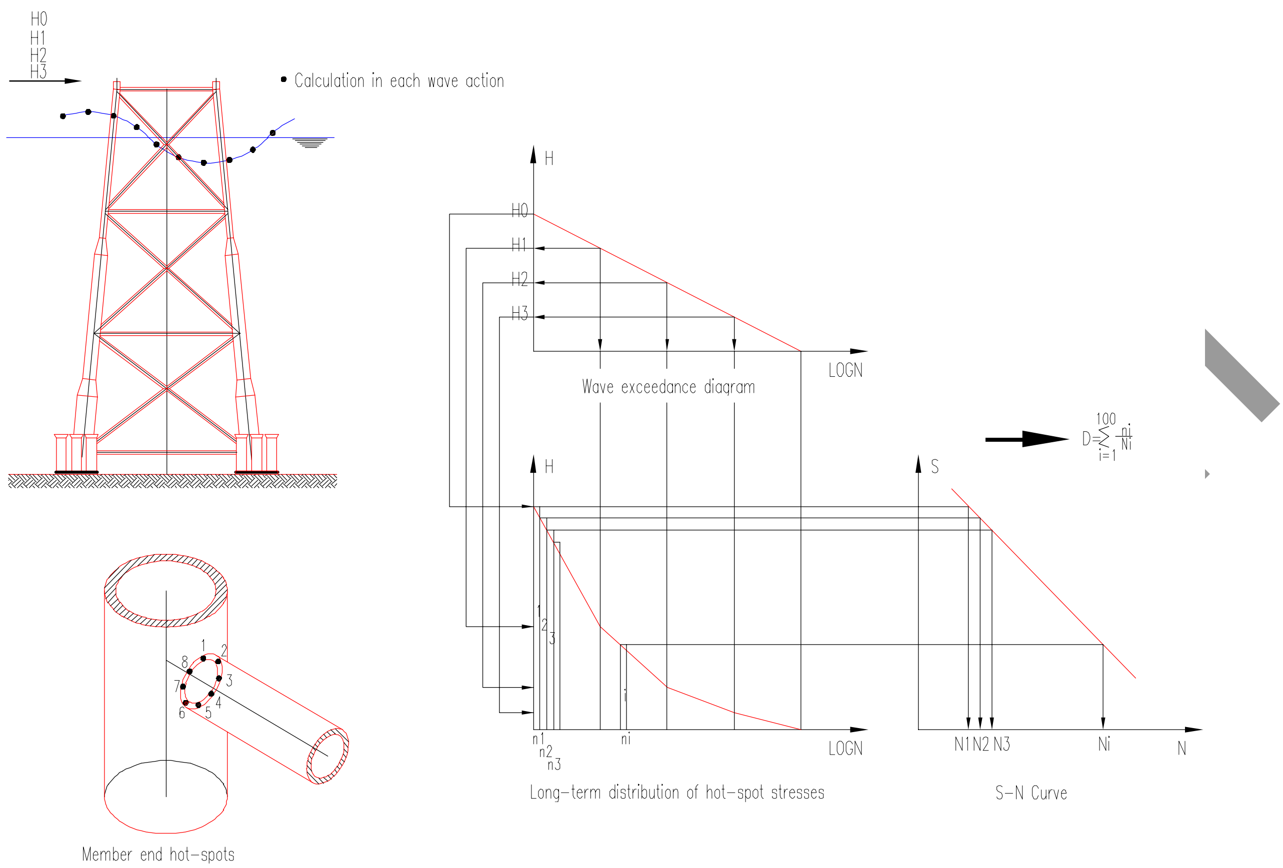

Figure K.4-2 Deterministic fatigue analysis procedure

Short crested waves (angular distribution of wave energy) may be taken into account if such effects are present at the location. When applying short crested waves, an increased number of wave approach directions should be used, normally not less than 12.

The dynamic analysis can be carried out by a modal superposition analysis, direct frequency response analysis or by mode syntehesis techniques.

Traditional jackets are drag dominated structures. For drag dominated structures stochastic analysis based upon linear extrapolation techniques may significantly underestimate action effects. When the effect of drag forces on the expected fatigue damage and the expected extreme responses are to be assessed, Morison-type wave actions are to be based on relevant non-linear models.

Structure-to-ground connections in the analysis shall be selected to adequately represent the response of the foundations. Structure-to-ground connections may normally be simulated by linear stiffness matrices. To linearize the actually non-linear soil response, these matrices should be developed based on a wave height which contributes significantly to the fatigue damage. The matrices shall account for the correlation between the rotational and translation degrees of freedom, which may be important for proper calculation of fatigue lives in the lower part of the structure.

Structural components in the jacket shall be classified according to consequences of failure and accessibility for inspection and repair as outlined in Section 8.

Structures or structural parts located in water depths below 150 m shall be considered as being not accessible for inspection and repair.

Fatigue design factors for typical components in jackets are given in

Table K.4-1 and Figure K.4-3.

Table K.4-1 Fatigue design factors in jackets

Classification of structural components based on damage consequence	**Access for inspection and repair**		
	No access or in the splash zone	**Accessibility**	
		Below splash zone	**Above splash zone**
• Brace/stub to chord welds in main loadtransferring joints in vertical plans, • Chord/cone to leg welds, between leg sections, • Brace to stub and brace to brace welds in main loadtransferring members in vertical plans, • Shear plates and yoke plates incl. stiffening • Piles and bucket foundation plates incl. stiffening	10	3	2
• Brace/stub to chord welds in joints in horisontal plans, • Chord/cone to brace welds and welds between sections in horisontal plans, • Appurtenance supports, • Anodes, doubler plates, • Outfitting steel	3	2	1

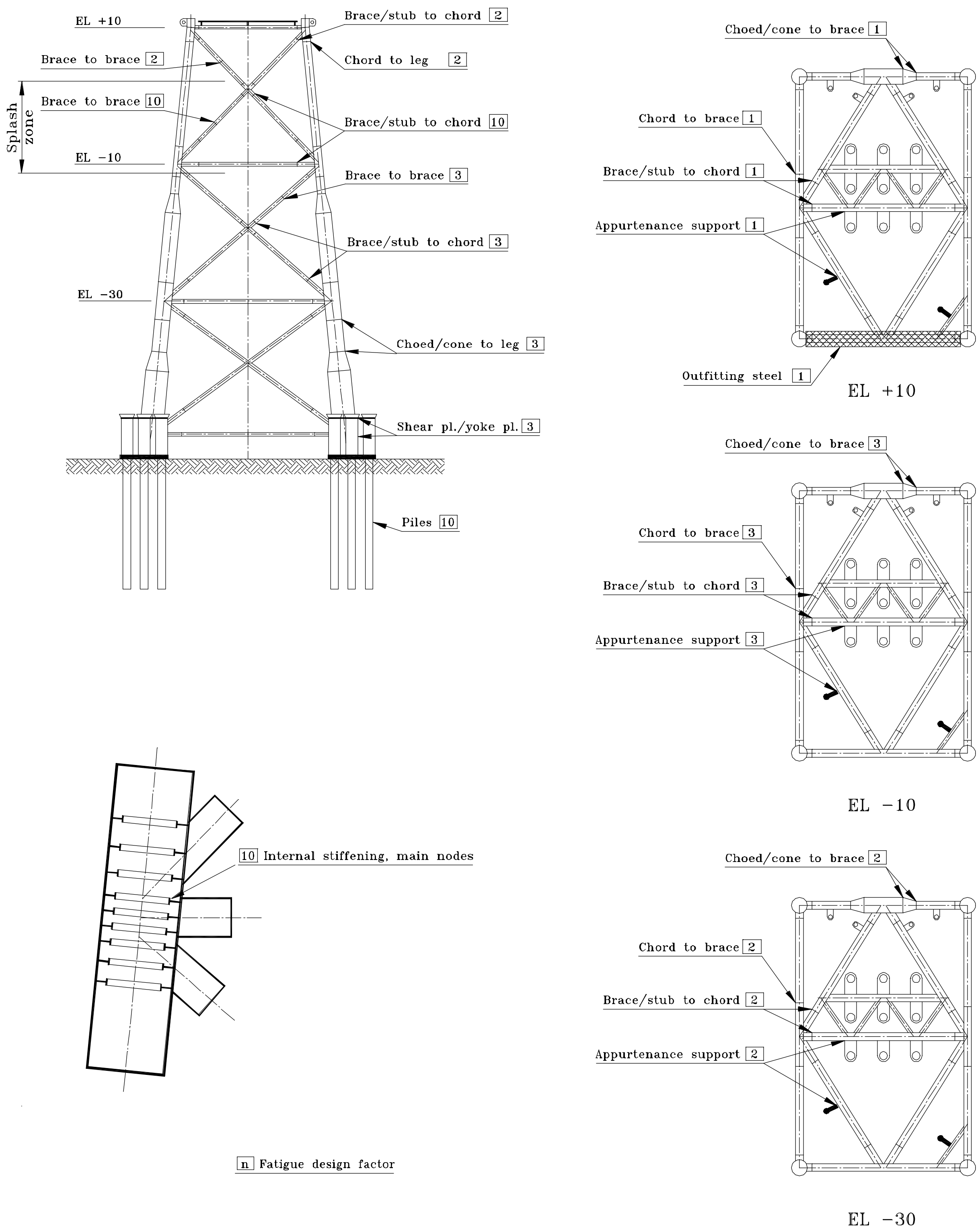

Figure K.4-3 Typical fatigue design factors in jackets

K.4.4.4 Accidental analysis

General requirements concerning the Accidental Limit State (ALS) of steel structures are given in Section 9.

The jacket is to be designed to be damage tolerant, i.e. credible accidental damages or events should not cause loss of global structural integrity.

A credible collision or dropped object against a bracing member or a nodal brace joint will reduce the action carrying ability of the member or the joint. Such members or joints are thus to be assumed non-effective when the global strength of the platform (residual strength) is to be assessed for combinations of design actions in the ALS condition. A credible collision or dropped object against a leg member will normally cause limited damages, and the residual strength of the platform is to be assessed taking due account of the residual strength of the damaged member for combination of design actions in the ALS condition. Details may be found in Annex A.

Pool fire analyses are normally performed to establish requirements to the platform passive and active fire protection system.

The extreme environmental wave design condition is normally limited to a wave action analysis study by comparing the 10^{-4} wave action by the factored wave action from the 10^{-2} event wave. Consequently, the structural response of the extreme environmental condition need not to be checked provided the following equation is fulfilled:

$$Q_{10^{-2}} \cdot \gamma_{f10^{-2}} \cdot \gamma_{M10^{-2}} \geq Q_{10^{-4}} \cdot \gamma_{f10^{-4}} \cdot \gamma_{M10^{-4}} \qquad \textbf{(K.4.1)}$$

where

$Q_{10^{-2}}$ = wave/current base shear and overturning moments action associated with the 10^{-2} event

$Q_{10^{-4}}$ = wave/current base shear and overturning moments action associated with the 10^{-4} event

$\gamma_{f10^{-2}}$ = load factor ULS = 1.3

$\gamma_{f10^{-4}}$ = load factor ALS = 1.0

$\gamma_{M10^{-2}}$ = material factor ULS = 1.15

$\gamma_{M10^{-4}}$ = material factor ALS = 1.0

K.4.4.5 Earthquake analysis

General requirements concerning earthquake analyses are given in NORSOK N-003.

Deck facilities that may respond dynamically to earthquake excitation (flare boom, derrick) are to be included in the structural model where relevant, in order to account for interaction effects due to correlation between closely spaced modes of vibration.

Earthquake analysis may be performed using response spectrum analysis or time history analysis. When the response spectrum method is used, the design spectrum shall be applied in one of the orthogonal directions parallell to a main structural axis. 2/3 of the design spectrum shall be applied in the other ortogonal direction and 2/3 in the vertical direction. The complete quadratic combination (CQC) method should be used for combining modal responses. The square root of the sum of the squares (SRSS) method should be used for combining directional responses.

The model used for earthquake analysis should satisfactorily simulate the behaviour of the actual structure. The number of vibration modes in the analysis should represent at least 90% of the total response energy of all modes. The number of modes to be included in the analysis is normally to be determined by a parametric study. Normally, 15-20 modes are sufficient to ensure an adequate representation of all response quantities. The number of modes may be larger if topside structures such as flare booms and derricks are incorporated in the global model and if their dynamic behaviour is required

Topside structures may normally be included in a simplified manner provided that such modelling will properly simulate the global stiffness and mass distributions.

Structure-to-ground connections are normally simulated by linear stiffness matrices. For jackets located in shallow to moderate water depths, conservative response values are usually achieved by a stiff soil assumption. However, for deep water jackets this may not be conservative.

Damping properties should be adequately assessed and included in the analysis as appropriate. In the absence of more accurate information, a modal damping ratio of 5 % of critical may be used. This value cover material damping and hydrodynamic damping, as well as radiation and hysteretic soil damping. Other damping ratios may be used where substantiated data exists.

K.4.4.6 Installation analysis

Transport and installation design and operation shall comply with the requirements given in NORSOK J-003, Marine Operations.

The recurrence periods to be considered for determination of environmental actions shall comply with the requirements given in NORSOK N-001.

K.5 SPECIAL DESIGNS

K.5.1 Member design

Reference is made to Section 6 for static strength requirements to tubular members.

Buckling factors for X-bracing's may be determined according to procedures which take into account the degree of lateral support furnished to the primary compression member by the cross member (tension member) and member end restraints.

Thickness transitions should be made by flushing inside of tubular to achieve good fatigue capabilities. Reference is made to Annex C for fatigue requirements to butt welds.

K.5.2 Tubular connections

Reference is made to Section 6 and 8 for static strength – and fatigue requirements to tubular member connections and conical transitions.

K.5.3 Grouted connection

Grouted pile connections shall be designed to satisfactorily transfer loads from the pile sleeve to the pile. The critical parameter for this design is the shear load transfer between the pile and the surrounding grout annulus.

The design interface transfer stress, $\tau_{b,Sd}$, is defined by:

$$\tau_{b,Sd} = \frac{N_{Sd}}{\pi \cdot D_p \cdot L_e} \quad \textbf{(K.5.1)}$$

where

N_{Sd} = design axial force [N]
D_p = outside diameter of pile [mm]
L_e = effective grouted connection length [mm]

In calculating the effective grouted connection length, $L_{e,}$ the following non-structural lengths shall be subtracted from the connection's nominal gross grouted length:

1. Where setting of a grout plug is to be used either as the primary means of sealing, or as a contingency sealing method in the event of packer failure, the grouted length allowed for this function shall be considered as non-structural.
2. To allow for potential weak interface zones, grout slump, etc. at each end of a connection, the greater of the following grouted lengths shall be considered as non-structural:
   - two thickness of the grout annulus ($2t_g$)
   - one shear key spacing (s)
3. Any grouted length that may not contribute effectively to the connection capacity, shall be considered as non-structural (e.g. when shear keys are used, the implications of possible over and under driving of piles shall be considered in relation to the number of keys present in the grouted length).

The characteristic interface transfer strength, f_{bk}, of a grouted connection shall be determined from the equation (K.5.2). The design interface transfer stress $\tau_{b,Sd}$, shall satisfy:

$$\tau_{b,Sd} \leq \frac{f_{bk}}{\gamma_M} \qquad \textbf{(K.5.2)}$$

where

$\tau_{b,Sd}$	=	design interface stress [MPa]
f_{bk}	=	characteristic interface transfer strength [MPa]
γ_M	=	material factor for interface transfer strength equal to 2.0

The inherent variability in the test data should be considered when calculating the characteristic strength and if the capacity is based on test results.

When a cement-water grout is used, the characteristic interface transfer strength, f_{bk}, shall be calculated as the minimum of equations (K.5.3) and (K.5.4). These equations define the two actual modes of failure of a connection; sliding at the grout steel interface and shearing through the grout matrix, respectively.

The characteristic interface transfer strength for grout steel interface sliding is given by:

$$f_{bk} = C_p \cdot \left[2 + 140 \cdot \left(\frac{h}{s} \right)^{0.8} \right] \cdot k^{0.6} \cdot f_{ck}^{0.3} \qquad \textbf{(K.5.3)}$$

The characteristic interface transfer strength for grout matrix shear failure is given by:

$$f_{bk} = \left[0.75 - 1.4 \cdot \left(\frac{h}{s} \right) \right] \cdot f_{ck}^{0.5} \qquad \textbf{(K.5.4)}$$

where

s	=	shear key spacing	
h	=	shear key height	
C_p	=	pile diameter scale factor	
	=	$(D_p/1000)^2 - (D_p/500) + 2$	for D_p<1000mm
	=	1.0	for $D_p \geq$1000mm
k	=	radial stiffness factor	
	=	$[(D_p / t_p) + (D_s / t_s)]^{-1} + (1/m)\cdot(D_g / t_g)^{-1}$	
f_{ck}	=	characteristic cube strength [MPa]	
D_p	=	outside diameter of pile	
t_p	=	wall thickness of pile	
D_s	=	outside diameter of pile sleeve	
t_s	=	wall thickness of pile sleeve	
D_g	=	outside diameter of grout annulus	
t_g	=	thickness of grout annulus	
m	=	steel-grout elastic modular ratio (to be taken as 18 in lieu of actual data)	

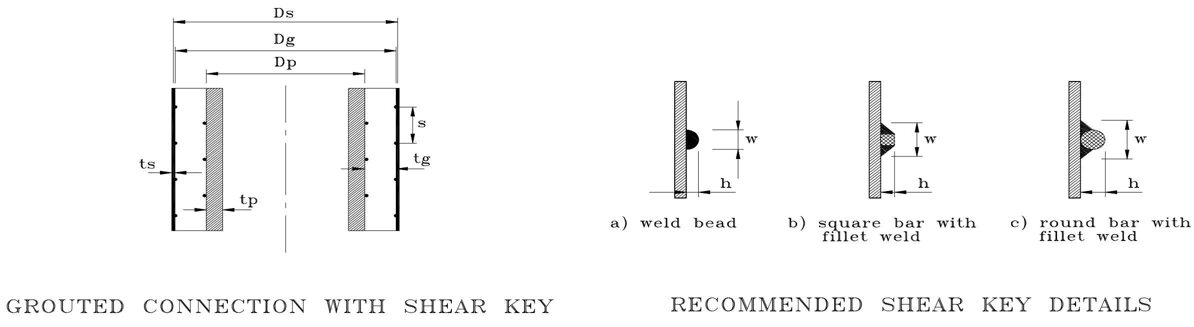

Figure K.5-1 Grouted connection with recommended shear key details

Equation (K.5.3) and (K.5.4) are valid for non-machined and uncoated tubulars, where mill scale has been fully removed. The recommendations are valid for the following range:

80 MPa	$\geq$	f_{ck}	$\geq$	20 MPa
0.10	$\geq$	h/s	$\geq$	0.0
40	$\geq$	D_p/t_p	$\geq$	20
140	$\geq$	D_s/t_s	$\geq$	30
45	$\geq$	D_g/t_g	$\geq$	10
0.02	$\geq$	k		
0.012	$\geq$	h/D_p		
16	$\geq$	D_p/s		
10	$\geq$	L_e/D_p	$\geq$	1
1.5	$\geq$	C_p		

A specified minimum 28 days grout strength of 40 MPa is recommended for design.

A shear key shall be a continuous hoop or a continuous helix. Where hoop shear keys are used, they shall be uniformly spaced, oriented perpendicular to the axis of the tube, and be of the same form, height and spacing on both the inner and outer tubes.

Where helical shear keys are used the following additional limitation shall be applied.

$2.5 \geq D_p/s$

and the characteristic interface transfer strength given in equation (K.5.3) and (K.5.4) shall be reduced by a factor of 0.75

Ungrouted pile on-bottom analysis of the structure shall be performed to determine the relative pile to sleeve movement during the maximum expected sea states in the 24 hour period after grouting. If relative axial movement at the grout steel interface exceeds 0.035%D_p during this period, the characteristic interface transfer strength calculated from Equation (K.5.4) shall be reduced by the following factor:

$1.0 - 0.1 \cdot (h/s) \cdot f_{ck}$

The above reduction factor is valid for relative movements, at either grout steel interface, not exceeding 0.35%D_p. Movements in excess of 0.35%D_p will cause higher levels of strength degradation and should therefore be avoided.

K.5.4 Cast items

Cast joints are defined as joints formed using a casting process. They can be of any geometry and of variable wall thickness.

The design of a cast joint is normally done by use of FE analyses. An acceptable design approach for strength is to limit stresses in the joint due to factored actions to below yielding of the material using appropriate yield criteria. Additionally, the FE analyses should provide SCF's for fatigue life calculations. Often, this design process is carried out in conjunction with the manufacturer of the cast joints.

Fatigue requirements to cast nodes are given in Annex C.

K.6 FOUNDATION DESIGN

K.6.1 General

K.6.1.1 Design principles

The principles of limit state design and the partial coefficient format shall be used. A general description of the method and the definition of the limit state categories are given in the ISO Standard 13819-1 /1/.

Material factors are to be applied to the soil shear strength as follows:

- for effective stress analysis, the tangent to the characteristic friction angle is to be divided by the material factor, γ_M
- for total shear stress analysis, the characteristic undrained shear strength is to be divided by the material factor, γ_M

For soil resistance to axial and lateral loading of piles, material factors are to be applied to the characteristic resistance as described in K.6.2.1. Material factors to be used are specified in subsection K.6.2.1 and K.6.2.2 and K.6.3.2.

K.6.1.2 Soil investigation

Generally, the extent of site investigations and the choice of investigation methods are to take into account the type, size and importance of the structure, uniformity of soil, seabed conditions and the actual type of soil deposits. The area to be covered by site investigations is to account for positioning and installation tolerances. The soil investigation should give basis for a complete foundation design, comprising evaluations for piled foundation of

- On-bottom stability of unpiled structure
- Lateral pile resistance
- Axial pile resistance
- Pile/soil/structure interaction
- Pile driveability predictions

For skirted foundations, the soil investigation should give basis for evaluations of

- Foundation stability
- Soil/structure interaction
- Skirt penetration
- Settlements

Site investigations are to provide relevant information about the soil to a depth below which possible existence of weak formations will not influence the safety or performance of the structure.

Site investigations are normally to comprise of the following type of investigations:

- site geology survey including assessment of shallow gas.
- topography survey of the seabed
- geophysical investigations for correlation with borings and in-situ testing

- soil sampling with subsequent laboratory testing
- in-situ tests, e.g. cone penetration tests

The soil investigation should provide the following type of geotechnical data:

- data for soil classification and description
- shear strength data and deformation properties, as required for the type of analysis to be carried out
- in-situ stress conditions

Further details and requirements related to the soil investigation are given in Norsk Standard NS3481 /2/ and NORSOK G-001.

K.6.2 Piled foundation

K.6.2.1 Axial pile resistance

General

Soil resistance against axial pile loads is to be determined by one, or a combination of, the following methods:

- load testing of piles
- semi-empirical pile resistance formulae based on pile load test data.

The soil resistance in compression is to be taken as the sum of accumulated skin friction on the outer pile surface and resistance against pile tip. In case of open-ended pipe piles, the resistance of an internal soil plug is to be taken into account in the calculation of resistance against pile tip. The equivalent tip resistance is to be taken as the lower value of the plugged (gross) tip resistance or the sum of skin resistance of internal soil plug and the resistance against pile tip area. The soil plug may be replaced or reinforced by a grout plug or equivalent in order to achieve fully plugged tip resistance.

The submerged weight of the pile below mudline should be taken into account.

For piles in tension, no resistance from the soil below pile tip is to be accounted for when the tip is located in cohesionless soils.

Examples of detailed calculation procedures may be found in /3/, /4/,/5/ and /9/. The relevance of alternative methods should be evaluated related to actual design conditions. The chosen method should as far as possible have support in a data base which fits the actual design conditions related to soil conditions, type and dimensions of piles, method of installation, type of loading etc. When such an ideal fit is not available, a careful evaluation of important deviations between data base and design conditions should be performed and conservative modifications to selected methods should be made.

Effect of cyclic loading

Effects of repeated loading are to be included as far as possible. In evaluation of the degradation, the influence of flexibility of the piles and the anticipated load history is to be included.

Cohesive soils

For piles in mainly cohesive soils, the skin friction is to be taken equal to or smaller than the undrained shear strength of undisturbed clay within the actual layer. The degree of reduction depends on the nature and strength of clay, method of installation (e.g. driven or drilled/grouted), time effects, geometry and dimensions of pile, load history and other factors. Especially, the time required to achieve full consolidation after installation of driven piles should be considered. Design conditions that may occur prior to full consolidation should be checked for reduced resistances.

The unit tip resistance of piles in mainly cohesive soils may be taken as 9 times the undrained shear strength of the soil near the pile tip.

Cohesionless soil

For piles in mainly cohesionless soils the skin friction may be related to the effective normal stresses against the pile surface by an effective coefficient of friction between the soil and the pile element. Examples of recommended calculation methods and limiting skin friction values may be found in /3/. The calculation procedures given in /9/ for cohesionless soils should not be used before considerably more extensive documentation is available or without a careful evaluation of limiting values.

The unit tip resistance of piles in mainly cohesionless soils may be calculated by means of conventional bearing capacity theory, taking into account a limiting value which may be governing for long piles /3/.

Material factors

For determination of design soil resistance against axial pile loads in ULS design, a material factor, γ_M=1.3 is to be applied to all characteristic values of soil resistance, e.g. to skin friction and tip resistance.

For individual piles in a group lower material factors may be accepted, provided that the pile group as a whole is designed with the required material factor. A pile group in this context is not to include more piles than those supporting one specific leg.

Group effects should be accounted for when relevant, as further detailed in K.6.2.2.

K.6.2.2 Lateral pile resistance

When lateral soil resistance governs pile penetrations, the design resistance is to be checked within the limit state categories ULS and ALS, using following material factors applied to characteristic resistance:

γ_M =1.3 for ULS condition

γ_M =1.0 for ALS condition

For calculation of pile stresses and lateral pile displacements, the lateral soil reaction is to be modelled using characteristic soil strength parameters, with the soil material factor γ_M=1.0

The non-linear mobilisation of soil resistance is to be accounted for.

The effect of cyclic loading should be accounted for in the lateral load-deflection (p-y) curves.

Recommended procedures for calculation of p-y curves may be found in /3/, /4/.

Scour

The effect of local and global scour on the lateral resistance should be accounted for. The scour potential may be estimated by sediment transport studies, given the soil particle sizes, current velocity etc. However, experience from nearby similar structures, where available, may be the most important guide in defining the scour criteria.

Group effects

When piles are closely spaced in a group, the effect of overlapping stress zones on the total resistance of the soil is to be considered for axial as well as lateral loading of the piles. The increased displacements of the soil volume surrounding the piles due to pile-soil-pile interaction and the effects of these displacements on interaction between structure and pile foundation is to be considered.

In evaluation of pile group effects, due considerations should be given to factors as:

- pile spacing
- pile type
- soil strength and deformation properties
- soil density
- pile installation method.

K.6.2.3 Foundation response analysis

The pile responses should preferably be determined from an integrated pile/soil/structure analysis, accounting for the soils' non-linear response and ensuring load-deflection compatibility between the structure and the pile/soil system. Such an analysis is normally carried out with characteristic soil strength parameters.

K.6.2.4 Installation

General

The structure shall be documented to have adequate foundation stability after touchdown, as well as before and after piling, when subjected to the environmental and accidental actions relevant during this period.

Pile stress check prior to driving

The pile should be checked with respect to yield and buckling (ULS) in the maximum possible inclined position for a design condition of maximum relevant equipment weight (e.g. hammer weight), plus pile self-weight. Current loads and possible dynamic effects should be accounted for.

Pile driving

It should be demonstrated by calculations that the indented driving equipment is capable of driving the pile to target penetration within the pile refusal criteria specified and without damaging the piles.

The soil resistance during driving used in the driveability analysis may account for the gradual reduction in the skin friction caused by driving. The set-up effects leading to increased skin friction after stop of driving should be taken into account for realistic duration of halts that may occur during driving.

Dynamic stresses caused by pile driving are to be assessed based upon recognised criteria or by using wave equation analysis. The sum of the dynamic driving stresses and the static stresses during the driving process is not to exceed the specified minimum yield strength.

Allowance for under- or over-drive to account for uncertainties in pile-driving predictions should be considered.

K.6.2.5 Pile fatigue

The pile fatigue damage should be evaluated and demonstrated to be within the requirements. Considering the pile as a structural component with no inspection access and a substantial consequence of damage, a fatigue design factor of 10 should be applied, as further outlined in Section K.4.4.3

It should be noted that the pile fatigue damage from the pile driving operation normally contributes substantially to the total fatigue damage, which is the sum of the partial damages from the pile driving and the environmental action during the design service life.

The fatigue design factor should be applied both to the pile driving damage and to the environmental damage.

K.6.2.6 Foundation simulation for jacket fatigue analysis

For a jacket fatigue analysis, the pile/soil system may be simulated by pile stiffness matrices generated from independent pile/soil analysis. To account for the soil non-linearity, the pile analysis for derivation of pile fatigue stiffness matrices should be performed at a representative fatigue load level.

K.6.3 Skirted foundation

K.6.3.1 General

Skirted foundations are an alternative to pile foundations for jacket structures. The base area can be of any shape and the skirts can be corrugated or plane. The skirts are penetrating the soil and contribute to increase the foundation capacity with respect to horizontal as well as vertical forces. Also, uplift may be resisted by the skirted foundation to a variable degree depending on the soil and loading conditions.

The skirted foundation may be used in both cohesive and cohesionless soils. The design for the main dimensions of the foundation should consider both the soil and the structural behaviour during operation as well as during installation. Possibility of inhomogenity of the soil over the area and local seabed variations should be accounted for.

K.6.3.2 Foundation capacity

In calculation of the foundation capacity one should consider the simultaneously acting vertical and horizontal forces and overturning moment acting on the skirted foundation. The acting forces should be determined from a soil structure interaction analysis. The stiffnesses used to simulate the foundation should account for the non-linearity of the soil and be compatible with the load level of the soil reactions obtained from the analysis. This may require iterative analyses unless an adequate fully integrated analysis tool is used.

The determination of the foundation capacity should account for the fact that most soils, even sandy soils, behave undrained for the short duration of a wave action, and that the cyclic loading effects may have significant effect on the undrained strength. The determination of foundation capacity should preferably be based on cyclic strengths derived by combining relevant storm load histories for the loads on the skirted foundations with the load dependent cyclic strengths determined from undrained cyclic soil tests. Depending on the reliability of determining cyclic strengths for the soil in question the use of large-scale field tests should be evaluated.

Special concerns for clay

In clay the soil can normally be considered fully undrained for all the accumulated effects of a design storm, and cyclic strengths can be determined based on recognised principles, as e.g. given in /4/, and described in more detail in /7/. It is important that the cyclic strength is determined for a realistic relation between average and cyclic shear stresses, representative for the storm load history on the skirted foundation.

Special concerns for sand

Even in sand, the transient type of loading from one wave will result in an undrained or partly undrained response in the soil. If the sand is dense dilatancy effects would lead to high shear resistance for a single transient loading, resulting in high capacity for any type of loading, including uplift. However, when the soil is exposed to repeated loading in an undrained state, accumulated pore pressures will result. Such pore pressures can ultimately lead to a 'true failure' for less dense sands, or a state of 'initial liquefaction' or 'cyclic mobility' for dense sands. In the latter case, large cyclic soil strains and corresponding displacements of the foundation may occur before the soil dilates and higher resistance is obtained. It is recommended that the definition of failure is based on an evaluation of tolerable cyclic or total strains in the soil.

Even though the sandy soils will be undrained or partly undrained from the transient loads from each wave, there may be a considerable dissipation of excess pore pressures through the duration of a storm. The combined effects of pore pressure build up from cyclic loading and dissipation may be accounted for. It should be aimed at defining an equivalent load history with a limited duration (as compared to the duration of the storm) for which the soil can be assumed to be basically undrained. An example of a calculation procedure is described in /10/. This load history may be used in defining cyclic shear strengths of the soil for calculation of cyclic capacities, or scaled directly as loads onto large-scale tests if such tests are performed.

The general failure mechanisms for skirted foundations in sand are described in more detail in part 2 of /6/, together with description of the design approach used for a jacket on dense sand.

Material factors

The minimum values of the material factors should be those specified by NPD, i.e. γ_M=1.25 for ULS design and γ_M=1.0 for ALS design.

There are elements in the design process for foundation stability design of a skirted foundation that may contribute to greater uncertainties than for other type of foundations. This especially relates to defining the cyclic strengths in sandy soils. Depending on the conservatism used in the design it may be necessary to increase the material factors generally recommended by NPD. It is recommended that the sensitivity of the main assumptions made in the design process is investigated before such a decision is made.

K.6.3.3 Skirt penetration

For installation of the skirted foundations the following design aspects should be considered

- A sufficient penetrating force to overcome the soil resistance against penetration should be provided by a combination of weight and suction.
- The suction should be applied in a controlled manner to prevent excessive soil heave inside the skirted foundation
- The skirted foundation should be designed to prevent buckling due to the applied suction
- Channelling

Penetration resistance

In calculation of penetration resistance, there are several uncertainties related to

- spatial variations in soil conditions; may be different at different corners
- general uncertainties in determination of basic soil parameters
- uncertainties in calculation of penetration resistance from basic soil parameters

The total range of expected resistances should be documented. The upper bound resistance will be governing for design of ballast and/or suction system and for structural design of the foundation. Depending of the conservatism applied in choice of parameters and method of calculation to cover up for the uncertainties listed above, safety factors may have to be applied to the calculated resistances.

In clayey soils it can be considered that the resistance is unaffected by the applied suction, and the resistance can be calculated as for skirts penetrated by weight only. Skirt penetration resistances may be determined through correlations with cone penetration resistances as given in /4/, relevant basically for overconsolidated clays. Alternatively the skin friction may be related to the remoulded shear strength of the clay, and tip resistance may be related to the intact shear strength through conventional bearing capacity factors.

In sand the penetration resistance will be effectively reduced by applied suction. The reduction is caused by

- general decrease of effective stresses inside the skirted foundation reducing the skin friction
- large gradients in effective stresses combined with large inwards gradients in the flow below the skirt tip level, which triggers an inward bearing failure beneath the skirt tip and thus significantly reduces the otherwise in sand dominating tip resistance.

The general governing mechanisms are described in /7/ and /8/. The reduction effect of suction on penetration resistance is dependent both on the geometry of the skirted foundations and on the soil stratification. When penetrating sand layers that are overlain by a clay layer, significantly higher suction will be required than in homogeneous sand. Reliable universal calculation methods for calculation of penetration resistance in sand when suction is applied are not generally available. The estimation penetration resistance should thus to a high extent be based on empirical evidence from relevant conditions, taking account for the effects described above.

Control of soil heave

The control of soil heave should be based on a detailed monitoring of penetration, tilt, heave of soil plug, suction pressures, etc. As long as there is a net weight from the jacket acting on the skirted foundation, the amount of soil heave inside the skirted foundation is normally controlled by controlling the discharge of water through the pumps. The restraining effects on a jacket with 4 or more legs should be taken into account. By applying suction to one skirted foundation only, the weight from the jacket on that skirted foundation will decrease. If the weight from the jacket is reduced to zero, significant soil heave may occur. This should be avoided. A penetration procedure should normally be aimed at, where the jacket as far as possible is penetrated in a level position.

K.6.3.4 Skirted foundation structural design

Reference is made to Section 6 and 8 for general static strength and fatigue requirements. The skirted foundation should be designed for all relevant design conditions including

- transportation forces
- forces when lowering the skirted foundations into the water and through the water line
- forces due to the motions of the jacket when the skirts start penetrating the seabed
- the suction applied during skirt penetration
- operational forces

The buckling resistance during penetration of the skirts should be checked for various stages of the penetration using calculation models that account for the boundary conditions both at the skirt top as well as within the soil.

The design for operational loading conditions should be based on soil reaction distributions that are derived from realistic soil/structure interaction analyses. Alternatively more simple approaches with obviously conservative soil reaction distributions can be used.

K.6.4 On-bottom stability

The foundation system for the jacket temporary on-bottom condition prior to installation of the permanent foundation system shall be documented to have the required foundation stability for the governing environmental conditions as specified, and for all relevant limit states.

K.7 DOCUMENTATION REQUIREMENTS FOR THE 'DESIGN BASIS' & 'DESIGN BRIEF'

K.7.1 General

Adequate planning shall be undertaken in the initial stages of the design process in order to obtain a workable and economic structural solution to perform the desired function. As an integral part of this planing, documentation shall be produced identifying design criteria and describing procedures to be adopted in the structural design of the platform.

Applicable codes, standards and regulations shall be identified at the commencement of the design.

When the design has been finalised, a summary document containing all relevant data from the design and fabricationand installation phase (DFI) shall be produced.

Design documentation (see below) shall, as far as practicable, be concise, non-voluminous, and, should include all relevant information for all relevant phases of the lifetime of the unit.

General requirements to documentation relevant for structural design are given in NORSOK, N-001, Section 5.

K.7.2 Design basis

A Design Basis Document shall be created in the initial stages of the design process to document the basis criteria to be applied in the structural design of the unit.

A summary of those items normally to be included in the Design Basis document is included below.

- Platform location and main fuctionalities,
- general description, main dimensions and water depth,
- applicable Regulations, Codes and Standards (including revisions and dates),
- service life of platform,
- topside interface requirements (including leg spacings, topside weight and c.o.g, appurtenance dimensions and routing),
- materials,
- coating and corrosion protection system,
- environmental and soil data,
- installation method,
- foundation system,
- system of units

K.7.3 Design brief

A Design Brief (DB) shall be created in the initial stages of the design process. The purpose of the Design Brief shall be to document the criteria and procedures to be adopted in the design of the jacket structure.

The Design Brief shall, as far as practicable, be concise, non-voluminous, and, should include all relevant limiting design criteria for the relevant design phase.

Design Briefs shall as a minimum cover the following main design phases and designs:

- Load-out
- Transportation
- Installation
- Inplace
- Fatigue
- Earthquake and accidental
- Foundation, on-bottom stability and pile driving

A summary of those items normally to be included in the Design Brief is included below.

Environmental design criteria

Limiting environmental design criteria (including all relevant parameters) for relevant conditions, including :

- wind, wave, current, earthquake, snow and ice description for relevant annual probability events.

Temporary phases

Design criteria for all relevant temporary phase conditions including, as relevant :

- limiting permanent, variable, environmental and deformation action criteria,
- essential design parameters and analytical procedures associated with temporary phases e.g. for load-out, transportation, lifting, installation, on-bottom stability and pile installation and driving,
- platform abandonment.

Operational design criteria

Design criteria for relevant operational phase conditions including :

- limiting permanent, variable, environmental and deformation action criteria,
- deck load description (maximum and mimimum) including variation in gravity,
- designing accidental event criteria (e.g. collision criteria, earthquake, explosion and fire),
- soil parameters.

Global structural analyses

A general description of models to be utilised in the global analysis including :

- description of global analysis model(s) including modelling for wave and current loading,
- foundation system,
- description of analytical procedures (including methodology, factors, dynamic representation and relevant parameters).

Structural evaluation

A general description of the structural evaluation process including :

- description of local analytical models,
- description of procedures to be utilised for combining global and local responses,
- criteria for member and joint code checking,
- description of fatigue analytical procedures and criteria (including design fatigue factors, SN-curves, basis for stress concentration factors (SCF's), etc.).

Miscellaneous

A general description of other essential design information, including :

- description of corrosion allowances, where applicable,
- in-service inspection criteria (as relevant for evaluating fatigue allowable cumulative damage ratios).

K.8 REFERENCES

/1/ International Standard ISO 13819-1: 'Petroleum and natural gas industries – Offshore structures, Part 1: General requirements'

/2/ Norges Standardiseringsforbund, NS3481: 'Soil investigation and geotechnical design for marine structures', 1989

/3/ American Petroleum Institute, API RP2A – LRFD: 'Planning, designing and constructing fixed offshore platforms – Load and Resistance Factor Design', 1993

/4/ DNV Class Note 30.4: 'Foundations', 1992

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DESIGN OF STEEL STRUCTURES

ANNEX L
SPECIAL DESIGN PROVISIONS FOR SHIP SHAPED UNITS

CONTENTS

L.1 INTRODUCTION

L.1.1 General

This Annex is intended to provide requirements and guidance to the structural design of ship shaped units constructed in steel, according to the provisions of relevant NORSOK standards. The Annex is intended as the rest of the standard to fulfill NPD regulations relating to loadbearing structures in the petroleum activities /4/. In addition it is intended that the unit shall fulfill technical requirements to standard ship hull design. Therefore references to the technical requirements in maritime standards such as NMD, IMO and DNV classification rules for guidance and requirements to design is also given.

The Annex is intended as being generally applicable to all types of conventional ship shaped structures, including the following variants:

- Floating Production Units (FPU)
- Floating Storage and Offloading Units (FSU)
- Floating Production, Storage and Offloading Units (FPSO)
- Floating Production, Drilling, Storage and Offloading Units (FPDSO).

In this Annex the above will collectively be referred to as “units”.

This Annex is intended to cover several variations with respect to conceptual solutions as listed below:

- Units intended for production which may be equipped with topside structures, supporting the production facilities.
- Units intended for storage with storage tanks together with facilities for offloading to shuttle tankers.
- Units intended for production will normally have a turret installed, while units intended for storage only, may have a buoy installed, replacing the turret.
- Units that can either be permanently moored on site or have a disconnectable mooring system. In the latter case, the unit may disconnect from its moorings and leave the site under its own power or assisted by tugs, to avoid certain exceptional events such as extreme storms, icebergs, hurricanes etc. Normally the mooring lines are connected to the turret or buoy, positioned forward of the midship area.

Requirements concerning mooring and riser systems other than the interfaces with the structure of the units are not explicitly considered in this Annex.

The intention of this Annex is to cover units weather vaning by rotating around a turret or a buoy. Fixed spread mooring arrangements (and similar) should be specially considered.

L.1.2 Definitions

Ship Shaped Floating Production Units:

A floating unit can be relocated, but is generally located on the same location for a prolonged period of time. Inspections and maintenance can be carried out on location. The Ship Shaped Floating Productions unit may consist of a ship shaped hull, with turret, and production equipment on the deck.

Ship Shape Floating Storage and Offloading Units:

A floating unit can be relocated, but is generally located on the same location for a prolonged period of time. Inspections and maintenance can be carried out on location. The Ship Shaped Floating Storage and Offloading units normally consist of a ship shaped hull equipped for crude oil storage. The crude oil may be transported to shore by shuttle tankers via an offloading arrangement.

Ship Shaped Floating Production, Storage and Offloading Units:

A floating unit can be relocated, but is generally located on the same location for a prolonged period of time. Inspections and maintenance are carried out on location. The Ship Shaped Floating Production, Storage and Offloading unit normally consists of a ship shaped hull, with turret, and production equipment on the deck. The unit is equipped for crude oil storage. The crude may be transported to shore by shuttle tankers via an offloading arrangement.

Ship Shaped Floating Production, Drilling, Storage and Offloading Units:

A floating unit can be relocated, but is generally located on the same location for a prolonged period of time. Inspections and maintenance are carried out on location. The Ship Shaped Floating Production, Drilling, Storage and Offloading unit normally consists of a ship shaped hull, with turret, and production and drilling equipment on the deck. The unit is equipped for crude oil storage. The crude may be transported to shore by shuttle tankers via an offloading arrangement.

Turret:

A device providing a connection point between the unit and the combined riser- and mooring-systems, allowing the unit to rotate around the turret (weather vane) without twisting the risers and mooring lines.

RCS:

Recognised Classification Society

L.2 BASIS OF DESIGN

L.2.1 Safety format

Generally, the design of units shall be based upon the partial factor design methodology as described in NORSOK N-001, based on direct calculated actions as described in L.5.

To ensure that experiences with traditional ships are taken into account in the design, units shall as a minimum, fulfil the relevant technical requirements given in the DNV Rules for Ship, Pt.3 Ch.1, /3/. Due consideration should be made with respect to the relevance of actions specified, taking into account possible increase in action level due to the differences in operation of the unit compared to normal trading ships.

L.2.2 Design criteria

The design of units should as a minimum, comply with the technical requirements given in the DNV Rules for Ships, Pt.3 Ch.1, /3/.

In cases where the unit is registered in a specific country (as though it was a merchant ship), the relevant requirements of the flag state authority shall additionally be complied with.

A unit may be designed to function in a number of operational modes, e.g. transit, operation and survival. Limiting design criteria for going from one mode to another shall be clearly established and documented when relevant. In these operational modes, the design criteria normally relate to consideration of the following items:

- Intact strength covering transit, temporary conditions (e.g. installation), extreme operating conditions and survival conditions (relevant only if normal operation of the unit is terminated when limiting weather conditions for the extreme operating condition are exceeded).
- Structural strength in damaged condition
- Compartmentation and stability

If it is intended to dry-dock the unit, the bottom structure shall be suitable strengthened to withstand such actions.

L.2.2.1 Non-operational phases

The structure shall be designed to resist relevant actions associated with conditions that may occur during all relevant stages of the life cycle of the unit. Such stages may include:

- fabrication,
- site moves,
- sea transportation,
- installation, and,
- decommissioning.

Structural design covering marine operation construction sequences shall be undertaken in accordance with NORSOK, N-001.

Marine operations should be undertaken in accordance with the requirements stated in NORSOK, J-003.

<u>Sea Transportation</u>

A detailed transportation assessment shall be undertaken which includes determination of the limiting environmental criteria, evaluation of intact and damage stability characteristics, motion response of the global system and the resulting, induced actions. The occurrence of slamming actions on the structure and the effects of fatigue during transport phases shall be evaluated when relevant.

Satisfactory compartmentation and stability during all floating operations shall be ensured.

Technical requirements to structural strength and stability as stated in relevant parts of the DNV Rules for Ships, /3/, e.g. Pt.3 Ch.1 and Ch. 4 should be complied with for transit conditions where such rules are found to be applicable.

Installation

Installation procedures of foundations (e.g. drag embedded anchors, piles, suction anchor or dead weights etc.) shall consider relevant static and dynamic actions, including consideration of the maximum environmental conditions expected for the operations.

Installation operations shall consider compartmentation and stability, and, dynamic actions on the mooring system(s). The actions induced by the marine environment involved in the operations and the forces exerted by the positioning equipment, such as fairleads and padeyes, shall be considered for local strength checks.

Decommissioning

Abandonment of the unit shall be planned for at the design stage. However, decommissioning phases for ship shaped units are normally not considered to provide design action conditions for the unit and may normally be disregarded at the design phase.

L.2.2.2 Operational phases

The unit shall be designed to resist all relevant actions associated with the operational phases of the unit. Such actions include ;

- Permanent actions (see L.4.2)
- Variable actions (see L.4.3)
- Deformation actions (see L.4.4)
- Environmental actions (see L.4.5)
- Accidental action (see L.4.6)
- Fatigue actions (see L.4.7)

Each mode of operation, such as production, work over, or, combinations thereof, shall be considered.

The unit may be designed to function in a number of operational modes, e.g. operational and survival. Limiting design criteria for going from one mode of operation to another mode of operation shall be clearly established and documented.

L.3 STRUCTURAL CLASSIFICATION AND MATERIAL SELECTION

L.3.1 Structural classification

Selection of steel quality, and requirements for inspection of welds, shall be based on a systematic classification of welded joints according to the structural significance and the complexity of the joints/connections as documented in Chapter 5.

In addition to in-service operational phases, consideration shall be given to structural members and details utilised for temporary conditions, e.g. fabrication, lifting arrangements, towing arrangements, etc.

Basic Considerations

Structural connections in ship shaped units designed in accordance with this Annex will normally not fall within the categorisation criteria relevant for Design Class DC1 or DC2. In particular, relevant failure of a single weld, or element, should not lead to a situation where the accidental limit state damaged condition is not satisfied. Structural connections will therefore be categorised within classification groups DC3, DC4 or DC5.

Consideration shall be given to address areas where through thickness tensile properties may be required.

Special consideration shall be given to ensure the appropriate inspection category for welds with high utilisation in fatigue if the coverage with standard local area allocation is insufficient.

Examples of typical design classes applicable to ship shaped units are stated below. These examples provide minimum requirements and are not intended to restrict the designer in applying more stringent requirements should such requirements be desirable.

Typical locations - Design class : DC3

- Structure in way of moonpool
- Structure in way of turret, fairlead and winches supporting structure
- Highly stressed areas in way of main supporting structures of heavy substructures and equipment e.g. anchor line fairleads, cranes substructure, gantry, topside support stools, flare boom, helicopter deck, davits, hawser brackets for shuttle tanker, towing brackets etc.

DC3 areas may be limited to local, highly stresses areas if the stress gradient at such connections is large.

Typical locations - Design class : DC4

In general all main loadbearing structural elements not described as DC3, such as:

- Longitudinal structure
- Transverse bulkheads
- Transverse frames an stringers
- Turret structure
- Accommodation

Typical locations - Design class : DC5

Non main loadbearing structural elements, such as:

- Non-watertight bulkheads,

- Tween decks
- Internal outfitting structure in general
- other non-loadbearing components

L.3.2 Material selection

Material specifications shall be established for all structural materials utilised in a ship shaped unit. Such materials shall be suitable for their intended purpose and have adequate properties in all relevant design conditions. Material selection shall be undertaken in accordance with the principles given in NORSOK M-001.

When considering criteria appropriate to material grade selection, adequate consideration shall be given to all relevant phases in the life cycle of the unit. In this connection there may be conditions and criteria, other than those from the in-service, operational phase, that provide the design requirements in respect to the selection of material. (Such criteria may, for example, be design temperature and/or stress levels during marine operations.)

Selection of steel quality for structural components shall normally be based on the most stringent Design Class of the joints involving the component.

Requirements to through-thickness strength shall be assessed.

The evaluation of structural resistance shall include relevant account of variations in material properties for the selected material grade. (e.g. Variation in yield stress as a function of thickness of the base material).

L.3.3 Inspection categories

Welding, and the extent of non-destructive examination during fabrication, shall be in accordance with the requirements stipulated for the appropriate 'inspection category' as defined in NORSOK, M-101.

The lower the extent of NDT selected, the more important is the representativeness of the NDT selected and performed. The designer should therefore exercise good engineering judgement in indicating priorities of locations for NDT, where variation of utility along welds is significant.

L.3.4 Guidance to minimum requirements

The following figures illustrate minimum requirements to selection of Design Class and Inspection Category for typical structural configurations of ship shaped units. The indicated Design Class and Inspection Categories should be regarded as guidance of how to apply the recommendations in chapter 5.

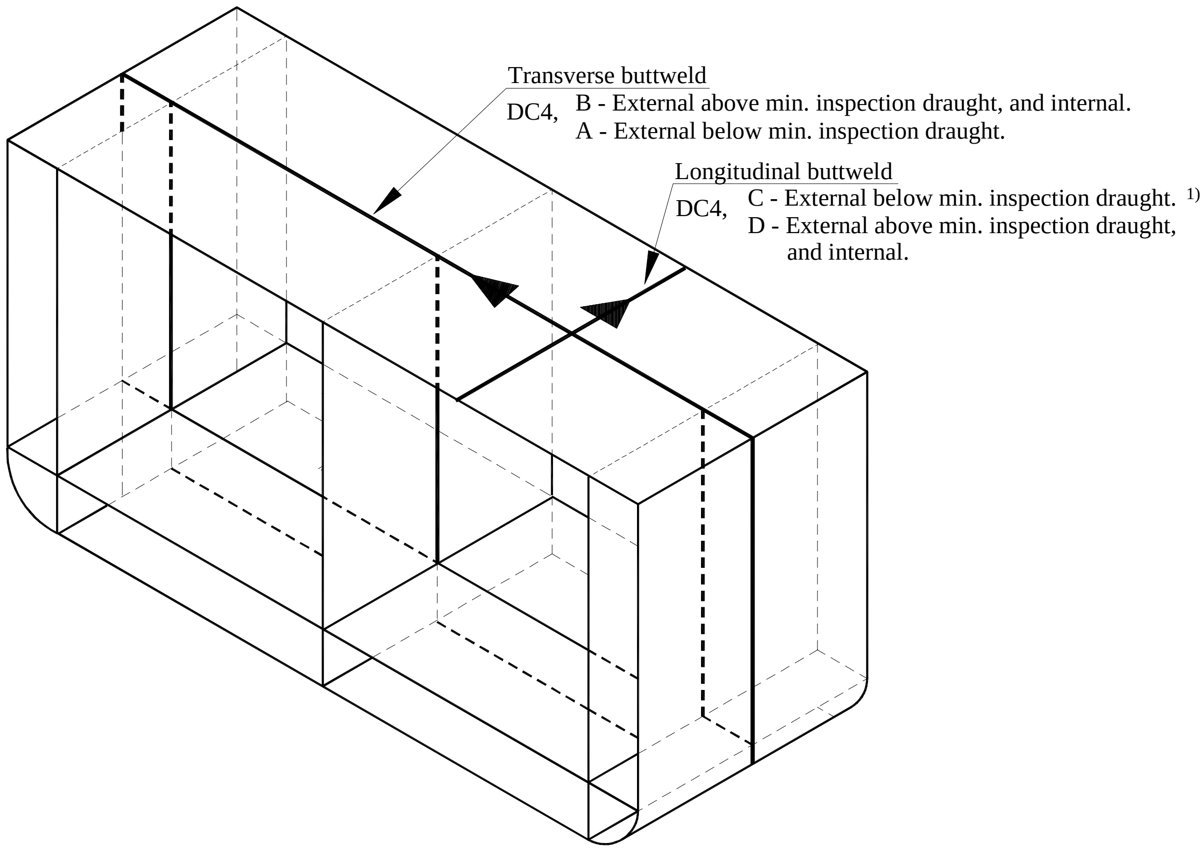

Figure L.3-1 Example of typical design classes and inspection categories for butt welds in the midship area

Key:

1. [DCn, ..] Design Class, n
2. [….., N] Inspection Category, N

Notes:

1. Inspection draught see L.7.3
2. The selection of the inspection categories is made on the following assumptions with reference to Table 5.3 and 5.4:

Transverse buttwelds, external above min. insp. draught and internal	High fatigue, (ref. Table 5.4), dominating dynamic principal stress transverse to the weld.
Transverse buttwelds, below min. insp. draught	High fatigue, (ref. Table 5.4), dominating dynamic principal stress transverse to the weld. No access for in-service inspection and repair.
Longitudinal buttwelds, external above min insp. draught and internal	Low fatigue, low tensile stress transverse to the weld.
Longitudinal buttwelds, below min. insp. draught	Low fatigue, low tensile stress, transverse to the weld. No access for in-service inspection and repair.

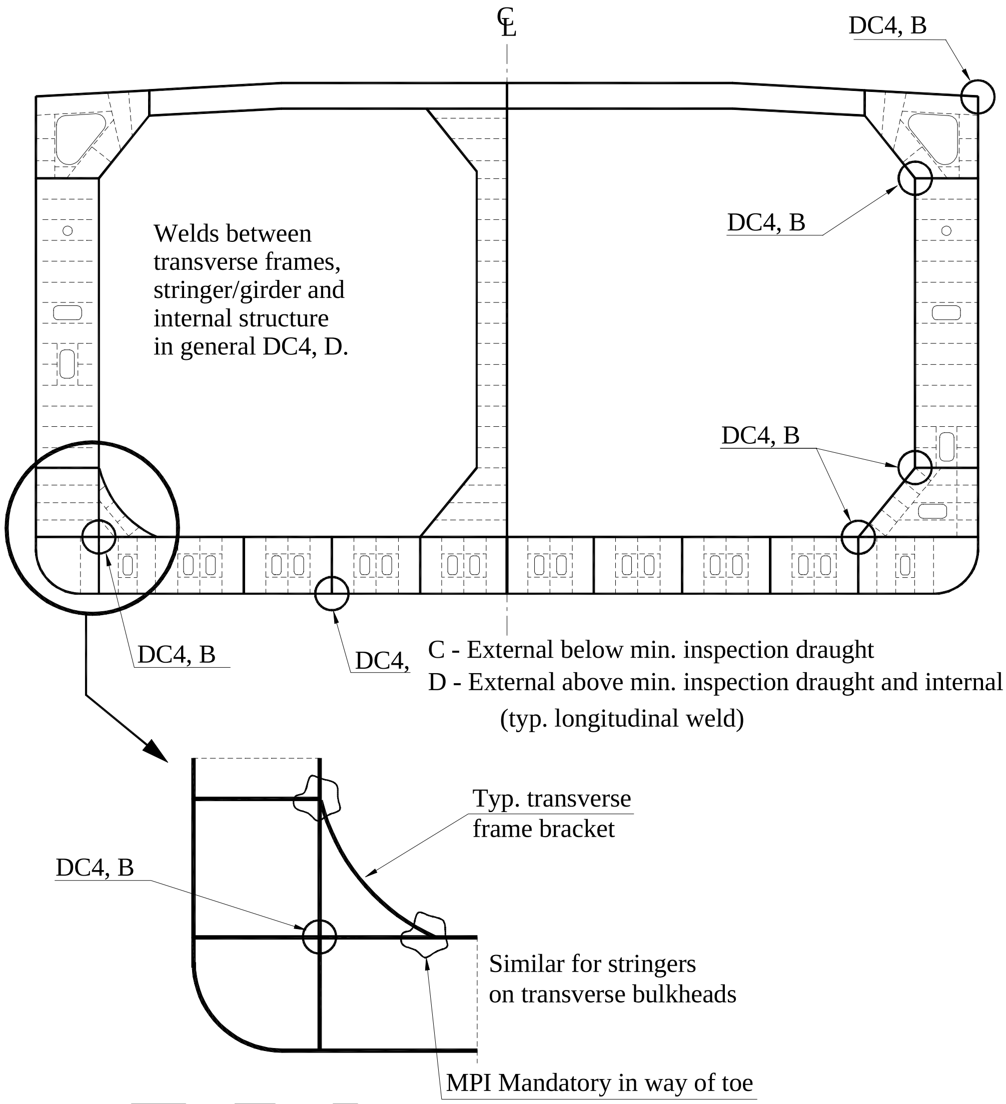

Figure L.3-2 Example of typical design classes and inspection categories for the transverse frames in the midship area

Key:

1. [DCn, ..] Design Class, n
2. [….., N] Inspection Category, N

Notes:

1. Inspection draft see L.7.3
2. The Design Classes and Inspection Categories should be applied along the whole length of the welds
3. For welds of longitudinal girders and stringers MPI should be mandatory in way of transverse frame intersection.

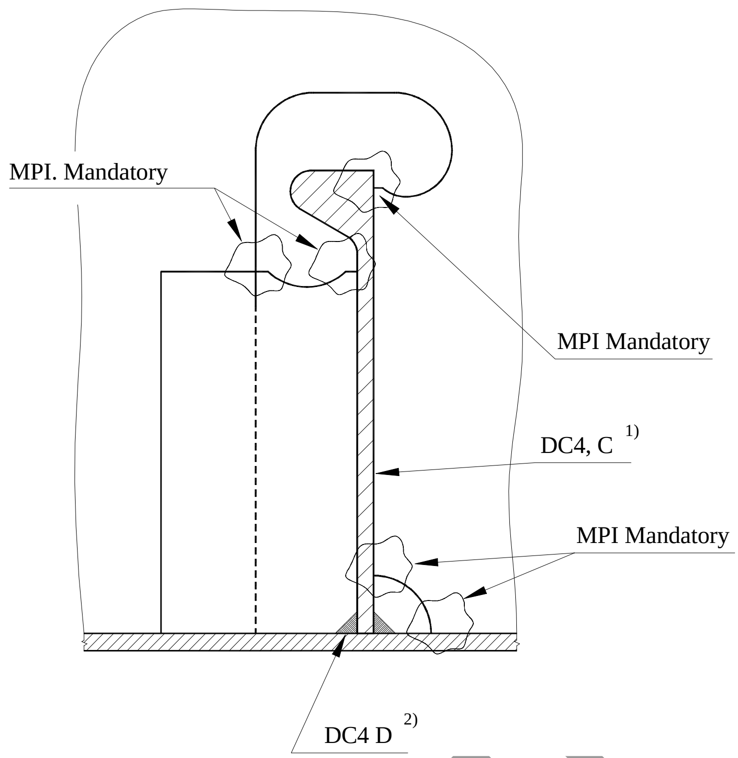

Figure L.3-3 Example of typical design classes and inspection categories longitudinal stiffeners connection to transverse frames / bulkheads

Key:

1. [DCn, ..] Design Class, n
2. [….., N] Inspection Category, N

Notes:

1. The selected inspection class is relevant for a high fatigue loading with principal stresses parallel to the weld and assuming in-service access (see table 5.4)
2. The selected inspection class is relevant for a low fatigue joint with low tensile stresses perpendicular to the weld

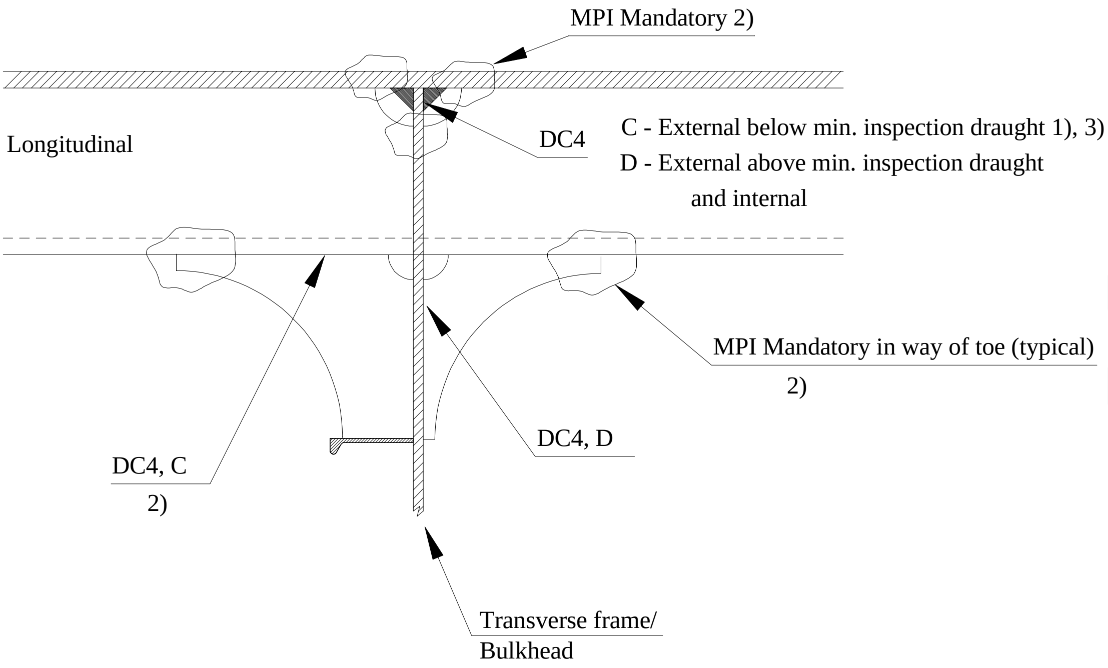

Figure L.3-4 Example of design classes and inspection categories longitudinal stiffeners connection to transverse frames / bulkheads

Key:

1. [DCn, ..] Design Class, n
2. […., N] Inspection Category, N

Notes:

1. Inspection draft see L.7.3
2. The proposed inspection categories are relevant for high fatigue loading, for longitudinal stiffener in the sideshell.
3. The proposed inspection categories are relevant for low fatigue loading with low tensile stresses transverse to the weld, and no access for repair below min. inspection draught.

L.4 DESIGN ACTIONS

L.4.1 General

Characteristic actions shall be used as reference actions in the partial coefficient method.

Design actions relevant for ship shaped units are in general defined in NORSOK N-003. Guidance concerning action categories relevant for ship shaped unit designs are given in the following.

Design action criteria dictated by operational requirements shall be fully considered. Examples of such requirements may be:

- drilling (if applicable), production, workover and combination thereof,
- consumable re-supply procedures and frequency,
- maintenance procedures and frequency,
- possible action changes in extreme conditions.

L.4.2 Permanent actions (G)

Permanent actions are actions that will not vary in magnitude, position or direction during the period considered. Permanent actions relevant for ship shaped unit designs are:

- 'lightmass' of the unit including mass of permanently installed modules and equipment, such as accommodation, helideck, cranes, drilling (if applicable) and production equipment,
- hydrostatic pressures resulting from buoyancy,
- pretension in respect to mooring, drilling (if applicable) and production systems (e.g. mooring lines, risers etc.).

L.4.3 Variable actions (Q)

Variable actions are actions that may vary in magnitude, position and direction during the period under consideration.

Except where analytical procedures or design specifications otherwise require, the value of the variable actions utilised in structural design should normally be taken as either the lower or upper design value, whichever gives the more unfavourable effect. Variable actions on deck areas for local, primary and global design are stated in NORSOK, N-003. For global design the factor to be applied to primary design actions (as stated NORSOK, N-001) is not, however, intended to limit the variable load carrying capacity of the unit in respect to non-relevant, variable load combinations. Global load cases should be established based upon 'worst case', representative variable load combinations, where the limiting global criteria is established by compliance with the requirements to intact and damage hydrostatic and hydrodynamic stability.

Global design action conditions, and associated limiting criteria, shall be documented in the stability manual for the unit, see L.9.

Variations in operational mass distributions (including variations in tank filling conditions) shall be adequately accounted for in the structural design.

Local design actions for all decks shall be documented on a ‘load-plan’. This plan shall clearly show the design uniform and concentrated actions for all deck areas for each relevant mode of operation.

Dynamic actions resulting from flow through air pipes during filling operations shall be adequately considered in the design of tank structures.

L.4.4 Deformation actions (D)

Deformation actions are actions due to deformations applied to the structure.

Deformation actions resulting from construction procedures and sequences (e.g. as a result of welding sequences, forced alignments of structure etc.) shall be accounted for when relevant.

Other relevant deformation action effects may include those resulting from temperature gradients, as for example:

- when hot-oil is stored in the cargo tanks,
- radiation from flaring operations.

Deflection imparted to topside structure due to G, Q or EA actions acting on the ship hull, shall not be considered as deformation actions.

L.4.5 Environmental actions (EA)

Environmental actions are actions caused by environmental phenomena. The characteristic value of an environmental action is the maximum or minimum value (whichever is the most unfavourable) corresponding to an action effect with a prescribed probability of exceedance.

Environmental conditions shall be described using relevant data for the relevant period and areas in which the ship shaped unit is to be fabricated, transported, installed and operated.

The long-term variation of environmental phenomena such as wind, waves and current shall be described by recognised statistical distributions relevant to the environmental parameter considered, (NORSOK N-003). Information on the joint probability of the various environmental actions may be taken into account if such information is available and can be adequately documented.

Consideration shall be given to responses resulting from the following listed environmental induced action effects that may be relevant for the design of a ship shaped unit:

- wave actions acting on the hull, (including variable sea pressure)
- wind actions,
- current actions,
- temperature actions,
- snow and ice actions.
- dynamic stresses for all limit states (e.g. with respect to fatigue see L.4.7)
- rigid body motion (e.g. in respect to air gap and maximum angles of inclination),
- sloshing,
- slamming (e.g. on bow and bottom in fore and aft ship)
- green water on the deck
- vortex induced vibrations (e.g. resulting from wind actions on structural elements in a flare tower),
- wear resulting from environmental actions at mooring and riser system interfaces with hull structures,

For ship shaped units with traditional, catenary mooring systems, earthquake actions may normally be ignored.

Further considerations in respect to environmental actions are given in NORSOK, N-003.

Analytical considerations with regard to evaluation of global response resulting from environmental action components are stated in L.5.

L.4.5.1 Mooring actions

A unit may be kept on station by various methods, depending on specific site criteria and operational goals. These methods may include several different types of station-keeping systems such as internal and submerged turret systems, external turret, CALM buoy, fixed spread mooring and dynamic positioning. Each mooring system configuration will impose actions into the hull structure which are characteristic to that system. These actions shall be addressed in the structural design of the unit, and combined with other relevant action components. Only mooring arrangements, which allows weather vaning of the unit, are covered by this annex.

L.4.5.2 Sloshing actions in tanks

Merchant ships are normally operated so that cargo tanks are typically either nearly full or nearly empty. For a floating production unit, in general, no restrictions should be imposed on partly filling of cargo tanks, and some cargo tanks may be partially filled most of the time. For partially filled tanks, the phenomenon of resonant liquid motion (sloshing) inside the tank should be considered.

Sloshing is defined as a dynamic magnification of internal pressures acting on the boundaries of cargo tanks and on internal structure within the tank, to a level greater than that obtained from static considerations alone. Sloshing occurs if the natural periods of the fluid and the vessel motions are close to each other. Major factors governing the occurrence of sloshing are:

- tank dimensions
- tank filling levels
- structural arrangements inside the tank (wash bulkheads, web frames etc.)
- transverse and longitudinal metacentric height (GM)
- vessel draught
- natural periods of ship and cargo in roll (transverse) and pitch (longitudinal) modes.

The pressure fields created by sloshing of the cargo/ballast may be considered, according to the requirements given in DNV Rules for Classification of Ships, /3/, Part 3, Chapter 1, Section 4, Design Loads, C300 Liquid in tanks. The DNV Rules differentiate between ordinary sloshing loads (non-impact) and sloshing impact actions.

For units to be operated in severe environments, direct calculation of sloshing pressures using site-specific criteria, should be considered.

L.4.5.3 Green water effect

The green water effect is the overtopping by water in severe wave conditions. Significant amounts of green water will influence the deck structural design, accommodation superstructure, equipment design and layout, and may induce vibrations in the hull. Normally the forward part of the deck and areas aft of midship will be most severe exposed to green water. Short wave periods are normally most critical.

Appropriate measures should be considered to avoid or minimize green water effects on the ship structure and topside equipment. These measures include bow shape design, bow flare, breakwaters and other protective structure. Adequate drainage arrangements shall be provided.

Exposed structural members or topside equipment on the weather deck shall be designed to withstand the loads induced by green water. For (horizontal) weather deck structural members this load will mainly be the hydrostatic load from the green water. For horizontal or vertical topside members this load will mainly be the dynamic action from the green water rushing over the deck.

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In lack of more exact information, for example from model testing, relevant technical requirements of the DNV Rules for Classification of Ships, /3/, may be applied:

- for weather deck structural members, Part 3, Chapter 1, Section 4, Design Loads, C200, Sea pressures
- for horizontal or vertical topside members, Part 3, Chapter 1, Section 10, C100, External pressures

In lieu of more exact actions, unprotected front bulkheads of deck houses located in the forward part of the vessel should be designed for loads 50% higher than the Rule actions. Horizontal or vertical topside members located in the midship or aft area of the ship are to be designed for loads 50% higher than the Rule loads.

Weather deck areas located in the forward part of the vessel shall be designed for loads 50% higher than the Rule loads at the ship side and 75% higher than the Rule loads at the ship center line.

L.4.5.4 Slamming in the fore and aft ship

Slamming is impact from waves which give rise to an impulsive pressure from the water, resulting in a dynamic transient action (whipping) of the hull. The most important locations to be considered with respect to slamming are the forward bottom structure, the bow flare and accommodation structure in the fore ship. Slamming forces should be taken into account in the turret design when relevant. Other locations on the hull e.g. exposed parts of the aft ship, which may be subject to wave impacts should be considered in each separate case.

The frequency of occurrence and severity of slamming are significantly influenced by the following:

- vessel draught,
- hull geometrical form,
- site environment,
- heading,
- forward speed (including current)
- position of superstructure.

The effects of slamming on the structure shall be considered in design particularly with regard to enhancement of global hull girder bending moments and shear forces induced by slamming, local strength aspects and limitations to ballast draft conditions.

In lack of more exact information, for example from model testing, relevant requirements of the DNV Rules for Classification of Ships, /3/, may be applied:

- for bottom slamming in the ship fore body, Part 3, Chapter 1, Section 6, Bottom Structures, H200, Strengthening against slamming
- for bow flare slamming in the ship fore body, Part 3, Chapter 1, Section 7, Side Structures, E300, Strengthening against bow impact.

For bottom slamming, in order to account for:

- 100 year return period
- Main wave direction head waves
- Long crested waves

the expression for coefficient c_2 in Part 3, Chapter 1, Section 6, Bottom Structures, H200, Strengthening against slamming, should be changed to:

$$c_2 = 1675\left(1 - \frac{12T_{BF}}{L}\right)$$

The resulting slamming impact pressures shall be multiplied by a factor 0.375 if they are to be applied as a mean pressure over a larger area (applicable for global structural evaluation).

For bow flare slamming, in order to account for:

- 100 year return period
- Main wave direction head waves
- Long crested waves

the expression for bow flare slamming pressures in the ship fore body, Part 3, Chapter 1, Section 7, Side Structures, E303, are to be multiplied by a factor:

$$fac = \left(1 + \frac{25}{L}\right)^2$$

The applicable speed V is not to be taken less then 8 knots.

The resulting bow impact pressures shall be multiplied by a factor 0.375 if they are to be applied as a mean pressure over a larger area (applicable for global structural evaluation).

L.4.6 Accidental actions (A)

Accidental actions are actions related to abnormal operation or technical failure.

In the design phase particular attention should be given to layout and arrangements of facilities and equipment in order to minimise the adverse effects of accidental events.

Risk analyses shall be undertaken to identify and assess accidental events that may occur and the consequences of such events. Structural design criteria for the ALS condition are identified from the risk analyses. Generally, the following ALS events are those required to be considered in respect to the structural design of a ship shaped unit :

- dropped objects (e.g. from crane handling),
- fire,
- explosion,
- collision,
- unintended flooding and counterflooding situations, and,
- abnormal wave events.

The counterflooding situations shall consider the possible loading conditions after an unintended flooding has occurred, to compensate for the heeling resulting from such flooding.
Further considerations in respect to accidental actions are given in Chapters 9 and Annex A.

L.4.7 Fatigue actions (F)

Repetitive actions, which may lead to possible significant fatigue damage, shall be evaluated. In respect to a ship shaped unit, the following listed sources of cyclic response shall, where relevant, be considered:

- waves (including those actions caused by dynamic pressure, sloshing / slamming and variable buoyancy),

- wind (especially when vortex induced vibration may occur),
- mechanical vibration (e.g. caused by operation of machinery),
- crane actions,
- full / empty variation of filling level in cargo tanks (low cycle).

The effects of both local and global dynamic response shall be properly accounted for when determining response distributions of repetitive action effects.

L.4.8 Combination of actions

Action coefficients and action combinations for the design limit states are, in general, given in NORSOK, N-003.

Structural strength shall be evaluated considering all relevant, realistic action conditions and combinations. Scantlings shall be determined on the basis of criteria that combine, in a rational manner, the effects of relevant global and local responses for each individual structural element.

A sufficient number of load conditions shall be evaluated to ensure that the most probable largest (or smallest) response, for the appropriate return period, has been established. (For example, maximum global, characteristic responses for a ship shaped unit may occur in environmental conditions that are not associated with the characteristic, largest, wave height. In such cases, wave period and associated wave steepness parameters are more likely to be governing factors in the determination of maximum/minimum responses.)

L.5 STRUCTURAL RESPONSE

L.5.1 General

The structure shall as a minimum comply with the technical requirements of the DNV Rules for Ship, /3/, Pt.3 Ch.1, see L.2.1, as applicable to a normally trading ship. In addition, the dynamic actions to which the hull may be subjected will vary from those associated with seagoing trading ships, and will require assessment.

This assessment may be based on results of model testing and/or by suitable direct calculation methods of the actual wave actions on the hull at the service location, taking into account relevant service related factors. When the design is based on direct calculations, the software used should have been verified by model testing.

Sufficient action conditions for all anticipated pre-service and in-service conditions shall be determined and analysed to evaluate the most unfavourable design cases for the hull girder global and local strength analysis.

For units intended for multi field developments the most onerous still water and site specific environmental actions shall be considered for the design. The action conditions shall be stated in the Operating Manual, see L.10.6.

The units shall comply with requirements taking into account site specific service. Factors which may influence the hull actions and shall be accounted for, include the following:

- Site specific environmental conditions
- Effect of mooring system
- Long term service at a fixed location
- Seas approaching predominantly from a narrow sector ahead
- Zero ship speed
- Range of operating action conditions
- Tank inspection requirements
- Different return period requirements compared with normal trading tankers

Site specific environment; for ship shaped units of normal form, strength standards set by RCS are based on criteria relating to a world-wide trading pattern.

Effect of mooring system; static and dynamic mooring and riser actions can be substantial. Their effects on the hull girder longitudinal bending moments and shear forces shall be accounted for in the design calculations.

Long term service at a fixed location; seagoing ships would generally spend a proportion of their time in sheltered water conditions. Permanently moored units would normally remain on station all the time and disconnectable units would only move off station in certain conditions and would generally remain in the local area. In addition the expectation of the field life may be in excess of 20 years. This shall be accounted for e.g. in the design of corrosion protection systems.

Seas approaching from a narrow sector ahead; for seagoing ships in severe weather, steps would generally be taken to minimise the effects of such conditions such as altering course or alternative routing. Moored production/storage units cannot take avoiding action and will weathervane due to the combined effects of waves, wind and current, resulting in a greater proportion of waves approaching from bow sector directions.

Zero ship speed; the effect of forward ship speed would enhance static predictions of hull bending moments and shear forces and other factors such as slamming. Due to zero forward speed this enhancement would not apply to moored production/storage units.

Range of operating action conditions; ocean going tankers traditionally have a fairly limited range of operational action conditions and would typically be “fully” loaded or in ballast. Due to their oil storage capability moored units however potentially have an almost unlimited range of action conditions. These effectively cover a full spectrum of cases from ballast through intermediate conditions to fully loaded returning to ballast via offloading.

Tank inspection requirements; ocean-going ships would generally be taken to dry dock for periodic survey and repair. Moored units can be inspected on station. Thus a full range of conditions covering each tank empty in turn, and tank sections empty should be addressed as appropriate for necessary access for inspection and maintenance. These conditions should be combined with appropriate site-specific environmental actions.

Different return period requirements compared with normal trading tankers; normal ship class rule requirements are based on providing adequate safety margins based on a 20 years return period. This standard specifies that the design shall be based on data having a return period of 100 years for the site specific action conditions.

Taking into consideration the conditions listed above, calculations shall be carried out to address all design action conditions including fully loaded, intermediate operating conditions, minimum loaded condition, inspection conditions with each tank empty in turn, etc. Still water bending moment and shear force distributions shall be calculated for each case and stated in the Operating Manual, see L.10.6.

L.5.2 Analysis models

L.5.2.1 General

Analysis models relevant for in-place ULS and FLS are described in this chapter.

The finite element modelling of the hull structure should be carried out according with principles given in the following. Four typical modelling levels are described below. Other equivalent modelling procedures may also be applied.

L.5.2.2 Global structural model (Model level 1)

In this model a relatively coarse mesh extending over the entire hull should be used. The overall stiffness of the primary members of the hull shall be reflected in the model. The mesh should be fine enough to satisfactorily take account of the following:

- Vertical hull girder bending including shear lag effects
- Vertical shear distribution between ship side and bulkheads
- Stress distribution around large penetrations such as the moonpool
- Horizontal hull girder bending including shear lag effects
- Termination of longitudinal members

The models should be used for analysing global wave response and still water response where found relevant.

L.5.2.3 Cargo tank model (Model level 2)

The cargo tank analysis shall be used to analyse local response of the primary hull structural members in the cargo area, due to relevant internal and external action combinations. The extent of the structural model shall be decided considering structural arrangements and action conditions.

The mesh fineness shall be decided based on the method of action application in the model. The model should normally include plating, stiffeners, girders, stringers, web-frames and major brackets. For units with topside the stiffness should be considered in the tank modelling. In some cases the topside structure should be included in the model.

The cargo tank model may be included in the global structural model, see L.5.2.2.

The finite element model should normally cover the considered tank, and one half tank outside each end of the considered tank. The effect of non-structural elements in the topside may introduce additional stiffness and should be considered.

Conditions of symmetry should as far as possible, be applied at each end of the finite element model. The model should be supported vertically by distributed springs at the intersections of the transverse bulkheads with ship sides and longitudinal bulkheads. The spring constants shall be calculated for each longitudinal bulkhead and ship sides based on actual bending and shear stiffness and for a model length of three tanks. If horizontal unbalanced loads are applied to the model in transverse direction e.g. heeling conditions, horizontal spring supports should be applied at the intersections of all continuous horizontal structural members.

The following basic actions are normally to be considered for the ULS condition:

- Cargo and ballast loading, static and dynamic
- External sea pressure, static and dynamic
- Topside loading, vertical static and dynamic and also horizontal acceleration actions

These ULS responses are to be combined with global stresses, see L.5.2.2.

The following basic actions are normally to be analysed in this model for the FLS condition:

- Dynamic cargo and ballast actions
- External sea pressure range
- Topside loading, vertical dynamic and also horizontal acceleration actions

These FLS responses are to be combined with global stresses, see L.5.2.2.

L.5.2.4 Turret analysis (Model level 2)

A finite element model should be made of the turret structure. The model should reflect the geometry and stiffness of the structure. A model with relatively coarse mesh may be used.

In cases where the stress distribution in the moonpool is dependent of the relative stiffness between unit hull and turret a model comprising both the unit hull and the turret should be made.

Modelling of interface areas may require finer mesh and use of gap elements etc. to describe the actual stress flow.

Following actions should normally be considered:

- Static and dynamic actions from the turret itself
- Mooring actions
- Riser actions

The analysis should cover both the ULS and the FLS condition.

L.5.2.5 Local structural analysis (Model level 3)

The purpose of the local structural analysis is to analyse laterally loaded local stiffeners (including brackets) subject to large relative deformations between girders.

The following typical areas should be given particular attention:

- Longitudinal stiffeners between transverse bulkhead and the first frame at each side of the bulkhead
- Vertical stiffeners at transverse bulkheads with horizontal stringers in way of inner bottom and deck connection
- Horizontal stiffeners at transverse bulkheads with vertical stringers in way of inner side and longitudinal bulkhead connection
- Corrugated bulkhead connections

Local structural models may be included in the cargo tank analysis or run separately with prescribed boundary deformations or boundary forces from the tank model.

L.5.2.6 Stress concentration models (Model level 4)

For fatigue assessment, fine element mesh models shall be made for critical stress concentration details, for details not sufficiently covered by stress concentration factors given in recognised standards, see Annex C.

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

The following typical areas may have to be considered:

- Hopper knuckles in way of web frames
- Topside support stools
- Details in way of the moonpool
- Other large penetrations in longitudinal loadbearing elements
- Longitudinal bulkhead terminations
- Stiffener terminations
- Other transition areas when large change in stiffness occur

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

Element size for stress concentration analyses is to be in the order of the plate thickness. Normally, shell elements may be used for the analysis. The correlation between different actions such as global bending, external and internal fluid pressure and acceleration of the topside should be considered in the fatigue assessment. For further details reference is made to DNV Classification Note 30.7, /11/.

The correlation between different actions such as global bending, external and internal fluid pressure and acceleration of the topside should be considered in the fatigue assessment.

L.5.3 Calculation of wave induced actions

According to Recognised Classification Society Rules, normal trading ships are designed according to the following environmental criteria:

- North Atlantic wave condition, 20 year return period
- Short crested sea
- Same probability for all wave directions relative to the ship

Specific requirements for weather vaning units covered by this Annex are given below.

L.5.3.1 General

Global linear wave induced actions such as bending moments and shear forces should be calculated by using either strip theory or three dimensional sink source (diffraction) formulation. Strip theory is a slender body theory and is not recommend when the length over beam ratio is less than 3.

Generally, the most importance global responses are midship vertical bending moments and vertical shear forces in the fore and aft body of the unit and the associated vertical bending moments. These responses should be calculated for head sea conditions. Horizontal and torsional moments may be of interest depending on the structural design, alone and in combination with other action components.

The calculation of wave induced actions may follow the following steps:

- Calculation of the relevant transfer functions (RAOs)
- Calculation of the 100 year linear response
- Evaluation of non-linear effects

When a 3-D diffraction program is used, the hydrodynamic model shall consist of sufficient number of facets. In general, the facets should be sufficiently modelled to describe the unit in a propitiate way. The size of the facets shall be determined with due consideration to the shortest wave length included in the hydrodynamic analysis. Smaller facets should be used in way of the water surface.

The mass model shall be made sufficiently detailed to give centre of gravity, roll radius of inertia and mass distribution as correct as practically possible.

L.5.3.2 Transfer functions

The following wave induced linear responses should be calculated:

- Motions in six degrees of freedom.
- Vertical bending moment at a sufficient number of positions along the hull. The positions have to include the areas where the maximum vertical bending moment and shear force occur and at the turret position. The vertical wave induced bending moment shall be calculated with respect to the section's neutral axis.
- Horizontal and torsional moment if applicable.

The linear transfer functions shall be calculated by either strip or 3D sink –source (diffraction) theory. The responses shall be calculated for a sufficient number of wave periods in the range from 4 – 35 seconds. For short crested sea at least seven directions in a sector ± 90° relative to head sea, with maximum 30° interval, should be considered.

L.5.3.3 Viscous damping

If roll resonance occurs within the range of wave periods likely to be encountered, the effect of non-linear viscous roll damping should be taken into account. The damping coefficients that are derived within linear potential theory reflect the energy loss in the system due to generation of surface waves from the ship motions. However, in the case of roll motion some special treatment is necessary.

Viscous effects, such as the production of eddies around the hull, will mainly act as a damping mechanism, especially at large roll angles, and these effects should be included. Furthermore, the effects from roll damping devices, like bilge keels, should be evaluated. The roll damping shall be evaluated for the return period in question. The sea state in question may be considered when the damping is calculated.

L.5.3.4 Extreme wave induced responses

Extreme wave induced responses shall be long term responses. The standard method for calculation of the 100-year wave induced long term responses (e.g. vertical moment and shear force) shall use a scatter diagram in combination with a wave spectrum (e.g. Jonswap) and RAOs.Short term predictions of the responses are to be calculated for all significant wave heights (H_s) and peak (T_p) or zero up-crossing (Tz) periods combinations within the scatter diagram. A Weibull distribution is fitted to the resulting range of the responses against probability of occurrence. This Weibull distribution is used to determine the response with a probability of occurrence corresponding to once every 100 years (100 year return period).

The short crested nature of the sea may be taken into account as a wave spreading function as given in NORSOK N-003.

The method described above is considered most accurate for estimating the 100-year value for the response in question. A short-term analysis based on the predicted 100-year wave height with corresponding T_z, will give comparable values if the following criteria are satisfied:

- The scatter diagram should be well developed with wave steepness approaching a constant value for the most extreme sea states
- The maximum wave induced response shall occur in a short term sea sate which is retraceable from the scatter diagram with a 100 year wave height steepness
- The Weibull fit to predict the response shall be good fit with low residual sum (deviations from the regressed line)

It should be noted that the method of calculating the 100-year wave induced response for a short-term sea state based upon the predicted 100 year wave height with corresponding single value T_z or T_p does not recognise that there are a range of possible T_z.(T_p). Therefore the range of possible T_z (T_p) within the 100-year return period should be investigated. The range of sea states with a 100 year return period should be found by developing a 100 year contour line from the scatter diagram. The wave bending moment should be calculated for several sea states on this contour line in order to find the maximum value.

L.5.3.5 Non-linear effects

Linear calculations as described above do not differentiate between sagging and hogging responses. The non-linearities shall be taken into account for the vertical bending moments and shear forces. The non-linear effects come from integration of the wave pressure over the instantaneous position of the hull relative to the waves, with the inclusion of bottom slamming, bow flare forces and deck wetness. These effects generally result in a reduction in hogging response and an increase in the sagging response (midship moment and shear fore in the fore body) compared with the linear response.

L.6 ULTIMATE LIMIT STATES (ULS)

L.6.1 Global strength

L.6.1.1 General

Ultimate strength capacity check shall be performed for all structure contributing directly to the longitudinal and transverse strength of the ship. Structure to be checked is all plates and continuous stiffeners including the following structure:

- Main deck, bottom and inner bottom
- Ship side, inner ship side and longitudinal bulkheads
- Stringers and longitudinal girders
- Foundations of turret and topside structure
- Transverse bulkheads
- Transverse web frames

Global actions on the hull girder shall be calculated by direct wave analyses, see L.5.3. Longitudinal wave bending moments, shear forces and dynamic external sea pressures should be calculated by a wave load analysis. Design accelerations should also be based on a wave load analysis. For unconventional designs torsional effects may also be of importance. The design moments, forces, pressures and accelerations shall be calculated for a representative number of sections.

Stillwater shear forces and bending moments should be calculated by direct analyses, taking in account all relevant loading conditions.

Internal static and dynamic pressures can be calculated by simplified formulas, see L.6.1.4.

Local stresses should be calculated by FE-analyses of relevant parts of the ship, see L.5.2. All relevant load conditions should be taken in account.

The hull girder strength shall be evaluated for relevant combinations of still water bending moment and shear force, and wave induced bending moment and shear force. The wave-induced bending moments and shear forces shall be calculated by means of an analysis carried out utilising the appropriate statistical site specific environmental data. Relevant non-linear action effects shall be accounted for, see L.5.3.

The following design format shall be applied:

$$\gamma_S M_S + \gamma_w M_w < M_G/\gamma_M \qquad \textbf{(L.6.1)}$$

$$\gamma_S Q_S + \gamma_w Q_w < Q_G/\gamma_M \qquad \textbf{(L.6.2)}$$

where:

M_G = Characteristic bending moment resistance of the hull girder calculated as an elastic beam

M_S = Characteristic design still water bending moment based on actual cargo and ballast conditions

M_W = Characteristic wave bending moment. Annual probability of exceedance of 10^{-2}.

Q_G = Characteristic shear resistance of the hull girder calculated as an elastic beam

Q_S = Characteristic design still water shear force based on actual cargo and ballast conditions

Q_W = Characteristic wave shear force. Annual probability of exceedance of 10^{-2}.

γ_M = Material factor

γ_S = Permanent and variable action factor

γ_W = Environmental action factor

The action factors shall be in accordance with NPD, see also Table L.6-1:

Table L.6-1

Action combinations	γ_S	γ_W
a	1.3	0.7
b	1.0	1.3

For combination of actions an action coefficient of 1.0 shall be applied for permanent actions where this gives the most unfavourable response.

The action coefficient for environmental actions may be reduced to 1.15 in action combination b, when the maximum stillwater bending moment represents between 20 and 50 % of the total bending moment. This reduction is applicable for the entire hull, both for shear forces and bending moments.

The action coefficient for permanent actions in action combination a, may be reduced to 1.2, if actions and responses are determined with great accuracy e.g. limited by the air-pipe height, and external static pressure to well defined draught.

Gross scantlings may be utilised in the calculation of hull structural resistance, provided a corrosion protection system in accordance with NORSOK N-001, is maintained.

The buckling resistance of the different plate panels shall be considered according to Section 8.

The global and local longitudinal stress components shall be combined in an appropriate manner with transverse stress and shear stress as relevant, see L.6.1.5.

L.6.1.2 Calculation of global stresses

Global longitudinal stresses should be calculated by FE-analysis, see L.5.2. For parallel parts of the midship, a simplified calculation of the hull girder section modulus can be performed. For units with moonpool / turret, a FE-analysis shall be carried out to describe the stress distribution in way of the openings, in particular in way of deck and bottom, and at termination of longitudinal strength elements. Global shear stresses shall be calculated considering the shear flow distribution in the hull.

L.6.1.3 Calculation of local transverse and longitudinal stresses

Local transverse and longitudinal stresses shall be superimposed with the global longitudinal stresses. Local stresses should be taken from a FEM analysis of a typical cargo area, see L.5.2. Phase information should be taken into account when available.

The FEM analysis shall include extreme hogging and sagging conditions as described in Figure L.6-1. All relevant variations in tank filling should be considered in the analysis and reflected in the Operation Manual. The following stress components can be found from the FEM analysis, and should be combined with other stress components as described in L.6.1.5:

- Transverse stresses in webframes
- Double shell and double bottom stresses
- Local shear stresses in panels

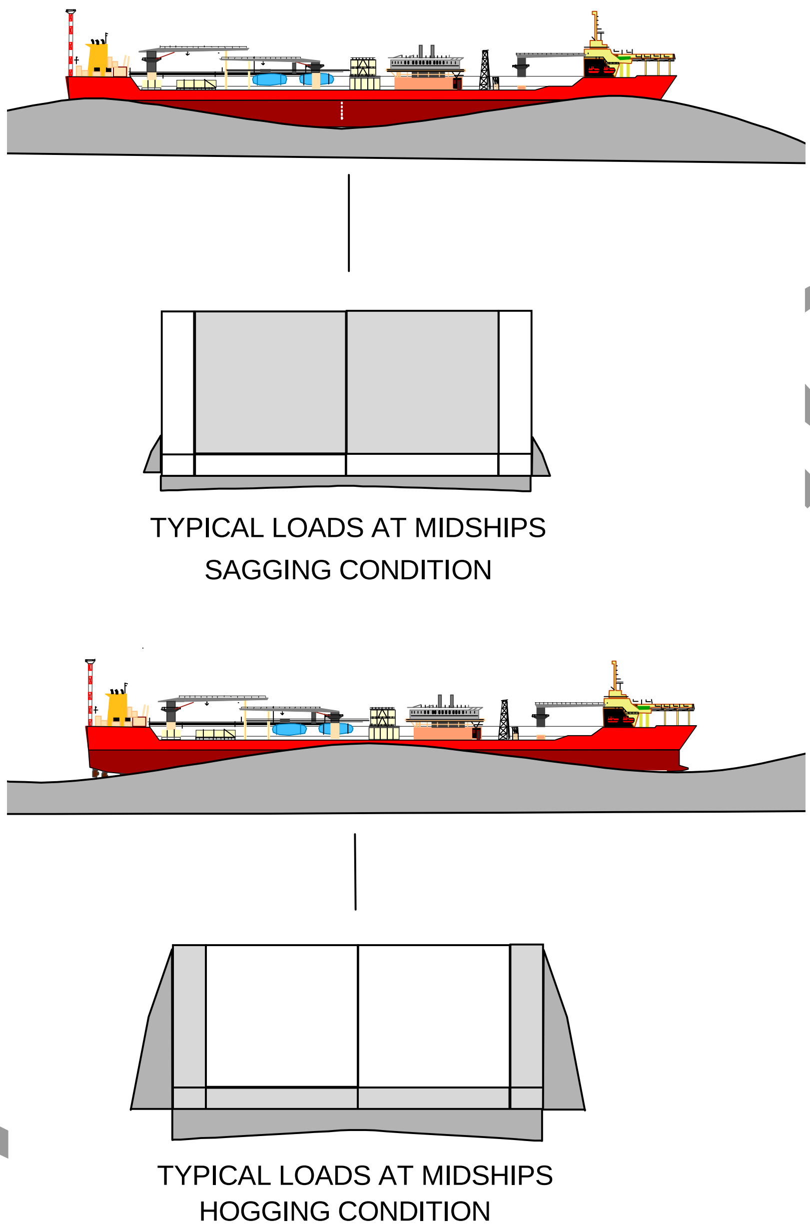

Figure L.6-1 Typical extreme sagging and hogging conditions

L.6.1.4 Calculation of local pressures

Sea pressure :
The dynamic sea pressure should be taken from a 3D hydrodynamic wave analysis. Action coefficients according to L.6.1.1 shall be considered.

Tank pressure:
The internal tank pressure shall be calculated in accordance with NORSOK N-003.

L.6.1.5 Combination of stresses

Total longitudinal design stress in the structure can be calculated as:

$$\sigma_{x,\ Total} = \sigma_{x,Global} + \sigma_{x,\ Local} \qquad \textbf{(L.6.3)}$$

Total transverse design stress in the structure is found directly from the FE-analysis:

$$\sigma_{y,\ Total} = \sigma_{y,\ Local} + \sigma_{y,\ Global} \qquad \textbf{(L.6.4)}$$

$\sigma_{y,\ Global}$ is normally relevant for the moonpool / turret area, and may be neglected in parallel parts of the midship.

Total design shear stress in the structure can be calculated as:

$$\tau_{x,\ Total} = \tau_{Global} + \tau_{\ Local} \qquad \textbf{(L.6.5)}$$

The combination of stresses should take into account actual stress directions and phase. However, if phase information is limited or uncertain, the maximum design value for each component may be combined as a 'worst-case' scenario.

Combination of typical stress components is shown in Figure L.6-2.

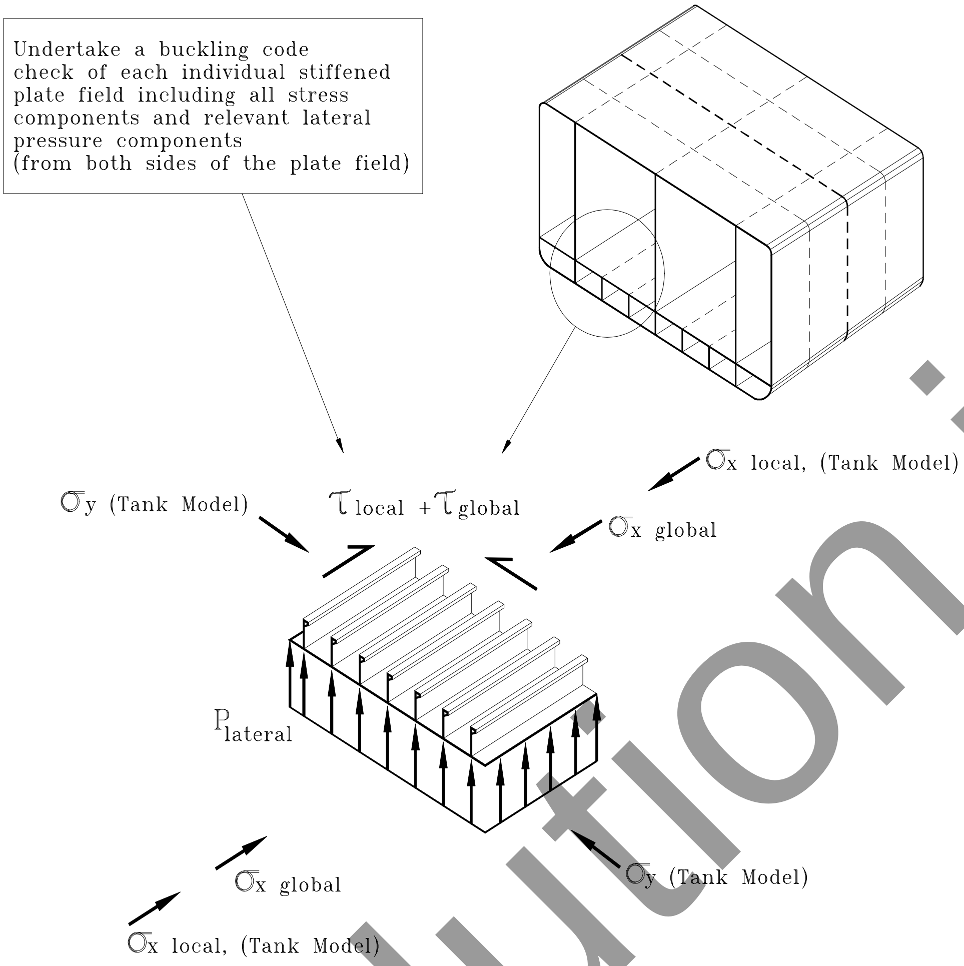

Figure L.6-2 Typical stress components acting on the hull beam.

L.6.1.6 Transverse structural strength

Transverse strength refers to the ability of the hull to resist lateral pressure and racking actions in combination with longitudinal action effects. This resistance is provided by means of transverse bulkheads, web frames, girders and stringers. Transverse strength should be evaluated using a finite element model of a specific portion of the hull, and the effects of process equipment deck actions should be included. The transverse strength shall meet the requirements given in L.6.1. The lateral pressure due to waves should be predicted using the hydrodynamic motion analysis approach, see L.5.3.

L.6.1.7 Capacity check

Capacity checks shall be performed in accordance with Chapter 6.

L.6.2 Local structural strength

Local structure not taking part of the overall strength of the unit may be designed in accordance with technical requirements given in the DNV Rules for Ships, Pt.3 Ch.1, /3/. Such structure may be in the following areas:

- Fore Ship.

- Aft Ship.
- Superstructure e.g. Process Deck and Accommodation.

For evaluation of slamming, sloshing and green sea effects, see L.4.5.2, L.4.5.3 and L.4.5.4.

L.6.3 Turret and turret area / moonpool

L.6.3.1 General

The following areas shall be considered as relevant, with respect to structural response from the mooring actions, combined with other relevant actions:

- Structure in way of moonpool opening in the ship hull.
- Turret structure including support, towards the unit hull.
- Structure in way of loading buoy support.
- Gantry structure including support.

L.6.3.2 Structure in way of moonpool opening in the unit hull

The structural strength shall be evaluated considering all relevant, realistic action conditions and combinations, see L.4. In particular action combinations due to the following shall be accounted for in the design:

- Turret bearing reactions.
- Overall hull bending moments and shear forces.
- Internal and external pressure actions, covering the intended range of draughts and action conditions, including non-symmetric cases as applicable.

Finite element analyses shall be carried out. Particular attention shall be given to critical interfaces. Functional limitations (bearing function) and structural strength shall be evaluated.

Continuity of primary longitudinal structural elements should be maintained as far as practicable in way of the turret opening. Reductions in hull section modulus shall be kept at a minimum and compensation shall be made where necessary.

L.6.3.3 Turret structure

A FEM analysis of the turret structure shall be performed, see L.5.2.4, demonstrating that the structural strength of the turret is acceptable. The structural strength shall be evaluated considering all relevant, realistic action conditions and combinations, see L.4. In particular, action combinations due to the following actions (as applicable) shall be accounted for in the design:

- Unit motion induced accelerations
- Mooring actions
- Riser actions
- Turret bearing reactions (calculated based on all the relevant actions on the turret)
- Moonpool deformations (based on hull bending moments and shear forces)
- Internal and external pressure actions, covering the intended range of draughts and load conditions including non-symmetric cases as applicable. Filling of void spaces to be accounted for (if relevant).
- Local actions from equipment and piping system (weight, thermal expansion, mechanical actions)
- Green seas
- Wave slamming

Boundary conditions for the model shall reflect the true configuration of the interface towards the unit.

Local analyses, (model level 3 & 4, see L.5.2.5 and L.5.2.6) shall be performed for structural areas, which are critical for the structural integrity of the turret. The following list contains typical areas which should be considered:

- Structure in vicinity of riser connection(s)
- Riser hang-off structure
- Structure in way of fairleads
- Hang-off structure for anchor line
- Local structure transferring bearing reactions
- Chain lockers
- Pipe supports (single supports and complex structures)
- Equipment supports
- Foundation for transfer system (especially for swivel solutions)
- Lifting appliances and pad-eyes including structure in way of these

L.6.4 Topside facilities structural support

The structural strength of topside facilities structural support shall be evaluated considering all relevant action conditions and combinations of actions. Scantlings shall be determined on the basis of criteria, which combine, in a rational manner, the effects of global and local responses for each structural element.

The following actions shall be considered:

- Permanent actions (weight of structures, process and drilling equipment, piping etc.)
- Live actions (equipment functional actions related to liquid, machinery, piping reaction forces, helicopter, cranes etc.)
- Wave actions
- Wave accelerations (inertia actions)
- Hull deflections due to tank filling, wave, temperature differences etc.
- Wind
- Snow and ice
- Green sea

L.6.5 Transit conditions

For self-propelled units, requirements given by the RCS (and the flag state) for seagoing ships cover the transit condition. Due consideration should be made with respect to dynamic actions, in particular due to roll motion, on the topside equipment.

L.7 FATIGUE LIMIT STATES (FLS)

L.7.1 General

General requirements and guidance concerning fatigue criteria are given in Chapter 8, and Annex C.

Evaluation of the fatigue limit states shall include consideration of all significant actions contributing to fatigue damage both in non-operational and operational design conditions.

The minimum fatigue life of the unit (before the design fatigue factor is considered) should be based upon a period of time not being less than the planned life of the structure.

Local effects, for example due to :

- slamming,
- sloshing,
- vortex shedding,
- dynamic pressures, and,
- mooring and riser systems,

shall be included in the fatigue damage assessment when relevant.

Calculations carried out in connection with the fatigue limit state may be undertaken without deducting the corrosion additions, provided a corrosion protection system in accordance with NORSOK, N-001 is maintained.

In the assessment of fatigue resistance, relevant consideration shall be given to the effects of stress raisers (concentrations) including those occurring as a result of :

- fabrication tolerances, (including due regard to tolerances in way of connections of large structural sections),
- cut-outs,
- details at connections of structural sections (e.g. cut-outs to facilitate construction welding).

L.7.2 Design fatigue factors

Criteria related to Design Fatigue Factors, are given in the Chapter 8. When determining the appropriate Design Fatigue Factor for a specific fatigue sensitive location, consideration shall be given to the following :

- Consideration of economic consequence of failure may indicate the use of larger design factors than those provided for as minimum factors.
- The categorisation : ‘Accessible / Above splash zone’ is, intended to refer to fatigue sensitive locations where the possibility for close-up, detailed inspection in a dry and clean condition exists. When all of these requirements are not fulfilled, the relevant design fatigue factor should be considered as being that appropriate for ‘Accessible / Below splash zone’, or, ‘No access or in the splash zone’ as relevant to the location being considered.
- Evaluation of likely crack propagation paths (including direction and growth rate related to the inspection interval), may indicate the use of a different Design Fatigue Factor than that which would be selected when the detail is considered in isolation, such that :
    - Where the likely crack propagation indicates that a fatigue failure, from a location satisfying the requirements for a 'Non-substantial' consequence of failure, may result in a 'Substantial' consequence of failure, such fatigue sensitive location is itself to be deemed to have a 'Substantial' consequence of failure.

- Where the likely crack propagation is from a location satisfying the requirement for a given 'Access for inspection and repair' category to a structural element having another access categorisation, such location is itself to be deemed to have the same categorisation as the most demanding category when considering the most likely crack path. For example, a weld detail on the inside (dry space) of a submerged shell plate should be allocated the same Fatigue Design Factor as that relevant for a similar weld located externally on the plate –see Figure L.7-1.

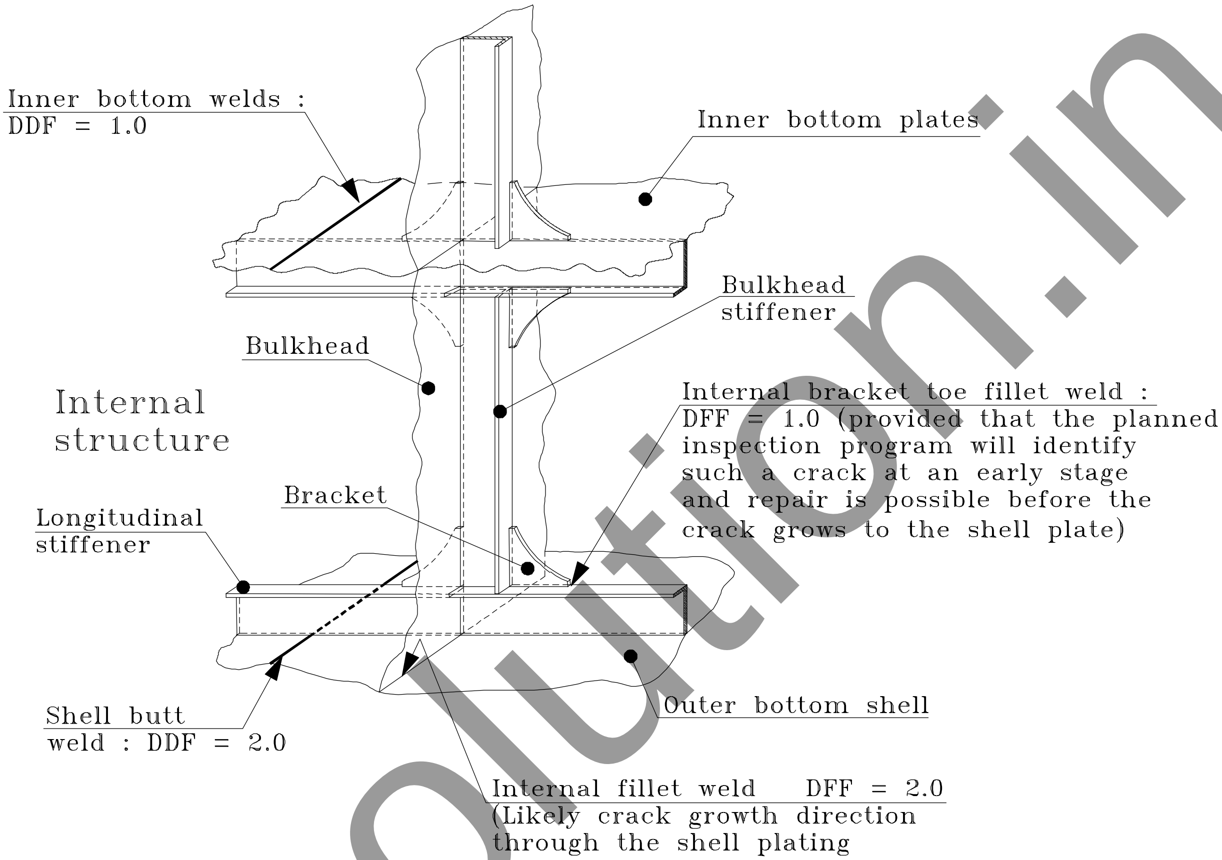

Figure L.7-1 Example illustrating considerations relevant for selection of design fatigue factors (DFF's) in locations considered to have 'Non-substantial' consequence of failure.

Notes:

1. Due to economic considerations (e.g. cost of repair to an external underwater structural element) the DFF's assigned a value of 2 in Figure L.7-1 may be considered as being more appropriately assigned a value of 3.)
2. The unit may be considered as "accessible and above the splash zone" (DFF = 1.0), see Annex C, if the survey extent e.g. given for main class (see DNV Ship Rules Pt.7 Ch.2) is followed i.e. drydocking for inspection and maintenance every 5 years.

L.7.3 Splash zone

The definition of 'splash zone' as given in NORSOK N-003, relates to a highest and lowest tidal reference. For ship shaped units, for the evaluation of the fatigue limit state, reference to the tidal datum should be substituted by reference to the draught that is intended to be utilised when condition monitoring is to be undertaken. The requirement that the extent of the splash zone is to extend 5 m above and 4 m below this draught may then be applied. (For application of requirements to corrosion addition, however, the normal operating draught should generally be considered as the reference datum).

If significant adjustment in draught of the unit is possible in order to provide for satisfactory accessibility in respect to inspection, maintenance and repair, account may be taken of this possibility in the determination of the Design Fatigue Factors. In such cases however, a margin of minimum 1 meter in respect to the minimum inspection draught should be considered when deciding upon the appropriate Design Fatigue Factor in relation to the criteria for 'Below splash zone' as opposed to 'Above splash zone'. Where draft adjustment possibilities exist, a reduced extent of splash zone may be applicable. Consideration should be given to operational requirements that may limit the possibility for ballasting / deballasting operations.

When considering utilisation of Remotely Operated Vehicle (ROV) inspection consideration should be given to the limitations imposed on such inspection by the action of water particle motion (e.g. waves). The practicality of such a consideration may be that effective underwater inspection by ROV, in normal sea conditions, may not be achievable unless the inspection depth is at least 10 metres below the sea surface.

L.7.4 Structural details and stress concentration factors

Fatigue sensitive details in the FPSO should be documented to have sufficient fatigue strength. Particular attention should be given to connection details of the following:

- Integration of the mooring system with hull structure
- Main hull bottom, side and decks
- Main hull longitudinal stiffener connections to transverse frames and bulkheads
- Main hull attachments; seats, supports etc.
- Openings in main hull
- Transverse frames
- Flare tower
- Riser interfaces
- Major process equipment seats

Selections of local details and calculations of stress concentration factors may be undertaken in accordance with DNV Classification Note 30.7, /11/. For details not covered in this document, stress concentration factors should be otherwise documented. Detailed finite element analysis may be utilized for determination of SCF's, according to procedure given in DNV Classification Note 30.7, /11/.

L.7.5 Design actions and calculation of stress ranges

L.7.5.1 Fatigue actions

An overview of fatigue actions is given in L.4.7. Site specific environmental data shall be used for calculation of long term stress range distribution. For units intended for multi field developments the site specific environmental actions for each field should be utilised considering the expected duration for each field. The most onerous environmental actions may be applied for the complete lifetime of the unit, as a conservative approach.

A representative range of action conditions shall be considered. It is generally acceptable to consider two action conditions, typically: ballast condition and the fully loaded condition, with appropriate amount of time at each condition, normally 50 % for each condition unless otherwise documented.

An appropriate range of wave directions and wave energy spreading shall be considered. For weather waning units, and in absence of more detailed documentation, the head sea direction shall be considered with the spreading taken as the most unfavourable between $\cos^2 - \cos^{10}$, see

NORSOK, N-003. Maximum spacing should be 30 degrees. Smaller spacing should be considered around the head-on heading, e.g. 15 degrees.

Typically, most unfavourable spreading will be $\cos^2$ for responses dominated by beam sea e.g. external side pressure, and $\cos^{10}$ for responses dominated by head sea e.g. global wave bending moment.

The following dynamic actions shall be included in a FLS analysis as relevant:

- Global wave bending moments
- External dynamic pressure due to wave and ship motion
- Internal dynamic pressure due to ship motion
- Sloshing pressures due to fluid motion in tanks for ships with long or wide tanks
- Loads from equipment and topside due to ship motion and acceleration

L.7.5.2 Topside structures

The following actions shall be considered for the topside structure:

- Hull deformations due to wave bending moment acting on the hull
- Wave induced accelerations (inertia actions)
- Vortex induced vibrations from wind
- Vibrations caused by operation of topside equipment

Additionally, the following low cycle actions should be considered where relevant for the topside structure:

- Hull deformations due to temperature differences
- Hull deformations due to change in filling condition e.g. ballasting / deballasting

The relevance of combining action components is dependent on the structural arrangement of the topside structure. Relevant stress components, both high cycle and low cycle, shall be combined, including phase information, when available. If limited phase information is available, the design may be based on ‘worst-case’ action conditions, by combination of maximum stress for each component.

L.7.5.3 Turret structure

The turret structure will normally be exposed to high dynamic action level. The choice of fatigue design factor for the turret should reflect the level of criticality and the access for inspection at the different locations, L.7.2.

The following actions shall be considered for the fatigue design of turret structures:

- Dynamic fluctuations of mooring line tension.
- Dynamic actions (tension and bending moment) from risers
- Varying hydrodynamic pressure due to wave action
- Varying hydrodynamic pressure due to unit accelerations (including added mass effects)
- Reactions in the bearing structure due to the other effects
- Inertia actions due to unit accelerations
- Fluctuating reactions in pipe supports due to thermal and pressure induced pipe deflections

Typical critical areas for fatigue evaluation listed in L.6.3.3.

L.7.5.4 Calculation of global dynamic stress ranges

Global stress ranges shall be determined from the global hull bending moments. If applicable, both vertical and horizontal bending moments shall be included. Shear lag effects and stress concentrations shall be considered.

L.7.5.5 Calculation of local dynamic stress ranges

Local stress ranges are determined from dynamic pressures acting on panels, accelerations acting on equipment and topside and other environmental actions resulting in local stresses to part of the structure.

Dynamic pressures shall be calculated from a 3D sink-source wave load analysis. The transfer function for the dynamic pressure could either be used directly to calculate local stress transfer functions and combined with the global stress transfer function, or a long-term pressure distribution could be calculated. As a minimum, the following dynamic pressures components shall be considered:

- Double hull stresses due to bending of double hull sections between bulkheads
- Panel stresses due to bending of stiffened plate panels
- Plate bending stresses due to local plate bending

For a description of calculation of local stress components, reference is made to DNV Classification Note 30.7, /11/.

L.7.5.6 Combination of stress components

Global and local stresses should be combined to give the total stress range for the detail in question. In general, the global and the local stress components differ in amplitude, phase and location. The method of combining these stresses for the fatigue damage calculation will depend on the location of the structural detail. A method for combination of actions is given in DNV Classification Note 30.7, /11/.

L.7.6 Calculation of fatigue damage

L.7.6.1 General

The basis for determining the acceptability of fatigue resistance, with respect to wave actions, shall be appropriate stochastic fatigue analyses. The analyses shall be undertaken utilising relevant site specific environmental data and take appropriate consideration of both global and local (e.g. pressure fluctuation) dynamic responses. (These responses do not necessarily have to be evaluated in the same model but the cumulative damage from all relevant effects should be considered when evaluating the total fatigue damage.)

Simplified fatigue analyses may form the basis of a 'screening' process to identify locations for which a detailed, stochastic fatigue analysis should be undertaken. Such simplified fatigue analysis shall be calibrated, see L.7.6.2.

Local, detailed FE-analysis (e.g. unconventional details with insufficient knowledge about typical stress distribution) should be undertaken in order to identify local stress distributions, appropriate SCF's, and/or extrapolated stresses to be utilised in the fatigue evaluation, see Annex C for further details. Dynamic stress variations through the plate thickness shall be documented and considered in such evaluations.

Explicit account shall be taken of any local structural details that invalidate the general criteria utilised in the assessment of the fatigue strength. Such local details may, for example be access openings, cut-outs, penetrations etc. in structural elements.

Principal stresses (see Annex C) should be utilised in the evaluation of fatigue responses.

L.7.6.2 Simplified fatigue analysis

Provided that the provisions stated in L.7.6.1, are satisfied, simplified fatigue analysis may be undertaken in order to establish the general acceptability of fatigue resistance. In all cases when a simplified fatigue analysis is utilized a control of the results of the simplified fatigue evaluation, compared to the stochastic results, shall be documented to ensure that the simplified analysis provides for a conservative assessment for all parts of the structure being considered.

The fatigue damage may be calculated based on a cumulative damage utilizing Miner-Palmgren summation, applying a Weibull distribution to describe the long-term stress range. The stress range shape parameter and the average zero-crossing frequency may be taken from the long-term stress distribution, utilizing the stress transfer function and environmental data for the operating area. A simplified method of fatigue analysis is described in DNV Classification Note 30.7, /11/.

L.7.6.3 Stochastic fatigue analysis

Stochastic fatigue analyses shall be based upon recognised procedures and principles utilising relevant site specific data.

Providing that it can be satisfactorily documented, scatter diagram data may be considered as being directionally specific. In such cases, the analyses shall include consideration of the directional probability of the environmental data. Relevant wave spectra shall be utilised. Wave energy spreading may be taken into account if relevant.

Structural response shall be determined based upon analyses of an adequate number of wave directions. Generally a maximum spacing of 30 degrees should be considered. Transfer functions should be established based upon consideration of a sufficient number of periods, such that the number, and values of the periods analysed :

- adequately cover the site specific wave data,
- satisfactorily describe transfer functions at, and around, the wave ‘cancellation’ and ‘amplifying’ periods. (Consideration should be given to take account that such ‘cancellation’ and ‘amplifying’ periods may be different for different elements within the structure), and,
- satisfactorily describe transfer functions at, and around, the relevant excitation periods of the structure.

The method is described in DNV Classification Note 30.7, /11/.

L.8 ACCIDENTAL LIMIT STATES (ALS)

Minimum impact energy levels, action combinations and allowable utilisation levels shall comply with the requirements given in NORSOK, N-003. Relevant actions shall be determined based on a Risk Analysis.

L.8.1 Dropped objects

The possibility of dropped objects impacting on the structural components of the unit shall be considered in design. Resistance to dropped objects may be accounted for by indirect means, such as, using redundant framing configurations, and, materials with sufficient toughness in affected areas.

The masses of the dropped objects from crane operation to be considered for design of units are normally taken based upon operational hook actions of the platform crane. Critical areas of dropped objects are to be determined based on crane operation sectors, crane reach and actual movement of actions assuming a drop direction within an angle to the vertical direction of 5^o in air and 15^o in water.

L.8.2 Fire

General guidance and requirements concerning accidental limit state events involving fire are given in Chapter 9 and Annex A.

L.8.3 Explosion

In respect to design considering actions resulting from explosions one, or a combination of the following main design philosophies are relevant ;

- Ensure that the probability of explosion is reduced to a level where it is not required to be considered as a relevant design action.
- Ensure that hazardous locations are located in unconfined (open) locations and that sufficient shielding mechanisms (e.g. blast walls) are installed.
- Locate hazardous areas in partially confined locations and design utilising the resulting relatively small overpressures.
- Locate hazardous areas in enclosed locations and install pressure relief mechanisms (e.g. blast panels) and design for the resulting overpressure.

Structural design accounting for large plate field rupture resulting from explosion actions should normally be avoided due to the uncertainties of the actions and the consequences of the rupture itself.

Structural support of the blast walls and the transmission of the blast action into main structural members shall be evaluated. Effectiveness of connections and the possible outcome from blast, such as flying debris, shall be considered.

L.8.4 Collision

Resistance to unit collisions may be accounted for by indirect means, such as, using redundant framing configurations, and, materials with sufficient toughness in affected areas.

Collision impact shall be considered for all elements of the unit, which may be impacted by sideways, bow or stern collision. The vertical extent of the collision zone shall be based on the depth and draught of attending units and the relative motion between the attending units and the unit.

To avoid possible penetration of a cargo tank, the side structure of the unit shall be capable of absorbing the energy of a vessel collision with an annual probability of 10^{-4} or at least a vessel of 5000 tonnes with an impacting contact speed of 2 m/s.

L.8.5 Unintended flooding

A procedure describing actions to be taken after relevant unintended flooding shall be prepared. Structural aspects related to counterflooding (if relevant) shall be investigated.

The unintended flooding conditions shall be considered in the turret and topside structure design.

L.8.6 Loss of heading control

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

The loss of heading control condition shall be considered in the topside and turret structure design.

L.9 COMPARTMENTATION & STABILITY

L.9.1 General

In the assessment of compartmentation and stability of a ship shaped unit, consideration shall be given to all relevant detrimental effects, including those resulting from:

- environmental actions,
- relevant damage scenarios,
- rigid body motions,
- the effects of free-surface, and,
- boundary interactions (e.g. mooring and riser systems).

An inclining test should be conducted when construction is as near to completion as practical in order to accurately determine the unit's mass and position of the centre of gravity. Changes in mass conditions after the inclining test should be carefully accounted for. The unit's mass and position of the centre of gravity shall be reflected in the unit's Operating Manual.

L.9.2 Compartmentation and watertight integrity

General requirements relating to compartmentation of ship shaped units are given in ISO 13819-1, /6/. Detailed provisions given in the IMO MODU Code relating to subdivision and freeboard should be complied with. Special provisions relating to the Norwegian petroleum activities are given in NORSOK N-001. Additionally, the design criteria for watertight and weathertight integrity as stated in the relevant rules of Recognised Classification Societies should normally be satisfied.

The number of openings in watertight structural elements shall be kept to a minimum. Where penetrations are necessary for access, piping, venting, cables etc., arrangements shall be made to ensure that the watertight integrity of the structure is maintained.

Where individual lines, ducts or piping systems serve more than one watertight compartment, or are within the extent of damage resulting from a relevant accidental event, arrangements shall be provided to ensure that progressive flooding will not occur.

L.9.3 Stability

Stability of a ship shaped unit shall satisfy the requirements as stated in the IMO MODU Code. Special provisions relating to the Norwegian petroleum activities are given in NORSOK N-001.

Adequacy of stability shall be established for all relevant in-service and temporary phase conditions. The assessment of stability shall include consideration of both the intact and damaged conditions.

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L.10 SPECIAL CONSIDERATION

L.10.1 Structural details

For the design of structural details consideration should be given to the following:

- The thickness of internal structure in locations susceptible to excessive corrosion.
- The design of structural details, such as those noted below, against the detrimental effects of stress concentrations and notches:
  - Details of the ends and intersections of members and associated brackets.
  - Shape and location of air, drainage, and lightening holes.
  - Shape and reinforcement of slots and cut-outs for internals.
  - Elimination or closing of weld scallops in way of butts, “softening” bracket toes, reducing abrupt changes of section or structural discontinuities.
- Proportions and thickness of structural members to reduce fatigue damage due to engine, propeller or wave-induced cyclic stresses, particularly for higher strength steel members.

L.10.2 Positioning of superstructure

Living quarters, lifeboats and other means of evacuation shall be located in non-hazardous areas and be protected and separated from production, storage, riser and flare areas.

Superstructure shall be located such that a satisfactory level of separation and protection is provided for.

L.10.3 Structure in way of a fixed mooring system

Local structure in way of fairleads, winches, etc. forming part of the position mooring system is, as a minimum, to be capable of withstanding forces equivalent to 1.25 times the breaking strength of the individual mooring line. The strength evaluation should be undertaken utilising the most unfavourable operational direction of the anchor line. In the evaluation of the most unfavourable direction, account shall be taken of relative angular motion of the unit in addition to possible line lead directions.

L.10.4 Inspection and maintenance

The structural inspection and maintenance programmes shall be consistent with NORSOK N-001.

L.10.5 Facilities for inspection on location

Inspections may be carried out on location based on approved procedures outlined in a maintenance system and inspection arrangement, without interrupting the function of the unit. The following matters should be taken into consideration to be able to carry out condition monitoring on location:

- Arrangement for underwater inspection of hull, propellers, thrusters, rudder and openings affecting the units seaworthiness.
- Means of blanking of all openings including side thrusters.
- Use of corrosion resistant materials for shafts, and glands for propeller and rudder.
- Accessibility of all tanks and spaces for inspection.
- Corrosion protection of hull.
- Maintenance and inspection of thrusters.
- Ability to gas free and ventilate tanks

- Provisions to ensure that all tank inlets are secured during inspection
- Measurement of wear in the propulsion shaft and rudder bearings.
- Testing facilities of all important machinery.

L.10.6 Action monitoring

An on-board loading instrument complying with the requirements of the DNV Ship Rules, /3/ shall be installed in order to be able to monitor still water bending moments and shear forces, and the stability of the unit.

The limitations for still water bending moments and shear forces shall be in accordance with maximum allowable stillwater bending moments and shear forces specified in the Operating Manual.

L.10.7 Corrosion protection

The corrosion protection arrangement shall be consistent with the references listed in NORSOK N-001.

L.11 DOCUMENTATION

L.11.1 General

Adequate planning shall be undertaken in the initial stages of the design process in order to obtain a workable and economic structural solution to perform the desired function. As an integral part of this planning, documentation shall be produced identifying design criteria and describing procedures to be adopted in the structural design of the unit.

Applicable codes, standards and regulations should be identified at the commencement of the design.

When the design has been finalised, a summary document containing all relevant data from the design and fabrication phase shall be produced.

Design documentation (see below) shall, as far as practicable, be concise, non-voluminous, and, should include all relevant information for all relevant phases of the lifetime of the unit.

General requirements to documentation relevant for structural design are given in NORSOK, N-001, Section 5.

L.11.2 Design basis

A Design Basis Document shall be created in the initial stages of the design process to document the basis criteria to be applied in the structural design of the unit.

A summary of those items normally to be included in the Design Basis document is included below.

Unit description and main dimensions

A summary description of the unit, including :

- general description of the unit (including main dimensions and draughts),
- main drawings including :
  - main structure drawings (This information may not be available in the initial stage of the design),
  - general arrangement drawings,
  - plan of structural categorisation,
  - load plan(s), showing description of deck uniform (laydown) actions,
  - capacity (tank) plan,
- service life of unit,
- position keeping system description.

Rules, regulations and codes

A list of all relevant, applicable, Rules, Regulations and codes (including revisions and dates), to be utilised in the design process.

Environmental design criteria

Environmental design criteria (including all relevant parameters) for all relevant conditions, including :

- wind, wave, current, snow and ice description for 10^{-1} ,10^{-2} and 10^{-4} annual probability events,
- design temperatures.

Stability and compartmentation

Stability and compartmentation design criteria for all relevant conditions including :

- external and internal watertight integrity plan,
- lightweight breakdown report,
- design loadcase(s) including global mass distribution,
- damage condition waterlines.(This information may not be available in the initial stage of the design.)

Temporary phases

Design criteria for all relevant temporary phase conditions including, as relevant :

- limiting permanent, variable, environmental and deformation action criteria,
- procedures associated with construction, (including major lifting operations),
- relevant ALS criteria.

Operational design criteria

Design criteria for all relevant operational phase conditions including :

- limiting permanent, variable, environmental and deformation action criteria,
- designing accidental event criteria (e.g. collision criteria),
- tank loading criteria (all tanks) including a description of system, with :
  - loading arrangements
  - height of air pipes,
  - loading dynamics,
  - densities,
- mooring actions.

In-service inspection criteria

A description of the in-service inspection criteria and general philosophy (as relevant for evaluating fatigue allowable cumulative damage ratios).

Miscellaneous

A general description of other essential design information, including :

- description of corrosion allowances, where applicable.

L.11.3 Design brief

A Design Brief Document shall be created in the initial stages of the design process. The purpose of the Design Brief shall be to document the intended procedures to be adopted in the structural design of the unit. All applicable limit states for all relevant temporary and operational design conditions shall be considered in the Design Brief.

A summary of those items normally to be included in the Design Brief Document is included below.

Analysis models

A general description of models to be utilised, including description of :

- global analysis model(s),

- local analysis model(s),
- loadcases to be analysed.

Analysis procedures

A general description of analytical procedures to be utilised including description of procedures to be adopted in respect to :

- the evaluation of temporary conditions,
- the consideration of accidental events,
- the evaluation of fatigue actions,
- the establishment of dynamic responses (including methodology, factors, and relevant parameters),
- the inclusion of 'built-in' stresses (if relevant),
- the consideration of local responses (e.g. those resulting from mooring and riser actions, ballast distribution in tanks as given in the operating manual etc.)

Structural evaluation

A general description of the evaluation process, including :

- description of procedures to be utilised for considering global and local responses,
- description of fatigue evaluation procedures (including use of design fatigue factors, SN-curves, basis for stress concentration factors (SCF's), etc.).
- description of procedures to be utilised for code checking.

L.11.4 Documentation for operation (DFO)

Documentation requirements covering structural design, as applicable in the operational phase of the unit (e.g. the DFI resume), are given in NORSOK Z-001.

L.12 REFERENCES

/1/ 'Offshore Installations: Guidance on design, construction and certification', U.K., Health & Safety Executive, Fourth Edition, HMSO

/2/ Rules for Classification of Mobile Offshore Units, Jan. 1997, DNV

/3/ Rules for Classification of Ship, latest valid edition, DNV

/4/ Regulations relating to loadbearing structures etc. Issued by the Norwegian Petroleum Directorate.

/5/ Regulations of 20 Dec. 1991, No.878, concerning stability, watertight subdivision and watertight / weathertight closing means on mobile offshore units, Norwegian Maritime Directorate.

/6/ 'Petroleum and Natural Gas Industries – Offshore Structures', Part 1 : General Requirements, International Standard, ISO 13819-1.

/7/ Salvesen, N., Tuck, E. O. and Faltinsen, O., "Ship Motions and Sea Loads".Trans. SNAME, Vol 78, 1970.

/8/ Tanaka, N., "A Study of the Bilge Keels, Part 4, on the Eddy-Making Resistance to the Rolling of a Ship Hull". Japan Society of Naval Architects, Vol. 109, 1960.

/9/ Torsethaugen, Knut, "Model for a Doubly Peaked Spectrum". Sintef Report No. SFT22 96204 dated 1996-02-20.

/10/ NORSOK Z-001 Documentation for Operation (DFO)

/11/ DNV Classification Note 30.7: Fatigue Assessment of Ship Structures

L.13 COMMENTARY

Comm. L.4.5.2 Sloshing actions in tanks

The non-impact sloshing loads in the DNV Rules are given at annual probability level 10^{-4} and represent the sloshing loads generally applicable for a FLS structural evaluation. These loads are governed by the inertia forces induced by the liquid in the tank. This means that the Weibull slope parameter applicable for a FLS structural evaluation, will be approximately equal to the Weibull slope parameter of the liquid acceleration in the longitudinal direction for the longitudinal sloshing mode and in the transverse direction for the transverse sloshing mode. Based on analysis of several offshore vessels ranging in length from 100-260m, a Weibull slope parameter $h=1.0$ has been determined as being appropriate for an FLS structural evaluation of ship shaped offshore units, both in the longitudinal sloshing mode and in the transverse sloshing mode. The non-impact sloshing loads should be applied on strength members as indicated in DNV Ship Rules Pt.3, Ch.1, Sec.4 C300 Liquid in tanks.

The sloshing impact loads in the DNV Rules are given at annual probability level 10^{-8} (20 year return period) and represent the sloshing impact loads generally applicable for an ULS structural evaluation. In the period 1990-1994, DNV performed a significant amount of sloshing model tests, using irregular excitation, with the main focus on sloshing impact loads. The duration of the tests was typically 2-4 hours in model scale. Based on statistical analysis of these model tests, a Weibull slope parameter $h\approx1.0$ has been determined as being representative for sloshing impact loading. To arrive at 100 year return period impact pressures, the 10^{-8} values should be multiplied by a factor 1.15. The impact sloshing loads should be applied on strength members as indicated in DNV Ship Rules Pt.3, Ch.1, Sec.4 C300 Liquid in tanks.

A wave which will impose the maximum wave bending moment on the vessel will have a wavelength of the order ships length. Provided that separation between the period of encounter for this wave T_e and the natural period T of the fluid in the tank is sufficient ($T\leq0.75T_e$), use of a reduced wave bending moment in calculating the allowable stress used in connection with sloshing loads for deck longitudinals is acceptable. The strength calculation for deck longitudinals may then be based on a wave bending moment reduced by 25% relative to the design wave bending moment.

The applicable number of cycles in a fatigue evaluation will in principle depend on the vessel pitch response period for the longitudinal sloshing mode and on the vessel roll response period for the transverse sloshing mode. A simplified expression for the applicable response period, both for the longitudinal and transverse sloshing mode, may be found in DNV Class Note 30.7, Section 2.1.2, $T_{resp}=4\cdot\log_{10}(L)$, where L is the rule ship length.

Comm. L.4.5.3 Green water effect

It is recommended that provisions are made during model testing for suitable measurements to determine design pressures for local structural design. This implies that model tests should be performed at design draught, for sea states with a spectrum peak period approximately 70-100% of the pitch resonance period of the vessel. The vessel model should be equipped with load cells on the weather deck at positions of critical structural members or critical topside equipment.

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DESIGN OF STEEL STRUCTURES

ANNEX M
SPECIAL DESIGN PROVISIONS FOR COLUMN STABILIZED UNITS

CONTENTS

M.1 INTRODUCTION

M.1.1 General

This Annex provides requirements and guidance to the structural design of column stabilised units, constructed in steel, in accordance with the provisions of this NORSOK standard.

The requirements and guidance documented in this Annex are generally applicable to all configurations of column stabilised units, including those with :

- ring (continuous) pontoons,
- twin pontoons or,
- multi-footing arrangements.

Such units may be kept on station by either a passive mooring system (e.g. anchor lines), or an active mooring system (e.g. thrusters), or a combination of these methods.

Requirements concerning mooring and riser systems are not considered in this Annex.

A column stabilised unit may be designed to function in a number of modes, e.g. transit, operational and survival. Limiting design criteria for going from one mode of operation to another mode of operation shall be clearly established and documented. Such limiting design criteria shall include relevant consideration of the following items :

- intact condition, structural strength,
- damaged condition, structural strength,
- air gap, and,
- compartmentation and stability.

For novel designs, or unproved applications of designs where limited or no direct experience exists, relevant analyses and model testing, shall be performed which clearly demonstrate that an acceptable level of safety is obtained.

M.1.2 Definitions

Column stabilised unit: A floating unit that can be relocated. A column stabilised unit normally consists of a deck structure with a number of widely spaced, large diameter, supporting columns that are attached to submerged pontoons.

Notes:

1. In the context of this Annex the term : 'column stabilised unit' is often abbreviated to the term 'unit'.
2. The definition for a column stabilised unit is in accordance with the ISO 13819-1 definition for a ‘semi-submersible’ unit.

M.1.3 Non-operational phases

The structure shall be designed to resist relevant actions associated with conditions that may occur during all stages of the life-cycle of the unit. Such stages may include :

- fabrication,
- site moves,
- mating,
- sea transportation,
- installation, and,
- decommissioning.

Structural design covering marine operation, construction sequences shall be undertaken in accordance with NORSOK, N-001.

Marine operations should be undertaken in accordance with the requirements stated in NORSOK J-CR-003.

All marine operations shall, as far as practicable, be based upon well proven principles, techniques, systems and equipment and shall be undertaken by qualified, competent personnel possessing relevant experience.

Structural responses resulting from one temporary phase condition (e.g. a construction or transportation operation) that may effect design criteria in another phase shall be clearly documented and considered in all relevant design workings.

If it is intended to dry-dock the unit the bottom structure shall be suitably strengthened to withstand such actions.

Fabrication

Planning of construction sequences and of the methods of construction shall be performed. Actions occurring in fabrication phases shall be assessed and, when necessary the structure and the structural support arrangement shall be evaluated for structural adequacy.

Major lifting operations shall be evaluated to ensure that deformations are within acceptable levels and that relevant strength criteria are satisfied.

Mating

All relevant action effects incurred during mating operations shall be considered in the design process. Particular attention should be given to hydrostatic actions imposed during mating sequences.

Sea transportation

A detailed transportation assessment shall be undertaken which includes determination of the limiting environmental criteria, evaluation of intact and damage stability characteristics, motion response of the global system and the resulting, induced actions. The occurrence of slamming actions on the structure and the effects of fatigue during transport phases shall be evaluated when relevant.

Satisfactory compartmentation and stability during all floating operations shall be ensured.

All aspects of the transportation, including planning and procedures, preparations, seafastenings and marine operations should comply with the requirements of the Warranty Authority.

Installation

Installation procedures of foundations (e.g. drag embedded anchors, piles, suction anchor or dead weights etc.) shall consider relevant static and dynamic actions, including consideration of the maximum environmental conditions expected for the operations.

The actions induced by the marine spread involved in the operations and the forces exerted on the structures utilised in positioning the unit, such as fairleads and padeyes, shall be considered for local strength checks.

Decommissioning

Abandonment of the unit shall be planned for in the design stage. Decommissioning phases for column stabilised units are however not normally considered to provide design load conditions for the structure of a unit and may therefore normally be disregarded in the design phase.

M.2 STRUCTURAL CLASSIFICATION AND MATERIAL SELECTION

M.2.1 Structural classification

Selection of steel quality, and requirements for inspection of welds, shall be based on a systematic classification of welded joints according to the structural significance and the complexity of the joints/connections as documented in Chapter 5.

In addition to in-service operational phases, consideration shall be given to structural members and details utilised for temporary conditions, e.g. fabrication, lifting arrangements, towing arrangements, etc.

Basic considerations

Structural connections in column stabilised units designed in accordance with this Annex should normally not fall within the categorisation criteria relevant for Design Class DC1 or DC2. In particular, relevant failure of a single weld, or element, should not lead to a situation where the accidental limit state condition is not satisfied. Structural connections will therefore be categorised within classification groups DC3, DC4 or DC5.

Consideration shall be given to address areas where through thickness tensile properties may be required.

Special consideration shall be given to ensure the appropriate inspection category for welds with high utilisation in fatigue if the coverage with standard local area allocation is insufficient.

Examples of typical design classes applicable to column stabilised units are stated below. These examples provide minimum requirements and are not intended to restrict the designer in applying more stringent requirements should such requirements be desirable.

Typical locations - Design class : DC3

Locations in way of connections at major structural interfaces, e.g. pontoon/pontoon, pontoon/column, column/deck, and, major brace connections, should generally be classified as DC3 due to the complexity of the connections and the general difficulty with respect to inspection and repair.

Local areas in way of highly stressed complex structural connections, e.g. in way of fairleads, riser supports, topside structures, or other similar locations with high complexity should also be classified as DC3.

DC3 areas may be limited to local, highly stressed areas if the stress gradient at such connections is large.

Typical locations - Design class : DC4

Except as provided for in the description for DC3 and DC5 structural categorisation, connections appropriate to general stiffened plate fields, including connections of structural elements supporting the plate fields, e.g. girders and stiffeners, should normally be classified in Design Class DC4.

General brace structural connections, (e.g. butt welds in brace plate fields) should be designated DC4, as should connections of the main supporting structure for topside components and equipment.

Typical locations - Design class : DC5

All welds for non-main loadbearing structural elements, including ; non-watertight bulkheads, mezzanine('tween) decks, and, deck superstructures may be classified as DC5.

The same applies for welds for internal outfitting structures in general.

M.2.2 Material selection

Material specifications shall be established for all structural materials utilised in a column stabilised unit. Such materials shall be suitable for their intended purpose and have adequate properties in all relevant design conditions. Material selection shall be undertaken in accordance with the principles given in Chapter 5.

When considering criteria appropriate to material grade selection, adequate consideration shall be given to all relevant phases in the life cycle of the unit. In this connection there may be conditions and criteria, other than those from the in-service, operational phase, that provide the design requirements in respect to the selection of material. (Such criteria may, for example, be design temperature and/or stress levels during marine operations.)

In the selection of material grades adequate consideration shall be given to the appropriateness of the design temperature including the definition of such. When considering design temperature related to material selection the applicability that standard NORSOK requirements to fabrication of steel structures (NORSOK, M-CR-101) are based upon a minimum design temperature of -14 degrees Celsius shall be evaluated.

Selection of steel quality for structural components shall normally be based on the most stringent Design Class of the joints involving the component.

Consideration shall be given to the presence of tensile stress in the direction of the thickness of the plate when determining the appropriate steel quality.

M.2.3 Inspection categories

Welding, and the extent of non-destructive examination during fabrication, shall be in accordance with the requirements stipulated for the appropriate 'inspection category' as defined in NORSOK, M-101. Determination of inspection category should be in accordance with the criteria given in table 5.3 and 5.4.

Inspection categories determined in accordance with NORSOK N-004 provide requirements to the minimum extent of required inspection. When considering the economic consequence that repair may entail, for example, in way of complex connections with limited or difficult access, it may be considered prudent engineering practice to require more demanding requirements to inspection than the required minimum.

When determining the extent of inspection, and the locations of required NDE, in additional to evaluating design parameters (for example fatigue utilisation) consideration should be given to relevant fabrication parameters including; positioning of block (section) connections, the possibility that manual welded connections may not achieve the quality that automatic weld locations achieve, etc.

The lower the extent of NDT selected, the more important is the representativeness of the NDT selected and performed. The designer should therefore exercise good engineering judgement in indicating madatory locations for NDT, where variation of utility along welds is significant.

M.2.4 Guidance to minimum requirements

Figure M.2-1 to Figure M.2-3 illustrate minimum requirements to selection of Design Class, and Inspection Category for typical column stabilised unit, structural configurations.

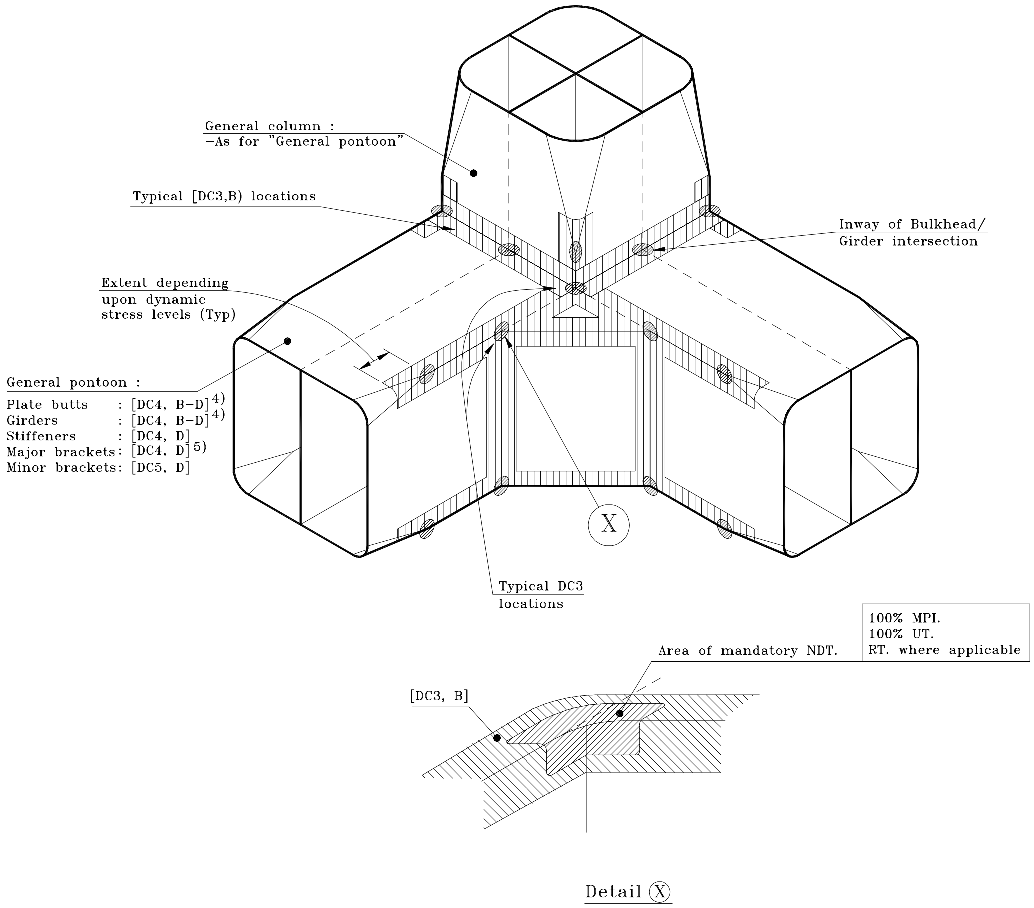

Figure M.2-1 Typical 'ring' pontoon design classes and inspection categories

Key:

1. [DCn, ..] Design Class n, N
2. [….., N] Inspection Category, N

Notes:

1. In way of the pontoon/column connection (except in way of brackets), the pontoon deckplate fields will normally be the continuous material. These plate fields should normally be material with documented through-thickness properties (i.e. Steel Quality Level I material).
2. Shaded areas indicated in the figures are intended to be three-dimensional in extent. This implies that, in way of these locations, the shaded area logic is not only to apply to the outer surface of the connection but is also to extend into the structure.
3. ‘Major brackets’ are considered, within the context of this figure, to be primary, load-bearing brackets supporting primary girders. (‘Minor brackets’ are all other brackets).
4. The inspection categories for general pontoon, plate butt welds and girder welds to the pontoon shell are determined based upon, amongst others: accessibility, fatigue utilisation, and dominating stress direction. (See NORSOK N-004, Tables 5.3 and 5.4). (e.g. Variations in dynamic stress levels across the section of the pontoon and also along the length of the pontoon may alter a general inspection category designation.)
5. Major bracket toes will normally be designated as locations with a mandatory requirement to MPI.

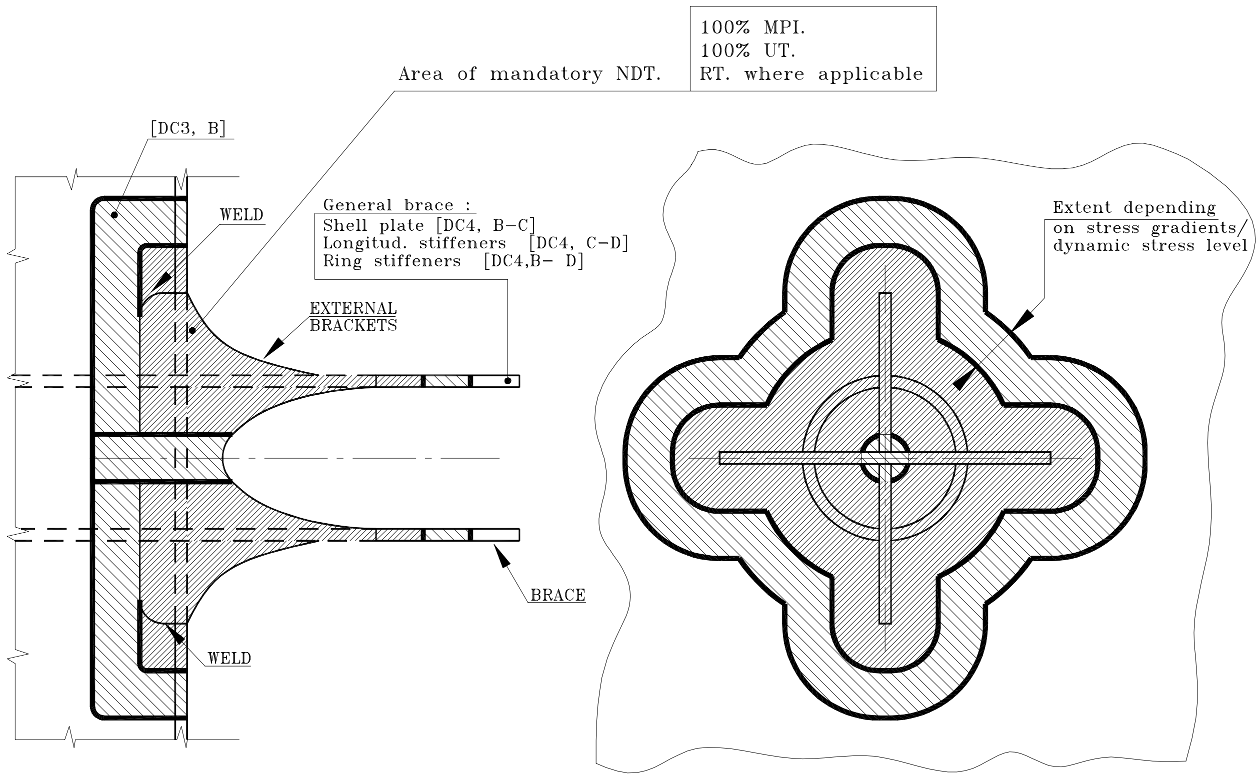

Figure M.2-2 Typical brace/column design classes and inspection categories

Key:

1. [DCn, ..] Design Class n, N
2. [….., N] Inspection Category, N

Notes:

1. In way of the column/brace connection the brace, and brace bracket plate fields, will normally be the continuous material. These plate fields should normally be material with documented through-thickness properties (i.e. Steel Quality Level I material).
2. Figure M.2-2 illustrates a relatively small brace connected to a column. For large brace connections the extent of the DC3 area may be limited somewhat to that illustrated..

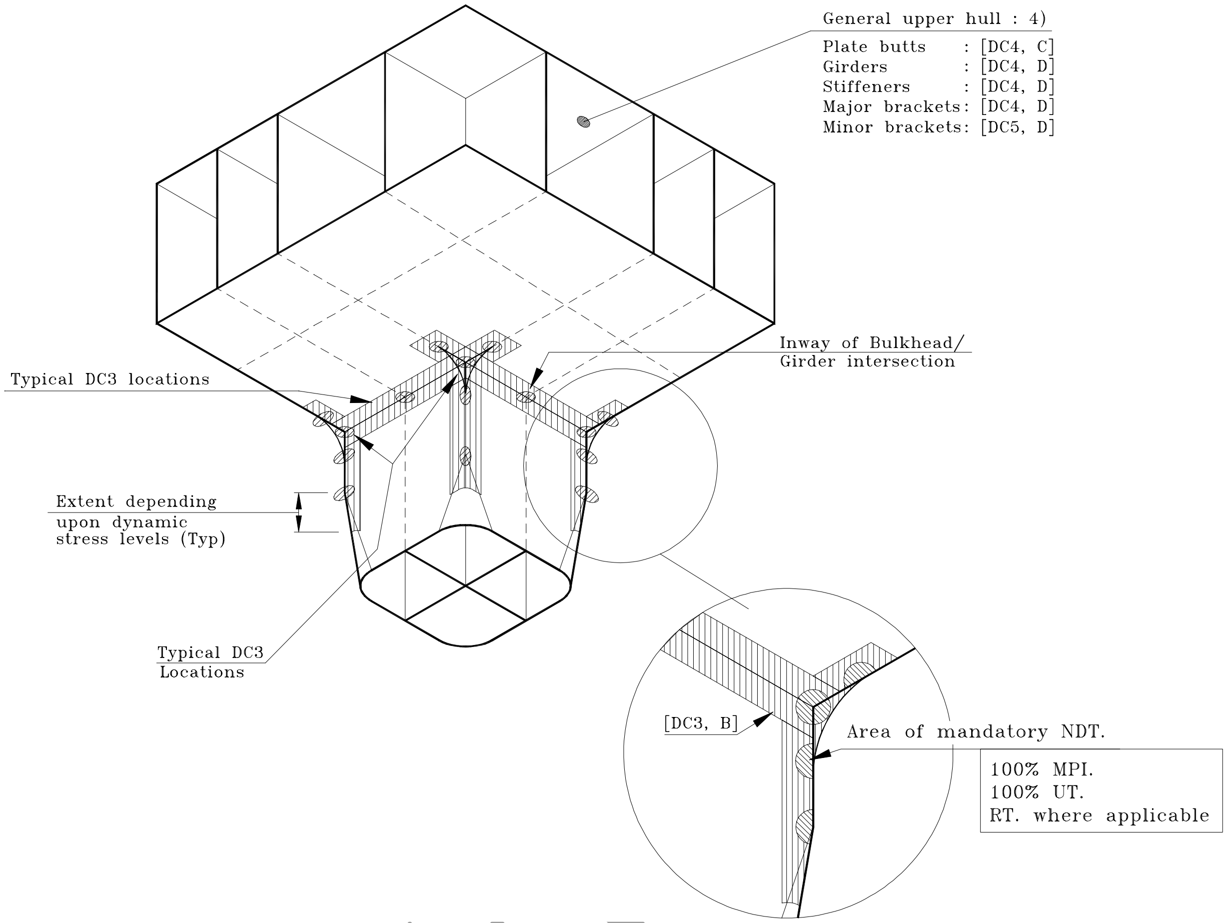

Figure M.2-3 Typical column/upper hull (deckbox) design classes and inspection categories

Key:

1. [DCn, ..] Design Class n, N
2. [….., N] Inspection Category, N

Notes:

1. In way of the column/upper hull connection (except in way of brackets) the upper hull deckplate fields will normally be the continuous material. These plate fields should normally be material with documented through-thickness properties (i.e. Steel Quality Level I material).
2. Shaded areas indicated in the figures are intended to be three-dimensional in extent. This implies that, in way of these locations, the shaded area logic is not only to apply to the outer surface of the connection but is also to extend into the structure.
3. ‘Major brackets’ are considered, within the context of this figure, to be primary, load-bearing brackets supporting primary girders. (‘Minor brackets’ are all other brackets).
4. The inspection categories stated for general upper hull are determined based upon, the assumption that the fatigue utilisation may be categorised as being 'low' (i.e. that NORSOK N-004, Table 5.3 is relevant). This assumption shall be verified.

M.3 ACTIONS

M.3.1 General

Characteristic actions are to be used as reference actions. Design actions are, in general, defined in NORSOK N-003. Guidance concerning action categories relevant for column stabilised unit designs are given in the following.

M.3.2 Permanent actions

Permanent actions are actions that will not vary in magnitude, position, or direction during the period considered, and include :

- 'lightmass' of the unit, including mass of permanently installed modules and equipment, such as accommodation, helideck, drilling and production equipment,
- hydrostatic pressures resulting from buoyancy, and,
- pretension in respect to mooring, drilling and production systems (e.g. mooring lines, risers etc.).

M.3.3 Variable actions

Variable actions are actions that may vary in magnitude, position and direction during the period under consideration.

Except where analytical procedures or design specifications otherwise require, the value of the variable actions utilised in structural design should normally be taken as either the lower or upper design value, whichever gives the more unfavourable effect. Variable actions on deck areas for local, primary and global design are stated in Table 5.1 in NORSOK, N-003. The factor to be applied to 'Global Design' actions (as stated in Table 5.1) is however not intended to limit the variable load carrying capacity of the unit in respect to non-relevant, variable load combinations. Global load cases should be established based upon 'worst case', representative variable load combinations, where the limiting global design criteria is established by compliance with the requirements to intact and damage hydrostatic and hydrodynamic stability. (See Commentary for a further discussion on this issue).

Global, design load conditions, and associated limiting criteria shall be documented in the stability manual for the unit, /1/.

Variations in operational mass distributions (including variations in tank load conditions in pontoons) shall be adequately accounted for in the structural design.

Design criteria resulting from operational requirements should be fully considered. Examples of such operations may be ;

- drilling, production, workover, and combinations thereof,
- consumable re-supply procedures,
- maintenance procedures,
- possible mass re-distributions in extreme conditions.

Local, design deck actions shall be documented on a 'load-plan'. This plan shall clearly show the design uniform and concentrated actions for all deck areas, for each relevant mode of operation.

Dynamic actions resulting from flow through air pipes during filling operations shall be adequately considered in the design of tank structures.

Operational actions imposed by supply boats while approaching, mooring, or, lying alongside the unit should be considered in the design. If fendering is fitted, the combined fender/structural system is to be capable of absorbing the energy of such actions. The resulting response in the structural system is to satisfy the requirements of the considered limit state. A typical design impact action would, for example, be a SLS design consideration of a 5000 tonne displacement vessel with a contact speed of 0.5 m/s.

M.3.4 Deformation actions

Deformation actions are actions due to deformations applied to the structure.

Deformation actions resulting from construction procedures and sequences (e.g. as a result of mating operations, welding sequences, forced alignments of structure etc.) shall be adequately accounted for.

Other relevant deformation action effects may include those resulting from temperature gradients, as for example :

- when hot-oil is stored in a compartment adjacent to the sea, and,
- radiation from flaring operations.

M.3.5 Environmental actions

Environmental actions are actions caused by environmental phenomena.

Environmental conditions shall be described using relevant data, for the relevant period and areas in which the unit is to be ; fabricated, transported, installed and operated. All relevant environmental actions shall be considered.

Amongst those environmental actions to be considered, in the structural design of a column stabilised unit, are :

- wave actions, (including variable pressure, inertia, wave 'run-up', and slamming actions)
- wind actions,
- current actions,
- temperature actions,
- snow and ice actions.

Amongst those environmental action induced responses to be considered, in the structural design of a column stabilised unit, are :

- dynamic stresses for all limit states (e.g. with respect to fatigue see M.3.7),
- rigid body motion (e.g. in respect to air gap and maximum angles of inclination),
- sloshing,
- slamming induced vibrations
- vortex induced vibrations (e.g. resulting from wind actions on structural elements in a flare tower),
- wear resulting from environmental actions at mooring and riser system interfaces with hull structures,

For column stabilised units with traditional, catenary mooring systems, earthquake actions may normally be ignored.

Further considerations with respect to environmental actions are given in NORSOK, N-003.

M.3.6 Accidental actions

Accidental actions are actions related to abnormal operation or technical failure.

Risk analyses shall be undertaken to identify and assess accidental events that may occur and the consequences of such events, (see Chapter 11). Relevant, structural design events for the ALS condition are identified from the risk analyses. The following ALS events are amongst those required to be considered in respect to the structural design of a column stabilised unit :

- dropped objects (e.g. from crane handling),
- fire,
- explosion,
- collision,
- unintended flooding, and,
- abnormal wave events.

Further considerations in respect to accidental actions are given in Chapter 11.

M.3.7 Repetitive actions

Repetitive actions, which may lead to possible significant fatigue damage, shall be evaluated. The following listed sources of fatigue actions shall, where relevant, be considered :

- waves (including those actions caused by slamming and variable (dynamic) pressures).
- wind (especially when vortex induced vibrations may occur),
- currents (especially when vortex induced vibrations may occur),
- mechanical vibration (e.g. caused by operation of machinery), and,
- mechanical loading/unloading (e.g. crane actions).

The effects of both local and global dynamic response shall be properly accounted for when determining response distributions related to fatigue actions.

M.3.8 Combination of actions

Load coefficients and load combinations for the design limit states are in general, given in NORSOK, N-001.

Structural strength shall be evaluated considering all relevant, realistic loading conditions and combinations. Scantlings shall be determined on the basis of criteria that combine, in a rational manner, the effects of relevant global and local responses for each individual structural element.

A sufficient number of load conditions shall be evaluated to ensure that the characteristic largest (or smallest) response, for the appropriate return period, has been established. (For example, maximum global, characteristic responses for a column stabilised unit may occur in environmental conditions that are not associated with the characteristic, largest, wave height. In such cases, wave period and associated wave steepness parameters are more likely to be governing factors in the determination of maximum/minimum responses.)

M.4 ULTIMATE LIMIT STATES (ULS)

M.4.1 General

General considerations in respect to methods of analysis are given in NORSOK N-003, Section 9.

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

It is normally not practical, in design analysis of column stabilised units, to include all relevant actions (both global and local) in a single model. Generally, a single model would not contain sufficient detail to establish local responses to the required accuracy, or to include consideration of all relevant actions and combinations of actions. (See Commentary for a more detailed discussion of this item). It is often the case that it is more practical, and efficient, to analyse different action effects utilising a number of appropriate models and superimpose the responses from one model with the responses from another model in order to assess the total utilisation of the structure. The modelling guidance given in the following therefore considers a proposed division of models in accordance with an acceptable analytical procedure. The procedures described are not however intended to restrict a designer to a designated methodology when an alternative methodology provides for an acceptable degree of accuracy, and includes all relevant action effects. Further, the modelling procedures and guidance provided are intended for establishing responses to an acceptable level of accuracy for final design purposes. For preliminary design, simplified models are recommended to be utilised in order to more efficiently establish design responses and to achieve a simple overview of how the structure responds to the designing actions.

M.4.2 Global models

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

An example of a global analysis model is shown in Figure M.4-1.

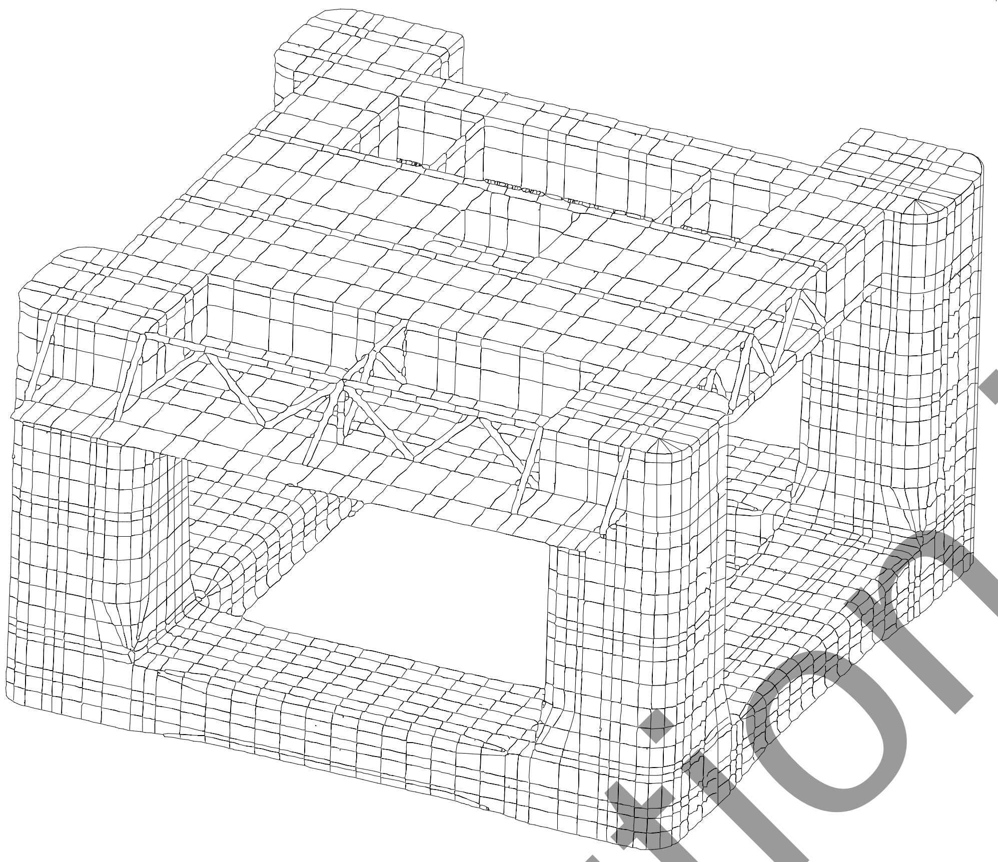

Figure M.4-1 Example of a global analysis model

M.4.2.1 Sea loading model

Global action effects

The environmental action model (e.g. simulating the appropriate wave conditions) shall be based upon site specific data in accordance with NORSOK N-003, Section 6. The stochastic nature of wave actions shall be adequately accounted for.

For extreme response analysis, the short-crested nature of the sea may be accounted for according to the requirements given in NORSOK N-003, section 6.2.2.3. Appropriate wave spectra criteria, as given in NORSOK N-003, section 6.2.2.3, based upon site specific environmental data shall be considered when simulating sea actions.

A simple, Morrison element model may normally be utilised in order to ascertain global actions in preliminary design stages. It may be prudent to apply a contingency factor to the global actions resulting from such a model. A contingency factor of 1.2 - 1.3 would generally be considered as being appropriate.

For final design a diffraction model should be created in order to more accurately establish the design actions.

Global viscous damping effects may be included in the model either by the inclusion of Morrison elements, or by the inclusion of relevant terms in a global damping matrix. Viscous effects are non-linear and should therefore be carefully considered in frequency domain analyses when such effects may be considered as significantly affecting the response, e.g. in the evaluation of motion response (in connection with establishing air gap and acceleration parameters).

Where hydrostatic actions are modelled explicitly (as opposed to an implicit inclusion from a diffraction model) it shall be ensured that adequate account is given to the inclusion of end-pressure action effects (e.g. in respect to axial actions acting on pontoon structures).

Local viscous (drag) action effects

For relatively slender members, viscous (drag) local action effects should be accounted for. In such cases, the inclusion of local drag action effects may be undertaken in a number of ways, for example :

- by use of hybrid (diffraction and Morrison) models of the structural elements,
- by only having a Morrison model of selected individual elements, or,
- by the inclusion of drag action effects by hand calculations.

M.4.2.2 Structural model

For large volume, thin-walled structures, three-dimensional finite element structural models created in shell (or membrane) finite elements, are normally required. For space frame structures consisting of slender members a three-dimensional, space frame representation of the structure may be considered adequate.

The stiffness of major structural connections (e.g. pontoon/column, column/deck) shall be modelled in sufficient detail to adequately represent the stiffness of the connection, such that the resulting responses are appropriate to the model being analysed.

It is preferable that actions resulting from the sea-loading model are mapped directly onto a structural model.

M.4.2.3 Mass modelling

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

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

Pontoon tank loading conditions

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

Responses for relevant load combinations with full pontoon tank loading conditions (max. ‘sagging’ condition) and empty tank loading conditions (max. ‘hogging’ condition) shall be considered. For column stabilised units with non-box type, upper hull arrangements it may also be necessary to investigate conditions with non-symmetric, diagonally distributed tank loading conditions. Response resulting from variations in pontoon tank load conditions may normally be considered as providing a significant contribution to the designing stress components in the areas of the upper and lower flanges of the pontoon deck and bottom plate fields. Such variations in stresses due to full/empty load combinations of pontoon tanks shall therefore be explicitly accounted for when considering logical combinations of global and local responses in the structural design.

M.4.2.4 Global analysis

Model testing shall be undertaken when non-linear effects cannot be adequately determined by direct calculations. In such cases, time domain analysis may also be considered as being necessary.

Where non-linear actions may be considered as being insignificant, or where such actions may be satisfactorily accounted for in a linear analysis, a frequency domain analysis can be undertaken. Transfer functions for structural response shall be established by analysis of an adequate number of wave directions, with an appropriate radial spacing. A sufficient number of periods shall be analysed to :

- adequately cover the site specific wave conditions,
- to satisfactorily describe transfer functions at, and around, the wave ‘cancellation’ and ‘amplifying’ periods, and,
- to satisfactorily describe transfer functions at, and around, the heave resonance period of the unit.

Global, wave-frequency, structural responses shall be established by an appropriate methodology, for example:

- a regular wave analysis,
- a 'design wave' analysis, or,
- a stochastic analysis.

These analytical methods are described in NORSOK N-003, Section 9.3.3.

Regular wave analysis :

A sufficient number of wave periods and wave directions shall be investigated to clearly demonstrate that maximum responses have been determined by the undertaken analysis. A wave directional radial spacing of maximum 15 degrees, should be utilised in a regular wave design analysis, (however it may be considered advisable to investigate with a spacing less than 15 degrees for structure with a substantial consequence of failure). Appropriate consideration shall be given to the modelling and inclusion of wave steepness characteristics.

Design wave analysis :

Design waves, utilised to simulate characteristic, global hydrodynamic responses shall be established based upon recognised analytical practices for column stabilised units. Appropriate consideration shall be given to the modelling and inclusion of wave steepness and spectral characteristics. A sufficient number of wave periods and wave directions shall be investigated to clearly demonstrate that maximum responses have been determined by the undertaken analysis. In design wave analysis a wave directional radial spacing of maximum 15 degrees or less, should normally be utilised.

The following characteristic responses will normally be governing for the global strength of a column stabilised unit :

- split (‘squeeze and pry’) force between pontoons,
- torsion (‘pitch connecting’) moment about a transverse horizontal axis,
- longitudinal shear force between the pontoons,
- combined longitudinal shear / split force between pontoons,
- longitudinal acceleration of deck mass,

- transverse acceleration of deck mass,
- vertical acceleration of deck mass.

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

Stochastic analysis :

Stochastic methods of analysis are, in principle recognised as providing the most accurate methods for simulating the irregular nature of wave actions. Long term, stress distributions shall be established based upon the relevant site specific, scatter diagram wave data.

In a stochastic design analysis, a wave directional radial spacing of 15 degrees or less, should be utilised.

Responses resulting from stochastic analytical procedures may however, not be particularly well suited to structural design as information concerning the simultaneity of internal force distribution is generally not available.

M.4.3 Local structural models

An adequate number of local structural models should be created in order to evaluate response of the structure to variations in local actions, e.g. in order to evaluate pressure acting across a structural section, lay-down loads acting on deck plate field, or, support point loads of heavy items of equipment.

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

Example of local pressure acting on a pontoon section :

An example of considerations that should be evaluated in connection with the load case of local pressures acting on a pontoon section of a column stabilised unit are given below.

Typical parameters contributing to variations in internal and external local pressure distributions acting on the pontoon tanks of a cross-section of a column stabilised unit are illustrated in Figure M.4-2. (For details concerning tank pressure actions reference should be made to NORSOK N-003, Section 5.4.)

A local structural model should be created with the intention of simulating local structural response for the most unfavourable combination of relevant actions.

When evaluating local response from the local model the following considerations may apply :

- The intention of the local model is to simulate the local structural response for the most unfavourable combination of relevant actions. Relevant combinations of internal and external pressures for tanks should be considered (for both the intact and damage load conditions). Maximum (and minimum) design pressures (both internal and external) should be investigated.
- If cross-section arrangements change along the length of the structure, a number of local models may be required in order to fully evaluate local response at all relevant sections.
- When internal structural arrangements are such that the structure is divided across its section (e.g. there may be a watertight centreline bulkhead or access/pipe tunnel in the pontoon) relevant combinations of tank loadings across the section shall be considered.

- In most cases, for external plate fields, relevant combinations of external and internal pressure loadcases will result in load conditions where the external and internal pressures are not considered to be acting simultaneously. For internal structures the simultaneous action of external and internal pressures loadcases may provide for the governing design loadcase. For internal structures the internal pressure should normally not be considered to act simultaneously on both sides of the structure.
- Actions are usually applied in the analysis model at the girder level and not at the individual stiffener level in order to ensure that local stiffener bending is not included in the model response as the stiffener bending response is explicitly included in the buckling code check, (see Chapter 6.5).
- For transversely stiffened structures (i.e. girders orientated in the transverse direction) the local responses extracted from the local model are normally responses in the structural transverse direction (σ_y in Figure M.4-3). For structural arrangements with continuous, longitudinal girder arrangements however a longitudinal response will also be of interest.
- For structural cross-sections without continuous longitudinal girder elements, two-dimensional structural models may be considered as being adequate.
- For space frame, beam models relevant consideration shall be given to shear lag effects.

Whilst this example considers the load case of local pressure acting on a pontoon section of a column stabilised unit, similar types of considerations are applicable in the evaluation of other local responses resulting from local actions on the column stabilised unit (e.g. lay-down actions).

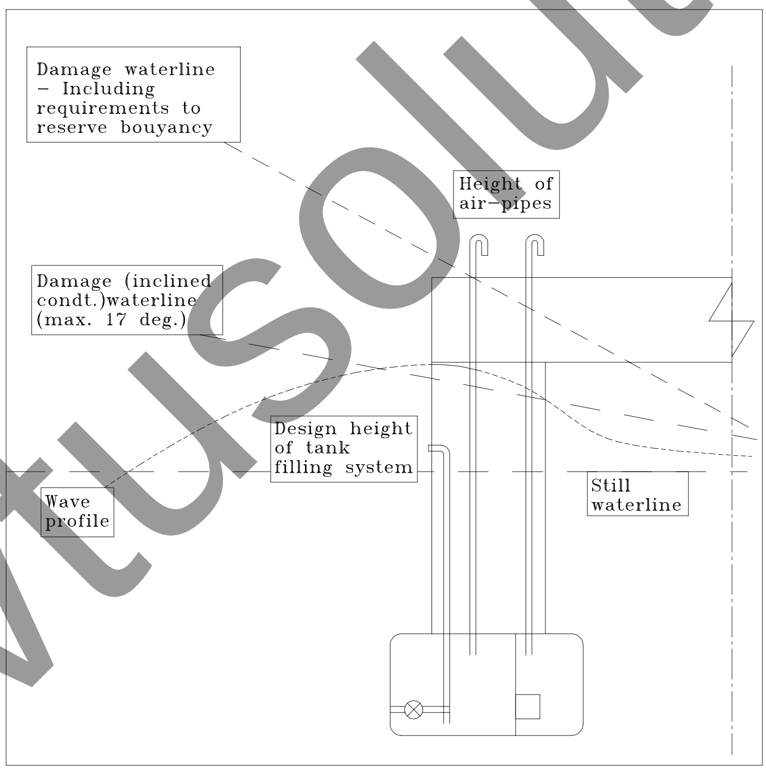

Figure M.4-2 Design action parameters acting as pressure on the pontoon tanks of a column stabilised unit.

M.4.4 Superimposing responses

The simultaneity of the responses resulting from the analysis models described in M.4.2 and M.4.3 may normally be accounted for by linear superposition of the responses for logical load combinations.

Structural utilisation shall be evaluated in accordance with the requirements of Chapter 6.

When evaluating responses by superimposing stresses resulting from a number of different models, consideration shall be given to the following :

- It should be ensured that responses from design actions are not included more than once. For example, when evaluating responses resulting from tank loading conditions (see M.4.2.3), it may be necessary to model a load condition, in the simplified model proposed in M.4.2.3, with a loadcase simulating that loadcase adopted in the global analysis model. In this way, stress variations resulting from relevant load conditions, when compared that load condition included in the global model may be established. Relevant peak responses may then be included in the capacity evaluation of the combined responses.
- Continuous, longitudinal structural elements, e.g. stiffened plate fields in the pontoon deck, bottom, sides, bulkheads, tunnels etc., located outside areas of global stress concentrations, may be evaluated utilising linear superposition of the individual responses as illustrated in Figure M.4-3 for a pontoon section. The summation of such responses should then be evaluated against the relevant structural capacity criteria. (In locations in way of column connections with pontoon and upper hull structures, global stress components become more dominant and should not be ignored.)
- When transverse stress components are taken directly from the local structural model an evaluation, that it is relevant to ignore transverse responses from the global model, shall be carried out. This may normally be undertaken by considering the transverse stresses in the global model along the length of the structural element and ensuring that no additional transverse stress components have been set-up as a result of global stress concentrations, 'skew' load conditions, or other interacting structural arrangements.
- Stiffener induced buckling failure normally tends to occur with lateral pressure on the stiffener side of the plate field. Plate induced buckling failure normally tends to occur with lateral pressure on the plate side. Relevant combinations of buckling code checking should therefore include evaluation of the capacity with relevant lateral pressure applied independently to both sides of the plate field.
- In order to ensure that local bending stress components resulting from action acting directly on the stiffeners (σ_{bs}, σ_{bp} -see Chapter 6) are included in the buckling code check, the lateral pressure should be explicitly included in the capacity check. Unless, for the case in question, there is always a pressure acting over the stiffened plate field being evaluated, the capacity checking should include a buckling check with no lateral pressure in addition to the case with lateral pressure.

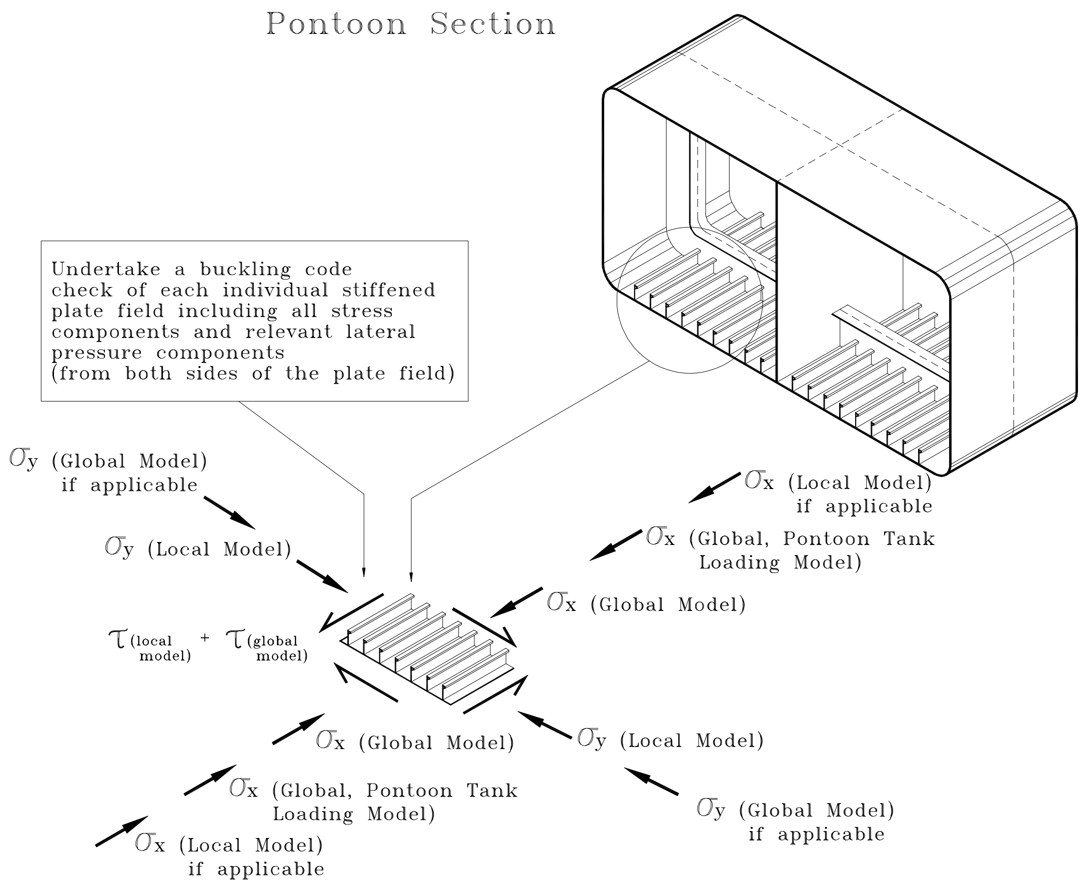

Figure M.4-3 Combination of stress components for buckling assessment of an individual stiffened plate field in a 'typical' pontoon section.

M.5 FATIGUE LIMIT STATES (FLS)

M.5.1 General

General requirements and guidance concerning fatigue criteria are given in Chapter 8, and Annex C.

Evaluation of the fatigue limit states shall include consideration of all significant actions contributing to fatigue damage both in non-operational and operational design conditions.

The minimum fatigue life of the unit (before the design fatigue factor is considered) should be based upon a period of time not being less than the planned life of the structure.

Assumptions related to the resistance parameters adopted in the fatigue design, e.g. with respect to corrosion protection, (see Annex C) shall be consistent with the in-service structure.

Local effects, for example due to :

- slamming,
- sloshing,
- vortex shedding,
- dynamic pressures, and,
- mooring and riser systems,

shall be included in the fatigue damage assessment when relevant.

In the assessment of fatigue resistance, relevant consideration shall be given to the effects of stress raisers (concentrations) including those occurring as a result of :

- fabrication tolerances. (Including due regard to tolerances in way of connections of large structural sections, e.g. as involved in mating sequences or section joints),
- cut-outs,
- details at connections of structural sections (e.g. cut-outs to facilitate construction welding).

M.5.2 Design fatigue factors

Criteria related to Design Fatigue Factors, are given in the Chapter 8.

Structural components and connections designed in accordance with this Annex should normally satisfy the requirement to damage condition according to the Accidental Limit State with failure in the actual joint defined as the damage. As such Design Fatigue Factors selected in accordance with Chapter 8, Table 8-1 should normally fall within the classification: "Without substantial consequences".

When determining the appropriate Design Fatigue Factor for a specific fatigue sensitive location, consideration shall be given to the following :

- Consideration that economic consequence of failure may indicate the use of larger design factors than those provided for as minimum factors.
- The categorisation : ‘Accessible / Above splash zone’ is, intended to refer to fatigue sensitive locations where the possibility for close-up, detailed inspection in a dry and clean condition exists. If any of these requirements are not fulfilled, the relevant design fatigue factor should be considered as being that appropriate for ‘Accessible / Below splash zone’, or, ‘No access or in the splash zone’ as relevant to the location being considered.
- Evaluation of likely crack propagation paths (including direction and growth rate related to the inspection interval), may indicate the use of a different Design Fatigue Factor than that which would be selected when the detail is considered in isolation, such that ;

- Where the likely crack propagation indicates that a fatigue failure, from a location satisfying the requirements for a 'Non-substantial' consequence of failure, may result in a 'Substantial' consequence of failure, such fatigue sensitive location is itself to be deemed to have a 'Substantial' consequence of failure.
- Where the likely crack propagation is from a location satisfying the requirement for a given ‘Access for inspection and repair’ category to a structural element having another access categorisation, such location is itself to be deemed to have the same categorisation as the most demanding category when considering the most likely crack path. For example, a weld detail on the inside (dry space) of a submerged shell plate should be allocated the same Design Fatigue Factor as that relevant for a similar weld located externally on the plate –see Figure M.5-1.

Implications of the above are that for internal structures, below or above splash zone, the Design Fatigue Factor (DFF) shall normally be 1 where access for inspection and repair is possible. Non-accessible areas, like welds covered with fire proofing and other areas inaccessible for inspection, shall be designed with DFF=3.

The hull shell below splash zone shall normally be assigned a DFF=2. This applies also to any attachment, internal or external, from where a fatigue crack can grow into the shell.

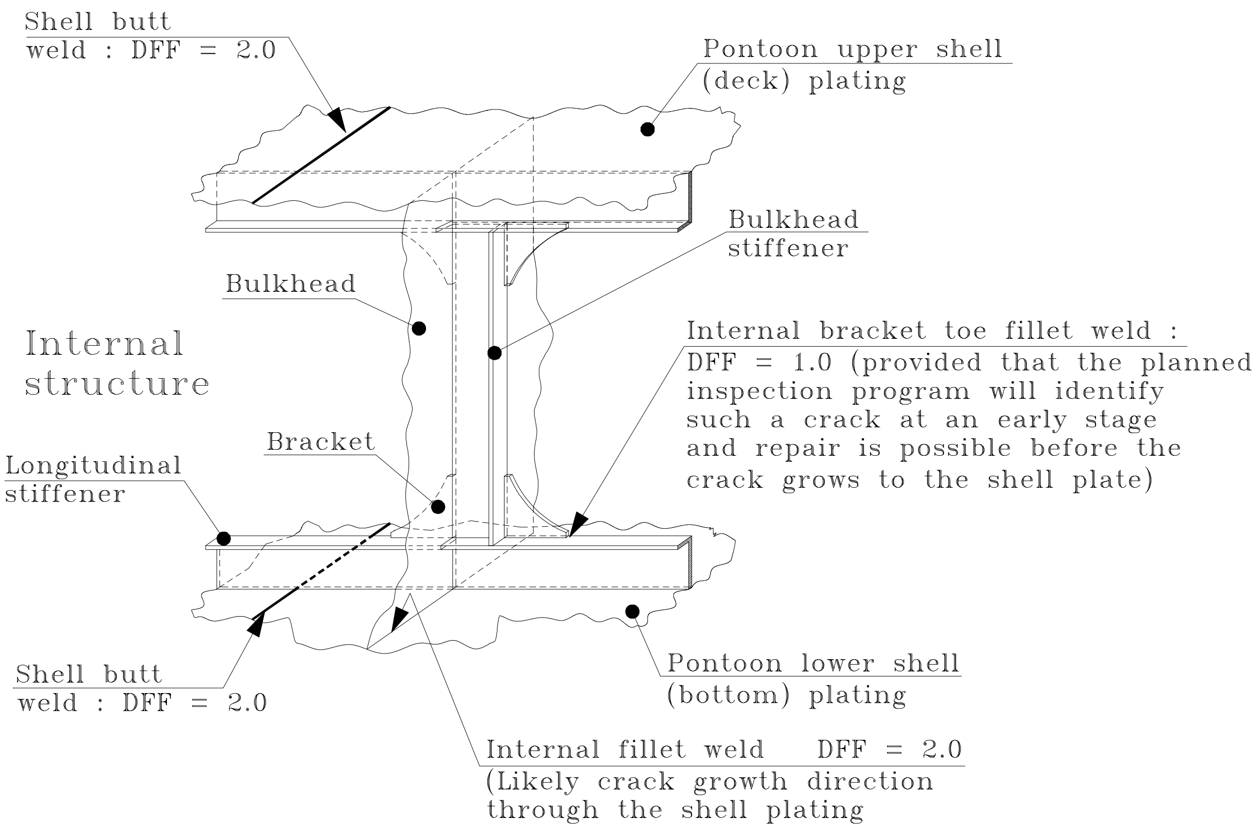

Figure M.5-1 Example illustrating considerations relevant for selection of design fatigue factors (DFF’s) in locations considered to have ‘non-substantial’ consequence of failure.

M.5.3 Splash zone

The definition of 'splash zone' as given in NORSOK N-003, Section 6.6.3, relates to a highest and lowest tidal reference. For column stabilised units, for the evaluation of the fatigue limit state, reference to the tidal datum should be substituted by reference to the draught that is intended to be utilised when condition monitoring is to be undertaken. The requirement that the extent of the splash zone is to extend 5m above and 4m below this draught may then be applied. (For application of requirements to corrosion addition, however, the normal operating draught should generally be considered as the reference datum).

If significant adjustment in draught of the unit is possible in order to provide for satisfactory accessibility in respect to inspection, maintenance and repair, account may be taken of this possibility in the determination of the Design Fatigue Factors. In such cases however, a sufficient margin in respect to the minimum inspection draught should be considered when deciding upon the appropriate Design Fatigue Factor in relation to the criteria for 'Below splash zone' as opposed to 'Above splash zone'. Where draft adjustment possibilities exist, a reduced extent of splash zone may be applicable. (See Commentary for further details).

The entire unit may be regarded as being above the splash zone if the unit is to be regularly dry-docked every 4-5 years.

M.5.4 Fatigue analysis

M.5.4.1 General

The basis for determining the acceptability of fatigue resistance, with respect to wave actions, shall be appropriate stochastic fatigue analyses. The analyses shall be undertaken utilising relevant site specific environmental data and take appropriate consideration of both global and local (e.g. pressure fluctuation) dynamic responses. (These responses do not necessarily have to be evaluated in the same model but the cumulative damage from all relevant effects should be considered when evaluating the total fatigue damage.)

Simplified fatigue analyses may form the basis of a 'screening' process to identify locations for which a detailed, stochastic fatigue analysis should be undertaken (e.g. at critical intersections). Such simplified fatigue analysis shall be calibrated –see M.5.4.2.

Local, detailed FE-analysis of critical connections (e.g. pontoon/pontoon, pontoon/column, column/deck and brace connections) should be undertaken in order to identify local stress distributions, appropriate SCF's, and/or extrapolated stresses to be utilised in the fatigue evaluation –see Annex C2.8 for further details. Dynamic stress variations through the plate thickness shall be documented and considered in such evaluations.

Explicit account shall be taken of any local structural details that invalidate the general criteria utilised in the assessment of the fatigue strength. Such local details may, for example be access openings, cut-outs, penetrations etc. in structural elements.

Principal stresses (see C2.2) should be utilised in the evaluation of fatigue responses.

M.5.4.2 Simplified fatigue analysis

Provided that the provisions stated in M.5.4.1, are satisfied, simplified fatigue analysis may be undertaken in order to establish the general acceptability of fatigue resistance. In all cases when a simplified fatigue analysis is utilised a control of the results of the simplified fatigue evaluation, compared to the stochastic results, shall be documented to ensure that the simplified analysis provides for a conservative assessment for all parts of the structure being considered.

Simplified fatigue analysis is particularly well suited for preliminary design evaluation studies. In such cases, a space frame model of the structure may be considered as being adequate. For final design however the basis model for the 'screening' evaluation should be a model of similar (or better) detail to that described in M.4.2.2.

Simplified fatigue analyses should be undertaken utilising appropriate design parameters that provide for conservative results. Normally a two-parameter, Weibull distribution (see C2.9) may be utilised to describe the long-term stress range distribution. In such cases the Weibull shape parameter ('h' –see Eqn.(M.5.1) below) should normally have a value between 1.0 - 1.1.

$$\Delta\sigma_{n_0} = \frac{(\ln n_0)^{\frac{1}{h}}}{(DFF)^{\frac{1}{m}}}\left[\frac{\bar{a}}{n_0\Gamma\left(1+m/h\right)}\right]^{\frac{1}{m}} \quad \textbf{(M.5.1)}$$

where :

n_0	is the total number of stress variations during the lifetime of the structure
$\Delta\sigma_{n_0}$	is the extreme stress range that is exceeded once out of n_0 stress variations. (The extreme stress amplitude $\Delta\sigma_{ampl\_n_0}$ is thus given by $\left(\Delta\sigma_{n_0}/2\right)$).
h	is the shape parameter of the Weibull stress range distribution
$\bar{a}$	is the intercept of the design S-N curve with the log N axis (see, for example, C2.3.3)
$\Gamma\left(1+m/h\right)$	is the complete gamma function (see C2.9)
m	is the inverse slope of the S-N curve (see C2.9)
DFF	is the Design Fatigue Factor.

Generally, the simplified global fatigue analysis should consider the 'F3', SN class curve (see C2.3), adjusted to include any thickness effect, as the minimum basis requirement. Areas not satisfying this requirement are normally to be excluded from the simplified fatigue evaluation 'screening' procedure. (This implies that connections with a more demanding SN fatigue class than F3, are not to be applied in the structure, e.g. if overlap connections are applied then the fatigue SN class is to be suitably adjusted.) The cumulative fatigue damage may then be obtained by considering the dynamic stress variation, $\Delta\sigma_{Actual(n_0)}$, which is exceeded once out of 'n_0' cycles (see eqn. (M.5.2)) with the allowable equivalent stress variation, $\Delta\sigma_{n_0}$, calculated from equation (M.5.1). The fatigue life thus obtained is found from eqn. (M.5.1)).

$$N_{Actual} = N_{Design}\left[\frac{\Delta\sigma_{n_0}}{\Delta\sigma_{Actual(n_0)}}\right]^{\mathbf{m}} \quad \textbf{(M.5.2)}$$

where :

N_{Actual}	is the actual (calculated) fatigue life
N_{Design}	is the target fatigue life
$\Delta\sigma_{n_0}$	is described in Eqn.(M.5.1)).
$\Delta\sigma_{Actual(n_0)}$	is the actual design, dynamic stress variation which is exceeded once out of 'n_0' cycles.

When the simplified fatigue evaluation involves utilisation of the dynamic stress responses resulting from the global analysis (as described in M.4.2), the response should be suitably scaled to the return period of the basis, minimum fatigue life of the unit. In such cases, scaling may normally be undertaken utilising the appropriate factor found from eqn (M.5.3).

$$\Delta\sigma_{n_0} = \Delta\sigma_{n_{100}} \left[\frac{\log n_0}{\log n_{100}} \right]^{\frac{1}{h}} \qquad \textbf{(M.5.3)}$$

where :

n_{100} is the number of stress variations (e.g. 100 years) appropriate to the global analysis

$\Delta\sigma_{n_{100}}$ is the extreme stress range that is exceeded once out of n_{100} stress variations.

(Other parameters are as for eqns. (M.5.1) and (M.5.2)).

M.5.4.3 Stochastic fatigue analysis

Stochastic fatigue analyses shall be based upon recognised procedures and principles utilising relevant site specific data.

Analyses shall include consideration of the directional probability of the environmental data. Providing that it can be satisfactorily documented, scatter diagram data may be considered as being directionally specific. Relevant wave spectra shall be utilised. Wave energy spreading may be taken into account if relevant.

Structural response shall be determined based upon analyses of an adequate number of wave directions. Generally a maximum radial spacing of 15 degrees should be considered. Transfer functions should be established based upon consideration of a sufficient number of periods, such that the number, and values of the periods analysed :

- adequately cover the site specific wave data,
- satisfactorily describe transfer functions at, and around, the wave ‘cancellation’ and ‘amplifying’ periods. (Consideration should be given to take account that such ‘cancellation’ and ‘amplifying’ periods may be different for different elements within the structure).
- satisfactorily describe transfer functions at, and around, the relevant excitation periods of the structure.

Stochastic fatigue analyses utilising simplified structural model representations of the unit (e.g. a space frame model) may form the basis of a ‘screening’ process to identify locations for which a, stochastic fatigue analysis, utilising a detailed model of the structure, should be undertaken (e.g. at critical intersections). When space frame, beam models are utilised in the assessment of the fatigue strength of large volume structures, (other than for preliminary design studies) the responses resulting from the beam models should be calibrated against a more detailed model to ensure that global stress concentrations are included in the evaluation process.

M.6 ACCIDENTAL LIMIT STATES (ALS)

M.6.1 General

General guidance and requirements concerning accidental events are given in Chapter 9, and Annex A.

Units shall be designed to be damage tolerant, i.e. credible accidental damages, or events, are not to cause loss of global structural integrity, (see M.9.1). The capability of the structure to redistribute actions should be considered when designing the structure.

In the design phase, attention shall be given to layout and arrangements of facilities and equipment in order to minimise the adverse effects of accidental events.

Satisfactory protection against accidental damage may be obtained by a combination of the following principles :

- reduction of the probability of damage to an acceptable level, and,
- reduction of the consequences of damage to an acceptable level.

Structural design in respect to the Accidental damage Limit State (ALS) shall involve a two-stage procedure considering :

- resistance of the structure to a relevant accidental event, and,
- capacity of the structure after an accidental event.

Global structural integrity shall be maintained both during and after an accidental event. Actions occurring at the time of a design accidental event and thereafter shall not cause complete structural collapse, or loss of hydrostatic (or hydrodynamic) stability (See M.8).

Requirements to compartmentation and stability in the damage condition are given in M.8.4. When the upper hull (deckbox) structure becomes buoyant in satisfying requirements to damage stability, /1/, consideration shall be given to the structural response resulting from such actions.

M.6.2 Dropped objects

Critical areas for dropped objects shall be determined on the basis of the actual movement of potential dropped objects (e.g. crane actions) relative to the structure of the unit itself. Where a dropped object is a relevant accidental event, the impact energy shall be established and the structural consequences of the impact assessed.

The impact energy at sea level is normally not to be taken less than 5 MJ for cranes with maximum lifting capacity of more than 30 tonnes, however, reduced impact energy may be acceptable for smaller cranes and special purpose cranes.

The impact energy below sea level is assumed to be equal to the energy at sea level, unless otherwise documented.

Generally, dropped object assessment will involve the following considerations :

- assessment of the risk and consequences of dropped objects impacting topside, wellhead (e.g. on the seafloor), and safety systems and equipment. The assessment shall identify the necessity of any local structural reinforcement or protection to such arrangements.
- assessment of the risk and consequences of dropped objects impacting externally on the hull structure. However the structural consequences are normally fully accounted for by the requirements for watertight compartmentation and damage stability (see M.8) and the requirement for structural redundancy of slender structural members (see M.9.1).

M.6.3 Fire

General guidance and requirements concerning accidental limit state events involving fire are given in Chapter 9 and Annex A.

M.6.4 Explosion

In respect to design considering actions resulting from explosions one, or a combination of the following main design philosophies are relevant :

- Ensure that the probability of explosion is reduced to a level where it is not required to be considered as a relevant design loadcase.
- Ensure that hazardous locations are located in unconfined (open) locations and that sufficient shielding mechanisms (e.g. blast walls) are installed.
- Locate hazardous areas in partially confined locations and design utilising the resulting, relatively small overpressures.
- Locate hazardous areas in enclosed locations and install pressure relief mechanisms (e.g. blast panels) and design for the resulting overpressure.

As far as practicable, structural design accounting for large plate field rupture resulting from explosion actions should normally be avoided due to the uncertainties of the actions and the consequences of the rupture itself.

Structural support of blast walls, and the transmission of the blast action into main structural members shall be evaluated when relevant. Effectiveness of connections and the possible outcome from blast, such as flying debris, shall be considered.

M.6.5 Collision

Collision shall be considered as a relevant ALS loadcase for all structural elements of the unit that may be impacted in the event of a collision. The vertical zone of impact shall be based on the depth and draught of the colliding vessel and the relative motion of the vessel and the unit.

In the assessment of the collision condition, the following general considerations with respect to structural strength will normally apply :

- An evaluation shall be undertaken in order to assess the extent of structural damage occurring to the unit at the time of impact.
- The extent of the local damage resulting from the collision should be compared to that damage extent implicit in the NMD regulations covering damage stability of the unit, /1/. Provided that the extent of local damage does not exceed that damage criteria stated in the NMD regulations, the unit shall satisfy the relevant damage stability requirements of the NMD. See also M.8.4. If the extent of the damage exceeds that damage criteria stated in the NMD regulations, /1/, an equivalent level of safety to that implicit in the NMD stability regulations should be documented.
- NMD damaged condition requirements, /1/, in respect to structural strength of watertight boundaries (including boundaries required for reserve buoyancy) shall be satisfied (see M.6.8).
- Global structural integrity of the unit after the collision shall be evaluated.
- Topside structural arrangements should be designed for the damaged (inclined) condition.

Considerations concerning structural evaluation at, and after, the time of the collision are given below.

Damage occurring at the time of collision

A structural evaluation shall be performed in order to document the extent of the local damage occurring to the unit at the time of impact.

If a risk analysis shows that the greatest relevant accidental event with regard to collision is a drifting vessel at 2m/s, with a displacement which does not exceed 5000 tonnes, the following kinetic energy occurring at the time of collision may be considered for the structural design ;

- 14 MJ for sideways collision, and,
- 11 MJ for bow or stern collision.

Local damage assessment may be undertaken utilising sophisticated non-linear analytical tools, however, simplified analytical procedures will normally be considered as being sufficient to evaluate the extent of damage occurring under the action of the collision.

Simplified local damage assessment of the collision event normally involves the following considerations :

- The typical geometry of the supply vessel, together with relevant force-deformation curves for side, bow and stern impact, documented in Annex A, may normally be utilised.
- In the local structural strength assessment the side, bow and stern profiles of the supply vessel are progressively 'stepped' into the collision zone of the column stabilised unit, see example shown in Figure M.6-1.

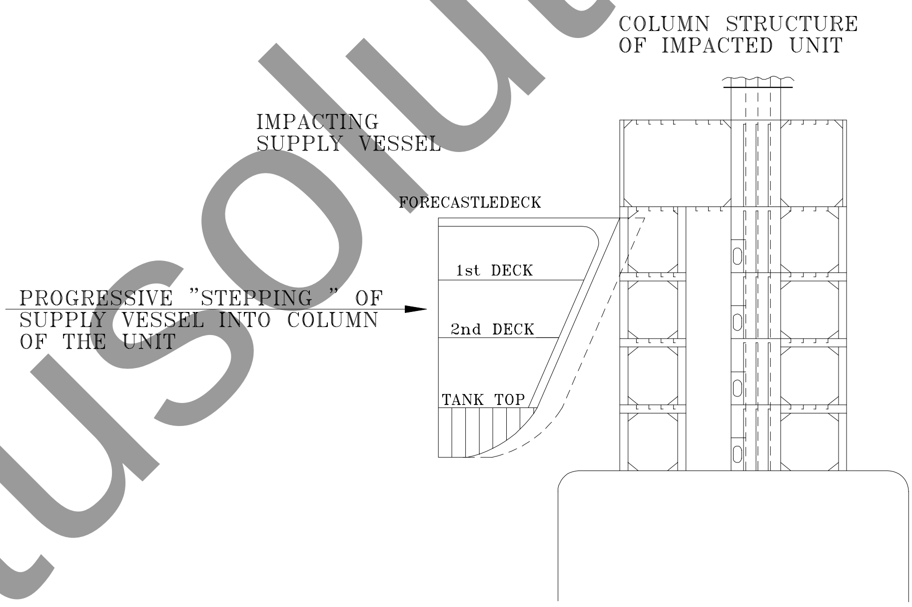

Figure M.6-1 Illustration of the bow profile of a supply vessel being 'stepped' into the structure of a column stabilised unit.

- By considering the local geometry of the supply vessel and the impacted structure, relevant force-deformation curves for the column stabilised unit may be produced.

- For a given action level the area under the force-deformation curves represents the absorption of energy. The distribution and extent of the damage results from the condition of equal collision force acting on the structures, and that the sum of the absorbed energies equals the portion of the impact energy dissipated as strain energy, i.e.

$$E_s + E_u = \int_0^{\Delta s} P_s(\delta_s)d\delta_s + \int_0^{\Delta u} P_u(\delta_u)d\delta_u \qquad \textbf{(M.6.1)}$$

Where :

P_s, P_u he force in the impacting vessel and the impacted unit respectively, and,

δ_s, δ_u he deformations in the impacting vessel and the impacted unit respectively.

This procedure is illustrated in Figure M.6-2.

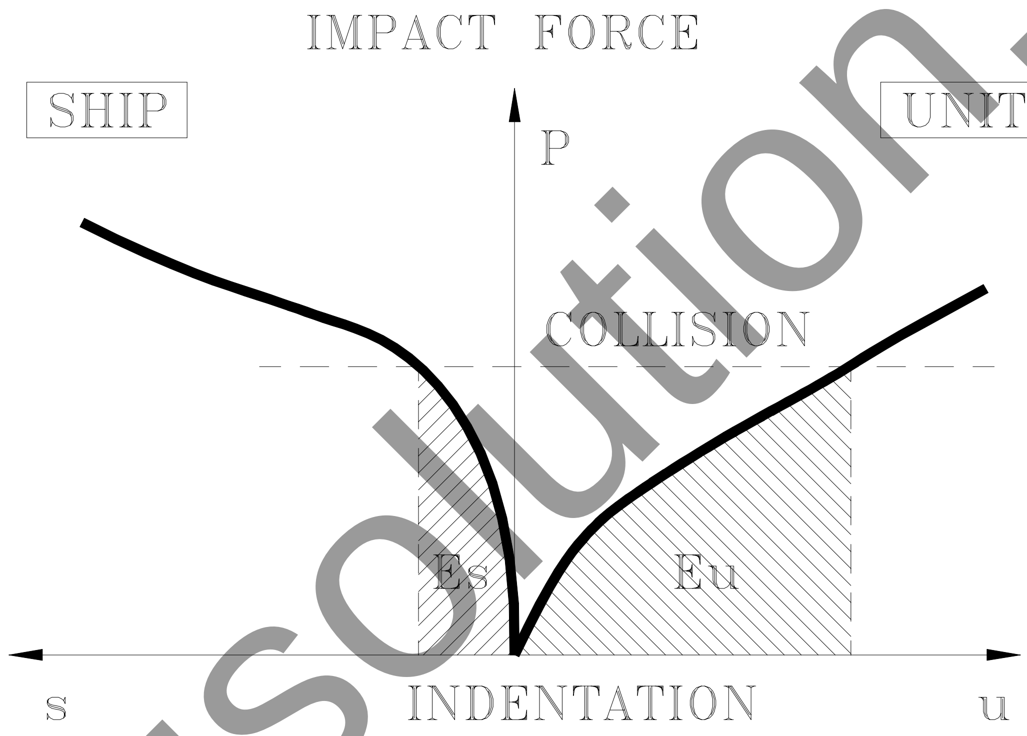

Figure M.6-2 Dissipation of strain energy global structural integrity after collision

Having evaluated the extent of local damage incurred by the relevant collision event (as described above) an assessment of the resulting, global structural capacity (considering environmental actions) shall be undertaken. In such an evaluation the following listed items are relevant :

- In cases where the impact damage is limited to local damage to the column particular consideration should be given to column/deckbox interfaces, and the damaged (impacted) section. (For typical column structures a simplified approach to assess the reduced capacity of the structure in way of the damaged location would be to assume that all the impacted (deformed) area is ineffective.)
- When counter-flooding is utilised as a means of righting the unit in an accidental event the actions resulting from such counter flooding shall be evaluated.

- In cases where the impact damage is limited to local damage to a single brace, redundancy requirements given in M.9.1 should adequately cover the required structural evaluation.
- NMD requirements to watertight boundary, structural strength in the damaged condition, see M.6.8, (including inclined angles resulting from requirements to reserve buoyancy) should be satisfied.
- In order to avoid risk assessment considerations in respect to implications of structural failure of topside structures in the inclined condition, (e.g. in respect to the possibility of progressive collapse in the event of structural failure) it is normal practice to consider the structural capacity of topside structural arrangements in the damaged condition.

Capacity exceedances may be accepted for local areas provided that adequate account is given to the redistribution of forces.

Due to the large heel angles in the damaged condition, the in-plane force component of the deckbox mass may be considerable. Normally part of the deck will be submerged and counteract this force. The most critical condition is therefore generally the heel angle corresponding to a waterplane just below the deckbox corner.

M.6.6 Unintended flooding

Considerations in respect to unintended flooding are generally system and stability design considerations rather than structural design. Requirements with respect to intact and reserve buoyancy conditions are normally considered to adequately cover any structural strength requirements (see M.6.8) in respect to unintended flooding.

M.6.7 Abnormal wave events

Air gap considerations with respect to evaluation of the abnormal wave events are given in M.7. Should any part of the structure receive wave impact actions in the abnormal wave condition the consequences of such wave impacts shall be evaluated.

M.6.8 Reserve buoyancy

Structural strength of watertight boundaries (both internal and external watertight boundaries, including all stiffeners and girders supporting the plate fields) shall comply with the requirements stated in NMD regulations, /1/.

M.7 AIR GAP

M.7.1 General

Requirements and guidance to air gap analyses for a column stabilised unit are given in NORSOK N-003, Section 9.2.5.4. Both ULS (10^{-2}) and ALS (10^{-4}) conditions shall be evaluated.

In the ULS condition, positive air gap should be ensured. However, wave impact may be permitted to occur provided that it can be demonstrated that such actions are adequately accounted for in the design and that safety to personnel is not significantly impaired. (See Commentary for more details).

Analysis undertaken to document air gap should be calibrated against relevant model test results. Such analysis shall include relevant account of :

- wave/structure interaction effects,
- wave asymmetry effects,
- global rigid body motions (including dynamic effects),
- effects of interacting systems (e.g. mooring and riser systems), and,
- maximum/minimum draughts.

Column ‘run-up’ action effects shall be accounted for in the design of the structural arrangement in way of the column/deckbox connection. These 'run-up' actions are to be treated as an environmental load component, however, they need not normally be considered as occurring simultaneously with other environmental responses. (For further considerations in respect to run-up effects reference should be made to NORSOK N-003).

Evaluation of air gap adequacy shall include consideration of all affected structural items including lifeboat platforms, riser balconies, overhanging deck modulus etc.

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M.8 COMPARTMENTATION & STABILITY

M.8.1 General

In the assessment of compartmentation and stability of a column stabilised unit, consideration shall be given to all relevant detrimental effects, including those resulting from :

- environmental actions,
- relevant damage scenarios,
- rigid body motions,
- the effects of free-surface, and,
- boundary interactions (e.g. mooring and riser systems).

The mass distribution of a column stabilised unit, including all associated permanent and variable actions, shall be determined to an appropriate accuracy by a relevant procedure. An inclining test should be conducted when construction is as near to completion as practical in order to accurately determine the unit's mass and position of the centre of gravity. Changes in load conditions after the inclining test should be carefully accounted for.

Monitoring of the mass and centre of gravity shall be performed during the entire life cycle of the unit. A procedure for control of the mass and position of the centre of gravity of the unit shall be incorporated into the design.

Note : Special provisions relating to the Norwegian activities

1. Special provisions relating to subdivision, stability and freeboard for units engaged in Norwegian petroleum activities are given in NORSOK N-001, Section 7.10.

M.8.2 Compartmentation and watertight integrity

Compartmentation, and watertight and weathertight integrity of a column stabilised unit shall satisfy the general requirements as stated in ISO 13819-1. Detailed provisions stated in IMO MODU Code, /2/, should also be satisfied.

The number of openings in watertight structural boundaries shall be kept to a minimum. Where penetrations are necessary for access, piping, venting, cables etc., arrangements shall be made to ensure that the watertight integrity of the structure is maintained.

Where individual lines, ducts or piping systems serve more than one watertight compartment, or are within the extent of damage resulting from a relevant accidental event, arrangements shall be provided to ensure that progressive flooding will not occur.

M.8.3 Stability

Stability of a column stabilised unit in the operational phase, shall satisfy the requirements of the relevant provisions stated in the IMO MODU Code, /2/.

Adequacy of stability shall be established for all relevant operational and temporary phase conditions. The assessment of stability shall include consideration of both the intact and damaged conditions. It shall be ensured that the assumed basis for the damage stability design criteria is compatible with accidental events identified as being relevant for the structure (see M.6 and M.8.4).

M.8.4 Damaged condition

The dimensioning extent of damage and loss of buoyancy shall be based on a risk analysis, see M.6.1. If such risk analysis shows that the greatest relevant accidental event with regard to collision is a drifting vessel with a displacement that does not exceed 5000 tonnes, the extent of damage indicated in the Norwegian Maritime Directorates, Regs. for Mobile Offshore Units, 1991, /1/, may be utilised. It is not necessary however to comply with the requirements in section 22, /1/, with regard to reserve buoyancy if the risk analysis demonstrates that there is no need for such reserve buoyancy.

In order to ensure adequacy of ballast systems, both with respect to unintentional flooding, (see M.6.6) and capacity in the damage condition, (see M.6.5), the requirements to ballast systems, stated in the Norwegian Maritime Directorate, Regs. concerning ballast systems on mobile offshore units, /3/, Sections 11 and 12 should be satisfied.

M.9 SPECIAL CONSIDERATIONS

M.9.1 Structural redundancy

Structural robustness shall, when considered necessary, be demonstrated by appropriate analysis. Slender, main load bearing, structural elements shall normally be demonstrated to be redundant in the accidental limit state condition.

Considerations with respect to redundancy of bracing systems are given in M.9.2.

M.9.2 Brace arrangements

For bracing systems (see Commentary M.9.1) the following listed considerations shall apply :

- Brace structural arrangements shall be investigated for relevant combinations of global and local actions.
- Structural redundancy of slender bracing systems (see M.9.1) shall normally include brace node redundancy (i.e. all bracings entering the node), in addition to individual brace element redundancy.
- Brace end connections (e.g. brace/column connections) shall normally be designed such that the brace element itself will fail before the end connection.
- Underwater braces should be watertight and have a leakage detection system to make it possible to detect cracking at an early stage.
- When relevant (e.g. in the self-floating, transit condition) the effect of slamming on braces shall be considered.

M.9.3 Structure in way of a fixed mooring system

Local structure in way of fairleads, winches, etc. forming part of the position mooring system is, as a minimum, to be capable of withstanding forces equivalent to 1.25 times the breaking strength of the individual mooring line. The strength evaluation should be undertaken utilising the most unfavourable operational direction of the anchor line. In the evaluation of the most unfavourable direction, account shall be taken of relative angular motion of the unit in addition to possible line lead directions.

M.9.4 Structural detailing

In the design phase particular attention should be given to structural detailing, and requirements to reinforcement in areas that may be subjected to high local stresses, for example :

- Design Class 3 (DC3) connections (see M.2.1),
- locations that may be subjected to wave impact (including column run-up actions),
- locations in way of mooring arrangements, and,
- locations that may be subjected to accidental, or operational, damage.

In way of DC3 connections, continuity of strength is normally to be maintained through joints with the axial stiffening members and shear web plates being made continuous. Particular attention should be given to weld detailing and geometric form at the point of the intersections of the continuous plate fields with the intersecting structure.

M.10 DOCUMENTATION REQUIREMENTS FOR THE 'DESIGN BASIS' & 'DESIGN BRIEF'

M.10.1 General

Adequate planning shall be undertaken in the initial stages of the design process in order to obtain a workable and economic structural solution to perform the desired function. As an integral part of this planing, documentation shall be produced identifying design criteria and describing procedures to be adopted in the structural design of the unit.

Applicable codes, standards and regulations should be identified at the commencement of the design.

A summary document containing all relevant data from the design and fabrication phase shall be produced.

Design documentation (see below) shall, as far as practicable, be concise, non-voluminous, and, should include all relevant information for all relevant phases of the lifetime of the unit.

General requirements to documentation relevant for structural design are given in NORSOK, N-001, Section 5.

M.10.2 Design basis

A Design Basis Document shall be created in the initial stages of the design process to document the basis criteria to be applied in the structural design of the unit.

A summary of those items normally to be included in the Design Basis document is included below.

Unit description and main dimensions

A summary description of the unit, including :

- general description of the unit (including main dimensions and draughts),
- main drawings including :
  - main structure drawings (This information may not be available in the initial stage of the design),
  - general arrangement drawings,
  - plan of structural categorisation,
  - load plan(s), showing description of deck uniform (laydown) actions,
  - capacity (tank) plan,
- service life of unit,
- position keeping system description.

Rules, regulations and codes

A list of all relevant, applicable, Rules, Regulations and codes (including revisions and dates), to be utilised in the design process.

Environmental design criteria

Environmental design criteria (including all relevant parameters) for all relevant conditions, including :

- wind, wave, current, snow and ice description for 10^{-1} ,10^{-2} and 10^{-4} annual probability events,
- design temperatures.

Stability and compartmentation

Stability and compartmentation design criteria for all relevant conditions including :

- external and internal watertight integrity plan,
- lightweight breakdown report,
- design loadcase(s) including global mass distribution,
- damage condition waterlines.(This information may not be available in the initial stage of the design.)

Temporary phases

Design criteria for all relevant temporary phase conditions including, as relevant :

- limiting permanent, variable, environmental and deformation action criteria,
- procedures associated with construction, (including major lifting operations),
- essential design parameters associated with temporary phases (e.g. for mating loadcases, mating weld-up sequences, crushing tube stiffness', for transit phases, transit speed etc.),
- relevant ALS criteria.

Operational design criteria

Design criteria for all relevant operational phase conditions including :

- limiting permanent, variable, environmental and deformation action criteria,
- designing accidental event criteria (e.g. collision criteria),
- tank loading criteria (all tanks) including a description of system, with :
  - loading arrangements
  - height of air pipes,
  - loading dynamics,
  - densities,
- mooring actions.

Air gap

Relevant basis information necessary for the assessment of air gap sufficiency, including:

- a description of the requirements to be applied in the ULS and ALS conditions,
- basis model test report (This information may not be available in the initial stage of the design).

In-service inspection criteria

A description of the in-service inspection criteria and general philosophy (as relevant for evaluating fatigue allowable cumulative damage ratios).

Miscellaneous

A general description of other essential design information, including :

- description of corrosion allowances, where applicable.

M.10.3 Design brief

A Design Brief Document shall be created in the initial stages of the design process. The purpose of the Design Brief shall be to document the intended procedures to be adopted in the structural design of the unit. All applicable limit states for all relevant temporary and operational design conditions shall be considered in the Design Brief.

A summary of those items normally to be included in the Design Brief Document is included below.

Analysis models

A general description of models to be utilised, including description of :

- global analysis model(s),
- local analysis model(s),
- loadcases to be analysed.

Analysis procedures

A general description of analytical procedures to be utilised including description of procedures to be adopted in respect to :

- the evaluation of temporary conditions,
- the consideration of accidental events,
- the evaluation of fatigue actions,
- air gap evaluation, (including locations to be considered, damping, inclusion of asymmetry factors, disturbed (radiated) wave considerations, combined motion response).
- the establishment of dynamic responses (including methodology, factors, and relevant parameters),
- the inclusion of 'built-in' stresses,
- the consideration of local responses (e.g. those resulting from mooring and riser actions, ballast distribution in pontoon tanks etc.)
- the consideration of structural redundancy.

Structural evaluation

A general description of the evaluation process, including :

- description of procedures to be utilised for considering global and local responses,
- description of fatigue evaluation procedures (including use of design fatigue factors, SN-curves, basis for stress concentration factors (SCF's), etc.).
- description of procedures to be utilised for code checking.

Air gap evaluation

A general description of the air gap evaluation procedure, including :

- description of procedures to be utilised for considering air gap sufficiency.

M.11 REFERENCES

/1/ Regulations of 20 Dec. 1991, No.878, concerning stability, watertight subdivision and watertight/weathertight closing means on mobile offshore units, Norwegian Maritime Directorate.

/2/ Code for the Construction and Equipment of Mobile Offshore Drilling Units, 1989, (1989 MODU CODE), International Maritime Organisation, London 1990.

/3/ Regulations of 20 Dec. 1991, No.879, concerning ballast systems on mobile offshore units, Norwegian Maritime Directorate.

/4/ 'Offshore Installations: Guidance on design, construction and certification', U.K., Health & Safety Executive, Fourth Edition, HMSO.

M.12 COMMENTARY

Comm. M.1.1 General

The content of this Annex is applicable to column stabilised units located at a single site over a prolonged period of time. Normally such units will be engaged in production activities. There may therefore be certain requirements and guidance given within this Annex that are not considered as being fully appropriate for 'mobile', column stabilised units, e.g. engaged in exploration drilling activities at a location for only a limited period of time.

Comm. M.3.3 Variable actions

Applicable variable actions acting on deck areas are stated in Table 5.1 in NORSOK, N-003. This table provides appropriate action factors for local, primary and global design. For floating units, full application of the global design action factors will lead to a load condition that, in practice, would not be possible to achieve as the total static mass is always required to be in balance with the buoyancy actions. Additionally requirements to stability and compartmentation will further restrict the practical distribution of variable actions.

In the assessment of global structural response the variable action design factors stated in Table 5.1 are not intended to limit the variable load carrying capacity of the unit in respect to non-relevant, variable load combinations. (Full application of the global design factors, stated in Table 5.1, to all variable actions would normally imply such a restriction.) Global design load conditions should therefore be established based upon 'worst case', representative variable load combinations, where the limiting global mass distribution criteria is established taking into account compliance with the requirements to intact and damage hydrostatic and hydrodynamic stability.

These limit global load conditions shall be fully documented in the design basis, see M.10.2.

Comm. M.4.2 Global models

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

- Single model solutions do not normally contain sufficient structural detailing. (For ULS structural assessment, response down to the level of the stress in plate fields between stiffeners is normally required). Examples of insufficient structural detailing may be :
  - internal structure is not modelled in sufficient detail to establish internal structural response to the degree of accuracy required,
  - element type, shape or fineness (e.g. mesh size) is insufficient.
- Single model solutions do not normally account for the full range of internal and external pressure combinations. Examples of effects that may typically not be fully accounted for include:
  - internal tank pressure up to the maximum design pressure,
  - maximum external pressures (e.g. if a 'design wave' analytical approach has been adopted, the maximum external pressure height is that height resulting from the design wave, which is not normally the maximum external pressure that the section may be subjected to),

- the full extent of internal and external pressure combinations,
- variations in tank loadings across the section of the pontoon (e.g. if the pontoon is sub-divided into a number of watertight compartments across its section),
- load conditions that may not be covered by the global analysis (e.g. damage, inclined conditions).

- Single model solutions do not normally account for the full range of 'global' tank loading conditions. Examples of global tank loading conditions that may typically not be fully accounted for include :
  - tank loading distributions along the length of the pontoon,
  - asymmetric tank loadings from one pontoon as compared to another.
- Single model solutions may not fully account for all action effects. Examples of load effects that may typically not be fully accounted for include :
  - viscous effects (drag loading) on slender members,
  - riser interface actions,
  - thruster actions.

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

Comm. M.5.2 Design fatigue factors

If significant adjustment in draught is possible in order to provide for satisfactory accessibility in respect to inspection, maintenance and repair, a sufficient margin in respect to the minimum inspection draught should be considered when deciding upon the appropriate Design Fatigue Factors. As a minimum this margin is to be at least 1 metre however it is recommended that a larger value be considered especially in the early design stages where sufficient reserve should be allowed for to account for design changes in the mass and the centre of mass of the unit. Consideration should further be given to operational requirements that may limit the possibility for ballasting/deballasting operations.

When considering utilisation of Remotely Operated Vehicle (ROV) inspection consideration should be given to the limitations imposed on such inspection by the action of water particle motion (e.g. waves). The practicality of such a consideration may be that effective underwater inspection by ROV, in normal sea conditions, may not be achievable unless the inspection depth is at least 10m below the sea surface.

Comm. M.6.5 Collision

In the damaged, inclined condition (i.e. after the collision event) the structure shall continue to resist the defined environmental conditions (i.e. 10^{-2} annual probability of exceedance criteria). In practice, it is not normally considered practicable to analyse this damaged, inclined condition as the deckbox structure becomes buoyant (due both to the static angle of inclination in the damaged condition and also due to rigid body motion of the unit itself). The global system of loading and response becomes extremely non-linear. Even with exhaustive model testing it would be difficult to ensure that all relevant responses are measured, for all designing sea-state conditions, and directions, for all relevant damage waterlines.

Further, it is considered to be extremely unlikely that a 10^{-2} environmental load event would occur before corrective action had taken place (e.g. righting of the unit to an even keel). NMD requirements to ballast systems for column stabilised units, /3/, (and required in M.8.4) include requirements that the ballast system shall be capable of restoring the unit to an upright position, and an acceptable draught as regards strength, within three hours after the collision damage event. On the assumption that the 5000 tonne, 2m/s, collision criteria is based upon a free drifting supply vessel with a drift velocity equal to $H_s/2$ (where H_s = significant wave height), within a 3 hour period, a 4m sign. wave height may be expected to increase to max. 6.5m sign. wave height. Hence, disregarding the damaged structural element itself, the ULS loadcase b. may generally be considered as providing more demanding design criteria than the ALS condition with 100-year load event. Additionally, in respect to global response, as soon as the deckbox starts to become buoyant the global actions resulting from the inclined deckbox mass rapidly become reduced.

With background in the above logic simplified 'engineering' solutions to structural design for the collision event, that do not explicitly require analyses involving 100 year environmental actions in the inclined (heeled) condition, are normally considered as being acceptable.

Comm. M.7.1 General

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

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

In the context of M.7, column run-up actions are not considered as resulting in negative air-gap responses.

Comm. M.9.1 Structural redundancy

The requirement concerning structural redundancy of a slender structural element is included within this Annex for the following listed reasons :

- In order to maintain compatibility with the requirements to column stabilised stated by the International Association of Classification Societies.
- In order to ensure that the structure exhibits ductility after first failure.
- In order to ensure that a single failure does not lead to a critical condition. (A critical condition may for example be a fatigue failure that has not been accounted for in design calculations due to the fact that, in the process of fabrication, details built into the structure do not exactly reflect those details that considered in the assessment of the design.)

Typically a member is to be considered as being slender if the reduced (relative) slenderness, $\bar{\lambda}$, is greater than 0.2, Section 5.4.

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DESIGN OF STEEL STRUCTURES

ANNEX N
SPECIAL DESIGN PROVISIONS FOR TENSION LEG PLATFORMS

CONTENTS

N.1 INTRODUCTION

N.1.1 General

This Annex provides requirements and guidance to the structural design of tension leg platforms, constructed in steel, in accordance with the provisions of this NORSOK standard.

The requirements and guidance documented in this Annex are generally applicable to all configurations of tension leg platforms.

For novel designs, or unproved applications of designs where limited or no direct experience exists, relevant analyses and model testing, shall be performed which clearly demonstrate that an acceptable level of safety is obtained which is not inferior to the safety level set forth by this Annex when applied to traditional designs.

N.1.2 Definitions

Terms

A Tension Leg Platform (TLP) is defined as a buoyant installation connected to a fixed foundation by pretensioned tendons. The tendons are normally parallel, near vertical elements, acting in tension, which restrain the motions of the platform in heavy, pitch and roll. The platform is compliant in surge, sway and yaw.

The TLP tendon system comprises all components associated with the mooring system between, and including the top connection(s) to the hull and the bottom connection(s) to the foundation(s). Guidelines, control lines, umbilicals etc. for tendon service and/or installation are considered to be included as part of the tendon system.

The TLP foundation is defined as those installations at, or in, the seafloor which serve as anchoring of the tendons and provides transfer of tendon actions to the foundation soil.

The TLP hull consists of buoyant columns, pontoons and intermediate structural bracings, as applicable.

The TLP deck structure is the structural arrangement provided for supporting the topside equipment. Normally, the deck serves the purpose of being the major structural components to ensure pontoons, columns and deck acting as one structural unit to resist environmental and gravity actions.

Ringing is defined as the high frequency resonant response induced by transient loads from high, steep waves.

Springing is defined as the high frequency resonant response induced by cyclic (steady state) in low to moderate seastates.

High Frequency (HF) responses are defined as TLP rigid body motions at, or near heave, roll and pitch eigenperiods.

Low Frequency (LF) responses is defined as TLP rigid body motions at, or near surge, sway and yaw eigenperiods.

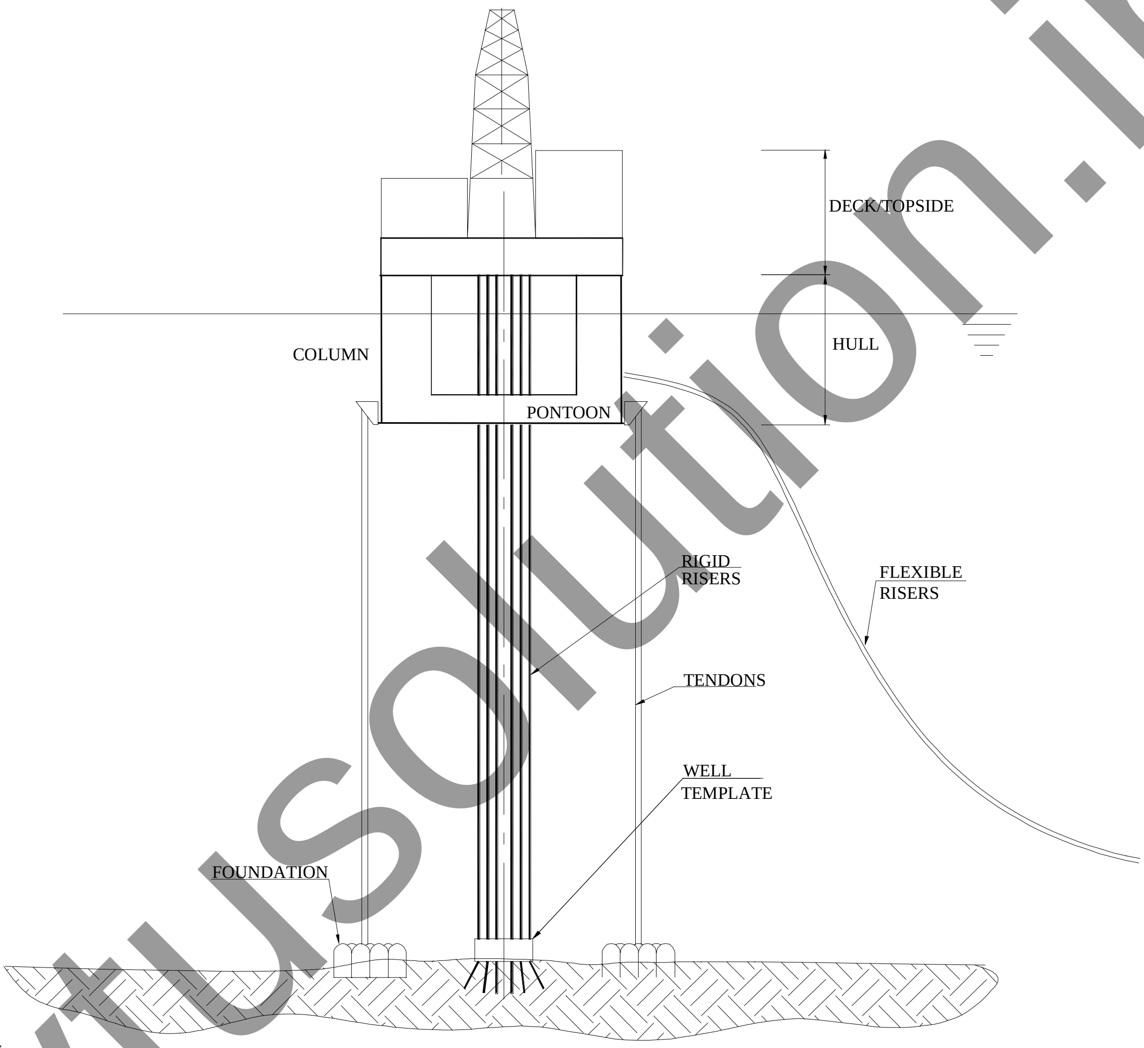


Figure N.1-1 Typical tension leg platform

N.1.3 Description of tendon system

Individual tendons are considered within this Annex as being composed of three major parts:

- Interface at the platform
- Interface at the foundation (seafloor)
- Link between platform and foundation

Tendon components at the platform interface shall adequately perform the following main functions:

- Apply, monitor and adjust a prescribed level of tension to the tendon
- Connect the tensioned tendon to the platform
- Transfer side actions and absorb bending moments or rotations of the tendon

Tendon components providing the link between the platform and the foundation consist of tendon elements (tubulars, solid rods etc.), termination at the platform interface and at the foundation interface, and intermediate connections of couplings along the length as required. The intermediate connections may take the form of mechanical couplings (threads, clamps, bolted flanges etc.), welded joints or other types of connections.

Tendon components at the foundation interface shall adequately perform the following main functions:

- Provide the structural connection between the tendon and the foundation
- Transfer side actions and absorb bending moments or rotations of the tendon

The tendon design may incorporate specialized components, such as:

- corrosion-protection system components,
- buoyancy devices,
- sensors and other types of instrumentation for monitoring the performance and condition of the tendons,
- auxiliary lines, umbilicals etc. for tendon service requirements and/or for functions not related to the tendons,
- provisions for tendons to be used as guidance structure for running other tendons or various types of equipment, and
- elastomeric elements.

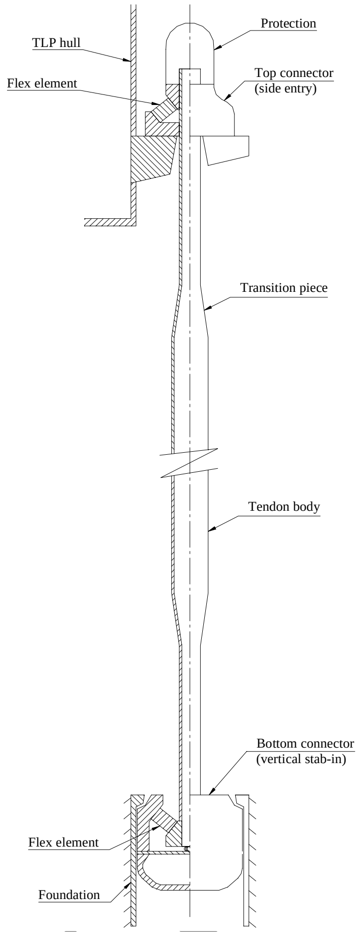


Figure N.1-2 Typical TLP tendon system

N.1.4 Non-operational phases

The structure shall be designed to resist relevant actions associated with conditions that may occur during all stages of the life-cycle of the unit. Such stages may include :

- fabrication,
- site moves,
- mating,
- sea transportation,
- installation, and,
- decommissioning.

Structural design covering marine operation, construction sequences shall be undertaken in accordance with NORSOK N-001.

Marine operations should be undertaken in accordance with the requirements stated in NORSOK J-003.

All marine operations shall, as far as practicable, be based upon well proven principles, techniques, systems and equipment and shall be undertaken by qualified, competent personnel possessing relevant experience.

Structural responses resulting from one temporary phase condition (e.g. a construction or transportation operation) that may effect design criteria in another phase shall be clearly documented and considered in all relevant design workings.

Fabrication

Planning of construction sequences and of the methods of construction shall be performed. Actions occurring in fabrication phases shall be assessed and, when necessary the structure and the structural support arrangement shall be evaluated for structural adequacy.

Major lifting operations shall be evaluated to ensure that deformations are within acceptable levels, and that relevant strength criteria are satisfied.

Mating

All relevant action effects incurred during mating operations shall be considered in the design process. Particular attention should be given to hydrostatic actions imposed during mating sequences.

Sea transportation

A detailed transportation assessment shall be undertaken which includes determination of the limiting environmental criteria, evaluation of intact and damage stability characteristics, motion response of the global system and the resulting, induced actions. The occurrence of slamming actions on the structure and the effects of fatigue during transport phases shall be evaluated when relevant.

In case of transportation (surface/sub surface) of tendons, this operation shall be carefully planned and analysed. Model testing shall be considered.

Satisfactory compartmentation and stability during all floating operations shall be ensured.

All aspects of the transportation, including planning and procedures, preparations, seafastenings and marine operations should comply with the requirements of the Warranty Authority.

Installation

Installation procedures of foundations (e.g. piles, suction anchor or gravity based structures) shall consider relevant static and dynamic actions, including consideration of the maximum environmental conditions expected for the operations.

For novel installation activities (foundations and tendons), relevant model testing should be considered.

Tendon stand-off (pending TLP installation) phases shall be considered with respect to actions and responses.

The actions induced by the marine spread mooring involved in the operations and the forces exerted on the structures utilised in positioning the unit, such as fairleads and padeyes, shall be considered for local strength checks.

Decommissioning

Abandonment of the unit shall be planned for in the design stage.

N.2 STRUCTURAL CLASSIFICATION AND MATERIAL SELECTION

N.2.1 General

Selection of steel quality, and requirements for inspection of welds, shall be based on a systematic classification of welded joints according to the structural significance and the complexity of the joints/connections as documented in Chapter 5.

In addition to in-service operational phases, consideration shall be given to structural members and details utilised for temporary conditions, e.g. fabrication, lifting arrangements, towing and installation arrangements, etc.

N.2.2 Structural classification

The structural classification guidance given below assumes that the tendon system is demonstrated to have residual strength and that the TLP structural system satisfies the requirements of the ALS condition with failure of the tendon (or a connection in the tendon) as the defined damage. (See also NORSOK N-004, Table 5-1). If this is not the case then design classes, DC3 and DC4 stated below should be substituted by design classes DC1 and DC2 and the inspection categories respectively upgraded in accordance with the requirements of NORSOK N-004, Tables 5-3 and 5-4 as applicable.

Examples of typical design classes applicable to the hull and deck structures of a TLP are given in Annex M.

Examples of typical design classes applicable to the pile foundation structures are given in Annex K.

Examples of considerations with respect to structural classification of tendons, tendon interfaces are given below. These examples provide minimum requirements and are not intended to restrict the designer in applying more stringent requirements should such requirements be desirable.

Typical locations - Design class : DC3

Locations in way of complex structural connections should be classified as DC3, such connections may occur at :

- tendon interfaces with the foundation and the TLP hull, and,
- complex tendon / tendon connections.

DC3 areas may be limited to local, highly stressed areas if the stress gradient at such connections is large.

Typical locations - Design class : DC4

Except as provided for in the description for DC3 structural categorisation, the following listed connections may appropriately be classified as being DC4 :

- simple tendon / tendon connections,
- interface arrangements outside locations of complex connections including general stiffened plate fields (e.g. at hull interface)

Consideration of the economic consequence of failure may however indicate the utilisation of a higher design class than the minimum class as given above.

Typical locations - Design class : DC5

DC5 locations are not normally relevant for tendons or tendon interfaces.

N.2.3 Material selection

Material specifications shall be established for all structural materials utilised in a TLP. Such materials shall be suitable for their intended purpose and have adequate properties in all relevant design conditions. Material selection shall be undertaken in accordance with the principles given in NORSOK M-DP-001.

When considering criteria appropriate to material grade selection, adequate consideration shall be given to all relevant phases in the life cycle of the unit. In this connection there may be conditions and criteria, other than those from the in-service, operational phase, that provide the design requirements in respect to the selection of material. (Such criteria may, for example, be design temperature and/or stress levels during marine operations.)

Selection of steel quality for structural components shall normally be based on the most stringent Design Class of the joints involving the component.

Through-thickness stresses and low temperature toughness requirements shall be assessed.

The evaluation of structural resistance shall include relevant account of variations in material properties for the selected material grade (e.g. Variation in yield stress as a function of thickness of the base material)

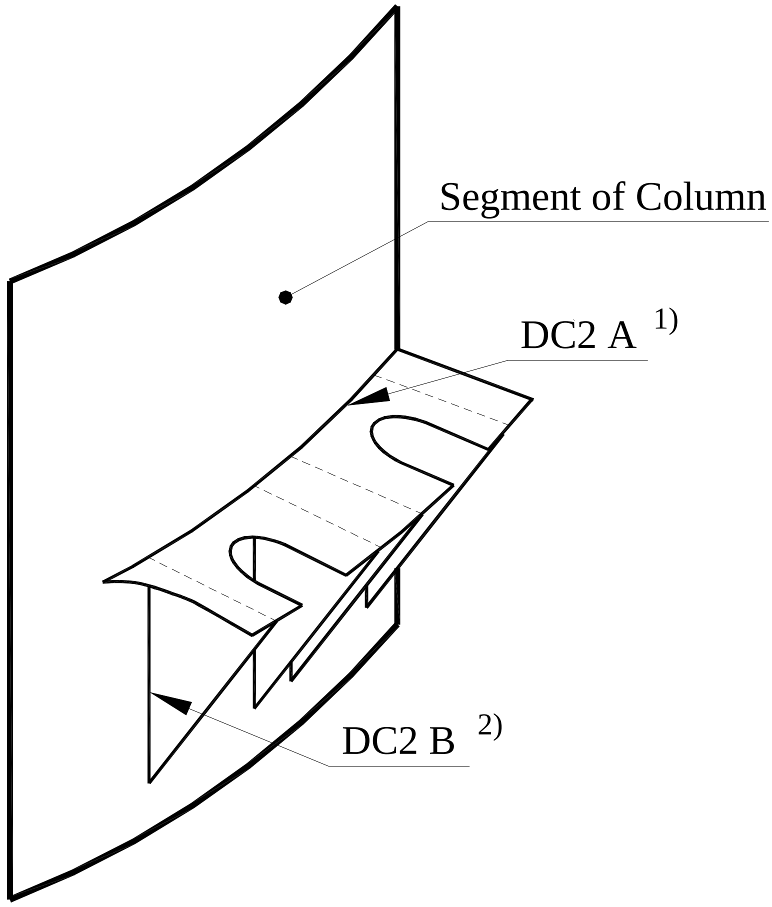
N.2.4 Inspection categories

Welding, and the extent of non-destructive examination during fabrication, shall be in accordance with the requirements stipulated for the appropriate 'inspection category' as defined in NORSOK, M-101. Determination of inspection category should be in accordance with the criteria given in Table 5-5.

The lower the extent of NDT selected, the more important is the representativeness of the NDT selected and performed. The designer should therefore exercise good engineering judgement in indicating mandatory locations for NDT, where variation of utility along welds is significant.

N.2.5 Guidance to minimum requirements

Figures N.2-1 and N.2-2 illustrate minimum requirements to selection of Design Class, and Inspection Category for typical TLP unit, structural configurations.



Figue N.2-1 Examples of typical hull porch design classes and inspection categories

1. *Inspection Category A is selected in accordance with Table 5.4 assuming high fatigue utilisation and principal stresses in the transverse direction.*
2. *Inspection Category B is selected in accordance with Table 5.4 assuming high fatigue and principal stresses longitudinal to the weld.*

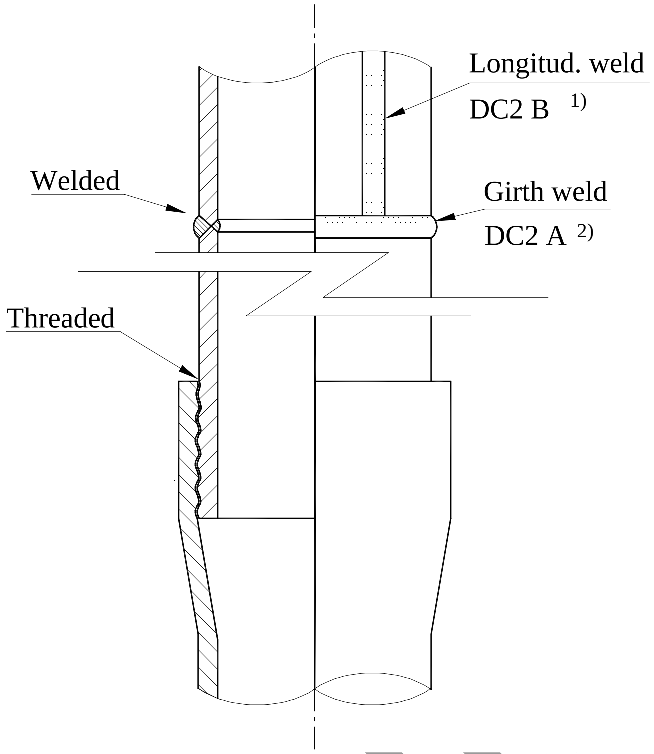


Figure N.2-2 Examples of design classes and inspection categories for typical tendon connections

1. *Inspection Category B is selected in accordance with Table 5.4 assuming high fatigue utilisation and principal stresses in the longitudinal direction.*
2. *Inspection Category A is selected in accordance with Table 5.4 assuming high fatigue and principal stresses transverse to the weld. Stricter acceptance criteria as per footnote 2 in Table 5.4 will apply.*

N.3 DESIGN CRITERIA

N.3.1 General

The following basic design criteria shall be complied with for the TLP design:

- The TLP is to be able to sustain all actions liable to occur during all relevant temporary and operating design conditions for all applicable limit states.
- Direct wave actions on the deck structure should not occur in the ultimate limit-state (ULS). Direct wave actions on the deck structure may be accepted in the ALS condition provided that such actions are adequately included in the design.

Operating tolerances shall be specified and shall be achievable in practice. Normally, the most unfavourable operating tolerances shall be included in the design. Active operation shall not to be dependent on high reliability of operating personnel in an emergency situation.

Note: *Active operation of the following may be considered in an emergency situation, as applicable:*

- *Ballast distribution*
- *Weight distribution*
- *Tendon tension*
- *Riser tension*

N.3.2 Design principles, tendons

Essential components of the tendon system shall be designed by the principle that, as far as practicable, they are to be capable of being inspected, maintained, repaired and/or replaced.

Tendon mechanical components shall, as far as practicable, be designed “fail to safe”. Consideration is to be given in the design to possible early detection of failure for essential components which cannot be designed according to this principle.

Certain vital tendon components may, due to their specialised and unproven function, require extensive engineering and prototype testing to determine:

- Confirmation of anticipated design performance
- Fatigue characteristics
- Fracture characteristics
- Corrosion characteristics
- Mechanical characteristics

The tendon system and the securing/supporting arrangements shall be designed in such a manner that a possible failure of one tendon is not to cause progressive tendon failure or excessive damage to the securing/supporting arrangement at the platform or at the foundation.

A fracture control strategy should be adopted to ensure consistency of design, fabrication and in service monitoring assumptions. The object of such a strategy is to ensure that the largest undetected flaw from fabrication of the tendons will not grow to a size that could induce failure within the design life of the tendon, or within the planned in-service inspection interval, within a reasonable level of reliability. Elements of this strategy include:

- Design fatigue life
- Fracture toughness
- Reliability of inspection during fabrication

- In-service inspection intervals and methods

Fracture mechanics should be used to define allowable flaw sizes, estimate crack growth rates and thus help define inspection intervals and monitoring strategies.

All materials liable to corrode shall be protected against corrosion. Special attention should be given to:

- Local complex geometries
- Areas that are difficult to inspect/repair
- Consequences of corrosion damage
- Possibilities for electrolytic corrosion

All sliding surfaces shall be designed with sufficient additional thickness against wear. Special attention should be given to the following:

- Cross-load bearings
- Seals
- Balljoints

Satisfactory considerations shall be given to settlement/subsidence which may be a significant factor in determining tendon-tension adjustment requirements.

N.4 ACTIONS

N.4.1 General

Characteristic actions are to be used as reference actions. Design actions are, in general, defined in NORSOK N-003. Guidance concerning action categories relevant for TLP designs are given in the following.

N.4.2 Different actions

All relevant actions that may influence the safety of the structure or its parts from commencement of construction to permanent decommissioning should be considered in design. The different actions are defined in NORSOK N-003. For the deck and hull of the TLP, the actions are similar to those also described in Annex M. Additional actions on the hull are those from ringing and springing.

The wave actions on the tendons can be described as recommended in NORSOK N-003 for slender structures with significant motions.

If of importance, the disturbance from hull and risers on the wave kinematics at the tendon locations should be accounted for.

The earthquake actions at the foundation of the tendons are described according to NORSOK N-003.

The following actions should be considered:

- Permanent Actions
- Variable Actions
- Deformation Actions
- Accidental Actions
- Environmental Actions

The environmental actions are to be summarized as:

- Wind Actions
  - Mean wind
  - Dynamic wind
  - Local wind

- Wave and current Actions
  - Actions on slender members
  - Actions induced by TLP motions
  - Slamming and shock pressure
  - Wave diffraction and radiation
  - Mean drift forces
  - Higher order non-linear wave actions (slowly varying, ringing and springing)
  - Wave enhancement
  - Vortex shedding effects

- Marine growth
- Snow and ice accumulation
- Direct ice action (icebergs and icefloes)
- Earthquake
- Temperature
- Tidal effects

N.5 GLOBAL PERFORMANCE

N.5.1 General

The selected methods of response analysis are dependent on the design conditions, dynamic characteristics, non-linearities in action and response and the required accuracy in the actual design phase. For a detailed discussion of the different applicable methods for global analysis of tension leg platforms, reference is made to API RP 2T, /5/.

The selected methods of analysis and models employed in the analysis shall include relevant non-linearities and motion-coupling effects. The approximations, simplifications and/or assumptions made in the analysis shall be justified, and their possible effects shall be quantified e.g. by means of simplified parametric studies.

During the design process, the methods for analytical or numerical prediction of important system response shall be verified (calibrated) by appropriate model tests.

Model tests may also be used to determine specific responses for which numerical or analytical procedures are not yet developed and recognized.

Relevant motions shall be determined, by relevant analysis techniques, for those applicable design conditions specified in NORSOK N-003. The basic assumptions and limitations associated with the different methods of analysis of global performance shall be duly considered prior to the selection of the methods.

The TLP should be analysed by methods as applicable to column-stabilised units (ref. Annex M) when the unit is free floating.

The method of platform-motion analysis as outlined in this Annex is one approximate method which may be applied. The designer is encouraged also to consider and apply other methods in order to discover the effects of possible inaccuracies etc. in the different methods.

N.5.2 Frequency domain analysis

Frequency domain HF, WF and LF analyses techniques may be applied for a TLP. Regarding action effects due to mean wind, current and mean wave drift, see NORSOK N-003.

For typical TLP geometries and tendon arrangements, the analysis of the total dynamic action effects may be carried out as:

- A high-frequency (HF) analysis of springing
- A wave-frequency (WF) analysis in all six degrees of freedom
- A low-frequency (LF) analysis in surge, sway and yaw

The following assumptions are inherent in adopting such on independent analysis approach:

- The natural frequencies in heave, pitch and roll are included in the wave-frequency analysis
- The natural frequencies in surge, sway and yaw are included in the low-frequency analysis
- The high and low natural frequencies are sufficient separate to allow independent dynamic analysis to be carried out
- The low-frequency excitation forces have negligible effect on the wave-frequency motions
- The low-frequency excitation forces have a negligible dynamic effect in heave, pitch and roll
- Tendon lateral dynamics are unimportant for platform surge/sway motions

Typical parameters to be considered for global performance analyses are different platform draft, tidal effects, storm surges, set down, settlement, subsidence, mispositioning and tolerances. Possible variations in vertical centre of gravity shall also be analysed (especially if ringing responses are important).

N.5.2.1 High frequency analyses

Frequency domain springing analyses shall be performed to evaluate tendon and TLP suspectability to springing responses.

Recognised analytical methods exist for determination of springing responses in tendons. These methods include calculation of Quadratic Transfer Functions for axial tendon (due to sum frequency actions on the hull) stresses which is the basis for determination of tendon fatigue due to springing.

Damping level applied in the springing response analyses shall be duly considered and documented.

N.5.2.2 Wave-frequency analyses

A wave-frequency dynamic analysis may normally be carried out by using linear wave theory in order to determine first-order platform motions and tendon response.

First order wave action analyses shall also serve as basis for structural response analyses. Finite wave action effects shall be evaluated and taken into acccount.

In linear theory, the response in regular waves (transfer functions) is combined with a wave spectrum to predict the response in irregular seas.

The effect of low-frequency set-down variations on the WF analysis is to be investigated by analysing at least two representative mean offset positions determined from the low-frequency analysis.

Wave approach headings shall be selected with basis in global configuration (e.g. number of columns).

N.5.2.3 Low-frequency analyses

A low-frequency dynamic analysis could be performed to determine the slow drift effects at early design stage due to fluctuating wind and second order wave actions.

Appropriate methods of analysis shall be used with selection of realistic damping levels. Damping coefficients for low-frequency motion analyses are important as the low-frequency motion may be dominated by resonant response.

N.5.3 Time domain analyses

For global motion response analyses, a time domain approach will be beneficial. In this type of analyses it is possible to include all environmental action effects and typical non-linear effects such as:

- Hull drag forces (including relative velocities)
- Finite wave amplitude effects
- Non-linear restoring (tendons, risers)

Highly non-linear effects such as ringing may also require a time domain analysis approach. Analytical methods exist for estimation of ringing responses. These methods can be used for the early design stage, but shall be correlated against model tests for the final design. Ringing and springing responses of hull and deck may however be analysed within the frequency domain with basis in model test results, or equivalent analytical results.

For deep waters a fully coupled time domain analysis of tendons, risers and platform may be required.

A relevant wave spectrum shall be used to generate random time series when simulating irregular wave elevations and kinematics.

Simulation length shall be long enough to obtain sufficient number of LF maxima (surge, sway, yaw).

Statistical convergence shall be checked by performing sensitivity analyses where parameters as input seed, simulation length, time step, solution technique etc. are varied.

Determination of extreme responses from time domain analyses shall be performed according to recognised principles.

Depending on selected TLP installation method, time domain analyses will probably be required to simulate the situation when the TLP is transferred from a free floating mode to the vertical restrained mode. Model testing shall also be considered in this context.

N.5.4 Model testing

Model testing will usually be required for final check of TLP designs. The main reason for model testing is to check that analytical results correlate with model tests.

The most important parameters to evaluate are:

- Air-gap
- First order motions
- Total offset
- Set-down
- WF motions versus LF motions
- Accelerations
- Ringing
- Springing

The model scale applied in testing shall be appropriate such that reliable results can be expected. A sufficient number of seastates need to be calibrated covering all limit states.

Wave headings and other variable parameters (water levels, vertical centre of gravity, etc.) need to be varied and tested as required.

If HF responses (ringing and springing) shows to be governing for tendon extreme and fatigue design respectively, the amount of testing may have to be increased to obtain confidence in results.

N.5.5 Action effects in the tendons

Action effects in the tendons comprise of steady-state and dynamic components.
The steady-state actions may be determined from the equilibrium condition of the platform, tendon and risers.

Tendon action effects arise from platform motions, any ground motions and direct hydrodynamic actions on the tendon.

Dynamic analysis of tendon response shall take into account the possibility of platform pitch, roll and heave excitation (springing and ringing effects).

Linearized dynamic analysis does not include some of the secondary wave effects, and may not model accurately extreme wave responses. A check of linear-analysis results using non-linear methods may be necessary. Model testing may also be used to confirm analytical results. Care shall be exercised in interpreting model-test results for resonant responses, particularly for actions due to platform roll, pitch and heave, since damping may not be accurately modelled.

Lift and overturning moment generated on the TLP hull by wind actions shall be included in the tendon response calculations.

Susceptibility to vortex induced vibrations shall be evaluated in operational and non-operational phases.

Interference (tendon/riser, tendon/tendon, tendon/hull, tendon/foundation) shall be evaluated for non-operational as well as the operational phase.

N.6 ULTIMATE LIMIT STATE (ULS)

N.6.1 General

General considerations in respect to methods of analysis are given in NORSOK N-003.

The TLP hull shall be designed for the loading conditions that will produce the most severe effects on the structure. A dynamic analysis shall be performed to derive at characteristic largest stresses in the structure.

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

It is normally not practical, in design analysis of TLP's, to include all relevant actions (both global and local) in a single model. Generally, a single model would not contain sufficient detail to establish local responses to the required accuracy, or to include consideration of all relevant actions and combinations of actions. It is often the case that it is more practical, and efficient, to analyse different action effects utilising a number of appropriate models and superimpose the response from one model with the responses from another model in order to assess the total utilisation of the structure. For preliminary design, simplified models are recommended to be utilised in order to more efficiently establish design responses and to achieve a simple overview of how the structure responds to the designing actions. For final design, a complete three dimensional model of the platform is required.

N.6.2 Hull

The following analysis procedure to obtain characteristic platform-hull response shall be applied:

a) *Steady-state analysis of the initial position.*

In this analysis, all vertical actions are applied (weights, live loads, buoyancy etc.) and equilibrium is achieved taking into account pretension in tendons and risers.

b) *Steady-state offset*

In this analysis the lateral steady-state wind, wave-drift and current actions are applied to the TLP resulting in a static offset position with a given set-down.

c) *Design wave analysis*

To satisfy the need for simultaneity of the responses, a design wave approach, see NORSOK N-003, may normally be used for maximum stress analysis.

The merits of the stochastic approach are retained by using the extreme stochastic values of some characteristic parameters in the selection of the design wave and is applied to the platform in its offset position. The results are superimposed on the steady-state solution to obtain maximum stresses.

d) *Spectral analysis*

Assuming the same offset position as described in b) and with a relevant storm spectrum, an analysis is carried out using 'n' wave frequencies from 'm' directions. Traditional spectral analysis methods should be used to compute the relevant response spectra and their statistics.

For a TLP hull, the following characteristic global sectional actions due to wave forces shall be considered as a minimum, see also Annex M:

- Split forces (transverse, longitudinal or oblique sea for odd columned TLP's)
- Torsional moment about a transverse and longitudinal, horizontal axis (in diagonal or near-diagonal)
- Longitudinal opposed forces between parallel pontoons (in diagonal or near-diagonal seas)
- Longitudinal, transverse and vertical accelerations of deck masses

It is recommended that a full stochastic wave action analysis is used as basis for the final design.

Local load effects (e.g. maximum direct environmental action of an individual member, wave slamming loads, external hydrostatic pressure, ballast distribution, internal tank pressures etc.) shall be considered. Additional actions from e.g high-frequency ringing accelerations shall be taken into account.

N.6.2.1 Structural analysis

For global structural analysis, a complete three-dimensional structural model of the platform is required.

Note: *Linear elastic space-frame analysis may be utilised if the torsional moments, for example as resulting from diagonal seas, are transferred mainly through a stiff bracing arrangement. Otherwise, finite-element analysis is required, see also Annex M..*

Additional detailed finite-element analyses may be required for complex joints and other complicated structural parts to determine the local stress distribution more accurately and/or to verify the results of a space-frame analysis, see also Annex M.

Where relevant local stress concentrations shall be determined by detailed finite-element analysis or by physical models. For standard details, however, recognised formulas will be accepted.

Supplementary manual calculations for members subjected to local actions may be required where appropriate.

If both static and dynamic action contributions are included in one analysis, the results shall be such that the contributions from both shall be individually identifiable.

Local environmental action effects, such as wave slamming and possible wave- or wind-induced vortex shedding, are to be considered as appropriate.

N.6.2.2 Structural design

Special attention shall be given to the structural design of the tendon supporting structures to ensure a smooth transfer and redistribution of the tendon concentrated actions through the hull structure without causing undue stress concentrations.

The internal structure in columns in way of bracings should to be designed stronger than the axial strength of the bracing itself.

Special consideration shall be given to the pontoon strength in way of intersections with columns, accounting for possible reduction in strength due to cut-outs and stress concentrations.

N.6.3 Deck

N.6.3.1 General

Structural analysis design of deck structure shall follow the principles as outlined in NORSOK N-004, Annex M. Additional actions (e.g. global accelerations) from high-frequency ringing and springing shall be taken into account when relevant.

N.6.3.2 Air gap

Requirements and guidance to air gap analyses for a TLP unit are given in NORSOK N-003.

In the ULS condition, positive air gap should be ensured. However, wave impact may be permitted to occur on any part of the structure provided that it can be demonstrated that such actions are adequately accounted for in the design and that safety to personnel is not significantly impaired.

Analysis undertaken to document air gap should be calibrated against relevant model test results. Such analysis shall include relevant account of:

- wave/structure interaction effects,
- wave asymmetry effects,
- global rigid body motions (including dynamic effects),
- effects of interacting systems (e.g. riser systems), and,
- maximum/minimum draughts (setdown, tidal surge, subsidence, settlement effects).

Column ‘run-up’ action effects shall be accounted for in the design of the structural arrangement in way of the column/deckbox connection. These 'run-up' actions should be treated as an environmental action component, however, they need not normally be considered as occurring simultaneously with other environmental responses.

Evaluation of air gap adequacy shall include consideration of all affected structural items including lifeboat platforms, riser balconies, overhanging deck modulus etc.

N.6.4 Tendons

N.6.4.1 Extreme tendon tensions

As a minimum the following tension components shall be taken into account:

- Pretension (static tension at MSL)
- Tide (tidal effects)
- Storm surge (positive and negative values)
- Tendon weight (submerged weight)
- Overturning (due to current, mean wind/drift load)
- Setdown (due to current, mean wind/drift load)
- WF tension (wave frequency component)
- LF tension (wind gust and slowly varying drift)
- Ringing (HF response)

Additional components to be considered are:

- Margins for fabrication, installation and tension reading tolerances.
- Operational requirements (e.g. operational flexibility of ballasting operations)

- Allowance for foundation mispositioning
- Field subsidence
- Foundation settlement and uplift

Bending stresses along the tendon shall be analysed and taken into account in design. For the constraint mode the bending stresses in tendon will usually be low. In case of surface, or subsurface tow (non-operational phase) the bending stresses shall be carefully analysed and taken into account in design. For nearly buoyant tendons the combination of environmental action (axial & bending) and high hydrostatic water pressure may be a governing combination.

Limiting combinations of tendon tension and rotations (flex elements) need to be established.

For specific tendon components such as couplings, flex elements, top and bottom connections etc. the stress distribution shall be determined by appropriate finite-element analysis.

If tendon tension loss is permitted, tendon dynamic analyses shall be conducted to evaluate its effect on the complete tendon system. Alternatively model tests may be performed. The reasoning behind this is that loss of tension could result in detrimental effects from tendon buckling and/or damage to flex elements.

N.6.4.2 Structural design

The structural design of tendons shall be carried out according to NORSOK N-001 and N-003 with the additional considerations given in this subsection.

When deriving maximum stresses in the tendons relevant stress components shall be superimposed on the stresses due to maximum tendon tension, minimum tendon tension or maximum tendon angle, as relevant.

Such additional stress components may be:

- Tendon-bending stresses due to lateral actions and motions of tendon
- Tendon-bending stresses due to flexelement rotational stiffness
- Thermal stresses in the tendon due to temperature differences over the cross sections
- Hoop stresses due to hydrostatic pressure

N.6.5 Foundations

Geotechnical field investigations and careful data interpretation shall form the basis for geotechnical design parameters.

Relevant combinations of tendon tensions and angles shall be analysed for the foundation design.

For gravity foundations the pretension shall be compensated by submerged weight of the foundation, whereas the varying actions may be resisted by for example suction and friction.

N.7 FATIGUE LIMIT STATE (FLS)

N.7.1 General

Structural parts where fatigue may be a critical mode of failure shall be investigated with respect to fatigue. All significant actions contributing to fatigue damage (non-operational and operational) design conditions shall be taken into account. For a TLP, the effects of springing and ringing resonant responses shall be considered for the fatigue limit state.

Fatigue design may be carried out by methods based on fatigue tests and cumulative damage analysis, methods based on fracture mechanics, or a combination of these.

General requirements to fatigue design are given in Chapter 8 and Annex C.

Careful design of details as well as stringent quality requirements to fabrication are essential in achieving acceptable fatigue strength. It is to be ensured that the design assumptions made concerning these parameters are achievable in practice.

The results of fatigue analyses shall be fully considered when the in-service inspection plans are developed for the platform.

N.7.2 Hull

Fatigue design of hull structure shall be performed in accordance to principles given in Annex M.

N.7.3 Deck

Fatigue design of deckstructure shall be performed in accordance to principles given in Annex M.

N.7.4 Tendons

All parts of the tendon system shall be evaluated for fatigue.

First order wave actions (direct/indirect) will usually be governing, however also fatigue due to springing shall be carefully considered. HF and WF tendon responses shall be combined realistically.

In case of wet transportation (surface/subsurface) to field these fatigue contributions shall be accounted for in design.

Vortex Shedding shall be considered and taken into account. This applies to operation and non-operational (e.g. tendon stand-off) phases.

Series effects (welds, couplings) shall be evaluated.

When fracture-mechanics methods are employed, realistic estimates of strains combined with maximum defect sizes likely to be missed with the applicable NDE methods shall be used

N.7.5 Foundation

Tendon responses (tension and angle) will be the main contributors for fatigue design of foundations. Local stresses shall be determined by use of finite element analyses.

N.8 SERVICEABILITY LIMIT STATE (SLS)

N.8.1 General

Considerations shall be given to the effect of deflections where relevant.

The stiffness of the structure and structural components shall be sufficient to prevent serious vibrations and to ensure safe operation of the platform.

Allowance for wear shall be considered in areas exposed to abrasion (e.g where cross load bearings may move relative to the TLP column).

N.9 ACCIDENTIAL LIMIT STATE (ALS)

N.9.1 General

General guidance and requirements concerning accidental events are given in Chapter 9 and Annex A.

Units shall be designed to be damage tolerant, i.e. credible accidental damages, or events, should not cause loss of global structural integrity. The capability of the structure to redistribute actions should be considered when designing the structure.

In the design phase, attention shall be given to layout and arrangements of facilities and equipment in order to minimise the adverse effects of accidental events.

Satisfactory protection against accidental damage may be obtained by a combination of the following principles :

- reduction of the probability of damage to an acceptable level, and,
- reduction of the consequences of damage to an acceptable level.

Structural design in respect to the Accidental damage Limit State (ALS) shall involve a two-stage procedure considering :

- resistance of the structure to a relevant accidental event, and,
- capacity of the structure after an accidental event.

Global structural integrity shall be maintained both during and after an accidental event. Actions occurring at the time of a design accidental event and thereafter shall not cause complete structural collapse.

Requirements to compartmentation and stability in the damage condition are given in N10. When the upper hull (deckbox) structure becomes buoyant in satisfying requirements to damage stability, /2/, consideration shall be given to the structural response resulting from such actions.

N.9.2 Hull and deck

The most relevant accidental events for hull and deck design are:

- dropped objects
- fire
- explosion
- collision
- unintended flooding

Compartmentation is a key issue for TLP's due to the fine balance between weight, buoyancy and pretensions. See N10.2.

N.9.3 Tendons

The most relevant accidental events for the tendons are:

- missing tendon
- tendon flooding
- dropped objects
- hull compartment(s) flooding

Missing tendon requires analysis with 100 year environment to satisfy the ALS. The same applies to tendon flooding, if relevant.

For accidental events leading to tendon failure the possible detrimental effect of the release of the elastic energy stored in the tendon may have on the surrounding structure shall be considered.

Dropped object may cause damage to the tendons and in particular the top and bottom connectors may be exposed. Shielding may be required installed.

Flooding hull compartments and the effects on design shall be analysed thoroughly.

N.9.4 Foundations

Accidental events to be considered for the foundations shall as a minimum be those listed for tendons.

N.10 COMPARTMENTATION & STABILITY

N.10.1 General

The TLP shall have draught marks pertinent to all phases of operation. The draught marks shall be located forward and aft on the platform and at starboard and port sides. The draught marks shall be clearly visible for operating personnel, unless instruments for continuous draught sensing and reading are provided.

When the construction of the platform is as close to completion as practical, an inclining test shall be undertaken in order to establish the position of the centre of gravity and the light weight.

The information obtained from the inclining test is to be continuously updated and adjustments shall be made to account for any items taken on or off the TLP after such tests.

Loading conditions during operating or temporary phases of the TLP shall have loads and centre-of-gravity location within the envelope of maximum/minimum allowable values. Information concerning these requirements is to be specified in the Operating Manual of the TLP.

Detailed provisions as stated in IMO MODU code, /4/ shall be satisfied.

N.10.2 Compartmentation and watertight integrity

Temporary conditions

For temporary phases where the tendons are not installed, the platform may be considered as a column-stabilised unit for which procedures and criteria concerning hydrostatic stability should be taken according to Annex M.

For temporary phases where tendons are partly engaged, adequate stability shall be documented. The stability in this condition may be provided by hydrostatic stability, by the tensioned tendons or by a combination.

Operating conditions

The tension in the tendons is to be sufficient to ensure the stability of the platform in the operating phase, both for the intact and damaged conditions.

Monitoring of weight, ballast and pretension shall be performed.

The following minimum accidental flooding criteria is normally to be assumed for the ALS condition:

a) Any one compartment adjacent to the sea is to be assumed flooded, regardless of cause.
b) Any one compartment containing sea water piping systems of other sources containing liquids, due to failure of such arrangements, is to be assumed flooded unless it can be adequately documented that such requirement is unwarranted.

Watertight integrity

Watertight closing appliances are required for those external openings up to the water levels defined by:

- The water level for an angle of heel equal to the first intercept in the intact or damage condition, whichever is greater (free-floating conditions)
- The water level corresponding to the required air-gap for deck clearance in the ULS condition

Where pipe-runs may lead to critical flooding scenarios, full account is to be taken of this possibility in the design of these pipe-runs and the size of the internal spaces they run to or from.

The number of openings in watertight bulkheads and decks are to be kept to a minimum compatible with the design and proper operation of the TLP. Where penetration of watertight decks and bulkheads are necessary for access, piping, ventilation, electrical cables etc., arrangements are to be made to maintain the watertight integrity.

Where valves are provided at watertight boundaries to provide watertight integrity, these valves are to be capable of being operated from a normally manned space. Valve position indicators are to be provided at a remote-control situation.

Watertight doors and hatches are to be remotely controlled from a central safe position and are also to be operable locally from each side of the bulkhead or deck. Indicators are to be provided at the remote-control position to indicate whether the doors are open or closed.

Where the tendon elements and the tendon-column annulus are designed as non-flooded members in service, such compartments are to be considered as TLP buoyancy compartments with respect to watertight integrity. Special consideration of closing devices (seals) for the tendon/column duct annulus is necessary to fulfil requirements as to “watertight closure”.

N.11 DOCUMENTATION REQUIREMENTS FOR THE 'DESIGN BASIS' & 'DESIGN BRIEF'

N.11.1 General

Adequate planning shall be undertaken in the initial stages of the design process in order to obtain a workable and economic structural solution to perform the desired function. As an integral part of this planning, documentation shall be produced identifying design criteria and describing procedures to be adopted in the structural design of the unit.

Applicable codes, standards and regulations shall be identified at the commencement of the design.

When the design has been finalised, a summary document containing all relevant data from the design and fabrication phase shall be produced.

Design documentation (see below) shall, as far as practicable, be concise, non-voluminous, and, should include all relevant information for all relevant phases of the lifetime of the unit.

General requirements to documentation relevant for structural design are given in NORSOK, N-001, Section 5.

N.11.2 Design basis

A Design Basis Document shall be created in the initial stages of the design process to document the basis criteria to be applied in the structural design of the TLP.

A summary of those items normally to be included in the Design Basis document is included below.

Unit description and main dimensions

A summary description of the unit, including :

- general description of the unit (including main dimensions and draughts),
- main drawings including :
  - main structure drawings (This information may not be available in the initial stage of the design),
  - general arrangement drawings,
  - plan of structural categorisation,
  - load plan(s), showing description of deck uniform (laydown) actions,
  - capacity (tank) plan,
- service life of unit,
- tendon system description.
- foundation system.

Rules, regulations and codes

A list of all relevant, applicable, Rules, Regulations and codes (including revisions and dates), to be utilised in the design process.

Environmental design criteria

Environmental design criteria (including all relevant parameters) for all relevant conditions, including :

- wind, wave, current, snow and ice description for 10^{-1} ,10^{-2} and 10^{-4} annual probability events

- water levels (tides, surges, subsidence)
- design temperatures

Stability and compartmentation

Stability and compartmentation design criteria for all relevant conditions including :

- external and internal watertight integrity plan
- lightweight breakdown report
- design loadcase(s) including global mass distribution
- damage condition waterlines.(This information may not be available in the initial stage of the design.)

Temporary phases

Design criteria for all relevant temporary phase conditions including, as relevant:

- limiting permanent, variable, environmental and deformation action criteria
- procedures associated with construction, (including major lifting operations)
- essential design parameters associated with temporary phases (e.g. for mating loadcases, mating weld-up sequences, crushing tube stiffness', for transit phases, transit speed etc.)
- relevant ALS criteria
- tendon installation/replacement
- foundation installation

Operational design criteria

Design criteria for all relevant operational phase conditions including :

- limiting permanent, variable, environmental and deformation action criteria
- designing accidental event criteria (e.g. collision criteria)
- tank loading criteria (all tanks) including a description of system, with :
  - loading arrangements
  - height of air pipes
  - loading dynamics
  - densities
- tendon criteria
- tendon monitoring philosophy
- foundation criteria
- tendon interface criteria (functionality, stiffness etc.)

Air gap

Relevant basis information necessary for the assessment of air gap sufficiency, including:

- a description of the requirements to be applied in the ULS and ALS conditions
- basis model test report (This information may not be available in the initial stage of the design)

In-service inspection criteria

A description of the in-service inspection criteria and general philosophy (as relevant for evaluating fatigue allowable cumulative damage ratios).

Miscellaneous

A general description of other essential design information, including :

- description of corrosion allowances, where applicable

N.11.3 Design brief

A Design Brief Document shall be created in the initial stages of the design process. The purpose of the Design Brief shall be to document the intended procedures to be adopted in the structural design of the TLP. All applicable limit states for all relevant temporary and operational design conditions shall be considered in the Design Brief.

A summary of those items normally to be included in the Design Brief Document is included below.

Analysis models

A general description of models to be utilised, including description of :

- global analysis model(s)
- local analysis model(s)
- loadcases to be analysed

Analysis procedures

A general description of analytical procedures to be utilised including description of procedures to be adopted in respect to :

- the evaluation of temporary conditions
- the consideration of accidental events
- the evaluation of fatigue actions
- air gap evaluation, (including locations to be considered, damping, inclusion of asymmetry factors, disturbed (radiated) wave considerations, combined motion response)
- the establishment of dynamic responses (including methodology, factors, and relevant parameters)
- the inclusion of 'built-in' stresses
- the consideration of local responses (e.g. those resulting from tendon and riser actions, ballast distribution etc.)
- the consideration of structural redundancy

Structural evaluation

A general description of the evaluation process, including :

- description of procedures to be utilised for considering global and local responses
- description of fatigue evaluation procedures (including use of design fatigue factors, SN-curves, basis for stress concentration factors (SCF's), etc.)
- description of procedures to be utilised for code checking

Air gap evaluation

A general description of the air gap evaluation procedure, including :

- description of procedures to be utilised for considering air gap sufficiency

N.12 REFERENCES

/1/ Regulations concerning Loadbearing Structures in the Petroleum Activities, Norwegian Petroleum Directorate, 1996.

/2/ Regulations of 20 Dec. 1991, No.878, concerning stability, watertight subdivision and watertight/weathertight closing means on mobile offshore units, Norwegian Maritime Directorate.

/3/ 'Petroleum and Natural Gas Industries-Offshore Structures', Part 1 : General Requirements, International Standard, ISO 13819-1.

/4/ Code for the Construction and Equipment of Mobile Offshore Drilling Units, 1989, (1989 MODU CODE), International Maritime Organisation, London 1990.

/5/ API RP 2T Recommended Practice for Planning, Designing, and Constructing Tension Leg Platforms, 2. edition August 1997.

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