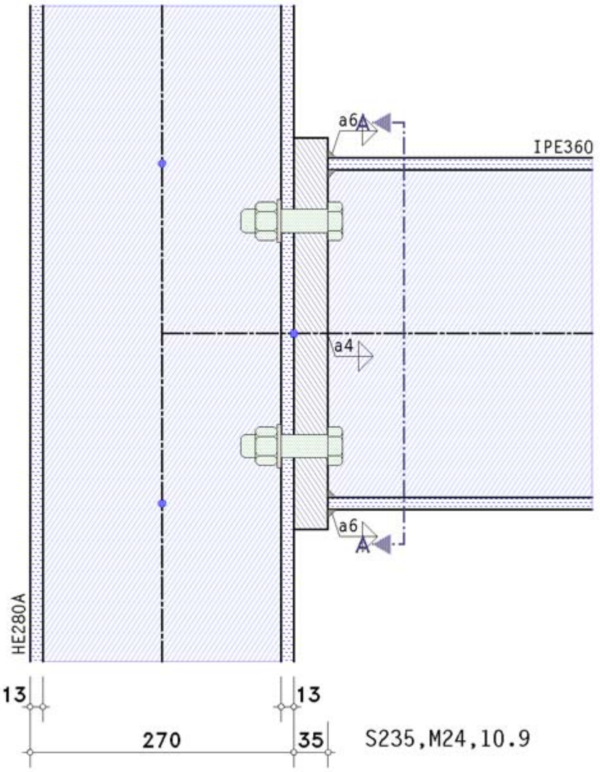
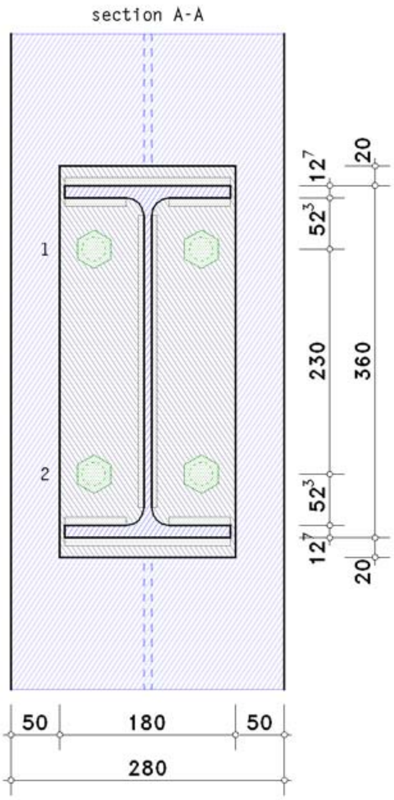
POS. 18: WAGENKNECHT BD.3 4.6.1

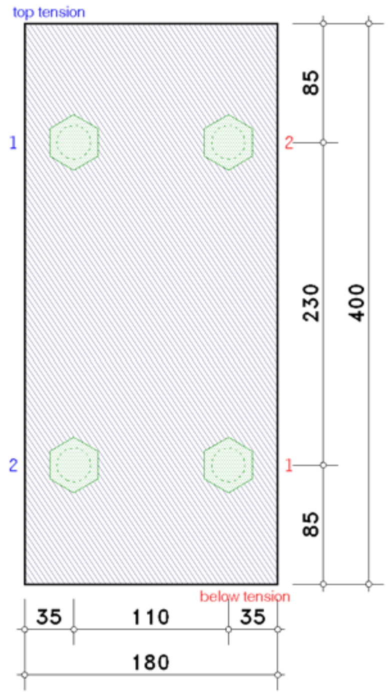
Rigid beam connection EC 3-1-8 (04.25), NA: Deutschland 4H-EC3BT version: 10/2019-2w

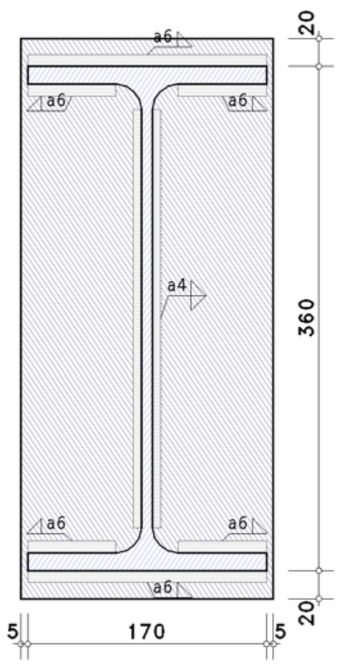
1. input report





details (section A - A)

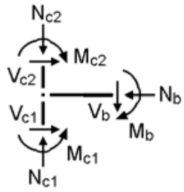




steel grade
steel grade S235
column parameters
section HE280A
bolts
bolt class 10.9, bolt size M24, normal wrench size
shear plane passes through the unthreaded portion of the bolt
beam parameters
section IPE360
verification parameters

bolted end-plate connection
thickness t_p = 35.0 mm, width b_p = 180.0 mm, length l_p = 400.0 mm
projections $h_{p,o}$ = 20.0 mm, $h_{p,u}$ = 20.0 mm
bolts in connection:
  2 bolt-rows with 2 bolts
  of these 1 bolt-row top in tension (row 1)
    and 1 bolt-row for shear transfer top (row 2)
  of these 1 bolt-row below in tension (row 2)
    and 1 bolt-row for shear transfer below (row 2)
  centre distance of the bolts to the lateral edge of the end-plate e_2 = 35.0 mm
  centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 85.0 mm
  centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 85.0 mm
  centre distance of the bolt-rows from each other $p_{1\text{-}2}$ = 230.0 mm
welds at the connection point:
  beam flange top: fillet weld, weld thickness a = 6.0 mm
  beam web: fillet weld, weld thickness a = 4.0 mm
  beam flange below: fillet weld, weld thickness a = 6.0 mm

internal forces and moments at the joint periphery refering to the system axes



Lc 1: $M_{b,Ed}$ = 1.00 kNm

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
no verification for cross-sections.
the welds are not regarded by calculation the T-stub resistance.
welds are not checked.
calculation of T-stub-resistance with the standard method.
Die shear resistance of end plate is not respected.

check of data
ok

distances between bolts at end-plate

horizontal:	e_2 = 35.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_2 = 35.0 mm < $4 \cdot t$ + 40 mm = 92.0 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) = 182.0 mm
top-below:	e_1 = 85.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 85.0 mm < $4 \cdot t$ + 40 mm = 92.0 mm
top-below:	p_1 = 230.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 230.0 mm > min($14 \cdot t$, 200 mm) = 182.0 mm !!
top-below:	e_1 = 85.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 85.0 mm < $4 \cdot t$ + 40 mm = 92.0 mm

bolt distance from column edge

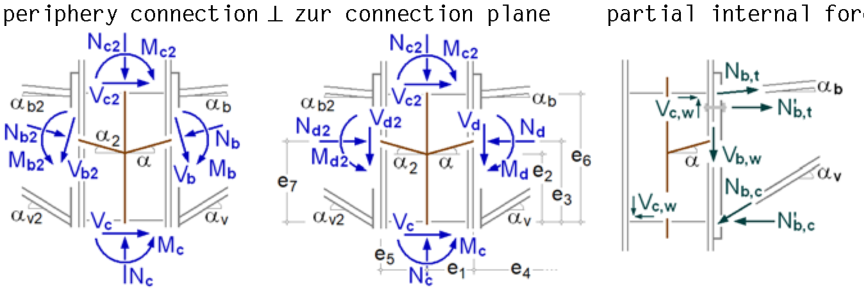
horizontal:	e_2 = 85.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_2 = 85.0 mm < $4 \cdot t$ + 40 mm = 92.0 mm

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

2. Lc 1

2.1. design values

periphery connection ⊥ zur connection plane   partial internal forces and moments



slope angle: $\alpha_b = \alpha = \alpha_v$ = 0°
distance: e_1 = 135.0 mm, e_3 = 173.6 mm, e_2 = 173.6 mm, e_6 = 347.3 mm

internal forces and moments perpendicular to the connection planes
periphery beam
M_d = 1.00 kNm

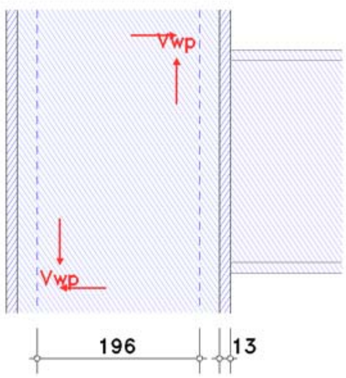
partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_p$ = 1.00 kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 2.88 kN, z_b = 347.3 mm, z_{bu} = 173.7 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b$ = 2.88 kN, z_b = 347.3 mm, z_{bo} = 173.7 mm

2.2. basic components

2.2.1. bc 1: Column web panel in shear

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 1.00$ kNm ($M_{j2} = 0$)

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

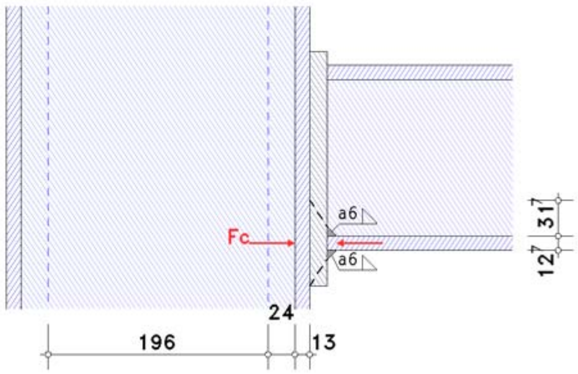


slenderness of column web $h_{wc}/t_{wc} = 30.50 < 72 \cdot \varepsilon/\eta = 60.00 \Rightarrow$ method applicable
plastic shear resistance $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_{wp}) / (3^{1/2} \cdot \gamma_{M0}) = 263.76$ kN
Beitrag of column flange:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/z_{wp} = 38.5$ kN, $z_{wp} = h_r = 288.6$ mm
plastic shear resistance plus Beitrag of column flange $V_{wp,Rd} = 302.3$ kN

2.2.2. bc 2: column web in transverse compression

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 1.00$ kNm ($M_{j2} = 0$)

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

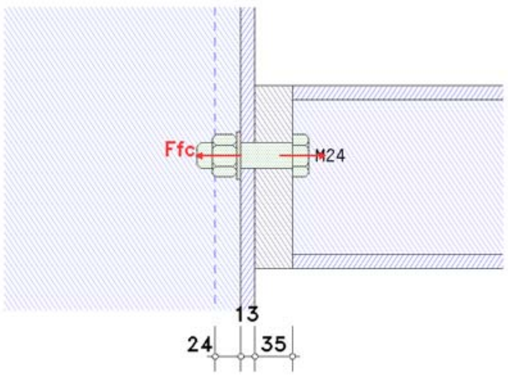


effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p = 261.2$ mm
reduction factor $k_w = 1.0$ ($\sigma_{com,Ed} = 0$)
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.882$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.851$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.672$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 329.82$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 255.20$ kN (decisive)

resistance of the upper beam flange:
effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p = 261.2$ mm
reduction factor $k_w = 1.0$ ($\sigma_{com,Ed} = 0$)
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.882$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.851$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.672$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 329.82$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 255.20$ kN (decisive)

2.2.3. 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 $n_b = 1$

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 199.8$ mm, $l_{eff,cp} = 199.8$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 233.4$ mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 1.98$ kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 2.32$ 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

$L_b = 70.7$ mm ≤ 227.2 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 249.54$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 335.36$ kN

mode 3: bolt failure

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

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 249.54$ kN

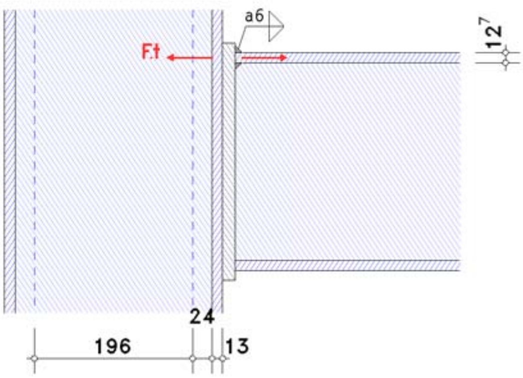
resistances and effective lengths of column flange in bending (per bolt-row)

$F_{t,fc,Rd,1} = 249.54$ kN, $l_{eff,1} = 199.8$ mm

2.2.4. bc 3: column web in transverse tension

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 1.00$ kNm ($M_{j2} = 0$)

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



each individual bolt-row:

reduction factor for interaction with shear stress $\beta = 1$ $\Rightarrow$ $\omega = 0.764$

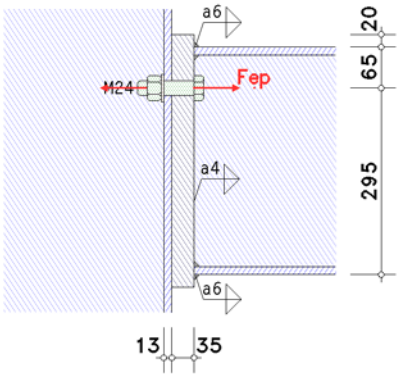
resistance eines unstiffeneden column webs with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 287.09$ kN, $b_{eff,t,wc} = 199.8$ mm

resistance of a column web with transverse tension (per bolt-row)

$F_{t,wc,Rd,1} = 287.09$ kN, $b_{eff,t,wc} = 199.8$ mm (s. bc 4)

2.2.5. bc 5: end-plate in bending

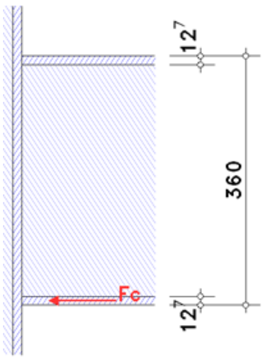


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

part of end-plate between beam flanges
equivalent T-stub flange (each individual bolt-row):
here: number of bolt-rows $n_b = 1$
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 245.2$ mm, $l_{eff,cp} = 292.0$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 245.2$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 17.64$ 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
$L_b = 70.7$ mm > 29.6 mm = L_b^* $\Rightarrow$ no prying forces !
mode 1 and 2: complete yielding of the T-stub flange and possibly coincident bolt failure
$F_{T,1\text{-}2,Rd} = (2 \cdot M_{pl,1,Rd}) / m = 759.32$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1\text{-}2,Rd}, F_{T,3,Rd}) = 507.60$ kN
resistances and effective lengths of end-plate in bending (per bolt-row):
$F_{ep,Rd,1} = 507.60$ kN, $l_{eff,1} = 245.2$ mm

2.2.6. bc 7: beam flange and web in compression

flange below: section class 1
web: section class 1
total: section class 1
section class of the beam: 1

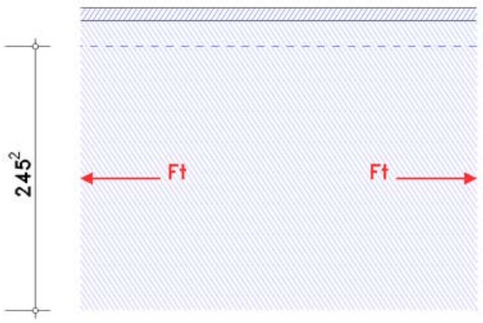


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 239.47$ kNm, $W_{pl} = 1019.04$ cm^3
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 689.53$ kN

resistance of the upper beam flange:
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 239.47$ kNm, $W_{pl} = 1019.04$ cm^3
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 689.53$ kN

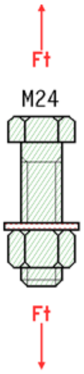
2.2.7. bc 8: beam web in tension



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

each individual bolt-row:
effective width $b_{eff,t,wb}$ = 245.2 mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 460.92 kN
resistance of a beam web in tension (per bolt-row)
$F_{t,wb,Rd,1}$ = 460.92 kN, $b_{eff,t,wb}$ = 245.2 mm (s. bc 5)

2.2.8. 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 $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2}$ = 266.59 kN, t_p = 13.0 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

2.3. connection capacity

transformation parameter: β_j = 1.00

2.3.1. moment resistance

distance of tension-bolt-row from centre of compression: h_1 = 288.6 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: $F_{tr,Rd}$ = 249.5 kN

resistance per bolt-row (tension)
row 1: $F_{tr,Rd}$ = 249.5 kN
$\Sigma F_{tr,Rd}^*$ = 249.5 kN

deductions acc. to EC 3-1-8, B.3.2.2(7)
decisive basic components: 1, 2, 7
row 1: $F_{tr,Rd}$ = 249.5 kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd}$ = 249.5 kN
$\Sigma F_{tr,Rd}$ = 249.5 kN
potential failure by basic component 4

resistance of flanges (compression)
$\Sigma F_{c,Rd}^*$ = 510.4 kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r)$ = 72.0 kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^*$ = 249.5 kN
compression resistance

$N_{j,c,Rd} = \Sigma F_{c,Rd}^{*} = 510.4$ kN

2.3.2. shear resistance

shear resistance of column web
decisive basic component: 1
$V_{wp,Rd} = 302.28$ kN

2.3.3. total

$M_{j,Rd} = 72.0$ kNm $N_{j,t,Rd} = 249.5$ kN $N_{j,c,Rd} = 510.4$ kN $V_{wp,Rd} = 302.3$ kN

2.4. rotational stiffness

stiffness coefficients
$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z) = 4.18$ mm
$k_2 = 0.7 \cdot b_{eff,c,wc} \cdot t_{wc} / d_c = 7.46$ mm
$k_3 = 0.7 \cdot b_{eff,t,wc} \cdot t_{wc} / d_c = 5.71$ mm, $b_{eff,t,wc} = 199.8$ mm
$k_4 = 0.9 \cdot l_{eff} \cdot t_{tc}^3 / m^3 = 12.29$ mm, $l_{eff} = 199.8$ mm, $m = 31.8$ mm
$k_5 = 0.9 \cdot l_{eff} \cdot t_p^3 / m^3 = 94.25$ mm, $l_{eff} = 245.2$ mm, $m = 46.5$ mm
$k_{10} = 1.6 \cdot A_s / L_b = 7.98$ mm, $L_b = t_{ges} + t_p + (t_k + t_m)/2 = 70.7$ mm, $t_{ges} = 48.0$ mm
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i) = 22849.9$ kNm/rad, $z = 288.6$ mm, $\Sigma(1/k_i) = 0.766$ mm^{-1}
$|M_{j,Ed}| = 1.00$ kNm $\leq$ 2/3 $M_{j,Rd} = 48.02$ kNm $\Rightarrow$ $\mu = 1$
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu = 22849.9$ kNm/rad
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd} = 0.003°$

3. 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