Danish Standard DS 412 And DS 449
Contents

Implementation of Danish Standard DS 412 And DS 449

The implementation of Danish Standard is according to:

STUCTURAL USE OF STEEL
DANSK STANDARD DS 412
1. EDITION APRIL 1983

PILE-SUPPORTED OFFSHORE STEEL STRUCTURES
1. EDITION APRIL 1983
DANSK STANDARD DS 449

PILE-SUPPORTED OFFSHORE STEEL STRUCTURES
PART 2, ANNEXES A-F
SUPPLEMENTARY GUIDE
1. EDITION OCTOBER 1983
DANSK STANDARD DS 449

The check covers design/utilization for pipe cross sections members. DS 412 code check is only available for circular pipes.

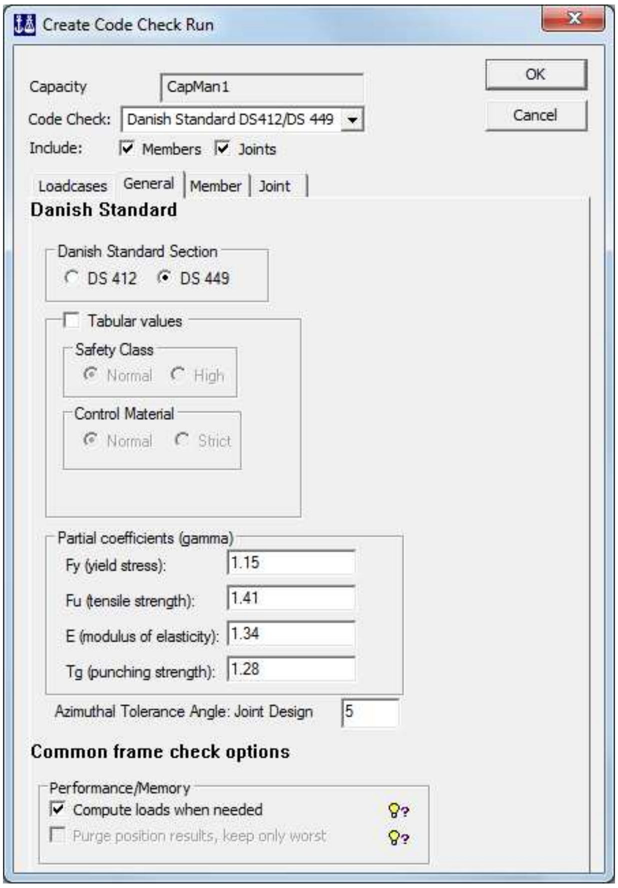
- Design is based on the principle that no applicable strength limit state shall be exceeded when the structure is subjected to all appropriate load combinations. The check covers checking of isolated members.
- Design of connections is not covered.
- Design for serviceability is not covered.
- For DS 412 von Mises stresses are verified.

Select Danish Standard from Create Code Check Run Dialog:

Define the general parameter:

The characteristics tensile strength, f_u, and the characteristic tensile yield stress, f_y, are determined as the minimum tensile strength and the minimum upper tensile yield stress, respectively, specified in the code of practice or standard for the actual type of steel. The compressive yield stress of the steel is taken as equal to the tensile yield stress. The partial coefficients are grouped according Safety Classes and the parameters are pre-defined.

The general parameters for the Danish Standard (DS) 412/449 are shown to the right. DS412 and DS449 are both made for handling tubular sections only.

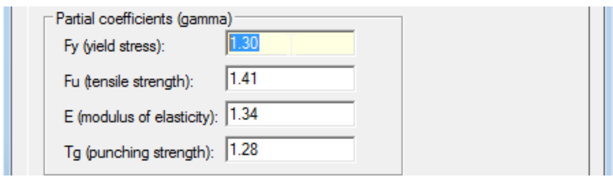


Danish Standard Section:

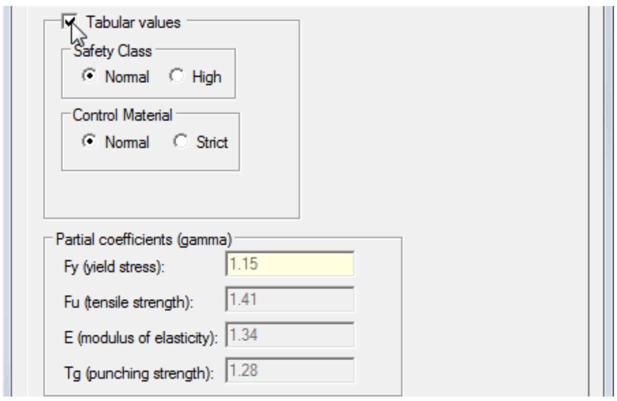
- DS 412 – Onshore structures
- DS 449 – Offshore structures

Safety Class
The default DS partial coefficients can be seen at the bottom of the dialog. Note that these are not the same for DS 412 and DS 449. We will first have a look at the settings for DS 449.

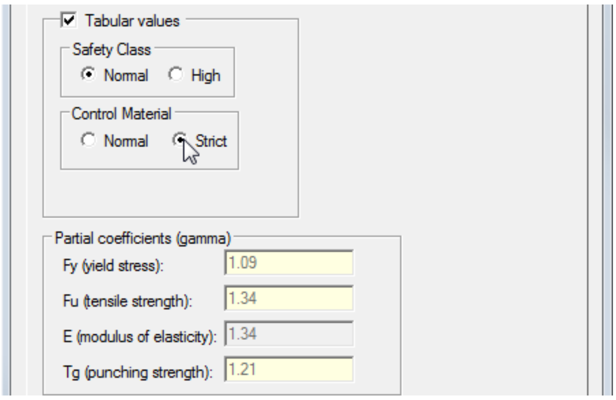
You can change the default partial coefficients. An example is shown in the illustration to the right



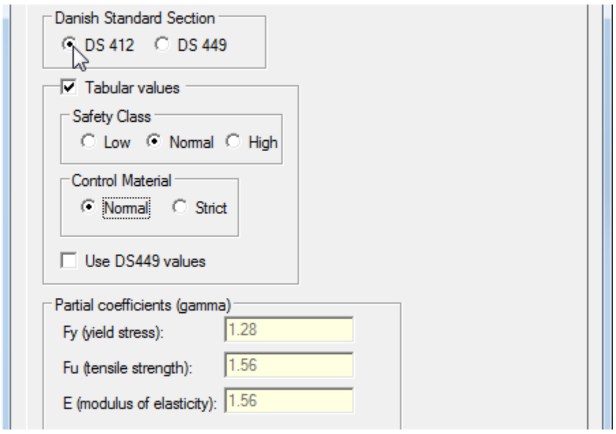
If you check off “Tabular values”, the choices under “Safety Class” and “Control Material” decide which partial coefficients that will be used.
Note that the setting we just changed (Fy) is back at the value decided by the tabular values settings.



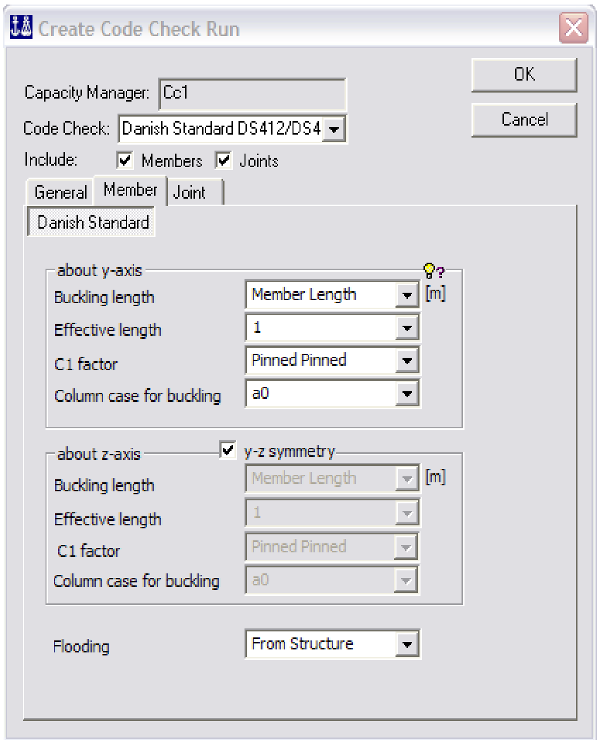
If we, as an example, change the control material from “Normal” to “Strict” by clicking the radio button, the partial coefficients change according to the new tabular values settings.



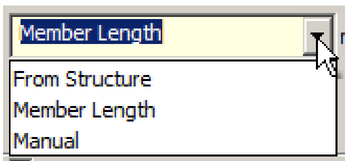
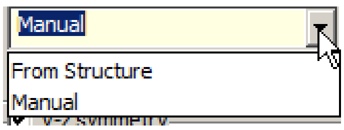
The settings for DS 412 are similar to the settings for DS 449, but there are a couple of differences:
DS 412 has a “Low” safety class.
There are no Tg (punching strength) for DS 412 since this standard do not support joints.



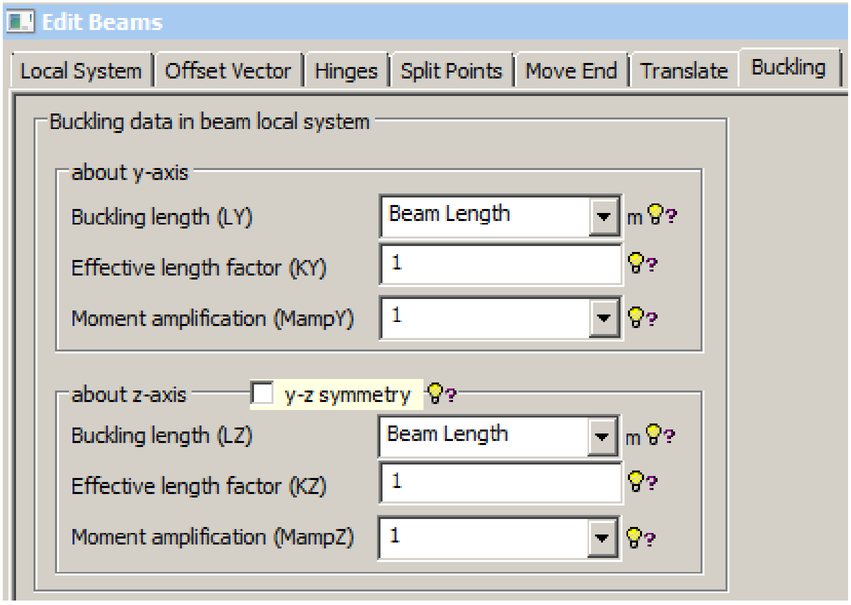
Definition of member specific parameters:



Options:

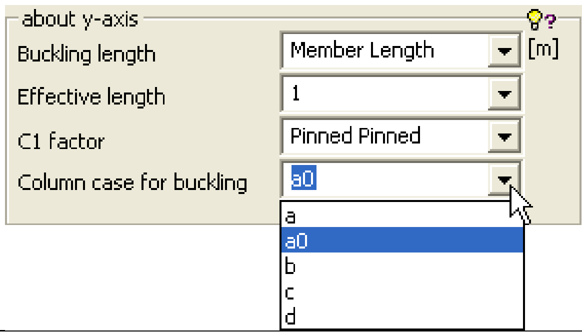
Buckling length	From Structure = use value/option assigned to the beam concept, ref. Edit Beam dialog (see below) Member Length = use the geometric length of the member (capacity model). Manual = specify the length to be used 
Effective length factor	From Structure = use value/option assigned to the beam concept, ref. Edit Beam dialog (see below) Manual = specify the factor to be used  The **<u>Edit Beam</u>** dialog may be used to define the buckling length and buckling factor to be used in the code checks (i.e. assigned to

<table>
<tr><td></td><td>the capacity members) based on the values/options assigned to the beam object. (The definition of Moment amplification is not in use in DS.)

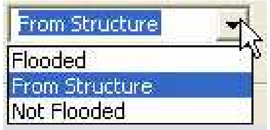


Note that the “From Structure” alternatives are only accepted in cases with one-to-one mapping between modelled beam and capacity member, else the default value/option will be used.</td></tr>
<tr><td>C1 factor</td><td>The C1 factor represents a non defined variable. According to the standard, the value is based on beam’s boundary conditions, and is necessary in order to compute Hydrostatic Critical Pressure. By default this parameter is set to Pinned-Pinned but other boundary conditions are also available.

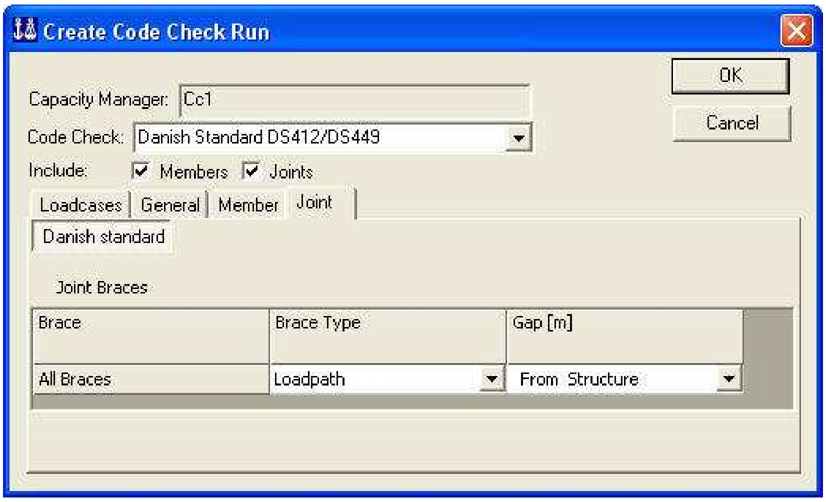

</td></tr>
<tr><td>Column case for buckling</td><td>The significance of the residual stresses on the shape and manufacture of the profile is considered. You can select between 5



cases, a0, a, b, c and d. A description of the cases can be found in Table V 6.2.1</td></tr>
</table>

About z-axis	By unchecking the "y-z symmetry" checkbutton you can change the same settings as for the y-axis.
Flooding	From Structure = use the properties assigned to the beam concepts using the properties defined from the "Create/Edit Hydro Property" dialog, or manually specify Flooded or Not Flooded. 

The global joint parameters for DS are shown to the right



Nomenclature Danish Standard

Member check Danish Standard

The print of all available results inclusive intermediate data from the Danish Standard check will report the following:

Member	Capacity model name (name of Beam(s) or part of beam representing the member)
Loadcase	Name of load case/combination under consideration
Position	Relative position along member longitudinal axis (start=0, end=1)
Status	Status regarding outcome of code check (OK or Failed)
UfTot	Value of governing usage factor
Formula	Reference to formula/check type causing the governing usage factor
SubCheck	Which check causes this result, here Danish Standard member check
	Status regarding any violation of geometric limitations: (add some information)
GeomCheck	

DS 412 Intermediate Values

uf_length_chord verification of chord's slenderness according DS 412, section 6.2.8.

uf_length_brace verification of brace's slenderness according DS 412, section 6.2.8.

ufD615 von Mises stress - Multi-axial stress field

uf1 usage factor for members in bending and compression according to:

$$\frac{N}{A}+\frac{N_{elx}}{N_{elx}-N}\frac{aM_x+Ne_x}{W_x}+\frac{N_{ely}}{N_{ely}-N}\frac{M_y}{W_y}\leq f_{yd}$$

uf2 usage factor for members in bending and compression according to:

$$\frac{N}{A}+\frac{N_{elx}}{N_{elx}-N}\frac{aM_x}{W_x}+\frac{N_{ely}}{N_{ely}-N}\frac{M_y+Ne_y}{W_y}\leq f_{yd}$$

gamma_fy412 partial coefficient of yield stress parameter for DS 412.

gamma_fu412 partial coefficient of ultimate stress parameter for DS 412.

gamma_e412 partial coefficient of Young modulus parameter for DS 412.

ed412 updated Young modulus according safety class chosen in DS 412.

fyd412 updated yield stress according safety class chosen in DS 412.

ek_ratio ration between equivalent geometrical imperfection and core radius. Base on the cross section properties and stress relived properties.

beta parameter used to compute the design critical stress for the compression member. (there is also a beta parameter on joint analysis)

sigmacr_comp design critical stress for the compression member.

sigmaely design critical stress according to the theory of Elasticity.

sigmaelz design critical stress according to the theory of Elasticity.

Nelx design critical axial force according to the theory of elasticity through direction x.

Nely design critical axial force according to the theory of elasticity through direction y.

sigmacr_tor is the design critical stress with regard to torsional buckling.

a distance between braces. (to move)

ky core radius – y direction.

kz core radius – z direction.

ey equivalent geometrical imperfection – y direction.

ez equivalent geometrical imperfection – z direction

lamdary is the relative slenderness ratio – direction y.

lambarz is the relative slenderness ratio – direction z.

DS 449 Intermediate Values

ufD124 combined actions usage factor according to DS449 D.1.2.4

gamma_fy449 partial coefficient of yield stress parameter for DS 449.

gamma_fu449 partial coefficient of ultimate stress parameter for DS 449.

gamma_e449 partial coefficient of Young modulus parameter for DS 449.

ed449 updated Young modulus according safety class chosen in DS 449.

fyd449 updated yield stress according safety class chosen in DS 449.

epsilon_a parameter used in order to compute local critical compressive design stress for tubular members subjected to axial forces and bending moments.

epsilon_b parameter used in order to compute local critical compressive design stress for tubular members subjected to axial forces and bending moments.

sigma_ad design stress caused by axial forces.

epsilon parameter used in order to compute local critical compressive design stress for tubular members subjected to axial forces and bending moments ($\varepsilon = \varepsilon_a + \varepsilon_b$).

sigma_el critical compressive design stress based on the theory of elasticity.

lamda_a parameter involved on computation of critical stresses according DS449 D1.2.2

sigma_crD122 local critical compressive design stress for tubular, members subjected to axial forces and bending moments.

alpha parameter used to compute k2 parameter and based on the ratio between radius and thickness.

C1 parameter used to compute k2 parameter for circular cylindrical shell as function of the boundary conditions.

k1 parameter involved on computation of critical stresses, in order to compute critical pressure, according DS449 D1.2.3

k2 parameter involved on computation of critical stresses, in order to compute parameter k1, according DS449 D1.2.3

sigma_cr_hydro critical design stress, according DS449 D1.2.3

pressure_cr critical hydrostatic design pressure.

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Structural Use of Steel - Translation Into English of Dansk Ingeniørforening's Code of Practice

2. EDITION APRIL 1983
DANSK STANDARD DS 412

This document is an excerpt from the complete description of the Danish Standard.

1 - INTRODUCTION

A steel structure is in conformity with Dansk Ingeniørforening's structural design codes when it satisfies the requirements of this code and the requirements of DS 409 The Safety of Structures.

The code applies to load-bearing steel structures in buildings, bridges, etc. For structures of special design and structures for which special requirements are made, the provisions of the present code will not form an adequate basis.

Guide: The present code alone does not form an adequate basis for e.g. nuclear power plants, extremely tall buildings, offshore structures, storage tanks, thin-plate structures and pressurized plants. The code applies to such structures only to the extent where the special codes covering these areas refer to provisions in the present code.

Codes for special areas are:

- DS 417 Welded storage tanks
- DS 446 Thin-plate structures
- DS 449 Pile supported offshore steel structures

In general, DS 446 covers structures with plate thickness less than 4 mm. However, DS 446 may under certain conditions also be applied to structures with plate thicknesses above 4 mm, cf. DS 446, chapter 1.

1.1 – Definitions

Nominal value
Numerical value used to characterize a quantity. The numerical value need not be equal to the actual value or quantity.

Static action
Action having such a low frequency that fatigue of the structural material or the joint is not the criterion for the load carrying capacity of the structure.

Strain
Unit deformation, e.g. elongation or shortening per unit length.

Stress range
The difference between maximum and minimum stress belonging to the individual stress variation, tensile stresses being assumed positive and compressive stresses negative.

Stress ratio
The ratio between minimum and maximum stress, tensile stresses being assumed positive and compressive stresses negative.

1.2 – Symbols

Specific symbols are:

A	cross sectional area
a	thickness of weld, distance
b	width
c	design factor
d	diameter
E	slope stress-strain curve for steel, modulus of elasticity
e	initial deflection
F	force
f	strength
h	depth
I	second moment of area of cross section
i	radius of gyration of cross-section
k	core radius of cross section, factor
l	length
M	moment
m	mean value
N	normal force, axial force
n	number
R	resistance
r	radius
t	thickness
W	section modulus for cross section
α	fillet weld angle, ration
β	stress ratio, ratio, angle
γ	partial coefficient
δ	coefficient of variation
λ	slenderness ratio
σ	axial stress
τ	shear stress

Subscripts

cr	critical
d	design
el	elastic –theoretical

f	dependent on load
h	embedding
i	penetration
$corr$	adjusted
l	longitudinal
m	dependent on material
max	maximum
mid	mean value
min	minimum
n	number, lower, nominal
p	pre-stress, proportionality
r	relative
red	reduced
s	column
t	transverse
u	fracture
v	stress range
y	yielding

Subscript d for design parameters has only been used in connection with the symbols f_y, f_u, E, and μ.

2 – PRELIMINARY INVESTIGATIONS

Generally, no special preliminary investigations are necessary in connection with steel structures.

3 - MATERIALS

3.1 – General

Materials with well-defined resistance properties corresponding to those assumed to verify the load-carrying capacity of the structure should be used.

3.1.1 – Material groups

Distinction is made between 7 groups of materials:

I. Ordinary hot-rolled, plain, weldable steels complying with the requirements of ISO 630, ISO 4952, or similar standards. The steel should have a specified lower limit for upper yield stress (prescribed 0.5 per cent stress, if any) and specified lower limit for the tensile strength in the range of 360-520 N/mm^2.
II. Hot-rolled, weldable, plain CMn steels with a specified lower limit for upper yield stress (specified 0.5 per cent stress, if any) and specified lower limit for the tensile

strength above about 520 N/mm^2. These steels should comply with the requirements of ISO 4950/2 or similar.

III. Heat treated steels. These steels should comply with the requirements of ISO 4950/3 or similar.
IV. Alloy and carbon steels. Steels according to ISO/R 1052 are assigned to this group.
V. Steel castings.
VI. Cast iron
VII. Other materials

Forged components are assigned to one of the above groups according to the analysis of the material and other properties.

Guide: The requirements for tensile strength and yield stress for ordinary structural steels, material group I specified in ISO 630, are given in Table 1.

Table 1 - Requirements for strength properties of steel according to ISO 630

	Tensile strength f_u N/mm^2	Upper yield stress f_y N/mm^2 for thickness t mm		
		$t \leq 16$	$16 < t \leq 40$	$40 < t \leq 63$
Fe 360	360-460	235	225	215
Fe 430	430-530	275	265	255
		$t \leq 16$	$16 < t \leq 35$	$35 < t \leq 50$
Fe 510	490-630	355	345	335

4 – ACTIONS

Not taking into account.

5 – SAFETY

5.1 The partial coefficient method

5.1.1 – Design material parameters

In the assessment of ultimate limit state the values of the partial coefficient method, γ_m, given in Table 2 are used for the different material parameters.
In the assessment of serviceability limit states $\gamma_m = 1.0$ is used.

5.1.2 – Characteristic material parameters

The characteristics tensile strength, f_u, and the characteristic tensile yield stress, f_y, are determined as the minimum tensile strength and the minimum upper tensile yield stress, respectively, specified in the code of practice or standard for the actual type of steel. The compressive yield stress of the steel is taken as equal to the tensile yield stress.

For steel of groups I and II the characteristic modulus of elasticity $E = 210\,000$ N/mm^2.

Table 2 - Partial coefficient γ_m of material parameters

	Safety class		
	Low	Normal	High
f_p, f_y, f_{yw} (also in the calculation for structural members in compression)			
normal material control	1.15	1.28	1.41
strict material control	1.09	1.21	1.34
f_u			
normal material control	1.41	1.56	1.72
strict material control	1.34	1.48	1.63
f_{fat}			
normal material control	1.76	1.95	2.15
strict material control	1.67	1.85	2.04
μ (coefficient of friction)			
friction joints	1.15	1.28	1.41
unlimited slip possible	1.28	1.42	1.56
E	1.41	1.56	1.72

6 – DESIGN AND CONSTRUCTION

6.1 – General

6.1.1 – Static action and fatigue action

Statically loaded structures should satisfy the requirements of the following sections:

6.1.2 – Design according to the theory of elasticity

In designing for safety against types of failure other than deflection, buckling, and lateral instability a linear-elastic stress-strain curve with the modulus of elasticity E up to the yield stress f_y may be assumed.

In designing against failures such as deflection, buckling, and lateral instability, account should be taken of the decreasing slope of the stress-strain curve above the proportionality stress f_p.

6.1.3 – Design according to the theory of plasticity

Statically loaded structures may de designed according to the theory of plasticity. It should thus be ensured that failure does not occur due to repeated yielding with opposite signs.

Guide: Alternating tensile yielding and compressive yielding in a structural member may cause failure of the material after relatively few stress alterations. On the safe side yielding with opposite signs can be avoided by permitting only plastic deformations to occur by the first loading of each individual load arrangement so that renewed loading by the live load only causes elastic deformations. This can be done by designing according to the theory of elasticity and by superimposing the stress resultants and stresses with the most favorable initial distribution of stress resultants and stresses. It is assumed that this initial distribution is statically permissible and that the same distribution is applied to all load arrangements.

6.1.4 – Slip of friction joints

Not applicable.

6.1.5 – Multi-axial stress field

For the tri-axial stress field the point of yielding can be determined from the von Mises yield criterion.

$$\sqrt{\frac{1}{2}(\sigma_1-\sigma_2)^2+\frac{1}{2}(\sigma_1-\sigma_3)^2+\frac{1}{2}(\sigma_2-\sigma_3)^2} = f_{yd}$$

where σ_1, σ_2, and σ_3 are the tree principal stresses inserted with signs.

Guide: for a plane stress field von Mises yield criterion can be written as

$$\sqrt{\sigma_x^2+\sigma_y^2+\sigma_x\sigma_y+3\tau_{xy}^2} = f_{yd}$$

where σ_x, σ_y, and τ_{xy} are the design stress components in two arbitrary sections at right angles, σ_x and σ_y are inserted with signs.

For the stress field pure shear with the design shear stress τ yielding may be assumed to occur when $\tau = 0.58 f_{yd}$.

6.1.6 – Strength reduction due to holes

Not applicable.

6.1.7 – Tension perpendicular to the surface

In the design of the structure of the structure it should be taken into account that structural steel has reduced mechanical properties perpendicular to the surface when lamination, micro lamination, and segregation occur.

Structural members subjected to tension perpendicular to the plate surface should not be laminated.

The tensile stress in the middle of the plate perpendicular to the direction of rolling, determined under the assumption that the outer force distributes uniformly at an angle of 45º, should not exceed $1/2\ f_{yd}$ unless a verification of the mechanical properties in the through-thickness direction, is carried out.

In the segregation zones of rimmed steel the tensile stress perpendicular to the surface of the profile should not exceed $1/2\ f_{yd}$.

6.1.8 – Direct transmission of forces in splices

Not relevant for this study.

6.1.9 – Combined action of different types of joints

Combined action of welds and dowelled joints should not be assumed in the same joint.

Combined action of welds and friction joints is permissible when it can be verified that the welds do not prevent wholly or partly the prestressing forces of the bolts from being transmitted directly through the contact surfaces of the members to be prestressed. It should furthermore be ensured that the individual members of the joint are not subjected to such plastic deformations that the load-carrying capacity becomes less than assumed.

6.2 Compressed structural members

In the assessment of the load-carrying capacity of structures and structural members, such as columns, beams, plates, arches, and frames, in which compression occurs account should be taken of the influence of the deflections.

Account should also be taken of local instability, if any, in the form of deflection of compression members, plate buckling, and torsional buckling.

In the design account should furthermore be taken of residual stresses due to the manufacturing process.

Finally, account should be taken of the geometrical imperfection, if any, such as initial deflection, intentional, and unintentional eccentric applications of force and variation of the cross sectional shape from one section to another.

An investigation of the load-carrying capacity corresponding to deflection should always be made.

If local instability could occur before the load-carrying capacity has been reached. It should furthermore be ensured that the requirements for failure as regards the serviceability are satisfied.

6.2.1 – Centrally loaded compression members

In the assessment of the load-carrying capacity of a theoretical axially loaded compression member the member should be designed for plane deflection as well as for spatial deflection (torsion) of the member as a whole. Furthermore, the member should have adequate safety against local instability.

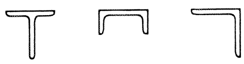
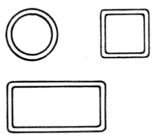
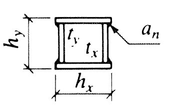
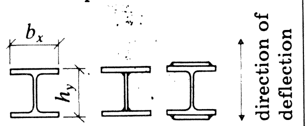
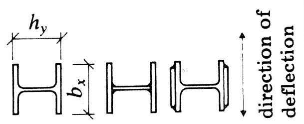
> Guide: Safety against spatial deflection may often be assumed to be allowed for without particular investigation, if it is shown that the member has adequate safety against both plane deflection as a whole and local instability. Provided the supports of the member are efficient in all directions of deflection, this applies to e.g. hot-rolled double-symmetric I-profiles and single-symmetric angles.

The design of a centrally loaded compression member may be carried out using the load-carrying capacity expression expressions reference is made to annex A.

The method indirectly includes the effect of residual stresses and geometric imperfections.

The significance of the residual stresses on the shape and manufacture of the profile considered. A grouping into 5 cases, a_0, a, b, c, and d, as shown in table V 6.2.1 is used. The expressions include a geometrical imperfection corresponding to an initial deflection of the compression member of $1/1000$ of the buckling length.

Table V 6.2.1. Fields of application for the cases a_0, a, b, c, and d.

type of profile		stress relieved	not stress relieved				
solid profiles 		b	c				
tees, channels, and angles 		b	c				
hollow sections 	seamless	a (a_0)	a (a)				
	welded	a (a_0)	b (a)				
	cold formed and welded		c				
box sections 	$a_n \le t/2$ or $h/t \ge 30$	a (a_0)	b (a)				
	$a_n > t/2$ and $h/t < 30$	a (a_0)	c				
			rolled			welded	
			$\frac{h_y}{b_x} > 1.2$	$\frac{h_y}{b_x} \le 1.2$	welded laminae	flame cut flange plates	edge-rolled flange plates
I- and H-profiles 		a (a_0)	a (a)	b (a)	b (a)	b (a) [c]	b (a) [c]
I- and H-profiles 		b (a)	b (b)	c (b)	a (a)	b (a) [c]	c (b) [d]

Cases without brackets apply to profiles with thickness of plate ≤ 40 mm of material group I. (See chapter 3 for material groups).

Cases with round brackets ( ) apply to profiles with thickness of plate ≤ 40 mm of material group II.

Cases with square brackets [ ] apply to profiles with thickness of plate > 40 mm.

As load-carrying capacity expressions

$$\frac{\sigma_{cr}}{f_{yd}} = 1 \qquad \text{for } \lambda_r \le 0.2$$

$$\frac{\sigma_{cr}}{f_{yd}} = \beta - \sqrt{\beta^2 - \frac{1}{\lambda_r^2}} \qquad \text{for } \lambda_r > 0.2$$

are used, where

$$\beta = \frac{1}{2\lambda_r^2}\left(\lambda_r^2 + 1 + \frac{e}{k}\right)$$

$$\frac{e}{k} = \begin{cases} 0.13(\lambda_r - 0.2) & \text{for case } a_0 \\ 0.21(\lambda_r - 0.2) & \text{for case } a \\ 0.34(\lambda_r - 0.2) & \text{for case } b \\ 0.49(\lambda_r - 0.2) & \text{for case } c \\ 0.76(\lambda_r - 0.2) & \text{for case } d \end{cases}$$

The following symbols relating to the actual direction of deflection have been used:

σ_{cr} the design critical stress for the compression member

f_{yd} the design yield stress

$\lambda_r = \sqrt{\frac{f_{yd}}{\sigma_{el}}} = \frac{l_s}{i}\frac{1}{\pi}\sqrt{\frac{f_{yd}}{E_d}}$ the relative slenderness ratio

σ_{el} the design critical stress according to the theory of elasticity (the “Euler stress”)

l_s the buckling length of the member

i the radius of gyration of the cross section

E_d the design modulus of elasticity

e equivalent geometrical imperfection, comprising both imperfection, comprising both imperfections of the material (residual stresses and curved stress-strain relation) and geometrical imperfections (initial deflection, eccentric application of force and variation of the cross-sectional shape from cross-section to cross-section)

$k = \frac{W}{A}$ core radius

W second moment of area corresponding to the tensile side of the deflection considered

A the cross-sectional are of the member.

The load-carrying capacity expressions are shown in figure V 6.2.1.

In the calculation to a initial deflection greater than $1/1000$ of the buckling length the same expressions are used, adding e_0/k to e/k where e_0 is the value by which the geometrical imperfection exceeds $1/1000$ of the buckling length. As an alternative in this case the expression for members in bending and compression may be used, cf. paragraph 6.2.2.

6.2.2 - Members in bending and compression

In the design of the load-carrying capacity of a member in compression which is laterally or eccentrically loaded, the member should be investigated for the same conditions as centrally loaded compression members, see paragraph 6.2.1.

Account should be taken of the moments including the additional moments from the axial force due to the deflection of the member.

Guide: The design with regard to plane deflection as a whole of a member in bending and compression with rectified lateral load and constant cross-section can be approximated by the load-carrying capacity expression

$$\frac{N}{A} + \frac{N_{el}}{N_{el} - N} \frac{|M| + Ne}{W} \leq f_{yd}$$

It is assumed that the member is incorporated in a structure with frames without sideways (prevented from horizontal displacement).

For $M = 0$ the expression is identical with the load carrying capacity expression for centrally loaded compression members in the guide of paragraph 6.2.1.

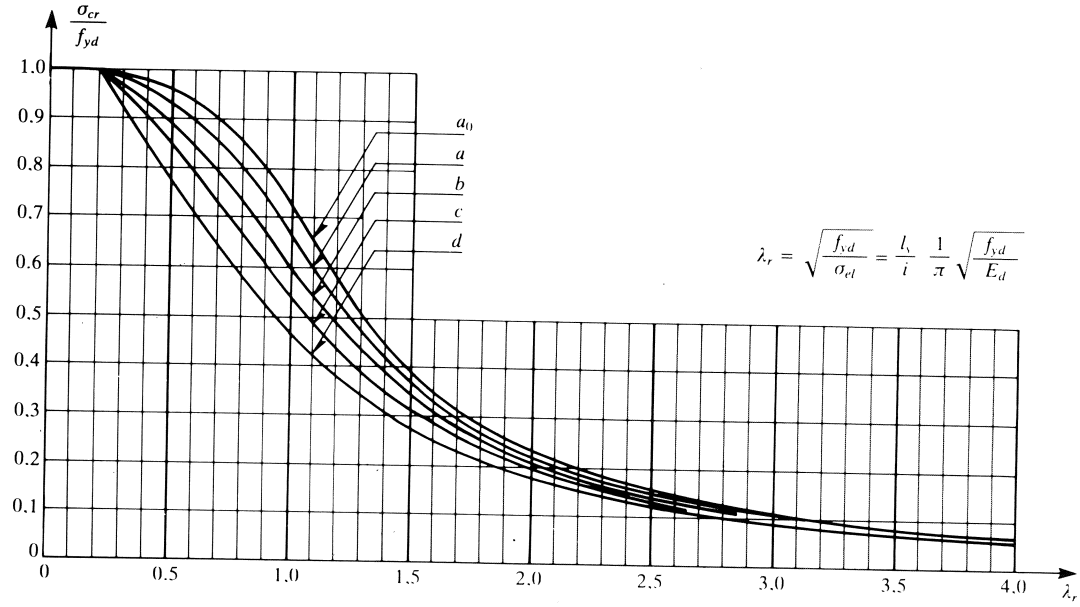


Figure V 6.2.1. Critical stress for members in compression.

For a member with unsymmetrical cross-section the design should be supplemented with an investigation of the tensile side of the member.

The following designations relating to the direction of deflection considered have been used:

N the design axial force in compression assumed positive

M the equivalent design external moment, see below

A area of the member

N_{el} the design critical axial force according to the theory of elasticity (the "Euler force")

e the equivalent geometrical imperfection determined from the expressions for e/k in the guide of paragraph 6.2.1 $e/k = 0$ for $\lambda_r \leq 0.2$

f_{yd} the design yield stress.

M is determined as the numerically greater of the values.

$$M = \begin{cases} 0.4M_1 + 0.6M_2 + M_0 \\ 0.4M_2 + M_0 \end{cases} \quad \text{if } M_0 \text{ and } M_2 \text{ have identical signs}$$

$$M = \begin{cases} 0.4M_1 + 0.6M_2 \\ 0.4M_2 \\ M_0 \end{cases} \qquad \text{if } M_0 \text{ and } M_2 \text{ have opposite signs}$$

However M should not be taken greater than the maximum resulting moment occurring in the member.

For a member supported at both ends

M_1 and M_2 are the moments at the extreme points of the member, M_2 being the numerical greater,

M_0 is the maximum moment from rectified lateral load perpendicular to the longitudinal direction of the member, determined under assumption that the member is simply supported.

For a member restrained at one end and free at the other

$M_1 = M_2$ is the moment at the free end,

M_0 is the moment at the restraint from lateral load perpendicular to the longitudinal direction of the member

Reference is made to figure V 6.2.2.

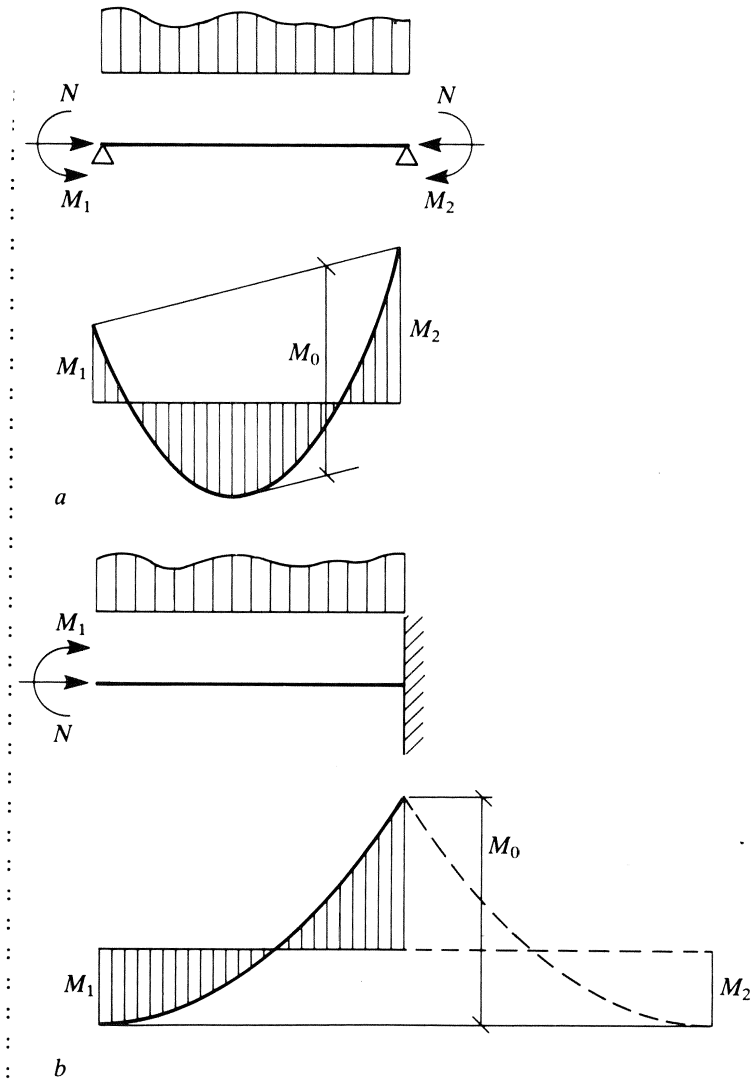


Figure V 6.2.2. Moments in a laterally loaded member.

<u>Note</u>: In order to compute with accuracy the bending moments, we use de actual bending moments along the beam and not the idealized way as shown on this section.

If the member has a geometrical imperfection corresponding to an initial deflection exceeding $1/1000$ of the buckling length, e should be added to the value by which the geometrical imperfection exceeds $1/1000$ of the buckling length.

If the member is subjected to bending moments around both principal axes, the design with regard to spatial deflection can be made analogous to the above by verification of the most demanding of the expressions

$$\frac{N}{A}+\frac{N_{elx}}{N_{elx}-N}\frac{aM_x+Ne_x}{W_x}+\frac{N_{ely}}{N_{ely}-N}\frac{M_y}{W_y}\leq f_{yd}$$

$$\frac{N}{A}+\frac{N_{elx}}{N_{elx}-N}\frac{aM_x}{W_x}+\frac{N_{ely}}{N_{ely}-N}\frac{M_y+Ne_y}{W_y}\leq f_{yd}$$

The subscripts x and y referring to the two principal axes, the x-axis being the stronger axis.

In the expressions $a = f_{yd}/\sigma_{cr}$, where σ_{cr} is the critical stress with regard to lateral instability, cf. paragraph 6.2.4.

For a member in bending and compression an ordinary stress investigation should be carried out in addition to the above investigations, proving that the stresses form the axial force and the external moments do not exceed the design yield stress.

6.2.4 – Torsional buckling

Beams should be designed for torsional buckling, i.e. instability caused by deflection of the compressed flange in its own plane.

Guide: the critical stress with regard to torsional buckling may be determined from the expression

$$\frac{\sigma_{cr}}{f_{yd}}=\frac{1}{\sqrt{1+\lambda_r^4}}$$

where

σ_{cr} is the design critical stress with regard to torsional buckling

$\lambda_r=\sqrt{\frac{f_{yd}}{\sigma_{el}}}$ is the relative slenderness ratio

σ_{el} is the design critical stress with regard to torsional buckling according to the theory of elasticity.

If the plastic properties of the material are utilized to attain the most favorable distribution of the stress resultants in a beam or frame system particular stability requirements with regard to torsional buckling should be satisfied.

6.2.5 - Special conditions for members in lattice structures

In the design of a member incorporated in a lattice structure or a built-up member in compression, consideration should be given to moments acting at the ends of the member. When determining the buckling length of such a member the rigidity of the restraints at the ends of the member should be taken into consideration.

> Guide: if the buckling length is determined from a reasonable assessment with due regard to the effect of secondary moments and the rigidity of the restraints, the member might, dependent on the conditions, be designed as a theoretically axially loaded compression members.
>
> In the assessment of the action of secondary moments, account should be taken of the ability of the flange to absorb construction eccentricities, if any, at the ends of the member, as well as to the moments from the deformations due to sideways of the frame and lateral load of the flanges. Furthermore, the ability of the support of the member to transmit moments should be assessed.
>
> When assessing the rigidity of restraints the bending and torsional rigidity of the flange should be considered and also if the flange is fully utilized in compression coincident with the lattice girder. Furthermore, account should be taken of the action of lattice girders subjected to tension and connected with the same support as the compressed lattice girder in consideration. The restraint of other lattice planes, if any, of the flange against torsion should also be taken into consideration. Finally, the rigidity of the support of the lattice girder should be assessed.
>
> For flange members the buckling length l_s will normally be equal to the distance between the supports l.
>
> Guide: for simple lacing, or lacing with battens, the buckling length may normally be assumed to be in the $0.7l - 1.0l$ range.

6.2.6 – Support of members in compression

If in the design a member or flange plate in compression is assumed to be prevented from deflection at certain points, the supporting system should have adequate strength and stiffness to comply with the requirements for safety against failure with regard to the load-carrying capacity.

> Guide: the design of a column or frame system supporting one or more members or flange plates in compression can be made in the following simplified way.
>
> In the support of an individual member or flange plate in compression, the supporting system should at one point of support at a time be designed for a force equal to 1.5 per cent of the compression force in the member acting at the point of support in the direction of deflection of the supported member. Other forces, if any, should be included.

It should furthermore be ensured that the rigidity of the supporting column or frame system is adequate to fulfill the requirements for resistance to deflection. The design coefficient of elasticity of the steel used should be used in the design of the rigidity.

If several members or flange plates in compression are prevented from deflection by a column or frame system these may, in the absence of a closer investigation, together with other actions be designed for forces equal to 1.5 percent and 1.0 per cent of the compression force in the first and second supported member or flange plate, respectively, and 0.5 per cent of the compression force in the rest of the supported members or flange plates. These forces should be assumed to act in the direction of deflection of the supported members, but at only one point of each supported member or flange. The specified order of the supported members should be chosen as unfavorable as possible for the structural member considered.

The rigidity of the column or frame system should correspond to the total sum of the rigidities of support required for each supported member or flange plate.

6.2.7 – Built-up members in compression

For a built-up member in compression comprising flanges connected together by cross members, e.g. lattice girders or battens, it should be verified that the resistance to deflection of the entire member or local deflection of the flanges is adequate or, where significant, a combination thereof. The rigidity of the cross members should be taken into account in the design.

In designing the flanges of the built-up member account should be taken of the increased flange force caused by deflection of the built-up member, due to lateral or eccentric loads.

It should be shown that the cross members of the built-up member have adequate strength to comply with requirements for the safety against failure, with regard to the load load-carrying capacity of the member as a whole.

If the cross members are in the form of lacings, where significant forces are introduced due to bending stiffnesses and length alterations such forces should also be taken into consideration.

Bracing members, if any, should be designed according to these rules as well.

Guide: the design of the cross members of the built-up member may be based on the assumption that the member, in addition to any shear forces due to external loads, if any, is loaded by a force that is 2.5 per cent of the axial force. This force is assumed to act in the plane considered and at right angles to the member. It is assumed that the initial deflection of the built-up member amounts to not more than $1/500$ of the length.

6.2.8 – Maximum slenderness ratio of members in compression

The slenderness ratio l_s/i of a member in compression assumed to be subjected to forces due to the load on the structure should not exceed 200.

The slenderness ratio of a bracing member loaded only by secondary compression forces should not exceed 250.

ANNEX A – LOAD-CARRYING CAPACITY EXPRESSIONS FOR MEMBERS IN COMPRESSION

The load-carrying capacity expressions given in the guide of paragraph 6.2.1 (the Perry-Robertson formula) may also be written in the form

$$\left(f_{yd} - \sigma_{cr}\right)\left(\sigma_{el} - \sigma_{cr}\right) = \frac{e}{k}\sigma_{el}\sigma_{cr}$$

The expression is theoretically correct for the following assumptions:

- Imperfections occur only in the form of geometrical imperfections as a sinusoidal initial deflection with amplitude e,
- Failure of load carrying capacity is defined as the exact load level where yielding in a fiber occurs.

For practical centrally loaded compression members where, in addition to material imperfections (residual stresses due to rolling and /or welding as well as decreasing slope of the stress-strain curve over the proportionality stress), the same expression may be used, but using an equivalent imperfection comprising both geometrical and material imperfections for e.
The given expressions for e have been chosen so that the resulting load-carrying capacity expressions give good agreement with the empirically determined tables for the load-carrying capacity given by CECM (Convention Européenne de la Construction Métallique).
For small values of λ_r, values of e obtained are less than the assumed geometrical imperfection of $1/1000$ of the buckling length. The ration has no practical importance, since columns in this slenderness are relatively unaffected by small geometrical imperfections.

PILE-SUPPORTED OFFSHORE STEEL STRUCTURES

TRANSLATION INTO ENGLISH OF DANSK INGENIØRFORENING'S CODE OF PRACTICE

1. EDITION APRIL 1983
DANSK STANDARD DS 449

1. INTRODUCTION

A pile supported offshore steel structure is in conformity with Dansk Ingeniøring structural design codes when it satisfies the requirements of this code and the requirements of DS 409 The Safety of Structures.
The code applies to pile-supported steel structures in the Danish part of the North Sea only.

> Guide: pile-supported offshore steel structures include pile-supported steel platforms, decks, accommodations, production and processing modules as well as bridges.

1.1 - General functional requirements

Reference is made to DS 409 The Safety of Structures.

1.2 – Basic concepts

Reference is made to DS 409 The Safety of Structures

1.2.1 – Safety classes

In general, offshore structures should be referred to high safety class, however, individual elements and structures may be referred to one of the other safety classes.

> Guide: where failure of a structure will involve negligible risk to life and will have minor consequences for society, the structure in question may be referred to normal safety class.
>
> An evaluation of the consequences for society should, among other thing allow for the effect on the surrounding environment and the reliability of oil and gas supply.
>
> Where failure of a structural member will involve negligible risk to life and will have minor consequences for society, structural member in question may be referred to a lower class than the structural itself.
>
> As examples of structural members which are not assumed to contribute to the load-carrying function of the main structure (independent structural members) and whose class is therefore determined independently of the main structure, mention can be made of deck plates (apart from heli-deck), stair, ladders, fenders, risers, mooring station, and rails.

1.5 – Symbols

C	shape factor
c	shear strength. Cohesion
d	diameter
E	modulus of elasticity
e	tolerance, initial deflection
f_y	yield stress
g	acceleration due to gravity
H	wave height
H_s	significant wave height
i	radius of gyration
l	length
n	number of actions
P	concentrated force
Re	Reynolds' number
T	wave period
μ	coefficient of friction
η	safety factor in Palmgren-Miner's formula
φ	angle of internal friction

5. SAFETY

5.1 – Limit states

Reference is made to DS 409 The Safety of Structures with the alterations and additions listed below.
The code presupposes that verification of safety is made by calculation and by use of the partial coefficient method.

5.1.1 Ultimate limit states

It should be demonstrated by means of calculations that the structure does not exceed the following ultimate limit states:

- Limit states of failure in parts of the structure due to instability (deflection of bars, lateral instability, or deflection of plates) or large, continuously increasing deformations which make it impossible to use the structure. In the following this limit state is termed *elastic ultimate limit state.*
- Limit state of total failure of the structure due to failure of foundation or progressive collapse. In the following this limit state is termed *plastic ultimate limit state.*
- Limit states of fatigue failure due to repeated action variations. In the following this limit state is termed *fatigue ultimate limit state*.

The calculations should be carried out with action combinations and partial coefficients as stated in section 5.2 and partial coefficients for material parameters and resistances as stated in Table 4

Allowance should be made for any action variation to which the structure will be exposed, the plastic deformations of the soil caused by action variations, possible changes in the sea floor level (scour) as well as the strength and deformation properties of the soil.

Elastic ultimate limit state

Normally the calculations should be carried out according to the theory of elasticity with allowance for the elastic/plastic properties of the soil.
Repeated plastic deformations (yielding) in the structure are not permissible.
In the calculations, the following should be demonstrated:

- That each action combination has a design reversible condition in which the actions can be absorbed by the structure and transferred to the soil without additional plastic deformations in the soil,
- That the stress at any point in the structure with the above-mentioned action combinations does not exceed the design yield stress and the design tensile strength, and
- That all structural members possess the required safety against stability failure.

Guide: the analysis to establish whether repeated plastic deformations occur may be undertaken by means of a shake-down analysis.

As an alternative to the shake-down analysis, a conservative but less comprehensive calculation may be made in a static, elastic state based on the following conditions:

- Utilization of the structure and piles is permitted up to the design strength only at one point and/or one pile or at several points/piles if it occurs simultaneously,
- The structure should be designed for extreme actions as defined in chapter 4, imposed as static actions,
- The influence of the alternating action on the strength and deformation properties of the soil should be taken into consideration when determining the characteristic parameters,
- The state of deformation of the soil should be determined for the most detrimental condition that may occur as result of possible action on the structure, and
- The sea floor level should be determined at the most detrimental level as a result of possible transport of sediments and erosion.

In the calculation of the stress resultants and pile reaction of the structure, the piles should be assumed to be restrained in the linear elastic structure. Each of the pile is characterized by a non-linear connection between the resulting force at the head of the pile and the corresponding displacements.

Therefore, both pile reactions and stress resultants in the structure have to be determined by an iterative calculation.

Plastic ultimate limit state

Calculations may be carried out according to the theory of plasticity and a fully developed ultimate limit state is permitted for ach action combination.

> Guide: this ultimate limit state primarily concern the design of the piles for the structure. Normally the structure itself can be regard as a rigid body in connection with a calculation of pile reactions based on the theory of plasticity. It should be demonstrated that the pile reactions can be absorbed by the structures.
>
> The individual members are permitted to yield or fail completely provided that the action can be absorbed by the structure as a whole.
>
> The design distribution of reactions on the individual piles may be undertaken regardless of the deformation conditions provided that actions and pile reactions constitute a force system in equilibrium on which the determination of the design ultimate limit state of both piles and structure should be based.

5.1.2 -.Serviceability limit states

The code does not make any specific requirements for the size of deformations, settlements, etc. which should be determined solely on the basis of serviceability criteria.

> Guide: the calculations of serviceability limit states should normally be carried out in accordance with the theory of elasticity with due allowance for non-linear material properties.

5.2 The partial coefficient method

Reference is made to DS 409 The Safety of Structures.

5.2.1 Action combinations

The design should make allowance for the action combinations stated in Table 3 together with the relevant partial coefficients.
In action combination 3.1, the action from waves, current, and wind should be determined so that the probability of transgression does not exceed $1/3$ within one year provided that a warning system is established for waves and wind during the period for which the structure is under repair. In the absence of a warning system, the probability of transgression should not exceed $1/7$ within one year. If the time of repair is assessed to last more than one year, the above-mentioned probabilities of transgression should be divided by the time of repair in years.
In action combination 3.2, waves, current, wind, snow, and ice may be disregarded and only one accidental action at a time should be considered. If it can be demonstrated that the structure can absorb action combination 3.2 without failure of any of structural members, action combination 3.1 may be omitted. In the design against fatigue, allowance should be made for all actions accruing in such combinations that a realistic picture is given of the variations in size and frequency of stresses in the individual structural members during the entire life of the structure. The fabrication, transportation, and installation phase should be included in the life of the

structure. In the design against fatigue, the partial coefficients for all actions should be assumed to be 1.0.

Table 3 – Action combinations and partial coefficients for actions

Type of action	Action combination								
	Use	Failure				failure	failure	accident	
		2.1				2.2	2.3	3.1	3.2
		a	b	c	d	a,b,c,d			
Permanent action									
Dead load	1.0	1.0	1.0	1.0	1.0	0.9	1.15	1.0	1.0
Dead load of soil and hydrostatic water pressure	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Variable action									
Imposed action	1.0	1.3[1]	1.0	1.0	1.0	as 2.1 a,b,c,d	0	1.0	1.0
Natural action Waves and current	ψ[2]	0.75	1.3	1.0	0.75		0.0	1.0[3]	0
Wind	ψ[2]	0.5	1.0	1.3	0.5		0.0	1.0[3]	0
Snow and ice	ψ[2]	0.5	0.5	0.5	1.3		0	0	0
Action from deformation	1.0	1.0	1.0	1.0	1.0	1.0	0	0	0
Accidental action	0	0	0	0	0	0	0	0	1.0

5.2.2 – Partial coefficients for material parameters

[1] In the determination of the imposed action from several platform decks, the value of 1.3 should be applied to the imposed action on one deck only, while the value of 1.0 should be applied to the action on the other decks. The value of 1.3 should be applied to the imposed action on the deck which gives the most detrimental combination of imposed actions on the individual decks.
[2] The values of natural actions which should be used in an analysis of serviceability limit states should be determined considering the consequences of the serviceability limit state being exceed. The action values may be determined from the characteristic actions used in the action combinations 2 by multiplying these actions by a factor of ψ<1. Alternatively, the actions of serviceability limit states may be determined on the basis of probabilities of transgression other than those used as a basis for the characteristic actions (98 percentile).
[3] The values of the actions should be determined as specified in the code text.

5.2.2.1 Serviceability limit states

The partial coefficients should be determined dependent on the consequences of transgression of the serviceability limit state in question. Normally, the partial coefficients are taken as 1.0.

5.2.2.2 Elastic and plastic ultimate limit state

In the action combinations 2 and 3.1, the values of the partial coefficients stated in Table 4 a should be used for steel parameters and geotechnical parameters, respectively.
The partial coefficients will depend on the safety class to which the structure/structural member in question is to be referred, see section 1.2.
In action combination 3.2 the partial coefficients are assumed to be 1.0 irrespective to safety class.

Table 4 – a Partial coefficients for steel parameter action combinations 2 and 3.1

	Safety Class	
Material parameters	Normal	High
f_y (yield stress)		
Normal material control	1.15	1.28
Strict material control	1.09	1.21
f_u (tensile strength)		
Normal material control	1.41	1.56
Strict material control	1.34	1.48
τ_g (punching strength)		
Normal material control	1.28	1.41
Strict material control	1.21	1.34
E (modulus of elasticity)	1.34	1.48
μ (coefficient of friction)		
Normal friction joints	1.09	1.21
Unlimited slip possible	1.34	1.48

For structures of low safety class, the partial coefficients should be determined in accordance with DS 412 The Structural Use of Steel.

> Guide: the partial coefficient for E should only be used for an analysis of the stability of the structure, including the stability of the individual structural members, as well as second order effects.

5. DESIGN AND CONSTRUCTION

6.3 – Steel structures

The structural design and the design verification of safety against failure should be in accordance with the regulations of DS 412 The Structural Use of Steel with the alterations and additions listed below

In the determination of stress resultants, allowance should be made for all eccentricities occurring. The influence on the distribution of stress resultants from deformations and from the actual rigidity of joints should be evaluated.

6.3.1 Structural members

6.3.1.1 Determination of stresses in tubular members

For tubular members, the stresses can normally be determined on the basis of the beam theory.
If there are any transition cones, correct allowance should be made for the changed distribution of stresses in the transition zones between the cylindrical and the conical parts.
If there are any reinforcing ribs or other reinforcing diaphragms in tubular members, allowance should be made for the changed distribution of stresses in the structural member as a result of the reinforcements.

6.3.1.2 Stability conditions for tubular

In an examination of total failure, the calculation of a column load-carrying capacity of a centrally influenced or moment-influenced tubular member should be in accordance with DS 412 The Structural Use of Steel. A slenderness ratio l/i exceeding 200 is not permitted.

> Guide: structural members on which major welding have been performed in the central part (pipe supports, anodes, fenders, ladders, etc.) without subsequent stress relieving should be treated as welded sections that have not been subject to stress relieving, see DS The Structural Use of Steel

An examination of local failure should include the following instances of failure:

- Buckling of pipe wall including reinforcements, if any
- Buckling of pipe wall between reinforcements, and
- Buckling of local reinforcements.

Locally reinforced tubular member should be designed in such a way that buckling of the section wall between reinforcements will occur before the general local buckling, i.e. before buckling of the section wall including reinforcements.

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TRANSLATION INTO ENGLISH OF DANSK INGENIØRFORENING'S CODE OF PRACTICE

PART 2, ANNEXES A-F
SUPPLEMENTARY GUIDE
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ANNEX D – STABILITY AND STRENGTH CONDITIONS OF TUBULAR MEMBERS AND TUBULAR JOINTS

D.1 Structural members

D.1.1 – General

This section contains guides for an evaluation of total failure and local failure of tubular members with a circular cross-section.
For tubular members with a maximum distance of

$$l \leq 0.95r\sqrt{\frac{r}{t}}$$

between points where the cross-section can be assumed to be restrained partly to prevent buckling as a whole partly to prevent local deformation, the case of total failure and the case of local failure may be considered independently of each other.
The load-carrying capacity of a structural member should thus be determined as the smaller of the load-carrying capacity of columns and the load-carrying capacity of the structural member in relation to local failure.
Total failure (the load-carrying capacity of columns) may be dealt with as a standard in DS 412 The Structural Use of Steel, whereas the load-carrying capacity in relation to local failure should be dealt with as stated in paragraph D.1.2.
For tubular members with a greater distance between the points of restraint than stated above, there may be an interaction between the cases of total failure and local failure which will reduce the resulting load-carrying capacity of the structural member.
The load-carrying capacity for combined total failure and local failure should in this case be determined by substituting the local design critical compressive stress σ_{cr} in the expressions for the load-carrying capacity of columns or beam columns of DS 412 The Structural Use of Steel as determined in formulas D.1.2.a, D.1.2.b, or D.1.2.c for the design yield stress of the steel f_{yd} .
For centrally loaded compression members this substitution corresponds formally to a reduction of the ordinate values of the columns curves, and for beam columns it corresponds to a reduction of the design strength.

D.1.2 - Local stability

Safety against local buckling of tubular members can be investigated by using the formulas of load-carrying capacity listed below.
Indirectly, these formulas make allowance for the effect of residual stresses and certain geometrical imperfections.

D.1.2.1 – Tolerances

The load-carrying capacity formulas for local stability presuppose that the following fabrication tolerances are observed:

Between circular welds:

- Measured along an arbitrary generatrix with a straight rail having a length of $l_r = 4\sqrt{rt}$, always provided that the length does not exceed 95 per cent of the distance between adjacent circular welds, the maximum deviation w should comply with $w/l_r < 0.01$, see figure D.1.2.1 a.
- Measured along an arbitrary circumference with a template having a radius of curvature equal to the theoretical outer radius of the cylinder and the length $l_r = 4\sqrt{rt}$, the maximum deviation w should comply with $w/l_r < 0.01$, see figure D.1.2.1 b.
- Measured along an arbitrary generatrix across the weld with a straight rail having a length of $l_r = 25t$, the maximum deviation w shall comply with $w/l_r < 0.01$, see figure D.1.2.1 c.

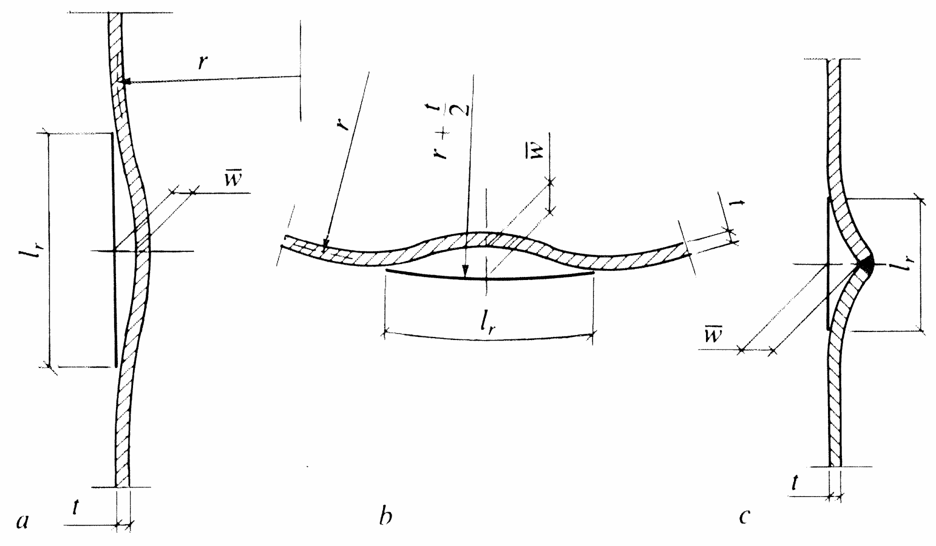


Figure D.1.2.1. Fabrication tolerances.

Where there is an interaction with an external excess hydrostatic pressure, the length of the radius of the pipe should not at any point differ by more than 0.5 per cent from the mean radius of the pipe. As regards control of the observance of this requirement, reference is made to BS 5500:1982.

If the imperfections are greater than stated above allowance should be made for this in the evaluation of the load-carrying capacity.

D1.2.2 Axial forces and moments

The local critical compressive design stress σ_{cr} for tubular, members subjected to axial forces and bending moments is found by:

$$\frac{\sigma_{cr}}{f_{yd}} = 1 \qquad \text{for } \lambda_a \leq 0.3$$

$$\frac{\sigma_{cr}}{f_{yd}} = 1.50 - 0.913\sqrt{\lambda_a} \qquad \text{for } 0.3 < \lambda_a \leq 1$$

where

$$\lambda_a = \sqrt{\frac{f_{yd}}{\varepsilon\sigma_{el}}}$$

$$\sigma_{el} = \frac{E_d}{\frac{r}{t}\sqrt{3(1-\nu^2)}}$$

$$\varepsilon = \frac{\varepsilon_a\sigma_{ad} + \varepsilon_b\sigma_{bd}}{\sigma_{ad} + \sigma_{bd}}$$

$$\varepsilon_a = \frac{0.83}{\sqrt{1 + 0.01\frac{r}{t}}}$$

$$\varepsilon_b = 0.1887 + 0.8113\varepsilon_a$$

D.1.2.3 - Hydrostatic over pressure

The local critical design pressure P_{cr} for tubular members subjected to an external hydrostatic excess pressure is found by:

$$P_{cr} = \frac{t}{r}\sigma_{cr}$$

Verification:

$$\sigma_c = \frac{p_d D}{2t} < \sigma_{cr}, \qquad \text{where } p_d = \rho g\left(H + D/2\right) \text{- design external pressure}$$

where

$$\frac{\sigma_{cr}}{f_{yd}} = k_1 - \sqrt{k_1^2 - k_2}$$

$$k_1 = 0.5\left(1 + \frac{1}{k_2} + 0.03\frac{r}{t}\right)k_2$$

and where

$$k_2 = \alpha C_1 \frac{0.855}{(1-\nu^2)^{0.75}} \cdot \frac{E_d}{f_{yd}} \cdot \frac{r}{l}\left(\frac{t}{r}\right)^{1.5} \qquad \text{for } 20\sqrt{\frac{t}{r}} \leq \frac{l}{r} \leq 1.63C_1\sqrt{\frac{r}{t}}$$

$$k_2 = \alpha \frac{E_d}{f_{yd}} \left(\frac{t}{r}\right)^2 \left[\frac{0.25}{1-\nu^2} + 2.03 \left(\frac{C_1}{\frac{l}{r}\sqrt{\frac{t}{r}}} \right)^4 \right] \qquad \text{for } \frac{l}{r} > 1.63 C_1 \sqrt{\frac{r}{t}}$$

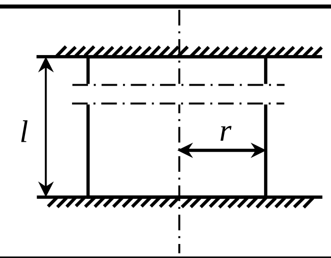
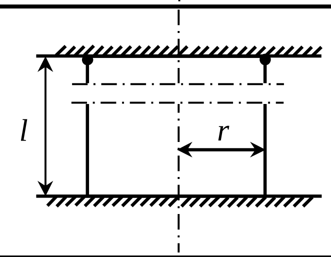
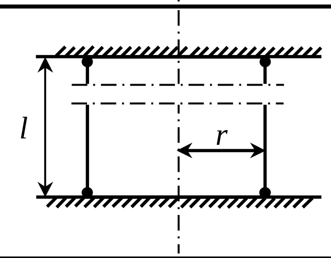
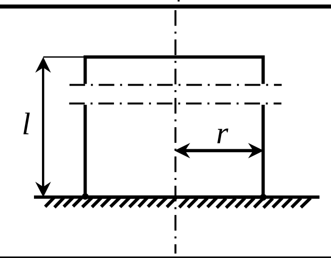
C_1 is determined by Table 5 and:

$$\alpha = 1 - \frac{1}{240} \cdot \frac{r}{t} \qquad \text{for } \frac{r}{t} \le 60$$

$$\alpha = 0.75 \qquad \text{for } \frac{r}{t} > 60$$

The above expression can be used for $l/r > 10$. For shorter cylinders the equations will lead to over-dimensioning and therefore, more exact methods of calculation are referred to in such cases.

Table 5 – Factor C_1 for a circular cylindrical shell as function of the boundary conditions

Boundary Conditions		C_1
	fixed fixed	1.4
	pinned fixed	1.2
	pinned pinned	1.0
	free fixed	0.6

D.1.2.4 – Combined actions

Where tubular members are simultaneously subjected to axial forces, bending moments and external hydrostatic over-pressure, the following formula should be used:

$$\frac{\sigma_a + \sigma_b}{(\sigma_{cr})_{ab}} + \left(\frac{p_d \frac{r}{t}}{(\sigma_{cr})_p} \right)^2 \leq 1$$

where

$(\sigma_{cr})_{ab}$	should be calculated on the basis of D.1.2 a (Axial Force and Moments)
$(\sigma_{cr})_p$	should be calculated on the basis of D.1.2 b (Hydrostatic Over-Pressure)
p_d	the design external pressure

In the case of combined total failure and local failure,

$$\sigma_{cr} = (\sigma_{cr})_{ab} \left(1 - \left(\frac{p_d \frac{r}{t}}{(\sigma_{cr})_p} \right)^2 \right)$$

should be used instead of the design yield stress of the steel.

D.2 Welded tubular joints

D2.1 – General

This section contains instructions as to low the strength of tubular joints may be demonstrated. The instructions apply only to joints that are not reinforced since it is not possible to provide general expressions of load-carrying capacity for joints reinforced by ribs, diaphragms, and the like.

The instructions concern plane Y, X, and K-shaped joints, see Figure 1, however, section D.2.6 contains instructions as to how they may be adapted to special joints.

D.2.2 – Welds

Not applicable.

D.2.3 – Design

The instructions presuppose that the minimum dimensions of the length of local reinforcements, by way of increasing thickness of material or stronger steel, are observed (as shown in Figure 2) The instructions do not cover K-joints with a free space between the braces of less than 50 mm or K-joints with an overlap b of less than 75 mm.

$a \geq 50\,\text{mm}$

$b \geq 75\,\text{mm}$

$l_1 \geq d_h/4$ and $l_1 \geq 300\,\text{mm}$

$l_2 \geq d_a/4$ and $l_2 \geq 600\,\text{mm}$

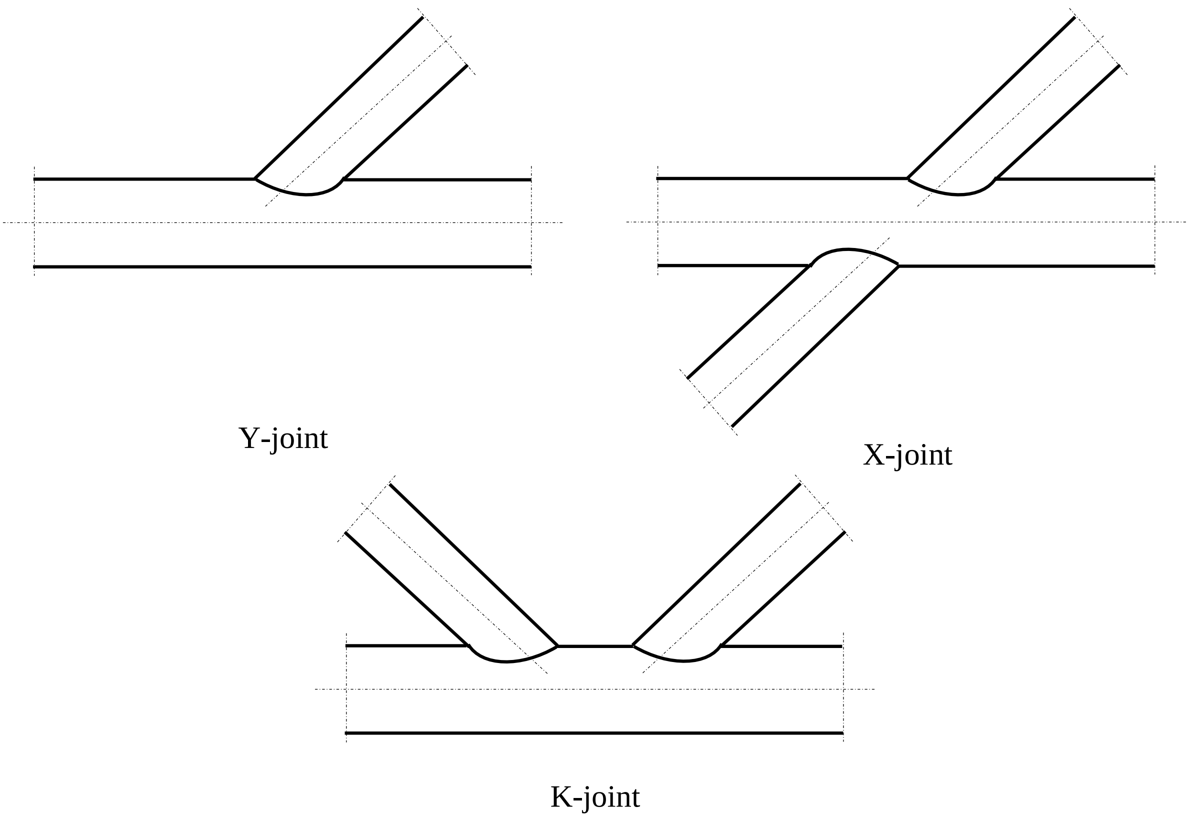


Figure 1 - Plane type joints

$0.2 \leq \beta \leq 1$

$10 \leq \gamma \leq 50$

$\tau \leq 1$

$30^\circ \leq \theta \leq 90^\circ$

The expressions for load-carrying capacity in section D.2.4 are based on test results within the parameter intervals of β, γ, τ, and θ stated in Figure 2. The expressions should not be used outside these intervals.

D.2.4 – Load-carrying capacity of joints

The characteristic load-carrying capacity of the joints as regards thrust and being moments in the connected braces should be determined by:

$$N_u = \frac{f_y t_h^2}{\sin\theta} C\mu \qquad \text{(D.2.4.1)}$$

$$M_u = \frac{f_y t_h^2}{\sin\theta} 0.8 d_a C\mu \qquad \text{(D.2.4.2)}$$

where

t_h is the thickness of the chord of the joint

$\mu = 1.22 - 0.5 \frac{|\sigma_{0d}|}{f_y / \gamma_m}$ always provided that it does nor exceed 1.0

γ_m is the partial coefficient of the punching strength

C should be determined by means of Table 6.

σ_{0d} design maximum axial stress in the chord at joint

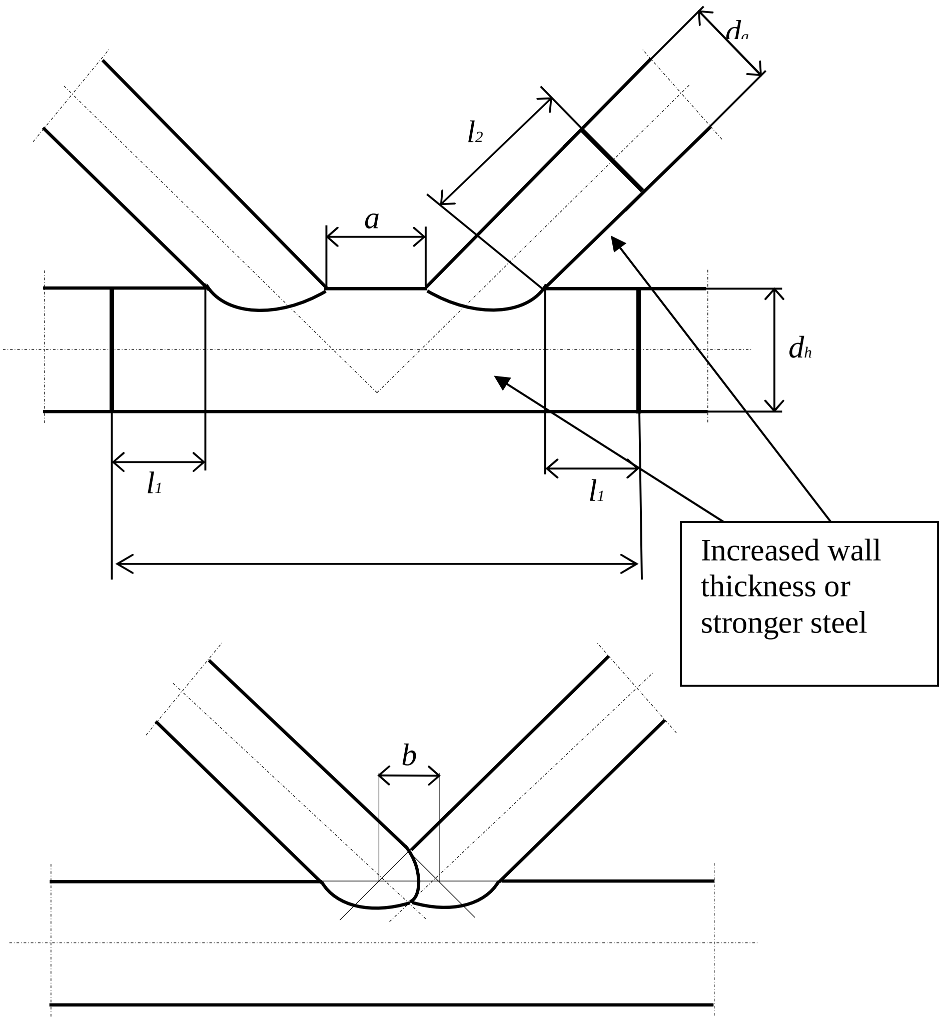


Figure 2 – Design requirements

Table 6 – Arithmetic value of *C*

type of action	type of joint		
	Y	X	K
N, tension	$3.4+19\beta$	$3.4+19\beta$	$(3.4+19\beta)C_\varsigma$
N, compression	$3.4+19\beta$	$(3.4+13\beta)C_\beta$	$(3.4+19\beta)C_\varsigma$
M_1	$3.4+19\beta$	$3.4+13\beta$	$3.4+19\beta$
M_2	$(3.4+7\beta)C_\beta$	$(3.4+5\beta)C_\beta$	$(3.4+7\beta)C_\beta$

$$C_\beta = \begin{cases} \dfrac{0.3}{\beta\left(1-\dfrac{5}{6}\beta\right)} & \text{for } \beta \geq 0.6 \\ 1.0 & \text{for } \beta < 0.6 \end{cases}$$

$$C_\varsigma = \begin{cases} 1.8 & \text{for } \varsigma < 0 \\ 1.8-0.8\varsigma & \text{for } 0<\varsigma<1 \\ 1.0 & \text{for } \varsigma \geq 1 \end{cases}$$

where

$$\varsigma = \frac{a}{d_{am}}$$

d_{am} is the mean of the two brace diameters of the K-joint.

The arithmetic value of C for thrust applies to:

- X-joints where the thrust of the diametrically opposite braces is in equilibrium
- K-joints where the components of the thrust of the braces perpendicular to the chord on the same side of the chord are in equilibrium
- Y-joints where the thrust of the brace is carried by the chord alone.

Where a joint carries part of the thrust of the braces as a K-joint and part of it as a Y-joint and/or X-joint, the size of C values of the individual types corresponding to their share of the thrust.

D.2.5 – Control of load-carrying capacity of chord and overlapping braces

It should be demonstrated that the wall of the chord has sufficient load-carrying capacity to comply with the following condition for each tubular joint and each individual combination

$$\frac{N_d}{N_u/\gamma_m} + \frac{2}{\pi}\arcsin\sqrt{\left(\frac{M_{1d}}{M_{1u}/\gamma_m}\right)^2 + \left(\frac{M_{2d}}{M_{2u}/\gamma_m}\right)^2} \leq 1$$

where

γ_m is the partial coefficient of the punching strength.

M_{1d}, M_{2d} design bending moments in braces that bend in and out of the place of the joint.

M_{1u}, M_{2u} characteristic strength of joint corresponding to M_{1d} and M_{2d}.

In the case of overlapping braces, it should in a similar way be demonstrated that the load-carrying capacity of the wall of the overlapping brace is sufficient.

The demonstration of load-carrying capacity may be based on reduced stress resultants corresponding to including only statically required moments.
Reduction of the stress resultants is allowed only where the joint has the presupposed strength after having been exposed to the plastic deformations required for a redistribution of the stress resultants.
Reduction of moments resulting in bending out of the plane of the joint should be effected with care and only where $\beta < 0.85$.
In the calculations of the load-carrying capacity of the brace, the fixed-end moments of the nodes should not be greater than those included in the demonstration of the load-carrying capacity of the joints.

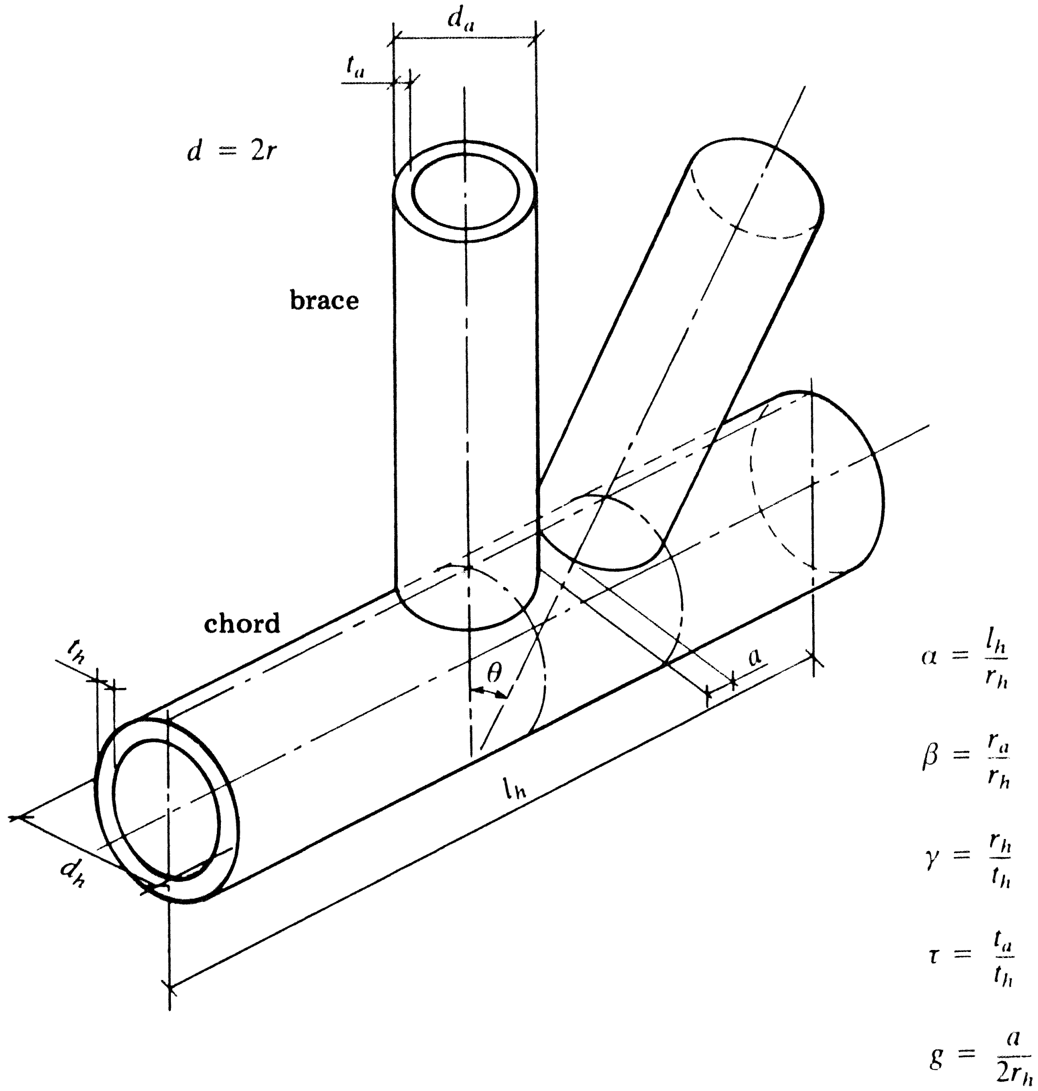


Figure E.2.2. Geometry and parameters of tubular joints.

D.2.6 – Non-plane joints

The general practice is to regard each plane in the joint with the axis of the chord as an independent plane joint not influenced by forces from the other planes of the joint.

D.3 List of symbols

a distance between braces

b length of overlap

d_{am} mean diameter of brace in K-joint

f_{yd} design yield stress

l_1, l_2 part of the length of reinforced tubular section

l_r length between points of measurement

M_{1d}, M_{2d} design bending moments in braces that bend in and out of the place of the joint.

M_{1u}, M_{2u} characteristic strength of joint corresponding to M_{1d} and M_{2d}

N_d design thrust in brace

N_u characteristic strength of joint corresponding to N_d

p_{cr} critical hydrostatic design pressure

r mean radius

t thickness of material

w fabrication tolerance

β ratio between the radii of the brace and the chord

γ ratio between the radius and the wall thickness of the chord

ς $= \frac{a}{d}$ or $\left(\frac{\pi r}{l}\right)^2$

σ_{ad} design stress caused by axial forces

σ_{bd} design stress caused by bending moments

σ_{el} critical compressive design stress based on the theory of elasticity

σ_{0d} design maximum axial stress in the chord at joint

τ ratio between the wall thickness of brace and chord