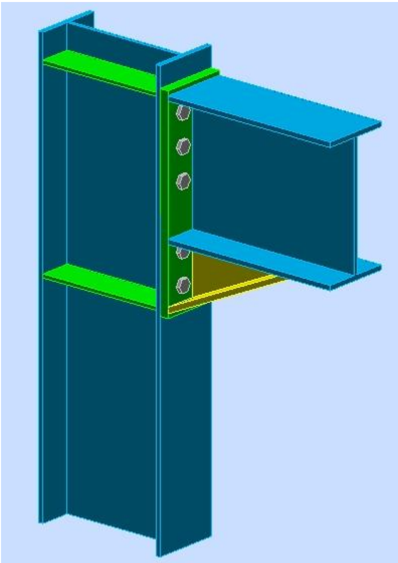
Metal Structures

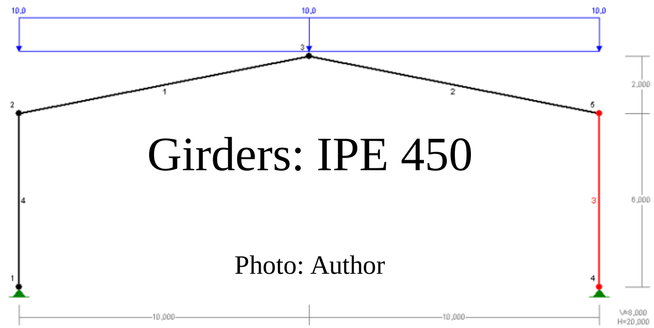
Design Project III

Steel hall – examples of calculation (part II)

III[rd] example of calculations - bolted joint girder-column

Photo: kilonewton.prv.pl





Girders: IPE 450

Photo: Author

$M_{Ed} = 311,6$ kNm

Columns: IPE 450

$N_{Ed,\ column} = 97,6$ kN

$N_{Ed,\ girder} = 63,9$ kN

$V_{Ed,\ girder} = 80,5$ kN

M_{Ed} is value of bending moment (the same in column and girder) important for resistance of joint;

$N_{Ed,\ girder}$, $N_{Ed,\ column}$ are axial forces in girder and column, important for resistance of joint; additionally this value should be checked for condition $N_{Ed} \leq 0,05\ N_{Rd}$;

$V_{Ed,\ girder}$ shear force in in girder important for resistance of join.

Calculation of bolted joint beam-column is the most complicated type of calculation in Metal Structures on Ist step of study. Algorithm of calculation is as follow:

- initial assumptions (#t / 5 – 34)
- resistance of welds (#t / 35 – 38)
- stiffness of joint (#t / 39 – 49)
- resistance of joint for bending moment (#t / 50 – 78)
- resistance of joint for shear force (#t / 79 – 81)

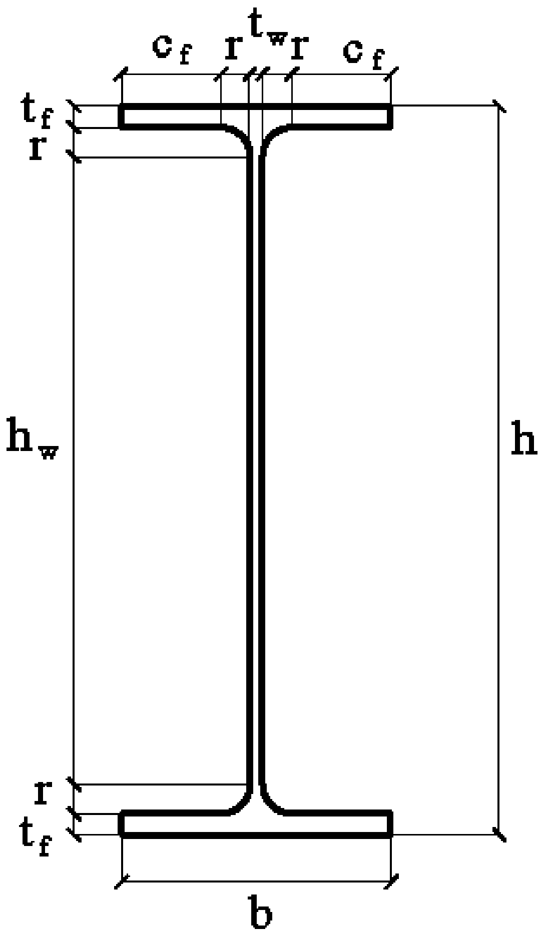


Photo: Author

IPE 450 [mm]

h	b	t_f	t_w	r
450	190	14,6	9,4	21

S235

$A = 98{,}82\ cm^2$

$J_y = 33\ 740\ cm^4$

$J_z = 1\ 676\ cm^4$

$W_{y,\ pl} = 1\ 500\ cm^3$

$W_{z,\ pl} = 276{,}4\ cm^3$

$A_{V,\ c} \approx 42{,}3\ cm^2$

$M_{Rd,\ y} = 352{,}50\ kNm$

$N_{Rd} = 2\ 322{,}27\ kN$

$V_{Rd} = 573{,}92\ kN$

Initial assumptions about geometry of joint

First checking:

EN 1993-1-8 6.2.3.(2)

„Provided that the axial force in the connected member does not exceed 5% of the design resistance $N_{pl, Rd}$ of its cross-section, the design moment resistance of a beam-to column joint or beam splice may be determined using the method given in 6.2.7."

Eurocode does not provide clear information on what happens if this value is exceeded. It is therefore best to:

$$N_{Ed, girder} \leq 0{,}05\ N_{Rd, girder}$$

$$N_{Ed, column} \leq 0{,}05\ N_{Rd, column}$$

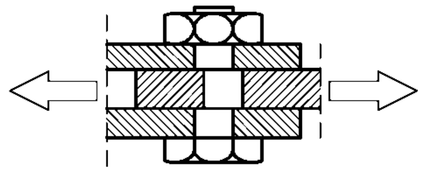
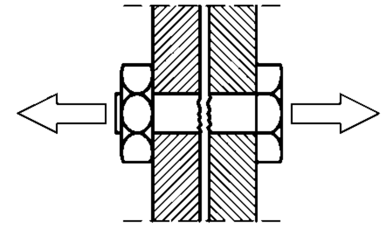
$0{,}05\ N_{Rd, girder} = 0{,}05\ N_{Rd, column} = 117{,}763$ kN

max ($N_{Ed, girder}$; $N_{Ed, column}$) = 97,6 kN

In analised case: axial force in join could be neglected. Only bending moment and shear force will be taken into consieration according to Eurocode procedures.

→ #15 / 13

Categories of bolted joints and loads

	A	B	C	D	E
					
Categories of bolted joint	A	B	C	D	E
Types of loads	Static without changing the direction of the bending moments; aerodynamic	Static with changing the direction of the bending moments; aerodynamic	Dynamic	Static; aerodynamic	Dynamic
Types of bolts	„normal"	preloaded		„normal"	preloaded

Changing the direction of the bending moment: various combinations of loads

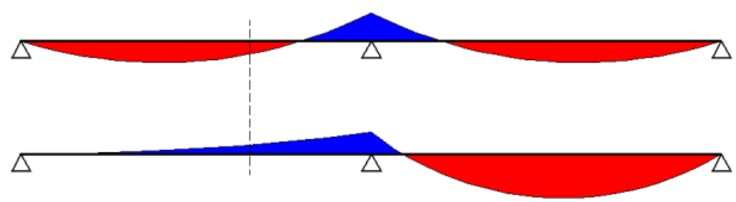


Photo: Author

Type of loads, act on hall: dead-weigth, snow, wind. There are no dynamic actions.

Lec #15 / 13 → cathegory of bolted joint D

„Normal bolts” (not preloaded), proposal: M24, class 6.8.

$$d = 24\ \text{mm}$$

$$A = 4{,}52\ \text{cm}^2$$

$$A_S = 3{,}53\ \text{cm}^2$$

$$d_0 = 26\ \text{mm}$$

$$f_{ub} = 600\ \text{MPa}$$

$$f_{yb} = 480\ \text{MPa}$$

Recommendation, diameter of bolts: function of thickness of elements

$$1{,}5\ t_{min} \leq d \leq 2{,}5\ t_{min}$$

$$\text{A, D:}\quad \Sigma t \leq 5d$$

$$\text{B, C, E:}\quad \Sigma t \leq 8d$$

$$f_{ub} > f_u$$

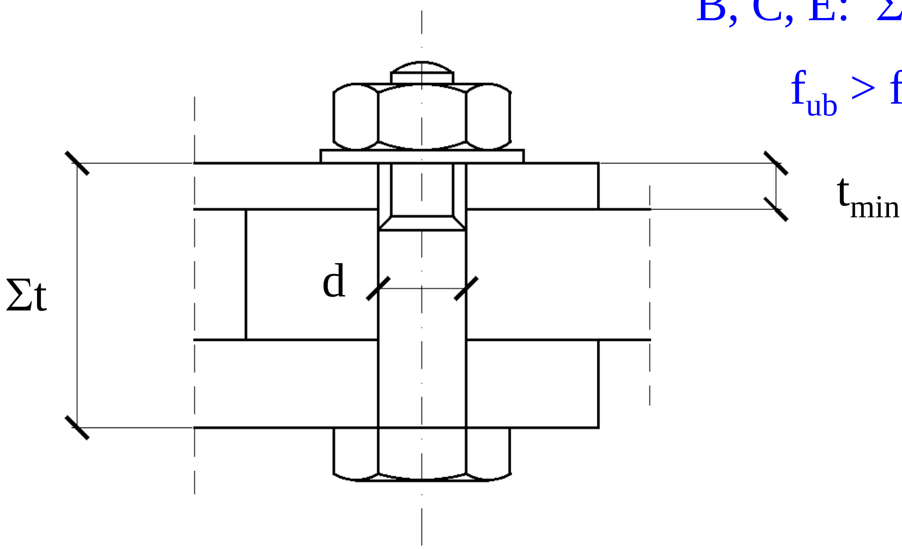
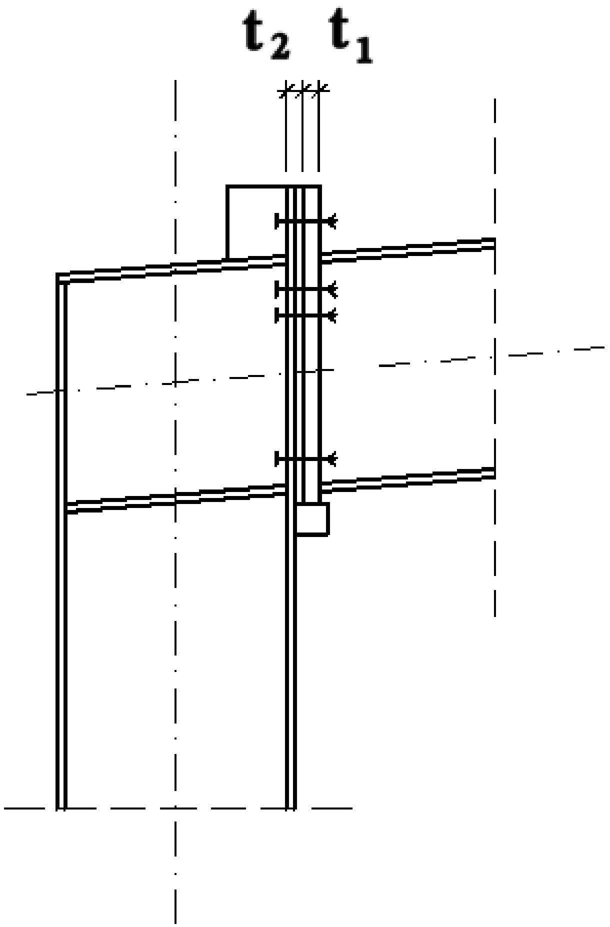


Photo: Author

Photo: parkersteel.co.uk

Recommendation type of bolt for category D: type SB with a thread along part of shank length according to EN 4017

Tension joint – thickness of end plate (t_1) and additional plate for column flange ($t_2 = t_{flange} + t_{additional\ plate}$)



$\min (t_1 ; t_2) \geq t$

→ #19 / 33

Bolted joint cathegory D:

$t = 2{,}0 \sqrt{\{c \, F_{t,Rd} / [f_y (2c + d)]\}}$

Bolted joint cathegory E:

$t = 1{,}25 \, d \, \sqrt[3]{\{f_{ub} / 1\,000 \, [\mathrm{MPa}]\}}$

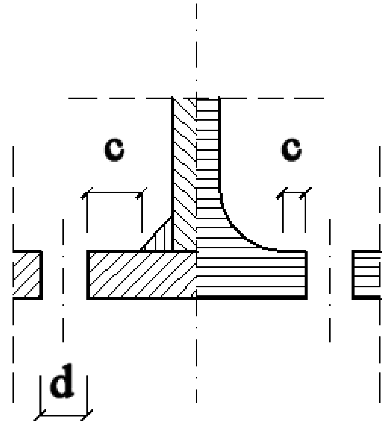


Photo: Author

Basic assuption: recommended value for distance between edge of cup and round part web-flange is minimum 5 mm. Real one: 8 mm.

16 mm – distance between edge of hole and round part web-flange → c

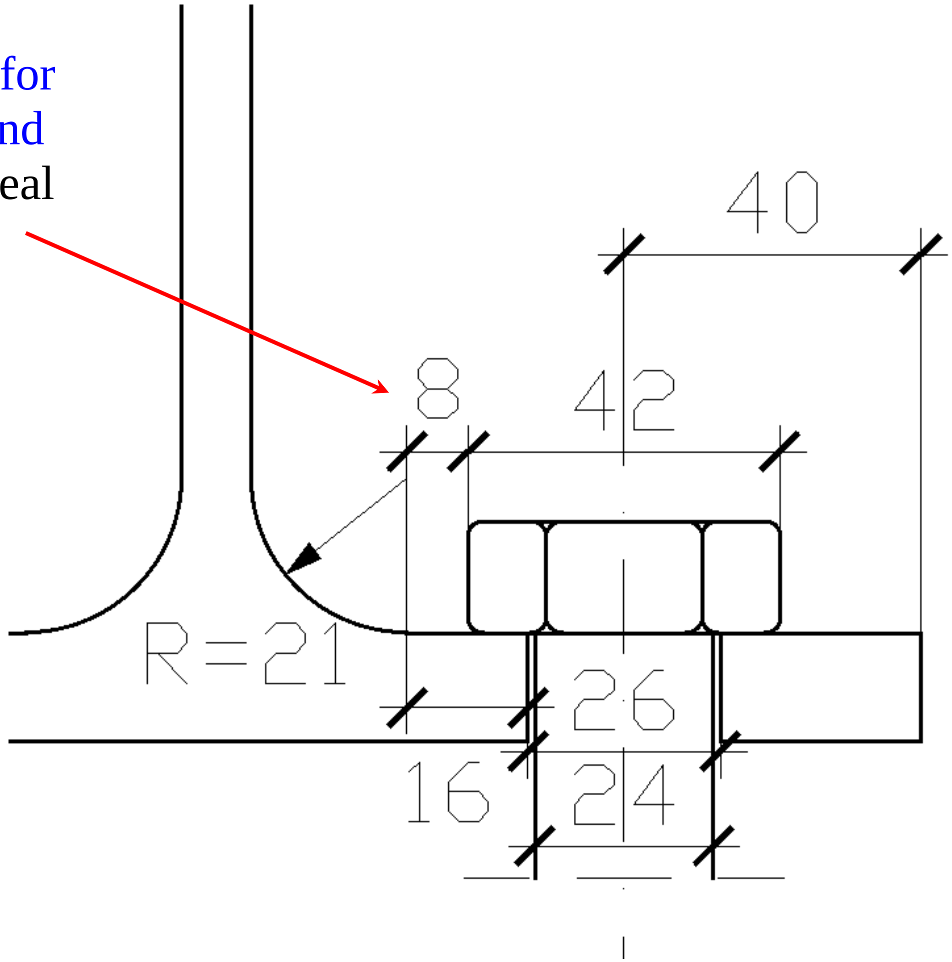


Photo: Author

Bolted joint cathegory D:

$t = 2{,}0 \sqrt{\{c\, F_{t,Rd} / [f_y\, (2c + d)]\}} = 27$ mm

Recommended thickness of end-plate and flange + plate = 27 mm

30 mm is assumed. To flange must be added plate 15 mm.

Column web 9,4 mm

Column flange 14,6 mm

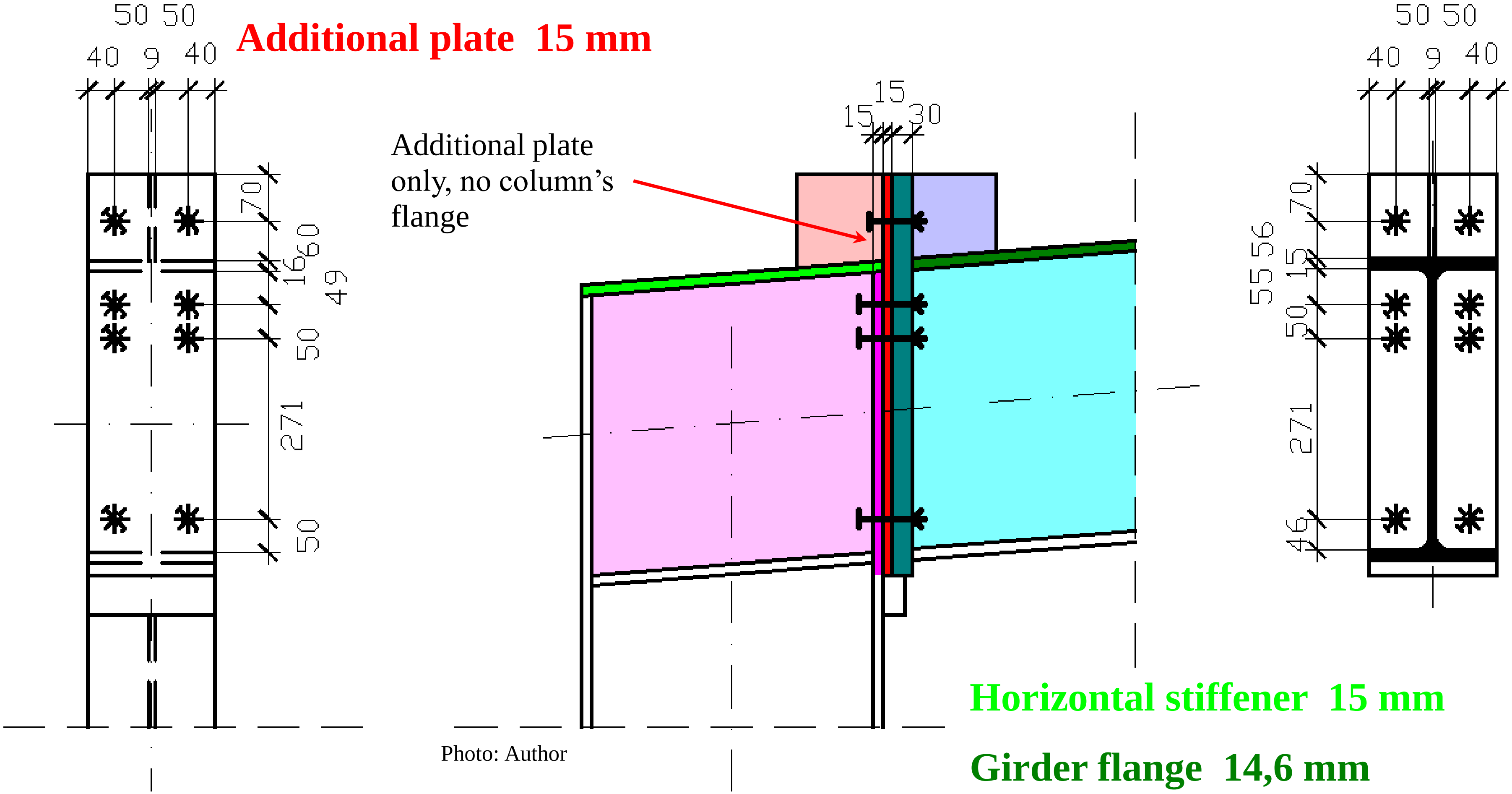
Vertical rib over column 10 mm

Additional plate 15 mm

Girder web 9,4 mm

Vertical rib over flange 10 mm

End plate 30 mm



Horizontal stiffener 15 mm

Girder flange 14,6 mm

Photo: Author

Checking dimensions according to Des # 2 examp 1 / 47:

e_1 = 70 mm

$p_{1,a}$ = 125 mm

$p_{1,b}$ = 50 mm

$p_{1,c}$ = 271 mm

e_2 = 40 mm

p_2 = 110 mm

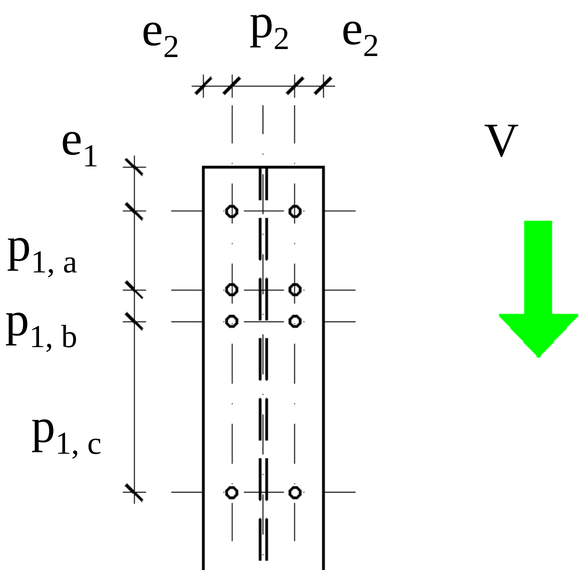


Photo: Author

$e_1 = 1{,}2\ d_0 \div 4\ t_{e,\ min} + 40\ mm =$

$= 32\ mm \div 160\ mm$ OK

$p_1 = 2{,}2\ d_0 \div \min(14\ t_{e,\ min}\ ;\ 200\ mm) =$

$= 58\ mm \div 200\ mm$

This is no „classical” shear joint, differences are accepted

$e_2 = 1{,}2\ d_0 \div 4\ t_{e,\ min} + 40\ mm =$

$= 32\ mm \div 160\ mm$ OK

$p_2 = 2{,}4\ d_0 \div \min(14\ t_{e,\ min}\ ;\ 200\ mm) =$

$= 63\ mm \div 200\ mm$ OK

Important notice: for various combinations of actions, there are possible different distributions of forces in top and bottom flanges.

→ #19 / 93

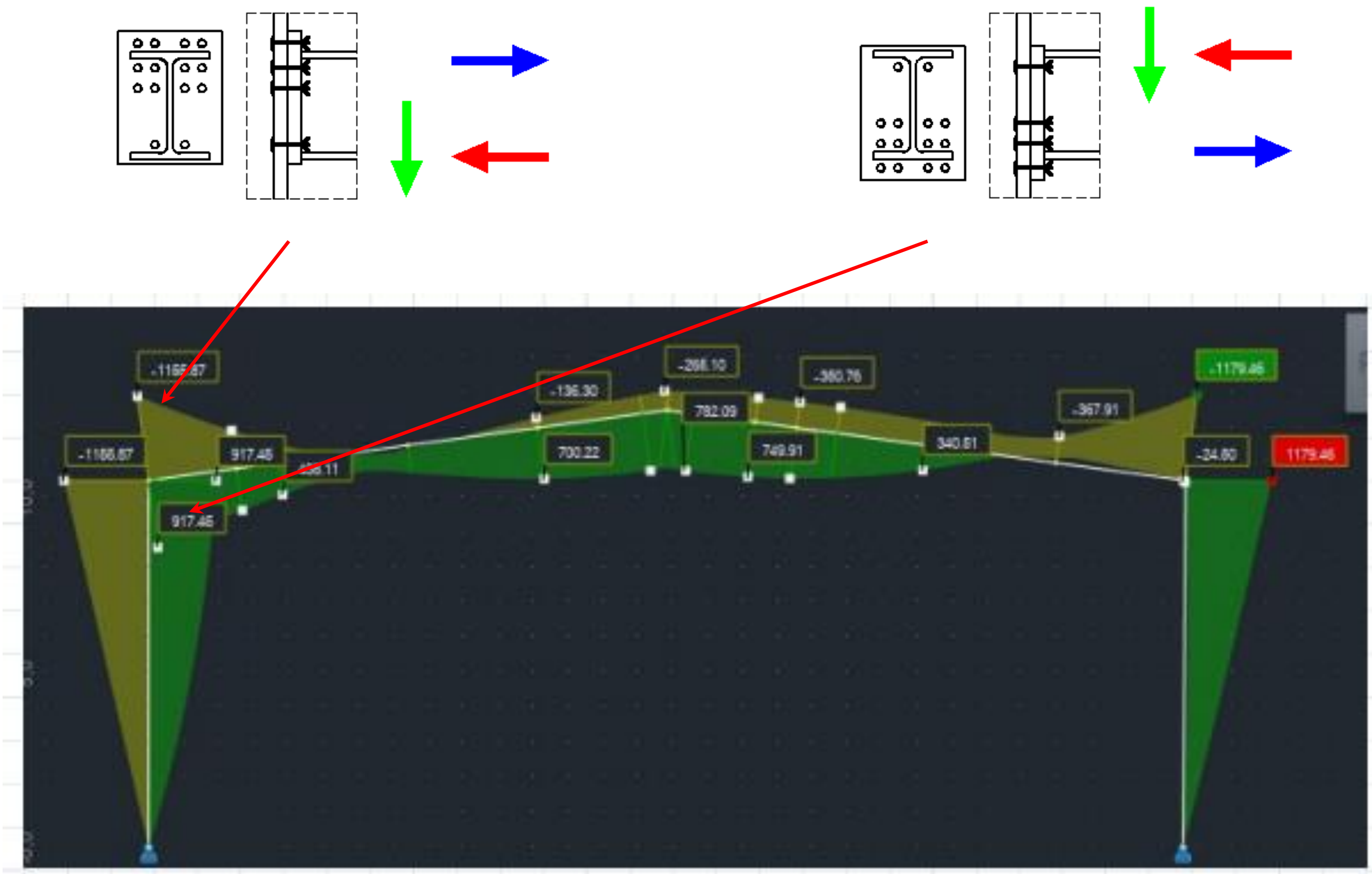


Photo: Author

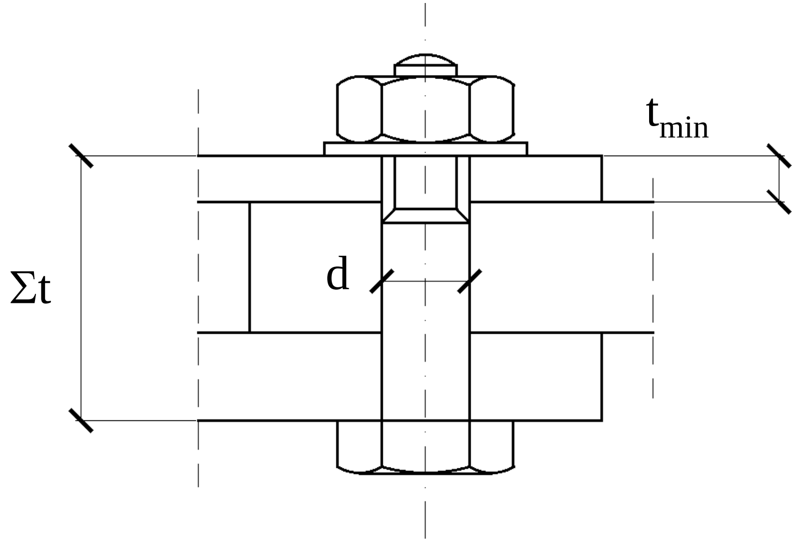


Photo: Author

For different load combinations it is possible that tension zone of joint may be at both the top and the bottom. Bolts in both parts must be identical.

t_{min} = 15 mm

Σt = 60 mm

$1,5\ t_{min} \leq d \leq 2,5\ t_{min}$ OK

D: $\Sigma t \leq 5d = 120$ mm OK

$f_{ub} = 600$ MPa $> f_u = (S235) = 360$ MPa OK

EN ISO 4014:

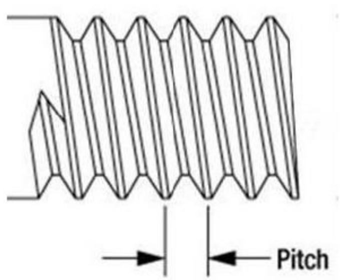


Photo: u-bolts-r-us.co.uk

Bolt	P [mm]
M16	2,0
M20	2,5
M24	3,0
M30	3,5

Length of bolt:

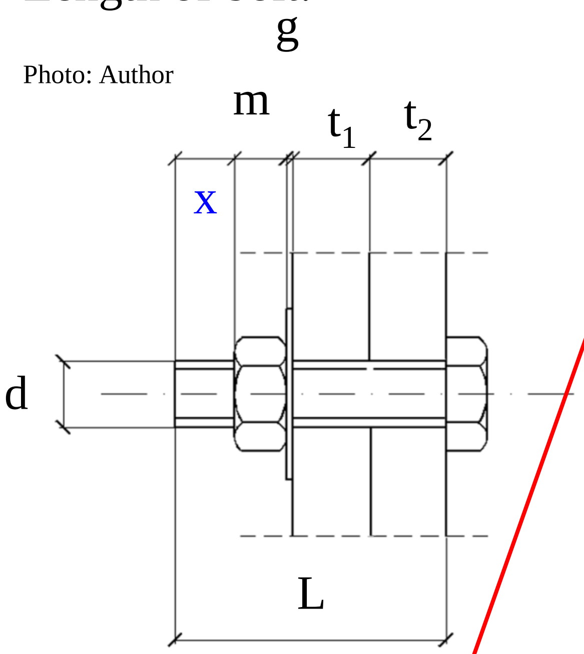


Photo: Author

t_1 = thickness of flange plate IPE 450 + thickness of additional plate; 15 + 15 = 30 mm;

t_2 = thickness of end plate, 30 mm;

additionally, gap for compensation of imperfections (fulfilled by pafking plate) must be taken into consideration, 10 mm

g = washer, 4,0 mm;

m = thickness of nut M24, 21 mm;

$P \leq x \leq d$, 3,0 ÷ 24 mm;

L = 15 + 15 + 30 + 10 + 4 + 21 + (3 ÷ 24) = 98 ÷ 119 mm

Column's position another than in project, bigger distance between both columns

L = 100 mm will be taken into consideration

Dimension (M)	Length (mm)	k (mm)	s (mm)	e (mm)	Weight (kg/1000 pc)
Search	Search	Search	Search	Search	Search
M20	130	12.5	30	32.95	340
M24	90	15	36	39.55	393
M24	100	15	36	39.55	423
M24	110	15	36	39.55	453
M24	120	15	36	39.55	483

Photo: rsgfasteners.com

423 kg / 1000 pcs

Welds:

$$0{,}2\ t_2 \leq a \leq 0{,}7\ t_1$$
$$t_2 \geq t_1$$

(#16 / 23)

Part	Sub-part	Thickness [mm]
Column	Flange	15
	Web	9
	Additional plate	15
	Horizontal stiffener	15
	Vertical rib	9
Girder	Flange	15
	Web	9
	End-plate	30
	Vertical rib	9

Part	Weld	Recommended thickness [mm]	Thickness taken into calculation [mm]
Column	Rib-plate	3 ÷ 6	6
	Rib-stiffener	3 ÷ 6	6
	Web-stiffener	3 ÷ 6	6
	Stiffener-plate	3 ÷ 10	10
	Stiffener-flange	3 ÷ 10	10
Girder	Rib-plate	6 ÷ 6	6
	Rib-flange	3 ÷ 6	6
	Web-plate	6 ÷ 6	6
	Flange-plate	6 ÷ 10	10

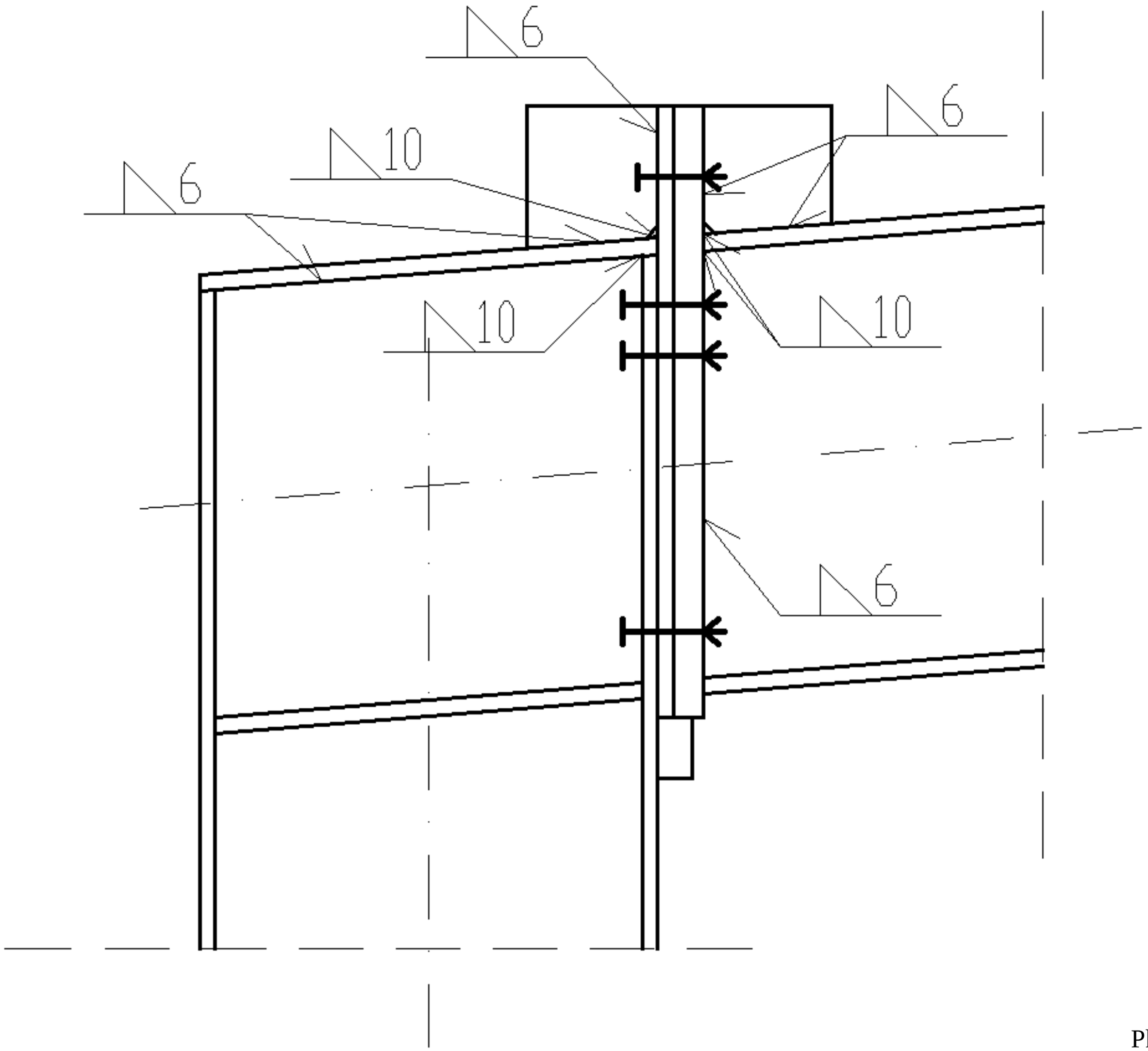
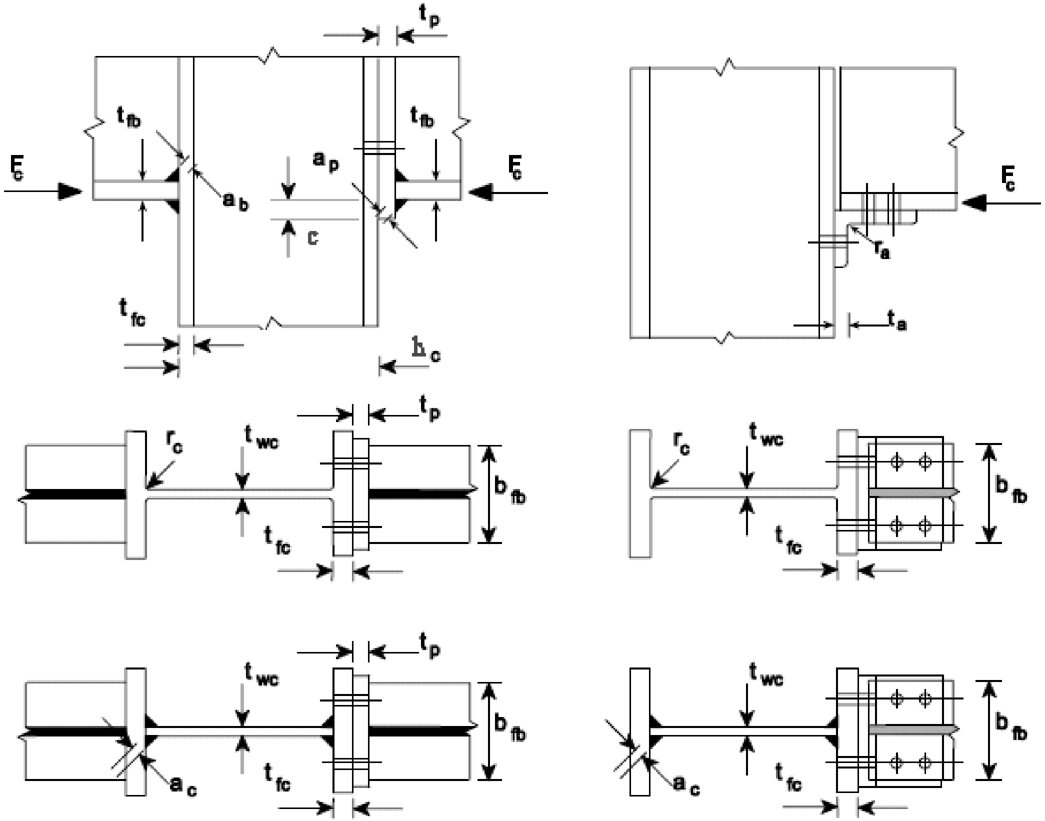


Photo: Author

Dimensions

→ #15 / 5



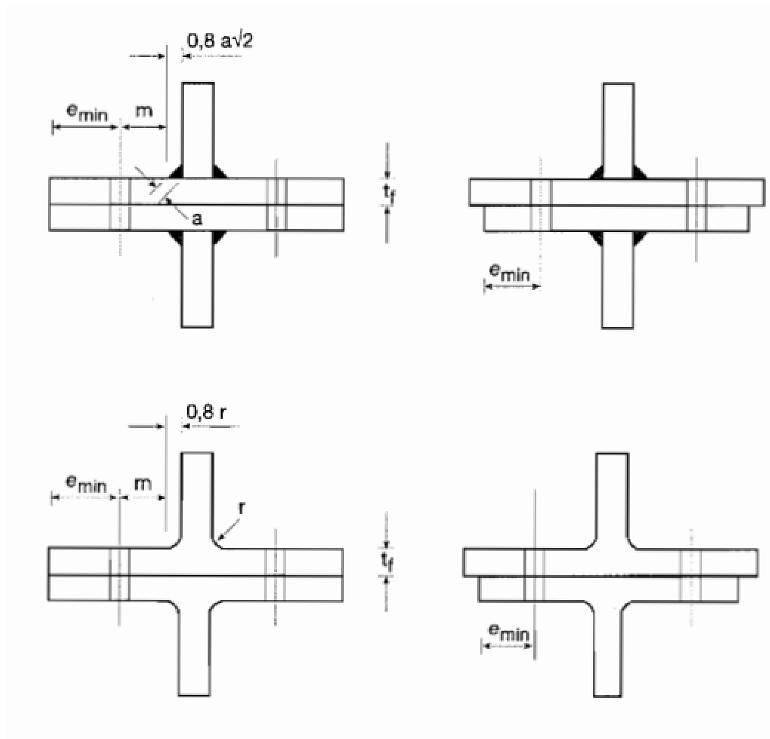
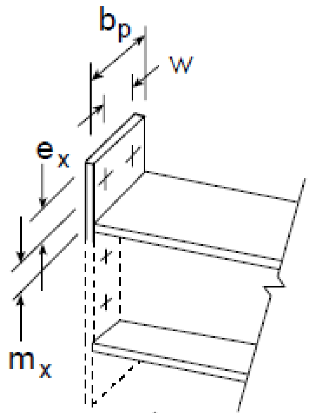
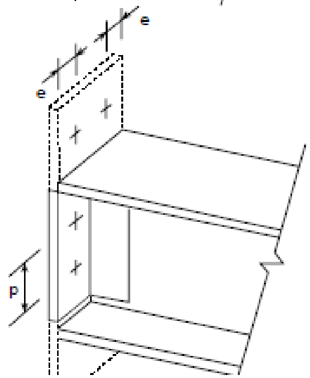
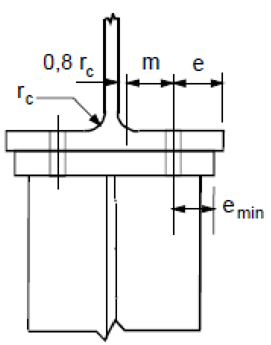


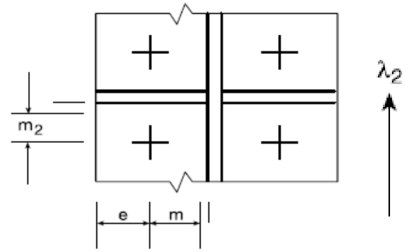
Photo: EN 1993-1-8 fig. 6.2 , fig. 6.6

Dimensions → #15 / 6



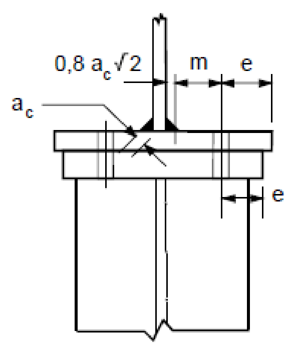






$$\lambda_1 = \frac{m}{m + e}$$

$$\lambda_2 = \frac{m_2}{m + e}$$



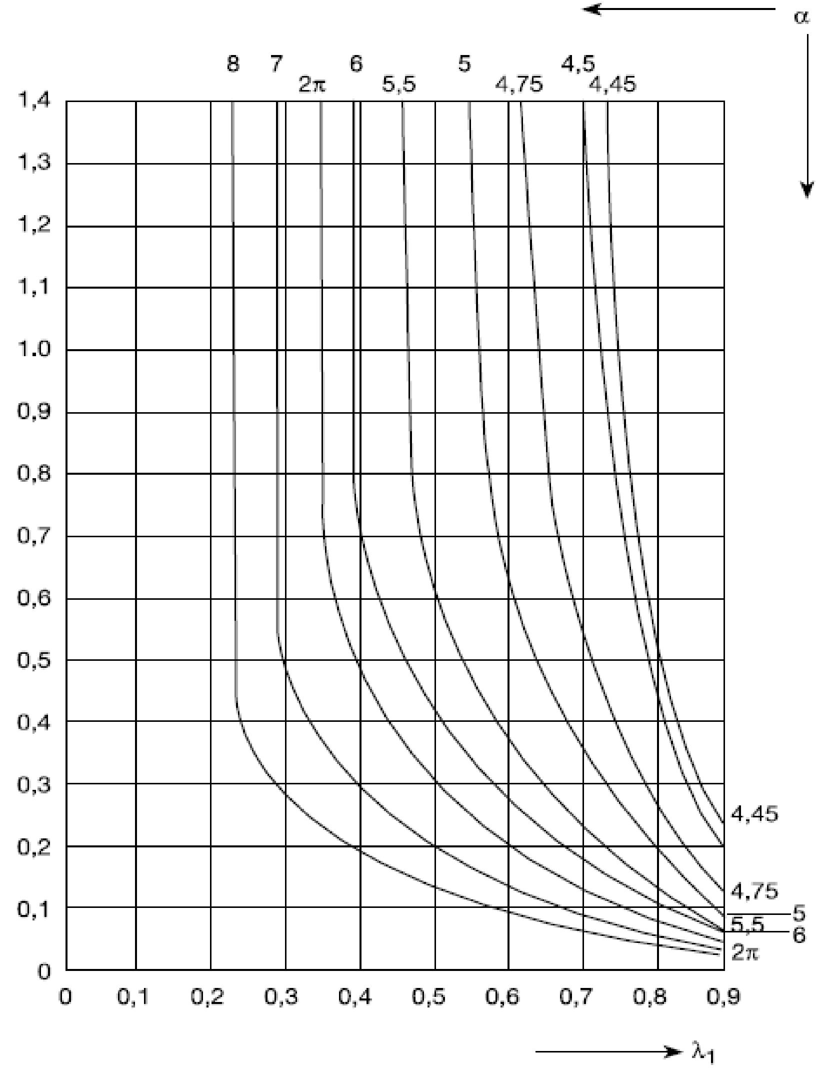
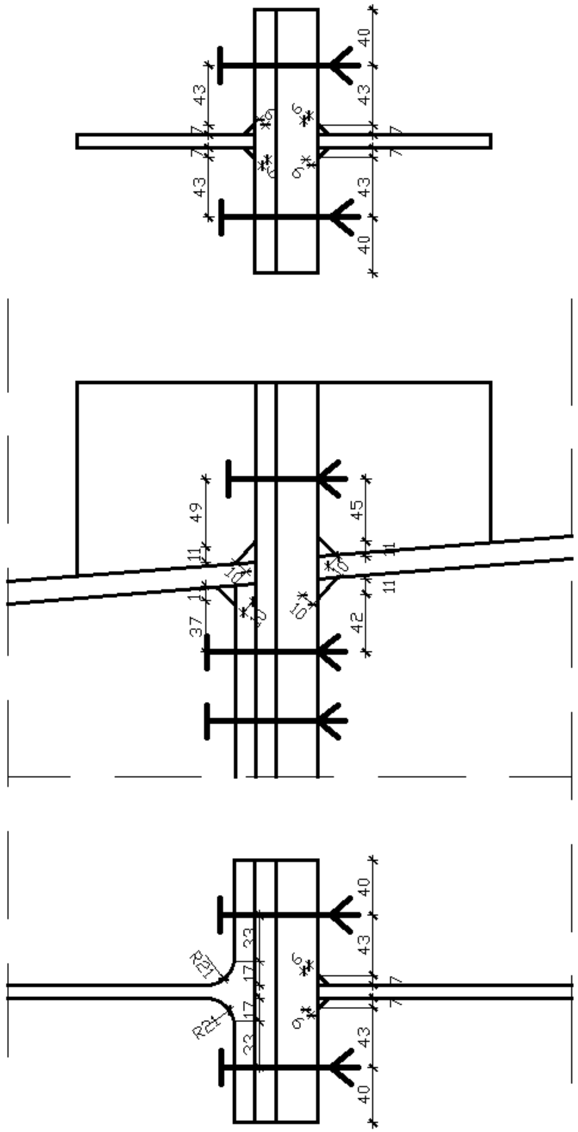


Photo: EN 1993-1-8 fig 6.8, 6.10. 6.11



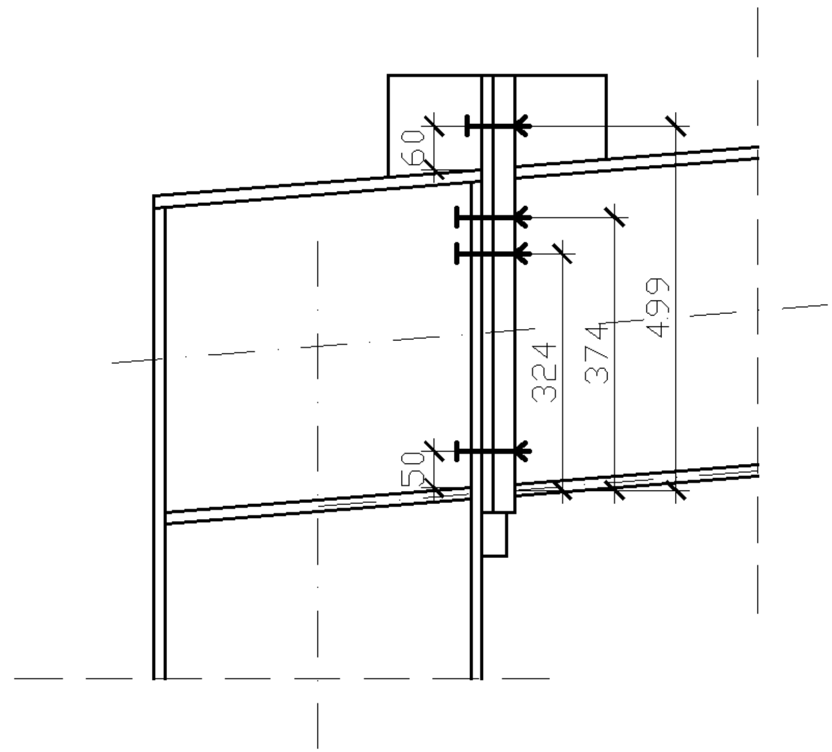


Photo: Author

Informal remarks on the effect of the reinforcing rib above the beam flange

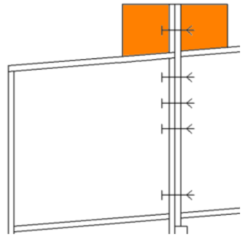
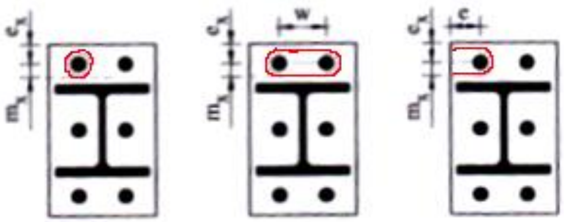


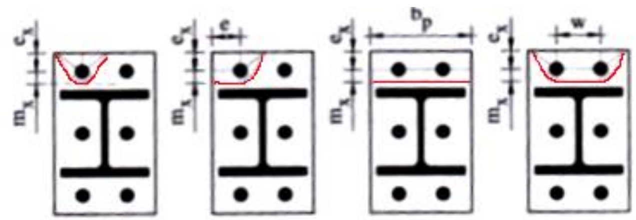
Photo: Author

Circular patterns



1 2 3

Non-circular patterns



1 2 3 4

Photo: J. Goczek, Ł. Supeł, M. Gajdzicki, Przykłady obliczeń konstrukcji stalowych, Politechnika Łódzka 2011

→ #15 / 26

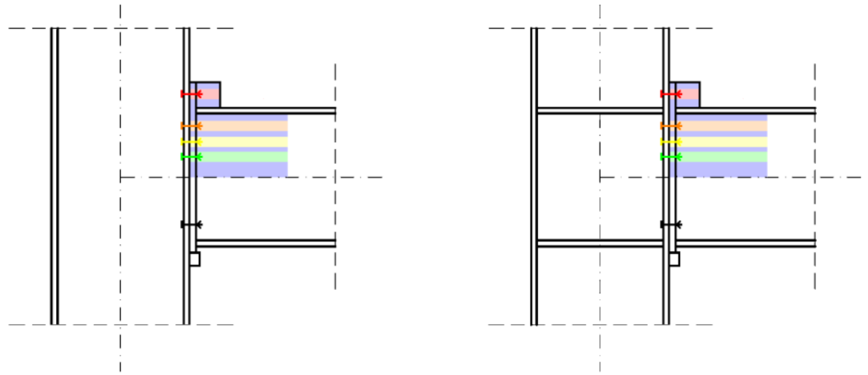
Formulas for effective lengths given, in the table, apply to specific failure mechanisms. Reinforcement of plate over beam flange by a vertical rib (→ #t / 4) will make **second** mechanism in circular (πm_x + w) and **third** + **fourth** mechanisms in non-circular ones (e +2m_x + 0,625e_x ; 0,5b_p) impossible (rib will prevent collapse involving both bolts at the same time). These formulas can be omitted from calculations.

End-plate / beam web

(Web stiffeners doesn't matter)

EN 1993-1-8 tab. 6.6

Photo: Author



Bolt-row location	Bolt-row considered individually		As part of a group of bolt-rows	
	Circular $l_{eff,\,cp}$	Non-circular $l_{eff,\,nc}$	Circular $\Sigma l_{eff,\,cp}$	Non-circular $\Sigma l_{eff,\,nc}$
Bolt-row outside tension flange of beam	min ($2\pi m_x$; $\pi m_x + w$; $\pi m_x + 2e$)	min ($4m_x + 1{,}25e_x$; $e + 2m_x + 0{,}625e_x$; $0{,}5b_p$; $0{,}5w + 2m_x + 0{,}625e_x$)	-	-
First bolt-row below tension flange of beam	$2\pi m$	αm	$\pi m + p$	$0{,}5p + \alpha m - 2m - 0{,}625e$
Other inner bolt-row	$2\pi m$	$4m + 1{,}25e$	$2p$	p
Other end row-bolt	$2\pi m$	$4m + 1{,}25e$	$\pi m + p$	$2m + 0{,}625e + 0{,}5p$

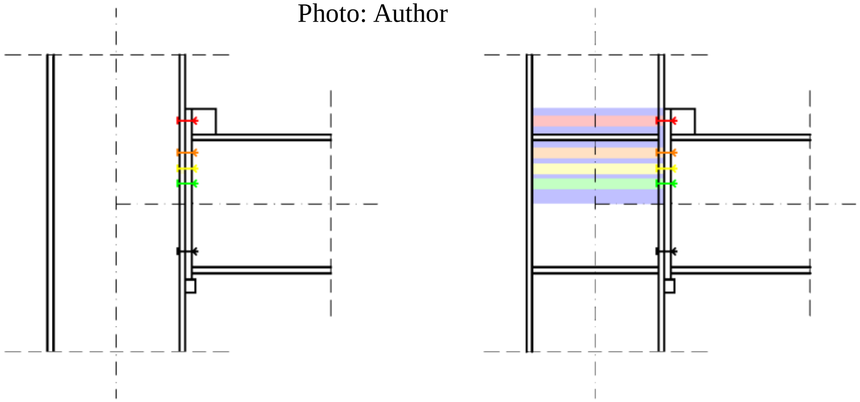
→ #15 / 23

→ #15 / 25

Stiffened column flange / stiffened column web

EN 1993-1-8 tab. 6.5

Photo: Author



Bolt-row location	Bolt-row considered individually		As part of a group of bolt-rows	
	Circular $l_{eff, cp}$	Non-circular $l_{eff, nc}$	Circular $\Sigma l_{eff, cp}$	Non-circular $\Sigma l_{eff, nc}$
End bolt-row adjacent to a stiffener	min ($2\pi m$; $\pi m + 2e_{1,1}$)	min ($e_{1,1} + \alpha m - 2m - 0{,}625e$; αm)	-	-
Bolt-row adjacent to a stiffener	$2\pi m$	αm	$\pi m + p$	$0{,}5p + \alpha m - 2m - 0{,}625e$
Other inner bolt-row	$2\pi m$	$4m + 1{,}25e$	$2p$	p
Other end bolt-row	min ($2\pi m$; $\pi m + 2e_{1,2}$)	min ($4m + 1{,}25e$; $2m + 0{,}625e + 2e_{1,2}$)	min ($\pi m + p$; $2e_{1,2} + p$)	min ($2m + 0{,}625e + 0{,}5p$; $e_{1,2} + 0{,}5p$)

Recommendation in literature

Type of bolts, according to #t / 19-20

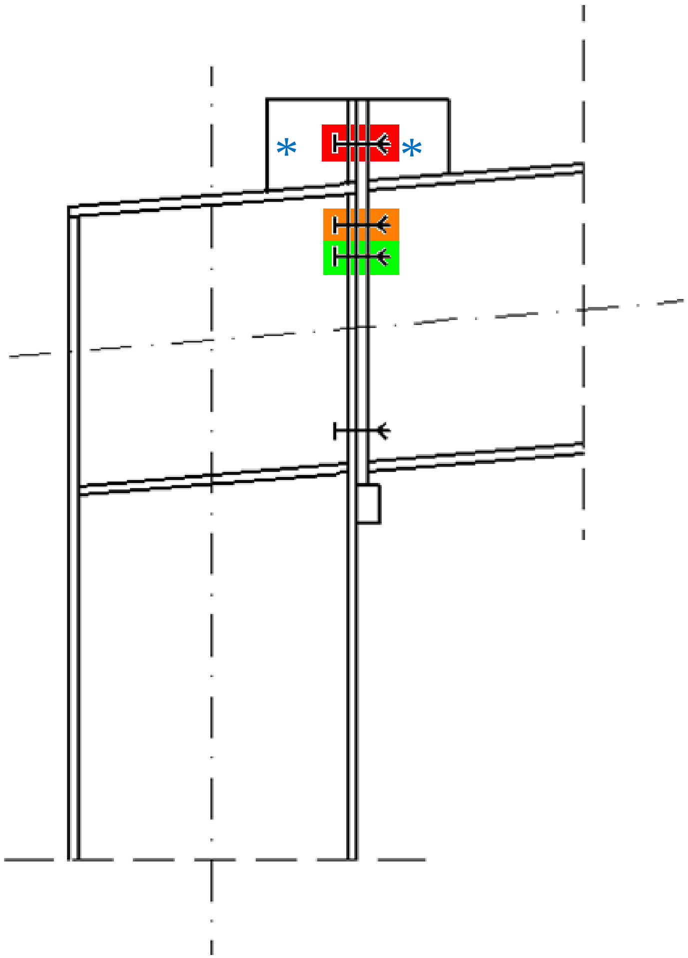


Photo: Author

First: row over flange of beam and horizontal stiffener on column, no cooperation with rest rows of bolt, influence of vertical rib;

Second: first row under flange and stiffener;

Third: other end row;

Column *	Girder *
$t_f = 15$ mm	$t_p = 30$ mm
$z = 499$ mm	$z = 499$ mm
$e_x = 70$ mm	$e_x = 70$ mm
$w = 110$ mm	$w = 110$ mm
$b_p = 190$ mm	$b_p = 190$ mm
$m = 43$ mm	$m = 43$ mm
$e = 40$ mm	$e = 40$ mm
$e_{min} = 40$ mm	$e_{min} = 40$ mm
$m_x = 49$ mm	$m_x = 45$ mm
$e_{1,1} = 60$ mm	
$e_{1,2} = 60$ mm	
$\lambda_1 = 0{,}518$	
$\lambda_2 = m_x / (m + e) = 0{,}590$	
$\alpha \approx 5{,}45$	

Column	Girder
t_f = 30 mm	t_p = 30 mm
z = 374 mm	z = 374 mm
w = 110 mm	w = 110 mm
b_p = 190 mm	b_p = 190 mm
m = 33 mm	m = 43 mm
e = 40 mm	e = 40 mm
e_{min} = 40 mm	e_{min} = 40 mm
p = 50 mm	p = 50 mm
m_2 = 37 mm	m_2 = 42 mm
λ_1 = 0,452	λ_1 = 0,452
λ_2 = 0,507	λ_2 = 0,575
$\alpha \approx 6{,}10$	$\alpha \approx 6{,}00$

Column	Girder
t_f = 30 mm	t_p = 30 mm
z = 324 mm	z = 324 mm
w = 110 mm	w = 110 mm
b_p = 190 mm	b_p = 190 mm
m = 33 mm	m = 43 mm
e = 40 mm	e = 40 mm
e_{min} = 40 mm	e_{min} = 40 mm
p = 50 mm	p = 50 mm
$e_{1,2}$ = 308 mm	

Stiffened column flange / stiffened column web

Bolt-row location	Bolt-row considered individually [mm]		As part of a group of bolt-rows [mm]	
	Circular $l_{eff, cp}$	Non-circular $l_{eff, nc}$	Circular $\Sigma l_{eff, cp}$	Non-circular $\Sigma l_{eff, nc}$
End bolt-row adjacent to a stiffener	min (**270** ; 275)	**193**	-	-
Bolt-row adjacent to a stiffener	**207**	**201**	**154**	**135**
Other end bolt-row	min (**207** ; 720)	min (**182** ; 707)	min (**154** ; 666)	min (**116** ; 333)

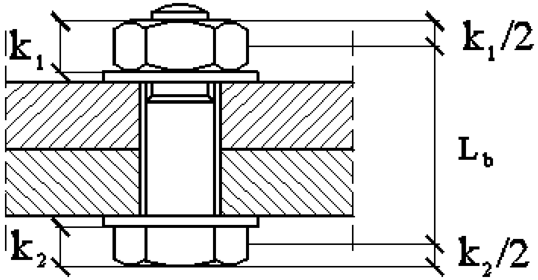
End-plate / beam web

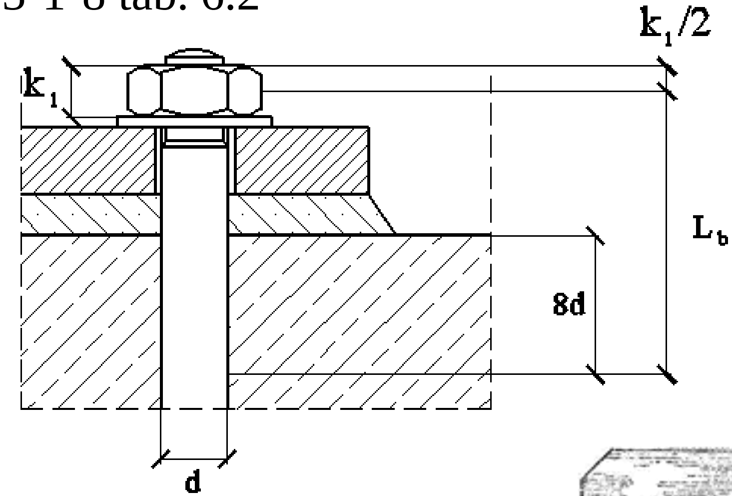
Bolt-row location	Bolt-row considered individually [mm]		As part of a group of bolt-rows [mm]	
	Circular $l_{eff, cp}$	Non-circular $l_{eff, nc}$	Circular $\Sigma l_{eff, cp}$	Non-circular $\Sigma l_{eff, nc}$
Bolt-row outside tension flange of beam	min (283 ; **221**)	min (268 ; **174**)	-	-
First bolt-row below tension flange of beam	**270**	**258**	**185**	**172**
Other inner bolt-row	**270**	**222**	**185**	**136**

Prying action - when it can occur?

Photo: Author

EN 1993-1-8 tab. 6.2





→ #15 / 18

$L_b \leq L_b^* \rightarrow$ Prying forces

$L_b > L_b^* \rightarrow$ No prying forces

$$L_b^* = 8{,}8\ m^3 A_s / (\Sigma l_{eff}\ t_f^3)$$

A_s – area of bolt cross-section in threated portion

t_f – the thickness of the thinnest plate

$m \rightarrow$ #t / 6

$\Sigma l_{eff} \rightarrow$ #t / 23 – 25, #t / 27

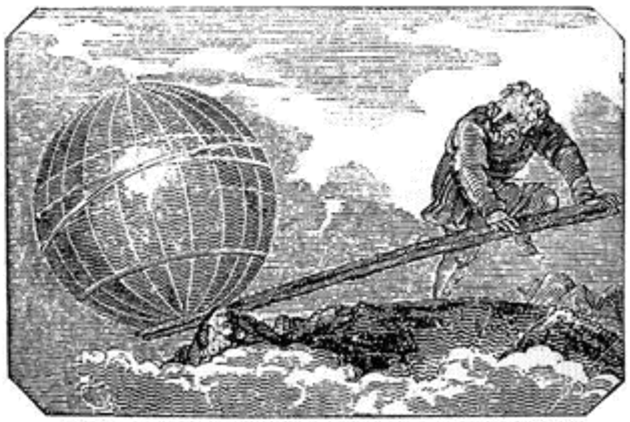


Photo: physics.weber.edu

Give me the place to stand, and I shall move the Earth

This point is an example of inconsequences and uncertainties in Eurocode.

There is no clear, what menas Σl_{eff}

Into calculation will be taken into consideration min ($\Sigma l_{eff, cp}$; $\Sigma l_{eff, nc}$)

L_b^* is function of Σl_{eff} , but for row of bolts over flange is no formulas for Σl_{eff}

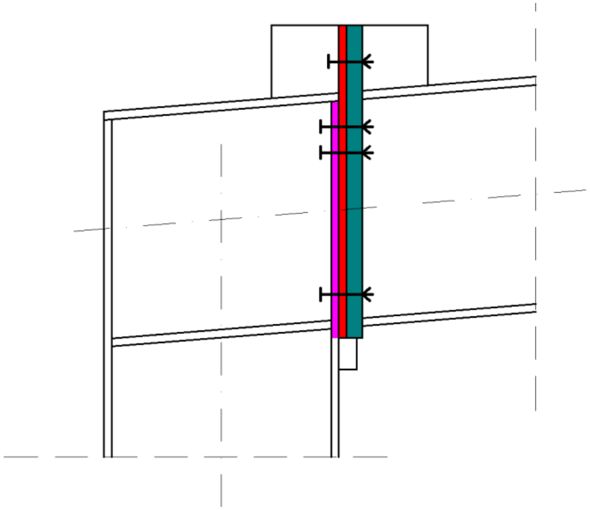
Into calculation is taken into consideration l_{eff} instead Σl_{eff}

Bolt-row location	Column		Endplate / beam	
	Circular $\Sigma l_{eff, cp}$	Non-circular $\Sigma l_{eff, nc}$	Circular $\Sigma l_{eff, cp}$	Non-circular $\Sigma l_{eff, nc}$
Bolt-row outside tension flange of beam	255	183	221	174
First bolt-row below tension flange of beam	154 + 154 = **308**	135 + 116 = **251**	185 + 100 = **285**	172 + 50 = **222**
Other inner bolt-row	154 + 154 = **308**	135 + 116 = **251**	185 + 100 = **285**	172 + 50 = **222**

Σl_{eff} means min (circular ; non-circular).

Bolt-row location	Column		Girder	
	L_b^* [mm]	L_b [mm]	L_b^* [mm]	L_b [mm]
(red)	379	62	53	62
(orange)	31	77	53	77
(green)	36	77	67	77

Photo: Author

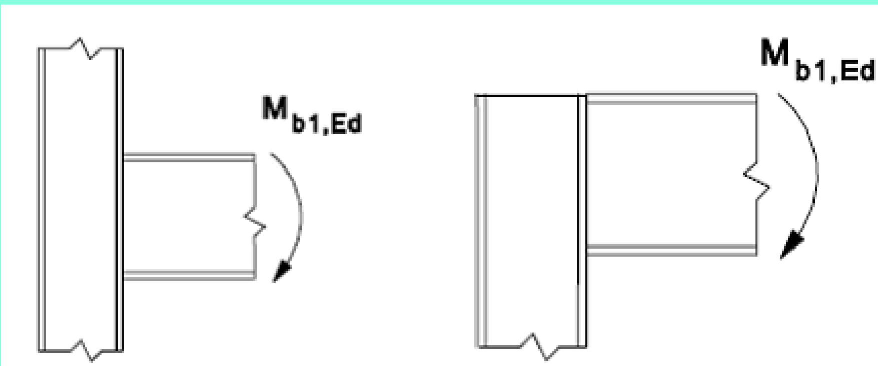
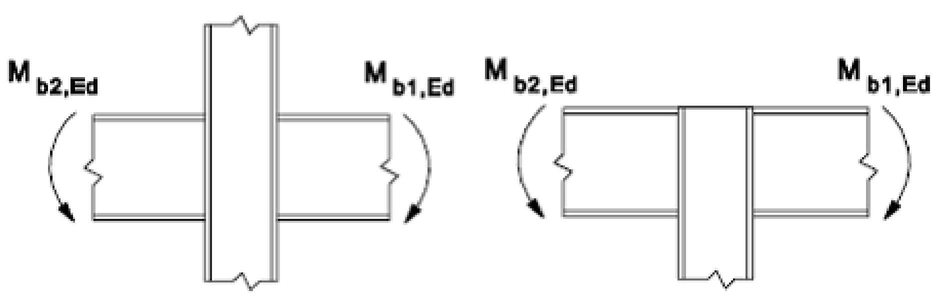


First row: only additional plate, no column flange. Thickness is only 15 mm, no 30 mm as for rest rows. Due to, impact of prying actions must be taken into consideration for I[st] row on column's flange.

Beam-column geometry

→ #15 / 10

EN 1993-1-8 tab. 5.4

Type of joint configuration	Action	Ttransformation parameter β
	$M_{b1,\ Ed}$	$\beta \approx 1$
	$M_{b1,\ Ed} = M_{b2,\ Ed}$	$\beta = 0$ *)
	$M_{b1,\ Ed} = M_{b2,\ Ed} > 0{,}0$	$\beta \approx 1$
	$M_{b1,\ Ed} = M_{b2,\ Ed} < 0{,}0$	$\beta \approx 2$
	$M_{b1,\ Ed} + M_{b2,\ Ed} = 0{,}0$	

*) in this cane the value of β is the exact value rater than an approximation

Effective areas in compressed part of column web

EN 1993-1-8 6.2.6.2

→ #15 / 28

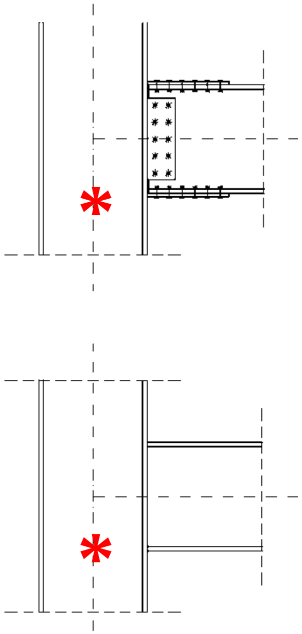
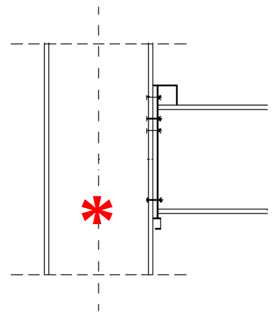
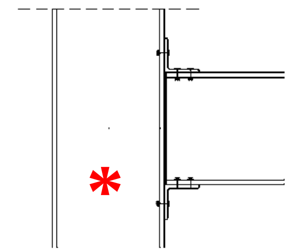
			
$b_{eff,\,c,\,wc}$	$t_{fb} + 2\sqrt{2}\,a_b + 5(t_{fc} + s)$	$t_{fb} + 2\sqrt{2}\,a_p + 5(t_{fc} + s) + s_p$	$2t_a + 0{,}6\,r_a + 5(t_{fc} + s)$

Photo: Author

Column:	s_p	s	d_{wc}
Welded I-beam	$\min(t_p + c\ ;\ 2t_p)$	$\sqrt{2}\, a_c$	$h_c - 2(t_{fc} + \sqrt{2}\, a_c)$
Hot rolled I-beam		r_c	$h_c - 2(t_{fc} + r_c)$

c – length of end-plate out off bottom flange of beam

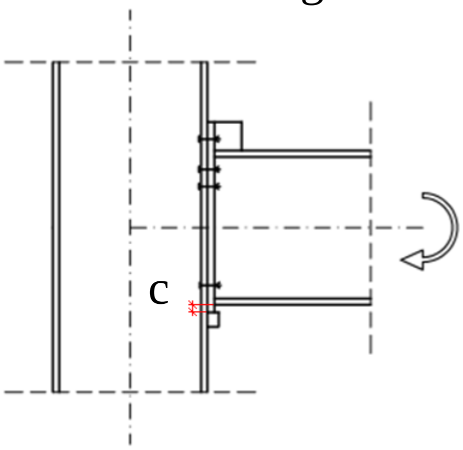


Photo: Author

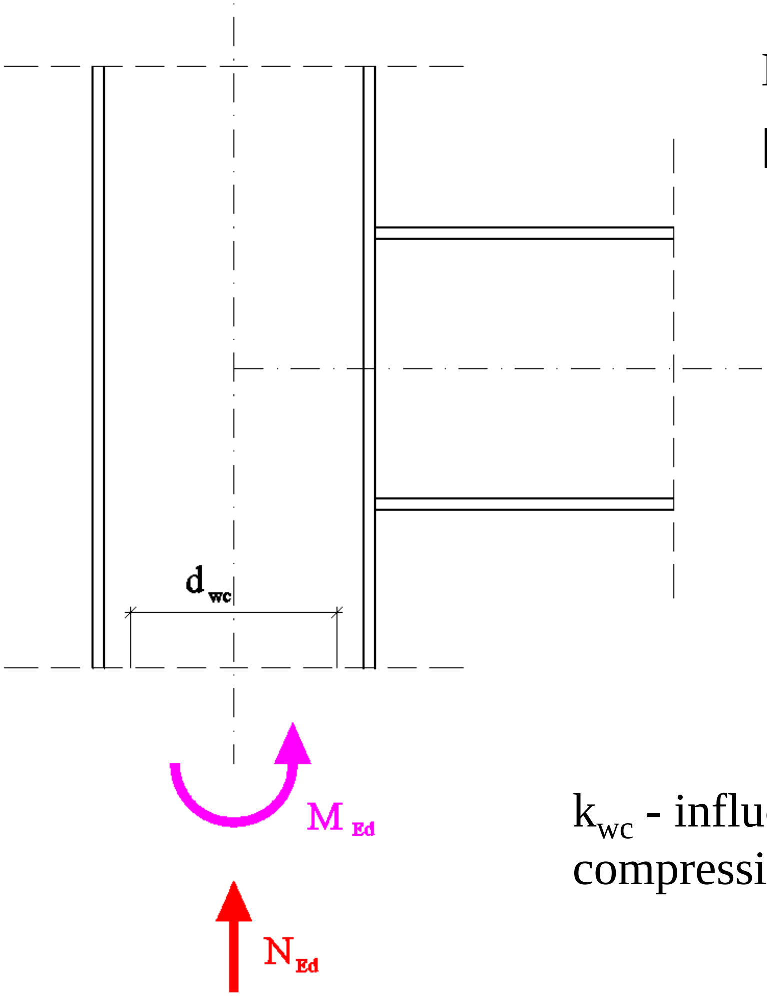
EN 1993-1-8 6.2.6.2

$\overline{\lambda}_p$	ρ
$\leq 0{,}72$	1,0
$> 0{,}72$	$(\overline{\lambda}_p - 0{,}2) / (\overline{\lambda}_p)^2$

$$\overline{l}_p = 0{,}932 \sqrt{[\ (b_{eff,\ c,\ wc}\ d_{wc}\ f_{y,\ wc}) / (E\ t_{wc}^{\ 2})\ ]}$$

ρ - simplified calculation of instability in compressed part of web - without full calculation of instability factor (χ), only reduction factor.

Effective areas in compressed part



Max compression for plane part of web (d_{dwc}):

$[\ s\ (N_{ed} + M_{Ed})\ ]_{dwc} = s_{com,\ Ed}$

$\sigma_{com,\ Ed}$ / $f_{y,\ wc}$	k_{wc}
$\leq 0{,}7$	1,0
$> 0{,}7$	$1{,}\ 7 - \sigma_{com,\ Ed} / f_{y,\ wc}$

EN 1993-1-8 6.2.6.2

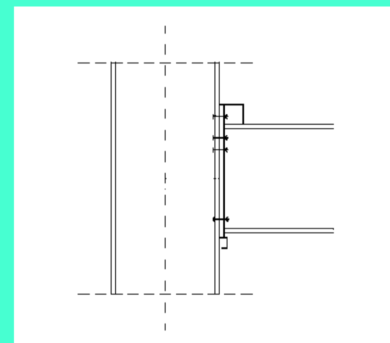
k_{wc} - influence of double - transverse and longitudinal - compression of the column flange

Photo: Author

Effective areas in compressed part

Photo: Author

EN 1993-1-8 6.2.6.2

			
$b_{eff,\,c,\,wc}$	$t_{fb} + 2\sqrt{2}\,a_b + 5(t_{fc} + s)$	$t_{fb} + 2\sqrt{2}\,a_p + 5(t_{fc} + s) + s_p$	$2t_a + 0{,}6\,r_a + 5(t_{fc} + s)$

Column:	s_p	s	d_{wc}
Welded I-beam	$\min\,(t_p + c\ ;\ 2t_p)$	$\sqrt{2}\,a_c$	$h_c - 2(t_{fc} + \sqrt{2}\,a_c)$
Hot rolled I-beam		r_c	$h_c - 2(t_{fc} + r_c)$

$s_p = 46$ mm

$s = 21$ mm

$d_{wc} = 379$ mm

$\mathbf{b_{eff,\,c,\,wc} = 267}$ **mm**

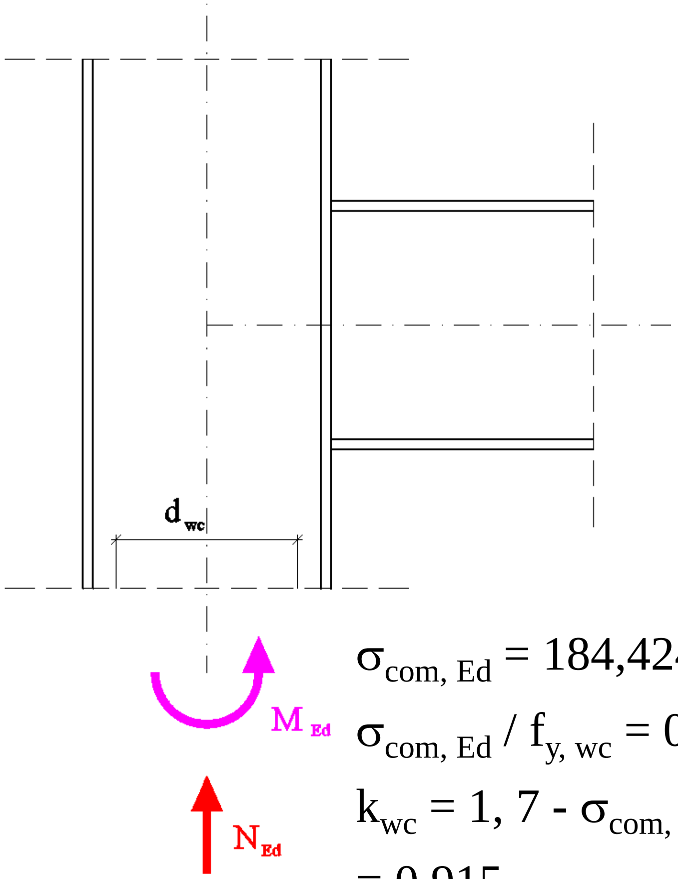
EN 1993-1-8 tab. 6.3

β	ω
$0{,}0 \leq \beta \leq 0{,}5$	$\omega = 1{,}0$
$0{,}5 \leq \beta < 1{,}0$	$\omega = \omega_1 + 2(1 - \beta)(1 - \omega_1)$
$\beta = 1{,}0$	$\omega = \omega_1$
$1{,}0 < \beta < 2{,}0$	$\omega = \omega_1 + 2(1 - \beta)(\omega_2 - \omega_1)$
$\beta = 2{,}0$	$\omega = \omega_2$

$$\omega = 1 / \sqrt{[1 + 1{,}3(b_{eff,\,c,\,wc}\ t_{wc} / A_{vc})^2]} = 0{,}686$$

Effective areas in compressed part

Photo: Author



EN 1993-1-8 6.2.6.2

Max compression for plane part of web (d_{dwc}):

$[\, \sigma\,(N_{Ed} + M_{Ed})\,]_{dwc} = \sigma_{com,\,Ed}$

$\sigma_{com,\,Ed} / f_{y,\,wc}$	k_{wc}
$\leq 0{,}7$	1,0
$> 0{,}7$	$1{,}7 - \sigma_{com,\,Ed} / f_{y,\,wc}$

$\sigma_{com,\,Ed} = 184{,}424$ Mpa

$\sigma_{com,\,Ed} / f_{y,\,wc} = 0{,}785$

$k_{wc} = 1{,}7 - \sigma_{com,\,Ed} / f_{y,\,wc} = 0{,}915$

λ_p	ρ
$\leq 0{,}72$	1,0
$> 0{,}72$	$(\lambda_p - 0{,}2) / (\lambda_p)^2$

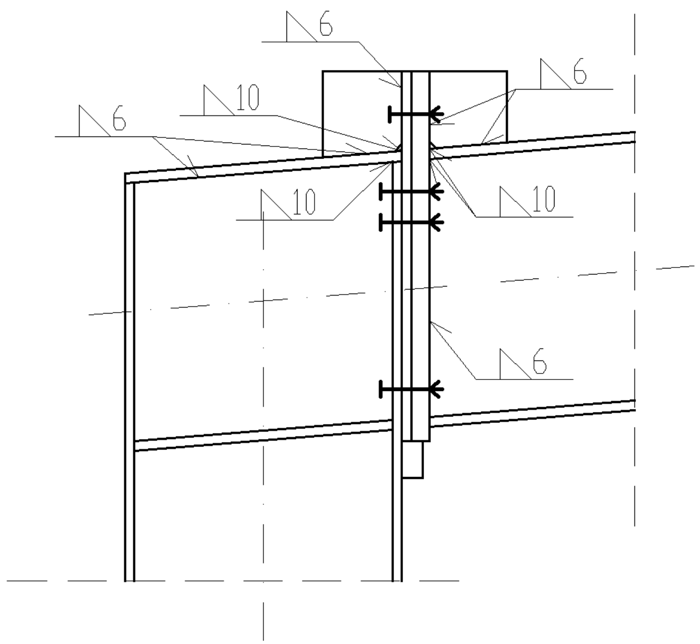
$\lambda_p = 0{,}932 \sqrt{[\, (b_{eff,\,c,\,wc}\, d_{wc}\, f_{y,\,wc}) / (E\, t_{wc}^2)\,]} = 1{,}055$

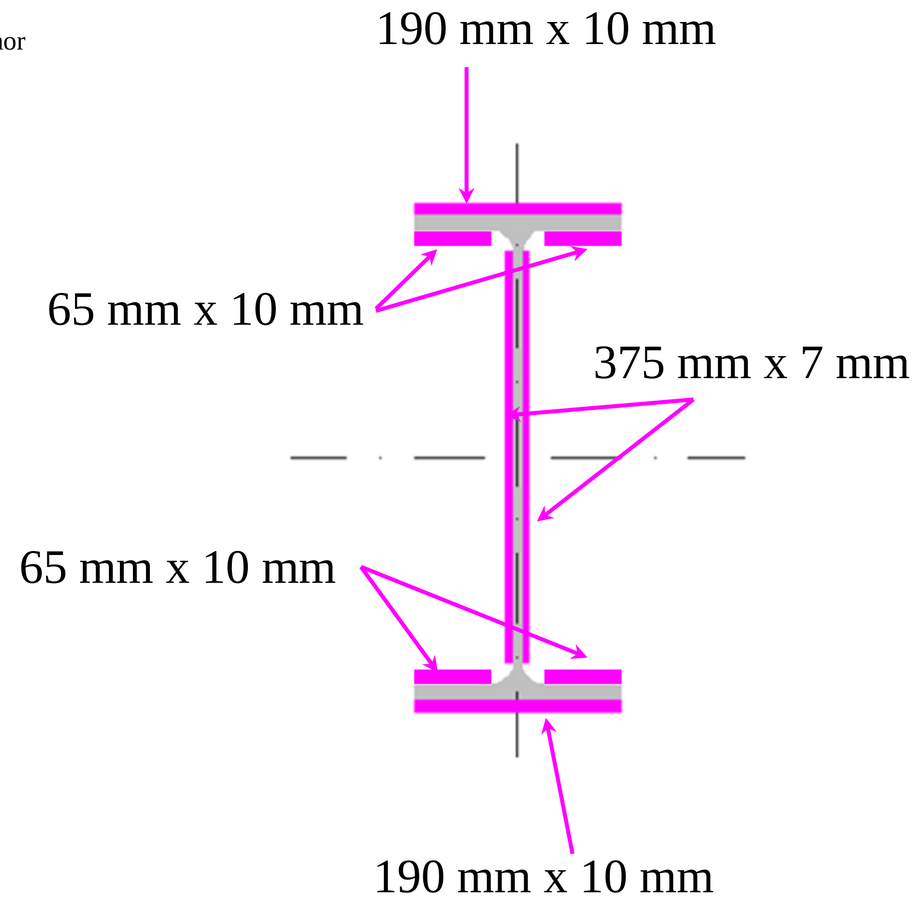
$\rho = 0{,}768$

Welds end-plate - girder

Welds between roof girder and end plate – according example presented on Lec #17 / 25

Photo: Author





$$A = 2 \cdot 1{,}0 \cdot 19{,}0 + 2 \cdot 0{,}6 \cdot 37{,}5 + 4 \cdot 1{,}0 \cdot 6{,}5 = 109{,}0\ cm^2$$

$$A_V = 2 \cdot 0{,}6 \cdot 37{,}5 = 45{,}0\ cm^2$$

$$J_y = 2 \cdot 0{,}6 \cdot 37{,}5^3 / 12 + 2 \cdot 1{,}0 \cdot 19{,}0 \cdot (45 / 2 + 0{,}5)^2 +$$

$$+ 4 \cdot 1{,}0 \cdot 6{,}5 \cdot (45 / 2 - 1{,}46 - 0{,}5)^2 = 36\ 344\ cm^4$$

$$W_{y,\ max} = J_y / (45 / 2 + 1{,}0) = 1\ 546{,}55\ cm^3$$

$$W_{y,\ web} = J_y / (37{,}5 / 2) = 1\ 938{,}347\ cm^3$$

$M_{Ed} = 311{,}6\ kNm$ $N_{Ed,\ girder} = 63{,}9\ kN$ $V_{Ed,\ girder} = 80{,}5\ kN$

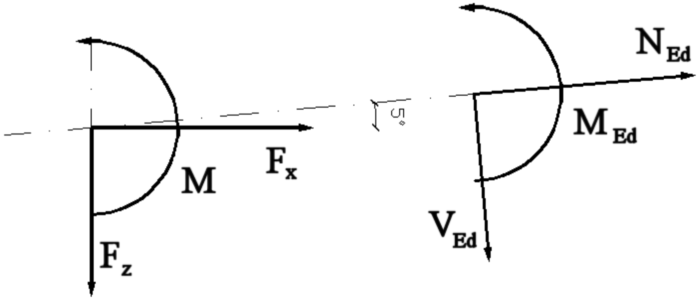


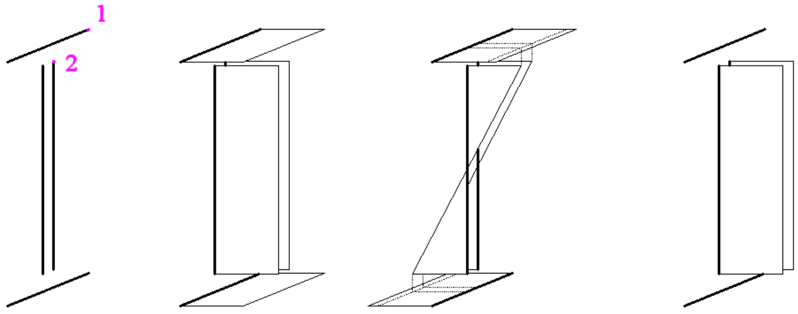
Photo: Author

$M_{Ed} = 311{,}6\ kNm$

$N_{Ed} = 70{,}673\ kN$

$V_{Ed} = 74{,}622\ kN$

Photo: Author



$$\sigma_1 (N_{Ed}) = \sigma_2 (N_{Ed}) = N_{Ed} / A = 6{,}484 \text{ MPa}$$

$$\sigma_1 (M_{Ed}) = M_{Ed} / W_{y,\, max} = 201{,}481 \text{ MPa}$$

$$\sigma_1 (M_{Ed}) = M_{Ed} / W_{y,\, web} = 160{,}755 \text{ MPa}$$

$$\tau_1 (V_{Ed}) = 0{,}000 \text{ MPa}$$

$$\tau_2 (V_{Ed}) = V_{Ed} / A_V = 16{,}583 \text{ MPa}$$

$$\sigma_1 = \sigma_1 (N_{Ed}) + \sigma_1 (M_{Ed}) = 207{,}965 \text{ MPa}$$

$$\sigma_2 = \sigma_2 (N_{Ed}) + \sigma_1 (M_{Ed}) = 167{,}239 \text{ MPa}$$

$$\tau_1 = 0{,}000 \text{ MPa}$$

$$\tau_2 = 16{,}583 \text{ MPa}$$

$$\sigma_{\perp 1} = \tau_{\perp 1} = \sigma_1 / \sqrt{2} = 147{,}053 \text{ MPa}$$

$$\sigma_{\perp 2} = \tau_{\perp 2} = \sigma_2 / \sqrt{2} = 118{,}256 \text{ MPa}$$

$$\tau_{\parallel 1} = \tau_1 = 0{,}000 \text{ MPa}$$

$$\tau_{\parallel 2} = \tau_2 = 16{,}583 \text{ MPa}$$

$$f_u / (\beta_w \gamma_{M2}) = 360,000 \text{ MPa}$$
$$0,9 f_u / \gamma_{M2} = 259,200 \text{ MPa}$$

For point 1

Condition 1: $\sqrt{[(\sigma_{\perp 1})^2 + 3(\tau_{\parallel 1}{}^2 + \tau_{\perp 1}{}^2)]} = 207,964 \text{ MPa} < 360,000 \text{ MPa}$

Condition 2: $\sigma_{\perp 1} = 147,053 \text{ MPa} < 259,200 \text{ MPa}$

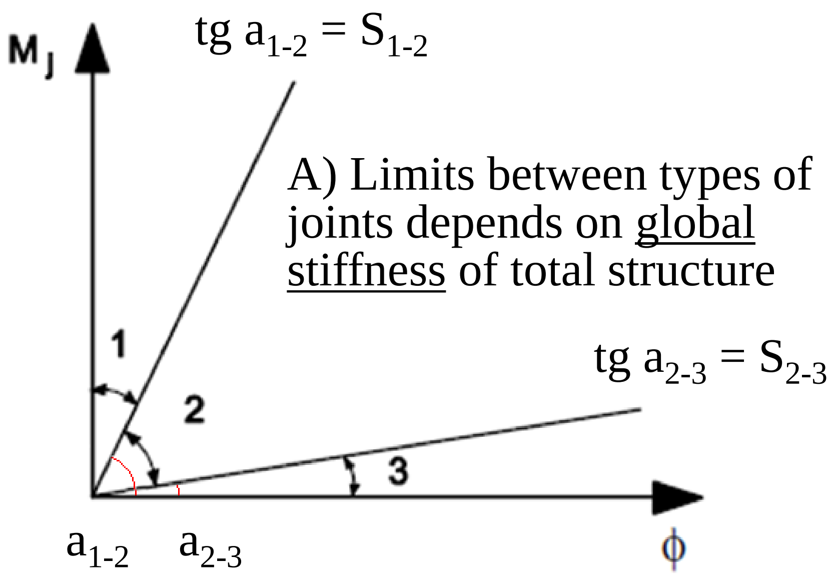
For point 2

Condition 1: $\sqrt{[(\sigma_{\perp 2})^2 + 3(\tau_{\parallel 2}{}^2 + \tau_{\perp 2}{}^2)]} = 169,688 \text{ MPa} < 360,000 \text{ MPa}$

Condition 2: $\sigma_{\perp 2} = 118,256 \text{ MPa} < 259,200 \text{ MPa}$

Stiffness of joint

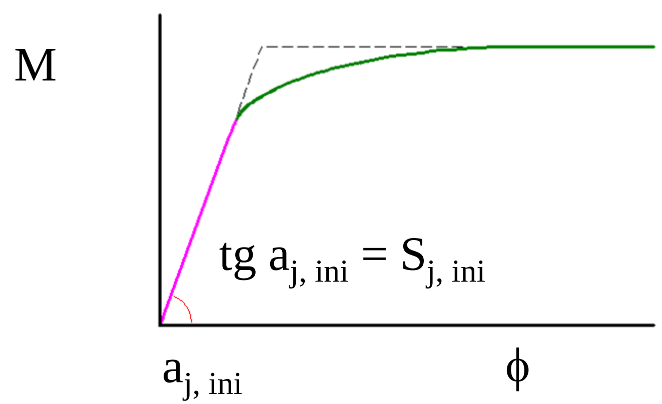
Proportion between local and global stiffness of joint – checking, if join is rigid.



A) Limits between types of joints depends on <u>global stiffness</u> of total structure

Photo: EN 1993-1-8 fig 5.4

Photo: Author



B) Characteristic of joint depends on <u>local stiffness</u> of sub-parts of joint

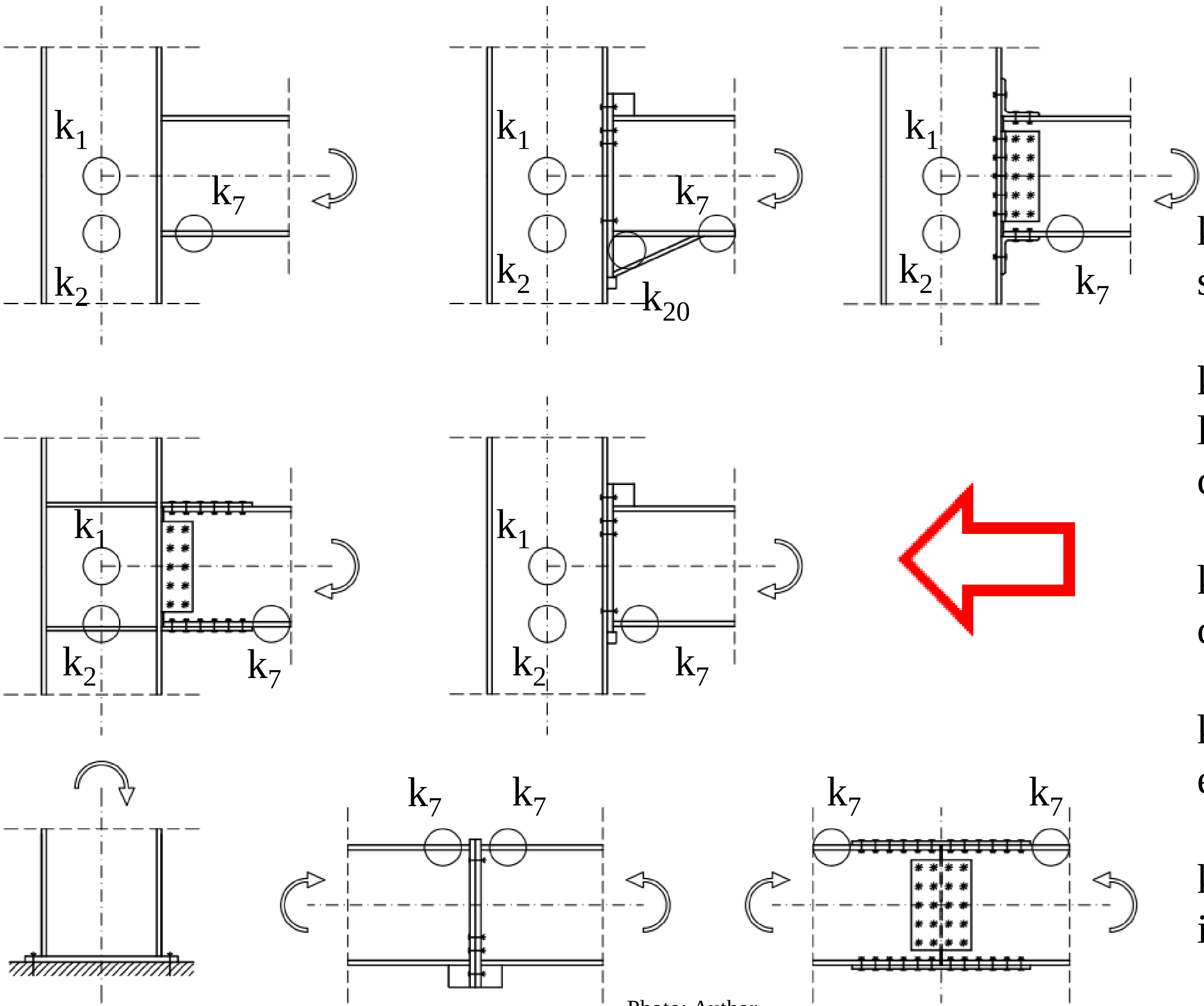


Photo: Author

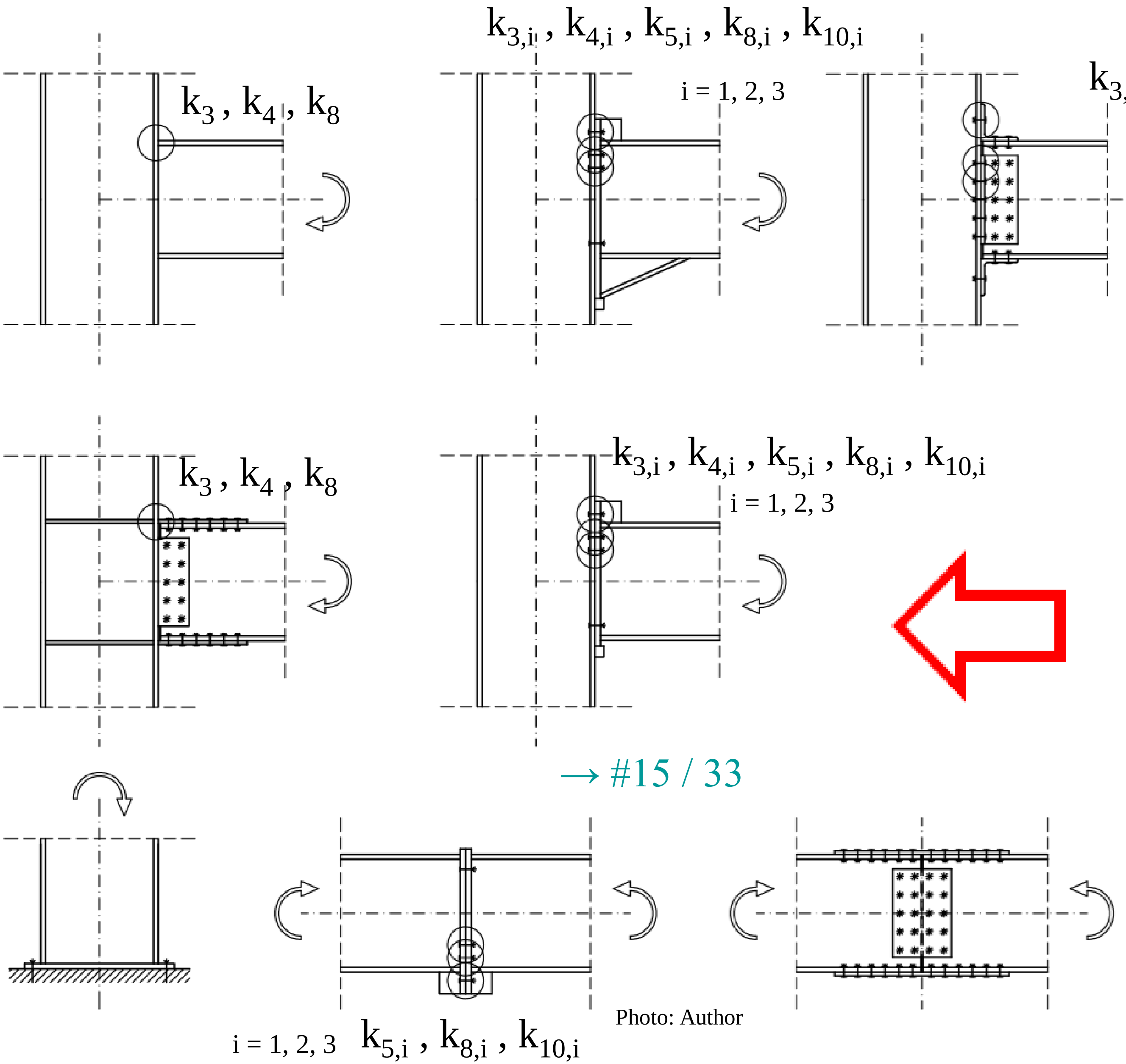
k_1 – column web in shear;

k_2 – column web in local transversal compression;

k_7 – flange of beam in compression;

k_{19} – welds (each on each positions);

k_{20} – haunched beam in compression;



→ #15 / 33

Photo: Author

k_3 – column web in local transversal tension;

k_4 – column flange in bending;

k_5 – end plate in bending;

k_6 – flange cleat in bending;

k_8 – beam web in tesnion;

k_{10} – bolt in tension;

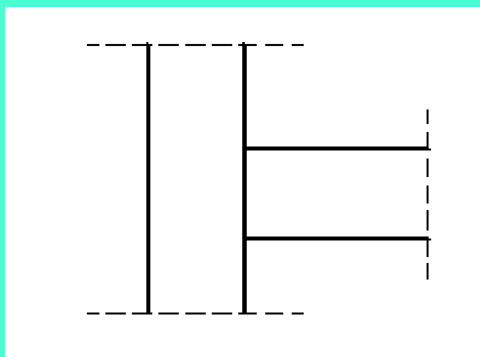
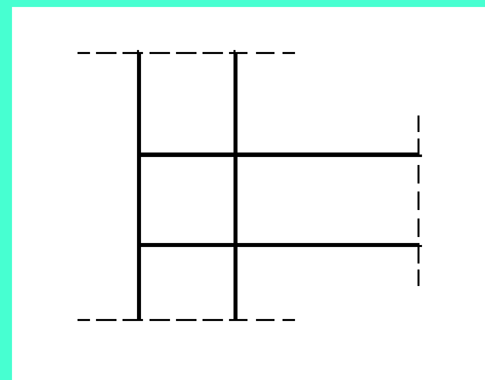
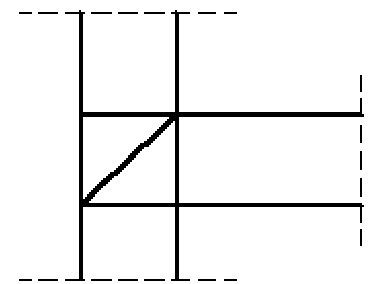
Values of k_i

k_1

Column web in shear

EN 1993-1-8 tab. 6.11

Photo: Author

 	
$0{,}38\, A_{vc} / \beta\, z$	$\rightarrow \infty$

z – between centres of gravities of compressed flange and center of I^{st} and III^{rd} row of bolts = between centres of gravities top and bottom flange = 412 mm

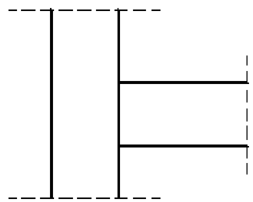
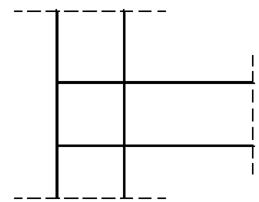
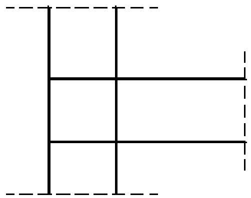
k_1 = 3,901 mm

k_2 — Column web in transversal compression

EN 1993-1-8 tab. 6.11

Photo: Author

k_3 — Column web in transversal tension

	Welded, bolted	Bolted	Welded
			
k_2	$0{,}7\ b_{eff,\ c,\ wc}\ t_{wc}\ /\ d_c$	$\rightarrow\ \infty$	
k_3	$0{,}7\ b_{eff,\ t,\ wc}\ t_{wc}\ /\ d_c$		$\rightarrow\ \infty$

Bolt-row location (column)	Bolt-row considered individually		As part of a group of bolt-rows	
	$k_3\ \ l_{eff,\ cp}$ [mm]	**$k_3\ \ l_{eff,\ nc}$ [mm]**	Circular $\Sigma l_{eff,\ cp}$	Non-circular $\Sigma l_{eff,\ nc}$
(red)	4,223	**3,019**		
(orange)	3,238	**3,144**		
(green)	3,238	**3,847**		

k_4

Column flange in bending $0,9\, l_{eff}\, t_{fc}^3 / m^3$

Bolt-row location (girder)	Bolt-row considered individually		As part of a group of bolt-rows	
	k_4 $l_{eff,\,cp}$ [mm]	**k_4 $l_{eff,\,nc}$ [mm]**	Circular $\Sigma l_{eff,\,cp}$	Non-circular $\Sigma l_{eff,\,nc}$
	10,315	**7,373**		
	139,970	**135,913**		
	139,970	**123,065**		

k_5

End-plate in bending $0,9\, l_{eff}\, t_p^3 / m^3$

Bolt-row location (girder)	Bolt-row considered individually		As part of a group of bolt-rows	
	k_5 $l_{eff,\,cp}$ [mm]	**k_5 $l_{eff,\,nc}$ [mm]**	Circular $\Sigma l_{eff,\,cp}$	Non-circular $\Sigma l_{eff,\,nc}$
	67,545	**53,180**		
	82,521	**78,853**		
	82,521	**67,851**		

k_7

Beam flange and beam web in compression

k_8

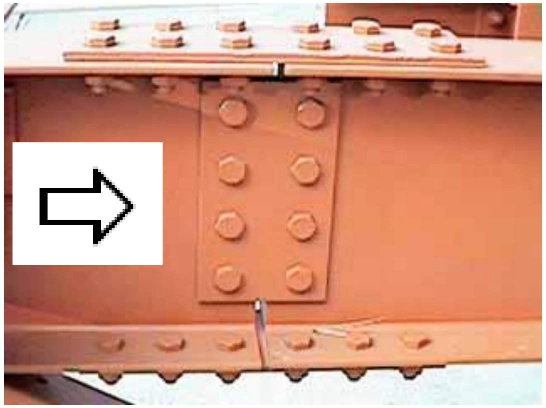
Beam web in tension

→ #15 / 40

k_9

Web / flange plate in tension or compression

Photo: amsd.co.uk



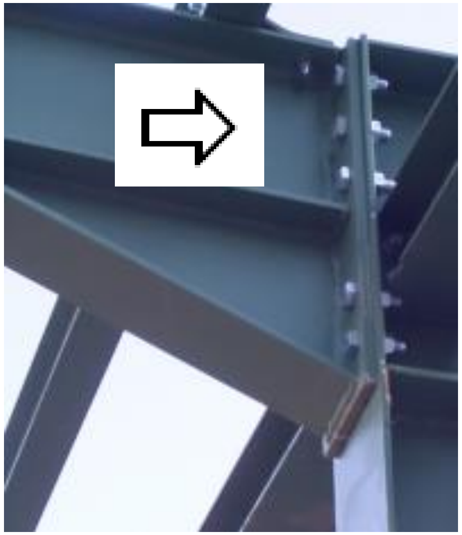


Photo: resources.scia.net

EN 1993-1-8 tab. 6.11 – no information about value. Parts important for resistance of joint only, not for stiffness. Stiffness of web, flange on plate in their planes is very big. This means, local stiffness can be taken into consideration as

$\rightarrow \infty$

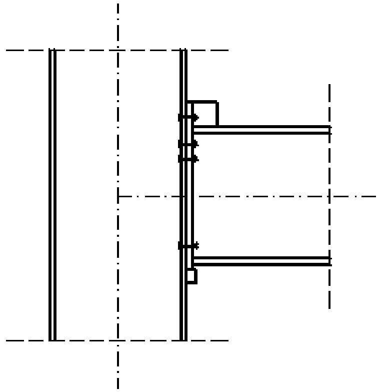
Stiffness of total join is in proportion to S (1 / k_i). When k_i tends to infinitive, its reversion tends to 0. This parts have no effect on total stiffness of joins.

k_{10} $1{,}6\, A_s / L_b$

Bolts in tension

EN 1993-1-8 tab. 6.11

Bolt-row location	**k_{10} [mm]**
	9,110
	7,335
	7,335



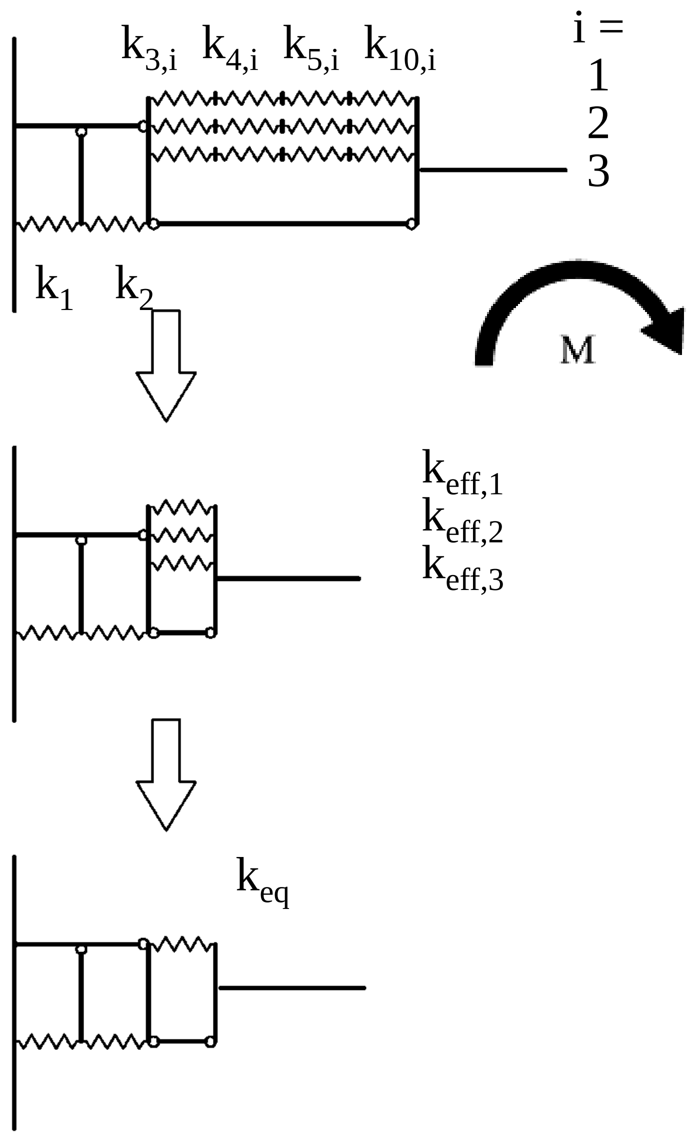


Photo: Author

Effective spring must be calculated for each row of bolts

$$k_{eff,\,r} = 1 / \Sigma\,(1 / k_{i,\,r})$$

$k_{i,\,r}$ – springs # 3, 4, 5, 7, 8, 9, 10 for row of bolts #r;
min value (circular; non-circular) for row of bolts considered individually

Row of bolts	k_3 [mm]	k_4 [mm]	k_5 [mm]	k_7 [mm]	k_8 [mm]	k_9 [mm]	k_{10} [mm]
(red)	3,019	7,373	53,180				9,110
(orange)	3,144	135,913	78,853		∞		7,335
(green)	3,847	123,065	67,851				7,335

Row of bolts	$1 / k_3$ [1/mm]	$1 / k_4$ [1/mm]	$1 / k_5$ [1/mm]	$1 / k_7$ [1/mm]	$1 / k_8$ [1/mm]	$1 / k_9$ [1/mm]	$1 / k_{10}$ [1/mm]
(red)	0,331 236	0,135 630	0,018 804				0,109 769
(orange)	0,318 066	0,007 358	0,012 682		0		0,136 333
(green)	0,259 943	0,008 126	0,014 738				0,136 333

Row of bolts	$\Sigma (1 / k_i)$ [1/mm]	$k_{eff,\, i}$ [mm]	h_i [mm]	$k_{eff,\, i}\, h_i^2$ [mm³]	$k_{eff,\, i}\, h_i$ [mm²]
	0,595	1,679	499	418 180,497	838,0371
	0,474	2,108	374	294 824,438	788,3006
	0,419	2,386	324	250 456,037	773,0125

Equivalent spring for three rows of bolts

$$k_{eff,\, r} = 1 / \Sigma\, (\, 1 / k_{i,\, r}\,)$$

$$z_{eq} = \Sigma\, (\, k_{eff,\, r}\, h_r^2) / \Sigma\, (\, k_{eff,\, r}\, h_r) = 401{,}551 \text{ mm}$$

$$\mathbf{k_{eq}} = \Sigma\, (\, k_{eff,\, r}\, h_r) / z_{eq} = \mathbf{5{,}975\ mm}$$

$$z\ (\rightarrow \#t / 42) = 412 \text{ mm}$$

$$\mathbf{S_{j,\, ini}} = E\, z^2 / [(1 / k_1) + (1 / k_2) + (1 / k_{eq})] =$$

$$= \mathbf{84{,}130\ GNm / rad}$$

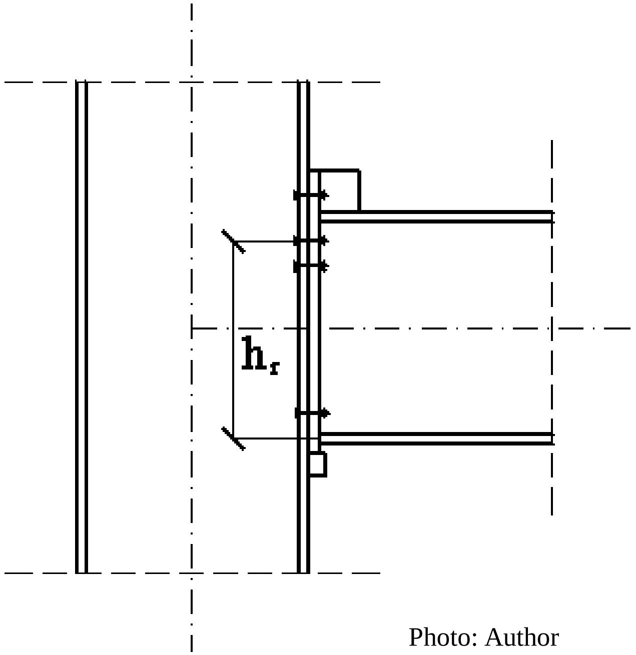
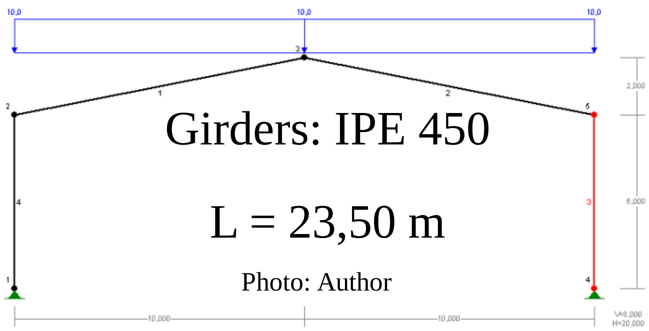


Photo: Author

Global stiffness



Girders: IPE 450

L = 23,50 m

Photo: Author

Columns: IPE 450

H = 5,45 m

J_y (IPE 450) = 33 740 cm^4 cm^4

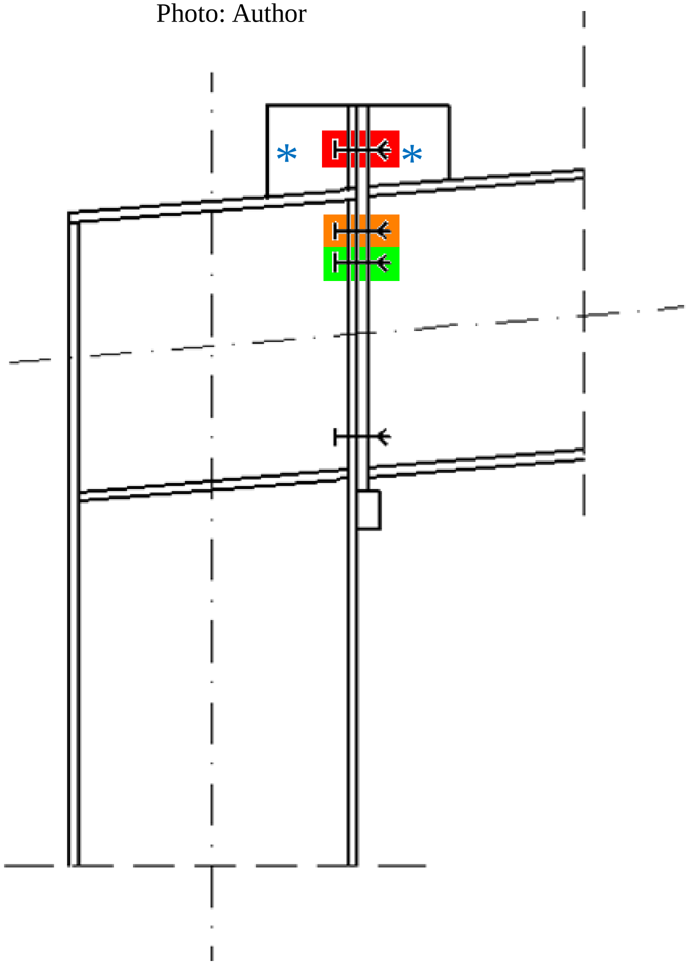
$S_{1\text{-}2} = k_b \, E \, J_b / l_b$

Non-braced frame, $(J_b / l_b) / (J_c / l_c) = 0{,}232 \geq 0{,}1 \rightarrow k_b = 25$

$S_{1\text{-}2}$ = 73,582 MNm / rad

$S_{j,\,ini}$ = 84,130 MNm / rad > $S_{1\text{-}2}$ = 73,582 MNm / rad → **rigid joint**

Photo: Author



Resistance for bending moment

Ist row of bolts lies over flange and stiffener. Due to, this row is completely separated from IInd and IIIrd and will be not taken into consideration in interaction with other rows.

Rows analised in such case:

Ist individually;

IInd individually;

IIIrd individually;

IInd and IIIrd as group.

<u>Resistances of beam flage in compression, column web in transversal compression and column web in shear are the same for each rows</u>.

Beam flange and beam web in compression

The same rules for welded and bolted joints

EN 1993-1-8 (6.21)

$F_{c,\,fb,\,Rd} = M_{c,\,Rd} / (h - t_{fp}) = M_{Rd,\,y} / (h - t_{fp})$ = 352,500 kNm / (0,450 m – 0,0146 m) =

= **809,600 kN**

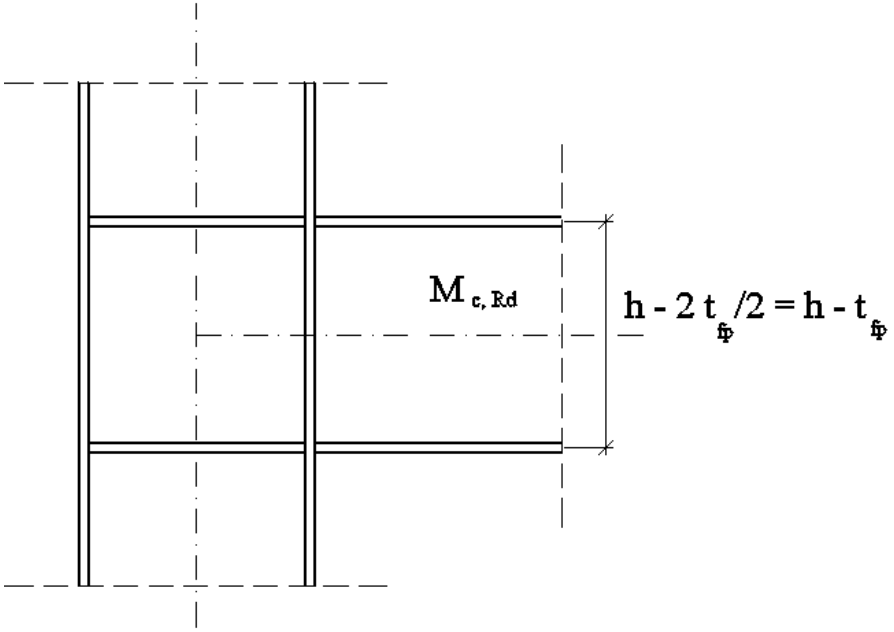
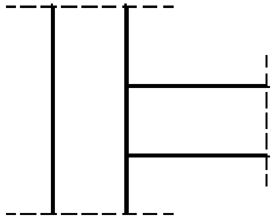
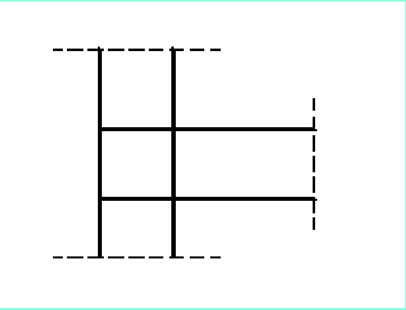


Photo: Author

Column web in transverse compression

The same rules for welded and bolted joints
EN 1993-1-8 6.2.6.2

Photo: Author

		
$F_{c,\,wc,\,Rd}$	$\min(1\ \omega\ k_{wc}\ b_{eff,\,c,\,wc}\ t_{wc}\ f_{y,\,wc}\ /\ \gamma_{M0}\ ;$ $\rho\ \omega\ k_{wc}\ b_{eff,\,c,\,wc}\ t_{wc}\ f_{y,\,wc}\ /\ \gamma_{M0}) =$ $= \min(1\ ;\ \rho)\ \omega\ k_{wc}\ b_{eff,\,c,\,wc}\ t_{wc}\ f_{y,\,wc}\ /\ \gamma_{M0}$	= resistance of stiffener (→ lecture #21)

Information in Eurocode is not copletely clear about influence of horizontal stiffener. The same in literature: in various sources are prezented various ways of calculation.

Most often way in literature:

Own resistance of web:

$F_{c,\,wc,\,Rd} = \min(1\ ;\ \rho)\ \omega\ k_{wc}\ b_{eff,\,c,\,wc}\ t_{wc}\ f_{y,\,wc}\ /\ \gamma_{M0} = 284{,}324$ kN

Resistance of stiffeners should be calculated according to the same way, as in II[nd] design project. Thickness of stiffeners should have not smaller value as thickness of beam flanges (15 mm).

Resistance for contact (according to Des 2 examp 3 / 36) = 479,400 kN

Stability (according to Des 2 examp 3 / 43) = 1257,438 kN

Min (479,400 ; 1257,438) = 479,400 kN

Total: $F_{cT,\,wc,\,Rd}$ = 284,324 kN + 479,400 kN = **763,724 kN**

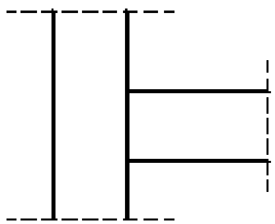
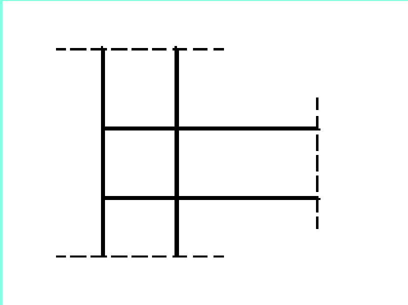
Column web in shear

The same for welded and bolted joints

EN 1993-1-8 6.2.6.1

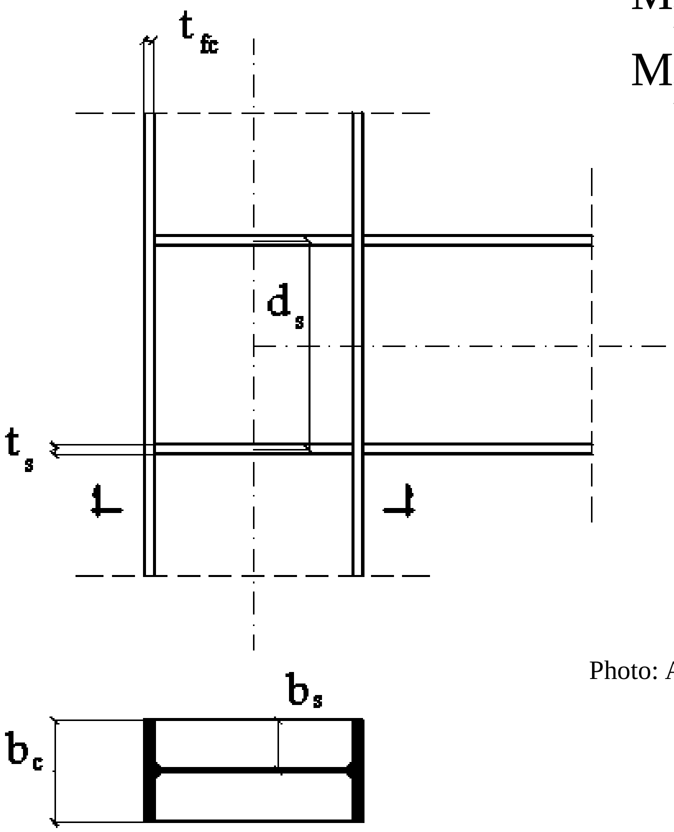
$$V_{wp,\,Rd} = [(0{,}9\, f_{y,\,wc}\, A_{vc}) / (\gamma_{M0}\, \sqrt{3})] + V_{wp,add,\,Rd}$$

Photo: Author

		
$V_{wp,add,\,Rd}$	0	$\min [\, 4M_{pl,\,fc,\,Rd} / d_s \;;\; (2M_{pl,\,fc,\,Rd} + 2M_{pl,\,st,\,Rd}) / d_s \,]$

$$[(0{,}9\, f_{y,\,wc}\, A_{vc}) / (\gamma_{M0}\, \sqrt{3})] = 516{,}524 \text{ kN}$$

EN 1993-1-8 6.2.6.1



$$M_{pl,\, fc,\, Rd} = 0{,}25\ b_c\ t_{fc}^{\ 2}\ f_{y,\, fc}\ /\ \gamma_{M0} = 2{,}512\ \text{kNm}$$

$$M_{pl,\, st,\, Rd} = 0{,}50\ b_s\ t_s^{\ 2}\ f_{y,\, s}\ /\ \gamma_{M0} = 2{,}379\ \text{kNm}$$

$$V_{wp,add,\, Rd} = \min\ (23{,}099\ \text{kN}\ \ ;\ \ 22{,}487\ \text{kN})$$

Photo: Author

$$V_{wp,\, Rd} = \mathbf{539{,}011\ kN}$$

Column web in tension

EN 1993-1-8 6.2.6.3

$$F_{t,\,wc,\,Rd} = \omega \;\; l_{eff,\,t,\,wc} \;\; t_{wc} \;\; f_{y,\,wc} \;\; / \;\; \gamma_{M0}$$

[kN]

Bolt-row location	Bolt-row considered individually		As part of a group of bolt-rows	
	Circular $l_{eff,\,cp}$	**Non-circular $l_{eff,\,nc}$**	Circular $\Sigma l_{eff,\,cp}$	**Non-circular $\Sigma l_{eff,\,nc}$**
(red)	409,151	**292,467**	Separated row	
(orange)	313,682	**304,590**	466,735 *	**380,359** *
(green)	313,682	**275,798**		

* Value for $l_{eff,\,cp} = \Sigma l_{eff,\,cp}\,(2) + \Sigma l_{eff,\,cp}\,(3)$ = 154 mm + 154 mm = 308 mm

* Value for $l_{eff,\,nc} = \Sigma l_{eff,\,nc}\,(2) + \Sigma l_{eff,\,nc}\,(3)$ = 135 mm + 116 mm = 251 mm

EN 1993-1-8 6.2.6.3:

(5) Stiffeners or supplementary web plates may be used to increase the design tension resistance of a column web.

(6) Transverse stiffeners and/or appropriate arrangements of diagonal stiffeners may be lIsed to increase the design resistance of the column web in tension.

It's not clear, what means increasing resistance by stiffenes. No formula are presented in Eurocode.

Some literature proposes to add resistance of stiffener to resistance of each bolt row. Another part of literature ignores these Eurocode points. The latter approach seems safer.

Beam web in tension

EN 1993-1-8 6.2.6.3

$$F_{t,\,wb,\,Rd} = \omega \;\; l_{eff,\,t,\,wb} \;\; t_{wb} \;\; f_{y,\,wb} \;\; / \;\; \gamma_{M0}$$

[kN]

Bolt-row location	Bolt-row considered individually		As part of a group of bolt-rows	
	Circular $l_{eff,\,cp}$	**Non-circular $l_{eff,\,nc}$**	Circular $\Sigma l_{eff,\,cp}$	**Non-circular $\Sigma l_{eff,\,nc}$**
(red)	334,897	**263,675**	Separated row	
(orange)	409,151	**390,966**	560,688 *	**466,735** *
(green)	409,151	**336,413**		

* Value for $l_{eff,\,cp} = \Sigma l_{eff,\,cp}\,(2) + \Sigma l_{eff,\,cp}\,(3)$ = 185 mm + 185 mm = 370 mm

* Value for $l_{eff,\,nc} = \Sigma l_{eff,\,nc}\,(2) + \Sigma l_{eff,\,nc}\,(3)$ = 172 mm + 136 mm = 308 mm

Column flange in bending

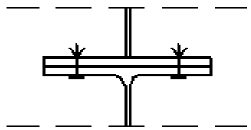
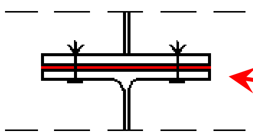
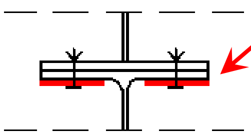
There are different rules for bolted and welded joints

Bolted joint:

We must analyse three different mechanism of destruction, according to prying actions phenomenon

→ #19 / 62

Resistances

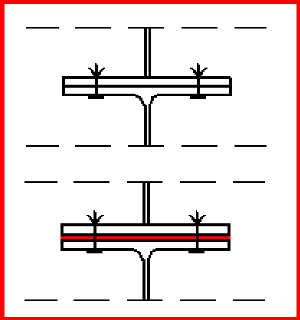
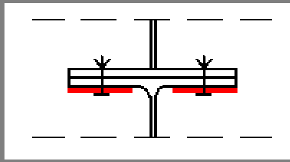
Photo: Author	Prying forces → #19 / 64		No prying forces: $F_{T,\,1\text{-}2,\,Rd}$
Mode 1: $F_{T,\,1,\,Rd}$ = min (1.1 ; 1.2)	Method 1.1	Method 1.2	$2\,M_{pl,\,1,\,Rd} / m$
  Reinforcing by additional plate(s)	$4\,M_{pl,\,1,\,Rd} / m$	$(8n - 2e_w)\,M_{pl,\,1,\,Rd}$ / $[2mn - e_w(m + n)]$	
	$(4\,M_{pl,\,1,\,Rd} + 2M_{bp,\,Rd}) / m$	$[(8n - 2e_w)\,4\,M_{pl,\,1,\,Rd} + 4n\,M_{bp,\,Rd}]$ / $[2mn - e_w(m + n)]$	
Mode 2: $F_{T,\,2,\,Rd}$	$(2\,M_{pl,\,2,\,Rd} + n\,\Sigma F_{t,\,Rd}) / (m + n)$		
Mode 3: $F_{T,\,3,\,Rd}$	$\Sigma F_{t,\,Rd}$		

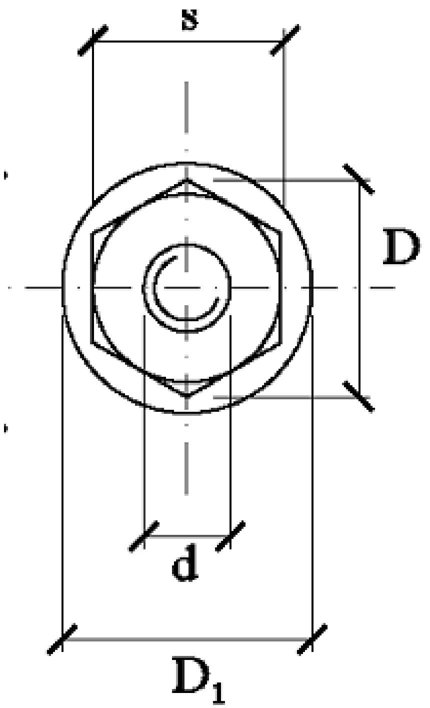
EN 1993-1-8 tab 6.2

Symbols → #19 / 69

Conclusion from #t / 28:

Bolt-row location	Column	Girder
(red)	Prying actions	No prying actions
(orange)	No prying actions	No prying actions
(green)	No prying actions	No prying actions

Photo: Author	Prying forces		No prying forces: $F_{T, 1\text{-}2, Rd}$
Mode 1: $F_{T, 1, Rd} = \min(1.1 ; 1.2)$	Method 1.1	Method 1.2	$2 M_{pl, 1, Rd} / m$
	$4 M_{pl, 1, Rd} / m$	$(8n - 2e_w) M_{pl, 1, Rd}$ / $[2mn - e_w(m + n)]$	
	$(4 M_{pl, 1, Rd} + 2M_{bp, Rd})$ / m	$[(8n - 2e_w) 4 M_{pl, 1, Rd} + 4n M_{bp, Rd}]$ / $[2mn - e_w(m + n)]$	
Mode 2: $F_{T, 2, Rd}$	$(2 M_{pl, 2, Rd} + n \Sigma F_{t, Rd}) / (m + n)$		
Mode 3: $F_{T, 3, Rd}$	$\Sigma F_{t, Rd}$	$\Sigma F_{t, Rd}$	$\Sigma F_{t, Rd}$



$$m \rightarrow \#t / 56$$

$$n = \min(e_{min} \; ; \; 1{,}25m)$$

$$e_w = d_w / 4$$

$$d_w = D \text{ or } D_1$$

$$l_{eff,1} = \min(l_{eff,cp} \; ; \; l_{eff,nc})$$

$$\Sigma l_{eff,1} = \min(\Sigma l_{eff,cp} \; ; \Sigma l_{eff,nc})$$

$$l_{eff,2} = l_{eff,nc}$$

$$\Sigma l_{eff,2} = \Sigma l_{eff,nc}$$

Photo: Author

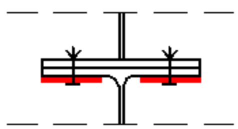
$\Sigma F_{t,Rd} \rightarrow \#19 / 80$

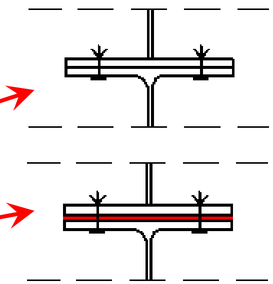
t_{fc} – thickness of column's flange

t_{bp} – thickness of addidional plate

$l_{eff} \rightarrow \#19 / 41, 42$

$\Sigma l_{eff} \rightarrow \#19 / 41, 42$





$t_{fc} = t_{fc,true}$ or $t_{fc,true} + t_{bp}$

	Bolt-row considered individually	As part of a group of bolt-rows
$M_{pl,1,Rd}$	$0{,}25\, l_{eff,1} \; t_{fc}^2 \; f_y \,/\, \gamma_{M0}$	$0{,}25\, \Sigma l_{eff,1} \; t_{fc}^2 \; f_y \,/\, \gamma_{M0}$
$M_{pl,2,Rd}$	$0{,}25\, l_{eff,2} \; t_{fc}^2 \; f_y \,/\, \gamma_{M0}$	$0{,}25\, \Sigma l_{eff,2} \; t_{fc}^2 \; f_y \,/\, \gamma_{M0}$
$M_{bp,Rd}$	$0{,}25\, l_{eff,1} \; t_{bp}^2 \; f_{y,bp} \,/\, \gamma_{M0}$	$0{,}25\, \Sigma l_{eff,1} \, t_{bp}^2 \; f_{y,bp} \,/\, \gamma_{M0}$

→ #19 / 65

EN 1993-1-8 tab 6.2

Photo: Author

$M_{pl,\,1,\,Rd}$ [kNm]

Bolt-row location	Bolt-row considered individually	As part of a group of bolt-rows
	min (Circular $l_{eff,\,cp}$ Non-circular $l_{eff,\,nc}$)	min (Circular $\Sigma l_{eff,\,cp}$ Non-circular $\Sigma l_{eff,\,nc}$)
	2,551	Separated row
	10,628	13,272
	9,623	

$M_{pl,\,2,\,Rd}$ [kNm]

Bolt-row location	Bolt-row considered individually	As part of a group of bolt-rows
	Non-circular $l_{eff,\,nc}$	Non-circular $\Sigma l_{eff,\,nc}$
	2,551	Separated row

No additional plates $\rightarrow t_{bp} = 0 \rightarrow M_{bp,\,Rd} = 0$

Mode 1: $F_{T, 1, Rd}$ [kN]

Bolt-row location	Bolt-row considered individually	As part of a group of bolt-rows
	237,302	Separated row
	988,648	1 235,904
	895,160	

Mode 3: $F_{T, 3, Rd}$ [kN]

Bolt-row location	$\Sigma F_{t, Rd}$	$\Sigma B_{p,Rd}$
	304,992	631,897
	304,992	1 263,795
	304,992	1 263,795

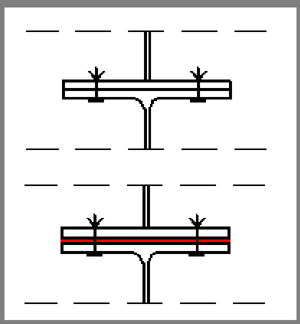
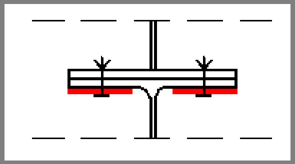
Mode 2: $F_{T, 2, Rd}$ [kN] (mix of modes 1 and 3)

Bolt-row location	Bolt-row considered individually	As part of a group of bolt-rows
(red)	219,056	Separated row
(orange)		
(green)		

Final result = min ($F_{T, 1, Rd}$, $F_{T, 2, Rd}$, $F_{T, 3, Rd}$)

Bolt-row location	Bolt-row considered individually	As part of a group of bolt-rows
(red)	**219,056**	Separated row
(orange)	**304,992**	**1 235,904**
(green)	**304,992**	

End plate in bending

	Prying forces		No prying forces: $F_{T, 1\text{-}2, Rd}$
Mode 1: $F_{T, 1, Rd}$ = min (1.1 ; 1.2)	Method 1.1	Method 1.2	
	$4 M_{pl, 1, Rd} / m$	$(8n - 2e_w) M_{pl, 1, Rd}$ / $[2mn - e_w(m + n)]$	$2 M_{pl, 1, Rd} / m$
	$(4 M_{pl, 1, Rd} + 2M_{bp, Rd}) / m$	$[(8n - 2e_w) 4 M_{pl, 1, Rd} + 4n M_{bp, Rd}]$ / $[2mn - e_w(m + n)]$	
Mode 2: $F_{T, 2, Rd}$	$(2 M_{pl, 2, Rd} + n \Sigma F_{t, Rd}) / (m + n)$		
Mode 3: $F_{T, 3, Rd}$	$\Sigma F_{t, Rd}$	$\Sigma F_{t, Rd}$	$\Sigma F_{t, Rd}$

$M_{pl, 1, Rd}$ [kNm]

Bolt-row location	Bolt-row considered individually	As part of a group of bolt-rows
	min (Circular $l_{eff, cp}$ Non-circular $l_{eff, nc}$)	min (Circular $\Sigma l_{eff, cp}$ Non-circular $\Sigma l_{eff, nc}$)
(red)	9,200	Separated row
(orange)	13,642	16,286
(green)	11,738	

Mode 1: $F_{T, 1, Rd}$ [kN]

Bolt-row location	Bolt-row considered individually	As part of a group of bolt-rows
(red)	427,907	
(orange)	634,512	757,488
(green)	545,954	

Mode 3: $F_{T,3,Rd}$ [kN]

Bolt-row location	$\Sigma F_{t,Rd}$	$\Sigma B_{p,Rd}$
(red)	304,992	1 263,795
(orange)	304,992	1 263,795
(green)	304,992	1 263,795

Final result = min ($F_{T,1,Rd}$, $F_{T,3,Rd}$)

Bolt-row location	Bolt-row considered individually	As part of a group of bolt-rows
(red)	**304,992**	Separated row
(orange)	**304,992**	**757,488**
(green)	**304,992**	

Equivalent resistance of I[st] row of bolts

The same values for each rows of bolts

$R_{eq,\,I}$ = min (resistance of column web under comperssion ; resistance of beam flange in compression ; resistance of column web in shear ; resistance of column web in tension around I[st] row of bolts ; **resistance of column flange under bending around I[st] row of bolts** ; resistance of end plate around I[st] row of bolts ; resistance of beam web around I[st] row of bolts) =

= min (763,724 ; 809,600 ; 539,011 ; 292,467 ; **219,056** ; 304,992 ; 263,675) =

= **219,056 kN**

The weaknest link is column flange in bending

Equivalent resistance of II^{nd} row of bolts

The same values for each rows of bolts

$R_{eq,\,II}$ = min (resistance of column web under comperssion ; resistance of beam flange in compression ; resistance of column web in shear ; **resistance of column web in tension around II^{nd} row of bolts** ; resistance of column flange under bending around II^{nd} row of bolts ; resistance of end plate around II^{nd} row of bolts ; resistance of beam web around II^{nd} row of bolts) =

= min (763,724 ; 809,600 ; 539,011 ; **304,590** ; 304,992 ; 304,992 ; 390,966) =

= **304,590 kN**

The weaknest link column web in tension

Equivalent resistance of III[rd] row of bolts

The same values for each rows of bolts

$R_{eq,\ III}$ = min [resistance of column web under comperssion ; resistance of beam flange in compression ; resistance of column web in shear ;

resistance of column web in tension around III[rd] row of bolts considered individually ; resistance of column flange under bending around III[rd] row of bolts considered individually ; resistance of end plate around III[rd] row of bolts considered individually ; resistance of beam web around III[rd] row of bolts considered individually) =

= min (763,724 ; 809,600 ; 539,011 ;

275,798 ; 304,992 ; 304,992 ; 336,413) =

= **275,798 kN**

The weaknest link is column web in tension

$R_{eq,\,I} = 219{,}056\ kN$; the weaknest link is column flange in bending

$R_{eq,\,II} = 304{,}590\ kN$; the weaknest link column web in tension

$R_{eq,\,III} = 275{,}798\ kN$; the weaknest link is column web in tension

Equivalent resistance of row of bolts can be effect of resistance sub-patrs of joint, not resistance od shank of bolt.

Additionally, points EN 1993-1-8 6.2.7.2 (7), (8) and (9) must be taken into consideration. Is possible, than deceasing of equivalent resistance of row of bolts must be applicated.

EN 1993-1-8   6.2.7.2 (7)

Balance between resistance of compressed zone and tensed zone:

$$\Sigma F_{t,\, Rd} \leq \min\,(F_{c,\, wc,\, Rd} \;\; ; \;\; F_{c,\, fb,\, Rd} \;\; ; \;\; V_{wp,\, Rd} / b)$$

$\Sigma F_{t,\, Rd}$ - total resistance of each rows of bolts in tensed part of joint, i.e. minimum of: (web of column in local tension ; flange of column in local bending ; end plate in local bending ; web of beam in local tension)

In alanysed case:

$$\Sigma F_{t,\, Rd} = R_{eq,\, I} + R_{eq,\, II} + R_{eq,\, III} = 219{,}056 + 304{,}590 + 275{,}798 = 799{,}444 \text{ kN}$$

$$\Sigma F_{t,\, Rd} = \min\,(763{,}724 \;\; ; \;\; 809{,}600 \;\; ; \;\; 539{,}011) = 539{,}011 \text{ kN}$$

$R_{eq,\, III}$ must be reduced

$$R_{eq,\, III} = \Sigma F_{t,\, Rd} - (R_{eq,\, I} + R_{eq,\, II}) = 15{,}363 \text{ kN}$$

EN 1993-1-8   6.2.7.2 (8)

Condition for resistance of tensed part:

$F_{t,\,r,\,Rd\,(individual)} \leq F_{t,\,r,\,Rd\,(group)}$

In analised case:

$(F_{t,\,r,\,Rd\,II} + F_{t,\,r,\,Rd\,III}) \leq F_{t,\,r,\,Rd\,(II\,+\,III)}$

$(F_{t,\,r,\,Rd\,II} + F_{t,\,r,\,Rd\,III}) = 304{,}590 + 15{,}363 = 319{,}955$ kN

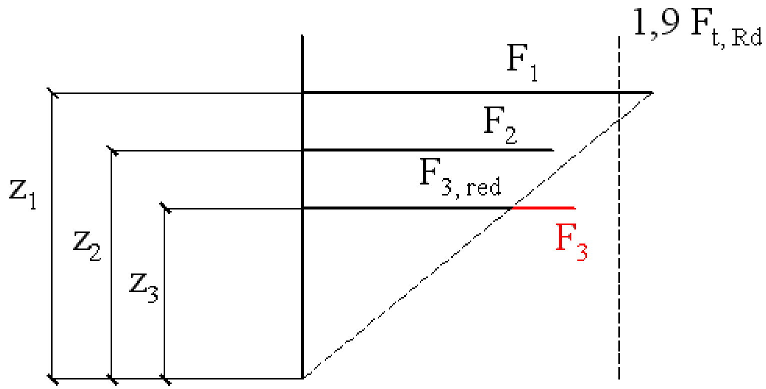
$F_{t,\,r,\,Rd\,(II\,+\,III)}$ = min (column web in tension for II+III, #t / 56 ; column flange in bending for II+III, #66 ; end plate in bending for II+III, #t / 69 ; beam web in tension for II+III, #t / 58) = min (380,359 ; 1 234,904 ; 757,488 ; 466,737) = 380,359 kN

319,955 kN < 380,359 kN

OK, no next reduction

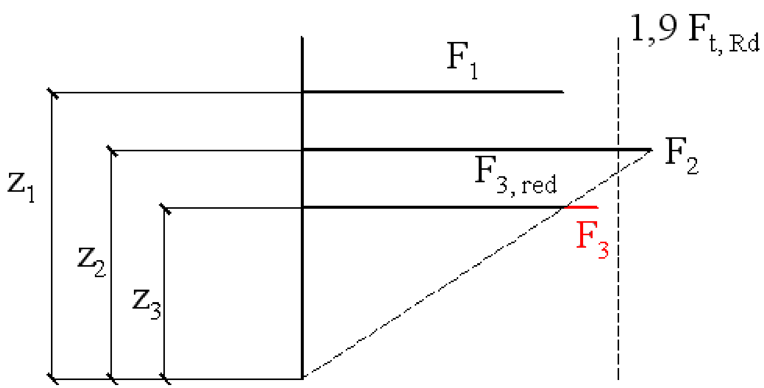
EN 1993-1-8   6.2.7.2 (9)

If for any row of bolts its equivalent resistance is greater than 1,9 resistance of bolt for axial force, equivalent resistances lower rows of bolt must be reduced by linear way.



$$F_{3,\ red} = F_1\ z_3 / z_1$$

→ #19 / 78



$$F_{3,\ red} = F_2\ z_3 / z_2$$

Photo: Author

NA 5: recommended for joints exposed to impact and vibration only

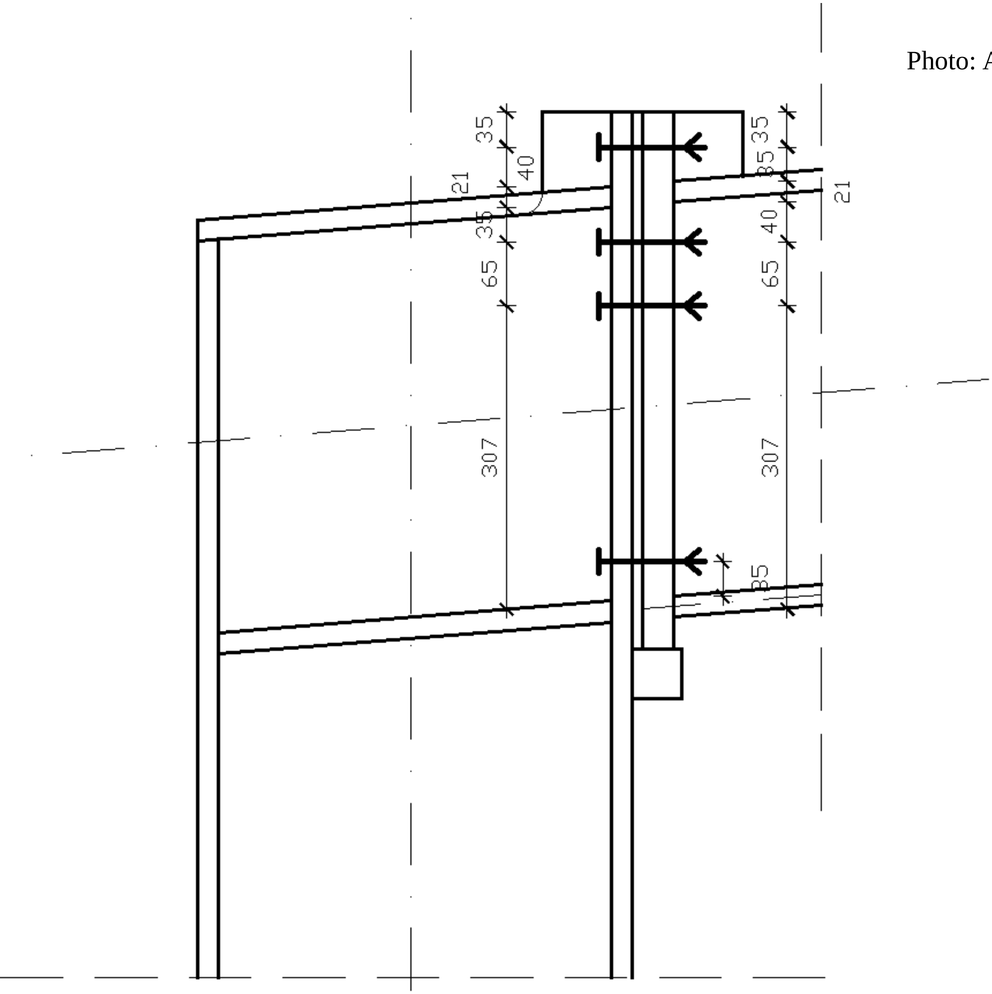


Photo: Author

Final resistance:

$$M_{Rd,\,j} = R_{eq,\,I}\, z_1 + R_{eq,\,II}\, z_2 + R_{eq,\,III}\, z_3 =$$

$$= 219{,}056 \cdot 0{,}499 + 304{,}590 \cdot 0{,}374 +$$
$$+ 15{,}363 \cdot 0{,}324 =$$

$$= 109{,}310 + 113{,}970 + 4{,}978 =$$

$$= 228{,}258 \text{ kNm}$$

$$M_{Ed,\,j} = 311{,}600 \text{ kNm}$$

$M_{Ed,\,j} / M_{Rd,\,j} =$ 1,365 > 1,0 Wrong

The biggest impact on final resistance is effect of reduction for III row of bolts because balance between compressed and tensed part of joint. Even if this problem could be overcome, next reduction: balance between row of bolts analised separately and as a group make similar reduction for III row of bolt. There is no chance to increase resistance of compressed part and resistance of gropus of rows to level, which no make reduction for III row of bolts.

For analised case only one solution could be applicated: haunched beam.

Thanks to haunched beam, lengths of arms z_i for rows of bolts will be increased. If each arm will be increased by 200 mm:

Photo: resources.scia.net

$$M_{Rd,\,j} = R_{eq,\,I}\, z_1 + R_{eq,\,II}\, z_2 + R_{eq,\,III}\, z_3 =$$
$$= 219{,}056 \cdot (0{,}499 + 0{,}200) + 304{,}590 \cdot (0{,}374 + 0{,}200) +$$
$$+ 15{,}363 \cdot (0{,}324 + 0{,}200) = 153{,}120 + 174{,}835 + 8{,}050 =$$
$$= 336{,}005 \text{ kNm}$$

$$M_{Ed,\,j} / M_{Rd,\,j} = 0{,}927 \text{ OK}$$

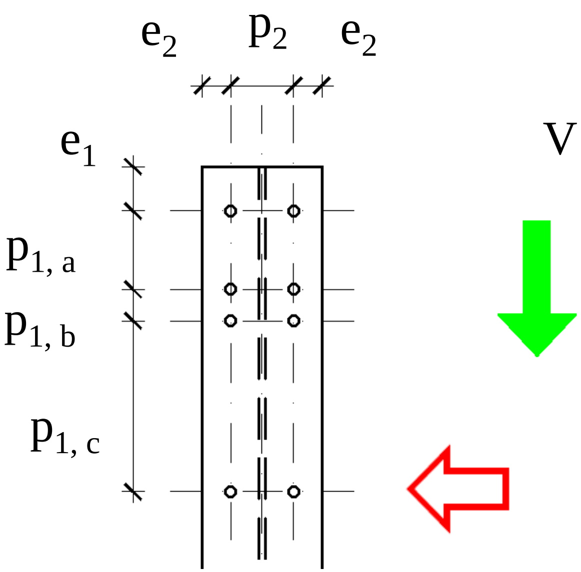


Photo: Author

Shear force acts on bolts in compressed part of joint. Class of bolts D in tension joint is analised as class A under shear force. This means, bolts should be analised for:

- shear of shank
- bearing resistance
- block tearing

Shear resistance for pair of bolts:

$F_{V,Rd} = 2\ n\ \alpha_v\ f_{ub}\ A_b\ /\ \gamma_{M2} = 169{,}440$ kN

$V_{Ed}\ /\ F_{V,Rd} = 0{,}475 < 1{,}0$ OK

Block tearing is calculated for one plate, not for complex cross-section. It should be rather calculation of netto area.

$A_{netto} = 178{,}0 - 2 \cdot 2{,}6 \cdot 2{,}1 = 159{,}08\ cm^2$

$R_{netto} = 4\ 672{,}925$ kN

$V_{Ed}\ /\ R_{netto} = 0{,}017 < 1{,}0$ OK

Bearing resistance

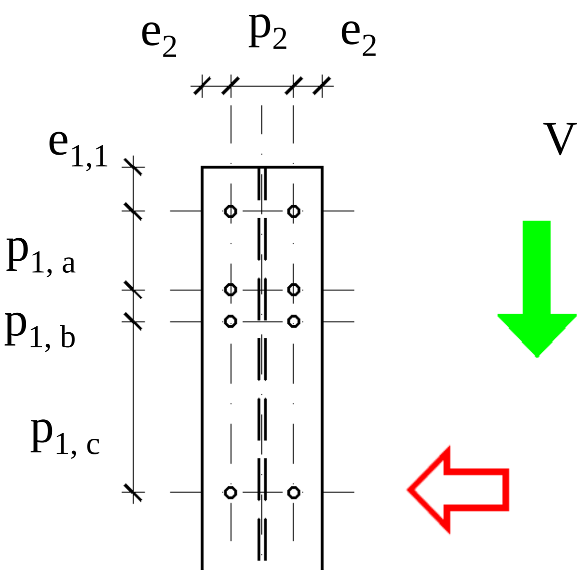


Photo: Author

Only one direction of force;

Very big distance to next row of bolts in direction of force → it is not completely clear, which type of bolts (inner / end) are analised pair. Taken into consideration:

$p_1 = p_{1,c} =$ **271 mm**

e_1 = distance to the bottom end of column ≈ 5,45 m

$e_1 = e_{1,1} + p_{1,a} + p_{1,b} + p_{1,c}$ = distance to the top end of end plate = **515 mm**

	α_d
Inner	$p_1 / 3d_0 - 0{,}25$
End	$e_1 / 3d_0$

$\alpha_d = 3{,}212$ or $6{,}603$

Bearing resistance

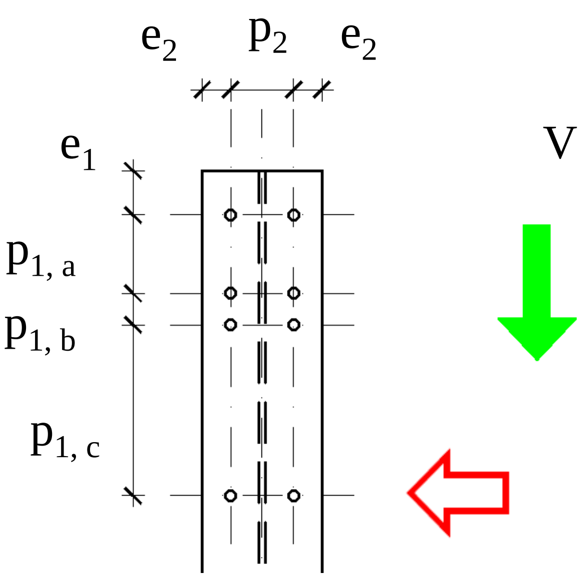


Photo: Author

Because of such big value of α_d , α_b will be taken in value 1,00. In perepndicular direction: edge bolts.

p_2 = 110 mm

e_2 = 40 mm

$$k_1 = \min(2{,}8\, e_2 / d_0 - 1{,}7\ ;\ 2{,}5) = 2{,}608$$

$$F_{b,Rd} = \beta_b\, k_1\, \alpha_b\, f_u\, d\, t_{min} / \gamma_{M2} = 540{,}795 \text{ kN}$$

For pair of bolts:

$$V_{Ed} / F_{b,Rd} = 0{,}149 < 1{,}0$$ OK

Thank you for attention

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