Strength and stability of aluminium members according to EN 1999-1-1 – Eurocode 9

Torsten Höglund
Royal Institute of Technology
Stockholm

Design values of loads and resistances

Design values of loads are given in Eurocode 0 and 1.

Eurodode 9 gives the design values of resistance at the ultimate limit state, e.g.

$$M_{\mathrm{Rd}} = \frac{M_{\mathrm{Rk}}}{\gamma_{\mathrm{M}}} = \frac{W_{\mathrm{el}} f_{\mathrm{o}}}{\gamma_{\mathrm{M1}}}$$ (class 3 cross section)

M_{Rd}	design value of bending moment resistance
M_{Rk}	characteristic value of bending moment resistance
$f_{\mathrm{o}} = R_{\mathrm{p0.2}}$	characteristic value of **0,2 % proof strength**
$\gamma_{\mathrm{M1}} = 1{,}1$	partial factor for **general yielding**
W_{el}	section modulus

For class 4 cross sections (slender sections, sections with large width/thickness ratio) W_{el} is replaced by W_{eff} for the effective cross section. However, if the deflection at the serviceability limit state is decisive then a simplified method may be used; see page 17.

Design values of loads and resistances

In a section with reduced strength due to welding (heat affected zone, HAZ)

$$M_{\mathrm{Rd}} = \frac{W_{\mathrm{el}}\rho_{\mathrm{u,haz}} f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}}$$

(in a section with HAZ across the section)

M_{Rd}	design value of bending moment resistance
f_{u}	characteristic value of **ultimate strength**
$\gamma_{\mathrm{M2}} = 1,25$	partial factor for failure
$\rho_{\mathrm{u,haz}}$	reduction factor for the **ultimate strength in HAZ**

Material properties

Table 3.2b - Characteristic values of 0,2% proof strength f_o and ultimate tensile strength f_u (unwelded and for HAZ), min elongation A, reduction factors $\rho_{o,haz}$ and $\rho_{u,haz}$ in HAZ, buckling class and exponent n_p for wrought aluminium alloys - Extruded profiles, extruded tube, extruded, rod/bar and drawn tube

Alloy EN-AW	Product form	Temper	Thickness t mm 1) 3)	f_o 1)	f_u 1)	A 5) 2)	$f_{o,haz}$ 4)	$f_{u,haz}$ 4)	HAZ-factor 4)		BC 6)	n_p 7)
				N/mm²		%	N/mm²		$\rho_{o,haz}$	$\rho_{u,haz}$		
6082	EP,ET,ER/B	T4	$t \le 25$	110	205	14	100	160	0,91	0,78	B	8
	EP/O, EP/H	T5	$t \le 5$	230	270	8	125	185	0,54	0,69	B	28
	EP/O,EP/H	T6	$t \le 5$	250	290	8			0,50	0,64	A	32
	ET	T6	$5 < t \le 15$	260	310	10	125	185	0,48	0,60	A	25
	ER/B	T6	$t \le 20$	250	295	8			0,50	0,63	A	27
			$20 < t \le 150$	260	310	8			0,48	0,60	A	25
	DT	T6	$t \le 5$	255	310	8			0,49	0,60	A	22
			$5 < t \le 20$	240	310	10			0,52	0,60	A	17

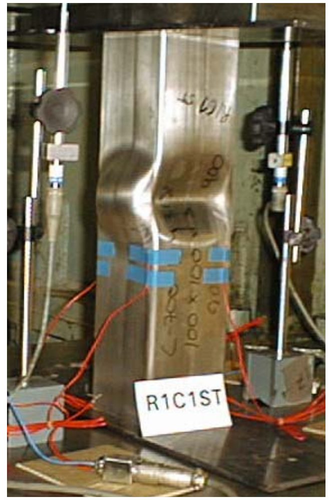
EP/O = Extruded open profiles

Part of Table 3.2 b.

Example: EN-AW 6082 T6, EP/O $t \le 5$ mm
Characteristic values

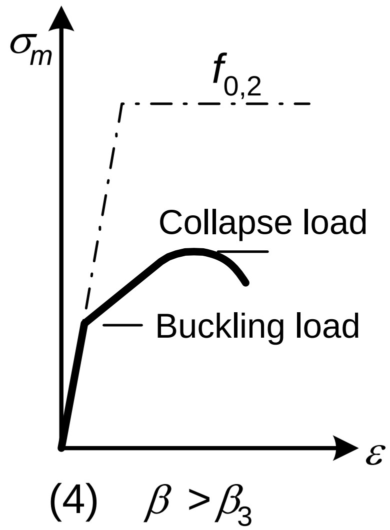
f_o = 250 MPa	0,2-proof stress
f_u = 290 MPa	ultimate strength
A_{50} = 8 %	elongation
$\rho_{o,haz}$ = 0,50	reduction factor for f_o in haz
$\rho_{u,haz}$ = 0,64	reduction factor for f_u in haz
BC = A	buckling class

Local buckling behaviour / cross section class 4

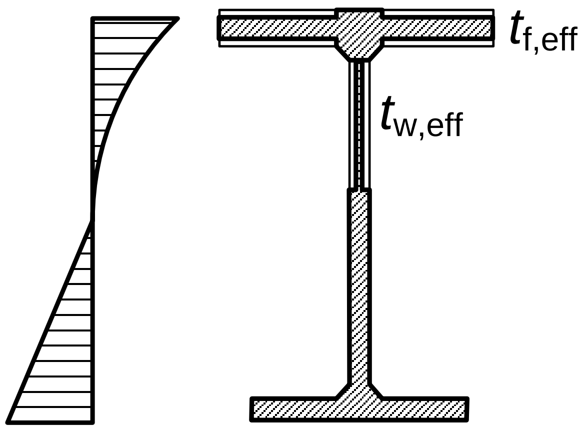


Except for massive sections and very stocky sections local buckling will occure in compressed parts at failure. However, the behaviour is different depending on the slenderness $\beta = b/t$ where b is the width and t is the thickness of the cross section part.

If $\beta > \beta_3$ where β_3 is roughly 6 for an outstand part and 22 for an internal part, then local buckling will occure before the compressive stress reach the 0,2 % proof stress f_o. Such a section part is called slender and the cross section is referred to as Class 4 cross section.



For very slender sections there is a post-buckling strength allowed for by using an effective cross section.

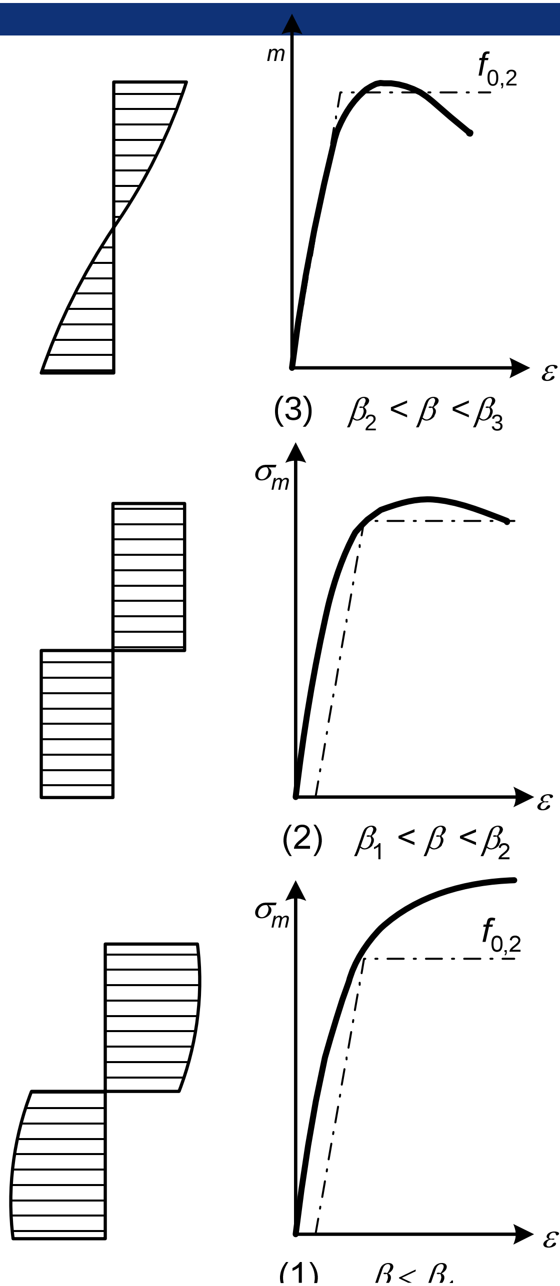


Cross section class 3, 2 and 1

If β for the most slender part of the cross section is $\beta < \beta_3$ and $\beta > \beta_2$ where β_2 is roughly 4,5 (16), then the cross section belong to class 3, non slender section. Then buckling will occur for a stress equal to or somewhat larger than f_o and some part of the cross section closer to the neutral axis (webs) may be larger than according to the theory of elasticity (linear stress distribution).

If β for the most slender part is less than β_2 then also parts of the cross section close to the neutral axis will reach f_o (class 2).

If $\beta_{max} < \beta_1$ = 3 (11) then rotation capacity is large enough for redistribution of bending moment using plastic global analysis (class 1).



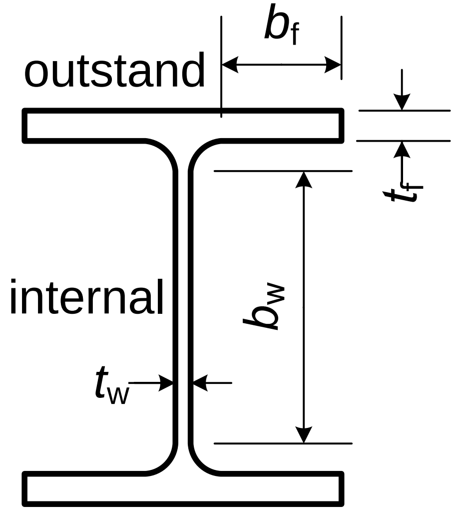
Local buckling - slenderness limits

The above given limits β_3 , β_2 and β_1 are valid for material buckling class A and f_o = 250 N/mm^2. For buckling class B and welded sections the limits are smaller.

(Buckling class is defined later)

Buckling class $\varepsilon = \sqrt{250/f_o}$	Internal part β_1/ε	β_2/ε	β_3/ε	Outstand part β_1/ε	β_2/ε	β_3/ε
A, without weld	11	16	22	3	4,5	6
A, with weld	9	13	18	2,5	4	5
B, without weld	13	16,5	18	3,5	4,5	5
B, with weld	10	13,5	15	3	3,5	4

For the web of a symmetric beam in bending $\beta = 0{,}4 b_w/t_w$

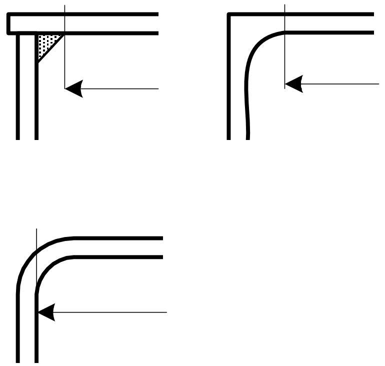


mm
$b_f = 70$
$t_f = 14$
$b_w = 90$
$t_w = 4$
$f_o = 250$

Example 1: Give cross section class

Loading	**Cross section class**			
	1	**2**	**3**	**4**
Axial compression			flange	web
Bending	web		flange	

Internal / outstand cross section part



For outstand cross section parts, *b* is the width of the flat part outside the fillet. For internal parts *b* is the flat part between the fillets, except for cold-formed sections and rounded outside corners.

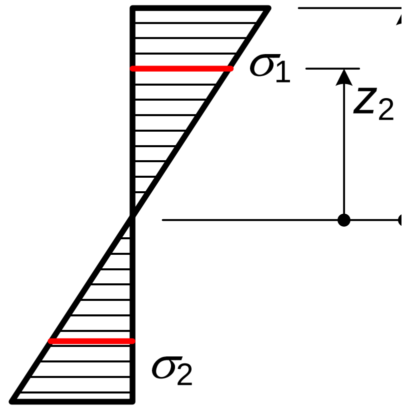
Stress gradient

For cross section parts with stress gradient ($\psi = \sigma_2/\sigma_1$) then

$\beta = \eta\, b_w/t_w$ where

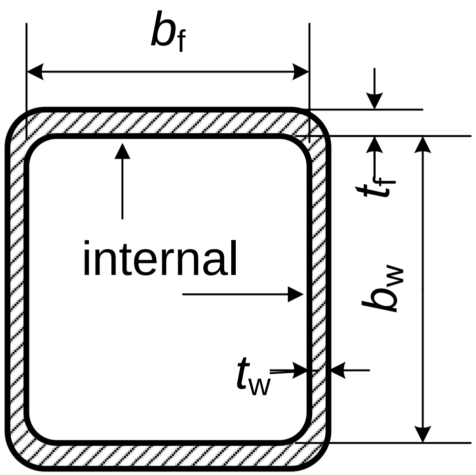
$\eta = 0{,}70 + 0{,}30\psi$ if $1 > \psi > -1$

$\eta = 0{,}80/(1-\psi)$ if $\psi < -1$



If the part is less highly stressed than the most severely stressed fibres in the section, a modified expression may be used for ε

$$\varepsilon = \sqrt{(250/f_o)\cdot(z_1/z_2)}$$



mm
$b_f = 140$
$t_f = 10$
$b_w = 180$
$t_w = 6$
$f_o = 250$

Example 2: Give cross section class

Loading	Cross section class 1	2	3	4
Axial compression			X	
Bending		X		

Axial force cross section resistance

For axial compression the cross section resistance (no flexural buckling) is the same for cross section ***class 1, 2 and 3***

$N_{\mathrm{Rd}} = A f_{\mathrm{o}} / \gamma_{\mathrm{M1}}$ where γ_{M1} = 1,1 = partial factor for material

For ***class 4*** cross section the cross section resistance is

$N_{\mathrm{Rd}} = A_{\mathrm{eff}} f_{\mathrm{o}} / \gamma_{\mathrm{M1}}$ where A_{eff} = area of effective cross section

This effective cross section is build up of section with effective thickness t_{eff} for the cross section parts that belong to class 4.

$t_{\mathrm{eff}} = \rho_{\mathrm{c}} t$ where ρ_{c} = reduction factor for local buckling $\rho_c = \frac{C_1}{(\beta / \varepsilon)} - \frac{C_2}{(\beta / \varepsilon)^2}$

Buckling class	Internal part		Outstand part	
	C_1	C_2	C_1	C_2
A, without weld	32	220	10	24
A, with weld	29	198	9	20
B, without weld	29	198	9	20
B, with weld	25	150	8	16

Bending moment resistance

For bending moment the formulae for the resistance is depending on cross section class. For ***class 2*** cross section the resistance is given by

$$M_{\mathrm{Rd},2} = M_{\mathrm{pl}} = W_{\mathrm{pl}} f_{\mathrm{o}} / \gamma_{\mathrm{M1}}$$ where W_{pl} = plastic section modulus $W_{\mathrm{pl}} = \sum A \cdot z$

For ***class 1*** cross section the resistance may be somewhat larger but M_{pl} is a good approximation.

For ***class 3*** cross section the resistance is somewhere between M_{pl} and M_{el} where

$$M_{\mathrm{el}} = W_{\mathrm{el}} f_{\mathrm{o}} / \gamma_{\mathrm{M1}}$$ with W_{el} = elastic section modulus $W_{\mathrm{el}} = I / e$

The actual resistance if found by interpolation

$$M_{\mathrm{Rd},3} = M_{\mathrm{el}} + (M_{\mathrm{pl}} - M_{\mathrm{el}}) \frac{\beta_3 - \beta}{\beta_3 - \beta_2}$$

However, in most cases M_{el} could be used as a conservative approximation

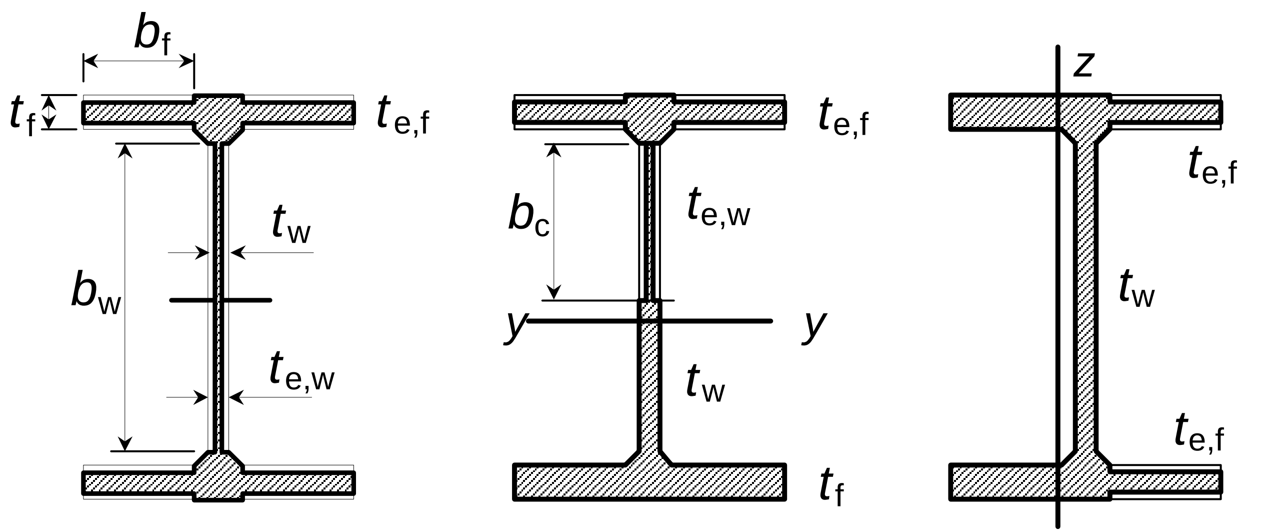
For ***class 4*** cross section the resistance is

$$M_{\mathrm{Rd},4} = W_{\mathrm{eff}} f_{\mathrm{o}} / \gamma_{\mathrm{M1}}$$ where W_{eff} = section modulus for effective cross section

Effective cross section

The effective cross section is different for axial force and bending moment.

No effective cross section is needed for the combined loading axial force and bending moment. The combination is solved using interaction formulae.



Effective section for axial compression

Effective section for y- axis bending

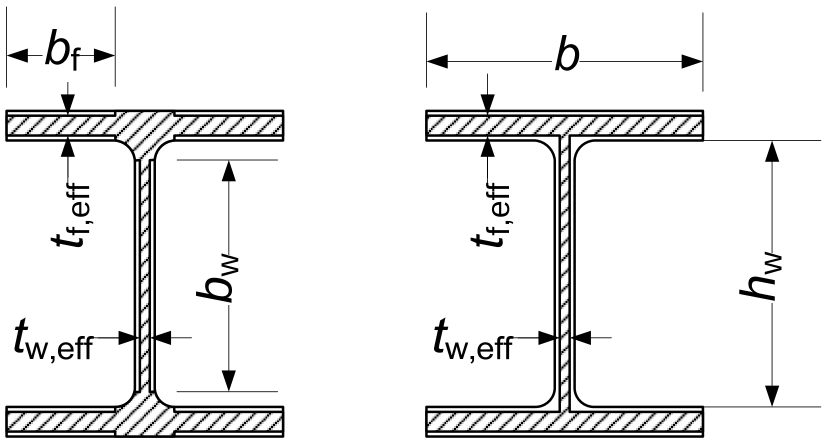
Effective section for z- axis bending

Effective cross section for axial force

The effective cross section is based on the effective thickness of the cross section parts.

If the cross section is symmetric, then the effective cross section is also symmetric.

If the cross section is asymmetric, then there might be a shift in the neutral axis. For axially compressed extruded profiles this shift is ignored i.e. the axial force is taken as acting in the centre of the effective cross section. For cold-formed sections the shift should be allowed for by adding a bending moment $\Delta M_{Ed} = N_{Ed} e_N$ where e_N is the shift in neutral axis for gross and effective cross section.



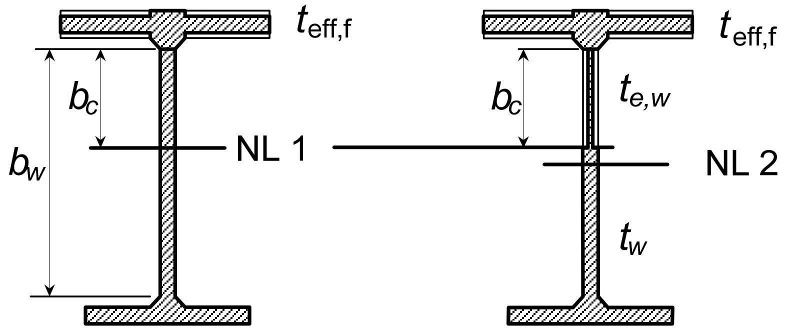
In principle only the flat parts between fillets need to be reduced, however, for simplicity, the whole flange or web may be reduced.

Effective cross section for bending moment

To find the effective cross section for bending moment is sometimes a tricky task and is not presented here in detail. Just a few comments:

- Local buckling may only occur on the compression side. For a member in bending, even if the cross section is symmetric, the effective section is asymmetric
- The neutral axis of the effective cross section is shifted closer to the tension side and the compressed part of the cross section is increased
- In principle an iteration procedure should be used, however, only two steps are necessary

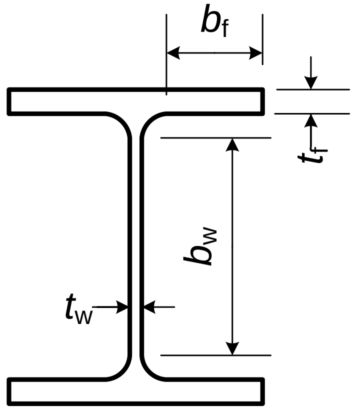
E.g. for an **I**-section the first step is to calculate the effective thickness of the compression flange and calculate the neutral axis for that section. The second step is to calculate the effective thickness of the web based on this neutral axis. This is then the effective cross section.



Step 1

Step 2, effective cross section

Effective cross section for bending moment



mm
$b_f = 70$
$t_f = 14$
$b_w = 90$
$t_w = 4$

From above we know the cross section class

Loading	Cross section class			
	1	2	3	4
Axial compression			**flange**	**web**
Bending	**web**		**flange**	

Compression, web $\rho_c = \frac{C_1}{(\beta/\varepsilon)} - \frac{C_2}{(\beta/\varepsilon)^2}$ $C_1 = 32,\quad C_2 = 220$ $\beta = 90/4 = 22{,}5$

$\varepsilon = 1,\ \beta_3 = 22$

$$\rho_c = \frac{32}{22{,}5} - \frac{220}{22{,}5^2} = 0{,}988$$

ρ_c is very close to one.
Use gross cross section

Compression and bending, flange $\beta = 70/14 = 5 \qquad \beta_3 = 6,\ \beta_2 = 4{,}5$

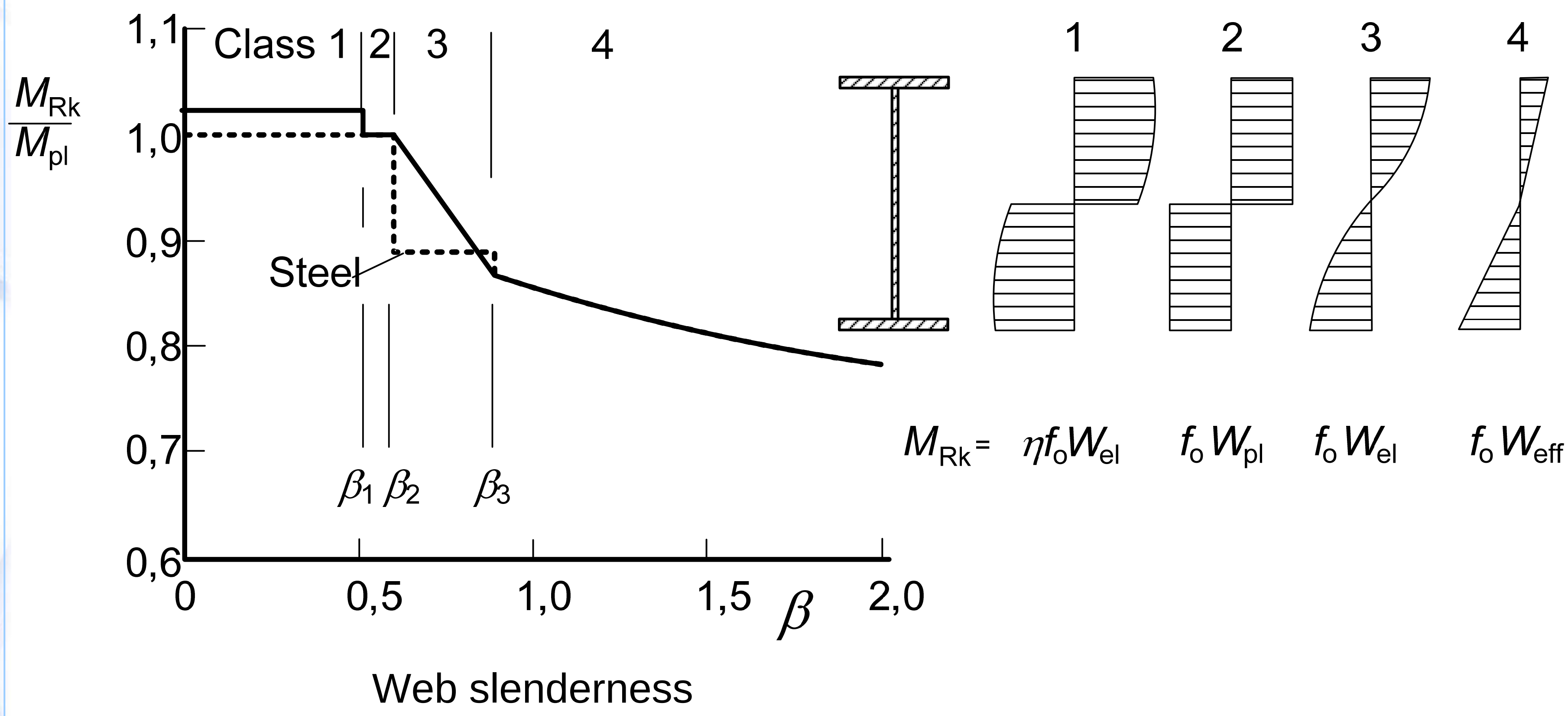
Which formula to be used?

$$M_{\mathrm{Rd},2} = W_{\mathrm{pl}} f_{\mathrm{o}} / \gamma_{\mathrm{M1}}$$

$$M_{\mathrm{Rd},3} = M_{\mathrm{el}} + (M_{\mathrm{pl}} - M_{\mathrm{el}}) \frac{\beta_3 - \beta}{\beta_3 - \beta_2}$$

$$M_{\mathrm{Rd},4} = W_{\mathrm{eff}} f_{\mathrm{o}} / \gamma_{\mathrm{M1}}$$

Summary for members in bending



If serviceability limit state is decisive

The relatively low elastic modulus of aluminium (compared to steel) means that the deflection at the serviceability limit state is often decisive. Then conservative design at the ultimate limit state can often be accepted.

For class 1, 2 and 3 cross section the resistance according to the theory of elasticity could be used e.g.

$$M_{\mathrm{Rd}} = \frac{W_{\mathrm{el}} f_{\mathrm{o}}}{\gamma_{\mathrm{M1}}}$$

corresponding to the horisontal line marked ”steel” on the previous slide.

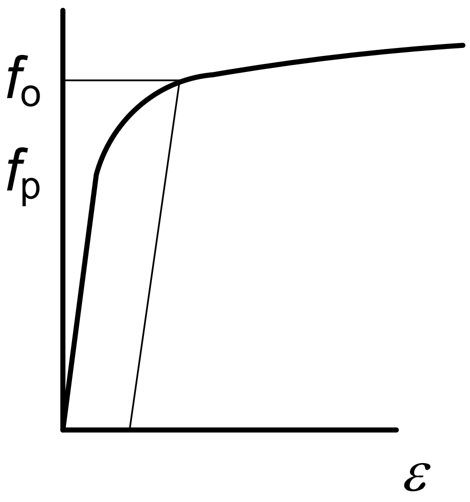
For class 4 cross section the resistance could be given by

$$M_{\mathrm{Rd}} = \rho_{\mathrm{c}} \cdot \frac{W_{\mathrm{el}} f_{\mathrm{o}}}{\gamma_{\mathrm{M1}}}$$

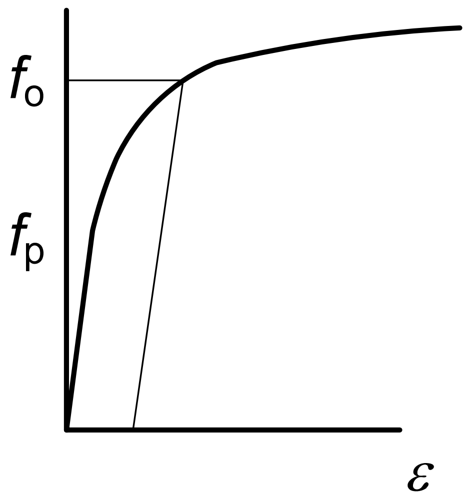
where ρ_c is the reduction factor for local buckling for the cross section part with the largest value of β / β_3. This might be rather conservative but no effective cross section need to be determined.

Buckling class

- Small residual stresses in extruded profiles mean that the buckling curves are not depending on the shape of the cross section (as for steel)
- Buckling curve depends on material and longitudinal welding
- Material buckling class A or B depends on the σ - ε–diagram for small strains (proportional limit - 0,2-proof stress ratio, f_p/f_o)
- Buckling class is given in Table 3.2 a and b

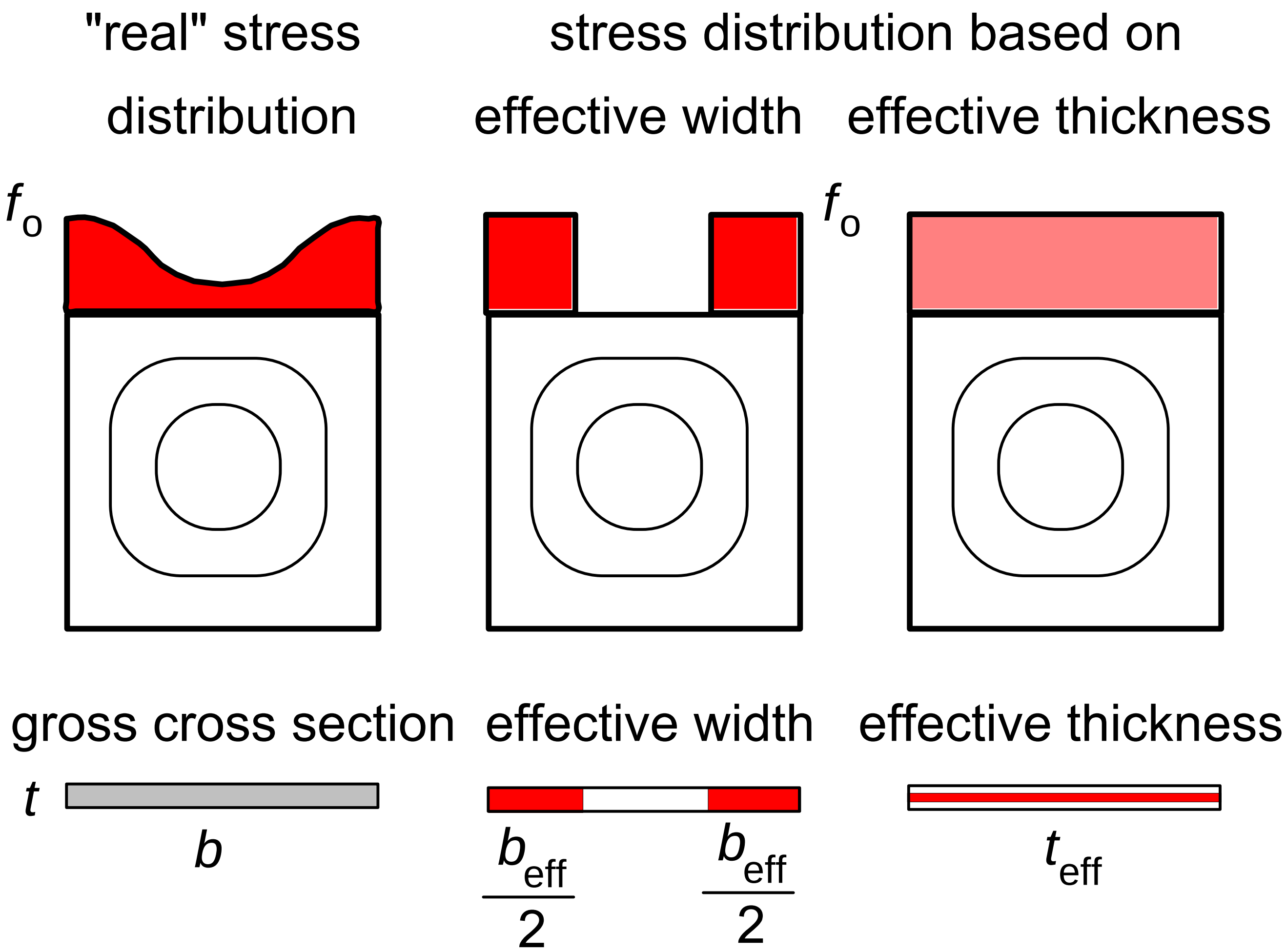


Buckling class A



Buckling class B

Effective width - effective thickness

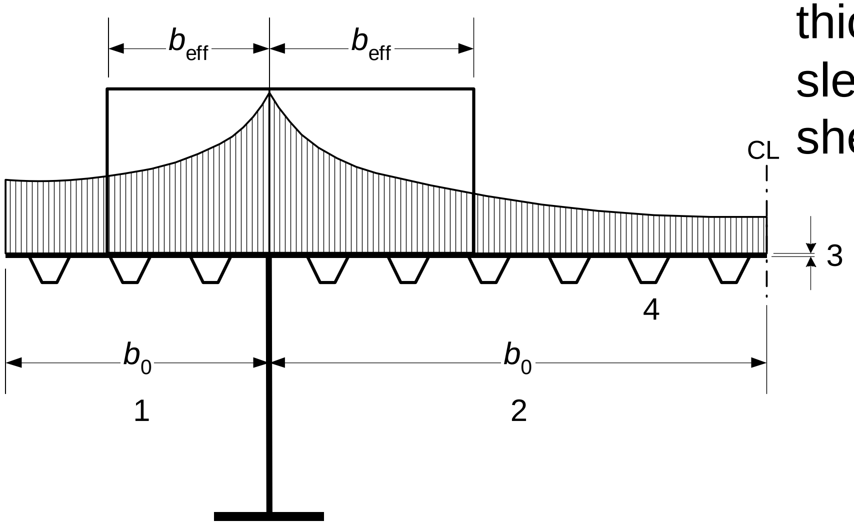


Why effective thickness?

Simple calculations

Easier to allow for combination of local buckling and HAZ

Easy to combine with shear lag where effective width is used



You only need to reduce the thickness, not to define start and stop of effective widths - especially important for aluminium profiles.

Within the HAZs the lesser of the reduction for local buckling and HAZ softening is used.

The effects of plate buckling on shear lag may be taken into account by first reducing the flange width to an effective width, then reducing the thickness to an effective thickness for local buckling basing the slenderness β on the effective width for shear lag. (National choice)

Heat Affected Zone, HAZ

Two reduction factors

$\rho_{o,haz}$ for 0,2 % proof strength and
$\rho_{u,haz}$ for ultimate strength

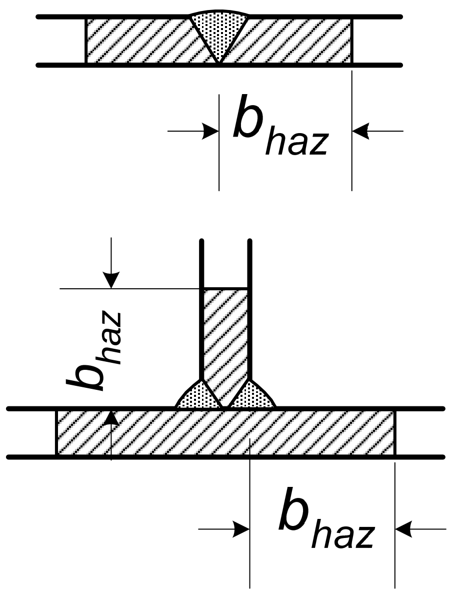
Example: Extruded profile, *t* < 5

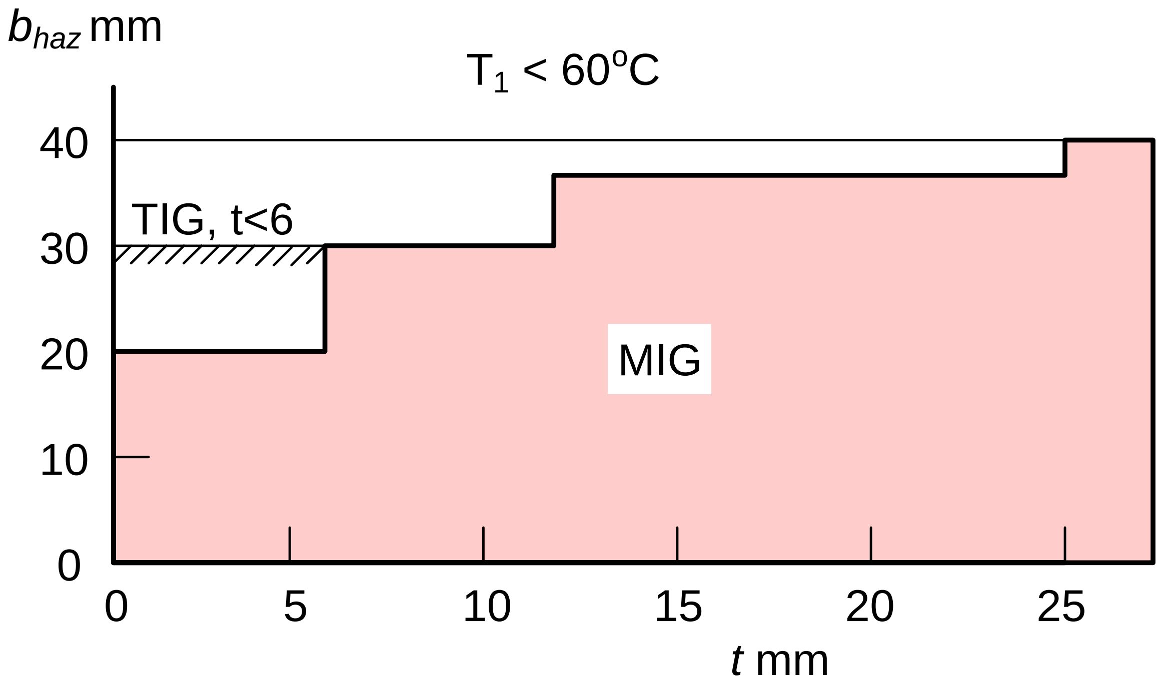
Alloy	Tem-per	0,2 % p. strength $\rho_{o,haz}$	Ultimate strength $\rho_{u,haz}$
6082	T4	0,91	0,78
	T5	0,54	0,69
	T6	0,50	0,64
7020	T6	0,71	0,80

Sheet, strip and plate, *t* < 5

Alloy	Tem-per	0,2 % p. strength $\rho_{o,haz}$	Ultimate strength $\rho_{u,haz}$
3005	H14	0,37	0,64
	H16	0,30	0,56
5754	H14	0,53	0,63
6082	T6	0,48	0,60

Width of heat affected zone





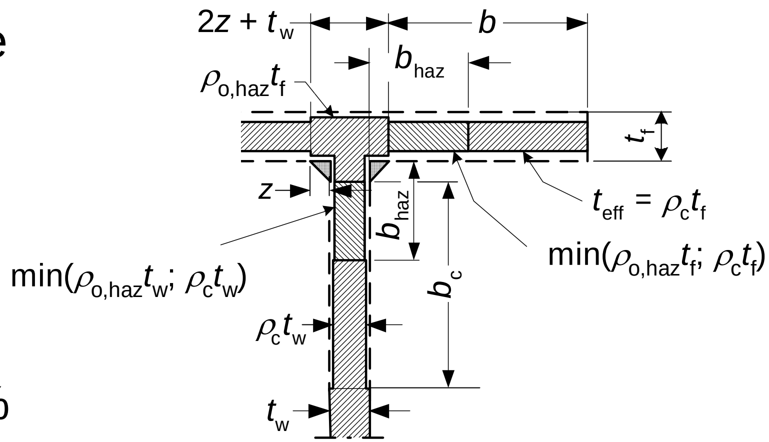
When $60^oC < T_1 < 120^oC$
multiply with

$1 + (T_1 - 60) / 120$	6xxx alloy
$1 + (T_1 - 60) / 80$	7xxx alloy

T_1 = interpass cooling temperature when multipass welds are laid

Longitudinally welded section

For a longitudinally welded section the loss of strength in the heat affected zone HAZ should be allowed for. The cross section classification is made as for extruded sections, except that the limits β_1, β_2 and β_3 are somewhat smaller.



For the resistance a reduced thickness is used within the widths b_{haz} of the HAZs

$$t_{\text{haz}} = \rho_{\text{o,haz}} t$$

where $\rho_{o,haz}$ is the reduction factor for the 0,2 % proof stress.

If the cross section belong to class 4 the effective thickness is the lesser of $\rho_c t$ and $\rho_{o,haz} t$ within b_{haz} and $\rho_c t$ besides HAZ.

Question 1: If a welded section is symmetric and belong to class 3 is then the reduced cross section due to HAZ asymmetric?

Question 2: If a welded section is symmetric and belong to class 4 is then the reduced cross section usually asymmetric?

Bending moment

Qu.	yes	no
1		X
2	X	

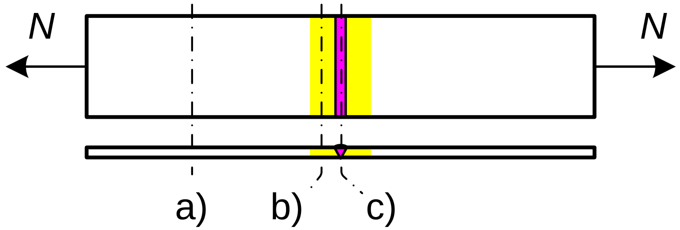
Member with transverse welds or welded attachments

For a member with a transverse cross weld the tension force resistance is the lesser of

a) The strength in the sections beside the weld and the HAZ

b) The strength in the HAZ

c) The strength of the weld



The strength of the sections besides the welds and the HAZs is based on the 0,2 % proof strength f_o whereas the strength in the HAZs is the ultimate strength $\rho_{u,haz} f_u$ and in the weld f_w, but with larger partial factors $\gamma_{M2} = \gamma_{Mw} = 1{,}25$.

So ,for a member in tension the resistance is the lesser of

a) $N_{o,Rd} = f_o A / \gamma_{M1}$

b) $N_{u,Rd} = \rho_{u,haz} f_u A / \gamma_{M2}$

c) $N_{w,Rd} = f_w A_w / \gamma_{Mw}$

Member with transverse welds or welded attachments

Question 1: Which is the lesser of the strength in HAZ and the weld for a tension member in EN-AW 6082-T6 with a but weld with $A_w = A$ made of filler metal 5356 ($\gamma_{M2} = \gamma_{Mw}$)

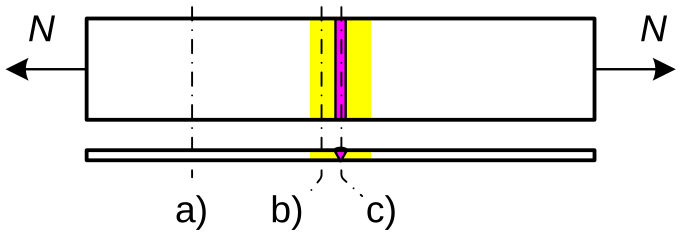
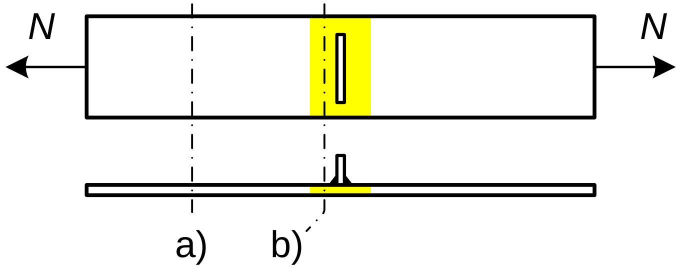


Table 8.8 $f_w = 210 N/mm^2$

Table 3.2b $\rho_{u,haz} f_u = f_{u,haz} = 185 N/mm^2$

Question 2: What is the difference for a member with an attachment?



a) $N_{o,Rd} = f_o A / \gamma_{M1}$

b) $N_{u,Rd} = \rho_{u,haz} f_u A / \gamma_{M2}$

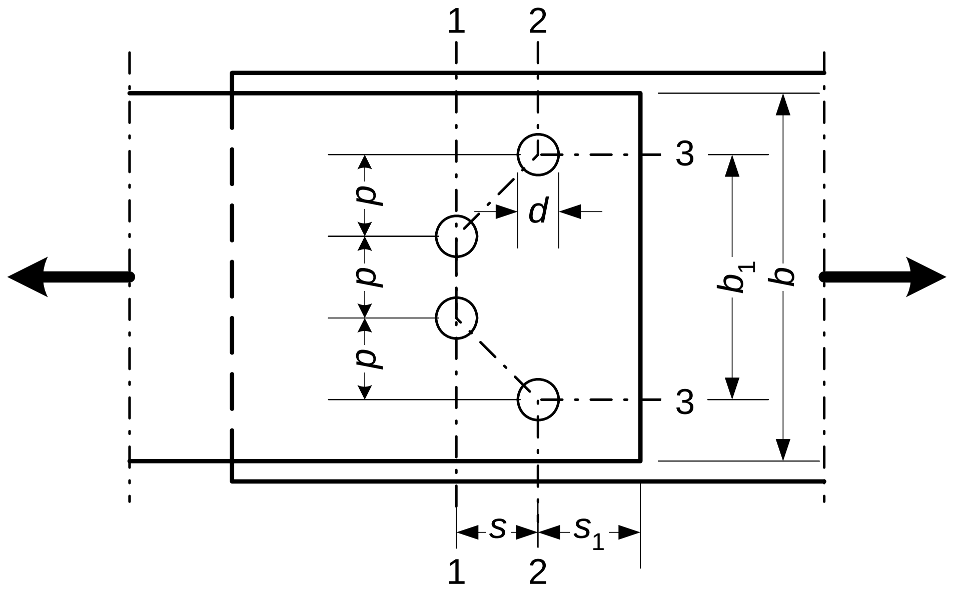
c) $N_{w,Rd} = f_w A_w / \gamma_{Mw}$ Formula c) is not applicable

Member with holes

For a member with (bolt) holes the resistance is the lesser of

a) The strength in the sections beside the holes

b) The strength in the section with the holes



A_{net} = min:

$t\,(b - 2d)$	line 1
$t\,(b - 4d + 2s^2/(4p))$	line 2
$t\,(b_1 + 2\times 0{,}65 s_1 - 4d + 2s^2/(4p))$	line 3

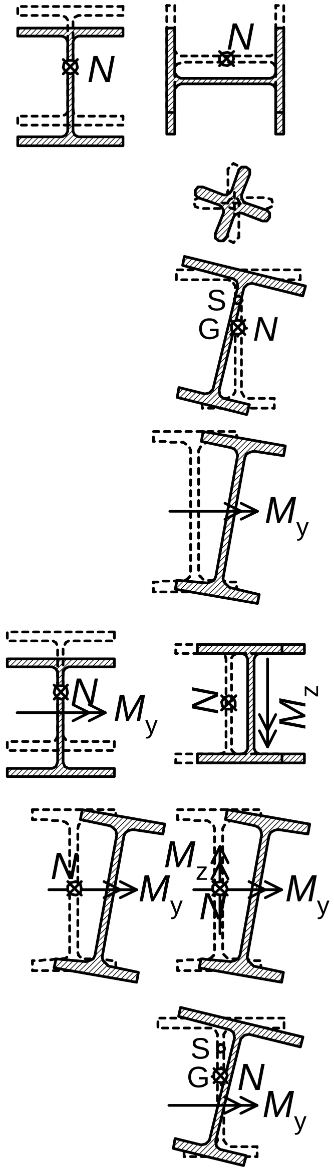
For a member in tension the resistance is the lesser of

a) $N_{\mathrm{o,Rd}} = f_{\mathrm{o}} A / \gamma_{\mathrm{M1}}$

b) $N_{\mathrm{u,Rd}} = 0{,}9\, f_{\mathrm{u}} A_{\mathrm{net}} / \gamma_{\mathrm{M2}}$ Note 0,9

The net area A_{net} shall be taken as the gross area less appropriate deductions for holes, see figure.

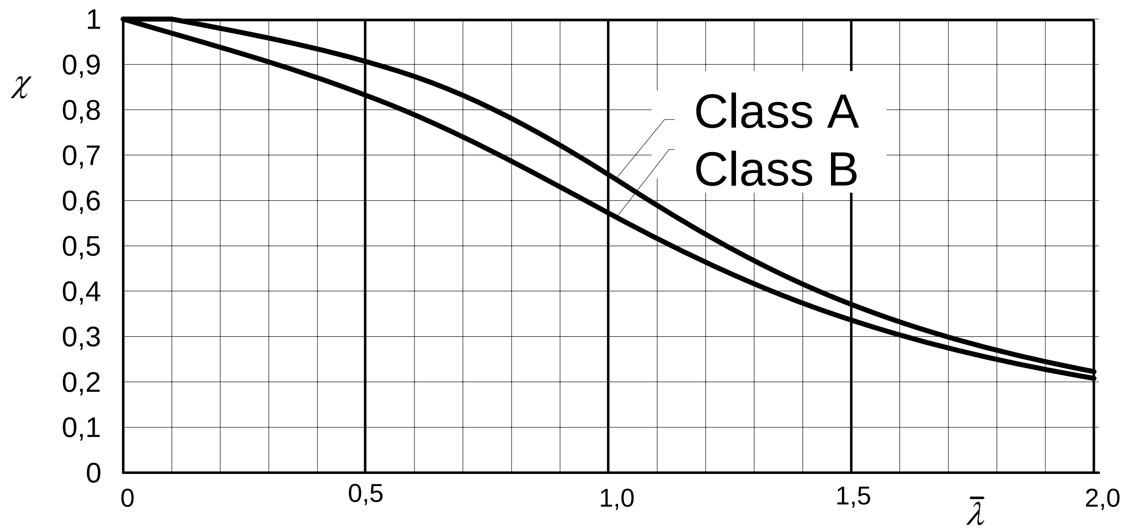
Flexural, torsion-flexural and lateral-torsional buckling



Axial force	(Flexural) buckling
	Torsional buckling
	Torsional-flexural buckling
Bending moment	Lateral-torsional buckling
Axial force and bending moment	Flexural buckling
	Lateral-torsional buckling

Flexural buckling

1. Critical load according to classic theory $N_{cr} = \frac{\pi^2 EI}{l_{cr}^2}$
2. Yield load $N_y = A_{eff} f_o$
3. Slenderness parameter $\bar{\lambda} = \sqrt{\frac{N_y}{N_{cr}}}$
4. Buckling class and reduction factor from formulae or diagram χ
5. Factor to allow for longitudinally or transverse welds $\kappa = 1$ for members without welds
6. Resistance $N_{b,Rd} = \kappa \chi N_y / \gamma_{M1}$



Flexural buckling, members with longitudinal welds

For members with *longitudinal welds*

Buckling class A

$$\kappa = 1 - \left(1 - \frac{A_1}{A}\right) 10^{-\bar{\lambda}} - \left(0{,}05 + 0{,}1\frac{A_1}{A}\right)\bar{\lambda}^{1{,}3(1-\bar{\lambda})}$$

where $A_1 = A - A_{\text{haz}}(1 - \rho_{\text{o,haz}})$

A_{haz} = area of HAZ

Buckling class B

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

$$\kappa = 1 + 0{,}04(4\bar{\lambda})^{(0{,}5-\bar{\lambda})} - 0{,}22\bar{\lambda}^{1{,}4(1-\bar{\lambda})} \qquad \text{if} \quad \bar{\lambda} > 0{,}2$$

Members with transverse welds at the ends

For members with *cross welds* the κ factor is depending on where the weld is placed along the member.

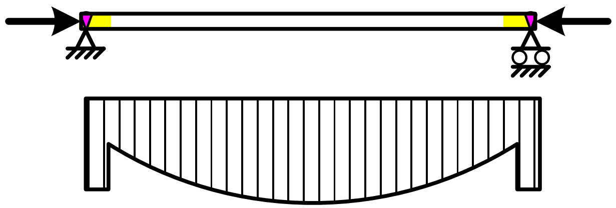
If the welds are *at the ends* then $\kappa = 1$ in the formula for flexural buckling (1). However, then a check is also needed of the section resistance at the ends where $\kappa = \omega_o$.(2)

$N_{b,Rd}$ is the lesser of 1 and 2

(1) $N_{b,Rd} = \chi N_y / \gamma_{M1}$ and

(2) $N_{Rd} = \omega_o N_y / \gamma_{M1}$ where

$$\omega_o = \frac{\rho_{u,haz} f_u / \gamma_{M2}}{f_o / \gamma_{M1}}$$



Utilization grade $\dfrac{N_{Ed}}{\chi N_y/\gamma_{M1}}$ $\dfrac{N_{Ed}}{\omega_o N_y/\gamma_{M1}}$

Transverse welds at any section

If the weld is at *a distance* x_s *from one end* then the resistance at that section is found for $\kappa = \omega_x$ (3). Furthermore the resistance for the member without weld should also be checked. (1)

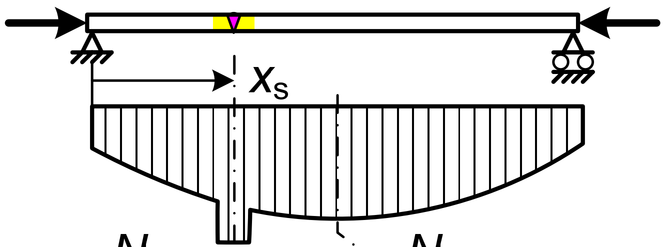
If the weld is at the centre of the member then $\omega_x = \omega_o$.

$N_{b,Rd}$ is the lesser of 1 and 3

(1) $N_{b,Rd} = \chi N_y / \gamma_{M1}$ and

(3) $N_{b,Rd} = \omega_x \chi N_y / \gamma_{M1}$

$$\omega_x = \frac{\omega_o}{\chi + (1-\chi)\sin(\pi x_s / l_{cr})}$$

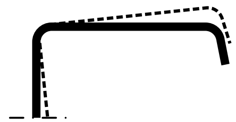


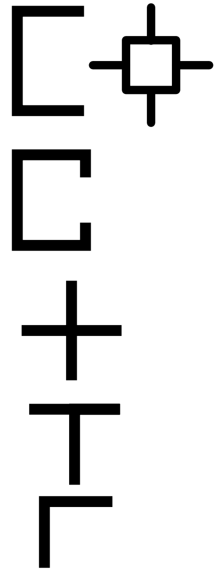
Utilization grade $\dfrac{N_{Ed}}{\omega_x \chi_{haz} N_y / \gamma_{M1}}$ $\dfrac{N_{Ed}}{\chi N_y / \gamma_{M1}}$

Note that at the weld χ_{haz} is based on $\bar{\lambda}_{haz} = \bar{\lambda}\sqrt{\omega_o}$ (6.68a)

Torsional and torsional-flexural buckling

(1) For sections containing reinforced outstands such that mode 1 would be critical in terms of local buckling, the member should be regarded as "general" and A_{eff} determined allowing for either or both local buckling and HAZ material.

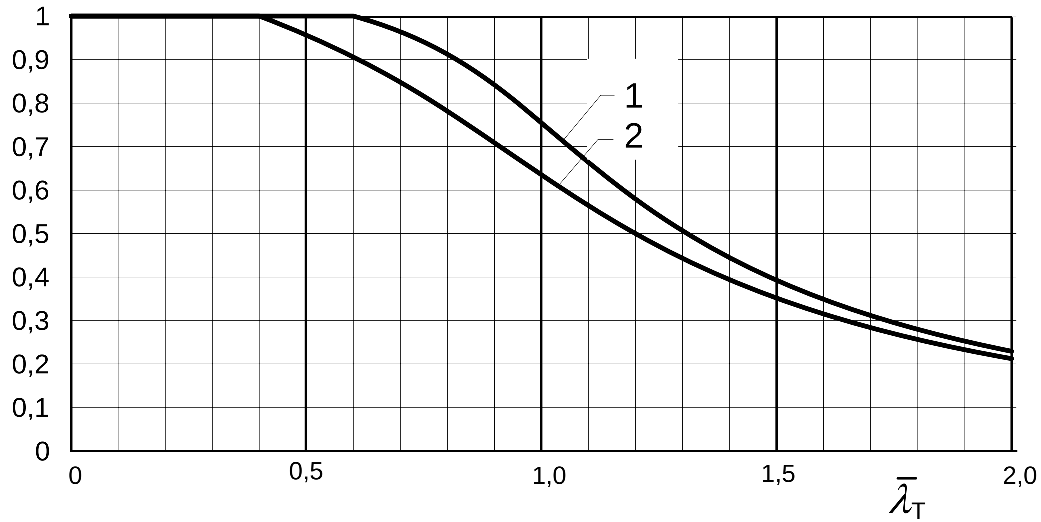




2) For sections such as angles, tees and cruciforms, composed entirely of radiating outstands, local and torsional buckling are closely related. When determining A_{eff} allowance should be made, where appropriate, for the presence of HAZ material but no reduction should be made for local buckling i.e. $\rho_c = 1$.

Formulae for critical load N_{cr} are given in Annex I of Eurocode 9 part 1-1.

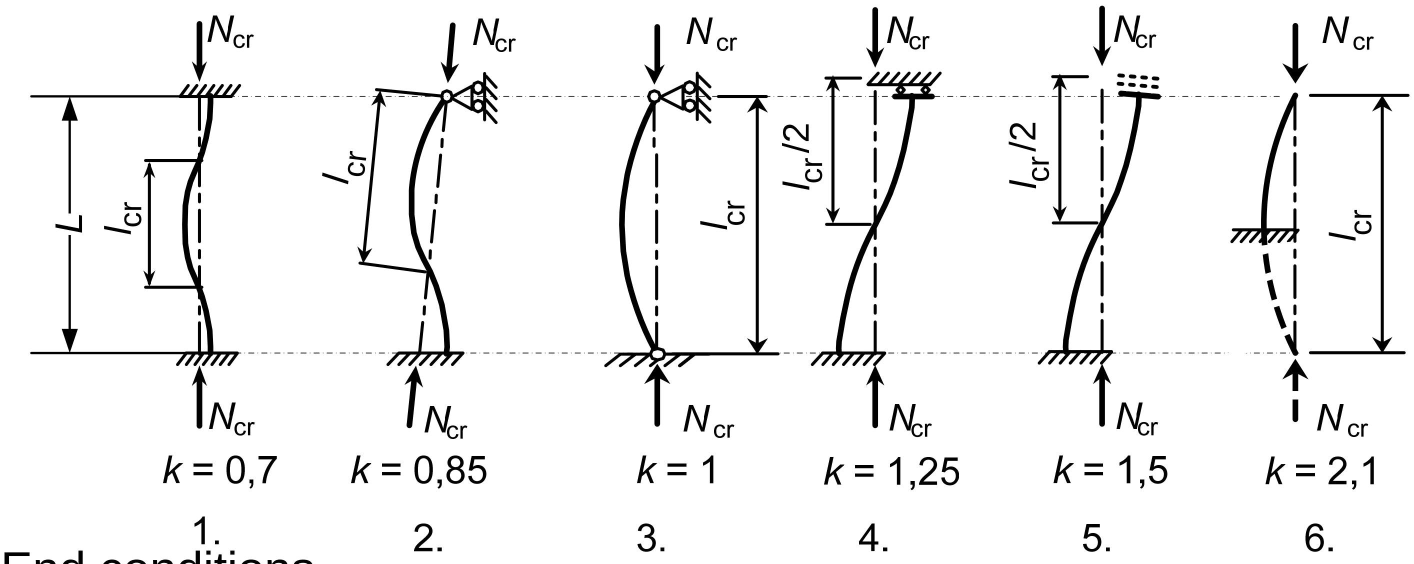
$$\overline{\lambda} = \sqrt{\frac{A_{eff}\ f_o}{N_{cr}}}$$



1 Cross section composed of radiating outstands,
2 General cross section

Buckling length factor *k*

The buckling length should be taken as $l_{cr} = kL$. The figure gives guidance for *k.*



End conditions

1. Held in position and restrained in rotation at both ends
2. Held in position at both ends and restrained in rotation at one end
3. Held in position at both ends, but not restrained in rotation
4. Held in position at one end, and restrained in rotation at both ends
5. Held in position and restrained in rotation at one end, and partially restrained in rotation but not held in position at the other end
6. Held in position and restrained in rotation at one end, but not held in position or restrained at the other end

EUROCODES
Background and Applications

Lateral-torsional buckling of beams

Critical moment

$$M_{\mathrm{cr}} = \frac{\pi}{L}\sqrt{EI_y\left(GK_{\mathrm{v}} + \frac{\pi^2 EK_{\mathrm{w}}}{L^2}\right)}$$

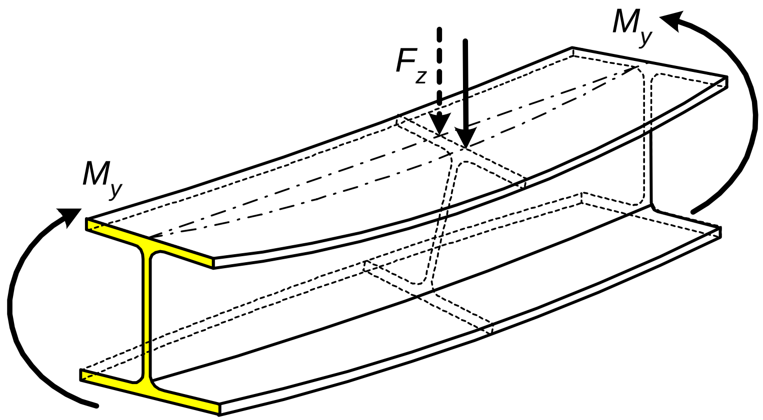
Slenderness parameter

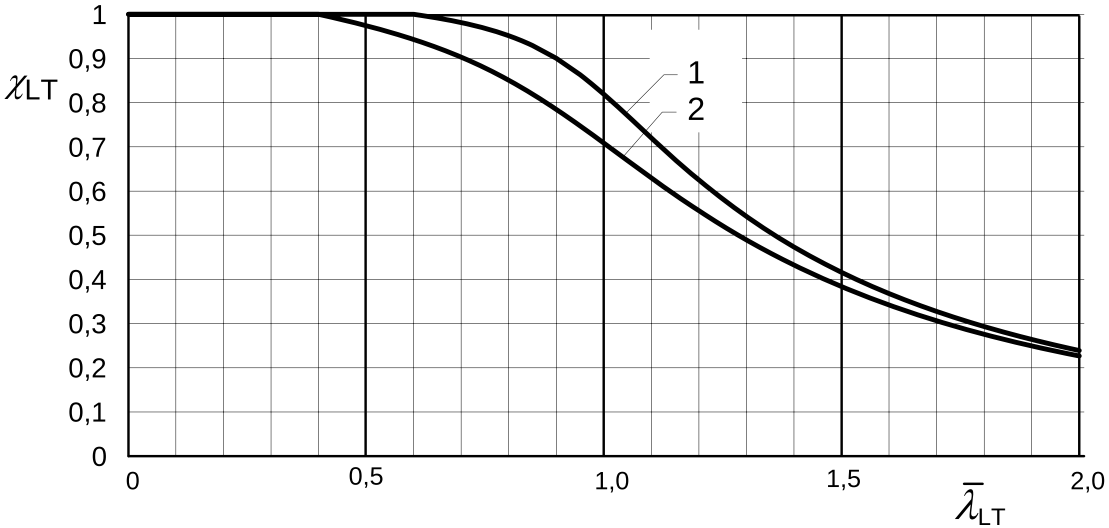
$$\bar{\lambda}_{\mathrm{LT}} = \sqrt{\frac{\alpha W_{\mathrm{el,y}} f_{\mathrm{o}}}{M_{\mathrm{cr}}}}$$

Reduction factor χ_{LT}

Resistance

$$M_{\mathrm{b,LT}} = \chi_{\mathrm{LT}} \alpha W_{\mathrm{el,y}} f_{\mathrm{o}} / \gamma_{M1}$$





1 Class 1 and 2 cross sections
2 Class 3 and 4 cross sections

Lateral-torsional buckling need not be checked

Lateral-torsional buckling need not be checked in any of the following circumstances

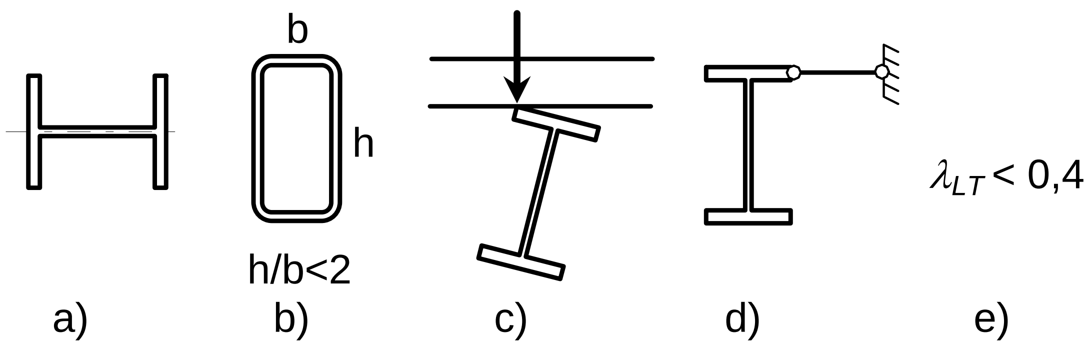
a) Bending takes place about the minor principal axis (symmetric profiles)

b) Hollow sections with *h/b* < 2

c) Rotation is prevented

d) The compression flange is fully restrained against lateral movement throughout its length

e) The slenderness parameter $\bar{\lambda}_{LT}$ between points of effective lateral restraint is less than 0,4.



Bending and axial compression

1 Classification of cross-sections for members with combined bending and axial forces is made for the loading components separately. No classification is made for the combined state of stress.

major axis (y-axis) bending:

$$\left(\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{y}}\,\omega_{\mathrm{x}}\,N_{\mathrm{Rd}}}\right)^{\xi_{\mathrm{yc}}}+\frac{M_{\mathrm{y,Ed}}}{\omega_0\,M_{\mathrm{y,Rd}}}\leq 1{,}00$$

minor axis (z-axis) bending:

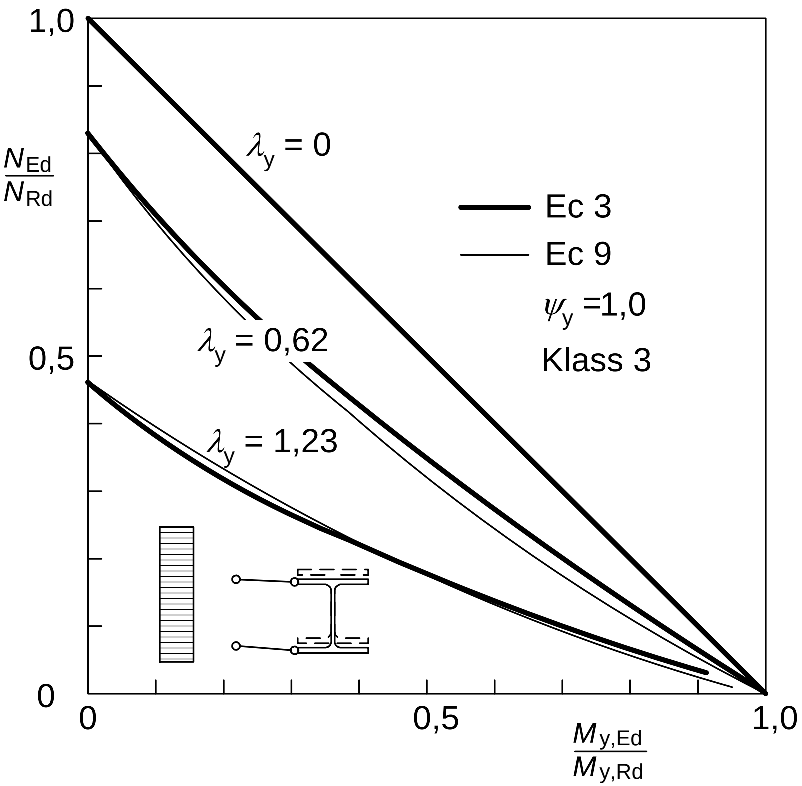
$$\left(\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{z}}\,\omega_{\mathrm{x}}\,N_{\mathrm{Rd}}}\right)^{\eta_c}+\left(\frac{M_{\mathrm{z,Ed}}}{\omega_0\,M_{\mathrm{z,Rd}}}\right)^{\xi_{zc}}\leq 1{,}00$$

2 A cross-section can belong to different classes for axial force, major axis bending and minor axis bending. The combined state of stress is accounted for in the interaction expressions. These interaction expressions can be used for all classes of cross-section. The influence of local buckling and yielding on the resistance for combined loading is accounted for by the resistances in the denominators and the exponents, which are functions of the slenderness of the cross-section.

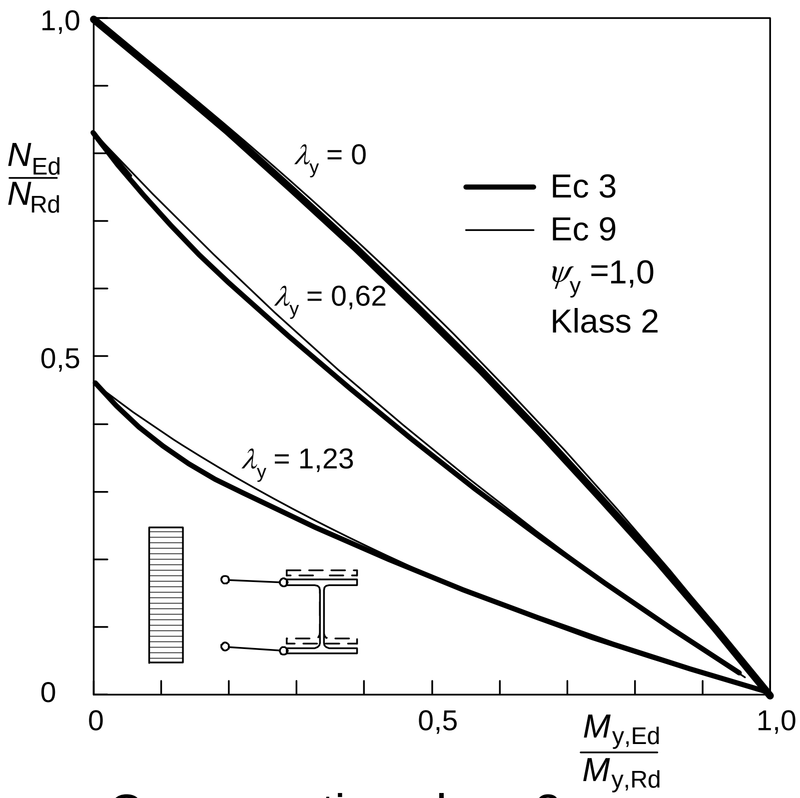
3 Section check is included in the check of flexural and lateral-torsional buckling

All exponents may conservatively be given the value 0,8. Alternative expressions depend on shape factors α_y or α_z and reduction factors χ_y or χ_z.

Comparison with Eurocode 3 for steel



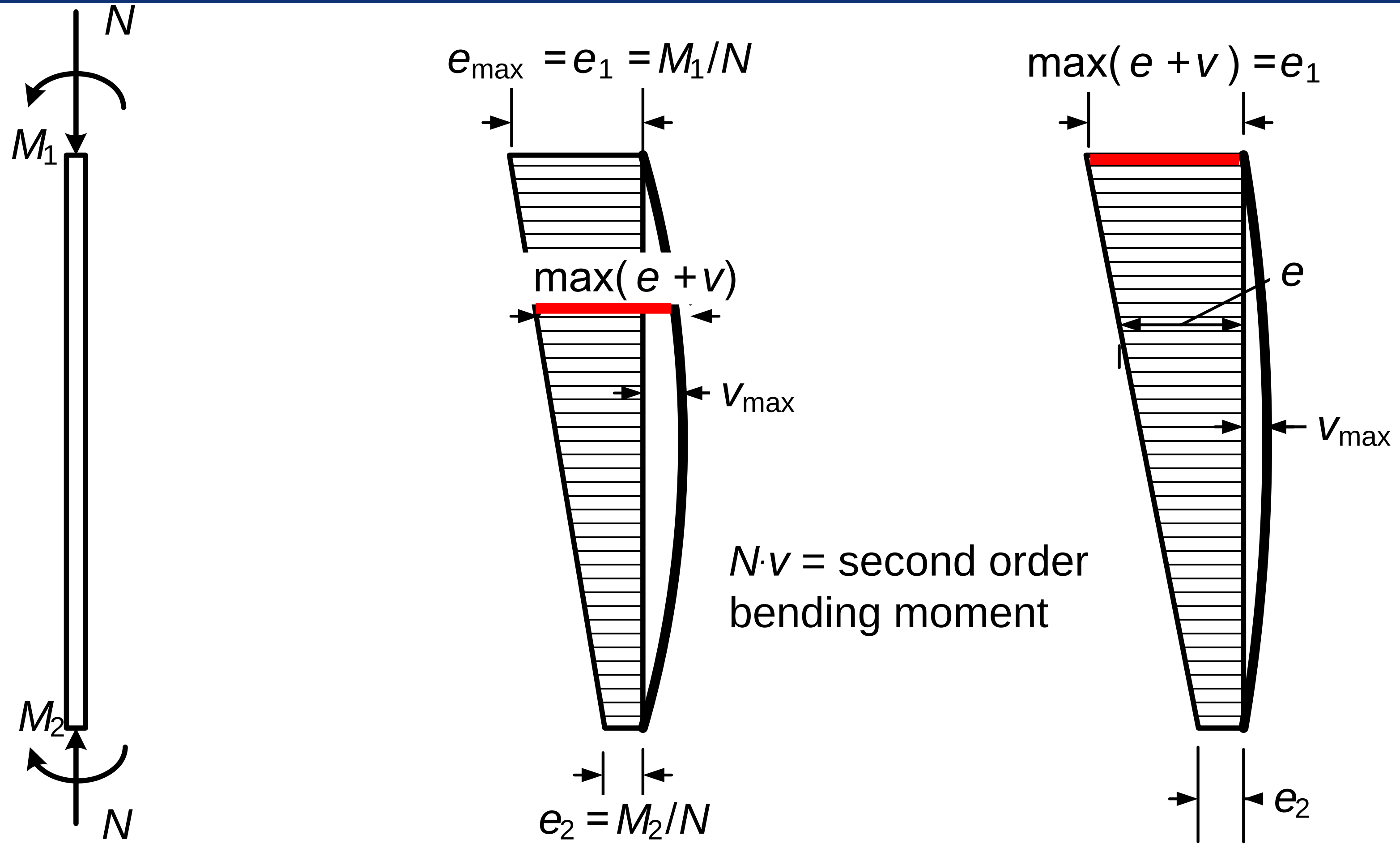
Cross section class 3



Cross section class 2

Major axis bending, constant bending moment

Design section



Basic case

$M_2 < M_1$

Max(e + v) occur in the span if N is large and/or the slenderness of the member is large

Max(e + v) occur at the end if M_1 is large and/or the slenderness of the member is small

Different end moments or transverse loads

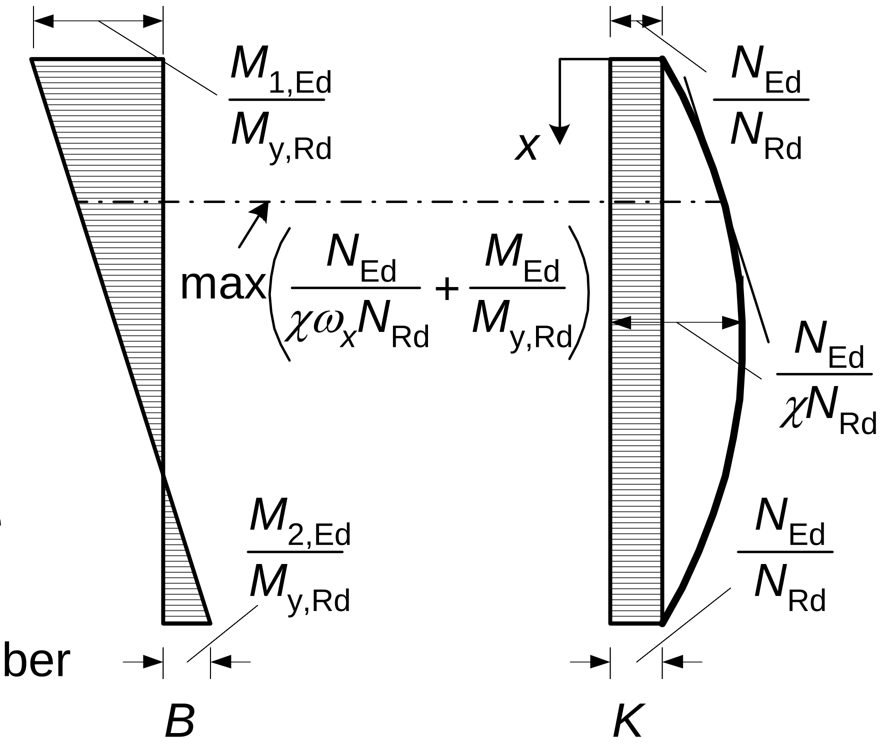
$$\left(\frac{N_{Ed}}{\chi_y \omega_x N_{Rd}}\right)^{\xi_{yc}} + \frac{M_{y,Ed}}{\omega_0 M_{y,Rd}} \leq 1{,}00$$

$$K \quad + \quad B \quad \leq 1$$

$$\omega_0 = 1$$

$$\omega_x = \frac{1}{\chi + (1-\chi)\sin\frac{\pi x}{l_{cr}}}$$

$\frac{1}{\omega_x}$ varies according to a sine curve and so also the first term K in the interaction formula



In principal all sections along the member need to be checked. However

$$\max\left(\left(\frac{N_{Ed}}{\chi_y \omega_x N_{Rd}}\right)^{\xi_{yc}} + \frac{M_{y,Ed}}{\omega_0 M_{y,Rd}}\right)$$

is found for $\cos\left(\frac{x\pi}{l_c}\right) = \frac{(M_{Ed,1} - M_{Ed,2})}{M_{Rd}} \cdot \frac{N_{Rd}}{N_{Ed}} \cdot \frac{1}{\pi(1/\chi - 1)}$ but $x \geq 0$

Equivalent moment

In **Eurocode 3 (steel)** the method with equivalent constant bending moment is used. Then for different bending moment distribution different coefficient are needed. One example is given below.

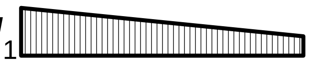
$$\frac{N_{\mathrm{Ed}}}{\chi_y N_{\mathrm{Rk}} / \gamma_{\mathrm{M1}}} + k_{yy} \frac{M_{y,\mathrm{Ed}}}{M_{y,\mathrm{Rk}} / \gamma_{\mathrm{M1}}} \leq 1$$

$$k_{\mathrm{yy}} = \frac{C_{\mathrm{my}}}{1 - \chi_{\mathrm{y}} \dfrac{N_{\mathrm{Ed}}}{N_{\mathrm{cr,y}}}}$$

Cross section class 3 and 4

$$k_{\mathrm{yy}} = \frac{C_{\mathrm{my}}}{\left(1 - \chi_{\mathrm{y}} \dfrac{N_{\mathrm{Ed}}}{N_{\mathrm{cr,y}}}\right) C_{\mathrm{yy}}}$$

Cross section class 1 and 2

For example for M_1  ψM_1

$$C_{\mathrm{my}} = 0,79 + 0,21\psi + 0,36(\psi - 0,33) \frac{N_{\mathrm{Ed}}}{N_{\mathrm{cr,y}}}$$

$$C_{\mathrm{yy}} = 1 + \left(w_{\mathrm{y}} - 1\right)\left[\left(2 - \frac{1,6}{w_{\mathrm{y}}} C_{\mathrm{my}}^2 \bar{\lambda}_y \left(1 + \bar{\lambda}_y\right)\right)\right]$$

$$w_{\mathrm{y}} = \frac{W_{\mathrm{pl,y}}}{W_{\mathrm{el,y}}} \leq 1,5$$

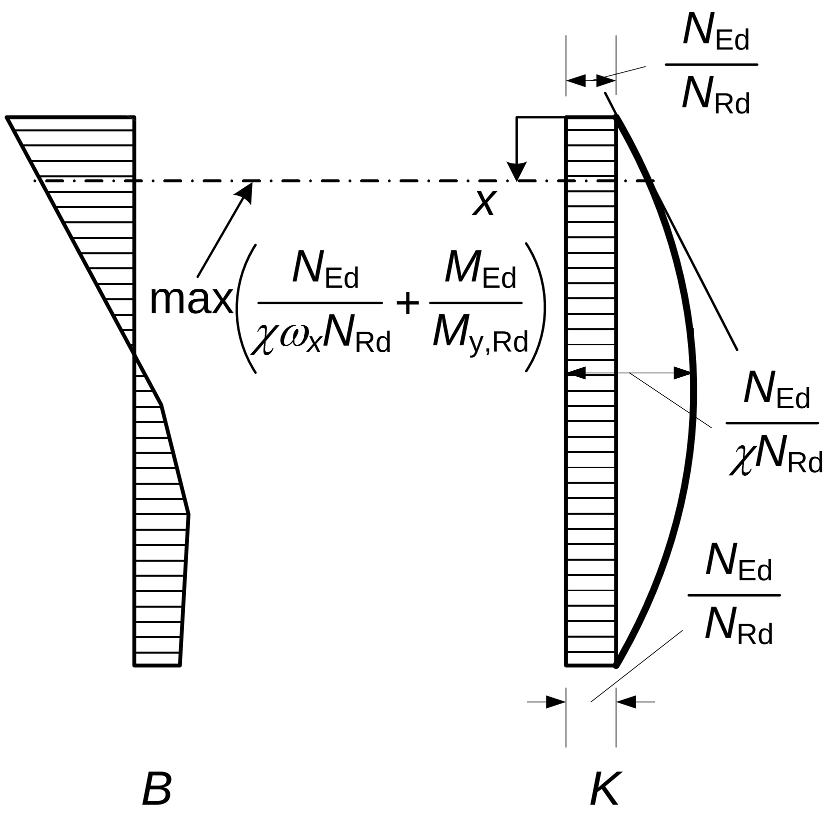
Arbitrary moment distribution

$$\left(\frac{N_{Ed}}{\chi_y \omega_x N_{Rd}}\right)^{\xi_{yc}} + \frac{M_{y,Ed}}{\omega_0 M_{y,Rd}} \leq 1{,}00$$

$$K \quad + \quad B \quad \leq 1$$

$$\omega_0 = 1$$

$$\omega_x = \frac{1}{\chi + (1-\chi)\sin\frac{\pi x}{l_{cr}}}$$



Member with transverse weld

For members with transverse (local) weld two checks should be made

1. As if there were no weld

$$\bar{\lambda} = \sqrt{\frac{N_y}{N_{cr}}} \rightarrow \chi$$

$$\left(\frac{N_{Ed}}{\chi\, \omega_x\, N_{Rd}}\right)^{\xi_{yc}} + \frac{M_{y,Ed}}{\omega_0\, M_{y,Rd}} \le 1{,}00$$

$$\omega_0 = 1$$

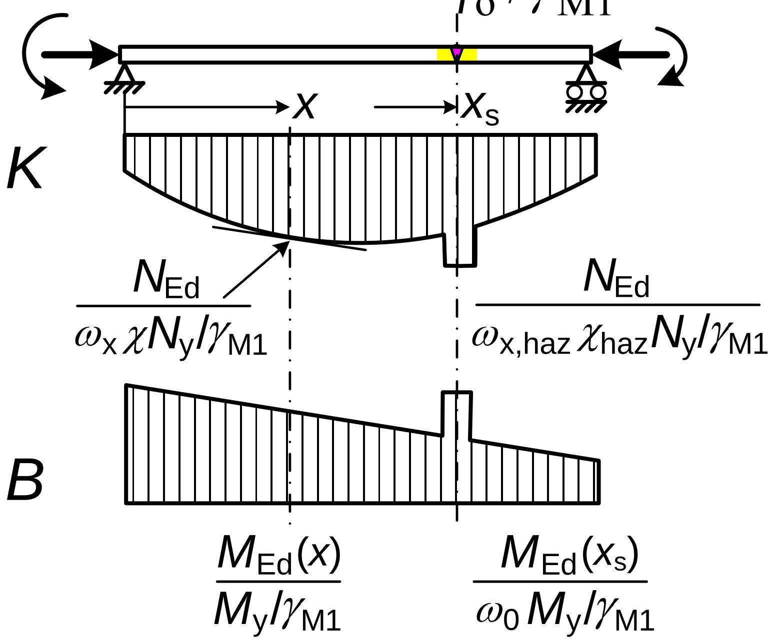
$$\omega_x = \frac{1}{\chi + (1-\chi)\sin\frac{\pi x}{l_{cr}}}$$

2. Check in the section with the weld

$$\bar{\lambda}_{haz} = \bar{\lambda}\sqrt{\omega_o} \rightarrow \chi_{haz}$$

$$\omega_{x.haz} = \frac{\omega_o}{\chi + (1-\chi)\sin(\pi x_s / l_{cr})} \quad \text{for } \chi = \chi_{haz}$$

$$\omega_o = \frac{\rho_{u,haz} f_u / \gamma_{M2}}{f_o / \gamma_{M1}}$$



Lateral-torsional buckling

Check for flexural buckling and

$$\left(\frac{N_{\mathrm{Ed}}}{\chi_{z}\,\omega_{x}\,N_{\mathrm{Rd}}}\right)^{\eta_c}+\left(\frac{M_{y,\mathrm{Ed}}}{\chi_{LT}\omega_{xLT}\,M_{y,\mathrm{Rd}}}\right)^{\gamma_c}+\left(\frac{M_{z,\mathrm{Ed}}}{\omega_0\,M_{z,\mathrm{Rd}}}\right)^{\xi_{zc}}\leq 1{,}00$$

As for flexural buckling all exponents may conservatively be given the value 0,8. Alternative expressions depend on shape factors α_y or α_z and reduction factors χ_y or χ_z.

The shape factors are:

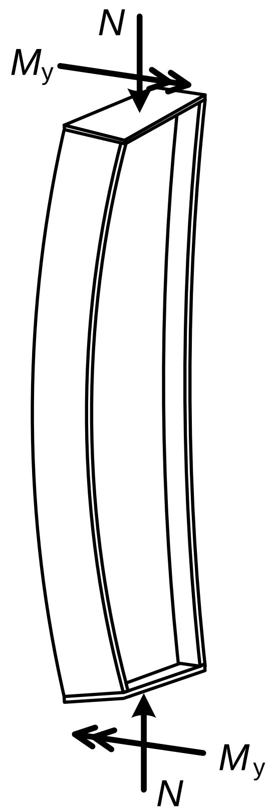
For class (1 and) 2 cross sections $\alpha = W_{pl}/W_{el}$

For class 3 cross sections α = between W_{pl}/W_{el} and 1

For class 4 cross sections $\alpha = W_{eff}/W_{el}$

Lateral-torsional buckling

If there are no lateral bending moment $M_{z,Ed}$ = 0 then



$$\left(\frac{N_{Ed}}{\chi_z \omega_x N_{Rd}}\right)^{\eta_c} + \left(\frac{M_{y,Ed}}{\chi_{LT} \omega_{x,LT} M_{y,Rd}}\right)^{\gamma_c} \leq 1$$

$\eta_c = 0{,}8$ or $\eta_0 \chi_z$ where $\eta_0 = 1$ or $\alpha_z^2 \alpha_y^2$ but $1 \leq \eta_0 \leq 2$

$\gamma_c = \gamma_0$ where $\gamma_0 = 1$ or α_z^2 but $1 \leq \gamma_0 \leq 1{,}56$

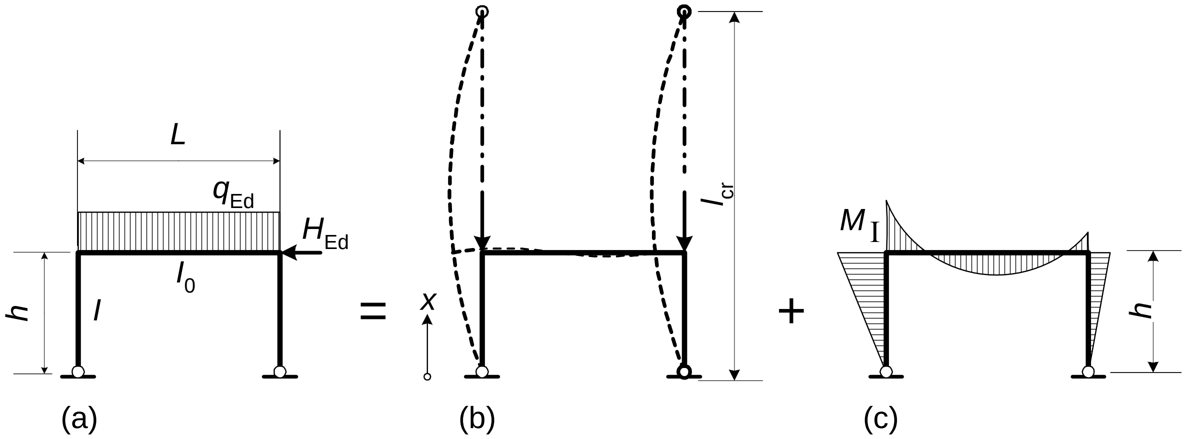
ω_x and $\omega_{x,LT}$ are coefficients which allow for HAZ across the member and/or of the moment distribution along the member. If there are no cross welds and constant moment then both ω are = 1 else

$$\omega_x = \frac{\omega_o}{\chi_z + (1 - \chi_z) \sin \frac{\pi x}{l_{cr}}} \qquad \omega_{x,LT} = \frac{\omega_o}{\chi_{LT} + (1 - \chi_{LT}) \sin \frac{\pi x}{l_{cr}}}$$

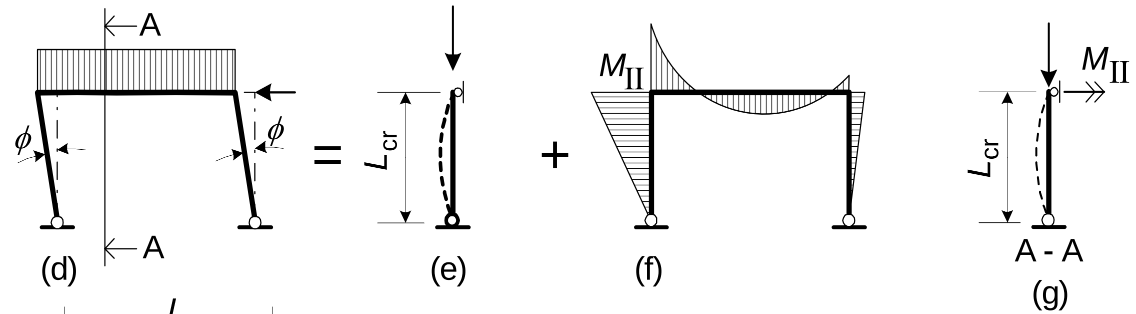
Design of frames

Three methods are possible:

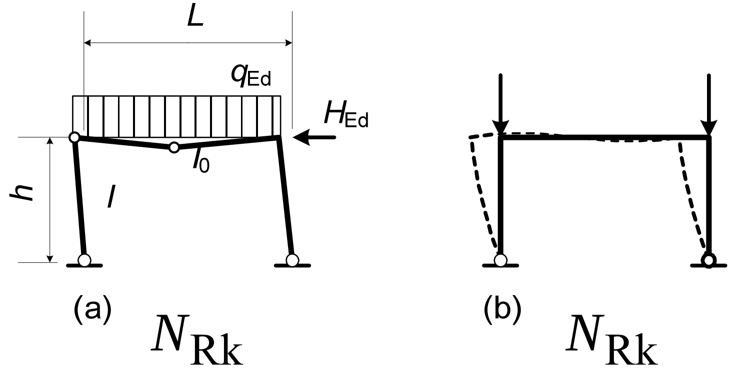
a) "Equivalent buckling length method"



b) "Equivalent sway imperfection method"

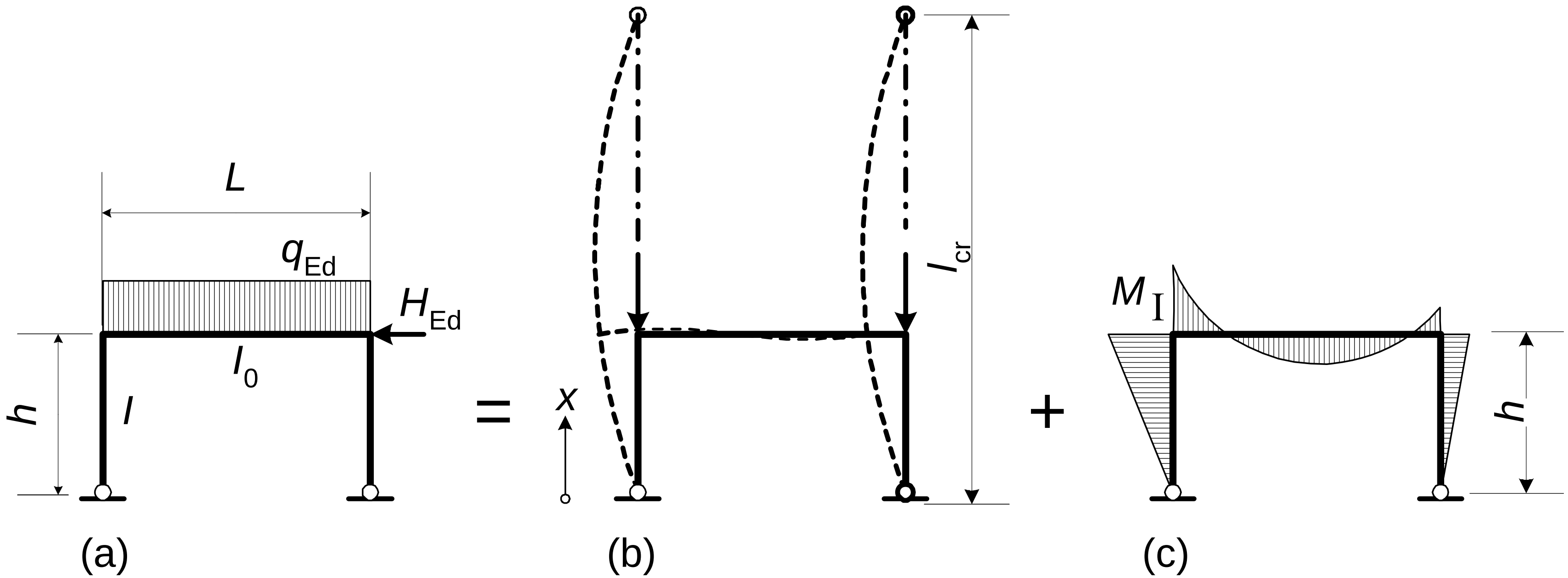


c) "Alternative method"



$$\bar{\lambda} = \sqrt{\frac{N_{Rk}}{N_{cr}}}$$

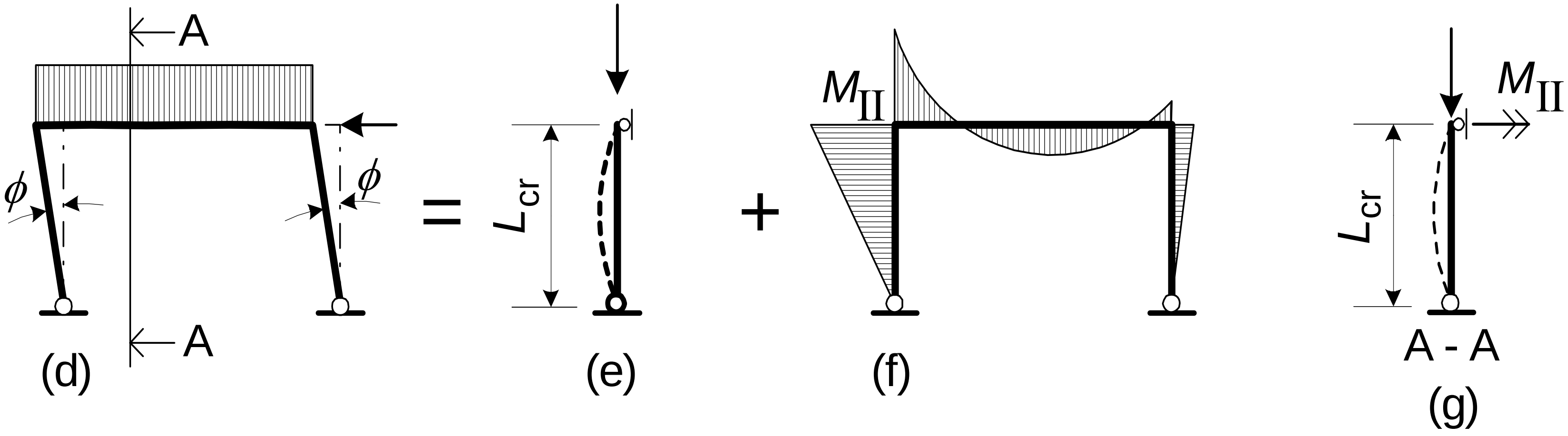
The equivalent column method



(a) System and load
(b) Equivalent column length
(c) First order bending moment

The second order bending moment is allowed for by the critical buckling length.

The equivalent sway method



(d) System, load and initial sway imperfection

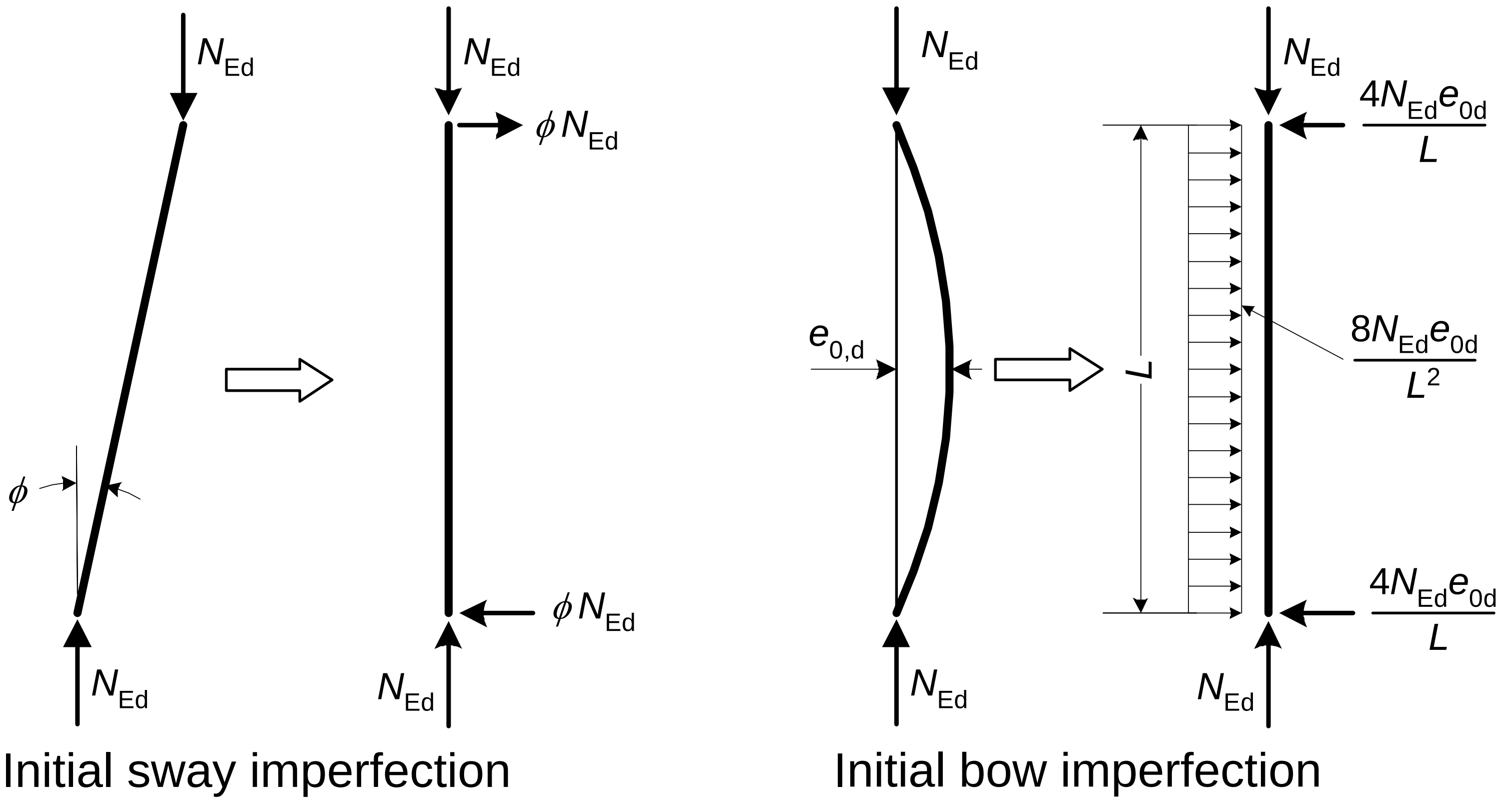
(e) Initial local bow imperfection and buckling length for flexural buckling

(f) Second order moment including moment from sway imperfection

(g) Initial local bow and buckling length for lateral-torsional buckling

Equivalent horizontal forces

The effect of initial sway imperfection and bow imperfection may be replaced by systems of equivalent horizontal forces introduced for each columns.



$$\frac{q_{\text{eqv}} L^2}{8} = N_{Ed} e_{0,d} \quad \text{gives} \quad q_{\text{eqv}} = \frac{8 N_{Ed} e_{0,d}}{L^2}$$

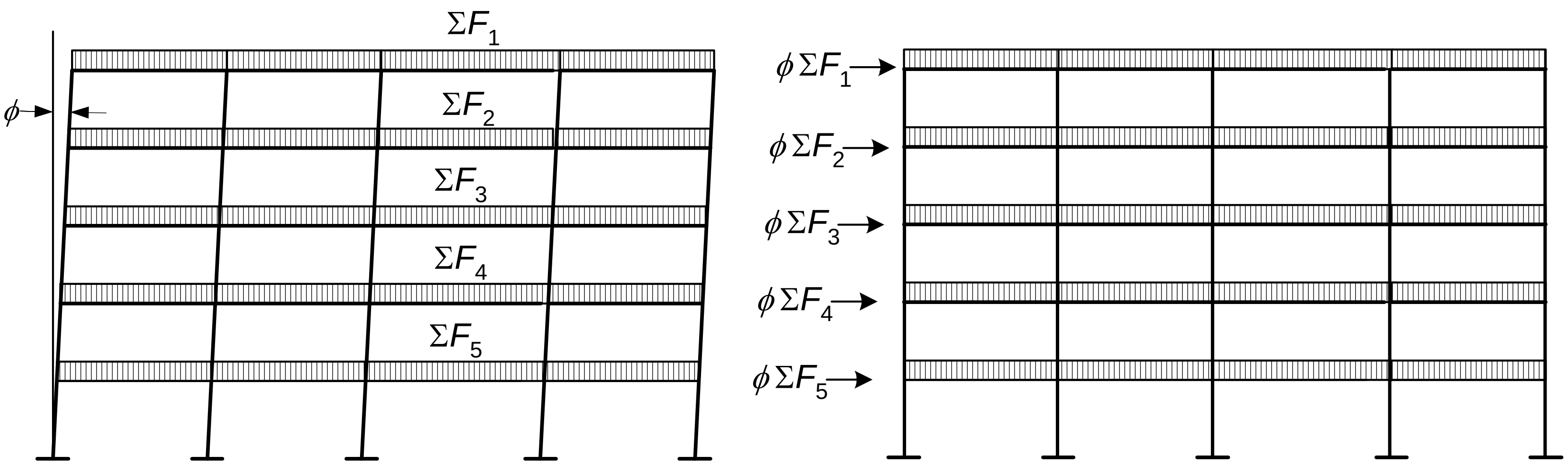
Initial sway imperfection

Initial sway $\phi = \phi_0 \cdot \alpha_h \cdot \alpha_m$

$$\phi_0 = \frac{1}{200} \qquad \alpha_h = \frac{2}{\sqrt{h}} \text{ but } \frac{2}{3} \le \alpha_h \le 1{,}0 \qquad \alpha_m = \sqrt{0{,}5\left(1 + \frac{1}{m}\right)}$$

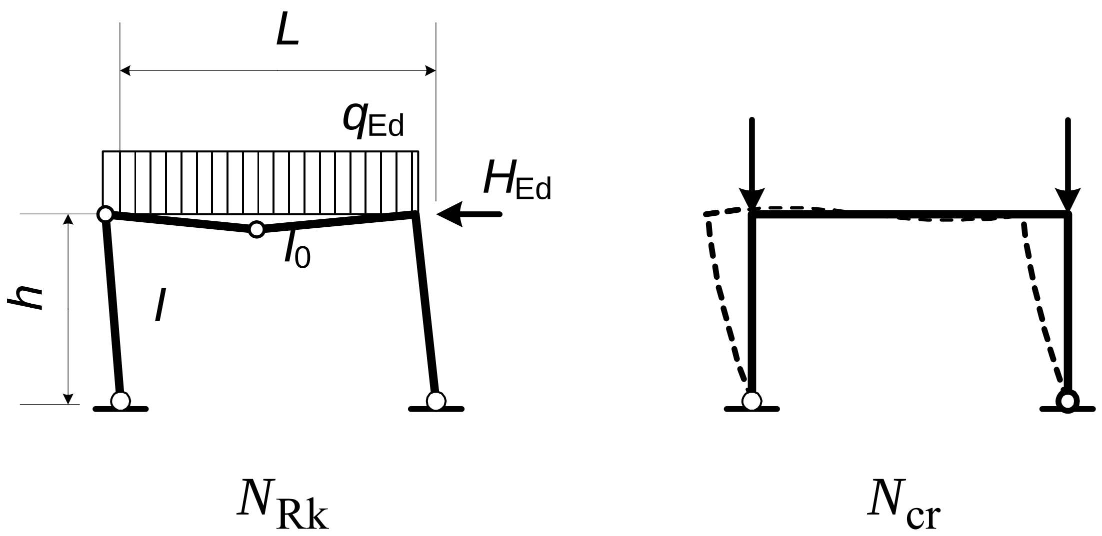
h = height in m meters

m = number of column in a row including only those columns which carry a vertical load N_{Ed} > 50 % of the average value for the columns



Equivalent horizontal forces

Alternative method



In principle $\bar{\lambda} = \sqrt{\dfrac{N_{Rk}}{N_{cr}}} \longrightarrow \chi \longrightarrow e_{0,d}$

Elastic or plastic global analysis

Elastic global analysis may be used in all cases.

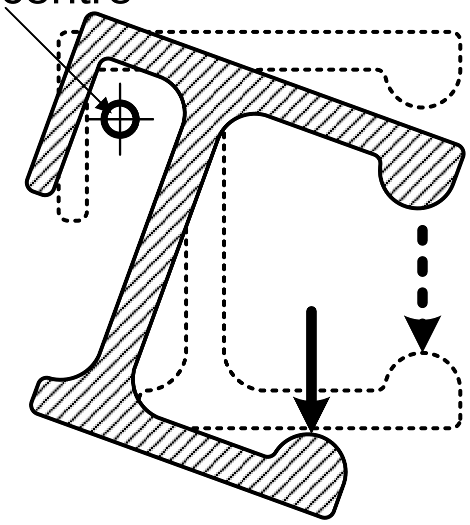
Plastic global analysis

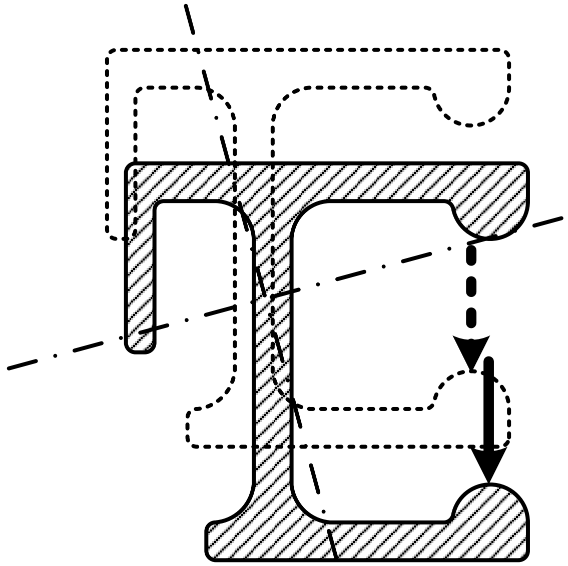
1. Plastic global analysis may be used only where the structure has sufficient rotation capacity at the actual location of the plastic hinge, whether this is in the members or in the joints. Where a plastic hinge occurs in a member, the member cross sections should be *double symmetric* or single symmetric with a plane of symmetry in the same plane as the rotation of the plastic hinge and it should satisfy the requirements for *cross section class 1*.
2. Where a plastic hinge occurs in a joint the *joint* should either have sufficient strength to ensure *the hinge remains in the member* or should be able to sustain the plastic resistance for a *sufficient rotation*.
3. Only *certain alloys* have the required ductility to allow sufficient rotation capacity.
4. Plastic global analysis should *not be used for beams with transverse welds* on the tension side of the member at the plastic hinge locations.
5. For plastic global analysis of beams recommendations are given in Annex H.
6. Plastic global analysis should only be used where the stability of members can be assured.

Torsion

Aluminium profiles are often asymmetric resulting in torsion. Example:

Shear centre

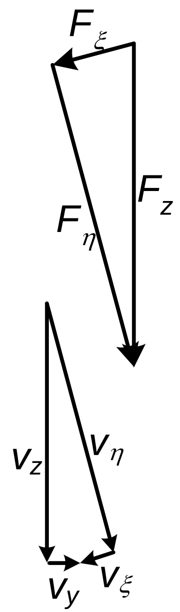




The beam is twisted around the shear centre

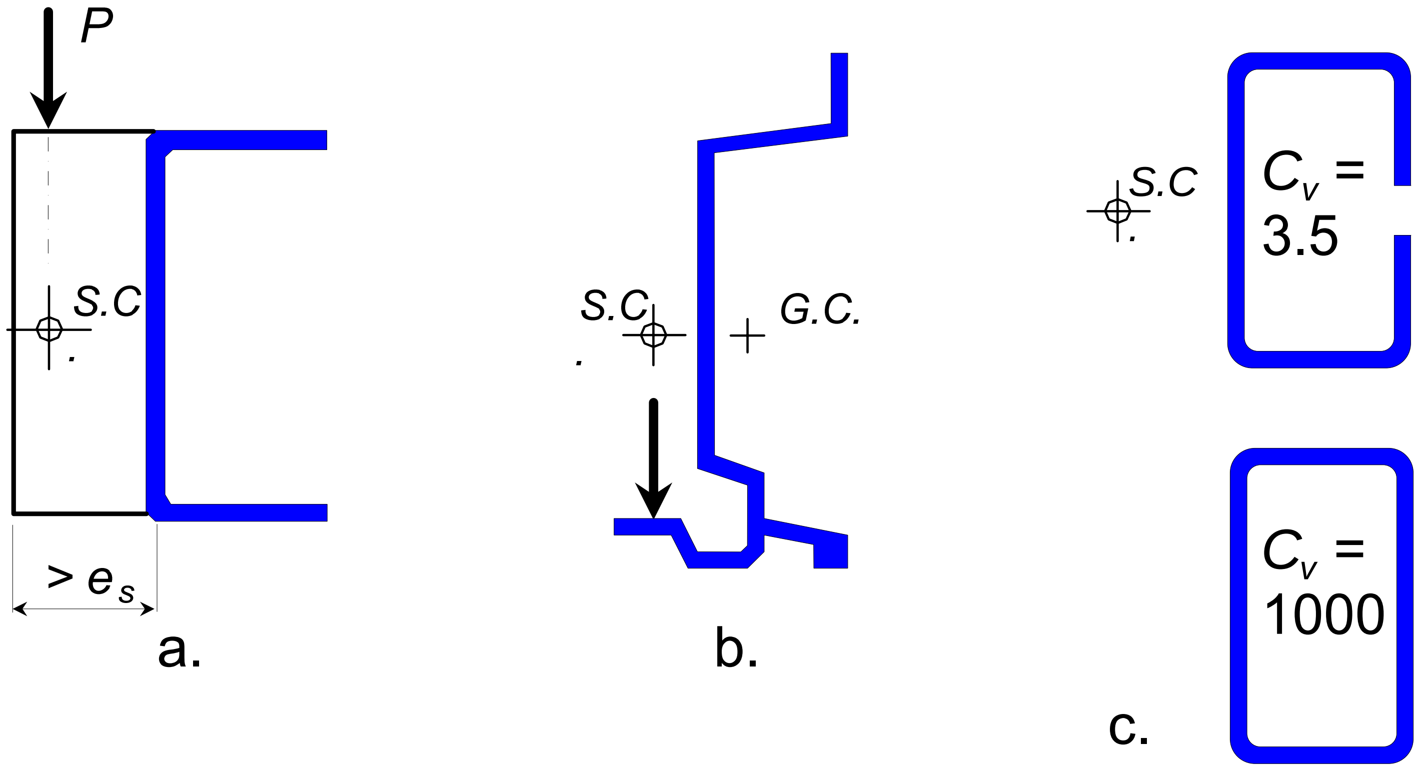
The deflection due to twisting may be larger than the deflection due to bending

1. Divide the load in the direction of the principal axes
2. Calculate the deflection in those directions
3. Calculate the vertical deflection



The load also deflects laterally, in this case to the left because the lateral deflection due to twist is larger than due to bending.

How to avoid torsion?



a. Add stiffeners

b. Change cross section so that the load acts through the shear centre

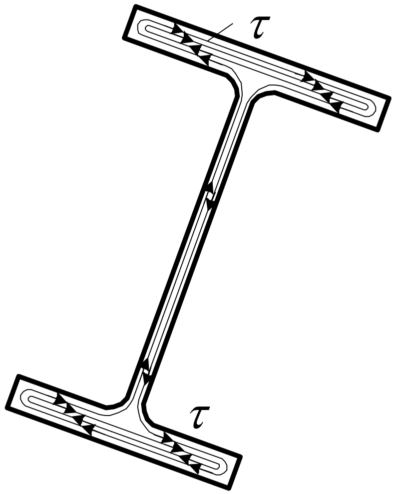
c. Use hollow sections

C_v = torsion stiffness (relative)

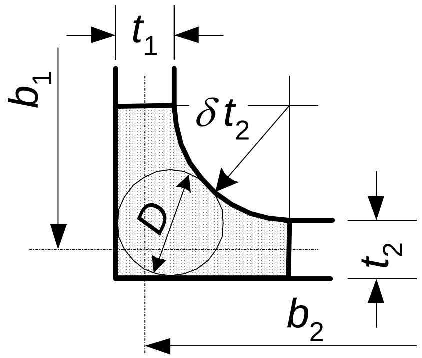
St Venants torsion resistance

For members subjected to torsion for which distortional deformations and warping torsion may be disregarded (St Venants torsion) the design value of the torsional moment at each cross-section shall satisfy

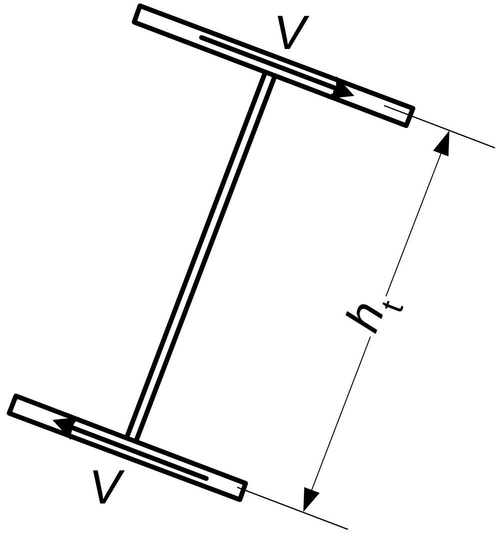
$$T_{\mathrm{Ed}} \le T_{\mathrm{Rd}} \quad \text{where } T_{\mathrm{Rd}} = W_{\mathrm{T,pl}}\, f_{\mathrm{o}} / (\sqrt{3}\, \gamma_{\mathrm{M1}})$$



St Venants torsion



Fillets increase torsion stiffness and strength considerably; see Annex J



Warping torsion

Warping torsion resistance

For members subjected to torsion for which distortional deformations may be disregarded but not warping torsion (Vlasov torsion) the total torsional moment at any cross-section should be considered as the sum of two internal effects:

The following stresses due to torsion should be taken into account:

- the shear stresses $\tau_{t,Ed}$ due to St. Venant torsion moment $T_{t,Ed}$
- the direct stresses $\sigma_{w,Ed}$ due to the bimoment B_{Ed} and shear stresses $\tau_{w,Ed}$ due to warping torsion moment $T_{w,Ed}$.

Check the von Mises yield criterion

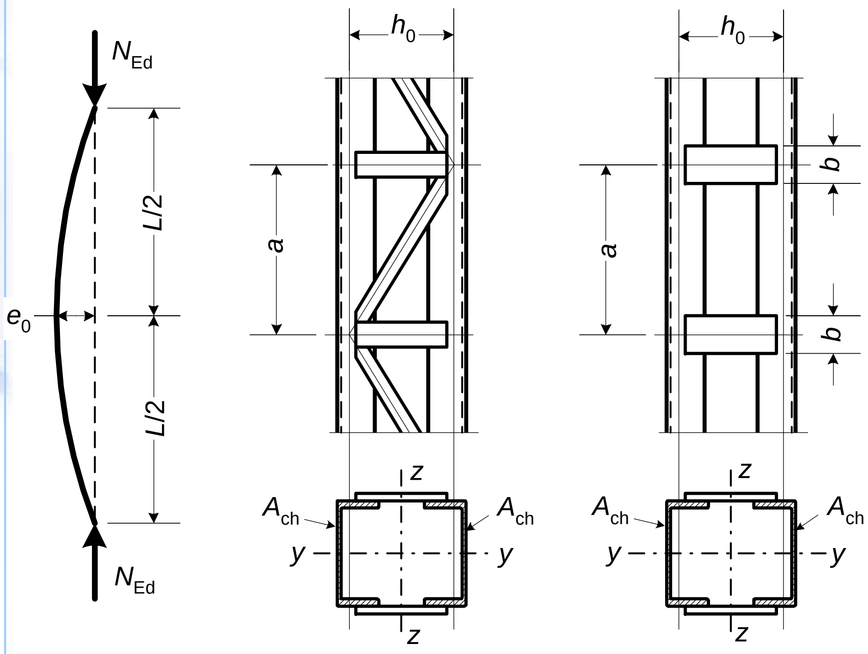
$$\left(\frac{\sigma_{x,Ed}}{f_o/\gamma_{M1}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_o/\gamma_{M1}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{f_o/\gamma_{M1}}\right)\left(\frac{\sigma_{z,Ed}}{f_o/\gamma_{M1}}\right) + 3\left(\frac{\tau_{Ed}}{f_o/\gamma_{M1}}\right)^2 \leq C$$

where $C = 1{,}2$

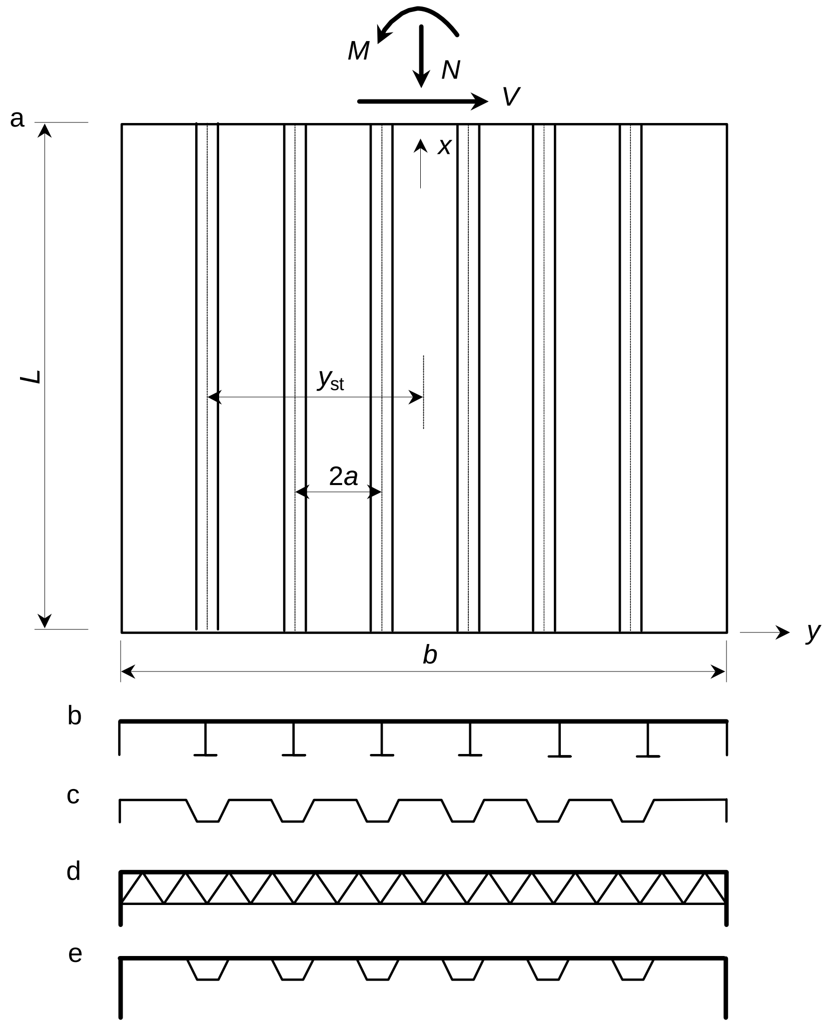
If the resultant force is acting through the shear centre there is no torsional moment due to that loading.

Formulae for the shear centre for some frequent cross-sections. see Annex J

Other structures covered in part 1-1



Built-up columns with lacings and battening [Eurocode 3]

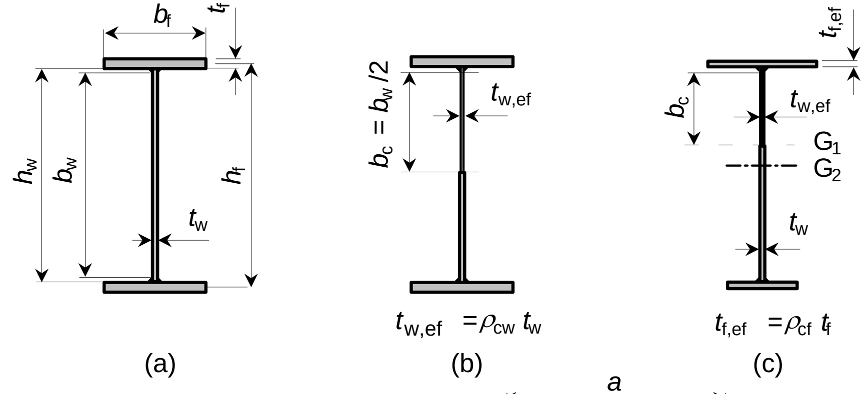


Un-stiffened and stiffened plates under in-plane loadings [2]

Plate girders

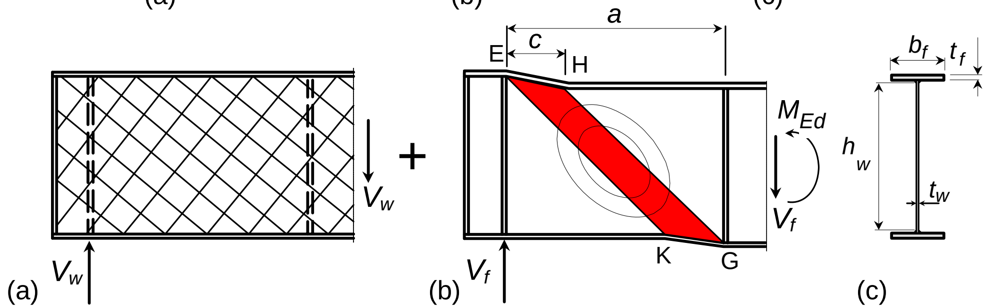
References

Bending



BS 8118 [4]
Höglund [2, 8]

Shear

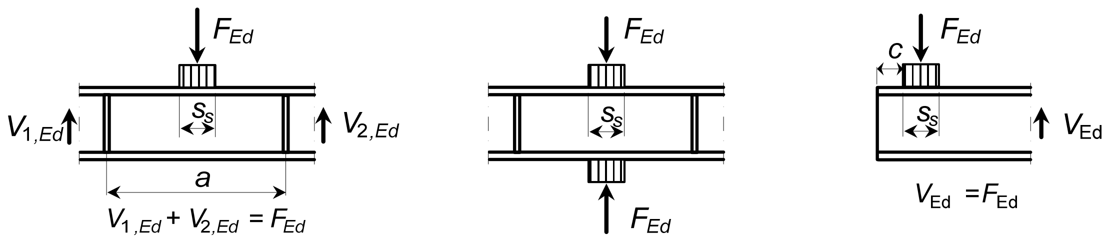


Rotated stress field + additional tension field

Höglund [5]

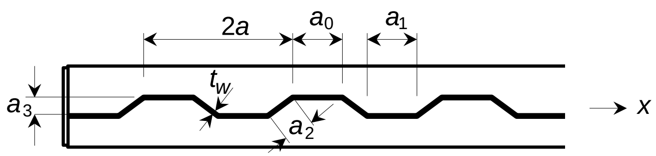
Others [6]

Patch loading



Lagerkvist [6j]
Tryland [6l]

Corrugated web



Höglund [5]
Benson [6a]
Ullman [12]

Worked examples and TALAT lectures

About 30 worked examples based on the ENV version of Eurocode 9 are available on the TALAT CD-ROM, also available at:

http://www.eaa.net/eaa/education/TALAT/2000/2300.htm

where also TALAT Lecture
2301 Design of Members and
2302 Design of Joints can be found

About 50 examples based on EN 1999 and updated lectures will be available on the EAA homepage.

A list of the examples are given at the end of this presentation

Main references

[1] Eurocode 9, EN 1999-1-1. *Eurocode 9: Design of Aluminium Structures – Part 1-1: General rules*. CEN (European Committee for Standardization) 2007.

[2] Höglund, T., *Design of members*. TALAT CD-ROM lecture 2301, (Training in Aluminium Application Technologies), European Aluminium Association. http://www.eaa.net/eaa/education/TALAT/2000/2300.htm

[3] Mazzolani (ed), Valtinat, Höglund, Soetens, Atzori, Langseth, *Aluminium Structural Design*. CISM Courses and Lectures No. 443, SpringerWienNewYork 2003

[4] BS 8118 *Structural use of aluminium*, Part 1. *Code of practice for design* Part 2. *Specification for material, workmanship and protection* 1991

[5] Höglund, T., *Shear Buckling resistance of Steel and Aluminium Plate Girders*. Thin-Walled Structures Vol. 29, Nos. 1-4, pp. 13-30, 1997

[6] References on Shear Buckling and Patch Loading

[a] Benson, P.G.(1992). Shear buckling and overall web buckling of welded aluminium girders. Royal Institute of Technology, Division of Steel Structures, Stockholm, PhD thesis

[b] Brown, K.E.P.(1990). The post-buckling and collapse behaviour of aluminium girders. *University of Wales* College of Cardiff, PhD thesis.

[c] Burt, C.A.(1987). The ultimate strength of aluminium plate girders. *University of Wales* College of Cardiff, PhD.

[d] Edlund, S., Jansson, R. and Höglund, T.(2001). Shear buckling of Welded Aluminium Girders. *9th Nordic Steel Construction Conference*, Helsinki.

[e] Evans, H.R. and Lee, A.Y.N.(1984). An appraisal, by comparison with experimental data, of new design procedures for aluminium plate girders. *Proc. Inst. Civ. Eng.* Structures & Buildings, Feb. 1984.

[f] Evans, H.R. and Hamoodi, M.J. (1987). The collapse of welded aluminium plate girders - an experimental study. *Thin-Walled Structures* 5.

[g] Evans, H.R. and Burt, C.(1990). Ultimate load determination for welded aluminium plate girders. Aluminium Structures: advances, design and construction. *Elsevier Applied Science*, London and New York.

[h] Höglund, T.(1972). Design of thin plate I girders in shear and bending with special reference to web buckling. *Royal Inst of Technology*, Dept. of Building Statics and Structural Engineering, Stockholm.

[i] Höglund, T.(1995). Shear buckling of Steel and Aluminium Plate Girders. *Royal Inst of Technology*, Dept. of Structural Engineering, Technical Report 1995:4, Stockholm

[j] Lagerqvist, O. (1994). Patch loading. Resistance of Steel Girders Subjected to Concentrated Forces. Ph.D. thesis, *Luleå University of Technology*, Division of Steel Structures, Luleå, Sweden.

[k] Rockey, K.C. and Evans, H.R.(1970). An experimental study of the ultimate load capacity of welded aluminium plate girders loaded in shear. Research Report, *University of Wales* College of Cardiff.

[l] Tryland, T. (1999). Aluminium and Steel Beams under Concentrated Loading. Dr.Ing. Thesis. *Norwegian University of Science and Technology*, Trondheim, Norway.

References on beam columns

See first of all [2]

[7] Höglund, T., *Approximativ metod för dimensionering av böjd och tryckt stång.* Royal Inst. of Technology, Division of Building Statics and Structural Engineering, Bulletin 77, Stockholm 1968

[8] Höglund, T., *Dimensionering av stålkonstruktioner.* Extract from the handbook Bygg, Chapter K18. The Swedish Institute of Steel Construction, Stockholm 1994

English Translation in: Höglund, T., *Steel structures, Design according to the Swedish Regulations for Steel Structures, BSK*. Dept. of Steel Structures, Royal Inst. of Technology, Stockholm 1988

[9] Edlund, S., *Buckling of T-section Beam-Columns in Aluminium with or without Transverse Welds.* Royal Inst. of Technology, Department of Structural Engineering, Stockholm 2000

References on local and overall buckling

[10] Langseth, M. and Hopperstad, O.S., *Local buckling of square thin-walled aluminium extrusions.* Thin-walled Structures, 27, pp. 117-126, 1996

[11] Hopperstad, O.S., Langseth, M. and Tryland, T., *Ultimate strength of aluminium alloy outstands in compression: experiments and simplified analysis.* Thin-walled Structures, 34, pp. 279-294, 1999

[12] Ullman, R., Shear Buckling of Aluminium Girders with Corrugated webs. Royal Inst. of Technology, Department of Structural Engineering, ISRN KTH/BKN/B-67-SE, Stockholm 2002

Thank you for your attention !

Eurocode 9, worked examples

Torsten Höglund

1. Mathcad formulations
2. Serviceability limit state
3. Axial tension
4. Bending moment
5. Axial force
6. Shear force
7. Concentrated force
8. Torsion
9. Axial force and bending moment
10. Nonlinear stress distribution
11. Trapezoidal sheeting
12. Shells

Mathcad formulations

The calculations in the following examples are set out in detail. In most cases, the designer can make simplifications when he/she has learned by experience which checks are not usually critical.

The examples are worked out in the mathematics program Mathcad, version 8. Some of the operators and notations used in the examples are explained below.

$x := 50.6 \cdot mm$ — Definition of value

$y \equiv 2.5 \cdot mm$ — Global definition

$x + y = 53.1mm$ — Calculation result

$a = b$ — Boolean equality

0.5 — Decimal point must be used

$c := (1 \;\; 3 \;\; 2)$ — Row vector

$\overrightarrow{(c \cdot d)}$ — Vectorise operator, i.e. perform arithmetical operation on each element of a vector or matrix

Example: $d := (2 \;\; 4 \;\; 3)$ $\quad a := \overrightarrow{(c \cdot d)}$

gives $a = (2 \;\; 12 \;\; 6)$

$g := \begin{pmatrix} 1 & 8 & 2 \\ 3 & 4 & 7 \\ 5 & 6 & 9 \end{pmatrix}$ — Matrix

c^T — Transpose, i.e. rows and columns are interchanged

Example: $c^T = \begin{pmatrix} 1 \\ 3 \\ 2 \end{pmatrix}$ $\quad g^T = \begin{pmatrix} 1 & 3 & 5 \\ 8 & 4 & 6 \\ 2 & 7 & 9 \end{pmatrix}$

$submatrix(a, 0, 1, 1, 2)$ — Part of matrix (a=matrix, 0 och 1 define rows, 1 and 2 define columns)

Normally, in a matrix, the first row is numbered 0 and the first column is numbered 0

Example: $g = \begin{pmatrix} 1 & 8 & 2 \\ 3 & 4 & 7 \\ 5 & 6 & 9 \end{pmatrix}$

$submatrix(g, 0, 1, 1, 2) = \begin{pmatrix} 8 & 2 \\ 4 & 7 \end{pmatrix}$

$augment(f, g)$ — Augmentation of matrices

Example: $augment\left(c^T, g\right) = \begin{pmatrix} 1 & 1 & 8 & 2 \\ 3 & 3 & 4 & 7 \\ 2 & 5 & 6 & 9 \end{pmatrix}$

$f^{\langle 1 \rangle}$ — Column

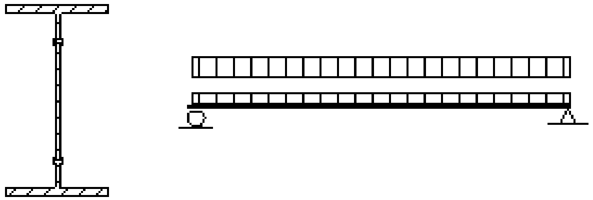
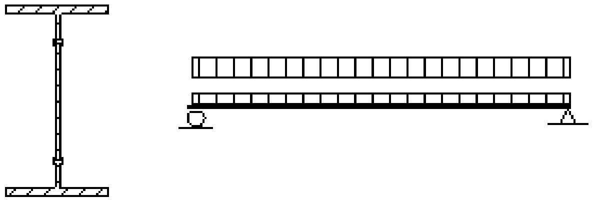
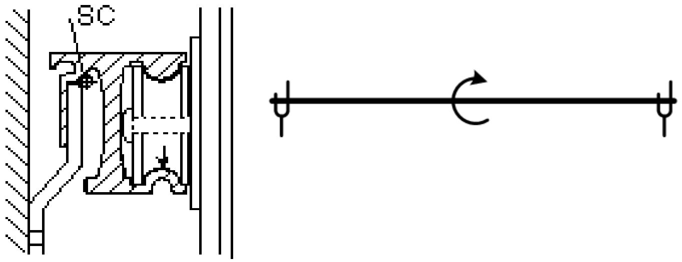
Example: $g^{\langle 1 \rangle} = \begin{pmatrix} 8 \\ 4 \\ 6 \end{pmatrix}$

A_{ef} — Notation (ef is not a subscript but part of variable notation)

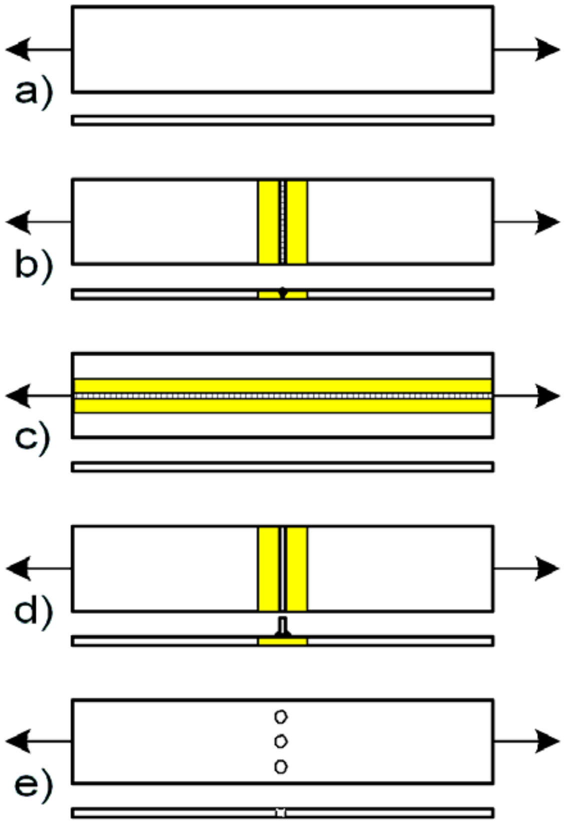
A_i — Subscript i

Example: $g_{1,2} = 7$

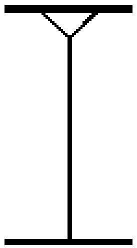
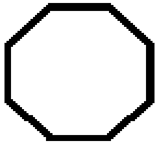
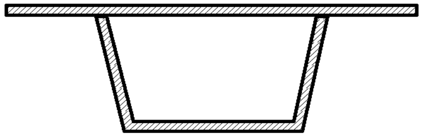
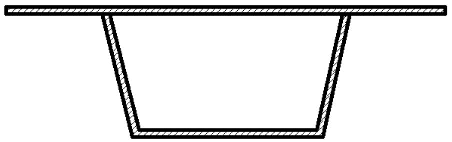
Serviceability limit state

No.	Cross section etc.	Description
2.1		Deflection of class 4 cross section girder made of two extrusions and one plate. Distributed load.
2.2		Simple method to check resistance and deflection of class 4 cross section girder. Distributed load.
2.3		Deflection of asymmetric extruded profile due to bending and torsion of concentrated load. Check of stresses included.

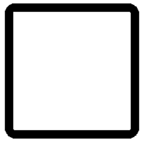
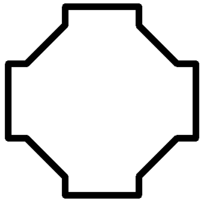
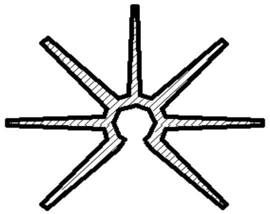
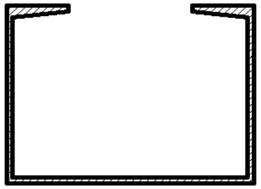
Axial tension

No.	Cross section etc.	Description
3.1	f_o, f_u, $f_{o,haz}$, $f_{u,haz}$ etc	Characteristic values of material properties
3.2		a) Axial tension force resistance of a plane plate
		b) Axial tension force resistance of a plate with MIG butt weld across the plate
		c) Axial tension force resistance of a plate with longitudinal butt weld
		d) Axial tension force resistance of a plate with welded attachment across the plate
		e) Axial tension force resistance of a plate with bolt holes

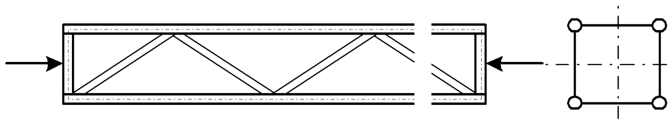
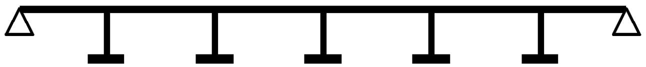
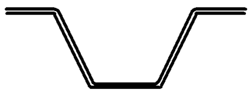
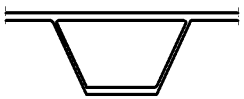
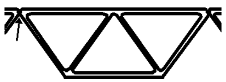
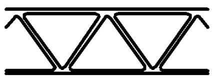
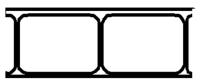
Bending moment

No.	Cross section etc.	Description
4.1		Bending moment resistance of cross section with closed cross section parts and outstands
4.2		Bending moment resistance of extruded hollow cross section
4.3		Bending moment resistance of welded hollow cross section with outstands. Class 2 cross section
4.4		Bending moment resistance of welded hollow cross section with outstands. Class 4 cross section

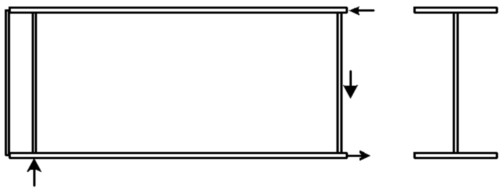
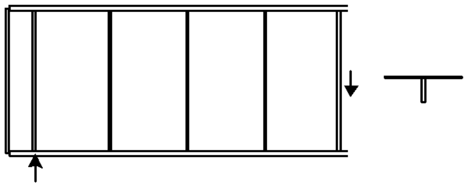
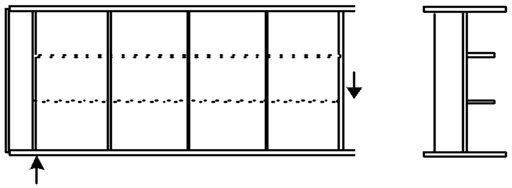
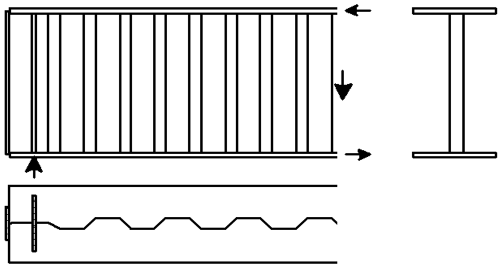
Axial compression 1

No.	Cross section etc.	Description
5.1		Axial compression force resistance of square hollow cross section (local and flexural buckling)
5.2		Axial compression force resistance of symmetric hollow extrusion (local and flexural buckling)
5.3		Axial compression force resistance of cross section with radiating outstands (torsional buckling)
5.4		Axial compression force resistance of channel cross section (distortional buckling)

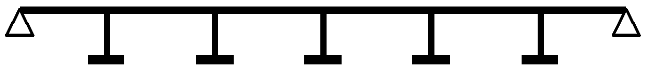
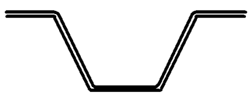
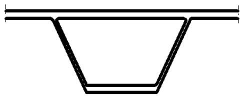
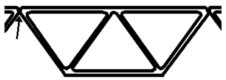
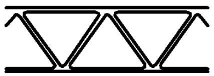
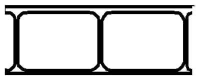
Axial compression 2

No.	Cross section etc.	Description
5.5		Axial force resistance of laced column
5.6	 a)   b) c)	Axial force resistance of orthotropic plate with a) open stiffeners b) trapezoidal stiffeners c) closed stiffeners
5.7	    a) b) c)	Axial force resistance of orthotropic double-skin plate a) profiles joined with grooves and tongues b) truss cross section c) frame cross section

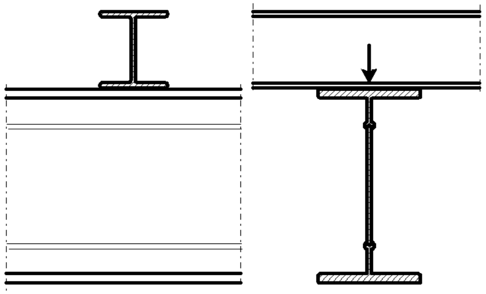
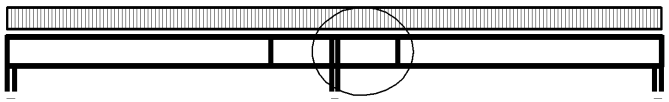
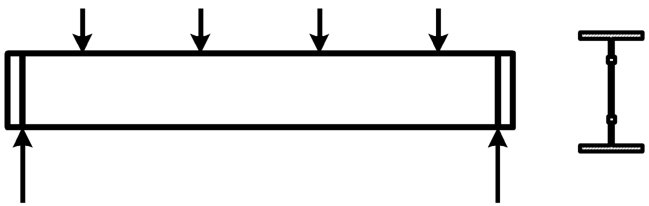
Shear force 1

No.	Cross section etc.	Description
6.1		Shear force resistance of a plate girder with no intermediate stiffeners incl. contribution from the flanges
6.2		Shear force resistance of a web with a) flexible intermediate stiffeners b) rigid intermediate stiffeners
6.3		Shear force resistance of a web with transverse and longitudinal stiffeners
6.4		Shear force resistance of a plate girder with corrugated web

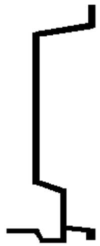
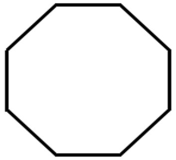
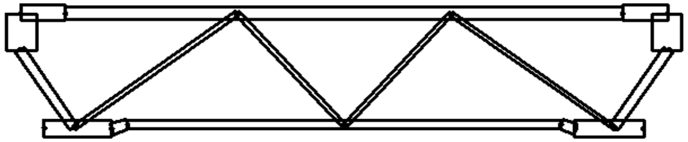
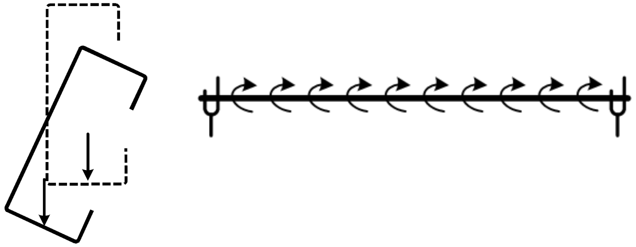
Shear force 2

No.	Cross section etc.	Description
6.5	 a)   b) c)	Shear force resistance of orthotropic plate with a) open stiffeners b) trapezoidal stiffeners c) closed stiffeners
6.6	    a) b) c)	Shear force resistance of orthotropic double-skin plate a) profiles joined with grooves and tongues b) truss cross section c) frame cross section

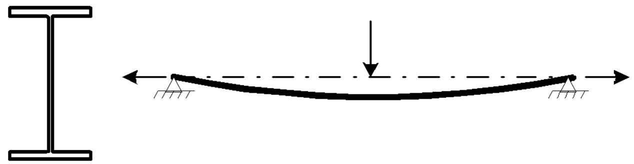
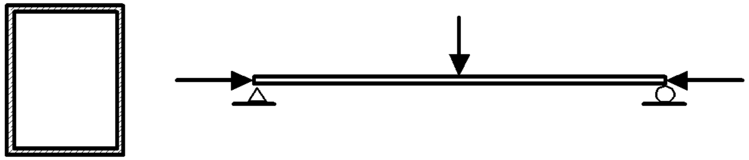
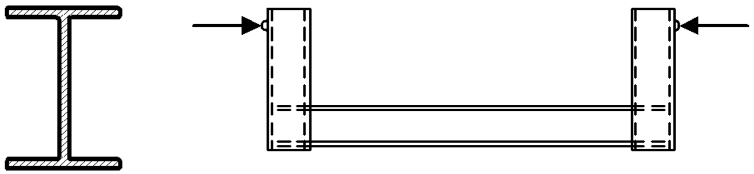
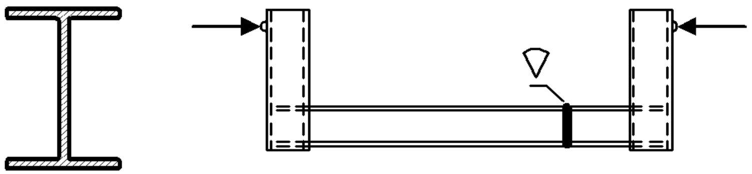
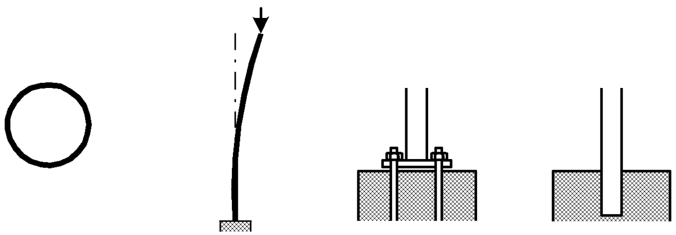
Concentrated force and interaction *M+V*

No.	Cross section etc.	Description
7.1		Concentrated force resistance of beam and plate girder (patch loading).
7.2		Interaction between shear force and bending moment for a plate girder at the support region.
7.3		Plate girder in shear, bending from concentrated forces. Rigid end post, no intermediate stiffeners.

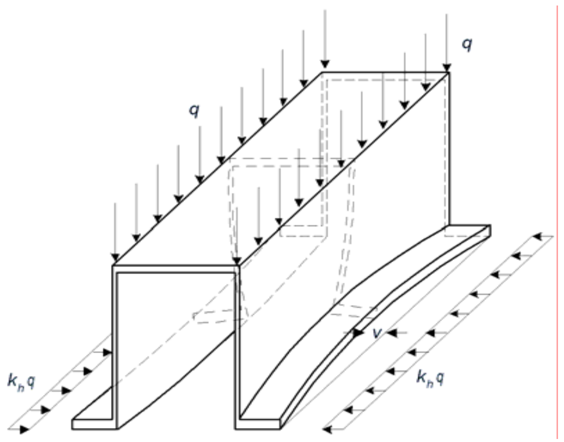
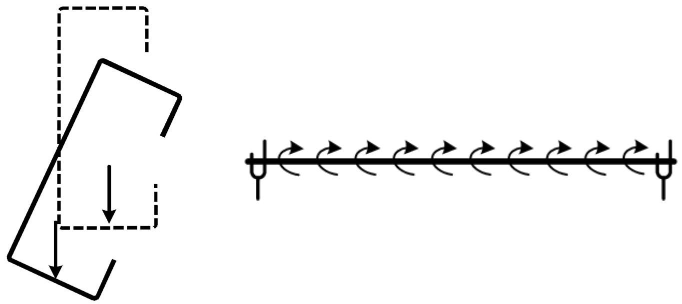
Torsion

No.	Cross section etc.	Description
8.1		Cross section shape to avoid torsion
8.2		Torsion constant for a hollow cross section
8.3		Torsion constant for a deck profile
8.4		Torsion and bending of thin-walled section

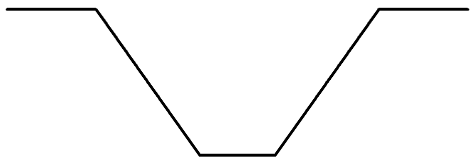
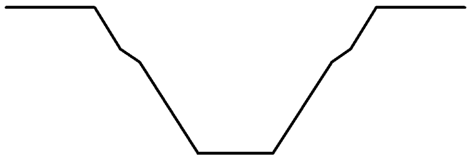
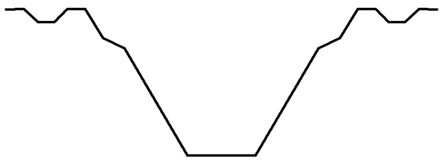
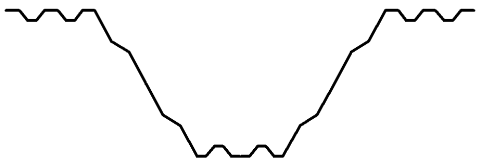
Axial force and bending moment

No.	Cross section etc.	Description
9.1		Tensile force and bending moment
9.2		Beam-column with rectangular hollow section
9.3		Beam-column with eccentric load
9.4		Eccentrically loaded beam-column with cross weld
9.5		Axial force resistance of cantilever column fixed to ground with bolted foot plate or fixed into a concrete block

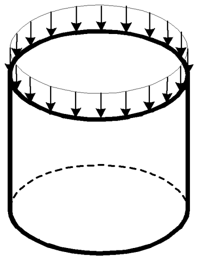
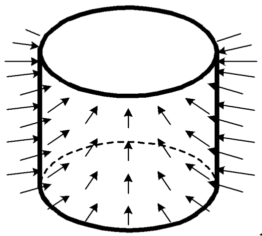
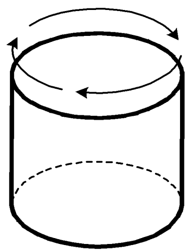
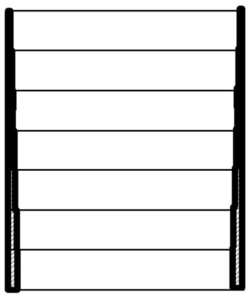
Nonlinear stress distribution

No.	Cross section etc.	Description
10.1		Transverse bending of asymmetric flanges
		Warping and bending of thin-walled member in torsion and bending. See 8.4 and also 2.2

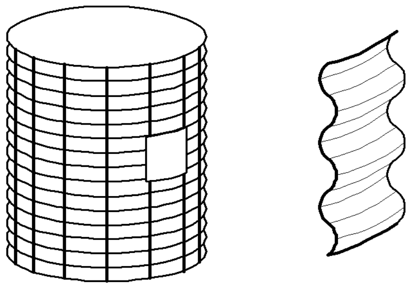
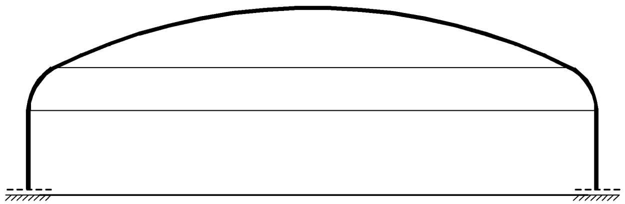
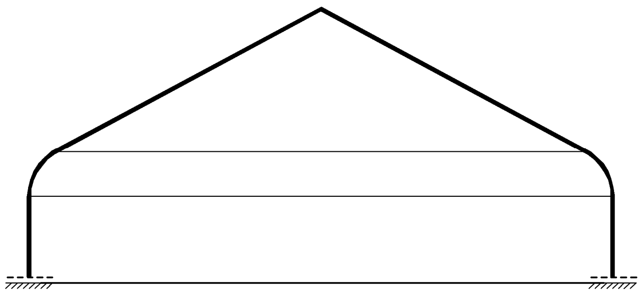
Trapezoidal sheeting

No.	Cross section etc.	Description
11.1		Trapezoidal sheeting without stiffeners
11.2		Trapezoidal sheeting with one stiffener in the webs
11.3		Trapezoidal sheeting with one stiffener in the top flanges and one stiffener in the webs
11.4		Trapezoidal sheeting with two stiffeners in the flanges and two stiffeners in the webs

Cylindrical shells

No.	Cross section etc.	Description
12.1		Cylindrical shell in a) Meridional (axial) compression and bending b) Meridional (axial) compression with coexistent internal pressure
12.2		Cylindrical shell in circumferential compression
12.3		Cylindrical shell in shear
12.4		Cylindrical shell with stepwise wall thickness in circumferential compression

Stiffened shells and shells with torus parts

No.	Cross section etc.	Description
12.5		Horizontally corrugated wall treated as an orthotropic shell. Axial compression and external pressure
12.6		Welded torispherical shell under external pressure
12.7		Welded toriconical shell under external pressure