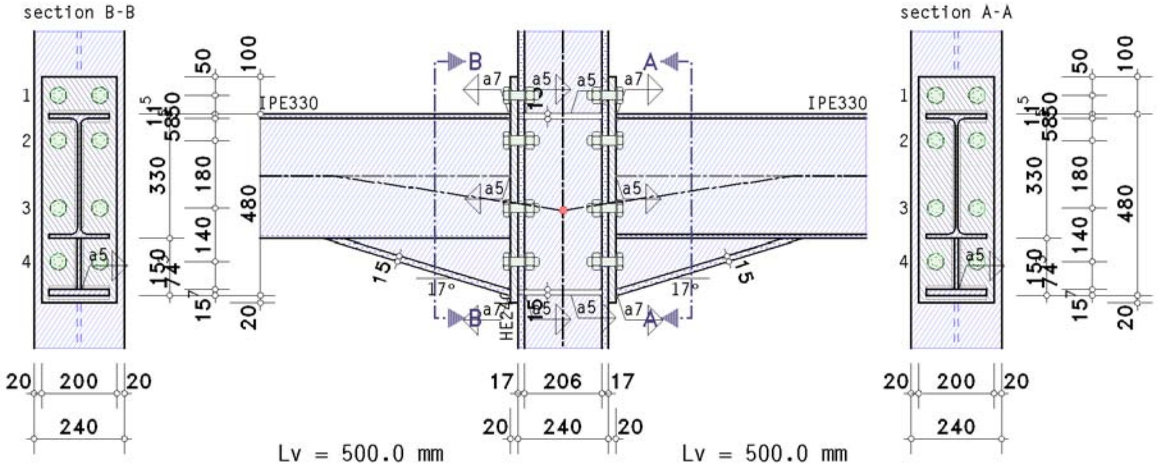
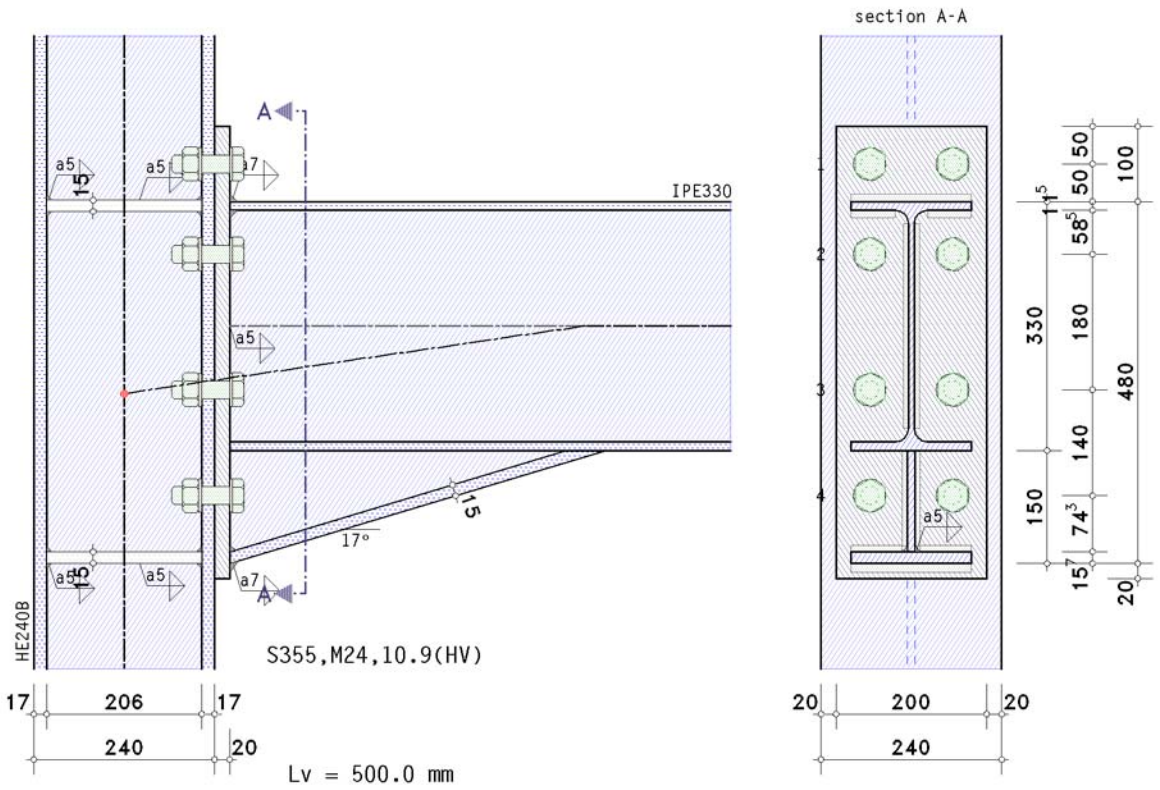
Rigid beam connection EC 3-1-8 (04.25), NA: Deutschland 4H-EC3BT version: 10/2019-2w

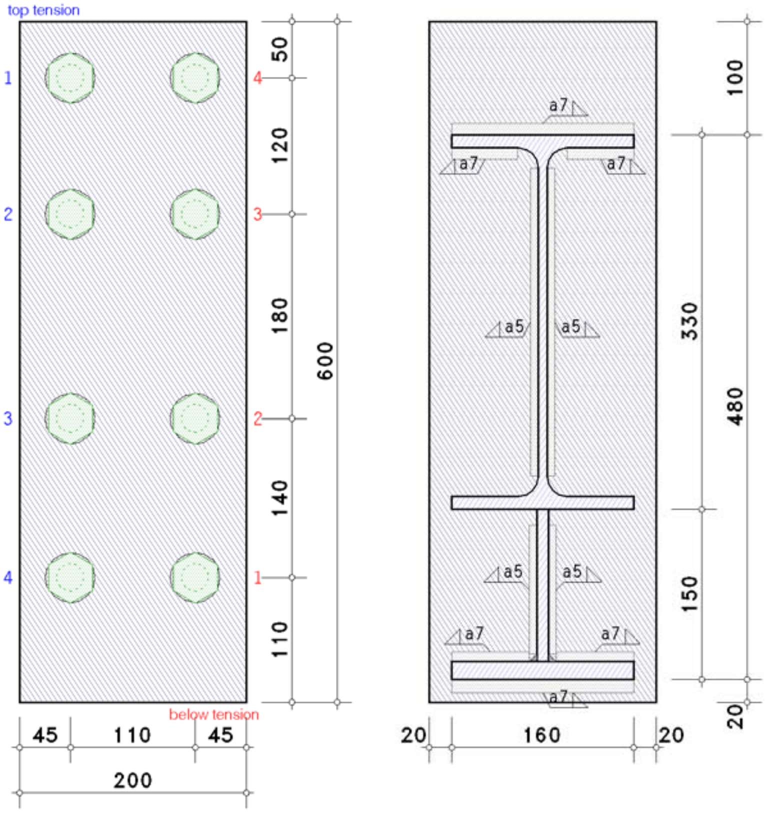
1. input report



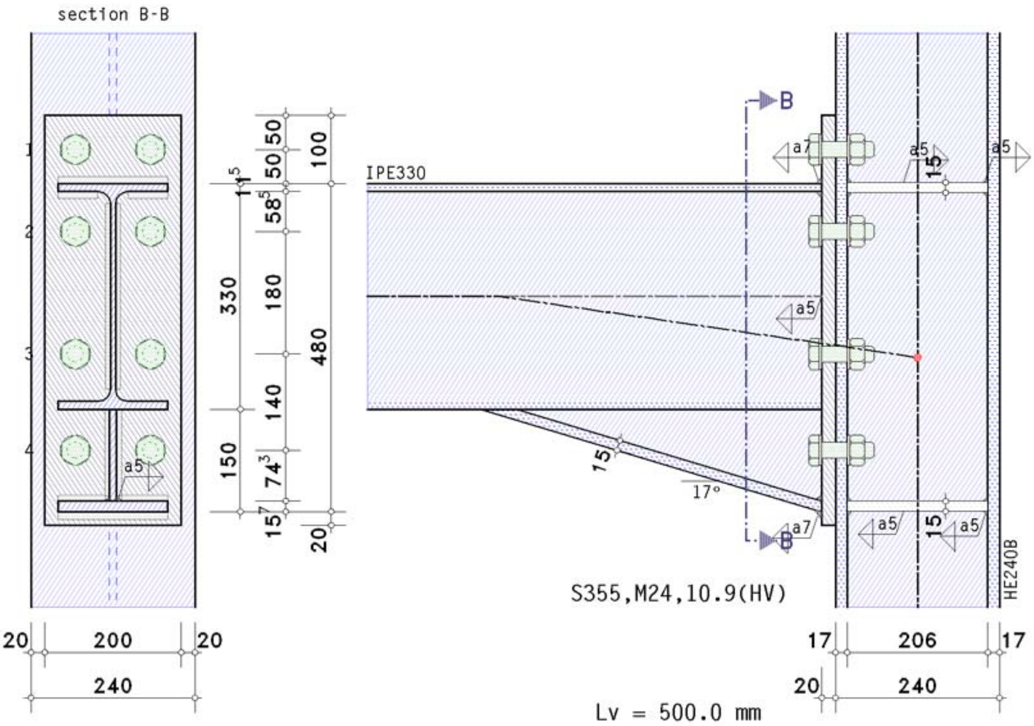
connection right



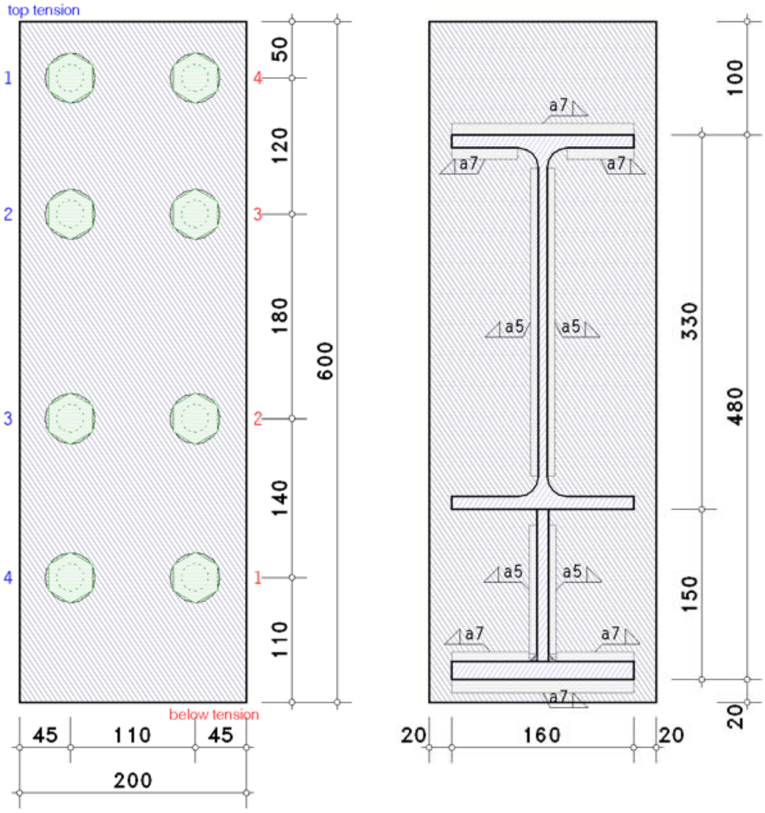
details (section A - A)



connection left



details (section B - B)



according to EC 3-1-8, 5.3 each connection side of two-sided beam-column connections is analysed independently of the other.

steel grade
steel grade S355
column parameters
section HE240B
reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 466.4 mm):
thickness t_{st} = 15.0 mm, width b_{st} = 100.0 mm, length l_{st} = 206.0 mm
recess at stiffeners c_{st} = 31.5 mm
welds $a_{st,f}$ = 5.0 mm, $a_{st,w}$ = 5.0 mm
double-sided beam-column joint, right
bolts
bolt class 10.9, bolt size M24
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}^* = 0.7 \cdot f_{yb} \cdot A_s$ = 222.1 kN)
shear plane passes through the unthreaded portion of the bolt
beam parameters
section IPE330
slope angle of haunch about the horizontal axis α_v = 16.70°
haunch length L_v = 500.0 mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b))$ = 150.0 mm
web thickness $t_{w,v}$ = 10.0 mm, flange width, thickness $b_{f,v}$ = 160.0 mm, $t_{f,v}$ = 15.0 mm, weld thickness a_v = 5.0 mm
total beam depth at the connection point $h_{ges} = h_b + h_v$ = 480.0 mm
verification parameters
bolted end-plate connection
thickness t_p = 20.0 mm, width b_p = 200.0 mm, length l_p = 600.0 mm
projections $h_{p,o}$ = 100.0 mm, $h_{p,u}$ = 20.0 mm
bolts in connection:
4 bolt-rows with 2 bolts
all bolt-rows considered individually
all bolt-rows for shear transfer (rows 1-4)
bolt groups generated automatically, considering all groups reg. row 1
centre distance of the bolts to the lateral edge of the end-plate e_2 = 45.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 50.0 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 110.0 mm
centre distance of the bolt-rows from each other p_{1-2} = 120.0 mm, p_{2-3} = 180.0 mm, p_{3-4} = 140.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 7.0 mm
beam web: fillet weld, weld thickness a = 5.0 mm
beam flange below: fillet weld, weld thickness a = 7.0 mm, angle φ = 107°
double-sided beam-column joint, left
bolts
bolt class 10.9, bolt size M24
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}^* = 0.7 \cdot f_{yb} \cdot A_s$ = 222.1 kN)

shear plane passes through the unthreaded portion of the bolt

beam parameters

section IPE330

slope angle of haunch about the horizontal axis α_v = 16.70°

haunch length L_v = 500.0 mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b))$ = 150.0 mm

web thickness $t_{w,v}$ = 10.0 mm, flange width, thickness $b_{f,v}$ = 160.0 mm, $t_{f,v}$ = 15.0 mm, weld thickness a_v = 5.0 mm

total beam depth at the connection point $h_{ges} = h_b + h_v$ = 480.0 mm

verification parameters

bolted end-plate connection

thickness t_p = 20.0 mm, width b_p = 200.0 mm, length l_p = 600.0 mm

projections $h_{p,o}$ = 100.0 mm, $h_{p,u}$ = 20.0 mm

bolts in connection:

4 bolt-rows with 2 bolts

all bolt-rows considered individually

all bolt-rows for shear transfer (rows 1-4)

bolt groups generated automatically, considering all groups reg. row 1

centre distance of the bolts to the lateral edge of the end-plate e_2 = 45.0 mm

centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 50.0 mm

centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 110.0 mm

centre distance of the bolt-rows from each other p_{1-2} = 120.0 mm, p_{2-3} = 180.0 mm, p_{3-4} = 140.0 mm

welds at the connection point:

beam flange top: fillet weld, weld thickness a = 7.0 mm

beam web: fillet weld, weld thickness a = 5.0 mm

beam flange below: fillet weld, weld thickness a = 7.0 mm, angle φ = 107°

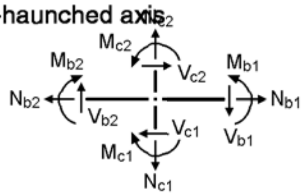
internal forces and moments in the intersection point of system axes refering to the non-haunched axis

Lc 1: $M_{j,b1,Ed}$ = -100.00 kNm $V_{j,b1,Ed}$ = 200.00 kN (right)

$M_{j,b2,Ed}$ = -100.00 kNm $V_{j,b2,Ed}$ = -200.00 kN (left)

$N_{j,c1,Ed}$ = -500.00 kN (below)

$N_{j,c2,Ed}$ = -100.00 kN (top)



partial safety factors for material

resistance of cross-sections γ_{M0} = 1.00

resistance of members in stability failure γ_{M1} = 1.10

resistance of bolts, welds, plates in bearing γ_{M2} = 1.25

prestressing of high strength bolts γ_{M7} = 1.10

notes

connection is verified due to EC 3-1-8 regardless of preloading.

however, connections may be constructed with prestressed high strength bolts.

for haunched beams, the bottom flange of the beam profile is not considered. a fictious profile is formed from the upper beam flange, the beam web and the haunch flange.

Only at calcualation of end plate the lower beam flange is respected as a stiffener.

no verification for cross-sections.

no verification of haunch connection to beam.

the welds of the haunch profile are not verified.

the welds are not regarded by calculation the T-stub resistance.

welds are not checked.

no verification of web stiffeners.

check of data

connection right:

ok

connection left:

ok

bolts right:

distances between bolts at end-plate

horizontal:	e_2 = 45.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_2 = 45.0 mm < $4 \cdot t$ + 40 mm = 108.0 mm
horizontal:	p_2 = 110.0 mm > $2.4 \cdot d_0$ = 62.4 mm,	p_2 = 110.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	e_1 = 50.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 50.0 mm < $4 \cdot t$ + 40 mm = 108.0 mm
top-below:	p_1 = 120.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 120.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	p_1 = 180.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 180.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	p_1 = 140.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 140.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	e_1 = 110.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 110.0 mm > $4 \cdot t$ + 40 mm = 108.0 mm !!

bolt distance from column edge

horizontal:	e_2 = 65.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_2 = 65.0 mm < $4 \cdot t$ + 40 mm = 108.0 mm

maximum values for spacings and edge distances should be in order to avoid local buckling and to prevent corrosion.

bolts left:

distances between bolts at end-plate

horizontal:	e_2 = 45.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_2 = 45.0 mm < $4 \cdot t$ + 40 mm = 108.0 mm
horizontal:	p_2 = 110.0 mm > $2.4 \cdot d_0$ = 62.4 mm,	p_2 = 110.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	e_1 = 50.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 50.0 mm < $4 \cdot t$ + 40 mm = 108.0 mm
top-below:	p_1 = 120.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 120.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	p_1 = 180.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 180.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	p_1 = 140.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 140.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm

top-below: e_1 = 110.0 mm > 1.2·d_0 = 31.2 mm, e_1 = 110.0 mm > 4·t + 40 mm = 108.0 mm
bolt distance from column edge
horizontal: e_2 = 65.0 mm > 1.2·d_0 = 31.2 mm, e_2 = 65.0 mm < 4·t + 40 mm = 108.0 mm
maximum values for spacings and edge distances should be in order to avoid local buckling and to prevent corrosion.

2. table of results

utilization/rotation of each joint (right)

Lc	$U_{\sigma,b}$	U_m	U_v	U_{ep}	U_{sb}	U	$S_{j,ini}$	S_j	φ_j
--	---	---	---	---	---	---	MNm/rad	MNm/rad	°
1	0.306	0.215	0.122	0.149	0.309	0.309	447.5	447.5	0.010

$U_{\sigma,b}$: stress utilization at the beam; U_m: utilization due to bending; U_v: utilization due to shear/bearing resistance
U_{ep}: utilization due to shear in end-plate; U_{sb}: utilization due to weld; U: utilization of each joint
U: utilization of each joint; $S_{j,ini}$: initial rotational stiffness; S_j: rotational stiffness
φ_j: rotation

utilization/rotation of each joint (left)

Lc	$U_{\sigma,b}$	U_m	U_v	U_{ep}	U_{sb}	U	$S_{j,ini}$	S_j	φ_j
--	---	---	---	---	---	---	MNm/rad	MNm/rad	°
1	0.306	0.215	0.122	0.149	0.309	0.309	447.5	447.5	0.010

$U_{\sigma,b}$: stress utilization at the beam; U_m: utilization due to bending; U_v: utilization due to shear/bearing resistance
U_{ep}: utilization due to shear in end-plate; U_{sb}: utilization due to weld; U: utilization of each joint
U: utilization of each joint; $S_{j,ini}$: initial rotational stiffness; S_j: rotational stiffness
φ_j: rotation

3. final result

utilization/rotation of the connection

	right			left				equilibrium			
Lc	$S_{j,ini}$	S_j	φ_j	$S_{j,ini}$	S_j	φ_j	U_j	ΣH	ΣV	ΣM	
--	MNm/rad	MNm/rad	°	MNm/rad	MNm/rad	°		kN	kN	kNm	
1	447.5	447.5	0.010	447.5	447.5	0.010	0.309*	0.00	0.00	0.00	ok

$S_{j,ini}$: initial rotational stiffness; S_j: rotational stiffness; φ_j: rotation; U_j: utilization of the connection; tolerances of equilibrium 1 kN / 1 kNm
*) maximum utilization

maximum utilization: max U = 0.309 < 1 ok
minimum rotational stiffness (right): min S_j = 447.5 MNm/rad, $S_{j,ini}$ = 447.5 MNm/rad, φ_j = 0.010°
minimum rotational stiffness (left): min S_j = 447.5 MNm/rad, $S_{j,ini}$ = 447.5 MNm/rad, φ_j = 0.010°

verification succeeded

4. Regulations

EN 1990, Eurocode 0: Grundlagen der Tragwerksplanung;
Deutsche Fassung EN 1990:2002 + A1:2005 + A1:2005/AC:2010, Ausgabe Dezember 2010
EN 1990/NA, Nationaler Anhang zur EN 1990, Ausgabe Dezember 2010

EN 1993-1-1, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau;
Deutsche Fassung EN 1993-1-1:2022, Ausgabe April 2025
EN 1993-1-1/NA, Nationaler appendix zur EN 1993-1-1, Ausgabe Oktober 2022

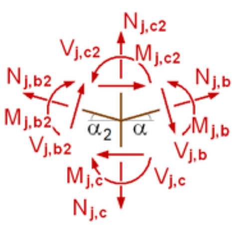
EN 1993-1-8, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-8: Bemessung von Anschlüssen;
Deutsche Fassung EN 1993-1-8:2024, Ausgabe April 2025
EN 1993-1-8/NA, Nationaler appendix zur EN 1993-1-8, Ausgabe November 2020

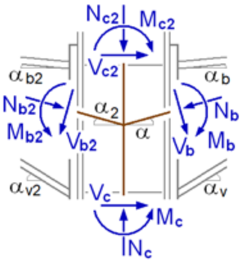
5. Lc 1 (decisive)

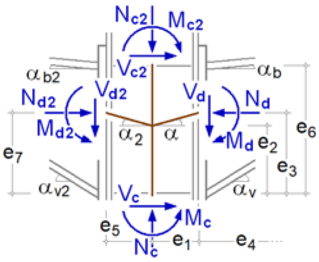
5.1. connection right

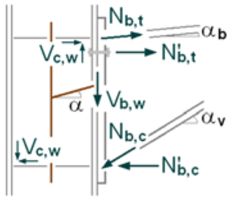
5.1.1. design values

internal forces at node | periphery connection ⊥ zur connection plane | partial internal forces and moments









slope angle: $\alpha_b = 0.00°$, $\alpha_v = 16.70°$ $\Rightarrow$ $\alpha = (\alpha_b+\alpha_v)/2 = 8.35°$
$\alpha_{b2} = 0.00°$, $\alpha_{v2} = 16.70°$ $\Rightarrow$ $\alpha_2 = (\alpha_{b2}+\alpha_{v2})/2 = 8.35°$
distance: $e_1 = 120.0$ mm, $e_3 = 234.4$ mm, $e_2 = 216.8$ mm, $e_5 = 120.0$ mm, $e_7 = 234.4$ mm, $e_6 = 466.4$ mm

internal forces and moments perpendicular to the connection planes

periphery beam (right)
$N_d = -29.04$ kN, $M_d = 75.74$ kNm, $V_d = 197.88$ kN
periphery beam (left)
$N_{d2} = -29.04$ kN, $M_{d2} = 75.74$ kNm, $V_{d2} = 197.88$ kN
periphery column (below)
$N_c = 500.00$ kN
periphery column (top)
$N_{c2} = 100.00$ kN

partial internal forces and moments

internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d \cdot t_p \cdot \tan(\alpha) - V_d \cdot t_p = 71.70$ kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b = 167.07$ kN, $z_b = 466.4$ mm, $z_{bu} = 214.3$ mm
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_v) = 144.10$ kN, $z_b = 466.4$ mm, $z_{bo} = 252.1$ mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = 0.00$ kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v) = 41.41$ kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 156.47$ kN

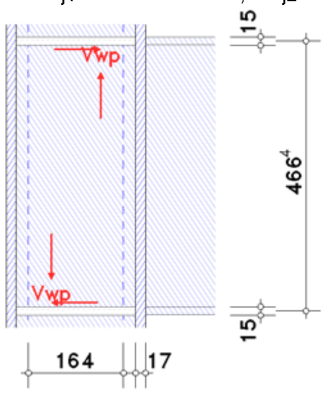
5.1.2. resistance of cross-section in the periphery

c/t-utilization reg. section class 2
flange below: section class 1, utilization $U_{c/t} = 0.457$
web: section class 1, utilization $U_{c/t} = 0.683$
total: section class 1, c/t-utilization $U_{c/t} = 0.683 < 1$ ok
plastic verification for $N_{Ed} = 29.04$ kN, $M_{y,Ed} = -71.70$ kNm, $V_{z,Ed} = 197.88$ kN
section class of the section $1 \le 2$ ok
shear buckling: $h_p/t_p = 60.38 > 72 \cdot \varepsilon/\eta = 48.82$ $\Rightarrow$ particular verification is required !!
Der cross-section ist plastic not acc. toweisbar !!
elastic verification for $N = 29.04$ kN, $M_y = -71.70$ kNm, $V_z = 197.88$ kN
verification: $\sigma_v = 108.81$ N/mm² $< \sigma_{v,Rd} = 355.00$ N/mm² $\Rightarrow$ $U_\sigma = 0.306 < 1$ ok
c/t-ratio: outstand flange: utilization $U_{c/t} = 0.145 < 1$ ok
internal compression parts: utilization $U_{c/t} = 0.215 < 1$ ok

5.1.3. basic components

5.1.3.1. bc 1: Column web panel in shear

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = |1 - M_{j2}/M_{j1} \cdot z_1/z_2| = 0.58 \le 2$
for $M_{j1} = 100.00$ kNm, $M_{j2} = 100.00$ kNm, $z_1 = 372.2$ mm, $z_2 = 234.8$ mm



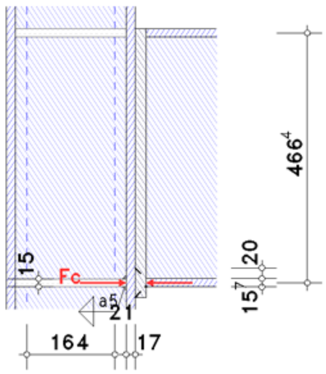
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

slenderness of column web $h_{wc}/t_{wc} = 20.60 < 72 \cdot \varepsilon/\eta = 48.82$ $\Rightarrow$ method applicable
plastic shear resistance without web stiffeners $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_{wp}) / (3^{1/2} \cdot \gamma_{M0}) = 442.71$ kN
Beitrag of column flange:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/z_{wp} = 110.8$ kN, $z_{wp} = h_r = 222.2$ mm
plastic shear resistance plus Beitrag of column flange $V_{wp,Rd} = 553.5$ kN
placing of intermediate web stiffeners:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/h_r + 2 \cdot M_{pl,st,Rd})/h_r = 110.83$ kN

plastic shear resistance with transverse stiffeners $V_{wp,Rd}$ = 664.36 kN

5.1.3.2. bc 2: column web in transverse compression

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = |1 - M_{j2}/M_{j1} \cdot z_1/z_2| = 0.58 \leq 2$
for M_{j1} = 100.00 kNm, M_{j2} = 100.00 kNm, z_1 = 372.2 mm, z_2 = 234.8 mm
longitudinal compressive stress in column web $\sigma_{com,Ed}$ = 47.18 N/mm²



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

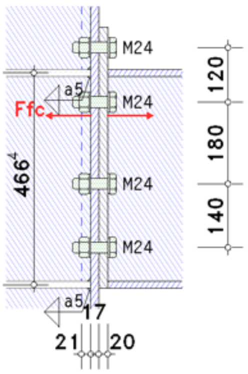
effective width of web in transverse compression $b_{eff,c} = t_{fb} + 5 \cdot (t_{fc}+s_c) + s_p$ = 245.7 mm
reduction factor k_w = 1.0 for $\sigma_{com,Ed}$ = 47.2 N/mm² $\leq 0.7 \cdot f_{y,w}$ = 248.5 N/mm²
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.769$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.928$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.941$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0}$ = 820.39 kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1}$ = 692.31 kN (decisive)
reinforcement of web with intermediate transverse stiffeners:
assumption: stiffeners do not buckle: section class 1, section class $1 \leq 3$ ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 466.4 mm
web height between the flanges h_{wc} = 206.0 mm
moment of inertia of stiffeners I_{st} = 1157.63 cm⁴
minimum moment of inertia for $a/h_{wc} = 2.26 \geq 2^{1/2}$: $I_{st,min}$ = 15.45 cm⁴ $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners I_T = 11.25 cm⁴
polar moment of inertia of stiffeners I_p = 127.81 cm⁴
$I_T / I_p \approx 0.088 > 0.009 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web A_{st} = 31.50 cm²
slenderness λ = 0.044
$\lambda \leq 0.2 \Rightarrow$ no deduction (χ = 1.0)
design value of resistance of flexural buckling $F_{c,w,Rd}$ = 1016.6 kN
maximum resistance:
$F_{c,w,Rd}$ = 1016.6 kN (resistance with transverse stiffeners)

resistance of the upper beam flange:
effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p$ = 261.3 mm
reduction factor k_w = 1.0 for $\sigma_{com,Ed}$ = 47.2 N/mm² $\leq 0.7 \cdot f_{y,w}$ = 248.5 N/mm²
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.793$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.911$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.937$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0}$ = 868.94 kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1}$ = 719.65 kN (decisive)
reinforcement of web with intermediate transverse stiffeners:
assumption: stiffeners do not buckle: section class 1, section class $1 \leq 3$ ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 466.4 mm
web height between the flanges h_{wc} = 206.0 mm
moment of inertia of stiffeners I_{st} = 1157.63 cm⁴
minimum moment of inertia for $a/h_{wc} = 2.26 \geq 2^{1/2}$: $I_{st,min}$ = 15.45 cm⁴ $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners I_T = 11.25 cm⁴
polar moment of inertia of stiffeners I_p = 127.81 cm⁴
$I_T / I_p \approx 0.088 > 0.009 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web A_{st} = 31.50 cm²
slenderness λ = 0.044
$\lambda \leq 0.2 \Rightarrow$ no deduction (χ = 1.0)
design value of resistance of flexural buckling $F_{c,w,Rd}$ = 1016.6 kN

maximum resistance:

$F_{c,w,Rd}$ = 1016.6 kN (resistance with transverse stiffeners)

5.1.3.3. bc 4: column flange in bending



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

equivalent T-stub flange (each individual bolt-row):

here: number of bolt-rows n_b = 1

row 1

effective length of the T-stub flange (column flange):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 208.6 mm, $l_{eff,cp}$ = 208.6 mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 224.7 mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 5.35 kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 5.76 kNm

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2}$ = 253.80 kN, k_2 = 0.90

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd}$ = 507.60 kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n))$ = 857.65 kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 436.33 kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 507.60 kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 436.33 kN

row 2

effective length of the T-stub flange (column flange):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 208.6 mm, $l_{eff,cp}$ = 208.6 mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 214.2 mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 5.35 kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 5.49 kNm

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2}$ = 253.80 kN, k_2 = 0.90

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd}$ = 507.60 kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n))$ = 857.65 kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 429.12 kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 507.60 kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 429.12 kN

row 3

effective length of the T-stub flange (column flange):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 208.6 mm, $l_{eff,cp}$ = 208.6 mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 214.1 mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 5.35 kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 5.49 kNm

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2}$ = 253.80 kN, k_2 = 0.90

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd}$ = 507.60 kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n))$ = 857.65 kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 428.99 kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 507.60 kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 428.99 kN

row 4

effective length of the T-stub flange (column flange):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 208.6 mm, $l_{eff,cp}$ = 208.6 mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 214.1 mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.35$ kNm
in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.49$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n-2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n-e_w \cdot (m+n)) = 857.65$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 428.99$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 428.99$ kN

resistances and effective lengths of column flange in bending (per bolt-row)

$F_{t,fc,Rd,1} = 436.33$ kN, $l_{eff,1} = 208.6$ mm
$F_{t,fc,Rd,2} = 429.12$ kN, $l_{eff,2} = 208.6$ mm
$F_{t,fc,Rd,3} = 428.99$ kN, $l_{eff,3} = 208.6$ mm
$F_{t,fc,Rd,4} = 428.99$ kN, $l_{eff,4} = 208.6$ mm

equivalent T-stub flange (group of bolts 1):

here: number of bolt-rows $n_b = 2$ (between stiffeners)
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 394.2$ mm, $\Sigma l_{eff,cp} = 568.6$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 394.2$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 10.11$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 1015.20$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n-2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n-e_w \cdot (m+n)) = 1620.88$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 834.73$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1015.20$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 834.73$ kN

equivalent T-stub flange (group of bolts 2):

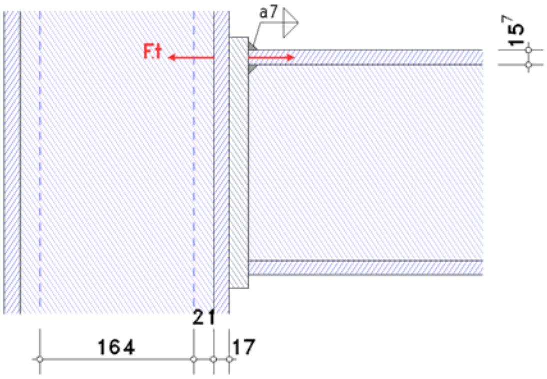
here: number of bolt-rows $n_b = 3$ (between stiffeners)
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 534.2$ mm, $\Sigma l_{eff,cp} = 848.6$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 534.2$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 13.70$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 1522.80$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n-2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n-e_w \cdot (m+n)) = 2196.48$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 1212.87$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1522.80$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 1212.87$ kN

resistances and effective lengths of column flange in bending (per bolt group):

$F_{ep,Rd,2\text{-}3} = 834.73$ kN, $\Sigma l_{eff} = 394.2$ mm, 2 rows
$F_{ep,Rd,2\text{-}4} = 1212.87$ kN, $\Sigma l_{eff} = 534.2$ mm, 3 rows

5.1.3.4. bc 3: column web in transverse tension

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = |1 - M_{j2}/M_{j1} \cdot z_1/z_2| = 0.58 \leq 2$
for $M_{j1} = 100.00$ kNm, $M_{j2} = 100.00$ kNm, $z_1 = 372.2$ mm, $z_2 = 234.8$ mm



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

each individual bolt-row:

row 1
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.951$

resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm

row 2
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.951$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm

row 3
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.951$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm

row 4
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.951$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm

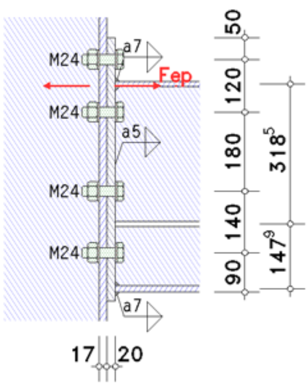
resistance of a column web with transverse tension (per bolt-row)
$F_{t,wc,Rd,1} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm (s. bc 4)
$F_{t,wc,Rd,2} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm (s. bc 4)
$F_{t,wc,Rd,3} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm (s. bc 4)
$F_{t,wc,Rd,4} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm (s. bc 4)

group of bolt-rows, group 1:
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.910$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 1273.89$ kN, $b_{eff,t,wc} = 394.2$ mm

group of bolt-rows, group 2:
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.893$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 1692.66$ kN, $b_{eff,t,wc} = 534.2$ mm

resistances of a column web with transverse tension (per bolt group):
$F_{t,wc,Rd,2\text{-}3} = 1273.89$ kN, $\Sigma b_{eff,t,wc} = 394.2$ mm (s. bc 4), 2 rows
$F_{t,wc,Rd,2\text{-}4} = 1692.66$ kN, $\Sigma b_{eff,t,wc} = 534.2$ mm (s. bc 4), 3 rows

5.1.3.5. bc 5: end-plate in bending



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

extended part of end-plate
in the extended part of the end-plate only one bolt-row is considered ($n_b = 1$).
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 100.0$ mm, $l_{eff,cp} = 222.2$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 100.0$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 3.55$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 419.98$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 352.74$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 352.74$ kN
resistance and effective length of end-plate in bending (projection)
$F_{t,ep,Rd,1} = 352.74$ kN, $l_{eff,1} = 100.0$ mm
part of end-plate between beam flanges
equivalent T-stub flange (each individual bolt-row):
here: number of bolt-rows $n_b = 1$
row 2
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 252.5$ mm, $l_{eff,cp} = 286.5$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 252.5$ mm
tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.96$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 975.17$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 450.03$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 450.03$ kN

row 3
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 239.7$ mm, $l_{eff,cp} = 286.5$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 239.7$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.51$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 925.60$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 439.97$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 439.97$ kN

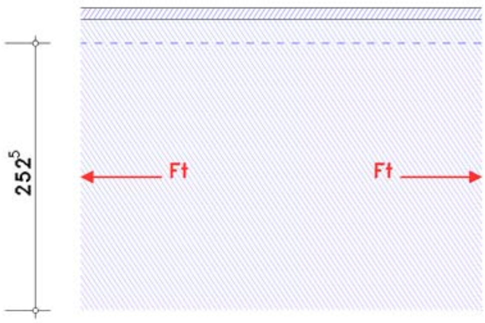
row 4
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 245.0$ mm, $l_{eff,cp} = 286.5$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 245.0$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.70$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 946.03$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 444.11$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 444.11$ kN

resistances and effective lengths of end-plate in bending (per bolt-row):
$F_{ep,Rd,2} = 450.03$ kN, $l_{eff,2} = 252.5$ mm
$F_{ep,Rd,3} = 439.97$ kN, $l_{eff,3} = 239.7$ mm
$F_{ep,Rd,4} = 444.11$ kN, $l_{eff,4} = 245.0$ mm

equivalent T-stub flange (group of bolts 1):
here: number of bolt-rows $n_b = 2$ (R2+R3)
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 433.5$ mm, $\Sigma l_{eff,cp} = 646.5$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 433.5$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 15.39$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 1015.20$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 1674.36$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 844.05$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1015.20$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 844.05$ kN

resistances and effective lengths of end-plate in bending (per bolt group):
$F_{ep,Rd,2\text{-}3} = 844.05$ kN, $\Sigma l_{eff} = 433.5$ mm, 2 rows

5.1.3.6. bc 8: beam web in tension



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

each individual bolt-row:

row 2

effective width $b_{eff,t,wb}$ = 252.5 mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 672.28 kN

row 3

effective width $b_{eff,t,wb}$ = 239.7 mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 638.10 kN

row 4

effective width $b_{eff,t,wb}$ = 245.0 mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 652.18 kN

resistance of a beam web in tension (per bolt-row)

$F_{t,wb,Rd,2}$ = 672.28 kN, $b_{eff,t,wb}$ = 252.5 mm (s. bc 5)

$F_{t,wb,Rd,3}$ = 638.10 kN, $b_{eff,t,wb}$ = 239.7 mm (s. bc 5)

$F_{t,wb,Rd,4}$ = 652.18 kN, $b_{eff,t,wb}$ = 245.0 mm (s. bc 5)

group of bolt-rows, group 1:

effective width $b_{eff,t,wb}$ = 433.5 mm (l_{eff} from bc 5)

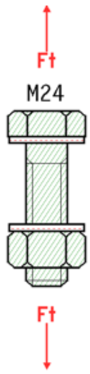
resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 1154.29 kN

resistances of a beam web in tension (per bolt group):

$F_{t,wb,Rd,2\text{-}3}$ = 1154.29 kN, $\Sigma b_{eff,t,wb}$ = 433.5 mm (s. bc 5), 2 rows

5.1.3.7. bc 10: bolts in tension



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

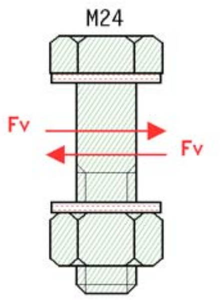
tension resistance of one bolt $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2}$ = 253.80 kN, k_2 = 0.90

punching shear load capicity of a bolt $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2}$ = 541.39 kN, t_p = 17.0 mm

tension-/punching shear load capicity for 2 bolts: $\Sigma F_{tp,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd})$ = 507.60 kN

5.1.3.8. bc 11: bolts in shear

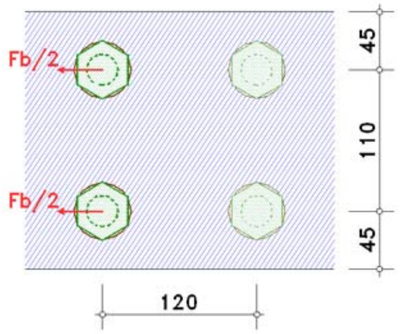
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



shear resistance $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A \,/\, \gamma_{M2}$ = 217.15 kN, α_v = 0.60
shear resistance of 2 bolts (1-shear): $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd}$ = 434.29 kN

5.1.3.9. bc 12: plate with bearing resistance

Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



row 1
end-plate:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 361.85 kN, k_m = 1.00, α_b = 1.92
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 361.85 kN, k_m = 1.00, α_b = 1.92
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 723.69 kN
<u>shear block of groups of bolts</u>
shearversagen in Kombination in tensionversagen of plate
failure type 1:
tension-shear resistance $V_{eff,Rd} = (A_{nt} \cdot f_u + \min(A_{gv} \cdot f_y,\ A_{nv} \cdot f_u)/3^{1/2}) \,/\, \gamma_{M2}$ = 822.53 kN
failure type 2:
tension-shear resistance $V_{eff,Rd} = (A_{nt} \cdot f_u + \min(A_{gv} \cdot f_y,\ A_{nv} \cdot f_u)/3^{1/2}) \,/\, \gamma_{M2}$ = 665.73 kN
failure type 3:
tension-shear resistance $V_{eff,Rd} = (0.5 \cdot A_{nt} \cdot f_u + \min(A_{gv} \cdot f_y,\ A_{nv} \cdot f_u)/3^{1/2}) \,/\, \gamma_{M2}$ = 567.73 kN
failure type 4:
tension resistance (without shearanteil) $V_{eff,Rd} = (A_{nt} \cdot f_u) \,/\, \gamma_{M2}$ = 1160.32 kN
resistance due to shear block: min $V_{eff,Rd}$ = 567.7 kN
<u>total</u>
bearing resistance incl. shear block: $\min(\Sigma F_{b,Rd},\ \min V_{eff,Rd})$ = 567.7 kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 852.99 kN
row 2
end-plate:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 564.48 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 564.48 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 1003.52 kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 852.99 kN
row 3
end-plate:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 564.48 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 564.48 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 1003.52 kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) \,/\, \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 852.99 kN

row 4

end-plate:

bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 564.48$ kN, $k_m = 1.00$, $\alpha_b = 3.00$

bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 564.48$ kN, $k_m = 1.00$, $\alpha_b = 3.00$

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 1003.52$ kN

column flange:

bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 479.81$ kN, $k_m = 1.00$, $\alpha_b = 3.00$

bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 479.81$ kN, $k_m = 1.00$, $\alpha_b = 3.00$

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 852.99$ kN

bearing resistance (4 rows)

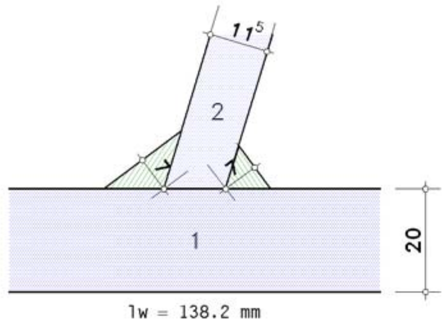
$\Sigma F_{b,Rd,1} = 567.73$ kN

$\Sigma F_{b,Rd,2} = 852.99$ kN

$\Sigma F_{b,Rd,3} = 852.99$ kN

$\Sigma F_{b,Rd,4} = 852.99$ kN

5.1.3.10. bc 19: weld



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

calculation with the simplified method

beam flange in compression

resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 1760.28$ kN/m, $f_{vw,d} = 251.5$ N/mm², $a = 7.0$ mm

total $F_{w,Rd} = 486.7$ kN

5.1.3.11. bc 20: haunched beam in compression

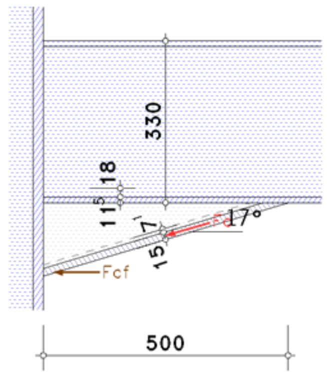
flange below: section class 1

web: section class 1

total: section class 1

section class of the beam in connection plane: 1

taking into account the moment-shear force-interaction $V_{Ed} = 200.0$ kN



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

assumption: haunch flange no hazard of buckling: 1 < 3 ok

connection haunch-column: (basic component 7: beam flange and web in compression)

beam height incl. haunch $h = h_b + h_v = 480.0$ mm

resistance only flange (without web, plastic: section class 1)

haunch flange width $b_{f,v} = 160.0$ mm < max $b_{f,v} = 535.1$ mm ok

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 413.04$ kNm, $W_{pl} = 1163.51$ cm³

resistance of flange and web in compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 889.52$ kN

refering to haunch flange $F_{c,f,Rd}/\cos(\alpha_v) = 928.69$ kN

total loading capacity of a haunched beam in compression

$F_{c,v,Rd} = F_{c,f,Rd} = 928.69$ kN

resistance refering to die connection plane $F_{c,v,Rd} \cdot \cos(\alpha_v) = 889.52$ kN

resistance of the upper beam flange (bc 7):

stress due to bending with shear force: $V_{Ed} = 200.0$ kN ≤ 351.3 kN $= 0.5 \cdot V_{pl,Rd}/2 \Rightarrow$ no effect

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 444.99$ kNm, $W_{pl} = 1253.49$ cm³

resistance of flange and web in compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 949.80$ kN

5.1.4. connection capacity

transformation parameter: $\beta_j = 0.58$

5.1.4.1. moment resistance

distance of tension-bolt-rows from centre of compression:
$h_1 = 522.2$ mm, $h_2 = 402.2$ mm, $h_3 = 222.2$ mm, $h_4 = 82.2$ mm

resistances acc. to EC 3-1-8, B.3.2.2(6) for bolt-rows considered individually
decisive basic components: 3, 4, 5, 8
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 429.0$ kN
row 4: $F_{tr,Rd} = 429.0$ kN

deductions acc. to EC 3-1-8, B.3.2.2(8) for bolt-rows as part of a group (column)
decisive basic components: 3, 4
group 1
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 405.6$ kN
group 2
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 405.6$ kN
row 4: $F_{tr,Rd} = 378.1$ kN

deductions acc. to EC 3-1-8, B.3.2.2(8) for bolt-rows as part of a group (end-plate)
decisive basic components: 5, 8
group 1
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 405.6$ kN

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 405.6$ kN
row 4: $F_{tr,Rd} = 378.1$ kN
$\Sigma F_{tr,Rd}^* = 1565.6$ kN

deductions acc. to EC 3-1-8, B.3.2.2(7)
decisive basic components: 1, 2, 20
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 107.7$ kN
row 4: $F_{tr,Rd} = 0.0$ kN

check acc. to EC 3-1-8, B.3.2.2(9)
decisive basic component: 10
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 107.7$ kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 107.7$ kN
row 4: $F_{tr,Rd} = 0.0$ kN
$\Sigma F_{tr,Rd} = 889.5$ kN
potential failure by basic component 4, 5, 20

resistance of flanges (compression)
$\Sigma F_{c,Rd}^* = 1839.3$ kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 380.7$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 1565.6$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 1839.3$ kN

5.1.4.2. shear/bearing resistance

resistance per bolt-row

decisive basic components: 11, 12

row 1: $F_{vr,Rd}$ = 434.3 kN
row 2: $F_{vr,Rd}$ = 434.3 kN
row 3: $F_{vr,Rd}$ = 434.3 kN
row 4: $F_{vr,Rd}$ = 434.3 kN

deductions depending on tension force (at full utilization of moment resistance)

decisive basic component: 10

row 1: $F_{vr,Rd}$ = 218.7 kN
row 2: $F_{vr,Rd}$ = 172.0 kN
row 3: $F_{vr,Rd}$ = 368.5 kN
row 4: $F_{vr,Rd}$ = 434.3 kN

resistance per bolt-row (shear/bearing resistance)

row 1: $F_{vr,Rd}$ = 218.7 kN
row 2: $F_{vr,Rd}$ = 172.0 kN
row 3: $F_{vr,Rd}$ = 368.5 kN
row 4: $F_{vr,Rd}$ = 434.3 kN
$\Sigma F_{vr,Rd}$ = 1193.6 kN

shear/bearing resistance

$V_{j,Rd} = \Sigma F_{vr,Rd}$ = 1193.6 kN

5.1.4.3. shear resistance

shear resistance of end plate

end-plate: $V_{ep,Rd}$ = 1708.73 kN
welds: $F_{w,Rd}$ = 1048.24 kN
shear resistance: $V_{ep,Rd} = F_{w,Rd}$ = 1048.24 kN

shear resistance of column web

decisive basic component: 1

$V_{wp,Rd}$ = 664.36 kN

5.1.4.4. total

$M_{j,Rd}$ = 380.7 kNm $N_{j,t,Rd}$ = 1565.6 kN $N_{j,c,Rd}$ = 1839.3 kN $V_{j,Rd}$ = 1193.6 kN $V_{wp,Rd}$ = 664.4 kN $V_{ep,Rd}$ = 1048.2 kN

5.1.5. verifications

5.1.5.1. verification of the connection capacity by means of the component method

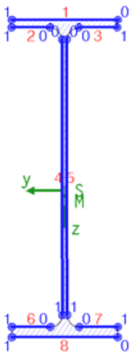
internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu}$ = 81.88 kNm, z_{bu} = 211.4 mm
shear force: $V_{Ed} = |V_d|$ = 197.88 kN
shear force: $V_{b,w,Ed}$ = 156.47 kN

$M_{Ed}/M_{j,Rd} = 0.215 < 1$ ok
shear/bearing resistance at 21.5% utilization of moment resistance $V_{j,Rd}$ = 1620.3 kN
$V_{Ed}/V_{j,Rd} = 0.122 < 1$ ok
$V_{b,w,Ed}/V_{ep,Rd} = 0.149 < 1$ ok

5.1.5.2. verification of welds at beam section

weld 1: beam flange in tension outer
welds 2,3: beam flange in tension inner
welds 4,5: beam web double-sided
weld 8: beam flange in compression outer
welds 6,7: beam flange in compression inner

calculation section:



weld 1: a_w = 7.0 mm l_w = 160.0 mm
weld 2: a_w = 7.0 mm l_w = 58.3 mm
weld 3: see weld 2
weld 4: a_w = 5.0 mm l_w = 416.8 mm
weld 5: see weld 4
weld 6: a_w = 7.0 mm l_w = 58.3 mm
weld 7: see weld 6
weld 8: a_w = 7.0 mm l_w = 160.0 mm

design values refering to centroid of the section:

N_{Ed} = 29.04 kN, $M_{y,Ed}$ = -75.74 kNm, $V_{z,Ed}$ = 197.88 kN

cross-sectional properties refering to centroid of the line cross-section:
ΣA_w = 80.39 cm², $A_{w,z}$ = 41.68 cm², Σl_w = 138.7 cm
$I_{w,y}$ = 27301.20 cm⁴, $I_{w,z}$ = 951.99 cm⁴, Δz_w = -19.3 mm
distribution of internal forces and moments:
weld 1: N_w = 78.16 kN
weld 2: N_w = 27.15 kN
weld 3: see weld 2
weld 4: N_w = 7.86 kN $M_{y,w}$ = -8.37 kNm
weld 5: see weld 4
weld 6: N_w = -24.08 kN
weld 7: see weld 6
weld 8: N_w = -71.00 kN
from conventionel distribution of shear force: $V_{z,w}$ = 197.88 kN
verifications in weld edges:
weld 1, pt. 0: $\sigma_{w,x}$ = 69.78 N/mm² $\Rightarrow U_w = 0.277 < 1$ ok
weld 2, pt. 0: $\sigma_{w,x}$ = 66.59 N/mm² $\Rightarrow U_w = 0.265 < 1$ ok
weld 4, pt. 0: $\sigma_{w,x}$ = 61.60 N/mm² $\tau_{w,z}$ = 47.47 N/mm² $\Rightarrow U_w = 0.309 < 1$ ok
pt. 1: $\sigma_{w,x}$ = -54.05 N/mm² $\tau_{w,z}$ = 47.47 N/mm² $\Rightarrow U_w = 0.286 < 1$ ok
weld 6, pt. 0: $\sigma_{w,x}$ = -59.04 N/mm² $\Rightarrow U_w = 0.235 < 1$ ok
weld 8, pt. 0: $\sigma_{w,x}$ = -63.39 N/mm² $\Rightarrow U_w = 0.252 < 1$ ok
Result:
weld 4, pt. 0: $\sigma_{w,x}$ = 61.60 N/mm² $\tau_{w,z}$ = 47.47 N/mm²
Max: $F_{w,Ed}$ = 388.83 kN/m $<$ $F_{w,Rd}$ = 1257.34 kN/m $\Rightarrow U_w = 0.309 < 1$ ok

verification of deflection forces (bc 19, simplified method)
compression flange: $F_{Rd} = F_{w,Rd}$ = 486.7 kN, F_{Ed} = 144.10 kN
F_{Ed} = 144.1 kN $<$ F_{Rd} = 486.7 kN $\Rightarrow U_{w,f} = 0.296 < 1$ ok

5.1.5.3. verification result

maximum utilization: max U = 0.309 $<$ 1 ok

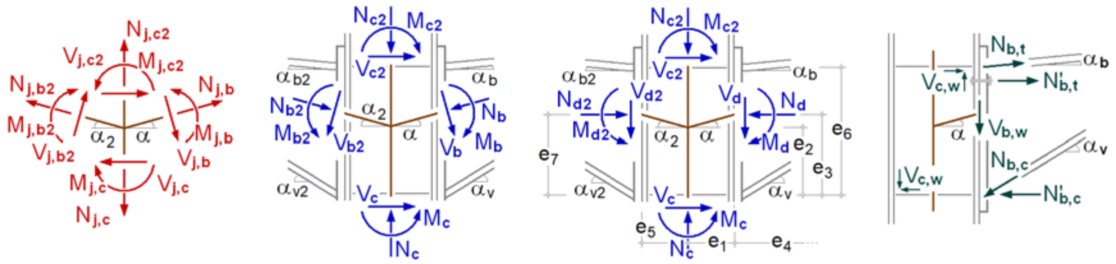
5.1.6. rotational stiffness

stiffness coefficients
equivalent stiffness coefficient for 4 tension-bolt-rows:
1: k_3 = 8.90 mm, k_4 = 25.21 mm, k_5 = 9.66 mm $\Rightarrow k_{eff,1} = 1 / \Sigma(1/k_{i,1})$ = 3.914 mm
2: k_3 = 8.90 mm, k_4 = 25.21 mm, k_5 = 19.18 mm $\Rightarrow k_{eff,2} = 1 / \Sigma(1/k_{i,2})$ = 4.899 mm
3: k_3 = 8.90 mm, k_4 = 25.21 mm, k_5 = 18.21 mm $\Rightarrow k_{eff,3} = 1 / \Sigma(1/k_{i,3})$ = 4.833 mm
4: k_3 = 8.90 mm, k_4 = 25.21 mm, k_5 = 18.61 mm $\Rightarrow k_{eff,4} = 1 / \Sigma(1/k_{i,4})$ = 4.861 mm
equivalent internal lever arm $z_{eq} = \Sigma(k_{eff,r}\cdot h_r^2) / \Sigma(k_{eff,r}\cdot h_r)$ = 388.36 mm
äquivalenter stiffness coefficient $k_{eq} = \Sigma(k_{eff,r}\cdot h_r) / z_{eq}$ = 14.130 mm
$k_2 = \infty$ (stiffened)
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E\cdot z^2) / \Sigma(1/k_i)$ = 447529.3 kNm/rad, $z = z_{eq}$ = 388.4 mm, $\Sigma(1/k_i)$ = 0.071 mm⁻¹
$|M_{j,Ed}|$ = 81.88 kNm $\leq$ 2/3 $M_{j,Rd}$ = 253.79 kNm $\Rightarrow \mu = 1$
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu$ = 447529.3 kNm/rad
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd}$ = 0.010°

5.2. connection left

5.2.1. design values

internal forces at node periphery connection ⊥ zur connection plane partial internal forces and moments



slope angle: α_b = 0.00°, α_v = 16.70° $\Rightarrow \alpha = (\alpha_b+\alpha_v)/2$ = 8.35°
α_{b2} = 0.00°, α_{v2} = 16.70° $\Rightarrow \alpha_2 = (\alpha_{b2}+\alpha_{v2})/2$ = 8.35°
distance: e_1 = 120.0 mm, e_3 = 234.4 mm, e_2 = 216.8 mm, e_5 = 120.0 mm, e_7 = 234.4 mm, e_6 = 466.4 mm

internal forces and moments perpendicular to the connection planes
periphery beam (right)
N_d = -29.04 kN, M_d = 75.74 kNm, V_d = 197.88 kN
periphery beam (left)
N_{d2} = -29.04 kN, M_{d2} = 75.74 kNm, V_{d2} = 197.88 kN
periphery column (below)

N_c = 500.00 kN
periphery column (top)
N_{c2} = 100.00 kN

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d \cdot t_p \cdot \tan(\alpha) - V_d \cdot t_p$ = 71.70 kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 167.07 kN, z_b = 466.4 mm, z_{bu} = 214.3 mm
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_v)$ = 144.10 kN, z_b = 466.4 mm, z_{bo} = 252.1 mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b)$ = 0.00 kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v)$ = 41.41 kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c}$ = 156.47 kN

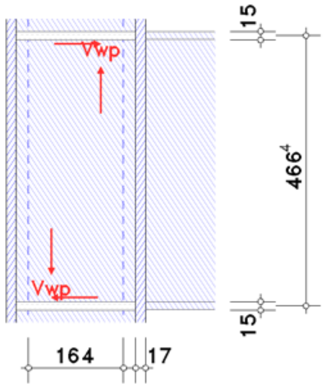
5.2.2. resistance of cross-section in the periphery

c/t-utilization reg. section class 2
flange below: section class 1, utilization $U_{c/t}$ = 0.457
web: section class 1, utilization $U_{c/t}$ = 0.683
total: section class 1, c/t-utilization $U_{c/t}$ = 0.683 < 1 ok
plastic verification for N_{Ed} = 29.04 kN, $M_{y,Ed}$ = -71.70 kNm, $V_{z,Ed}$ = 197.88 kN
section class of the section 1 ≤ 2 ok
shear buckling: h_p/t_p = 60.38 > $72 \cdot \varepsilon/\eta$ = 48.82 ⇒ particular verification is required !!
Der cross-section ist plastic not acc. toweisbar !!
elastic verification for N = 29.04 kN, M_y = -71.70 kNm, V_z = 197.88 kN
verification: σ_v = 108.81 N/mm² < $\sigma_{v,Rd}$ = 355.00 N/mm² ⇒ U_σ = 0.306 < 1 ok
c/t-ratio: outstand flange: utilization $U_{c/t}$ = 0.145 < 1 ok
internal compression parts: utilization $U_{c/t}$ = 0.215 < 1 ok

5.2.3. basic components

5.2.3.1. bc 1: Column web panel in shear

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = |1 - M_{j2}/M_{j1} \cdot z_1/z_2| = 0.58 \le 2$
for M_{j1} = 100.00 kNm, M_{j2} = 100.00 kNm, z_1 = 372.2 mm, z_2 = 234.8 mm

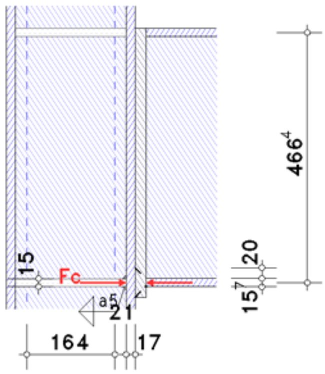


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

slenderness of column web h_{wc}/t_{wc} = 20.60 < $72 \cdot \varepsilon/\eta$ = 48.82 ⇒ method applicable
plastic shear resistance without web stiffeners $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_{wp}) / (3^{1/2} \cdot \gamma_{M0})$ = 442.71 kN
Beitrag of column flange:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/z_{wp}$ = 110.8 kN, $z_{wp} = h_r$ = 222.2 mm
plastic shear resistance plus Beitrag of column flange $V_{wp,Rd}$ = 553.5 kN
placing of intermediate web stiffeners:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/h_r + 2 \cdot M_{pl,st,Rd})/h_r$ = 110.83 kN
plastic shear resistance with transverse stiffeners $V_{wp,Rd}$ = 664.36 kN

5.2.3.2. bc 2: column web in transverse compression

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = |1 - M_{j2}/M_{j1} \cdot z_1/z_2| = 0.58 \leq 2$
for $M_{j1} = 100.00$ kNm, $M_{j2} = 100.00$ kNm, $z_1 = 372.2$ mm, $z_2 = 234.8$ mm
longitudinal compressive stress in column web $\sigma_{com,Ed} = 47.18$ N/mm²



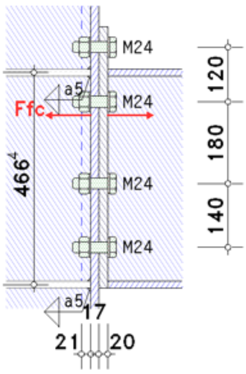
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

effective width of web in transverse compression $b_{eff,c} = t_{fb} + 5 \cdot (t_{fc}+s_c) + s_p = 245.7$ mm
reduction factor $k_w = 1.0$ for $\sigma_{com,Ed} = 47.2$ N/mm² $\leq 0.7 \cdot f_{y,w} = 248.5$ N/mm²
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.769$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.928$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.941$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 820.39$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 692.31$ kN (decisive)
reinforcement of web with intermediate transverse stiffeners:
assumption: stiffeners do not buckle: section class 1, section class $1 \leq 3$ ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 466.4 mm
web height between the flanges $h_{wc} = 206.0$ mm
moment of inertia of stiffeners $I_{st} = 1157.63$ cm⁴
minimum moment of inertia for $a/h_{wc} = 2.26 \geq 2^{1/2}$: $I_{st,min} = 15.45$ cm⁴ $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners $I_T = 11.25$ cm⁴
polar moment of inertia of stiffeners $I_p = 127.81$ cm⁴
$I_T / I_p \approx 0.088 > 0.009 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web $A_{st} = 31.50$ cm²
slenderness $\lambda = 0.044$
$\lambda \leq 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd} = 1016.6$ kN
maximum resistance:
$F_{c,w,Rd} = 1016.6$ kN (resistance with transverse stiffeners)

resistance of the upper beam flange:
effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p = 261.3$ mm
reduction factor $k_w = 1.0$ for $\sigma_{com,Ed} = 47.2$ N/mm² $\leq 0.7 \cdot f_{y,w} = 248.5$ N/mm²
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.793$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.911$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.937$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 868.94$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 719.65$ kN (decisive)
reinforcement of web with intermediate transverse stiffeners:
assumption: stiffeners do not buckle: section class 1, section class $1 \leq 3$ ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 466.4 mm
web height between the flanges $h_{wc} = 206.0$ mm
moment of inertia of stiffeners $I_{st} = 1157.63$ cm⁴
minimum moment of inertia for $a/h_{wc} = 2.26 \geq 2^{1/2}$: $I_{st,min} = 15.45$ cm⁴ $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners $I_T = 11.25$ cm⁴
polar moment of inertia of stiffeners $I_p = 127.81$ cm⁴
$I_T / I_p \approx 0.088 > 0.009 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web $A_{st} = 31.50$ cm²
slenderness $\lambda = 0.044$
$\lambda \leq 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd} = 1016.6$ kN
maximum resistance:

$F_{c,w,Rd} = 1016.6$ kN (resistance with transverse stiffeners)

5.2.3.3. bc 4: column flange in bending



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

equivalent T-stub flange (each individual bolt-row):

here: number of bolt-rows $n_b = 1$

row 1

effective length of the T-stub flange (column flange):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 208.6$ mm, $l_{eff,cp} = 208.6$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 224.7$ mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.35$ kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.76$ kNm

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 857.65$ kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 436.33$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 436.33$ kN

row 2

effective length of the T-stub flange (column flange):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 208.6$ mm, $l_{eff,cp} = 208.6$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 214.2$ mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.35$ kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.49$ kNm

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 857.65$ kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 429.12$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 429.12$ kN

row 3

effective length of the T-stub flange (column flange):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 208.6$ mm, $l_{eff,cp} = 208.6$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 214.1$ mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.35$ kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.49$ kNm

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 857.65$ kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 428.99$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 428.99$ kN

row 4

effective length of the T-stub flange (column flange):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 208.6$ mm, $l_{eff,cp} = 208.6$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 214.1$ mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.35$ kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.49$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 857.65$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 428.99$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 428.99$ kN

resistances and effective lengths of column flange in bending (per bolt-row)

$F_{t,fc,Rd,1} = 436.33$ kN, $l_{eff,1} = 208.6$ mm
$F_{t,fc,Rd,2} = 429.12$ kN, $l_{eff,2} = 208.6$ mm
$F_{t,fc,Rd,3} = 428.99$ kN, $l_{eff,3} = 208.6$ mm
$F_{t,fc,Rd,4} = 428.99$ kN, $l_{eff,4} = 208.6$ mm

equivalent T-stub flange (group of bolts 1):
here: number of bolt-rows $n_b = 2$ (between stiffeners)
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 394.2$ mm, $\Sigma l_{eff,cp} = 568.6$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 394.2$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 10.11$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 1015.20$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 1620.88$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 834.73$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1015.20$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 834.73$ kN

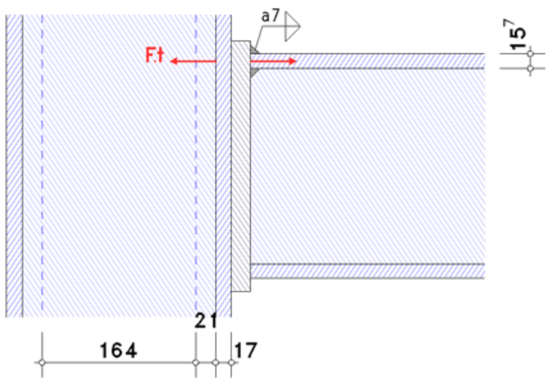
equivalent T-stub flange (group of bolts 2):
here: number of bolt-rows $n_b = 3$ (between stiffeners)
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 534.2$ mm, $\Sigma l_{eff,cp} = 848.6$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 534.2$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 13.70$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 1522.80$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 2196.48$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 1212.87$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1522.80$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 1212.87$ kN

resistances and effective lengths of column flange in bending (per bolt group):

$F_{ep,Rd,2-3} = 834.73$ kN, $\Sigma l_{eff} = 394.2$ mm, 2 rows
$F_{ep,Rd,2-4} = 1212.87$ kN, $\Sigma l_{eff} = 534.2$ mm, 3 rows

5.2.3.4. bc 3: column web in transverse tension

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = |1 - M_{j2}/M_{j1} \cdot z_1/z_2| = 0.58 \leq 2$
for $M_{j1} = 100.00$ kNm, $M_{j2} = 100.00$ kNm, $z_1 = 372.2$ mm, $z_2 = 234.8$ mm



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

each individual bolt-row:
row 1
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.951$
resistance eines unstiffeneden column webs with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm
row 2
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.951$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm
row 3
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.951$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm
row 4
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.951$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm

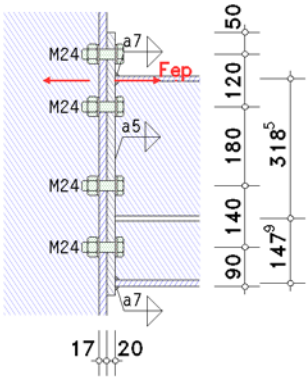
resistance of a column web with transverse tension (per bolt-row)
$F_{t,wc,Rd,1} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm (s. bc 4)
$F_{t,wc,Rd,2} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm (s. bc 4)
$F_{t,wc,Rd,3} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm (s. bc 4)
$F_{t,wc,Rd,4} = 704.13$ kN, $b_{eff,t,wc} = 208.6$ mm (s. bc 4)

group of bolt-rows, group 1:
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.910$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 1273.89$ kN, $b_{eff,t,wc} = 394.2$ mm

group of bolt-rows, group 2:
reduction factor for interaction with shear stress $0.5 < \beta < 1 \Rightarrow \omega = 0.893$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 1692.66$ kN, $b_{eff,t,wc} = 534.2$ mm

resistances of a column web with transverse tension (per bolt group):
$F_{t,wc,Rd,2-3} = 1273.89$ kN, $\Sigma b_{eff,t,wc} = 394.2$ mm (s. bc 4), 2 rows
$F_{t,wc,Rd,2-4} = 1692.66$ kN, $\Sigma b_{eff,t,wc} = 534.2$ mm (s. bc 4), 3 rows

5.2.3.5. bc 5: end-plate in bending



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

extended part of end-plate
in the extended part of the end-plate only one bolt-row is considered ($n_b = 1$).
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 100.0$ mm, $l_{eff,cp} = 222.2$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 100.0$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 3.55$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 419.98$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 352.74$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 352.74$ kN
resistance and effective length of end-plate in bending (projection)
$F_{t,ep,Rd,1} = 352.74$ kN, $l_{eff,1} = 100.0$ mm

part of end-plate between beam flanges
equivalent T-stub flange (each individual bolt-row):
here: number of bolt-rows $n_b = 1$
row 2
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 252.5$ mm, $l_{eff,cp} = 286.5$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 252.5$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.96$ kNm

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n-2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n-e_w \cdot (m+n)) = 975.17$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 450.03$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 450.03$ kN

row 3
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 239.7$ mm, $l_{eff,cp} = 286.5$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 239.7$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.51$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n-2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n-e_w \cdot (m+n)) = 925.60$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 439.97$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 439.97$ kN

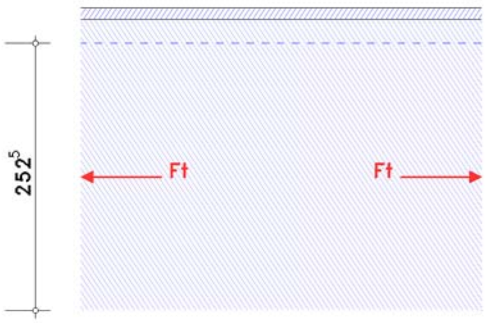
row 4
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 245.0$ mm, $l_{eff,cp} = 286.5$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 245.0$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.70$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n-2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n-e_w \cdot (m+n)) = 946.03$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 444.11$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 444.11$ kN

resistances and effective lengths of end-plate in bending (per bolt-row):
$F_{ep,Rd,2} = 450.03$ kN, $l_{eff,2} = 252.5$ mm
$F_{ep,Rd,3} = 439.97$ kN, $l_{eff,3} = 239.7$ mm
$F_{ep,Rd,4} = 444.11$ kN, $l_{eff,4} = 245.0$ mm

equivalent T-stub flange (group of bolts 1):
here: number of bolt-rows $n_b = 2$ (R2+R3)
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 433.5$ mm, $\Sigma l_{eff,cp} = 646.5$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 433.5$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 15.39$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 1015.20$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n-2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n-e_w \cdot (m+n)) = 1674.36$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 844.05$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1015.20$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 844.05$ kN

resistances and effective lengths of end-plate in bending (per bolt group):
$F_{ep,Rd,2-3} = 844.05$ kN, $\Sigma l_{eff} = 433.5$ mm, 2 rows

5.2.3.6. bc 8: beam web in tension



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

each individual bolt-row:

row 2

effective width $b_{eff,t,wb}$ = 252.5 mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 672.28 kN

row 3

effective width $b_{eff,t,wb}$ = 239.7 mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 638.10 kN

row 4

effective width $b_{eff,t,wb}$ = 245.0 mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 652.18 kN

resistance of a beam web in tension (per bolt-row)

$F_{t,wb,Rd,2}$ = 672.28 kN, $b_{eff,t,wb}$ = 252.5 mm (s. bc 5)

$F_{t,wb,Rd,3}$ = 638.10 kN, $b_{eff,t,wb}$ = 239.7 mm (s. bc 5)

$F_{t,wb,Rd,4}$ = 652.18 kN, $b_{eff,t,wb}$ = 245.0 mm (s. bc 5)

group of bolt-rows, group 1:

effective width $b_{eff,t,wb}$ = 433.5 mm (l_{eff} from bc 5)

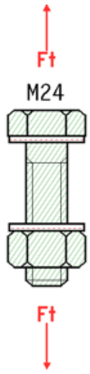
resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 1154.29 kN

resistances of a beam web in tension (per bolt group):

$F_{t,wb,Rd,2-3}$ = 1154.29 kN, $\Sigma b_{eff,t,wb}$ = 433.5 mm (s. bc 5), 2 rows

5.2.3.7. bc 10: bolts in tension



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

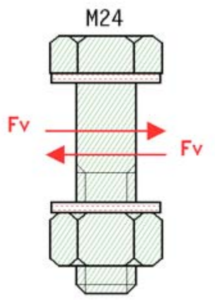
tension resistance of one bolt $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2}$ = 253.80 kN, k_2 = 0.90

punching shear load capicity of a bolt $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2}$ = 541.39 kN, t_p = 17.0 mm

tension-/punching shear load capicity for 2 bolts: $\Sigma F_{tp,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd})$ = 507.60 kN

5.2.3.8. bc 11: bolts in shear

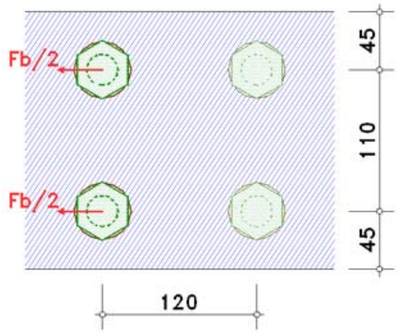
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



shear resistance $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2}$ = 217.15 kN, α_v = 0.60
shear resistance of 2 bolts (1-shear): $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd}$ = 434.29 kN

5.2.3.9. bc 12: plate with bearing resistance

Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

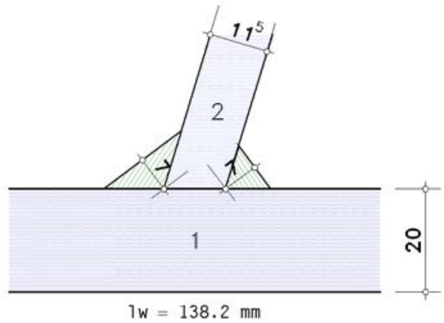


row 1
end-plate:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 361.85 kN, k_m = 1.00, α_b = 1.92
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 361.85 kN, k_m = 1.00, α_b = 1.92
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 723.69 kN
shear block of groups of bolts
shearversagen in Kombination in tensionversagen of plate
failure type 1:
tension-shear resistance $V_{eff,Rd} = (A_{nt} \cdot f_u + \min(A_{gv} \cdot f_y,\ A_{nv} \cdot f_u)/3^{1/2}) / \gamma_{M2}$ = 822.53 kN
failure type 2:
tension-shear resistance $V_{eff,Rd} = (A_{nt} \cdot f_u + \min(A_{gv} \cdot f_y,\ A_{nv} \cdot f_u)/3^{1/2}) / \gamma_{M2}$ = 665.73 kN
failure type 3:
tension-shear resistance $V_{eff,Rd} = (0.5 \cdot A_{nt} \cdot f_u + \min(A_{gv} \cdot f_y,\ A_{nv} \cdot f_u)/3^{1/2}) / \gamma_{M2}$ = 567.73 kN
failure type 4:
tension resistance (without shearanteil) $V_{eff,Rd} = (A_{nt} \cdot f_u) / \gamma_{M2}$ = 1160.32 kN
resistance due to shear block: min $V_{eff,Rd}$ = 567.7 kN
total
bearing resistance incl. shear block: $\min(\Sigma F_{b,Rd},\ \min V_{eff,Rd})$ = 567.7 kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 852.99 kN
row 2
end-plate:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 564.48 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 564.48 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 1003.52 kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 852.99 kN
row 3
end-plate:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 564.48 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 564.48 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 1003.52 kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 479.81 kN, k_m = 1.00, α_b = 3.00
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 852.99 kN

row 4
end-plate:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 564.48$ kN, $k_m = 1.00$, $\alpha_b = 3.00$
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 564.48$ kN, $k_m = 1.00$, $\alpha_b = 3.00$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 1003.52$ kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 479.81$ kN, $k_m = 1.00$, $\alpha_b = 3.00$
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 479.81$ kN, $k_m = 1.00$, $\alpha_b = 3.00$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 852.99$ kN

bearing resistance (4 rows)
$\Sigma F_{b,Rd,1} = 567.73$ kN
$\Sigma F_{b,Rd,2} = 852.99$ kN
$\Sigma F_{b,Rd,3} = 852.99$ kN
$\Sigma F_{b,Rd,4} = 852.99$ kN

5.2.3.10. bc 19: weld

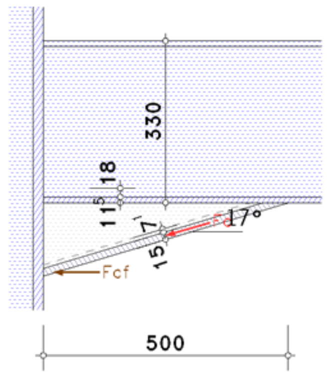


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

calculation with the simplified method
beam flange in compression
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 1760.28$ kN/m, $f_{vw,d} = 251.5$ N/mm², $a = 7.0$ mm
total $F_{w,Rd} = 486.7$ kN

5.2.3.11. bc 20: haunched beam in compression

flange below: section class 1
web: section class 1
total: section class 1
section class of the beam in connection plane: 1
taking into account the moment-shear force-interaction $V_{Ed} = 200.0$ kN



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

assumption: haunch flange no hazard of buckling: 1 < 3 ok
connection haunch-column: (basic component 7: beam flange and web in compression)
beam height incl. haunch $h = h_b + h_v = 480.0$ mm
resistance only flange (without web, plastic: section class 1)
haunch flange width $b_{f,v} = 160.0$ mm < max $b_{f,v} = 535.1$ mm ok
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 413.04$ kNm, $W_{pl} = 1163.51$ cm³
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 889.52$ kN
refering to haunch flange $F_{c,f,Rd} / \cos(\alpha_v) = 928.69$ kN

total loading capacity of a haunched beam in compression
$F_{c,v,Rd} = F_{c,f,Rd} = 928.69$ kN
resistance refering to die connection plane $F_{c,v,Rd} \cdot \cos(\alpha_v) = 889.52$ kN

resistance of the upper beam flange (bc 7):
stress due to bending with shear force: $V_{Ed} = 200.0$ kN ≤ 351.3 kN $= 0.5 \cdot V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 444.99$ kNm, $W_{pl} = 1253.49$ cm³
resistance of flange and web in compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 949.80$ kN

5.2.4. connection capacity

transformation parameter: $\beta_j = 0.58$

5.2.4.1. moment resistance

distance of tension-bolt-rows from centre of compression:
$h_1 = 522.2$ mm, $h_2 = 402.2$ mm, $h_3 = 222.2$ mm, $h_4 = 82.2$ mm

resistances acc. to EC 3-1-8, B.3.2.2(6) for bolt-rows considered individually
decisive basic components: 3, 4, 5, 8
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 429.0$ kN
row 4: $F_{tr,Rd} = 429.0$ kN

deductions acc. to EC 3-1-8, B.3.2.2(8) for bolt-rows as part of a group (column)
decisive basic components: 3, 4
group 1
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 405.6$ kN
group 2
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 405.6$ kN
row 4: $F_{tr,Rd} = 378.1$ kN

deductions acc. to EC 3-1-8, B.3.2.2(8) for bolt-rows as part of a group (end-plate)
decisive basic components: 5, 8
group 1
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 405.6$ kN

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 405.6$ kN
row 4: $F_{tr,Rd} = 378.1$ kN
$\Sigma F_{tr,Rd}^{*} = 1565.6$ kN

deductions acc. to EC 3-1-8, B.3.2.2(7)
decisive basic components: 1, 2, 20
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 107.7$ kN
row 4: $F_{tr,Rd} = 0.0$ kN

check acc. to EC 3-1-8, B.3.2.2(9)
decisive basic component: 10
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 107.7$ kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 352.7$ kN
row 2: $F_{tr,Rd} = 429.1$ kN
row 3: $F_{tr,Rd} = 107.7$ kN
row 4: $F_{tr,Rd} = 0.0$ kN
$\Sigma F_{tr,Rd} = 889.5$ kN
potential failure by basic component 4, 5, 20

resistance of flanges (compression)
$\Sigma F_{c,Rd}^{*} = 1839.3$ kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 380.7$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^{*} = 1565.6$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^{*} = 1839.3$ kN

5.2.4.2. shear/bearing resistance

resistance per bolt-row
decisive basic components: 11, 12
row 1: $F_{vr,Rd}$ = 434.3 kN
row 2: $F_{vr,Rd}$ = 434.3 kN
row 3: $F_{vr,Rd}$ = 434.3 kN
row 4: $F_{vr,Rd}$ = 434.3 kN

deductions depending on tension force (at full utilization of moment resistance)
decisive basic component: 10
row 1: $F_{vr,Rd}$ = 218.7 kN
row 2: $F_{vr,Rd}$ = 172.0 kN
row 3: $F_{vr,Rd}$ = 368.5 kN
row 4: $F_{vr,Rd}$ = 434.3 kN

resistance per bolt-row (shear/bearing resistance)
row 1: $F_{vr,Rd}$ = 218.7 kN
row 2: $F_{vr,Rd}$ = 172.0 kN
row 3: $F_{vr,Rd}$ = 368.5 kN
row 4: $F_{vr,Rd}$ = 434.3 kN
$\Sigma F_{vr,Rd}$ = 1193.6 kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd}$ = 1193.6 kN

5.2.4.3. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd}$ = 1708.73 kN
welds: $F_{w,Rd}$ = 1048.24 kN
shear resistance: $V_{ep,Rd} = F_{w,Rd}$ = 1048.24 kN

shear resistance of column web
decisive basic component: 1
$V_{wp,Rd}$ = 664.36 kN

5.2.4.4. total

$M_{j,Rd}$ = 380.7 kNm $N_{j,t,Rd}$ = 1565.6 kN $N_{j,c,Rd}$ = 1839.3 kN $V_{j,Rd}$ = 1193.6 kN $V_{wp,Rd}$ = 664.4 kN $V_{ep,Rd}$ = 1048.2 kN

5.2.5. verifications

5.2.5.1. verification of the connection capacity by means of the component method

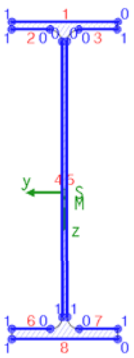
internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu}$ = 81.88 kNm, z_{bu} = 211.4 mm
shear force: $V_{Ed} = |V_d|$ = 197.88 kN
shear force: $V_{b,w,Ed}$ = 156.47 kN

$M_{Ed}/M_{j,Rd} = 0.215 < 1$ ok
shear/bearing resistance at 21.5% utilization of moment resistance $V_{j,Rd}$ = 1620.3 kN
$V_{Ed}/V_{j,Rd} = 0.122 < 1$ ok
$V_{b,w,Ed}/V_{ep,Rd} = 0.149 < 1$ ok

5.2.5.2. verification of welds at beam section

weld 1: beam flange in tension outer
welds 2,3: beam flange in tension inner
welds 4,5: beam web double-sided
weld 8: beam flange in compression outer
welds 6,7: beam flange in compression inner

calculation section:



weld 1: a_w = 7.0 mm l_w = 160.0 mm
weld 2: a_w = 7.0 mm l_w = 58.3 mm
weld 3: see weld 2
weld 4: a_w = 5.0 mm l_w = 416.8 mm
weld 5: see weld 4
weld 6: a_w = 7.0 mm l_w = 58.3 mm
weld 7: see weld 6
weld 8: a_w = 7.0 mm l_w = 160.0 mm

design values refering to centroid of the section:
N_{Ed} = 29.04 kN, $M_{y,Ed}$ = -75.74 kNm, $V_{z,Ed}$ = 197.88 kN

cross-sectional properties refering to centroid of the line cross-section:
ΣA_w = 80.39 cm², $A_{w,z}$ = 41.68 cm², Σl_w = 138.7 cm
$I_{w,y}$ = 27301.20 cm⁴, $I_{w,z}$ = 951.99 cm⁴, Δz_w = -19.3 mm
distribution of internal forces and moments:

weld 1:	N_w = 78.16 kN	
weld 2:	N_w = 27.15 kN	
weld 3:	see weld 2	
weld 4:	N_w = 7.86 kN	$M_{y,w}$ = -8.37 kNm
weld 5:	see weld 4	
weld 6:	N_w = -24.08 kN	
weld 7:	see weld 6	
weld 8:	N_w = -71.00 kN	

from conventionel distribution of shear force: $V_{z,w}$ = 197.88 kN
verifications in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x}$ = 69.78 N/mm²		$\Rightarrow U_w = 0.277 < 1$ ok
weld 2,	pt. 0:	$\sigma_{w,x}$ = 66.59 N/mm²		$\Rightarrow U_w = 0.265 < 1$ ok
weld 4,	pt. 0:	$\sigma_{w,x}$ = 61.60 N/mm²	$\tau_{w,z}$ = 47.47 N/mm²	$\Rightarrow U_w = 0.309 < 1$ ok
	pt. 1:	$\sigma_{w,x}$ = -54.05 N/mm²	$\tau_{w,z}$ = 47.47 N/mm²	$\Rightarrow U_w = 0.286 < 1$ ok
weld 6,	pt. 0:	$\sigma_{w,x}$ = -59.04 N/mm²		$\Rightarrow U_w = 0.235 < 1$ ok
weld 8,	pt. 0:	$\sigma_{w,x}$ = -63.39 N/mm²		$\Rightarrow U_w = 0.252 < 1$ ok

Result:
weld 4, pt. 0: $\sigma_{w,x}$ = 61.60 N/mm² $\tau_{w,z}$ = 47.47 N/mm²
Max: $F_{w,Ed}$ = 388.83 kN/m $<$ $F_{w,Rd}$ = 1257.34 kN/m $\Rightarrow U_w = 0.309 < 1$ ok

verification of deflection forces (bc 19, simplified method)
compression flange: $F_{Rd} = F_{w,Rd}$ = 486.7 kN, F_{Ed} = 144.10 kN
F_{Ed} = 144.1 kN $<$ F_{Rd} = 486.7 kN $\Rightarrow U_{w,f} = 0.296 < 1$ ok

5.2.5.3. verification result

maximum utilization: max U = 0.309 $<$ 1 ok

5.2.6. rotational stiffness

stiffness coefficients
equivalent stiffness coefficient for 4 tension-bolt-rows:
1: k_3 = 8.90 mm, k_4 = 25.21 mm, k_5 = 9.66 mm $\Rightarrow$ $k_{eff,1} = 1 / \Sigma(1/k_{i,1})$ = 3.914 mm
2: k_3 = 8.90 mm, k_4 = 25.21 mm, k_5 = 19.18 mm $\Rightarrow$ $k_{eff,2} = 1 / \Sigma(1/k_{i,2})$ = 4.899 mm
3: k_3 = 8.90 mm, k_4 = 25.21 mm, k_5 = 18.21 mm $\Rightarrow$ $k_{eff,3} = 1 / \Sigma(1/k_{i,3})$ = 4.833 mm
4: k_3 = 8.90 mm, k_4 = 25.21 mm, k_5 = 18.61 mm $\Rightarrow$ $k_{eff,4} = 1 / \Sigma(1/k_{i,4})$ = 4.861 mm
equivalent internal lever arm $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r)$ = 388.36 mm
äquivalenter stiffness coefficient $k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq}$ = 14.130 mm
$k_2 = \infty$ (stiffened)
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i)$ = 447529.3 kNm/rad, $z = z_{eq}$ = 388.4 mm, $\Sigma(1/k_i)$ = 0.071 mm⁻¹
$|M_{j,Ed}|$ = 81.88 kNm $\leq$ 2/3 $M_{j,Rd}$ = 253.79 kNm $\Rightarrow \mu = 1$
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu$ = 447529.3 kNm/rad
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd}$ = 0.010°