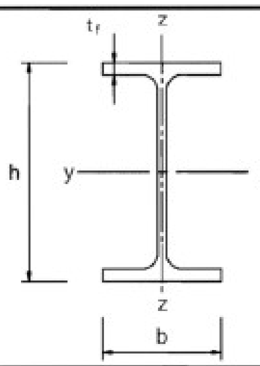
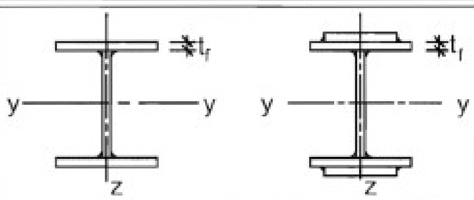
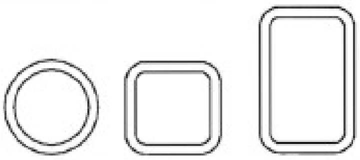
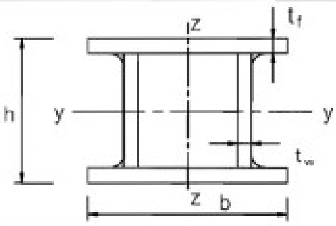
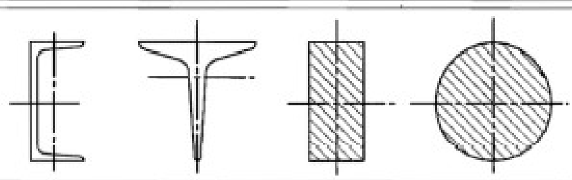
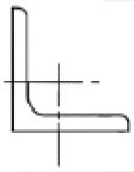
Steel Structures 1.

Cross section		Limits		Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S 460
Rolled sections		$h/b > 1{,}2$	$t_f \le 40$ mm	y – y z – z	a b	a_0 a_0
		$h/b > 1{,}2$	40 mm $< t_f \le 100$	y – y z – z	b c	a a
		$h/b \le 1{,}2$	$t_f \le 100$ mm	y – y z – z	b c	a a
		$h/b \le 1{,}2$	$t_f > 100$ mm	y – y z – z	d d	c c
Welded I-sections			$t_f \le 40$ mm	y – y z – z	b c	b c
			$t_f > 40$ mm	y – y z – z	c d	c d
Hollow sections			hot finished	any	a	a_0
			cold formed	any	c	c
Welded box sections			generally (except as below)	any	b	b
			thick welds: $a > 0{,}5t_f$ $b/t_f < 30$ $h/t_w < 30$	any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b

SZÉCHENYI 2020

PÉCSI TUDOMÁNYEGYETEM
UNIVERSITY OF PÉCS

Attila Fülöp

Steel Structures 1.

Pécs

2019

The Steel Structures 1 course material was developed under the project EFOP 3.4.3-16-2016-00005 "Innovative university in a modern city: open-minded, value-driven and inclusive approach in a 21st century higher education model".

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SZÉCHENYI 2020

Magyarország Kormánya

Európai Unió
Európai Szociális Alap

BEFEKTETÉS A JÖVŐBE

Attila Fülöp

Steel Structures 1.

Pécs

2019

A Steel Structures 1. tananyag az EFOP-3.4.3-16-2016-00005 azonosító számú, „Korszerű egyetem a modern városban: Értékközpontúság, nyitottság és befogadó szemlélet egy 21. századi felsőoktatási modellben” című projekt keretében valósul meg.

EFOP-3.4.3-16-2016-00005 Korszerű egyetem a modern városban:
Értékközpontúság, nyitottság és befogadó szemlélet
egy 21. századi felsőoktatási modellben

Dr. Fülöp Attila

Steel Structures 1.

Pécs

2019

A **Steel Structures 1.** c. tananyag az EFOP-3.4.3-16-2016-00005 azonosító számú, „Korszerű egyetem a modern városban: Értékközpontúság, nyitottság és befogadó szemlélet egy 21. századi felsőoktatási modellben” című projekt keretében valósul meg.

EN 1993-1-1 (2005) (English): Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]

Table 3.1: Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel

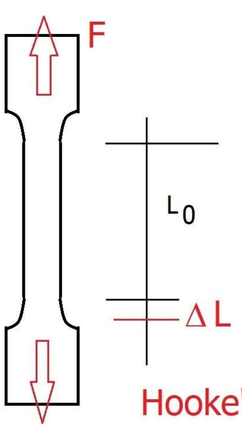
Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 80$ mm	
	f_y [N/mm²]	f_u [N/mm²]	f_y [N/mm²]	f_u [N/mm²]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	[AC2⟩ 490 ⟨AC2]	335	470
S 450	440	550	410	550

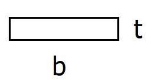
3.2.6 Design values of material coefficients

(1) The material coefficients to be adopted in calculations for the structural steels covered by this Eurocode Part should be taken as follows:

- modulus of elasticity $E = 210\,000 \text{ N/mm}^2$
- shear modulus $G = \dfrac{E}{2(1+\nu)} \approx 81\,000 \text{ N/mm}^2$
- Poisson's ratio in elastic stage $\nu = 0{,}3$
- coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6}$ *per K* (for $T \leq 100$ °C)

NOTE For calculating the structural effects of unequal temperatures in composite concrete-steel structures to EN 1994 the coefficient of linear thermal expansion is taken as $\alpha = 10 \times 10^{-6}$ *per K*.





$A = b \times t$

f_y yield strength

$E = 21\,000$ kN/cm2

$\sigma = \dfrac{F}{A}$

$f_y = 23{,}5 - 35{,}5$ kN/cm2

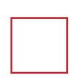
$\varepsilon = \Delta L / L_0$

$f_u = 36{,}0 - 51{,}0$ kN/cm2

Hooke's law: $\sigma = E \times \varepsilon$

f_u ultimate strength

E Young's modulus of elasticity



a= 1 cm

6.2.3 Tension

AC1 (1)P The design value of the tension force N_{Ed} at each cross section shall satisfy: AC1

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1{,}0 \tag{6.5}$$

(2) For sections with holes the design tension resistance $N_{t,Rd}$ should be taken as the smaller of:

a) the design plastic resistance of the gross cross-section

$$N_{pl,Rd} = \frac{A f_y}{\gamma_{M0}} \tag{6.6}$$

b) the design ultimate resistance of the net cross-section at holes for fasteners

$$N_{u,Rd} = \frac{0{,}9 A_{net} f_u}{\gamma_{M2}} \tag{6.7}$$

EN 1993-1-8 (2005) (English): Eurocode 3: Design of steel structures - Part 1-8: Design of joints [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]

3.10.3 Angles connected by one leg and other unsymmetrically connected members in tension

(1) The eccentricity in joints, see 2.7(1), and the effects of the spacing and edge distances of the bolts, should be taken into account in determining the design resistance of:

- unsymmetrical members;
- symmetrical members that are connected unsymmetrically, such as angles connected by one leg.

(2) A single angle in tension connected by a single row of bolts in one leg, see Figure 3.9, may be treated as concentrically loaded over an effective net section for which the design ultimate resistance should be determined as follows:

with 1 bolt: $$N_{u,Rd} = \frac{2{,}0(e_2 - 0{,}5d_0)t\, f_u}{\gamma_{M2}} \qquad \text{... (3.11)}$$

with 2 bolts: $$N_{u,Rd} = \frac{\beta_2\, A_{net}\, f_u}{\gamma_{M2}} \qquad \text{... (3.12)}$$

with 3 or more bolts: $$N_{u,Rd} = \frac{\beta_3 \; A_{net} \; f_u}{\gamma_{M2}} \qquad \text{... (3.13)}$$

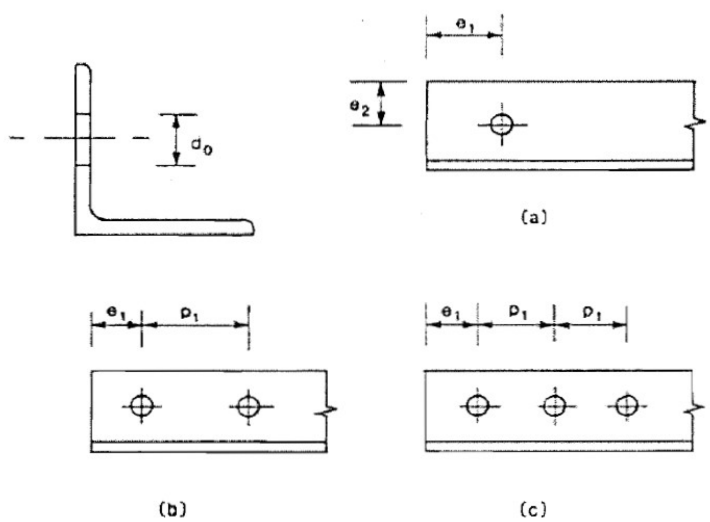
where:

β_2 and β_3 are reduction factors dependent on the pitch p_1 as given in Table 3.8. For intermediate values of p_1 the value of β may be determined by linear interpolation;

A_{net} is the net area of the angle. For an unequal-leg angle connected by its smaller leg, A_{net} should be taken as equal to the net section area of an equivalent equal-leg angle of leg size equal to that of the smaller leg.

Table 3.8: Reduction factors β_2 and β_3

Pitch	p_1	$\leq 2,5\ d_o$	$\geq 5,0\ d_o$
2 bolts	β_2	0,4	0,7
3 bolts or more	β_3	0,5	0,7



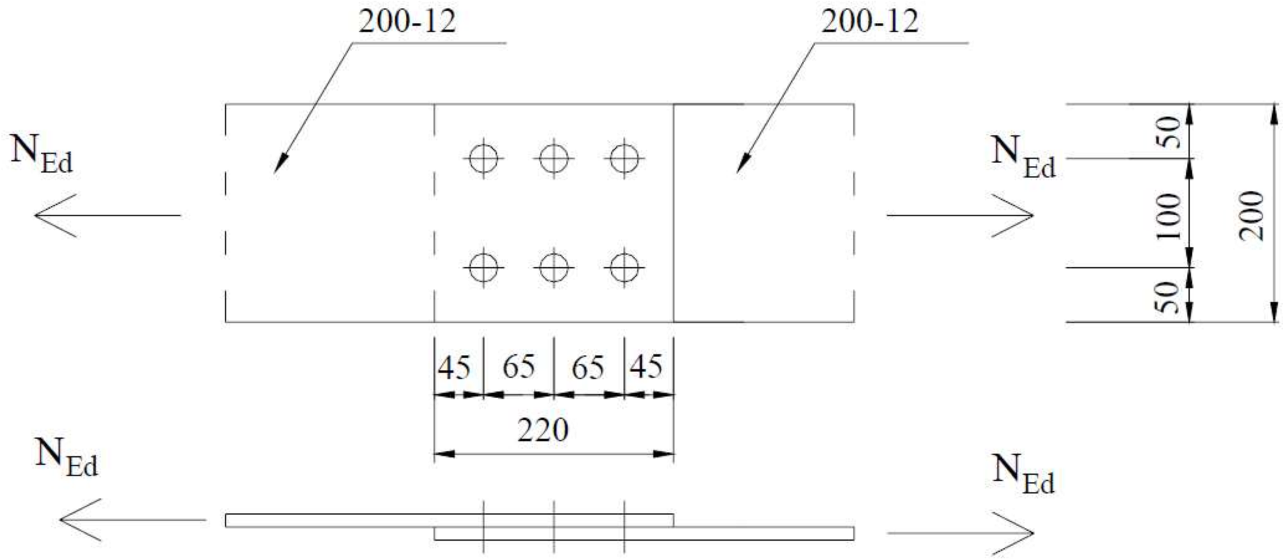
a) 1 bolt

b) 2 bolts

c) 3 bolts

Figure 3.9: Angles connected by one leg

<u>SAMPLE 1</u>



$N_{Ed} = 450kN$

S235 $\quad f_y = 23{,}5\ kN/cm^2 \quad f_u = 36{,}0\ kN/cm^2$

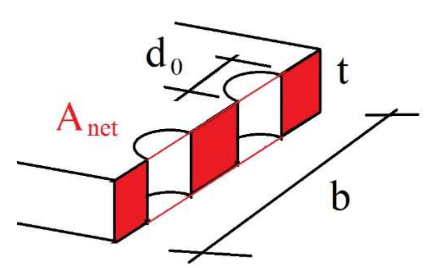
bolts $\quad$ M24, 8.8 $\rightarrow d_0 = 26\,mm$

$$N_{t,Rd} = min\begin{pmatrix} N_{pl,Rd} = \dfrac{A \cdot f_y}{\gamma_{M0}} \\ \\ N_{u,Rd} = 0{,}9 \cdot \dfrac{A_{net} \cdot f_u}{\gamma_{M2}} \end{pmatrix}$$

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{20 \cdot 1{,}2 \cdot 23{,}5}{1{,}0} = 564{,}0\ kN$$

$$N_{u,Rd} = 0{,}9 \cdot \frac{A_{net} \cdot f_u}{\gamma_{M2}} = 0{,}9 \cdot \frac{(20 - 2 \cdot 2{,}6) \cdot 1{,}2 \cdot 36}{1{,}25} = 460{,}3\ kN$$

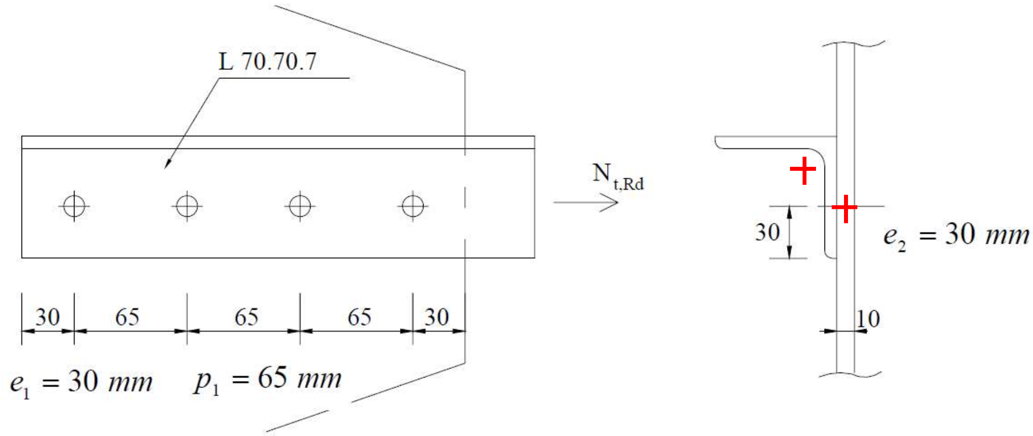
$$N_{t,Rd} = N_{u,Rd} = 460{,}3\ kN \geq N_{Ed} = 450\ kN$$



$A_{net} = A - 2 \times d_0 \times t = b \times t - 2 \times d_0 \times t = (b - 2 \times d_0) \times t$

$N_{Ed} / N_{t.Rd} = 450 / 460{,}3 = 0.98 < 1.0 \rightarrow$ satisfies! OK!

SAMPLE 2



L70.70.7 $A = 9{,}4\ cm^2$

S275 $f_y = 27{,}5\ kN/cm^2$ $f_u = 43{,}0\ kN/cm^2$

M16, 8.8 $\rightarrow d_0 = 18\ mm$

$$N_{t,Rd} = min\begin{pmatrix} N_{pl,Rd} = \dfrac{A \cdot f_y}{\gamma_{M0}} \\ \\ N_{u,Rd} = \beta \cdot \dfrac{A_{net} \cdot f_u}{\gamma_{M2}} \end{pmatrix}$$

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{9{,}4 \cdot 27{,}5}{1{,}0} = 258{,}5\ kN$$

Table 3.8: Reduction factors β_2 and β_3

Pitch	p_1	$\leq 2{,}5\ d_o$	$\geq 5{,}0\ d_o$
2 bolts	β_2	0,4	0,7
3 bolts or more	β_3	0,5	0,7

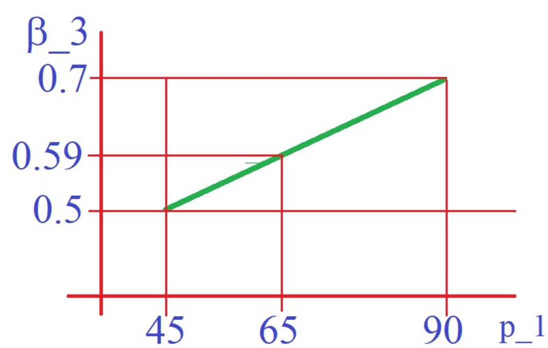
2.5 × 18 = 45 mm < p_1 = 65 mm < 5 × 18 mm = 90 mm

β = 0.5 + (0.7 - 0.5) / (90 - 45) × (65 – 45) = 0,59

$$\beta = 0{,}3 + 0{,}08\frac{65}{18} = 0{,}59$$

$$N_{u,Rd} = 0{,}59 \cdot \frac{A_{net} \cdot f_u}{\gamma_{M2}} = 0{,}59 \cdot \frac{(9{,}4 - 1{,}8 \cdot 0{,}7) \cdot 43}{1{,}25} = 165{,}2\ kN$$

$$N_{t,Rd} = N_{u,Rd} = 165{,}2\ kN$$



6.2.4 Compression

AC1 (1)P The design value of the compression force N_{Ed} at each cross-section shall satisfy: AC1

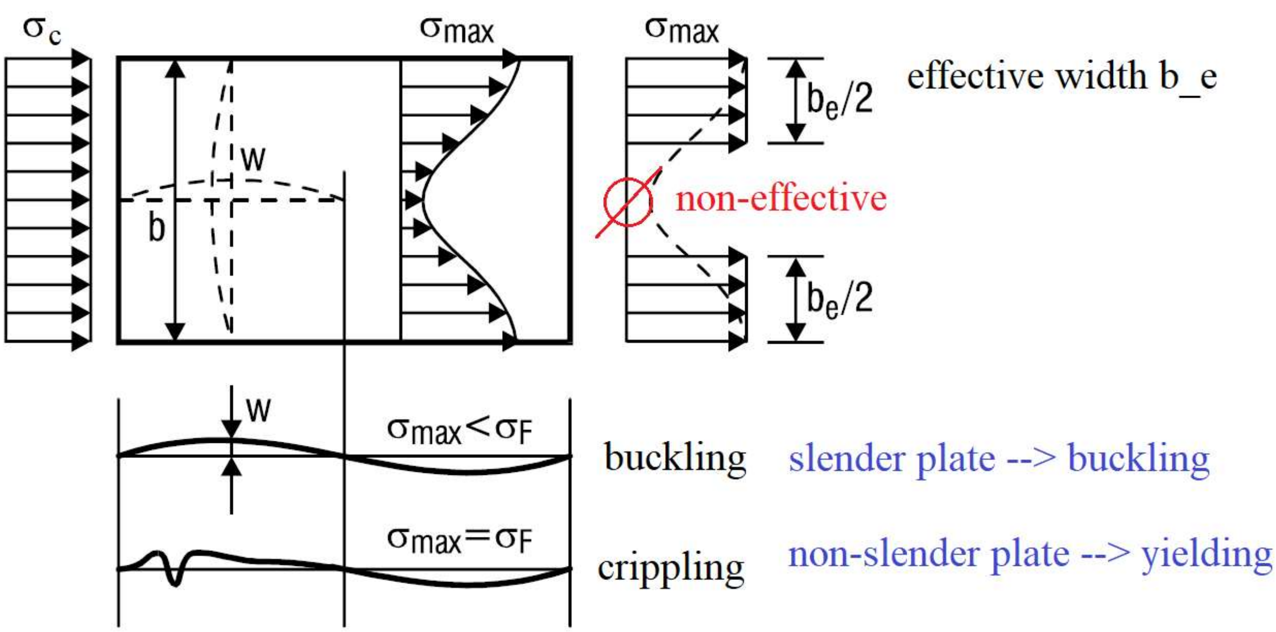
$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1{,}0 \tag{6.9}$$

(2) The design resistance of the cross-section for uniform compression $N_{c,Rd}$ should be determined as follows:

$$N_{c,Rd} = \frac{A f_y}{\gamma_{M0}} \quad \text{for class 1, 2 or 3 cross-sections} \tag{6.10}$$

$$N_{c,Rd} = \frac{A_{eff} f_y}{\gamma_{M0}} \quad \text{for class 4 cross-sections} \tag{6.11}$$

(3) Fastener holes except for oversize and slotted holes as defined in EN 1090 need not be allowed for in compression members, provided that they are filled by fasteners.



EN 1993-1-1 (2005) (English): Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings

5.5 Classification of cross sections

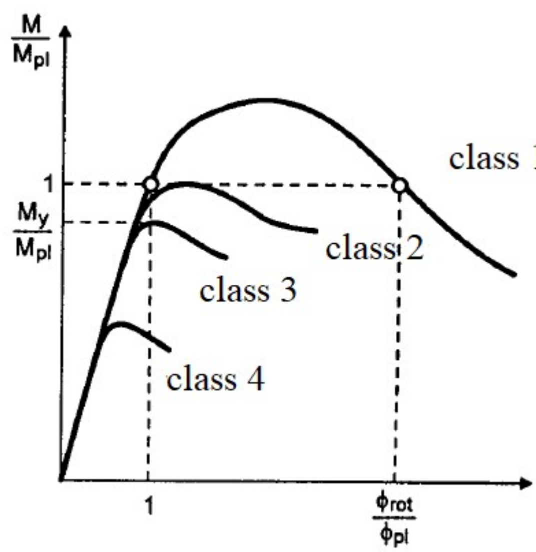
5.5.1 Basis

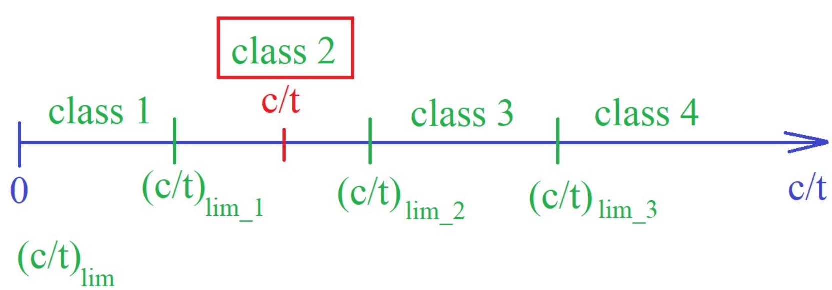
(1) The role of cross section classification is to identify the extent to which the resistance and rotation capacity of cross sections is limited by its local buckling resistance.

5.5.2 Classification

(1) Four classes of cross-sections are defined, as follows:

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.
- Class 3 cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.



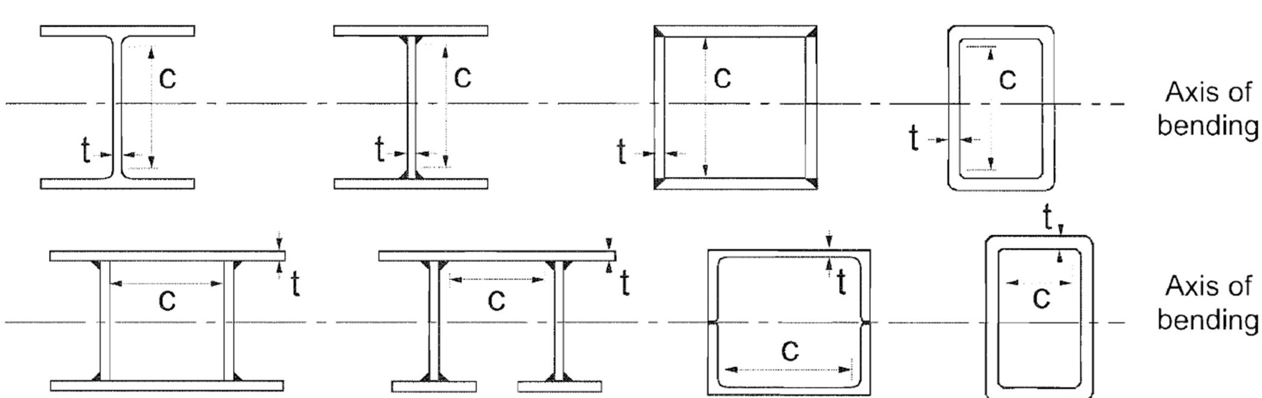


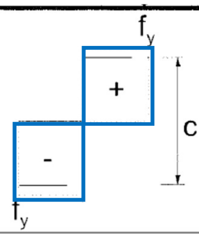
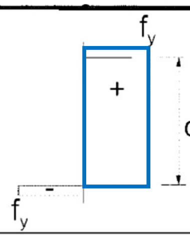
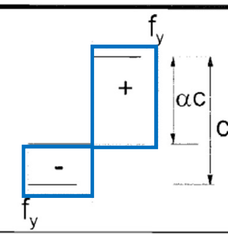
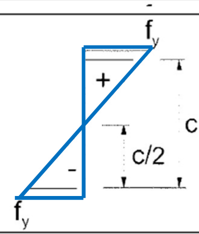
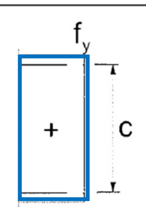
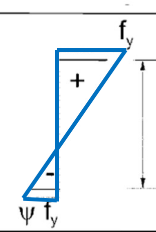
1 - internal webplate / outstand flanges
2 - internal forces --> N(-), M, N(-) - M
3 - steel grade --> ε

width-to-thickness ratio (plate slenderness) c / t >=< $(c/t)_{limit}$ → class of the plate

Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts

Internal compression parts



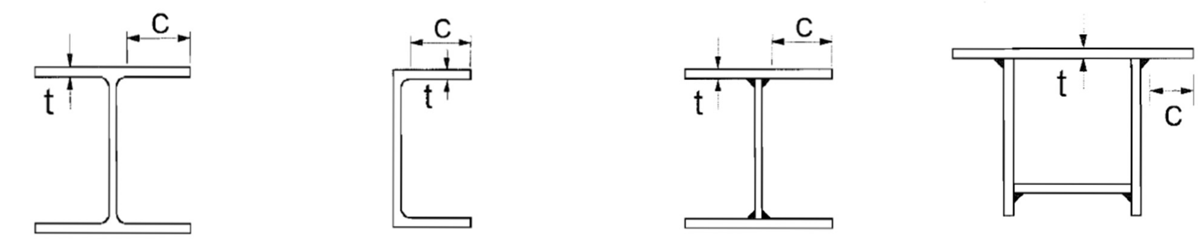
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1^{*)}$: $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

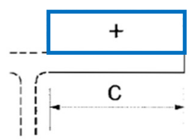
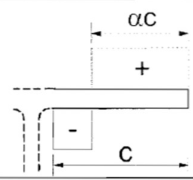
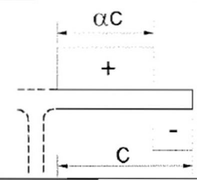
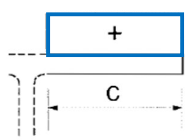
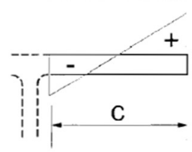
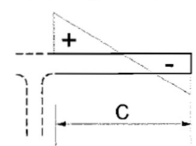
*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

Outstand flanges



Rolled sections | Welded sections

Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \leq 9\varepsilon$	$c/t \leq \frac{9\varepsilon}{\alpha}$	$c/t \leq \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \leq 10\varepsilon$	$c/t \leq \frac{10\varepsilon}{\alpha}$	$c/t \leq \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

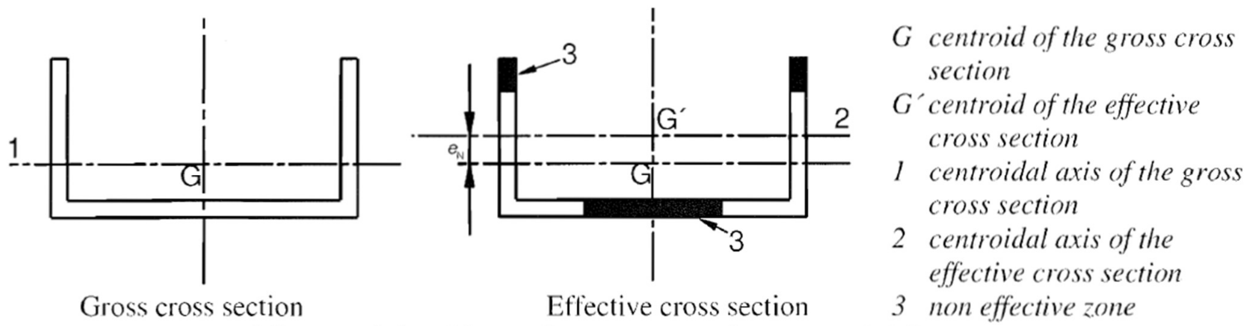


Figure 4.1: Class 4 cross-sections - axial force

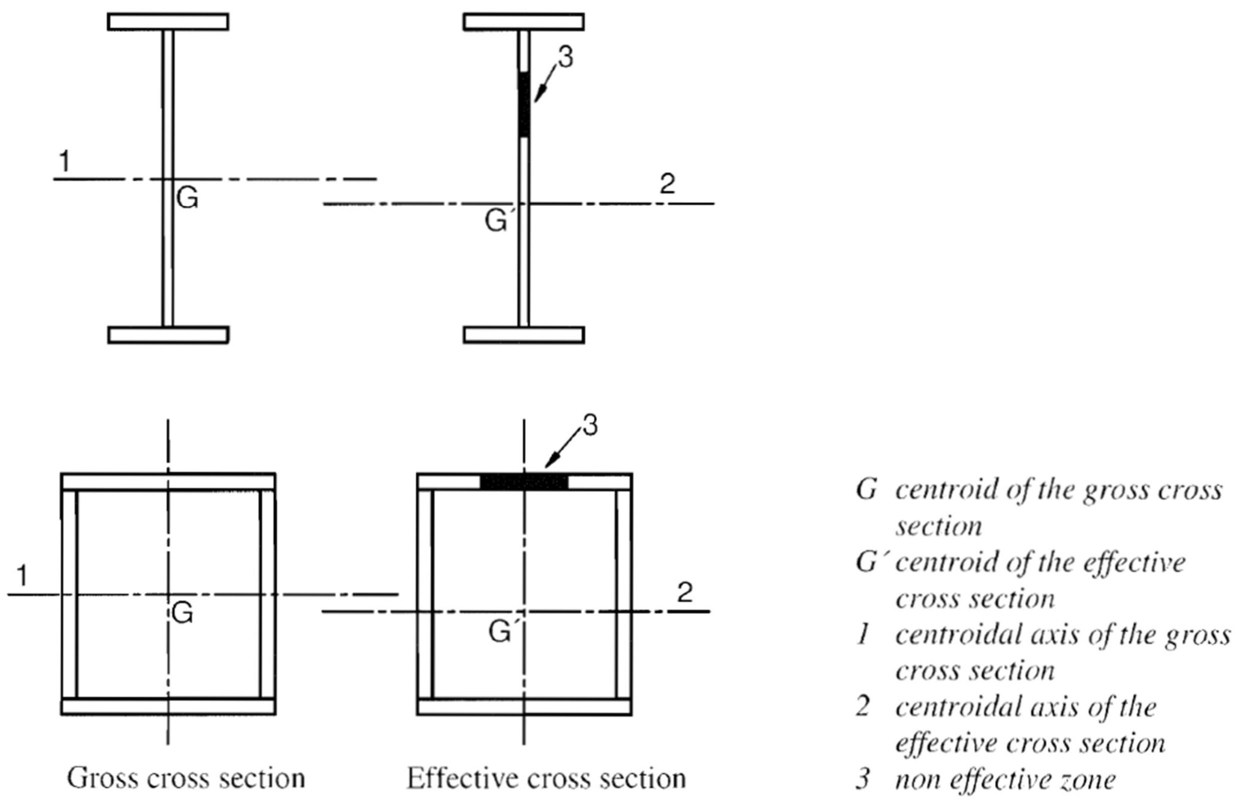


Figure 4.2: Class 4 cross-sections - bending moment

4.4 Plate elements without longitudinal stiffeners

(1) The effective[p] areas of flat compression elements should be obtained using Table 4.1 for internal elements and Table 4.2 for outstand elements. The effective[p] area of the compression zone of a plate with the gross cross-sectional area A_c should be obtained from:

$$A_{c,eff} = \rho\, A_c \tag{4.1}$$

where ρ is the reduction factor for plate buckling.

(2) The reduction factor ρ may be taken as follows:

- internal compression elements:

$$\rho = 1{,}0$$

$$\rho = \frac{\overline{\lambda}_p - 0{,}055\,(3+\psi)}{\overline{\lambda}_p^2} \le 1{,}0$$

- outstand compression elements:

$$\rho = 1{,}0$$

$$\rho = \frac{\overline{\lambda}_p - 0{,}188}{\overline{\lambda}_p^2} \le 1{,}0$$

where $\overline{\lambda}_p = \sqrt{\dfrac{f_y}{\sigma_{cr}}} = \dfrac{\overline{b}/t}{28{,}4\,\varepsilon\,\sqrt{k_\sigma}}$

ψ is the stress ratio determined in accordance with 4.4(3) and 4.4(4)

$\overline{b}$ is the appropriate width to be taken as follows (for definitions, see Table 5.2 of EN 1993-1-1)

b_w	for webs;
b	for internal flange elements (except RHS);
b - 3 t	for flanges of RHS;
c	for outstand flanges;
h	for equal-leg angles;
h	for unequal-leg angles;

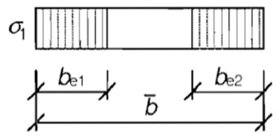
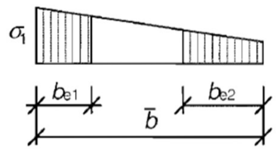
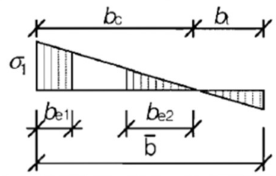
k_σ is the buckling factor corresponding to the stress ratio ψ and boundary conditions. For long plates k_σ is given in Table 4.1 or Table 4.2 as appropriate;

t is the thickness;

σ_{cr} is the elastic critical plate buckling stress see equation (A.1) in Annex A.1(2) and Table 4.1 and Table 4.2;

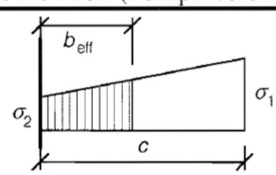
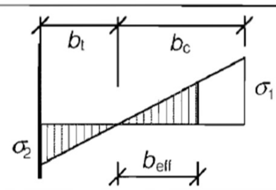
$$\varepsilon = \sqrt{\frac{235}{f_y\left[N/mm^2\right]}}$$

Table 4.1: Internal compression elements

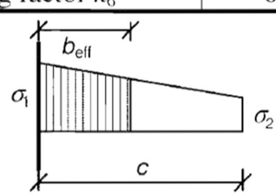
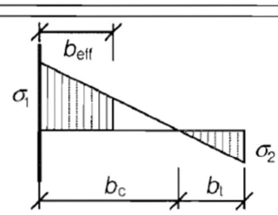
Stress distribution (compression positive)	Effective^{p} width b_{eff}
	<u>$\psi = 1$:</u> $b_{eff} = \rho\ \bar{b}$ $b_{e1} = 0{,}5\ b_{eff}$ $\quad b_{e2} = 0{,}5\ b_{eff}$
	<u>$1 > \psi \geq 0$:</u> $b_{eff} = \rho\ \bar{b}$ $b_{e1} = \dfrac{2}{5-\psi} b_{eff}$ $\quad b_{e2} = b_{eff} - b_{e1}$
	<u>$\psi < 0$:</u> $b_{eff} = \rho\ b_c = \rho\ \bar{b} / (1-\psi)$ $b_{e1} = 0{,}4\ b_{eff}$ $\quad b_{e2} = 0{,}6\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1	[AC1] $-1 > \psi \geq -3$ [AC1]
Buckling factor k_σ	4,0	$8{,}2 / (1{,}05 + \psi)$	7,81	$7{,}81 - 6{,}29\psi + 9{,}78\psi^2$	23,9	$5{,}98\ (1 - \psi)^2$

Table 4.2: Outstand compression elements

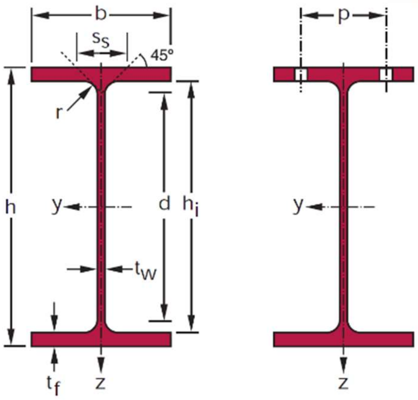
Stress distribution (compression positive)	Effective^{p} width b_{eff}
	<u>$1 > \psi \geq 0$:</u> $b_{eff} = \rho\ c$
	<u>$\psi < 0$:</u> $b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	0	-1	$1 \geq \psi \geq -3$
Buckling factor k_σ	0,43	0,57	0,85	$0{,}57 - 0{,}21\psi + 0{,}07\psi^2$

Stress distribution (compression positive)	Effective^{p} width b_{eff}
	<u>$1 > \psi \geq 0$:</u> $b_{eff} = \rho\ c$
	<u>$\psi < 0$:</u> $b_{eff} = \rho\ b_c = \rho\ c / (1-\psi)$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0{,}578 / (\psi + 0{,}34)$	1,70	$1{,}7 - 5\psi + 17{,}1\psi^2$	23,8

SAMPLE 1



HE 280 B column, $N_{c,Rd}$ = ?

Steel grade S235

$f_y = 235\ N/mm^2$, $f_u = 360\ N/mm^2$, $\varepsilon = 1{,}0$

<u>classification</u>:

flange: $c_f / t_f = (b_f / 2 - t_w / 2 - r) / t_f = (28/2 - 1{,}05/2 - 2{,}4) / 1{,}8 = \underline{6.15}$

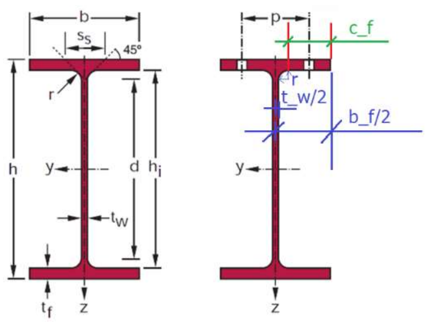
$c_f / t_f = \underline{6.15} < 9\varepsilon = 9 \rightarrow$ class 1

compressed flange 9ε ; 10ε ; 14ε ,

web: $c_w / t_w = (h - 2 \times t_f - 2 \times r) / t_w = (280 - 2 \times 18 - 2 \times 24) / 10{,}5 = 196 / 10{,}5 = \underline{18.7} < 33\varepsilon = 33 \rightarrow$ class 1

compressed web: 33ε ; 38ε ; 42ε

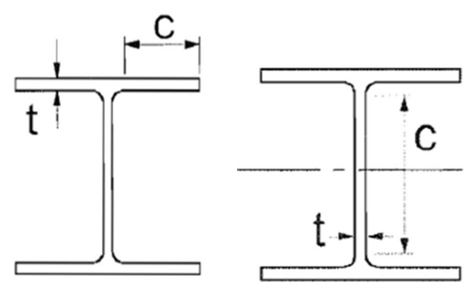
The HE 280 B column in case pure compression is **<u>class 1</u>**.



$$N_{c,Rd} = \frac{A\, f_y}{\gamma_{M0}}$$

$N_{c.Rd} = 131.4 \times 23.5 / 1.0$

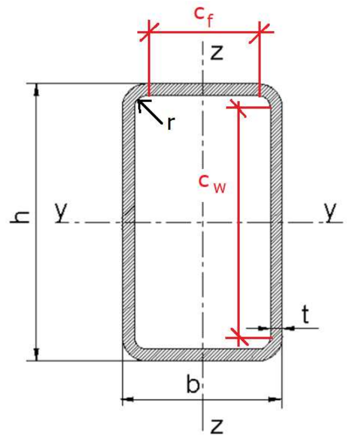
$N_{c.Rd} = \underline{3\,087.9}$ kN



Denominación Designation Designazione G kg/m	Dimensiones Dimensions Dimensioni h mm	b mm	t_w mm	t_f mm	r mm	A mm^2 x 10^2	Dimensiones de construcción Dimensions for detailing Dimensioni di costruzione h_i mm	d mm	Ø	p_{min} mm	p_{max} mm	Superficie Surface Superficie A_L m^2/m	A_G m^2/t
HE 280 AA* 61,2	264	280	7	10	24	78,0	244	196	M 27	110	178	1,593	26,01
HE 280 A 76,4	270	280	8	13	24	97,3	244	196	M 27	112	178	1,603	20,99
HE 280 B 103	280	280	10,5	18	24	131,4	244	196	M 27	114	178	1,618	15,69
HE 280 M 189	310	288	18,5	33	24	240,2	244	196	M 27	122	186	1,694	8,984

Denominación Designation Designazione G kg/m	Propiedades del perfil / Section properties / Proprietà del profilato — eje fuerte y-y strong axis y-y asse forte y-y I_y mm^4 x 10^4	$W_{el.y}$ mm^3 x 10^3	$W_{pl.y}$♦ mm^3 x 10^3	i_y mm x 10	A_{vz} mm^2 x 10^2	eje débil z-z weak axis z-z asse debole z-z I_z mm^4 x 10^4	$W_{el.z}$ mm^3 x 10^3	$W_{pl.z}$♦ mm^3 x 10^3	i_z mm x 10	s_s mm	I_t mm^4 x 10^4	I_w mm^6 x 10^9	Classification ENV 1993-1-1 pure bending y-y S 235	S 355	S 460	pure compression S 235	S 355	S 460	EN 10025:1993	EN 10113-3:1993	EN 10225:2001
HE 280 AA 61,2	10560	799,8	873,1	11,63	27,52	3664	261,7	399,4	6,85	55,12	36,22	590,1	3	4	4	3	4	4	✓	✓	✓
HE 280 A 76,4	13670	1013	1112	11,86	31,74	4763	340,2	518,1	7,00	62,12	62,10	785,4	2	3	4	2	3	4	✓	HI	HI
HE 280 B 103	19270	1376	1534	12,11	41,09	6595	471,0	717,6	7,09	74,62	143,7	1130	1	1	2	1	1	2	✓	HI	HI
HE 280 M 189	39550	2551	2966	12,83	72,03	13160	914,1	1397	7,40	112,6	807,3	2520	1	1	1	1	1	1	✓	HI	HI

SAMPLE 2



60 × 40 × 2,5 rectangular hollow section, $N_{c,Rd}$ = ?

Steel grade S275

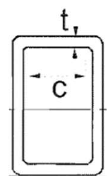
f_y = 275 N/mm², f_u = 430 N/mm², ε = 0,92

folding radius r = t!

<u>classification</u>:

flange: $c_f / t_f = (b - 2 \times t - 2 \times r) / t = (40 - 2\times2{,}5 - 2\times2{,}5) / 2{,}5 = \underline{12}$

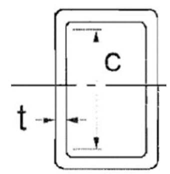
$c_f / t_f = \underline{12} < 33\varepsilon = 33 \times 0{,}92 = 30{,}36$ → class 1



compressed flange 33ε ; 38ε ; 42ε,

web: $c_w / t_w = (h - 2 \times t - 2 \times r) / t = (60 - 2 \times 2{,}5 - 2 \times 2{,}5) / 2{,}5 = \underline{20} < 33\varepsilon = 33 \times 0{,}92 = 30{,}36$ → class 1

compressed web: 33ε ; 38ε ; 42ε



60 × 40 × 2,5 rectangular hollow section in case pure compression is <u>**class 1**</u>.

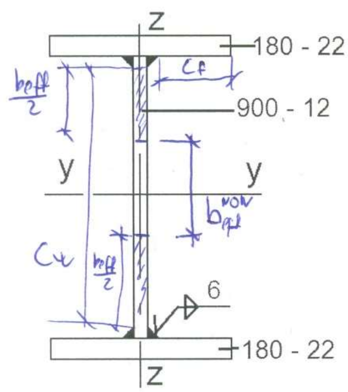
$$N_{c,Rd} = \frac{A f_y}{\gamma_{M0}}$$

$N_{c.Rd}$ = 4.7 × 27.5 / 1.0 = 129,25 kN

h × b	t	M	A	I		i		W_{el}		W_{pl}		I_T	W_T	A_L	L
				y-y	z-z	y-y	z-z	y-y	z-z	y-y	z-z				
mm	mm	kg/m	cm²	cm⁴	cm⁴	cm	cm	cm³	cm³	cm³	cm³	cm⁴	cm³	m²/m	m
50 × 30	2,5	2,90	3,70	11,9	5,28	1,80	1,20	4,78	3,52	5,96	4,14	11,5	6,52	0,156	345
	3,0	3,42	4,36	13,8	6,02	1,78	1,17	5,50	4,01	6,95	4,80	13,2	7,60	0,155	292
	3,2	3,63	4,62	14,4	6,29	1,77	1,17	5,77	4,20	7,32	5,05	13,9	8,01	0,155	276
	4,0	4,41	5,62	16,9	7,25	1,73	1,14	6,75	4,83	8,72	5,97	16,2	9,54	0,153	227
	5,0	5,33	6,79	19,4	8,17	1,69	1,10	7,74	5,45	10,2	6,95	15,5	11,2	0,151	188
60 ×40	2,5	3,69	4,70	23,0	12,1	2,21	1,61	7,67	6,07	9,37	7,05	24,7	10,8	0,196	271
	3,0	4,37	5,56	26,7	14,0	2,19	1,59	8,91	7,01	11,0	8,24	28,7	12,6	0,195	229
	3,2	4,63	5,90	28,1	14,7	2,18	1,58	9,38	7,36	11,6	8,70	30,3	13,4	0,195	216
	4,0	5,67	7,22	33,3	17,3	2,15	1,55	11,1	8,65	14,0	10,4	35,9	16,1	0,193	176
	5,0	6,90	8,79	39,0	20,0	2,11	1,51	13,0	10,0	16,6	12,3	41,9	19,2	0,191	145
	6,0	8,05	10,3	43,7	22,1	2,06	1,47	14,6	11,1	19,0	14,0	47,0	21,9	0,190	124
	6,3	8,38	10,7	44,9	22,7	2,05	1,46	15,0	11,3	19,6	14,5	48,3	22,7	0,189	119

SAMPLE 3

1. Determine the **tension and compression resistances** of the welded steel I-section (classification)! Steel grade S235: f_y = 235 N/mm², f_u = 360 N/mm², ε = 1,0 (compressed flange 9ε ; 10ε ; 14ε , compressed web: 33ε ; 38ε ; 42ε) $k_\sigma = 4$



$A = 90 \cdot 1{,}2 + 2 \cdot 18 \cdot 2{,}2 = \underline{187{,}2\ cm^2}$

$N_{pl,Rd} = \dfrac{187{,}2 \cdot 23{,}5}{1{,}0} = \boxed{4399{,}2\ kN}$

Classification

flange $\dfrac{c_f}{t_f} = \dfrac{\frac{180}{2} - \frac{12}{2} - 6 \cdot \sqrt{2}}{22} = \dfrac{75{,}51}{22} = 3{,}43 < 9 \rightarrow$ class 1

web $= \dfrac{c_w}{t_w} = \dfrac{900 - 2 \cdot 6 \cdot \sqrt{2}}{12} = \dfrac{883}{12} = 73{,}59 > 42 \rightarrow$ class 4

I-section class 4.

Effective area

web: $\bar{\lambda}_p = \dfrac{73{,}59}{28{,}4 \cdot 1 \cdot \sqrt{4}} = 1{,}30 \rightarrow \rho = \dfrac{1{,}30 - 0{,}055\,(3+1)}{1{,}30^2} = 0{,}639 < 1$ ✓

$b_{eff} = \rho \cdot c_w = 0{,}639 \cdot 883 = 564{,}2\ mm$

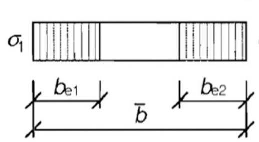
$b_{eff}^{NON} = 883 - 564{,}2 = 318{,}8\ mm$

$A_{eff} = A - b_{eff}^{NON} \cdot t_w = 187{,}2 - 31{,}88 \cdot 1{,}2 = 148{,}9\ cm^2$

$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\sqrt{k_\sigma}} \qquad \rho = \frac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{\bar{\lambda}_p^2} \le 1{,}0$$

$N_{c,Rd} = \dfrac{148{,}9 \cdot 23{,}5}{1{,}0} = \boxed{3499{,}2\ kN}$

$$N_{c,Rd} = \frac{A_{eff}\, f_y}{\gamma_{M0}}$$

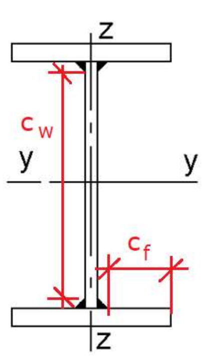


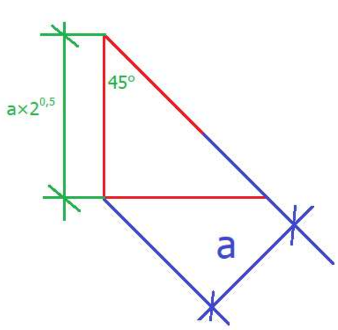
$\psi = 1$:

$b_{eff} = \rho\ \bar{b}$

$b_{e1} = 0{,}5\ b_{eff}$ $\quad b_{e2} = 0{,}5\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	1
Buckling factor k_σ	4,0





Classification:

flange:

$c_f / t_f = (b_f / 2 - t_w / 2 - a \times 2^{0.5}) / t_f$

web:

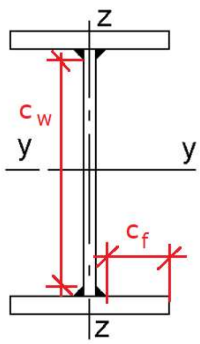
$c_w / t_w = (h_w - 2 \times a \times 2^{0.5}) / t_w$

SAMPLE 4

Determine the **tension and compression resistances** of the welded steel I-section (classification)!

Steel grade S355: $f_y = 355\ N/mm^2 = 35{,}5\ kN/cm^2$, $f_u = 510\ N/mm^2$, $\varepsilon = 0{,}81$

(compressed flange 9ε ; 10ε ; 14ε , compressed web: 33ε ; 38ε ; 42ε), $k_{\sigma.flange} = 0{,}43$, $k_{\sigma.web} = 4{,}0$



flange 400-14, web 1100-8, a = 4 mm

$A = 110 \times 0{,}8 + 2 \times 40 \times 1{,}4 = 200\ cm^2$

$N_{t.Rd} = A \times f_y / \gamma_{M0} = 200 \times 35{,}5 / 1{,}0 =$ **7 100** kN

Classification:

flange: $c_f / t_f = (b_f / 2 - t_w / 2 - a \times 2^{0.5}) / t_f = (400/ 2 - 8/ 2 - 4 \times 2^{0.5}) / 14 =$

$= 190{,}3 / 14 =$ **13,6** $> 14 \times \varepsilon = 14 \times 0{,}81 = 11{,}34 \rightarrow$ class 4

web: $c_w / t_w = (h_w - 2 \times a \times 2^{0.5}) = (1100 - 2 \times 4 \times 2^{0.5}) / 8 = 1088 / 8 =$ **136,1** $> 42 \times \varepsilon = 42 \times 0{,}81 =$

$= 34{,}02 \rightarrow$ class 4

➔ welded I-section in case of pure compression with the steel grade S355 **class 4 section**!

Effective width – effective area:

flange:

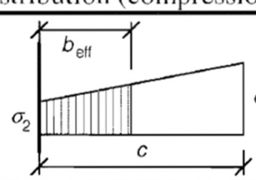
$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\sqrt{k_\sigma}}$$

$= 190.3 / 28.4 / 0.81 / 0.43^{0.5} = 12{,}62$

$$\rho = \frac{\bar{\lambda}_p - 0{,}188}{\bar{\lambda}_p^2} \le 1{,}0$$

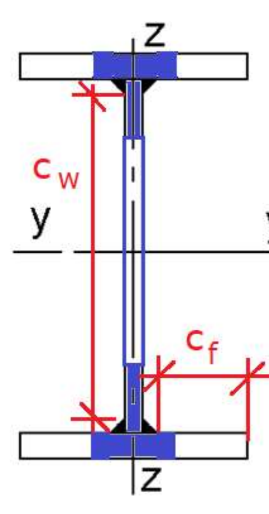
$= (12{,}62 - 0.188) / 12.62^2 = 0.078$

$b_{eff} = 0{,}078 \times 190{,}3 = 14{,}8$ mm

Stress distribution (compression positive)	Effective^p width b_{eff}
	$1 > \psi \ge 0$: $b_{eff} = \rho\, c$

$\psi = \sigma_2/\sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_σ	0,43	$0{,}578 / (\psi + 0{,}34)$	1,70	$1{,}7 - 5\psi + 17{,}1\psi^2$	23,8

web:



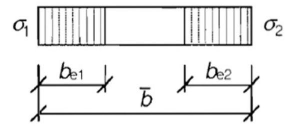
$$\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28{,}4\,\varepsilon\sqrt{k_\sigma}}$$

$= 136.1 / 28.4 / 0.81 / 4^{0.5} = 2.96$

$$\rho = \frac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{\bar{\lambda}_p^2} \le 1{,}0$$

$= (2.96 - 0.055 \times (3+1)) / 2.96^2 = 0.31$

$b_{eff.w} = \rho \times c_w = 0.31 \times 1088 = 337.3$ mm
$b_{e1} = b_{e2} = 0.5 \times b_{eff} = 0.5 \times 337.3 = 168.6$ mm
$b_{eff}{}^{NON}{}_{.w} = (1 - \rho) \times c_w = (1 - 0.31) \times 1088 = 750.7$ mm



$\underline{\psi = 1}$:

$b_{eff} = \rho\ \bar{b}$

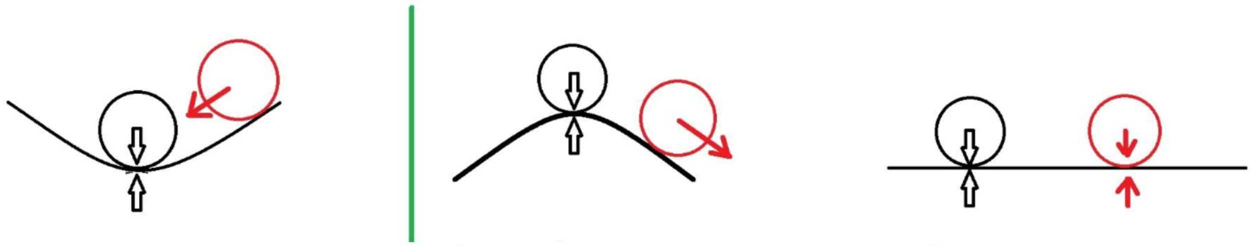
$b_{e1} = 0{,}5\ b_{eff}$ $\quad b_{e2} = 0{,}5\ b_{eff}$

$\psi = \sigma_2/\sigma_1$	1
Buckling factor k_σ	4,0

$A_{eff} = A - (b_{eff}{}^{NON}{}_{.w} \times t_w + 4 \times b_{eff}{}^{NON}{}_{.f} \times t_f = 200 - (75.07 \times 0.8 + 4 \times 2.3 \times 1.4) = 127.06$ cm^2

$N_{c.Rd} = \dfrac{A_{eff}\ f_y}{\gamma_{M0}}$ $= 127.06 \times 35.5 / 1.0 = \underline{4\ 510.6}$ kN

Stability theory

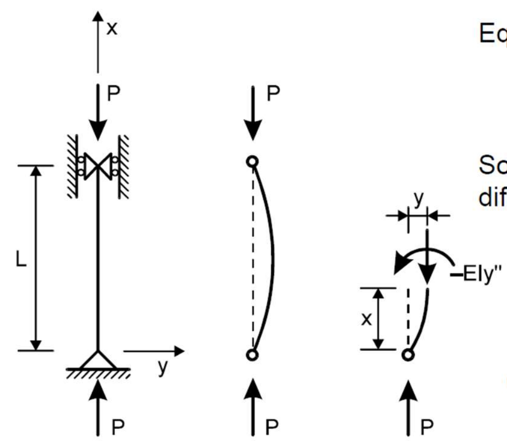


returns --> STABLE

does not return --> UNSTABLE

does not return --> INDIFFERENT - UNSTABLE

Compression load:



Internal resisting moment: $M_x = -EI \cdot y''$

Equilibrium equation: $EI \cdot y'' + P \cdot y = 0$

$$k^2 = \frac{P}{EI} \qquad y'' + k^2 \cdot y = 0$$

Solution of the linear homogeneous differential equation:

$$y = K \cdot e^{m \cdot x} \qquad K \cdot e^{m \cdot x} \cdot (m^2 + k^2) = 0$$

$$m = \pm k \cdot \mathrm{i} \qquad y = C_1 \cdot e^{k \cdot x \cdot \mathrm{i}} + C_2 \cdot e^{-k \cdot x \cdot \mathrm{i}}$$

$$e^{\pm k \cdot x \cdot i} = \cos kx \pm \mathrm{i} \cdot \sin kx \qquad A = C_1 \cdot \mathrm{i} - C_2 \cdot \mathrm{i}; \quad B = C_1 + C_2,$$

$$\boxed{y = A \cdot \sin kx + B \cdot \cos kx}$$

Euler-column [1744]

Boundary conditions:

$$x = 0 \quad \rightarrow \quad y = 0;$$
$$x = L \quad \rightarrow \quad y = 0.$$

First of these conditions:

$$B = 0 \qquad y = A \cdot \sin kx$$

Second of these conditions:

$$A \cdot \sin kL = 0$$

Trivial solution: $A = 0$

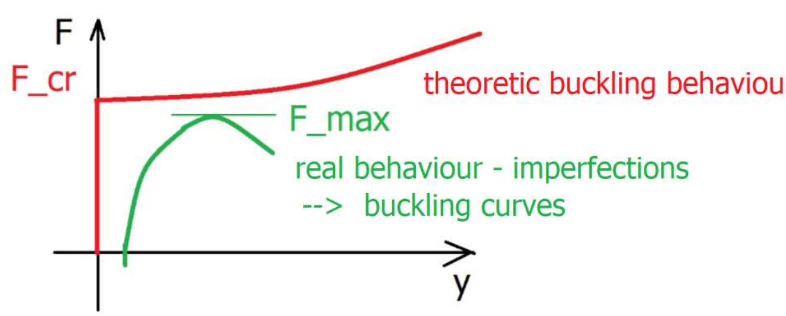
Critical solution: $\sin kL = 0$

$$k \cdot L = n \cdot \pi \qquad n = 1,2,\ldots$$

$$P = \frac{n^2 \cdot \pi^2 \cdot EI}{L^2} \qquad y = A \cdot \sin \frac{n \cdot \pi \cdot x}{L}$$

$$n = 1: \boxed{P_E = \frac{\pi^2 \cdot EI}{L^2}} \qquad \boxed{y = A \cdot \sin \frac{\pi x}{L}}$$

Effective length: $l = L$



6.3 Buckling resistance of members

6.3.1 Uniform members in compression

6.3.1.1 Buckling resistance

(1) A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1{,}0 \tag{6.46}$$

where N_{Ed} is the design value of the compression force;

$N_{b,Rd}$ is the design buckling resistance of the compression member.

(3) The design buckling resistance of a compression member should be taken as:

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \quad \text{for Class 1, 2 and 3 cross-sections} \tag{6.47}$$

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \quad \text{for Class 4 cross-sections} \tag{6.48}$$

where χ is the reduction factor for the relevant buckling mode.

6.3.1.2 Buckling curves

(1) For axial compression in members the value of χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ should be determined from the relevant buckling curve according to:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1{,}0 \tag{6.49}$$

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

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} \quad \text{for Class 1, 2 and 3 cross-sections}$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \quad \text{for Class 4 cross-sections}$$

α is an imperfection factor

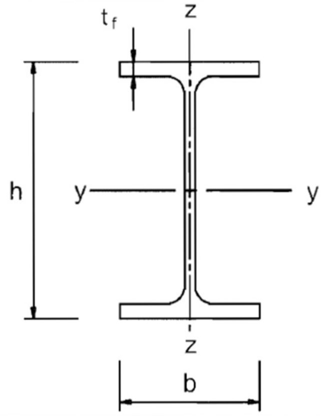
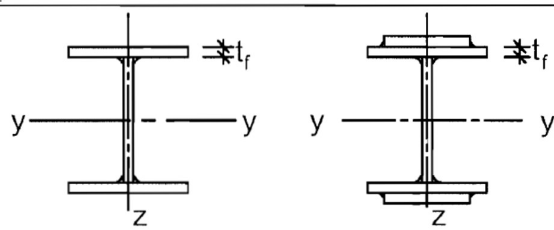
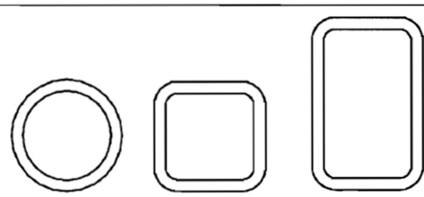
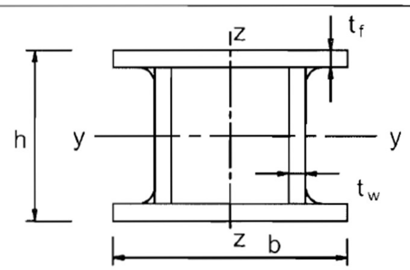
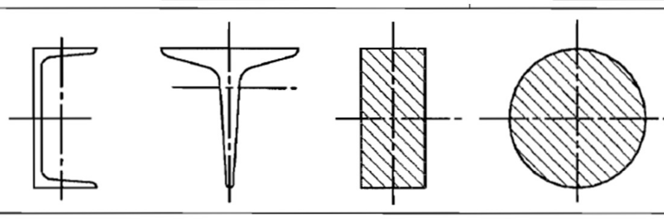
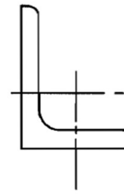
N_{cr} is the elastic critical force for the relevant buckling mode based on the gross cross sectional properties.

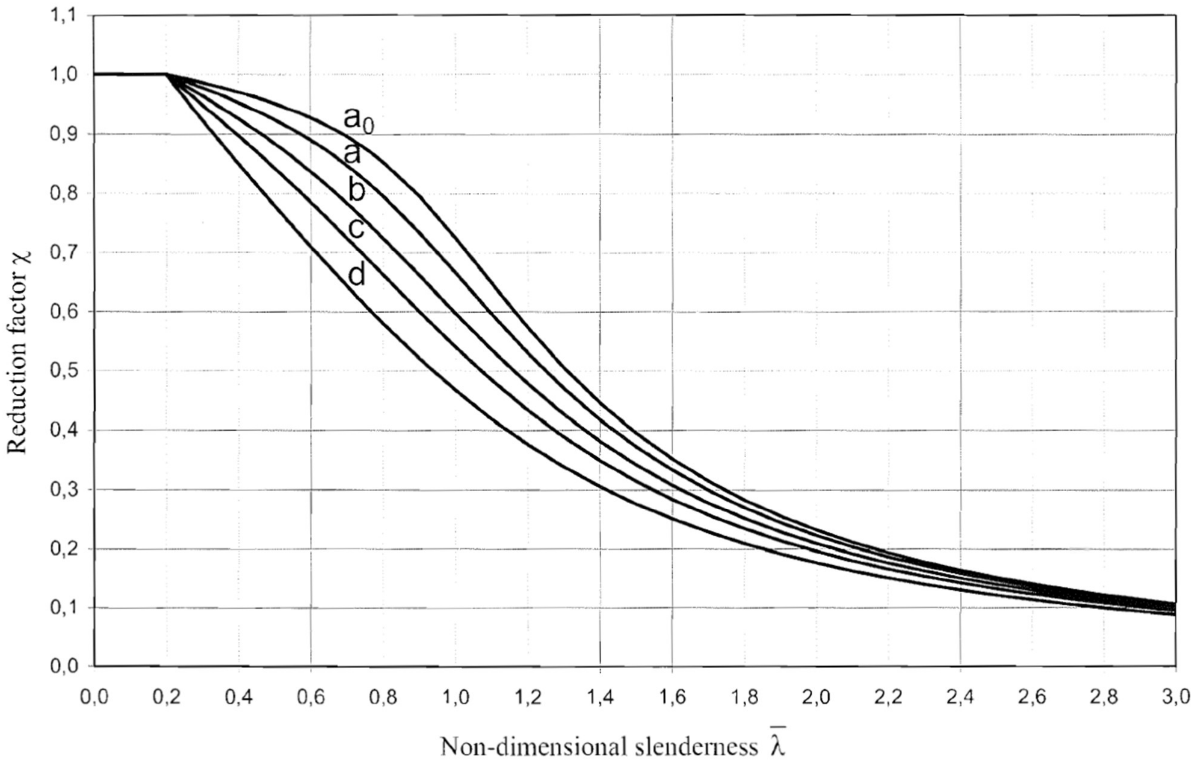
(2) The imperfection factor α corresponding to the appropriate buckling curve should be obtained from Table 6.1 and Table 6.2.

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

Table 6.2: Selection of buckling curve for a cross-section

	Cross section	Limits		Buckling about axis	Buckling curve S 235, S 275, S 355, S 420	Buckling curve S 460
Rolled sections		$h/b > 1{,}2$	$t_f \le 40$ mm	y – y z – z	a b	a_0 a_0
			$40 \text{ mm} < t_f \le 100$	y – y z – z	b c	a a
		$h/b \le 1{,}2$	$t_f \le 100$ mm	y – y z – z	b c	a a
			$t_f > 100$ mm	y – y z – z	d d	c c
Welded I-sections		$t_f \le 40$ mm		y – y z – z	b c	b c
		$t_f > 40$ mm		y – y z – z	c d	c d
Hollow sections		hot finished		any	a	a_0
		cold formed		any	c	c
Welded box sections		generally (except as below)		any	b	b
		thick welds: $a > 0{,}5t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b



(4) For slenderness $\bar{\lambda} \le 0{,}2$ or for $\dfrac{N_{Ed}}{N_{cr}} \le 0{,}04$ the buckling effects may be ignored and only cross sectional checks apply.

(1) The non-dimensional slenderness $\bar{\lambda}$ is given by:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1} \qquad \text{for Class 1, 2 and 3 cross-sections} \qquad (6.50)$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff}f_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{\sqrt{\dfrac{A_{eff}}{A}}}{\lambda_1} \qquad \text{for Class 4 cross-sections} \qquad (6.51)$$

where L_{cr} is the buckling length in the buckling plane considered

i is the radius of gyration about the relevant axis, determined using the properties of the gross cross-section

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93{,}9\varepsilon$$

$$\varepsilon = \sqrt{\frac{235}{f_y}} \qquad (f_y \text{ in N/mm}^2)$$

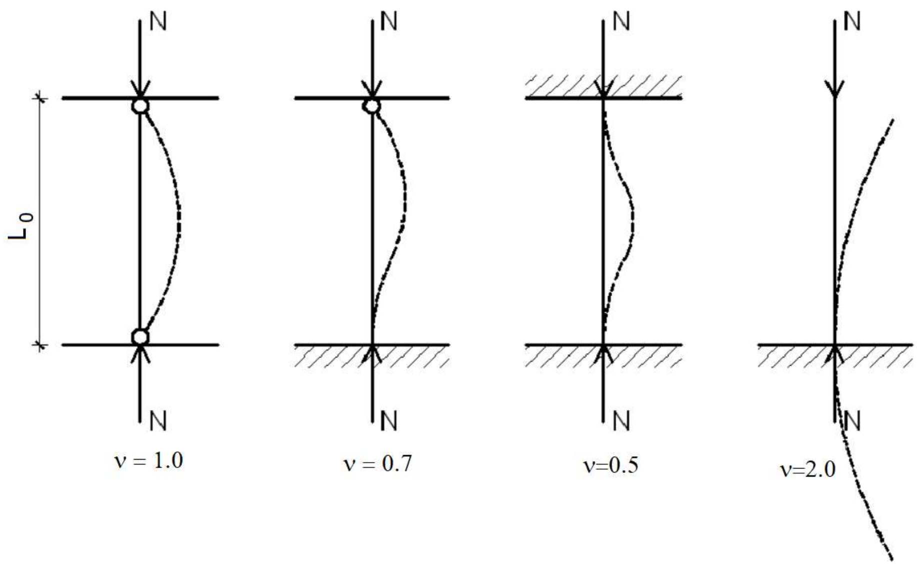
S235 $\lambda_1 = 93{,}9$
S275 $\lambda_1 = 86{,}8$
S355 $\lambda_1 = 76{,}4$
S420 $\lambda_1 = 70{,}2$
S460 $\lambda_1 = 67{,}1$

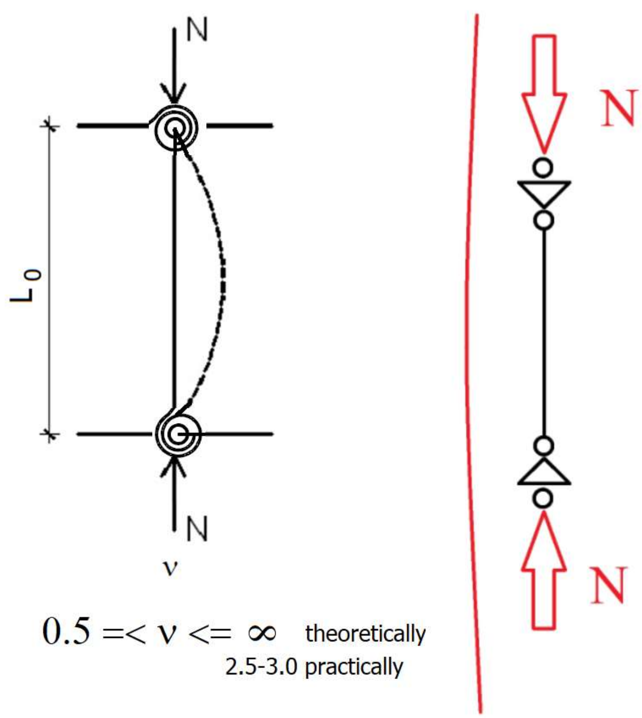
$$\bar{\lambda}_y = \frac{L_{cr}}{i\ \lambda_1} = \frac{\nu_y \times L_0}{i_y\ \lambda_1}$$

Calculation steps for flexural buckling resistance:

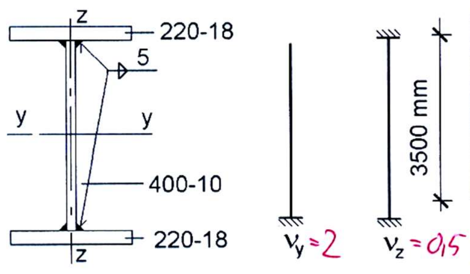
1) classification of the cross-section: class 1, 2 or 3 ←→ class 4
2) sectional properties: A, I_y, I_z, i_y, i_z
3) buckling lengths: $L_y = \nu_y \times L_0$, $L_z = \nu_z \times L_0$
4) slendernesses $\lambda_1 = 93.9 \times \varepsilon$, $\lambda_y = L_y / i_y$, $\lambda_z = L_z / i_z$,
5) non-dimensional slendernesses: $\bar{\lambda}_y$ and $\bar{\lambda}_z$
6) selecting buckling curves and calculation of χ_y and χ_z buckling reduction factors
7) $N_{b.Rd} = \chi_{min} \times A \times f_y / \gamma_{M1}$, $\gamma_{M1} = 1.0$

buckling length





2. Determine the **buckling resistance** of the welded I-section (classification)! The geometry and support conditions see on the figure. Steel grade S235. f_y = 235 N/mm², f_u = 360 N/mm², ε = 1,0



$\nu_y = 2$ $\nu_z = 0{,}5$

Kihajlási görbe kiválasztása adott keresztmetszethez:

Keresztmetszet	Korlátozások	Kihajlási tengely	Kihajlási görbe S 235, S 275, S 355, S 420	Kihajlási görbe S 460
Hegesztett I szelvények	$t_f \le 40$ mm	y – y z – z	b c	b c
	$t_f > 40$ mm	y – y z – z	c d	c d

Adott kihajlási görbéhez tartozó α imperfekciós tényezők:

Kihajlási görbe	a_0	a	b	c	d
Imperfekciós tényező	0,13	0,21	0,34	0,49	0,76

$$\phi = \frac{1 + \alpha \cdot (\bar{\lambda} - 0{,}2) + \bar{\lambda}^2}{2} \qquad \chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}}, \ \chi \le 1{,}0$$

flange: $\frac{c_f}{t_f} = \frac{\frac{220}{2} - \frac{10}{2} - 5\cdot\sqrt{2}}{18} = \boxed{5{,}44 < 9 \rightarrow Cl1}$

web: $\frac{c_w}{t_w} = \frac{400 - 2\cdot 5 \cdot \sqrt{2}}{10} = \boxed{38{,}6 < 42 \rightarrow Cl3}$

section $\boxed{CL3}$

$A = 40 \cdot 1 + 2 \cdot 22 \cdot 1{,}8 = \boxed{119{,}2 \text{ cm}^2}$

$I_y = \frac{1 \cdot 40^3}{12} + 2\left(\frac{22 \cdot 1{,}8^3}{12} + 22 \cdot 1{,}8 \cdot 20{,}9^2\right) = \boxed{39\,950 \text{ cm}^4}$

$I_z = \frac{40 \cdot 1^3}{12} + 2 \cdot \frac{1{,}8 \cdot 22^3}{12} = \boxed{3\,198 \text{ cm}^4}$

$i_y = \sqrt{\frac{39950}{119{,}2}} = \boxed{18{,}31 \text{ cm}}$; $\bar{\lambda}_y = \frac{2 \cdot 350}{18{,}31 \cdot 93{,}9} = \boxed{0{,}407}$ „b" curve $\rightarrow \alpha = 0{,}34$

$i_z = \sqrt{\frac{3198}{119{,}2}} = \boxed{5{,}18 \text{ cm}}$; $\bar{\lambda}_z = \frac{0{,}5 \cdot 350}{5{,}18 \cdot 93{,}9} = \boxed{0{,}360}$ „c" curve $\rightarrow \alpha = 0{,}49$

$\phi_y = \frac{1 + 0{,}34(0{,}407 - 0{,}2) + 0{,}407^2}{2} = \boxed{0{,}618}$; $\chi_y = \frac{1}{0{,}618 + \sqrt{0{,}618^2 - 0{,}407^2}} = \boxed{0{,}923}$

$\phi_z = \frac{1 + 0{,}49(0{,}360 - 0{,}2) + 0{,}360^2}{2} = \boxed{0{,}604}$; $\chi_z = \frac{1}{0{,}604 + \sqrt{0{,}604^2 - 0{,}360^2}} = \boxed{0{,}918}$

$N_{b.Rd} = \frac{0{,}918 \cdot 119{,}2 \cdot 23{,}5}{1{,}0} = \boxed{2571{,}5 \text{ kN}}$

2. Determine the **buckling resistance** of the welded I-section (classification)! The geometry and support conditions see on the figure.
Steel grade S275. f_y = 275 N/mm², f_u = 430 N/mm², λ_1=86,8, ε = 0,92

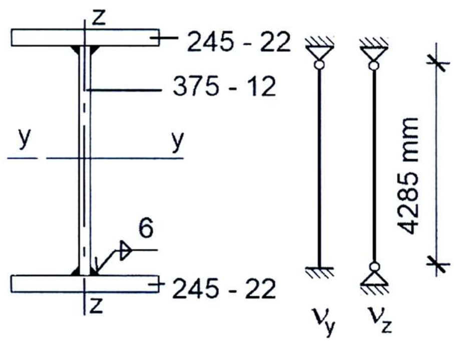


Table 6.2: Selection of buckling curve for a cross-section

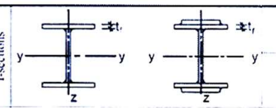
Cross section		Limits	Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	S 460
Welded I-sections		$t_f \le 40$ mm	y – y	b	b
			z – z	c	c
		$t_f > 40$ mm	y – y	c	c
			z – z	d	d

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \le 1{,}0$$

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

flange: $\frac{c_f}{t_f} = \frac{\frac{245}{2} - \frac{12}{2} - 6\sqrt{2}}{22} = \boxed{4{,}91} < 9 \cdot 0{,}92 = 8{,}28 \rightarrow Cl1$

web: $\frac{c_w}{t_w} = \frac{375 - 2 \cdot 6 \cdot \sqrt{2}}{12} = \boxed{29{,}84} < 33 \cdot 0{,}92 = 30{,}36 \rightarrow Cl1$ $\Big\}$ $\boxed{Cl1}$

$$A = 37{,}5 \cdot 1{,}2 + 2 \cdot 24{,}5 \cdot 2{,}2 = \boxed{152{,}8\ cm^2}$$

$$I_y = \frac{1{,}2 \cdot 37{,}5^3}{12} + 2\left(\frac{24{,}5 \cdot 2{,}2^3}{12} + 24{,}5 \cdot 2{,}2 \cdot 19{,}85^2\right) = \boxed{47793\ cm^4}$$

$$I_z = \frac{37{,}5 \cdot 1{,}2^3}{12} + 2 \cdot \frac{2{,}2 \cdot 24{,}5^3}{12} = \boxed{5398\ cm^4}$$

$$i_y = \sqrt{\frac{47793}{152{,}8}} = \boxed{17{,}7\ cm} \rightarrow \bar{\lambda}_y = \frac{0{,}7 \cdot 428{,}5}{17{,}7 \cdot 86{,}8} = \boxed{0{,}195} \rightarrow \text{„b" curve } \alpha = 0{,}34$$

$$i_z = \sqrt{\frac{5398}{152{,}8}} = \boxed{5{,}94\ cm} \rightarrow \bar{\lambda}_z = \frac{1{,}0 \cdot 428{,}5}{5{,}94 \cdot 86{,}8} = \boxed{0{,}831} \rightarrow \text{„c" curve } \alpha = 0{,}49$$

$$\Phi_y = 0{,}5\left(1 + 0{,}34(0{,}195 - 0{,}2) + 0{,}195^2\right) = \boxed{0{,}518}$$

$$\Phi_z = 0{,}5\left(1 + 0{,}49(0{,}831 - 0{,}2) + 0{,}831^2\right) = \boxed{1{,}000}$$

$$\chi_y = \frac{1}{0{,}518 + \sqrt{0{,}518^2 - 0{,}195^2}} = \boxed{1{,}002} \rightarrow \chi_y = 1{,}0$$

$$\chi_z = \frac{1}{1{,}000 + \sqrt{1^2 - 0{,}831^2}} = \boxed{0{,}643} \rightarrow \chi_{min} = 0{,}643$$

$$N_{b,Rd} = 0{,}643 \cdot \frac{152{,}8 \cdot 27{,}5}{1} = \boxed{2701{,}9\ kN}$$

2. Determine the **buckling resistance** of the welded I-section (classification)! The geometry and support conditions see on the figure.

Steel grade S235. $f_y = 235\ N/mm^2$, $f_u = 360\ N/mm^2$, $\lambda_1 = 93,9$, $\varepsilon = 1,00$

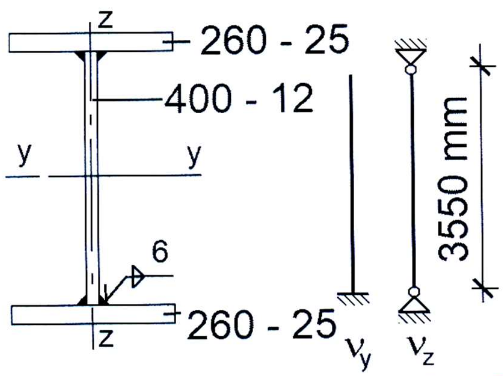


Table 6.2: Selection of buckling curve for a cross-section

	Cross section	Limits	Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	S 460
Welded I-sections		$t_f \le 40$ mm	y – y	b	b
			z – z	c	c
		$t_f > 40$ mm	y – y	c	c
			z – z	d	d

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \le 1,0$$

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

flange: $\frac{c_f}{t_f} = \frac{\frac{260}{2} - \frac{12}{2} - 6\cdot\sqrt{2}}{25} = \boxed{4,62 < 9 \text{ Cl1}}$

web: $\frac{c_w}{t_w} = \frac{400 - 2\cdot 6\cdot\sqrt{2}}{12} = \boxed{31,92 < 33 \text{ Cl1}}$ $\boxed{\text{Cl1}}$

$A = 40\cdot 1,2 + 2\cdot 26\cdot 2,5 = \boxed{178\ cm^2}$

$I_y = \frac{1,2\cdot 40^3}{12} + 2\left(\frac{26\cdot 2,5^3}{12} + 26\cdot 2,5\cdot 21,25^2\right) = \boxed{65171\ cm^4}$

$I_z = \frac{40\cdot 1,2^3}{12} + 2\cdot\frac{2,5\cdot 26^3}{12} = \boxed{7329\ cm^4}$

$i_y = \sqrt{\frac{65171}{178}} = \boxed{19,13\ cm}$; $\bar{\lambda}_y = \frac{2\cdot 355}{19,13\cdot 93,9} = \boxed{0,395} \xrightarrow{\text{„b"}} \alpha = 0,34$

$i_z = \sqrt{\frac{7329}{178}} = \boxed{6,42\ cm}$; $\bar{\lambda}_z = \frac{1\cdot 355}{6,42\cdot 93,9} = \boxed{0,589} \xrightarrow{\text{„c"}} \alpha = 0,49$

$\Phi_y = \frac{1 + 0,34(0,395 - 0,2) + 0,395^2}{2} = \boxed{0,611}$; $\chi_y = \frac{1}{0,611 + \sqrt{0,611^2 - 0,395^2}} = \boxed{0,928}$

$\Phi_z = \frac{1 + 0,49(0,589 - 0,2) + 0,589^2}{2} = \boxed{0,769}$; $\chi_z = \frac{1}{0,769 + \sqrt{0,769^2 - 0,589^2}} = \boxed{0,792}$

$N_{b.Rd} = \frac{0,792\cdot 178\cdot 23,5}{1,0} = \boxed{3312,9\ kN}$

2. Determine the **buckling resistance** of the welded I-section (classification)! The geometry and support conditions see on the figure.
Steel grade S275. f_y = 275 N/mm^2, f_u = 430 N/mm^2, λ_1=86,8, ε = 0,92

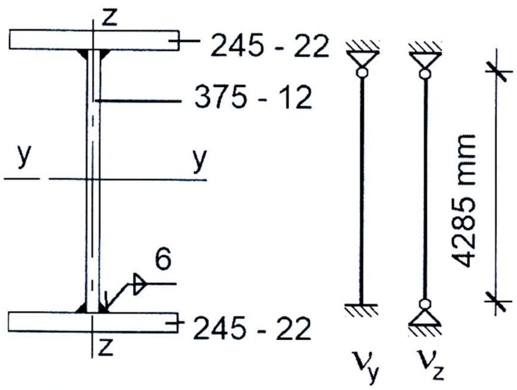


Table 6.2: Selection of buckling curve for a cross-section

Cross section	Limits	Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	S 460
Welded I-sections	$t_f \le 40$ mm	y – y z – z	b c	b c
	$t_f > 40$ mm	y – y z – z	c d	c d

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \le 1{,}0$$

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

$$c_f: \frac{\frac{245}{2} - \frac{12}{2} - 6\sqrt{2}}{22} = \boxed{4,91} < 8,28 \text{ cl 1}$$

$$c_w = \frac{375 - 2\cdot 6\sqrt{2}}{12} = \boxed{29,84} < 30,36 \text{ cl 1}$$

cl 1

$$A = 37,5\cdot 1,2 + 2\cdot 24,5\cdot 2,2 = \underline{152,8 \text{ cm}^2}$$

$$I_y = \frac{1,2\cdot 37,5^3}{12} + 2\left(\frac{24,5\cdot 2,2^3}{12} + 24,5\cdot 2,2\cdot 19,85^2\right) = \boxed{47792,5 \text{ cm}^4}$$

$$I_z = \frac{37,5\cdot 1,2^3}{12} + 2\cdot\frac{2,2\cdot 24,5^3}{12} = \boxed{5398 \text{ cm}^4}$$

$$i_y = \sqrt{\frac{47792,5}{152,8}} = \boxed{17,69 \text{ cm}} \rightarrow \bar{\lambda}_y = \frac{0,7\cdot 428,5}{17,69\cdot 86,8} = \boxed{0,195} \xrightarrow{\text{"b"}} \boxed{\chi_y = 1}$$

$$i_z = \sqrt{\frac{5398}{152,8}} = \boxed{5,94 \text{ cm}} \rightarrow \bar{\lambda}_z = \frac{1\cdot 428,5}{5,94\cdot 86,8} = \boxed{0,831} \xrightarrow{\text{"c"}}$$

$$\Phi_z = \frac{1 + 0,49(0,831 - 0,2) + 0,831^2}{2} = \boxed{0,9999}$$

$$\chi_z = \frac{1}{0,9999 + \sqrt{0,9999^2 - 0,831^2}} = \boxed{0,6427} \ (0,6433)$$

$$N_{b,Rd} = 0,6427\cdot 152,8\cdot 27,5/1,0 = \boxed{2700,6 \text{ kN}}$$

Bolted Connections

EN 1993-1-8 (2005) (English): Eurocode 3: Design of steel structures - Part 1-8: Design of joints [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]

3.4.1 Shear connections

(1) Bolted connections loaded in shear should be designed as one of the following:

a) **Category A: Bearing type**
In this category bolts from class 4.6 up to and including class 10.9 should be used. No preloading and special provisions for contact surfaces are required. The design ultimate shear load should not exceed the design shear resistance, obtained from 3.6, nor the design bearing resistance, obtained from 3.6 and 3.7.

b) **Category B: Slip-resistant at serviceability limit state**
In this category preloaded bolts in accordance with 3.1.2(1) should be used. Slip should not occur at the serviceability limit state. The design serviceability shear load should not exceed the design slip resistance, obtained from 3.9. The design ultimate shear load should not exceed the design shear resistance, obtained from 3.6, nor the design bearing resistance, obtained from 3.6 and 3.7.

c) **Category C: Slip-resistant at ultimate limit state**
In this category preloaded bolts in accordance with 3.1.2(1) should be used. Slip should not occur at the ultimate limit state. The design ultimate shear load should not exceed the design slip resistance, obtained from 3.9, nor the design bearing resistance, obtained from 3.6 and 3.7. In addition for a connection in tension, the design plastic resistance of the net cross-section at bolt holes $N_{net,Rd}$, (see 6.2 of EN 1993-1-1), should be checked, at the ultimate limit state.

3.4.2 Tension connections

(1) Bolted connection loaded in tension should be designed as one of the following:

a) **Category D: non-preloaded**
In this category bolts from class 4.6 up to and including class 10.9 should be used. No preloading is required. This category should not be used where the connections are frequently subjected to variations of tensile loading. However, they may be used in connections designed to resist normal wind loads.

b) **Category E: preloaded**
In this category preloaded 8.8 and 10.9 bolts with controlled tightening in conformity with 1.2.7 Reference Standards: Group 7 should be used.

Table 3.2: Categories of bolted connections

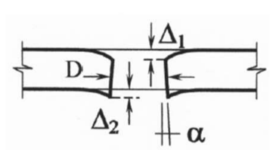
Category	Criteria	Remarks
Shear connections		
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used.
B slip-resistant at serviceability	$F_{v,Ed,ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at serviceability see 3.9.
C slip-resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ AC2 $\sum F_{v,Ed} \leq N_{net,Rd}$ AC2	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at ultimate see 3.9. $N_{net,Rd}$ see 3.4.1(1) c).
Tension connections		
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used. $B_{p,Rd}$ see Table 3.4.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. $B_{p,Rd}$ see Table 3.4.
The design tensile force $F_{t,Ed}$ should include any force due to prying action, see 3.11. Bolts subjected to both shear force and tensile force should also satisfy the criteria given in Table 3.4.		

Table 3.1: Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

Holes

- Normal
  +1 mm for M14
  +2 mm for M 16 up M 24
  +3 mm for M 27 and bigger



d – nominal bolt diameter

d_0 – the hole diameter for a bolt

M14 → bolt is a **M**etric d = 14 mm, d_0 = 15 mm

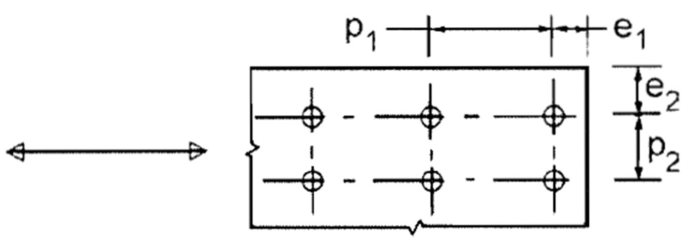
https://eurocodeapplied.com/design/en1993/bolt-design-properties

Strength properties for bolt steel according to EN 1991-1-8 Table 3.1

		Bolt class						
Symbol	**Description**	**4.6**	**4.8**	**5.6**	**5.8**	**6.8**	**8.8**	**10.9**
f_{yb} (MPa)	Yield strength	240	320	300	400	480	640	900
f_{ub} (MPa)	Ultimate tensile strength	400	400	500	500	600	800	1000

Design properties for metric hex bolts (Typical coarse pitch thread)

	Dimensions		Hole diameter d_0 [mm]				Areas	
Size	**Nominal diameter d [mm]**	**Nut width across flats s [mm]**	**Normal round hole**	**Oversize round hole**	**Short slotted hole**	**Long slotted hole**	**Gross area (unthreaded part) A_g [mm^2]**	**Stress area (threaded part) A_s [mm^2]**
M14	14	21	15	17	18×15	35.0×15	154	115
M16	16	24	18	20	22×18	40.0×18	201	157
M18	18	27	20	22	24×20	45.0×20	254	192
M20	20	30	22	24	26×22	50.0×22	314	245
M22	22	34	24	26	28×24	55.0×24	380	303
M24	24	36	26	30	32×26	60.0×26	452	353
M27	27	41	30	35	37×30	67.5×30	573	459



a) Symbols for spacing of fasteners

e_1 is the end distance from the centre of a fastener hole to the adjacent end of any part, measured in the direction of load transfer, see Figure 3.1;

e_2 is the edge distance from the centre of a fastener hole to the adjacent edge of any part, measured at right angles to the direction of load transfer, see Figure 3.1;

p_1 is the spacing between centres of fasteners in a line in the direction of load transfer, see Figure 3.1;

p_2 is the spacing measured perpendicular to the load transfer direction between adjacent lines of fasteners, see Figure 3.1;

Table 3.3: Minimum and maximum spacing, end and edge distances

Distances and spacings, see Figure 3.1	Minimum	Maximum[1) 2) 3)]		
		Structures made from steels conforming to EN 10025 except steels conforming to EN 10025-5		Structures made from steels conforming to EN 10025-5
		Steel exposed to the weather or other corrosive influences	Steel not exposed to the weather or other corrosive influences	Steel used unprotected
End distance e_1	$1{,}2d_0$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Edge distance e_2	$1{,}2d_0$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Spacing p_1	$2{,}2d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm
Spacing p_2 [5)]	$2{,}4d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm

1) Maximum values for spacings, edge and end distances are unlimited, except in the following cases:

- for compression members in order to avoid local buckling and to prevent corrosion in [AC2⟩ exposed members (the limiting values are given in the table) and; ⟨AC2]
- for exposed tension members [AC2⟩ to prevent corrosion (the limiting values are given in the table). ⟨AC2]

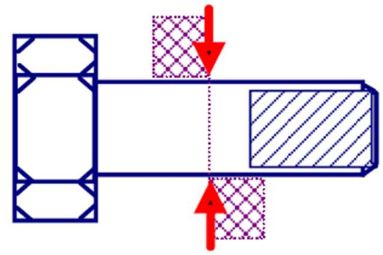
2) The local buckling resistance of the plate in compression between the fasteners should be calculated according to EN 1993-1-1 using $0{,}6\,p_1$ as buckling length. Local buckling between the fasteners need not to be checked if p_1/t is smaller than $9\,\varepsilon$. The edge distance should not exceed the local buckling requirements for an outstand element in the compression members, see EN 1993-1-1. The end distance is not affected by this requirement.

3) t is the thickness of the thinner outer connected part.

5) For staggered rows of fasteners a minimum line spacing of $p_2 = 1{,}2d_0$ may be used, provided that the minimum distance, L, between any two fasteners is greater or equal than $2{,}4d_0$, see Figure 3.1b).

https://eurocodes.jrc.ec.europa.eu/doc/2014_07_WS_Steel/presentations/06_Eurocodes_Steel_Workshop_WALD.pdf

Resistance in shear in one shear plane



$$F_{v,Rd} = \frac{\alpha_v \; A \, f_{ub}}{\gamma_{M2}}$$

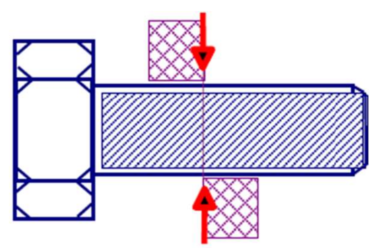
where the shear plane
passes through the **unthreaded portion of the bolt**

$\alpha_v = 0{,}6$

A	is the gross cross section of the bolt
f_{ub}	is ultimate tensile strength for bolt
γ_{M2}	s partial safety factors for resistance of bolts

$A = d^2 \times \pi /4, \quad \gamma_{M2} = 1.25$

Resistance in shear in one shear plane



$$F_{v,Rd} = \frac{\alpha_v \; A \, f_{ub}}{\gamma_{M2}}$$

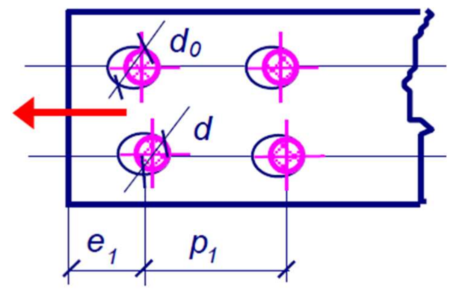
where the shear plane
passes through the **threaded portion** of the bolt

- for classes 4.6, 5.6 and 8.8:
$\boxed{\alpha_v = 0{,}6}$

- for classes 4.8, 5.8, 6.8 and 10.9
$\boxed{\alpha_v = 0{,}5}$

A	is the tensile stress area of the bolt A_s
f_{ub}	is ultimate tensile strength for bolt
γ_{M2}	s partial safety factors for resistance of bolts

Resistance in bearing

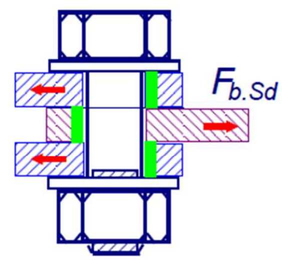


$$F_{b,Rd} = \frac{k_1\, \alpha_b\, d\, t\, f_u}{\gamma_{M2}}$$

where α_b is the smallest of α_d, $\dfrac{f_{ub}}{f_u}$ or 1,0

In the direction of load transfer

- for end bolts: $\alpha_d = \dfrac{e_1}{3\, d_0}$, for inner bolts $\alpha_d = \dfrac{p_1}{3\, d_0} - \dfrac{1}{4}$



Perpendicular to the direction of load transfer

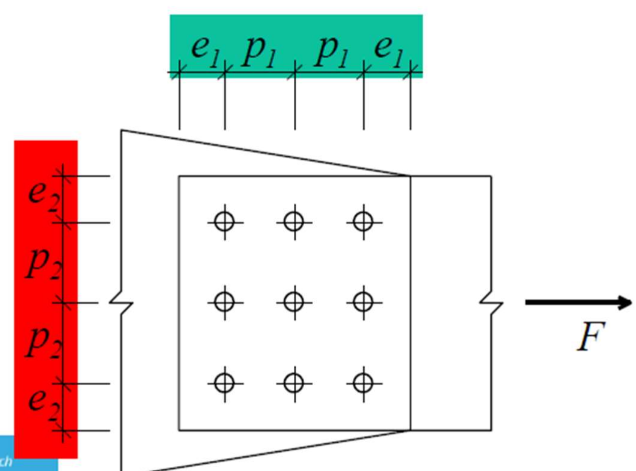
- for edge bolts k_1 is the smallest of $2{,}8\dfrac{e_2}{d_0} - 1{,}7$ or 2,5
- for inner bolts k_1 is the smallest of $1{,}4\dfrac{p_2}{d_0} - 1{,}7$ or 2,5

Influence of distances to force

$$F_{b,Rd} = \frac{k_1\, \alpha_b\, d\, t\, f_u}{\gamma_{M2}}$$

Prallel to acting force

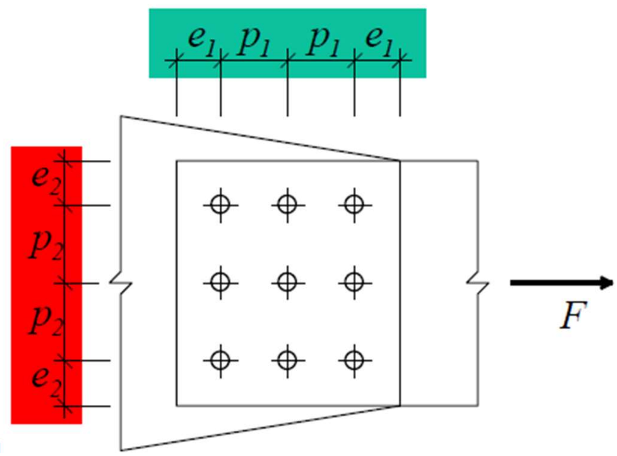
$$\alpha_b = min \begin{Bmatrix} \dfrac{e_1}{3\, d_0} \\ \dfrac{p_1}{3\, d_0} - 0{,}25 \\ \dfrac{f_{ub}}{f_u} \\ 1 \end{Bmatrix}$$



Perpendicular to acting force

$$F_{b,Rd} = \frac{k_1 \alpha_b d\, t\, f_u}{\gamma_{M2}}$$

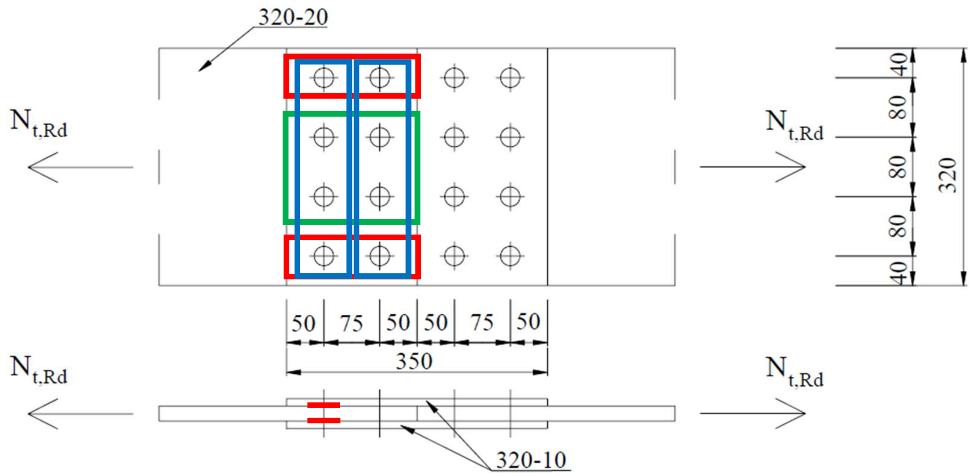
$$k_1 = \min \begin{Bmatrix} 2{,}8\,\dfrac{e_2}{d_0} - 1{,}7 \\ 1{,}4\,\dfrac{p_2}{d_0} - 1{,}7 \\ 2{,}5 \end{Bmatrix}$$



A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$

https://eurocodes.jrc.ec.europa.eu/doc/2014_07_WS_Steel/presentations/06_Eurocodes_Steel_Workshop_WALD.pdf

Sample 1: Design a full-strength double sheared splice connection



$e_1 = 50\ mm$

$p_1 = 75\ mm$

$e_2 = 40\ mm$

$p_2 = 80\ mm$

S275 $\quad f_y = 27{,}5\ kN/cm^2 \qquad f_u = 43{,}0\ kN/cm^2$

M24, 5.6 $\rightarrow d_0 = 26\ mm$

$f_{yb} = 30{,}0\ kN/cm^2 \qquad f_{ub} = 50{,}0\ kN/cm^2$

Tensile resistance of the plate: $N_{t,Rd}$

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{32 \cdot 2{,}0 \cdot 27{,}5}{1{,}0} = 1760{,}0\ kN$$

$$N_{u,Rd} = 0{,}9 \cdot \frac{A_{net} \cdot f_u}{\gamma_{M2}} = 0{,}9 \cdot \frac{(32 - 4 \cdot 2{,}6) \cdot 2{,}0 \cdot 43}{1{,}25} = 1337{,}42\ kN$$

$$N_{t,Rd} = N_{u,Rd} = 1337{,}47\ kN$$

<u>Resistance of 1 bolt:</u>

Shear resistance:

$$F_{v,Rd} = 2 \cdot \frac{\alpha_v \cdot f_{ub} \cdot A}{\gamma_{M2}} = 2 \cdot \frac{0{,}6 \cdot 50 \cdot \frac{2{,}4^2 \cdot \pi}{4}}{1{,}25} = \boxed{217{,}15\ kN}$$

Bearing resistance:

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}}$$

k_1 in case of edge bolts, perpendicularly to the load transfer direction

$$k_1 = min\begin{pmatrix} 2{,}8 \cdot \frac{e_2}{d_0} - 1{,}7 = 2{,}8 \cdot \frac{40}{26} - 1{,}7 = 2{,}61 \\ 2{,}5 \end{pmatrix} \qquad \rightarrow k_1 = 2{,}5$$

k_1 in case of inner bolts, perpendicularly to the load transfer direction

$$k_1 = min\begin{pmatrix} 1{,}4 \cdot \frac{p_2}{d_0} - 1{,}7 = 1{,}4 \cdot \frac{80}{26} - 1{,}7 = 2{,}61 \\ 2{,}5 \end{pmatrix} \qquad \rightarrow k_1 = 2{,}5$$

α_b in case of end bolts, parallel with the load transfer direction

$$\alpha_b = min\begin{pmatrix} \frac{e_1}{3 \cdot d_0} = \frac{50}{3 \cdot 26} = 0{,}64 \\ \frac{f_{ub}}{f_u} = \frac{50}{43} = 1{,}16 \\ 1 \end{pmatrix} \qquad \rightarrow \alpha_b = 0{,}64$$

α_b in case of inner bolts, parallel with the load transfer direction → don't have such a bolt!

$$\alpha_b = min\begin{pmatrix} \frac{p_1}{3 \cdot d_0} - \frac{1}{4} \\ \frac{f_{ub}}{f_u} \\ 1 \end{pmatrix}$$

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \frac{2{,}5 \cdot 0{,}64 \cdot 43 \cdot 2{,}4 \cdot 2{,}0}{1{,}25} = 264{,}19\ kN$$

$$n_{sz} = \frac{N_{t,Rd}}{F_{v,Rd}} = \frac{1337{,}47}{217{,}15} = 6{,}16$$

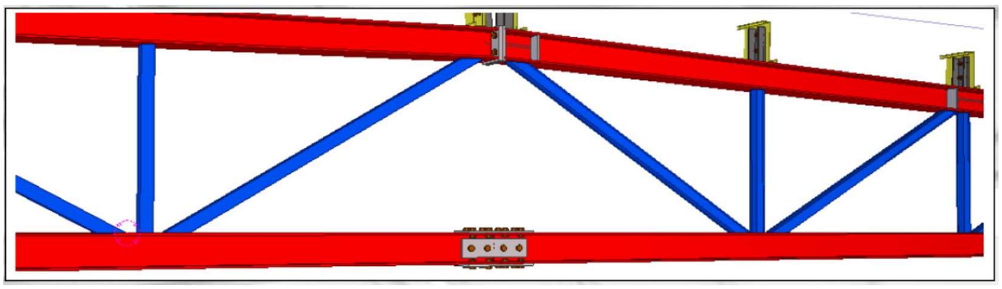
$$n_{alk} = 8\ db \qquad \rightarrow \qquad 2\text{x}4\ \text{db}$$

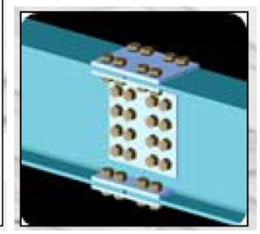
Tensile resistance of joint plates

$A_{net}^{plate} <= A_{net}^{joint\ plates}$!!

$A_{net}^{plate} = (32 – 4 × 2.6) × 2.0 <= A_{net}^{joint\ plates} = 2 × (32 – 4 × 2.6) × 1.0$ → satisfies!

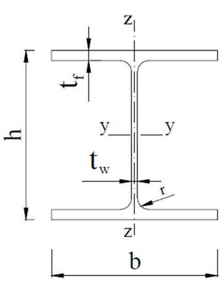
Sample 2: Design a full-strength double sheared splice connection



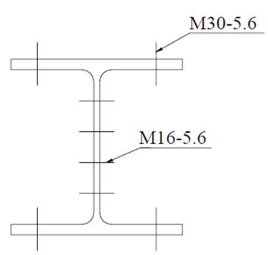


Desing the full-strength double sheared splice connection of HE 400 B tensioned bar!

S235 $\quad f_y = 23{,}5\ kN/cm^2 \quad f_u = 36{,}0\ kN/cm^2$



$b = 300\ mm$
$t_f = 24{,}0\ mm$
$h = 400\ mm$
$t_w = 13{,}5\ mm$
$A = 198\ cm^2$



M30, 5.6 → $d_0 = 33\ mm$

M16, 5.6 → $d_0 = 18\ mm$

$f_{yb} = 30{,}0\ kN/cm^2 \quad f_{ub} = 50{,}0\ kN/cm^2$

$$F_{Ed} = F_{Ek,g} \cdot \gamma_g + F_{Ek,q} \cdot \gamma_q = 900 \cdot 1{,}35 + 1600 \cdot 1{,}5 = 3615\ kN$$

$A_f = 30 × 2.4 = 72\ cm^2$

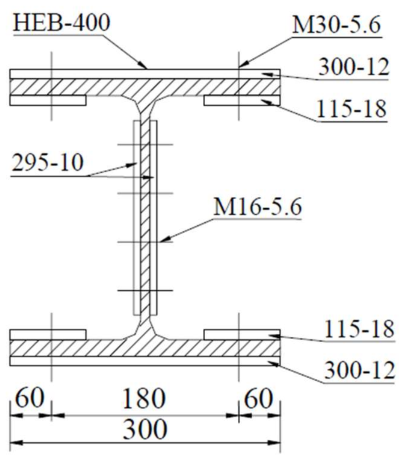
$A_w = A – 2 × A_f = 198 – 2 × 72 = 54\ cm^2$

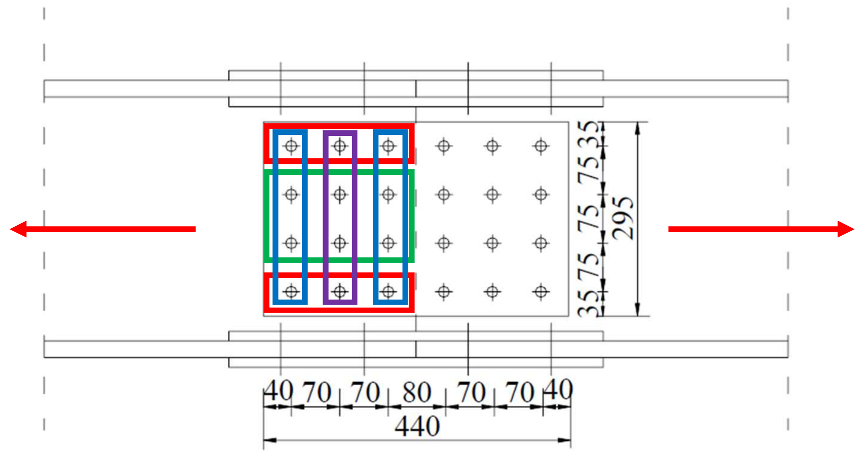
$F_{Ed,f} = A_f / A × F_{Ed} = 72 / 198 × 3615 =$ **1314,5 kN**

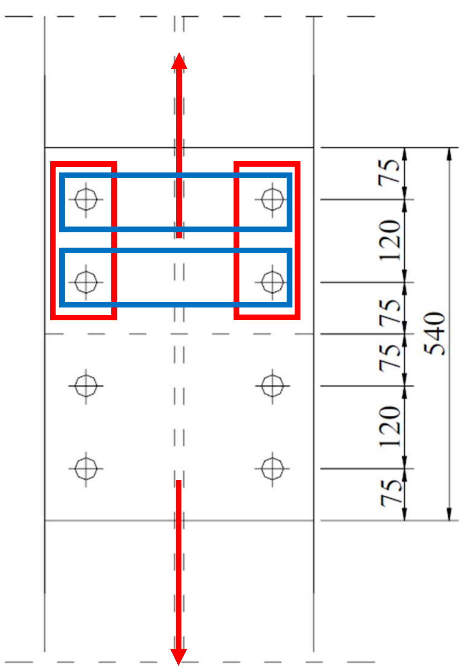
$F_{Ed,w} = A_w / A × F_{Ed} = 54 / 198 × 3615 =$ **985,9 kN**

Tensile resistance of HE 400 B bar

$$N_{t,Rd} = min\begin{pmatrix} N_{pl,Rd} = \dfrac{A \cdot f_y}{\gamma_{M0}} \\ N_{u,Rd} = 0,9 \cdot \dfrac{A_{net} \cdot f_u}{\gamma_{M2}} \end{pmatrix} \quad \begin{matrix} N_{pl,Rd} = \dfrac{A \cdot f_y}{\gamma_{M0}} = \dfrac{198 \cdot 23,5}{1,0} = 4653\ kN \\ N_{u,Rd} = 0,9 \cdot \dfrac{A_{net} \cdot f_u}{\gamma_{M2}} = 0,9 \cdot \dfrac{(198 - 4 \cdot 3,3 \cdot 2,4 - 4 \cdot 1,8 \cdot 1,35) \cdot 36}{1,25} = \boxed{4059,07\ kN} \end{matrix}$$







$e_{1.w}$ = 40 mm, $p_{1.w}$ = 70 mm, $e_{2.w}$ = 35 mm, $p_{2.w}$ = 75 mm

$e_{1.min.w} = e_{2.min.w} = 1.2 \times d_0 = 1.2 \times 18 = 21.6$ mm

$p_{1.min.w} = 2.2 \times d_0 = 2.2 \times 18 = 39.6$ mm

$p_{2.min.w} = 2.4 \times d_0 = 2.4 \times 18 = 43.2$ mm

$e_{1.f}$ = 75 mm, $p_{1.f}$ = 120 mm, $e_{2.f}$ = 55 mm, ($p_{2.f}$ = 0 mm)

$e_{1.min.f} = e_{2.min.f} = 1.2 \times d_0 = 1.2 \times 33 = 39.6$ mm

$p_{1.min.f} = 2.2 \times d_0 = 2.2 \times 33 = 72.6$ mm

($p_{2.min.f} = 2.4 \times d_0 = 2.4 \times 33 = 79.2$ mm)

Bolted connection of the flanges:

Resistance of 1 piece of M30 bolt:

Shear resistance:

$$F_{v,Rd}^{30} = 2 \cdot \frac{\alpha_v \cdot f_{ub} \cdot A_b}{\gamma_{M2}} = 2 \cdot \frac{0{,}6 \cdot 50 \cdot \frac{3^2 \cdot \pi}{4}}{1{,}25} = \boxed{339{,}3\ kN}$$

Bearing resistance:

$e_{1.f}$ = 75 mm, $p_{1.f}$ = 120 mm, $e_{2.f}$ = 55 mm

k_1 in case of edge bolts, perpendicularly to the load transfer direction

$$k_1 = min\begin{pmatrix} 2{,}8 \cdot \frac{e_2}{d_0} - 1{,}7 = 2{,}8 \cdot \frac{55}{33} - 1{,}7 = 2{,}96 \\ 2{,}5 \end{pmatrix} \qquad \rightarrow k_1 = 2{,}5$$

α_b in case of end bolts, parallel with the load transfer direction

$$\alpha_b = min\begin{pmatrix} \frac{e_1}{3 \cdot d_0} = \frac{75}{3 \cdot 33} = 0{,}76 \\ \frac{f_{ub}}{f_u} = \frac{50}{36} = 1{,}38 \\ 1 \end{pmatrix} \qquad \rightarrow \alpha_b = 0{,}76$$

$$F_{b,Rd}^{30} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \frac{2{,}5 \cdot 0{,}76 \cdot 36 \cdot 3{,}0 \cdot 2{,}4}{1{,}25} = 393{,}98\ kN$$

$F_{Ed,f} = A_f / A \times F_{Ed}$ = 72 / 198 × 3615 = **1314,5 kN**

$$n_{sz,öv} = \frac{F_{Ed,f}}{F_{v,Rd}^{30}} = \frac{1314{,}5}{339{,}3} = 3{,}9\ db \qquad n_{alk} = 4\ db \quad \rightarrow \quad \text{2x2 db}$$

db (darab) = peace

Tensile resistance of joint plates

$A_{net.f} <= A_{net.f}^{joint\ plates}$!!

$A_{net.f} = (30 - 2 \times 3.3) \times 2.4 <= A_{net.f}^{joint\ plates} = (30 - 2 \times 3.3) \times 1.2 + 2 \times (11.5 - 3.3) \times 1.8$

$A_{net.f} = 56.16\ cm^2 < A_{net.f}^{joint\ plates} = 57.6\ cm^2$ → satisfies!

Bolted connection of the web plate:

Resistance of 1 piece of M16 bolt:

Shear resistance:

$$F_{v,Rd}^{16} = 2 \cdot \frac{\alpha_v \cdot f_{ub} \cdot A_b}{\gamma_{M2}} = 2 \cdot \frac{0{,}6 \cdot 50 \cdot \frac{1{,}6^2 \cdot \pi}{4}}{1{,}25} = \boxed{96{,}5\ kN}$$

Bearing resistance:

$e_{1.w}$ = 40 mm, $p_{1.w}$ = 70 mm, $e_{2.w}$ = 35 mm, $p_{2.w}$ = 75 mm

k_1 in case of edge bolts, perpendicularly to the load transfer direction

$$k_1 = min\begin{pmatrix} 2{,}8 \cdot \frac{e_2}{d_0} - 1{,}7 = 2{,}8 \cdot \frac{35}{18} - 1{,}7 = 3{,}74 \\ 2{,}5 \end{pmatrix} \qquad \rightarrow k_1 = 2{,}5$$

k_1 in case of inner bolts, perpendicularly to the load transfer direction

$$k_1 = min\begin{pmatrix} 1{,}4 \cdot \frac{p_2}{d_0} - 1{,}7 = 1{,}4 \cdot \frac{75}{18} - 1{,}7 = 4{,}13 \\ 2{,}5 \end{pmatrix} \qquad \rightarrow k_1 = 2{,}5$$

α_b in case of end bolts, parallel with the load transfer direction

$$\alpha_b = min\begin{pmatrix} \frac{e_1}{3 \cdot d_0} = \frac{40}{3 \cdot 18} = 0{,}74 \\ \frac{f_{ub}}{f_u} = \frac{50}{36} = 1{,}38 \\ 1 \end{pmatrix} \qquad \rightarrow \alpha_b = 0{,}74$$

α_b in case of inner bolts, parallel with the load transfer direction

$$\alpha_b = min\begin{pmatrix} \frac{p_1}{3\cdot d_0} - \frac{1}{4} = \frac{70}{3\cdot 18} - \frac{1}{4} = 1{,}05 \\ \frac{f_{ub}}{f_u} = \frac{50}{36} = 1{,}38 \\ 1 \\ \end{pmatrix} \rightarrow \alpha_b = 1{,}0$$

$$F_{b,Rd}^{16} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \frac{2{,}5 \cdot 0{,}74 \cdot 36 \cdot 1{,}6 \cdot 1{,}35}{1{,}25} = 115{,}08\ kN$$

$F_{Ed,w} = A_w / A \times F_{Ed}$ = 54 / 198 × 3615 = **985,91 kN**

$$n_{sz,g} = \frac{F_{Ed,w}}{F_{v.Rd}^{16}} = \frac{985{,}91}{96{,}5} = 10{,}2\ db \qquad n_{alk} = 12\ db \rightarrow \quad 3x4\ \text{db}$$

db (darab) = peace

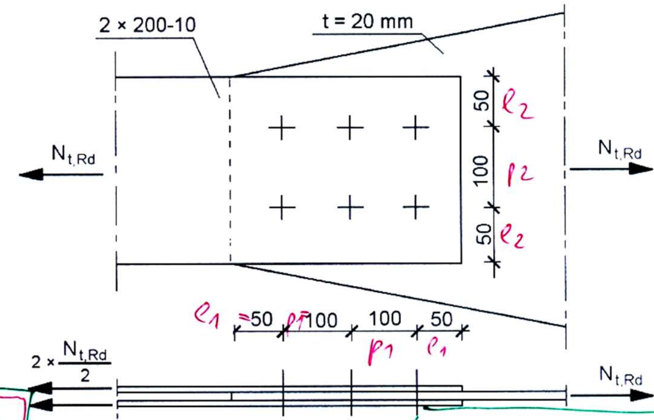
Tensile resistance of joint plates

$A_{net.w} <= A_{net.w}^{joint\ plates}$!!

$A_{net.w}$ = 54 – 4 × 1.8 × 1.35 = 44.28 cm^2 **<=** $A_{net.w}^{joint\ plates}$ = 2 × (29,5 – 4 × 1.8) × 1.0 = 44.6 cm^2

→ satisfies!

3. Determine the **tension resistance** of the **2 × 200-10** bar ($N_{t,Rd}$)! Design the double sheared bolted connection to $N_{t,Rd}$ ($n_{necessary}$ =?, $n_{applied}$ =?)! Bolts: M24 8.8, steel grade S275. f_y = 275 N/mm², f_u = 430 N/mm²



$$N_{pl,Rd} = \frac{2 \cdot 20 \cdot 1 \cdot 27{,}5}{1} = \boxed{1100\ kN}$$

$$N_{u,Rd} = 0{,}9 \frac{(20 - 2 \cdot 2{,}6) \cdot 2 \cdot 43}{1{,}25} = \boxed{916{,}4\ kN}$$

$$\boxed{\begin{array}{l} e_1^{min} = e_2^{min} = 1{,}2 \cdot 26 = 31{,}2\ mm < 50\ mm \\ p_1^{min} = 2{,}2 \cdot 26 = 57{,}2\ mm < 100\ mm \\ p_2^{min} = 2{,}4 \cdot 26 = 62{,}4\ mm < 100\ mm \end{array}}$$

$$F_{v,Rd} = 2 \cdot \frac{0{,}6 \cdot 80 \cdot \frac{2{,}4^2 \pi}{4}}{1{,}25} = \boxed{347{,}4\ kN}$$

$$k_1 = \min\left\{2{,}8 \cdot \frac{50}{26} - 1{,}7 = \boxed{3{,}68};\ 1{,}4 \frac{100}{26} - 1{,}7 = \boxed{3{,}68};\ 2{,}5\right\} = \boxed{2{,}5}$$

$$\alpha_b = \min\left\{\frac{50}{3 \cdot 26} = \boxed{0{,}64};\ \frac{100}{3 \cdot 26} - \frac{1}{4} = \boxed{1{,}03};\ \frac{80}{43} = \boxed{1{,}86};\ 1\right\} = \begin{cases} 0{,}64 & \text{end bolts} \\ \boxed{1} & \text{inner bolts} \end{cases}$$

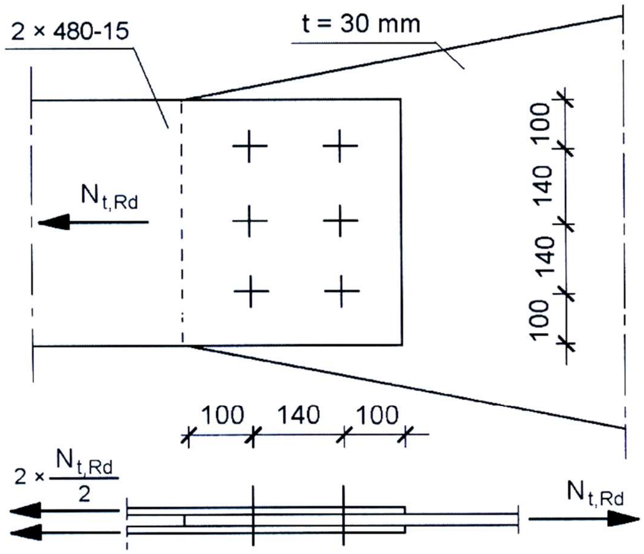
$$F_{b,Rd}^{end} = \frac{2{,}5 \cdot 0{,}64 \cdot 43 \cdot 2{,}4 \cdot 2}{1{,}25} = \boxed{264{,}2\ kN}$$

$$F_{b,Rd}^{inner} = \frac{2{,}5 \cdot 1{,}0 \cdot 43 \cdot 2{,}4 \cdot 2}{1{,}25} = \boxed{412{,}8\ kN}$$

$$N_{bolt} = 264{,}2\ kN$$

$$n_{nec} = \frac{916{,}4}{264{,}2} = \boxed{3{,}47} \rightarrow n_{all} = \boxed{2 \times 2 = 4}$$

3. Determine the **tension resistance** of the **2 × 480-15** bar ($N_{t,Rd}$)! Design the double sheared bolted connection to $N_{t,Rd}$ (**$n_{necessary}$ =?, $n_{applied}$ =?**)! Bolts: M20 10.9, steel grade S275. f_y = 275 N/mm^2, f_u = 430 N/mm^2.



$$e_1^{min} = e_2^{min} = 1,2 \cdot 22 = 26,4 \text{ mm} < 100 \text{ mm} \checkmark$$

$$p_1^{min} = 2,2 \cdot 22 = 48,4 \text{ mm} < 140 \text{ mm} \checkmark$$

$$p_2^{min} = 2,4 \cdot 22 = 52,8 \text{ mm} < 140 \text{ mm} \checkmark$$

$$N_{pl,Rd} = \frac{2 \cdot 48 \cdot 1,5 \cdot 27,5}{1} = \boxed{3960 \text{ kN}}$$

$$N_{u,Rd} = 0,9 \frac{(48 - 3 \cdot 2,2) \cdot 2 \cdot 1,5 \cdot 43}{1,25} = \boxed{3845 \text{ kN}}$$

$$F_{v,Rd} = 2 \cdot \frac{0,6 \cdot 100 \cdot \frac{2^2 \pi}{4}}{1,25} = \boxed{301,6 \text{ kN}}$$

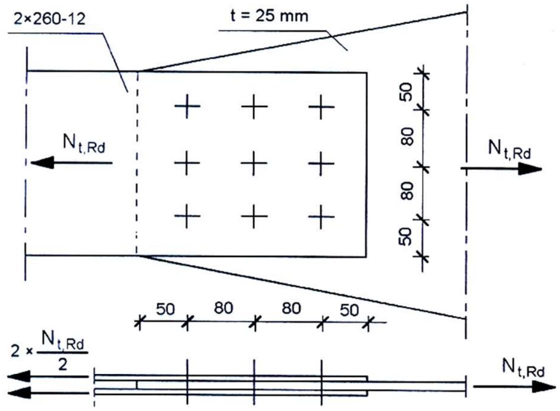
$$\alpha_b = \min\left\{ \frac{100}{3 \cdot 22} = \boxed{1,52};\ \frac{100}{43} = \boxed{2,33};\ 1 \right\} = \boxed{1}$$

$$k_1 = \min\left\{ 2,8 \cdot \frac{100}{22} - 1,7 = \boxed{11,03},\ 1,4 \cdot \frac{140}{22} - 1,7 = \boxed{7,21},\ 2,5 \right\} = \boxed{2,5}$$

$$F_{b,Rd} = \frac{2,5 \cdot 1 \cdot 43 \cdot 2 \cdot 3}{1,25} = \boxed{516 \text{ kN}}$$

$$n_{necess} = \frac{3845}{301,6} = \boxed{12,7} \rightarrow n_{appl} = 3 \times 5 = \boxed{15}$$

3. Determine the **tension resistance** of the **2 × 260-12** bar ($N_{t,Rd}$)! Design the double sheared bolted connection to $N_{t,Rd}$ (**$n_{necessary}$ =?, $n_{applied}$ =?**)! Bolts: M20 10.9, steel grade S355, f_y = 355 N/mm², f_u = 510 N/mm².



csavar	átmérő d, mm	furatátmérő d_0, mm	keresztmetszeti terület A, mm²	húzási feszültség-keresztmetszet A_s, mm²	átmérő a kigombolódás számításához d_m, mm
M12	12	13	113	84,3	20,5
M14	14	15	154	115	23,7
M16	16	18	201	157	24,6
M18	18	20	254	192	29,1
M20	20	22	314	245	32,4
M22	22	24	380	303	34,5

$e_1^{min} = e_2^{min} = 1{,}2 \cdot 22 = 26{,}4\,mm < 50\,mm$ ✓

$p_1^{min} = 2{,}2 \cdot 22 = 48{,}4\,mm < 80\,mm$ ✓

$p_2^{min} = 2{,}4 \cdot 22 = 52{,}8\,mm < 80\,mm$ ✓

$$N_{pl,Rd} = \frac{2 \cdot 26 \cdot 1{,}2 \cdot 35{,}5}{1{,}0} = \boxed{2215{,}2\ kN}$$

$$N_{u,Rd} = 0{,}9\,\frac{(26 - 3 \cdot 2{,}2) \cdot 2 \cdot 1{,}2 \cdot 51}{1{,}25} = \boxed{1709{,}7\ kN}$$

$$F_{v,Rd} = 2 \cdot \frac{0{,}6 \cdot 100 \cdot \frac{2^2 \pi}{4}}{1{,}25} = \boxed{301{,}6\ kN}$$

$$k_1 = \min\left\{2{,}8 \cdot \frac{50}{22} - 1{,}7 = \boxed{4{,}66};\ 1{,}4 \cdot \frac{80}{22} - 1{,}7 = \boxed{3{,}39};\ 2{,}5\right\} = \boxed{2{,}5}$$

$$\alpha_b = \min\left\{\frac{50}{3 \cdot 22} = \boxed{0{,}76};\ \frac{80}{3 \cdot 22} - \frac{1}{4} = \boxed{0{,}96};\ \frac{100}{51} = \boxed{1{,}96};\ 1\right\} = \begin{cases} 0{,}76 & \text{end bolts} \\ 0{,}96 & \text{inner bolts} \end{cases}$$

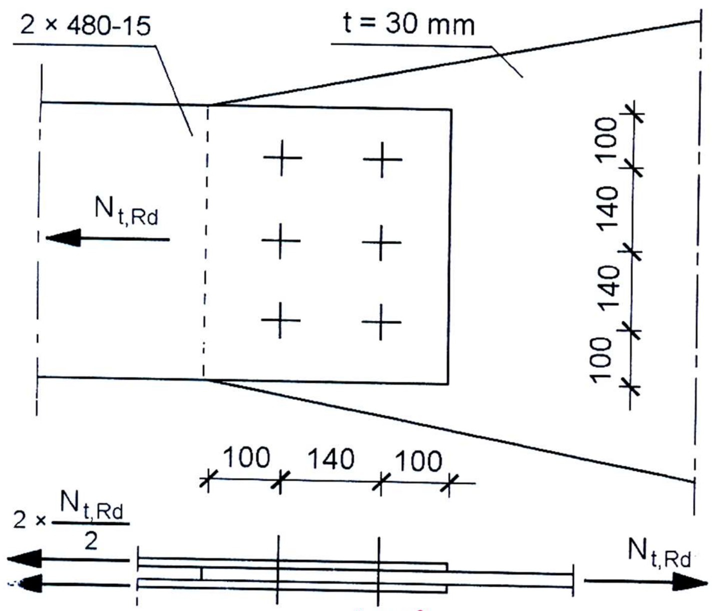
$$F_{b,Rd}^{End} = \frac{2{,}5 \cdot 0{,}76 \cdot 51 \cdot 2 \cdot 1{,}2 \cdot 2}{1{,}25} = \boxed{372{,}1\ kN}$$

$$F_{b,Rd}^{inner} = \frac{2{,}5 \cdot 0{,}96 \cdot 51 \cdot 2 \cdot 2 \cdot 1{,}2}{1{,}25} = \boxed{470{,}0\ kN}$$

$$N_{bolt} = 301{,}6\ kN$$

$$n_{nec} = \frac{1709{,}7}{301{,}6} = \boxed{5{,}67} \rightarrow n_{appl.} = \boxed{3 \times 2 = 6}$$

3. Determine the **tension resistance** of the **2 × 480-15** bar ($N_{t,Rd}$)! Design the double sheared bolted connection to $N_{t,Rd}$ (**$n_{necessary}$ =?, $n_{applied}$ =?**)! Bolts: M24 10.9, steel grade S275. f_y = 275 N/mm², f_u = 430 N/mm².



$$N_{pl,Rd} = \frac{2 \cdot 48 \cdot 1{,}5 \cdot 27{,}5}{1} = \boxed{3960\ kN}$$

$$N_{u,Rd} = 0{,}9 \cdot \frac{(48 - 3 \cdot 2{,}6) \cdot 2 \cdot 1{,}5 \cdot 43}{1{,}25} = \boxed{3734\ kN}$$

$$F_{v,Rd} = 2 \cdot \frac{0{,}6 \cdot 100 \cdot \overbrace{\frac{2{,}4^2\pi}{4}}^{4{,}52}}{1{,}25} = \boxed{434{,}3\ kN}$$

$$\alpha_b = \min\left\{ \frac{100}{3 \cdot 26} = \boxed{1{,}28};\ \frac{140}{3 \cdot 26} - \frac{1}{4} = 1{,}54;\ \frac{100}{43} = \boxed{2{,}33};\ 1{,}0 \right\} = \boxed{1{,}0}$$

$$k_1 = \min\left\{ 2{,}8 \cdot \frac{100}{26} - 1{,}7 = \boxed{9{,}07};\ 1{,}4 \cdot \frac{140}{26} - 1{,}7 = \boxed{5{,}84};\ 2{,}5 \right\} = \boxed{2{,}5}$$

$$F_{b,Rd} = \frac{2{,}5 \cdot 1 \cdot 43 \cdot 2{,}4 \cdot 3}{1{,}25} = \boxed{619{,}2\ kN}$$

$$n_{nec} = \frac{3734}{434{,}3} = \boxed{8{,}6} \rightarrow n_{appl} = \boxed{9 = 3 \times 3}$$

Non-preloaded tensioned connection

3.4.2 Tension connections

(1) Bolted connection loaded in tension should be designed as one of the following:

a) **Category D: non-preloaded**
In this category bolts from class 4.6 up to and including class 10.9 should be used. No preloading is required. This category should not be used where the connections are frequently subjected to variations of tensile loading. However, they may be used in connections designed to resist normal wind loads.

b) **Category E: preloaded**
In this category preloaded 8.8 and 10.9 bolts with controlled tightening in conformity with 1.2.7 Reference Standards: Group 7 should be used.

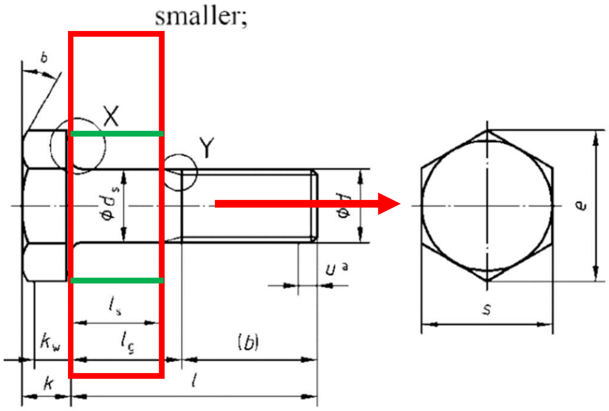
Table 3.2: Categories of bolted connections

Category	Criteria	Remarks
	Shear connections	
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used.
B slip-resistant at serviceability	$F_{v,Ed,ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at serviceability see 3.9.
C slip-resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ AC2 $\sum F_{v,Ed} \leq N_{net,Rd}$ AC2	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at ultimate see 3.9. $N_{net,Rd}$ see 3.4.1(1) c).
	Tension connections	
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used. $B_{p,Rd}$ see Table 3.4.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. $B_{p,Rd}$ see Table 3.4.
The design tensile force $F_{t,Ed}$ should include any force due to prying action, see 3.11. Bolts subjected to both shear force and tensile force should also satisfy the criteria given in Table 3.4.		

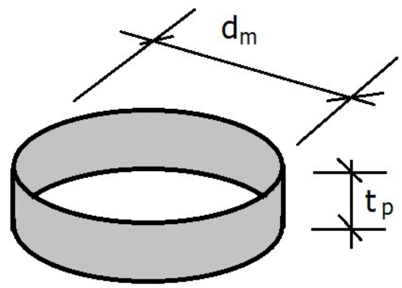
Table 3.4: Design resistance for individual fasteners subjected to shear and/or tension

Failure mode	Bolts
Tension resistance [2)]	$F_{t,Rd} = \dfrac{k_2\, f_{ub}\, A_s}{\gamma_{M2}}$ where $k_2 = 0{,}63$ for countersunk bolt, otherwise $\boxed{k_2 = 0{,}9.}$
Punching shear resistance	$B_{p,Rd} = 0{,}6\,\boxed{\pi\, d_m\, t_p}\, f_u / \gamma_{M2}$

d_m is the mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller;



$$d_m = (e + s) / 2$$



Punching strength of bolts

The punching resistance of the bolt $B_{p,Rd}$ should be verified against the applied tensile load $F_{t,Ed}$ in accordance with *EN1993-1-8 Table 3.4*:

$B_{p,Rd} = 0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u / \gamma_{M2}$

where:

- d_m is the mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller.
- t_p is the plate thickness under the bolt or nut.
- f_u is the ultimate tensile strength of the steel plate.
- γ_{M2} is the partial safety factor for the resistance of bolts in accordance with *EN1993-1-8 §2.2(2) Table 2.1* and the National Annex. The recommended value in EN1993-1-8 is $\gamma_{M2} = 1.25$.

The value of the mean diameter d_m is estimated as follows. The distance across flats s of the nut is given in the standard ISO 898-2. By approximately ignoring the corner rounding for a perfect hexagon the relation of the distance across points s' and the distance across flats s is $s' = s / \cos(30°) = 1.1547 \cdot s$. Therefore the mean diameter d_m is approximately:

$d_m = (s + 1.1547 \cdot s) / 2 = 1.07735 \cdot s$

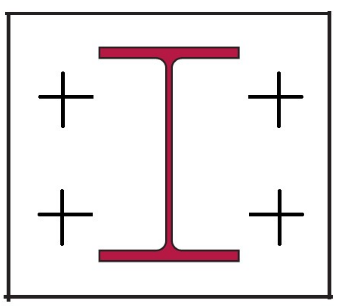
Size	Dimensions: Nominal diameter d [mm]	Dimensions: Nut width across flats s [mm]
M14	14	21
M16	16	24
M18	18	27
M20	20	30
M22	22	34
M24	24	36
M27	27	41
M30	30	46

Design properties for metric hex bolts (Typical coarse pitch thread)

	Dimensions		Hole diameter d_0 [mm]				Areas	
Size	Nominal diameter d [mm]	Nut width across flats s [mm]	Normal round hole	Oversize round hole	Short slotted hole	Long slotted hole	Gross area (unthreaded part) A_g [mm²]	Stress area (threaded part) A_s [mm²]
M14	14	21	15	17	18×15	35.0×15	154	115
M16	16	24	18	20	22×18	40.0×18	201	157
M18	18	27	20	22	24×20	45.0×20	254	192
M20	20	30	22	24	26×22	50.0×22	314	245
M22	22	34	24	26	28×24	55.0×24	380	303
M24	24	36	26	30	32×26	60.0×26	452	353
M27	27	41	30	35	37×30	67.5×30	573	459

Sample – tensioned bolted connection → full-strength connection

Denominación Designation Designazione		Dimensiones Dimensions Dimensioni						Dimensiones de construcción Dimensions for detailing Dimensioni di costruzione					Superficie Surface Superficie	
	G kg/m	h mm	b mm	t_w mm	t_f mm	r mm	A mm^2	h_i mm	d mm	Ø	p_{min} mm	p_{max} mm	A_L m^2/m	A_G m^2/t
							x 10^2							
HE 200 B	61,3	200	200	9	15	18	78,1	170	134	M 27	100	100	1,151	18,78



HE 200 B bar, S235 → f_y = 23.5 kN/cm^2, f_u = 36 kN/cm^2

A_{HE200B} = 78.1 cm^2 → $N_{pl.Rd}$ = 78.1 × 23.5 / 1.0 = **<u>1 835.35</u>** kN

4 × M16 5.6 → d = 16 mm, d_0 = 18 mm, f_{ub} = 50 kN/cm^2

A_s = 1,57 cm^2 , d_m = 1.07735 × s = 1.07735 × 2,4 = 2,59 cm

t_p = 15 mm

Tension resistance of 1 bolt:

tension resistance:

$$F_{t,Rd} = \frac{k_2 \; f_{ub} \; A_s}{\gamma_{M2}}$$ = 0.9 × 50 × 1,57 / 1.25 = <u>56.52</u> kN

punching shear resistance:

$$B_{p,Rd} = 0{,}6\,\pi\, d_m\, t_p f_u / \gamma_{M2}$$

$B_{p.Rd}$ = 0.6 × π × 2.59 × 1.5 × 36 / 1.25 = <u>210.9</u> kN

Tension resistance of the connection:

$F_{t.Rd.conn}$ = 4 × 56.52 = <u>226.08</u> kN < $N_{pl.Rd}$ = 78.1 × 23.5 / 1.0 = **<u>1 835.35</u>** kN → will fail!

4 × M22 8.8 → d = 22 mm, d_0 = 24 mm, f_{ub} = 80 kN/cm^2

A_s = 3,03 cm^2 , d_m = 1.07735 × s = 1.07735 × 3,4 = 3,43 cm
t_p = 15 mm

$$F_{t,Rd} = \frac{k_2 \; f_{ub} \; A_s}{\gamma_{M2}}$$ = 0.9 × 80 × 3,43 / 1.25 = <u>197.6</u> kN

$$B_{p,Rd} \quad = \quad 0{,}6 \, \pi \, d_m \, t_p \, f_u \, / \, \gamma_{M2}$$

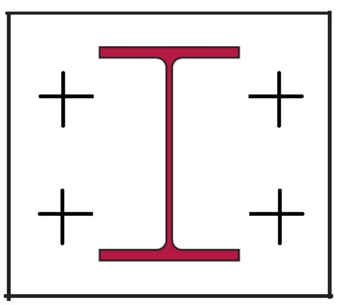
$B_{p.Rd}$ = 0.6 × π × 3,43 × 1,5 × 36 / 1.25 = <u>297.9</u> kN

$F_{t.Rd.conn}$ = 4 × 197.6 = <u>790.4</u> kN < $N_{pl.Rd}$ = 78.1 × 23.5 / 1.0 = <u>**1 835.35**</u> kN → will fail!

Necessary number of bolts:

n_{nec} = 1 835.35 / 197.6 = 9.3 bolts → **n_{appl} = 10 bolts!**

alternate sample:



HE 100 A bar, S235 → f_y = 23.5 kN/cm^2, f_u = 35 kN/cm^2

A_{HE100A} = 21.2 cm^2 → $N_{pl.Rd}$ = 21.2 × 23.5 / 1.0 = <u>**498.2**</u> kN

4 × M22 8.8 → $F_{t.Rd.conn}$ = 4 × 197.6 = <u>790.4</u> kN → satisfies!

4 × M20 10.9 bolts, A_s = 1,92 cm^2 , d_m = 1.07735 × s = 1.07735 × 3 = 3,02 cm

$$F_{t,Rd} = \frac{k_2 \; f_{ub} \; A_s}{\gamma_{M2}}$$ = 0.9 × 100 × 1,92 / 1.25 = <u>138.2</u> kN

$$B_{p,Rd} \quad = \quad 0{,}6 \, \pi \, d_m \, t_p \, f_u \, / \, \gamma_{M2}$$

$B_{p.Rd}$ = 0.6 × π × 3,02 × 1,5 × 36 / 1.25 = <u>245.9</u> kN

$F_{t.Rd.conn}$ = 4 × 138.2 = <u>552.8</u> kN > $N_{pl.Rd}$ = 21.2 × 23.5 / 1.0 = <u>**498.2**</u> kN → satisfies!

4.5.1 Length of welds

(1) ⟦AC2⟩ The effective length of a fillet weld l_{eff} ⟨AC2⟧ should be taken as the length over which the fillet is full-size. This maybe taken as the overall length of the weld reduced by twice the effective throat thickness a. Provided that the weld is full size throughout its length including starts and terminations, no reduction in effective length need be made for either the start or the termination of the weld.

(2) A fillet weld with an effective length less than 30 mm or less than 6 times its throat thickness, whichever is larger, should not be designed to carry load.

4.5.2 Effective throat thickness

(1) The effective throat thickness, a, of a fillet weld should be taken as the height of the largest triangle (with equal or unequal legs) that can be inscribed within the fusion faces and the weld surface, measured perpendicular to the outer side of this triangle, see Figure 4.3.

(2) The effective throat thickness of a fillet weld should not be less than 3 mm.

(3) In determining the design resistance of a deep penetration fillet weld, account may be taken of its additional throat thickness, see Figure 4.4, provided that preliminary tests show that the required penetration can consistently be achieved.

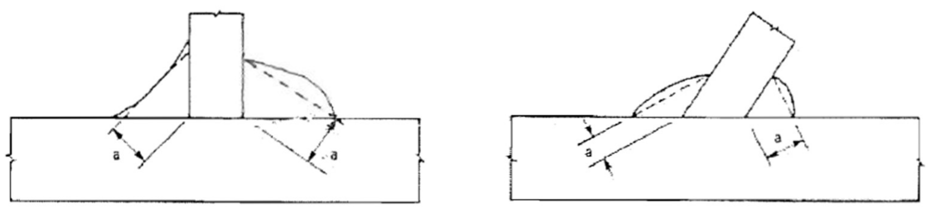


Figure 4.3: Throat thickness of a fillet weld

4.5.3.3 Simplified method for design resistance of fillet weld

$$F_{w,Ed} \leq F_{w,Rd}$$

where:

$F_{w,Ed}$ is the design value of the weld force per unit length;

$F_{w,Rd}$ is the design weld resistance per unit length.

$F_{w.Ed} = R / l_{tot}$ [kN / cm]

where

R – is resultant of the external forces [kN]
l_{tot} – is the total lengths of welds in the investigated connection [cm]

$$F_{w,Rd} = f_{vw.d}\, a \qquad [\ kN\ /\ cm\ ]$$

where:

$f_{vw.d}$ is the design shear strength of the weld.

$$f_{vw.d} = \frac{f_u / \sqrt{3}}{\beta_w \gamma_{M2}} \qquad [\ kN\ /\ cm^2\ ]$$

f_u is the nominal ultimate tensile strength of the weaker part joined;

β_w is the appropriate correlation factor taken from Table 4.1.

Table 4.1: Correlation factor β_w for fillet welds

Standard and steel grade			Correlation factor β_w
EN 10025	**EN 10210**	**EN 10219**	
S 235 S 235 W	S 235 H	S 235 H	0,8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,9
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,0
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,0

4.5.3.2 Directional method

$$[\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)]^{0,5} \leq f_u / (\beta_w \gamma_{M2}) \quad \text{and} \quad \sigma_\perp \leq 0.9 f_u / \gamma_{M2}$$

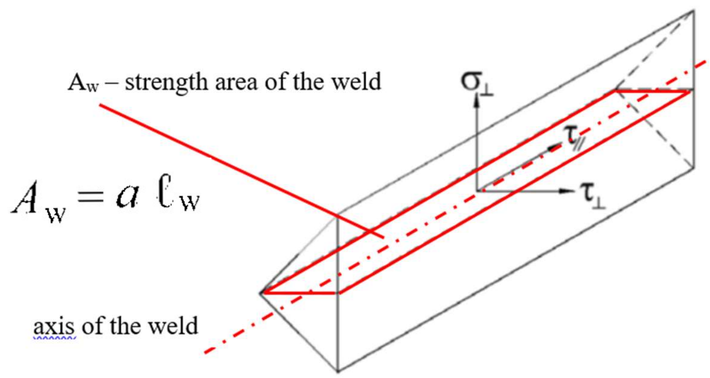
where:

f_u is the nominal ultimate tensile strength of the weaker part joined;

β_w is the appropriate correlation factor taken from Table 4.1.

$$\sqrt{\sigma_\perp^2 + 3\left(\tau_\perp^2 + \tau_{II}^2\right)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$$

$$\sigma_\perp \leq 0,9 \frac{f_u}{\gamma_{M2}}$$



$\sigma_\perp$ is the normal stress perpendicular to the throat

$\tau_\perp$ is the shear stress (in the plane of the throat) perpendicular to the axis of the weld

$\tau_\parallel$ is the shear stress (in the plane of the throat) parallel to the axis of the weld.

Sample 1:

S235 $\qquad f_y = 23{,}5\ kN/cm^2 \qquad f_u = 36{,}0\ kN/cm^2 \qquad \beta_w = 0{,}8$

$t = 10\ mm$

$F_{Ed} = 150\ kN$

Simplified method:

$$f_{vw,d} = \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} = \frac{36}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} = 20{,}78\ kN/cm^2$$

$$F_{w,Rd} = f_{vw,d} \cdot a = 20{,}78 \cdot 0{,}5 = 10{,}39\ kN/cm$$

$$F_{w,Ed} = \frac{F_{Ed}}{2 \cdot \ell}$$

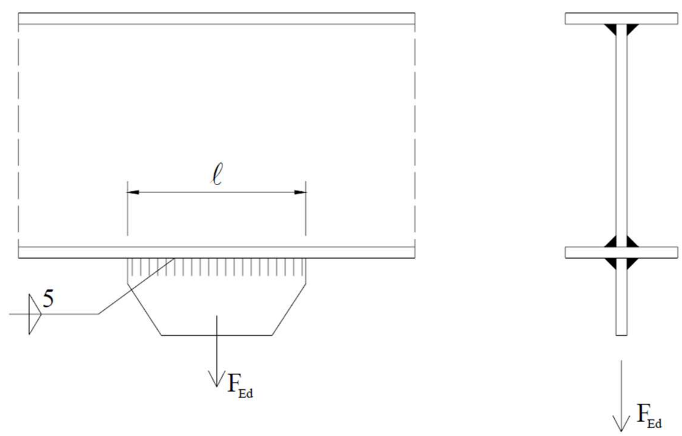
$$F_{w,Ed} \le F_{w,Rd}$$

$$\frac{F_{Ed}}{2 \cdot \ell} \le 10{,}39$$

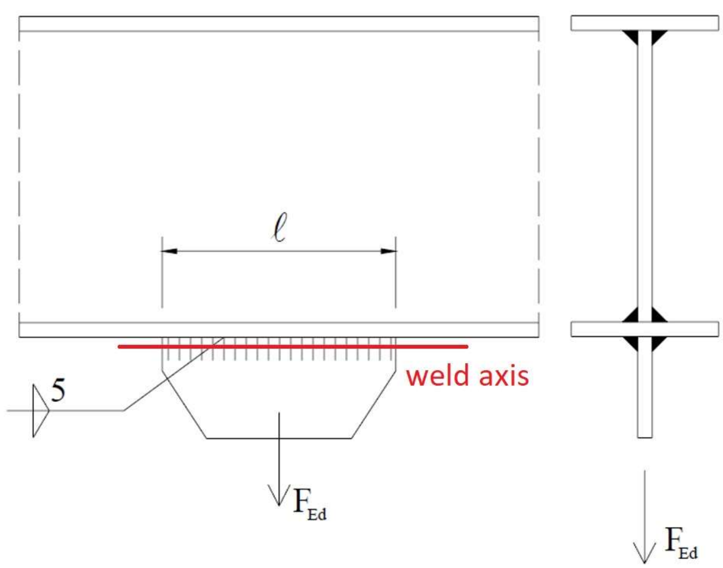
$$\frac{150}{2 \cdot \ell} \le 10{,}39$$

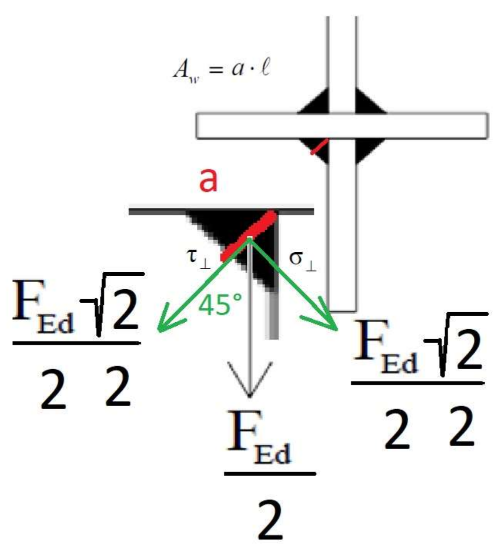
$$\ell \ge \frac{150}{2 \cdot 10{,}39}$$

$$\ell \ge 7{,}21\ cm \quad \rightarrow \quad \ell_{alk} = 8\ cm$$



Directional method:





$$A_w = a \cdot \ell = 0{,}5 \cdot \ell$$

$$\tau_{\perp} = \sigma_{\perp} = \frac{F_{Ed}}{2}\frac{\sqrt{2}}{2}\frac{1}{A_w} = \frac{150}{2}\frac{\sqrt{2}}{2}\frac{1}{0{,}5 \cdot \ell} \; ; \qquad \tau_{II} = 0$$

$$\sqrt{\sigma_{\perp}^2 + 3\left(\tau_{\perp}^2 + \tau_{II}^2\right)} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}$$

$$\sqrt{\left(\frac{150 \cdot \sqrt{2}}{1{,}0 \cdot \ell \cdot 2}\right)^2 + 3\left(\frac{150 \cdot \sqrt{2}}{1{,}0 \cdot \ell \cdot 2}\right)^2} \leq \frac{36}{0{,}8 \cdot 1{,}25}$$

$$2 \cdot \frac{150 \cdot \sqrt{2}}{1{,}0 \cdot \ell \cdot 2} \leq 36$$

$$\ell \geq 5{,}89\ cm \ \rightarrow \ \ell_{alk} = 6\ cm$$

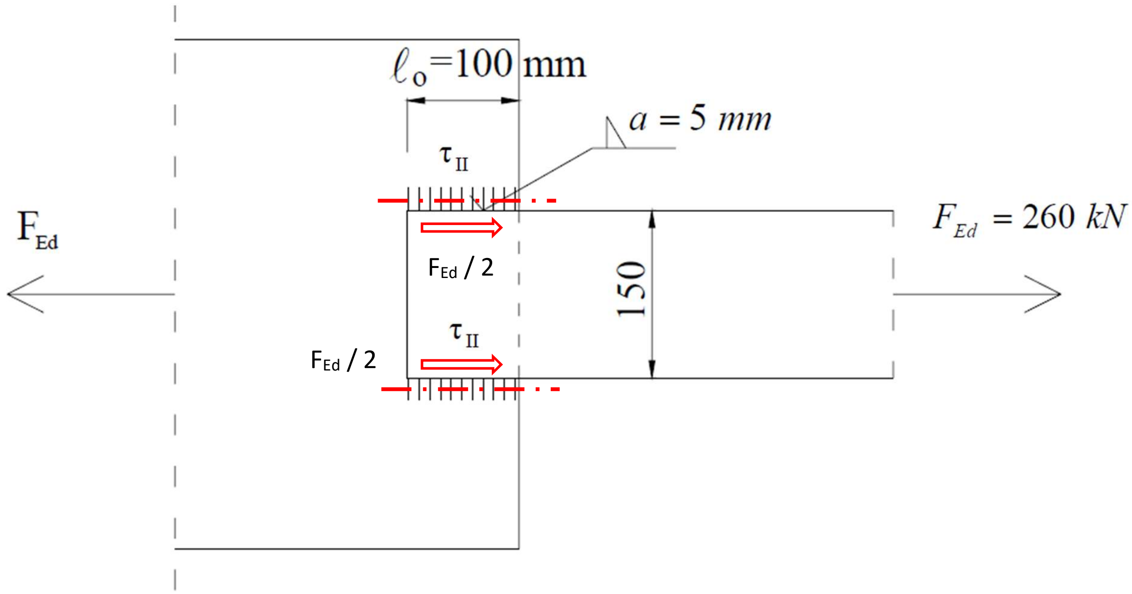
$$\sigma_{\perp} \leq \frac{f_u}{\gamma_{M2}}$$

$$\frac{150 \cdot \sqrt{2}}{1{,}0 \cdot \ell \cdot 2} \leq \frac{36}{1{,}25} = 28{,}8$$

$$\ell \geq 3{,}68\ \mathrm{cm}$$

Sample 2:

S235 $f_y = 23{,}5\ kN/cm^2$ $f_u = 36{,}0\ kN/cm^2$ $\beta_w = 0{,}8$



<u>Simplified method:</u>

$$f_{vw,d} = \frac{f_u}{\sqrt{3}\cdot\beta_w\cdot\gamma_{M2}} = \frac{36}{\sqrt{3}\cdot 0{,}8\cdot 1{,}25} = 20{,}78\ kN/cm^2$$

$$F_{w,Rd} = f_{vw,d}\cdot a = 20{,}78\cdot 0{,}5 = 10{,}39\ kN/cm$$

$$F_{Rd} = F_{w,Rd}\cdot\Sigma\ell = 10{,}39\cdot 2\cdot 10 = 207{,}8\ kN$$

F_{Ed} = 260 kN > F_{Rd} = 207,8 kN → welded connection FAILS!

<u>Directional method:</u>

$$\tau_{II} = \frac{F_{Ed}}{2\cdot\ell\cdot a} = \frac{260}{2\cdot 10\cdot 0{,}5} = 26{,}0\ kN/cm^2; \qquad \sigma_\perp = \tau_\perp = 0$$

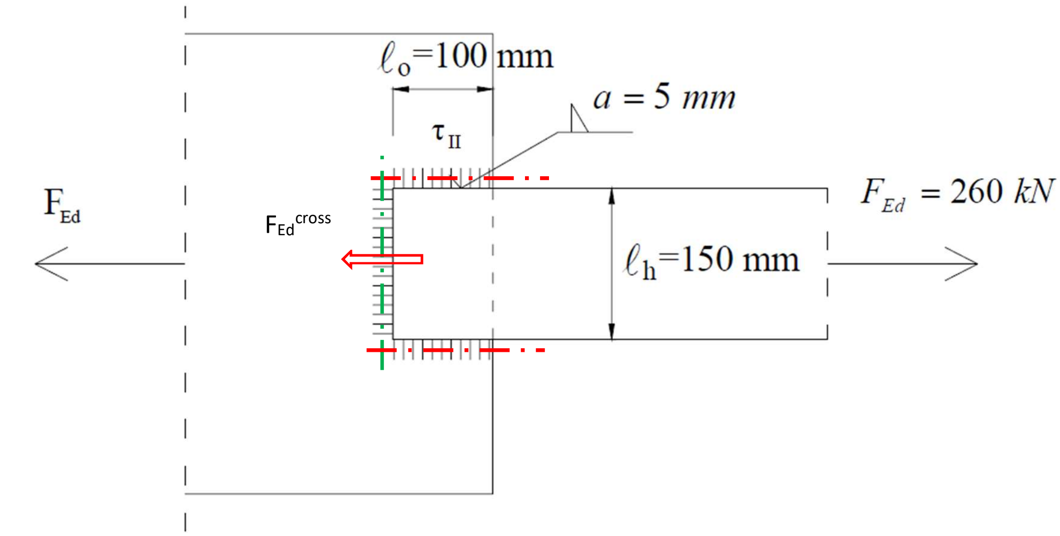
$$\sqrt{\sigma_\perp^2 + 3\left(\tau_\perp^2 + \tau_{II}^2\right)} \le \frac{f_u}{\beta_w\cdot\gamma_{M2}}$$

$$\sqrt{3}\cdot\tau_{II} = \sqrt{3}\cdot 26{,}0 = 45{,}0\ kN/cm^2 > \frac{f_u}{\beta_w\cdot\gamma_{M2}} = 36\ kN/cm^2$$

→ welded connection FAILS!

Sample 3:

S235 $\quad f_y = 23{,}5\ kN/cm^2 \quad f_u = 36{,}0\ kN/cm^2 \quad \beta_w = 0{,}8$



<u>Simplified method:</u>

$$f_{vw,d} = \frac{f_u}{\sqrt{3}\cdot\beta_w\cdot\gamma_{M2}} = \frac{36}{\sqrt{3}\cdot 0{,}8\cdot 1{,}25} = 20{,}78\ kN/cm^2$$

$$F_{w,Rd} = f_{vw,d}\cdot a = 20{,}78\cdot 0{,}5 = 10{,}39\ kN/cm$$

$$F_{Rd} = F_{w,Rd}\cdot\Sigma\ell = 10{,}39\cdot(2\cdot 10 + 15) = 363{,}65\ kN$$

F_{Ed} = 260 kN < F_{Rd} = 363.65 kN → welded connection is satisfying!

<u>Directional method:</u>

<u>side fillet welds:</u>

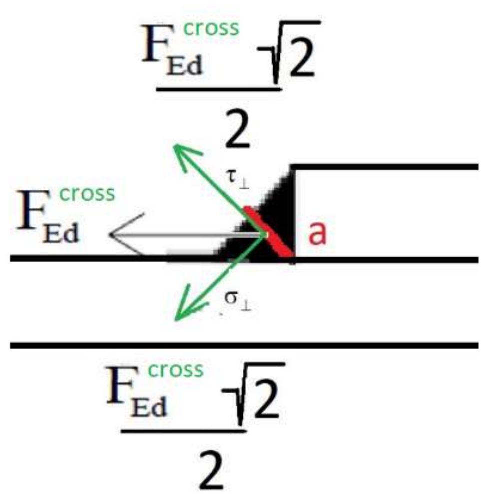
$$\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_{II}^2)} \le \frac{f_u}{\beta_w \cdot \gamma_{M2}}$$

$$\sqrt{3}\cdot\tau_{II} \le \frac{f_u}{\beta_w\cdot\gamma_{M2}}$$

$$\tau_{II} \le \frac{f_u}{\beta_w\cdot\gamma_{M2}\cdot\sqrt{3}} = 20{,}78\ kN/cm^2$$

$$F^{o}_{w,Rd} = \tau_{II}\cdot a\cdot\Sigma\ell = 20{,}78\cdot 0{,}5\cdot(10+10) = 207{,}8\ kN$$

$F^{side}_{w,Rd}$ = 207.8 kN



F_{Ed}^{cross} = 260 kN – 207.8 kN = <u>52.2 kN</u>

$$\sigma_{\perp} = \tau_{\perp} = \frac{F_h}{\Sigma a \cdot \ell} \cdot \frac{\sqrt{2}}{2} = \frac{52{,}2}{0{,}5 \cdot 15} \frac{\sqrt{2}}{2} = 4{,}92\ kN/cm^2; \qquad \tau_{II} = 0$$

$$\sqrt{\sigma_{\perp}^2 + 3 \cdot \tau_{\perp}^2} \leq \frac{f_u}{\beta_W \cdot \gamma_{M2}}$$

$$\sqrt{4{,}92^2 + 3 \cdot 4{,}92^2} = 9{,}84\ kN/cm^2 \leq \frac{36}{0{,}8 \cdot 1{,}25} = 36{,}0\ kN/cm^2$$

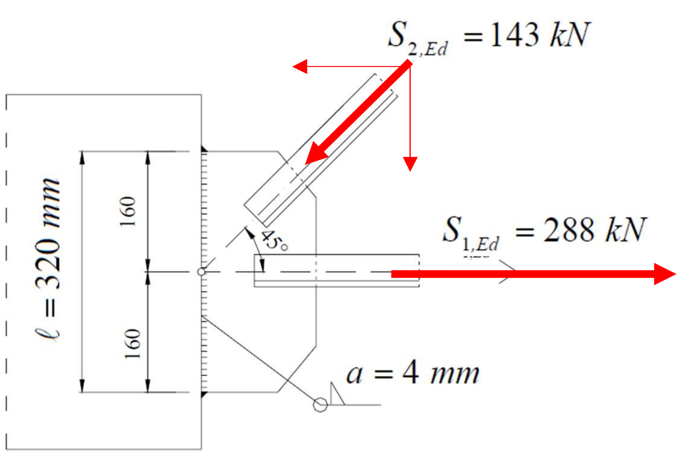
→ cross fillet weld is satisfying!

$$\sigma_{\perp} \leq \frac{f_u}{\gamma_{M2}}$$

→ cross fillet weld is satisfying!

$$4{,}92 \leq \frac{36}{1{,}25} = 28{,}8\ kN/cm^2$$

Sample 4:



S235 $\quad f_y = 23{,}5\ kN/cm^2 \quad f_u = 36{,}0\ kN/cm^2 \quad \beta_w = 0{,}8$

Simplified method:

$$f_{vw,d} = \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} = \frac{36}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} = 20{,}78\ kN/cm^2$$

$$F_{w,Rd} = a \cdot \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} = 0{,}4 \cdot \frac{36}{\sqrt{3} \cdot 0{,}8 \cdot 1{,}25} = 8{,}314\ kN/cm = 831{,}4\ kN/m$$

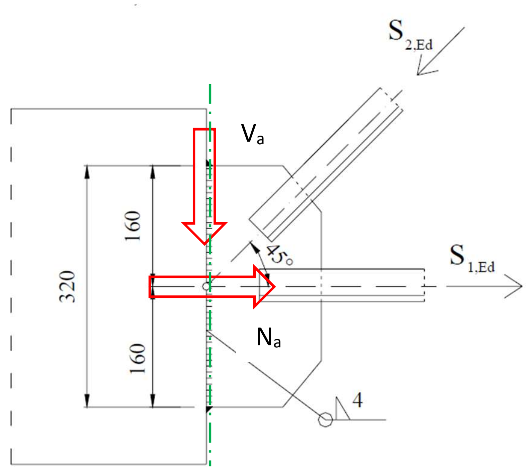
$$N_a = S_{1,Ed} - S_{2,Ed} \cdot cos\,45^\circ = 288 - \frac{143}{\sqrt{2}} = 186{,}9\ kN$$

$$V_a = S_{2,Ed} \cdot sin\ 45^\circ = \frac{143}{\sqrt{2}} = 101{,}1\ kN$$

$$F_{w,Ed} = \frac{\sqrt{N_a^2 + V_a^2}}{2 \cdot \ell} = \frac{\sqrt{186{,}9^2 + 101{,}1^2}}{2 \cdot 0{,}32} = 332{,}0\ kN/m \le F_{w,Rd} = 831{,}4\ kN/m$$

→ welded connection is satisfying!

Directional method:



sin 45° = cos 45° = 1/ $(2)^{0.5}$ = $(2)^{0.5}$/ 2

$$A_w = 2 \cdot 0{,}4 \cdot 32 = 25{,}6\ cm^2$$

$$\tau_\perp = \sigma_\perp = \frac{N_a}{\sqrt{2} \cdot A_w} = \frac{186{,}9}{\sqrt{2} \cdot 25{,}6} = 5{,}16\ kN/cm^2$$

$$\tau_{II} = \frac{V_a}{A_w} = \frac{101{,}1}{25{,}6} = 3{,}95\ kN/cm^2$$

$$\sqrt{\sigma_\perp^2 + 3\left(\tau_\perp^2 + \tau_{II}^2\right)} \le \frac{f_u}{\beta_w \cdot \gamma_{M2}}$$

$$\sqrt{5{,}16^2 + 3 \cdot \left(5{,}16^2 + 3{,}95^2\right)} = 12{,}38\ kN/cm^2 \le \frac{36}{0{,}8 \cdot 1{,}25} = 36\ kN/cm^2$$

$$\sigma_\perp = 5{,}16\ kN/cm^2 \le \frac{f_u}{\gamma_{M2}} = 28{,}8\ kN/cm^2$$

→ welded connection is satisfying!

a) **Chord face failure** (plastic failure of the chord face) or chord plastification (plastic failure of the chord cross-section);

b) **Chord side wall failure** (or **chord web failure**) by yielding, crushing or instability (crippling or buckling of the chord side wall or chord web) under the compression brace member;

c) **Chord shear failure**;

d) **Punching shear** failure of a hollow section chord wall (crack initiation leading to rupture of the brace members from the chord member);

e) **Brace failure** with reduced effective width (cracking in the welds or in the brace members);

f) **Local buckling** failure of a brace member or of a hollow section chord member at the joint location.

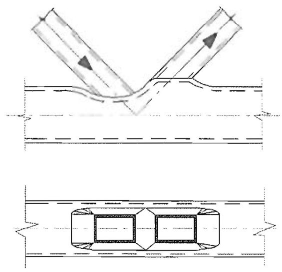
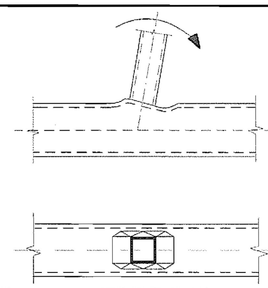
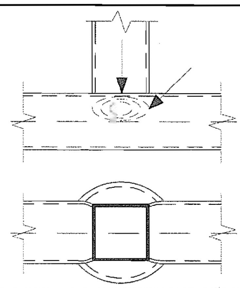
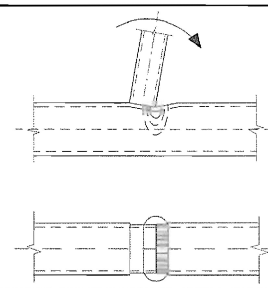
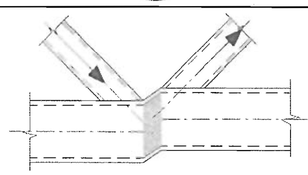
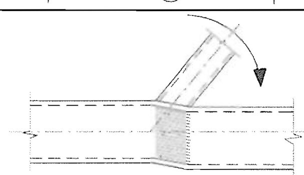
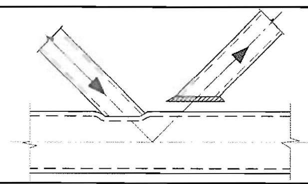
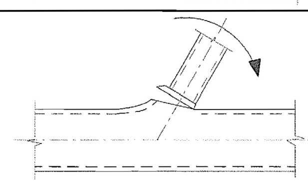
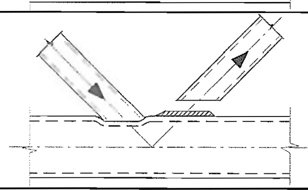
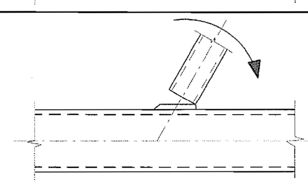
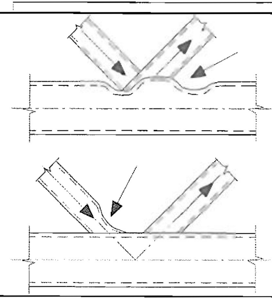
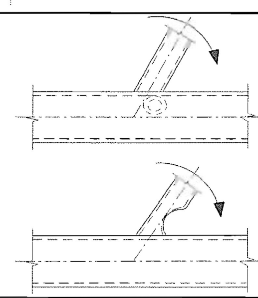
Mode	Axial loading	Bending moment
a		
b		
c		
d		
e		
f		

Figure 7.3: Failure modes for joints between RHS brace members and RHS chord members

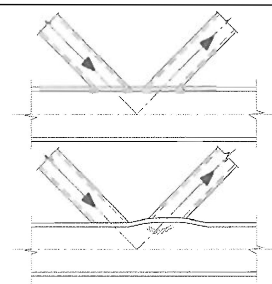
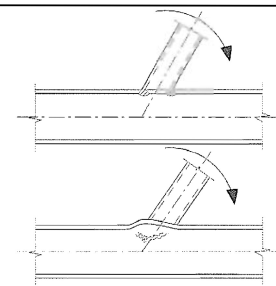
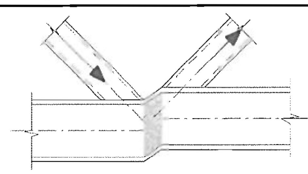
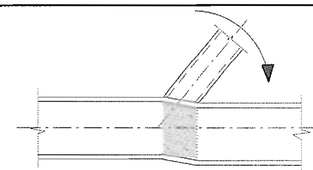
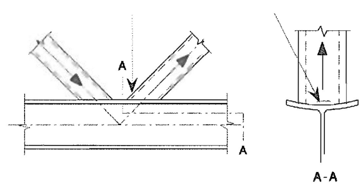
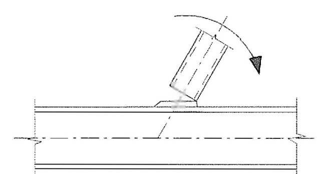
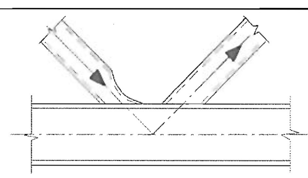
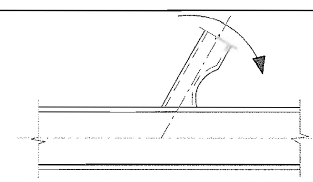
Mode	Axial loading	Bending moment
a	–	–
b		
c		
d	–	–
e		
f		

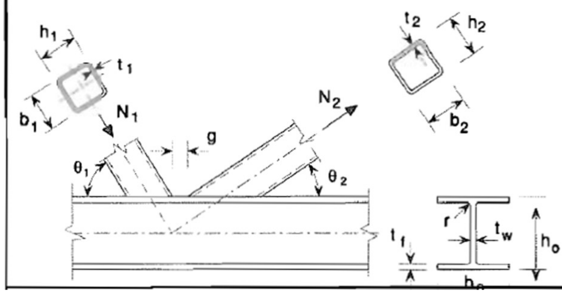
Figure 7.4: Failure modes for joints between CHS or RHS brace members and I or H section chord members

Design the resistance of a welded K-joint
RHS brace bars (truss bars) and hot rolled I-section chord

Table 7.20: Range of validity for welded joints between CHS or RHS brace members and I or H section chord members

Type of joint	Joint parameter [i = 1 or 2, j = overlapped brace]					
	d_w/t_w	b_i/t_i and h_i/t_i or d_i/t_i		h_i/b_i	b_0/t_f	b_i/b_j
		Compression	Tension			
X	Class 1 and $d_w \leq 400$ mm	⟨AC2⟩ Class 1 or 2 ⟨AC2⟩ and $\frac{h_i}{t_i} \leq 35$ $\frac{b_i}{t_i} \leq 35$ $\frac{d_i}{t_i} \leq 50$	$\frac{h_i}{t_i} \leq 35$ $\frac{b_i}{t_i} \leq 35$ $\frac{d_i}{t_i} \leq 50$	$\geq 0{,}5$ but $\leq 2{,}0$	⟨AC2⟩ Class 1 or 2 ⟨AC2⟩	–
T or Y K gap N gap	⟨AC2⟩ Class 1 or 2 ⟨AC2⟩ and $d_w \leq 400$ mm			1,0		–
K overlap N overlap				$\geq 0{,}5$ but $\leq 2{,}0$		$\geq 0{,}75$ ⟨AC2⟩ $25\% \leq \lambda_{ov} \leq \lambda_{ov,lim.}$ 1) ⟨AC2⟩

Table 7.21: Design resistances of welded joints between RHS or CHS brace members and I or H section chords

Type of joint	Design resistance [i = 1 or 2, j = overlapped brace]	
K and N gap joints [i = 1 or 2]	⟨AC2⟩ Chord web yielding ⟨AC2⟩ ⟨AC2⟩ $N_{1,Rd} = \frac{f_{y0} t_w b_w}{\sin\theta_1} / \gamma_{M5}$ ⟨AC2⟩	Brace failure need not be checked if: $g/t_f \leq 20 - 28\beta$; $\beta \leq 1{,}0 - 0{,}03\gamma$ where $\gamma = b_0/2t_f$ and for CHS: $0{,}75 \leq d_1/d_2 \leq 1{,}33$ or for RHS: $0{,}75 \leq b_1/b_2 \leq 1{,}33$
	Brace failure $N_{i,Rd} = 2 f_{yi} t_i p_{eff} / \gamma_{M5}$	
	Chord shear $N_{i,Rd} = \frac{f_{y0} A_v}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$ $N_{0,Rd} = \left[(A_0 - A_v) f_{y0} + A_v f_{y0} \sqrt{1 - (V_{Ed}/V_{pl,Rd})^2} \right] / \gamma_{M5}$	

$A_v = A_0 - (2-\alpha)\, b_0\, t_f + (t_w + 2r)\, t_f$ For RHS brace: AC2 $\alpha = \sqrt{\dfrac{1}{(1+4g^2/(3t_f^2))}}$ AC2	AC2 $p_{eff} = t_w + 2r + 7t_f f_{y0}/f_{yi}$ but for T, Y, X joints and K and N gap joints: $p_{eff} \le b_i + h_i - 2t_i$ but for K and N overlap joints: $p_{eff} \le b_i$ AC2	$b_w = \dfrac{h_i}{\sin\theta_i} + 5(t_f + r)$ but $b_w \le 2t_i + 10\,(t_f + r)$
For CHS brace: $\alpha = 0$	$b_{e,ov} = \dfrac{10}{b_j/t_j}\dfrac{f_{yj}t_j}{f_{yi}t_i} b_i$ but $b_{e,ov} \le b_i$	

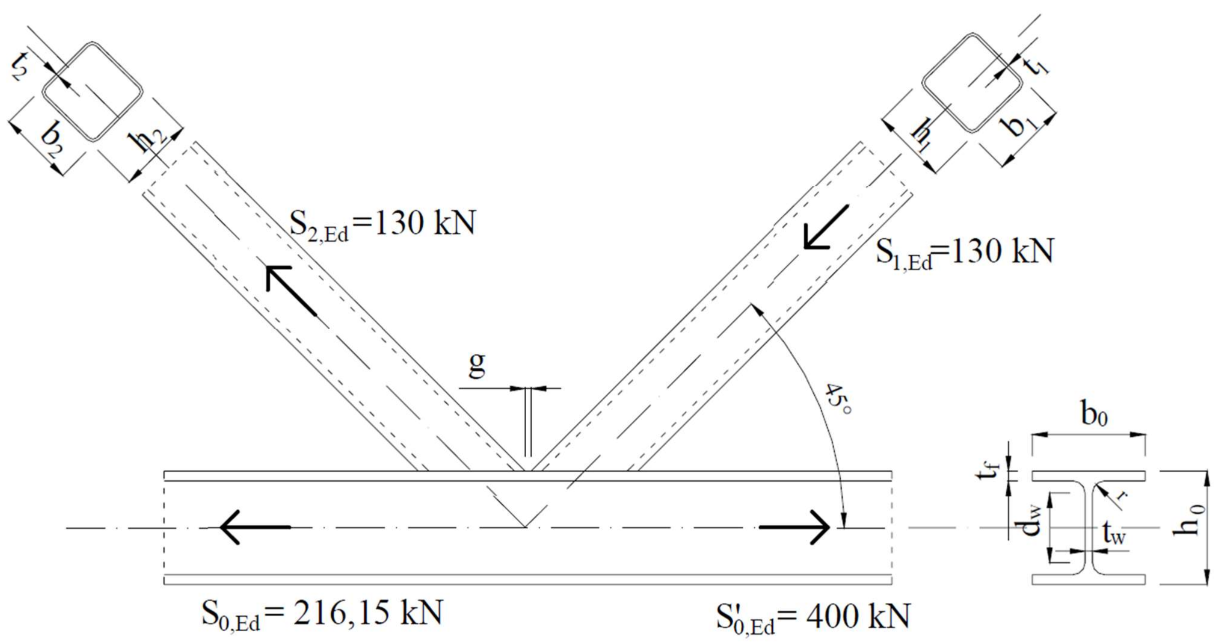
Sample

The welded K joint on the figure is at the bottom chord of a steel truss. The chord bar is an HE 120 AA hot-rolled I-section, truss bars are 80×80×4 sections, the directions are 45°.

a) Determine the resistance of the K-joint!

b) Let's check the a = 4 mm fillet welds between the truss and chord bars!

Steel grade: S275, f_y = 27,5 kN/cm^2, f_u = 43,0 kN/cm^2, β_w = 0,85



chord: HE 120 AA		compressed truss 80×80×4	tensioned truss 80×80×4
b_0 = 120 mm	t_f = 5,5 mm	b_1 = 80 mm	b_1 = 80 mm
h_0 = 109 mm	t_w = 4,2 mm	h_1 = 80 mm	h_1 = 80 mm
r = 12 mm		t_1 = 4 mm	t_1 = 4 mm
A_{vz0} = 6,9 cm^2	A_0 = 18,6 cm^2		

gap: g = 20 mm

a) Resistance of the welded K-joint in case of I-sectioned chord bar

Range verification:

- chord bar

1. $d_w = h_0 - 2t_f - 2r = 109 - 2 \cdot 5{,}5 - 2 \cdot 12 = 74\ mm < 400\ mm$ → Ok

2. class 1 or 2

$c_w = h_0 - 2r - 2t_f = 109 - 2 \cdot 12 - 2 \cdot 5{,}5 = 74\ mm$

$$\frac{c_w}{t_w} = \frac{74}{4{,}2} = 17{,}62 < 33 \cdot \varepsilon = 33 \cdot 0{,}92 = 30{,}36$$

The web plate is class 1 Ok

- compressed truss bar

3. $\frac{b_1}{t_1} = \frac{80}{4} = 20{,}0 \leq 35$ → Ok

4. $\frac{h_1}{t_1} = \frac{80}{4} = 20{,}0 \leq 35$ → Ok

5. $\frac{h_1}{b_1} = \frac{80}{80} = 1{,}0$ → Ok

7. class 1 or 2

$c_f = h_1 - 2 \cdot r - 2 \cdot t_1 = 80 - 2 \cdot 8 - 2 \cdot 4 = 56\ mm$

$$\frac{c_f}{t_1} = \frac{56}{4} = 14{,}0 < 33 \cdot \varepsilon = 33 \cdot 0{,}92 = 30{,}36$$

The bar is class 1 → Ok

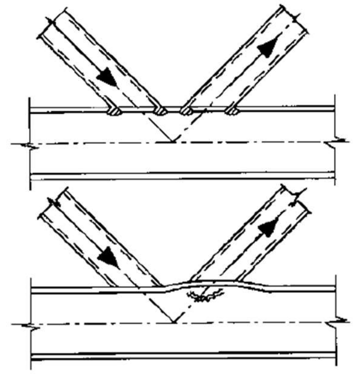
- tensioned truss bar

8. $\frac{b_2}{t_2} = \frac{80}{4} = 20{,}0 \leq 35$ → Ok

9. $\frac{h_2}{t_2} = \frac{80}{4} = 20{,}0 \leq 35$ → Ok

11. $\frac{h_2}{b_2} = \frac{80}{80} = 1{,}0$ → Ok

Chord web yielding



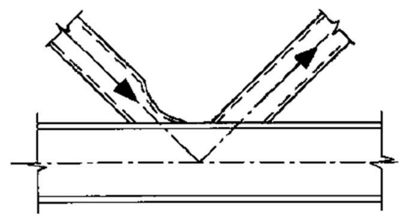
$$N_{i,Rd} = \frac{f_{y0} t_w b_w}{\sin\theta_i \cdot \gamma_{M5}} = \frac{27{,}5 \cdot 0{,}42 \cdot 18{,}30}{\sin 45 \cdot 1{,}0} = 298{,}92\ kN$$

where:

$$b_w = \min\left[\frac{h_i}{\sin\theta_i} + 5(t_f + r);\quad 2t_i + 10(t_f + r)\right]$$

$$b_w = \min[200{,}64;\ 183{,}0] = 183{,}0\ mm$$

Brace failure



$$N_{i,Rd} = 2 f_{yi} t_i p_{eff} / \gamma_{M5} = 2 \cdot 27{,}5 \cdot 0{,}4 \cdot 6{,}67 / 1{,}0 = 146{,}74 kN$$

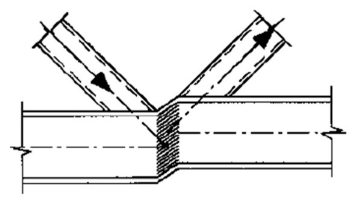
where:

$$p_{eff} = \min\left[t_w + 2r + 7 t_f f_{y0} / f_{yi};\ \ b_i + h_i - 2t_i\right]$$

$$p_{eff} = \min[66{,}7; 152] = 66{,}7 mm$$

Chord shear

1. criterion of truss bars



$$N_{i,Rd} = \frac{f_{y0} A_v}{\sqrt{3}\sin\theta_i \cdot \gamma_{M5}} = \frac{27{,}5 \cdot 8{,}52}{\sqrt{3}\sin 45 \cdot 1{,}0} = 191{,}30\ kN$$

where:

$$\alpha = \sqrt{\frac{1}{1 + 4g^2 / 3t_f^2}} = \sqrt{\frac{1}{1 + 4 \cdot 20^2 / 3 \cdot 5{,}5^2}} = 0{,}238$$

$$A_v = A_0 - (2 - \alpha) \cdot b_0 t_f + (t_w + 2r) \cdot t_f$$

$$A_v = 18{,}6 - (2 - 0{,}238) \cdot 12{,}0 \cdot 0{,}55 + (0{,}42 + 2 \cdot 1{,}2) \cdot 0{,}55 = 8{,}52\ cm^2$$

2. criterion of chord bar

$$N_{0,Rd} = \frac{(A_0 - A_v) f_{y0} + A_v f_{y0} \sqrt{1 - (V_{Ed} / V_{pl,Rd})^2}}{\gamma_{M5}}$$

where:

V_{Ed} : vertical component of the truss bar force

$$V_{Ed} = S_{1,Ed} \sin 45 = 130 \cdot \sin 45 = 91{,}92\ kN$$

$$V_{pl,Rd} = \frac{A_{v0} \cdot f_{y0}}{\sqrt{3} \cdot \gamma_{M0}} = \frac{6{,}9 \cdot 27{,}5}{\sqrt{3} \cdot 1{,}0} = 109{,}55\ \text{kN}$$

substituting:

$$N_{0,Rd} = \frac{(18{,}6 - 8{,}52) \cdot 27{,}5 + 8{,}52 \cdot 27{,}5 \sqrt{1 - (91{,}92 / 109{,}55)^2}}{1{,}0} = 404{,}67\ \text{kN}$$

$$N_{i,Rd} = min\ [298{,}92\ kN;\ 146{,}74\ kN;\ 191{,}30\ kN]$$

$N_{i,Rd} = 146{,}74\ kN > S_{1,Ed} = 130\ kN \quad \rightarrow \quad$ truss bar is satisfying

$N_{0,Rd} = 404{,}67\ kN > S'_{0,Ed} = 400\ kN \quad \rightarrow \quad$ chord bar is satisfying

b) Let’s check the a = 4 mm fillet welds between the truss and chord bars!

We apply full-strength connection, therefore the fillet weld must resist against the tension resistance of the truss bar.

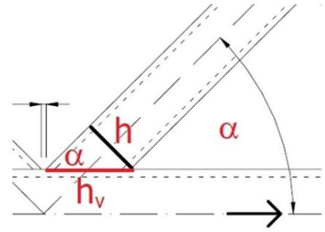
We apply the simplified method.

$$f_{vw,d} = \frac{f_u}{\beta_w \cdot \gamma_{M2} \cdot \sqrt{3}} = \frac{43}{0{,}85 \cdot 1{,}25 \cdot \sqrt{3}} = 23{,}37\ kN/cm^2$$

$$F_{w,Rd} = f_{vw,d} \cdot a = 0{,}4 \cdot 23{,}37 = 9{,}35\ kN/cm$$

$$N_{t,Rd} = N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{11{,}74 \cdot 27{,}5}{1{,}0} = 332{,}85\ kN$$

$$K = 2 \cdot (b_2 + \frac{h_2}{sin\,45°}) = 2 \cdot (80 + 80 \cdot \sqrt{2}) = 386{,}3\ mm$$



$\sin \alpha = h / h_v$

$h_v = h / \sin \alpha$

$$\frac{N_{t,Rd}}{K} = \frac{322{,}85}{38{,}63} = 8{,}36\ kN/cm$$

$$F_{w,Rd} = 9{,}35\ kN/cm > \frac{N_{t,Rd}}{K} = 8{,}36\ kN/cm$$

→ the fillet weld is satisfying