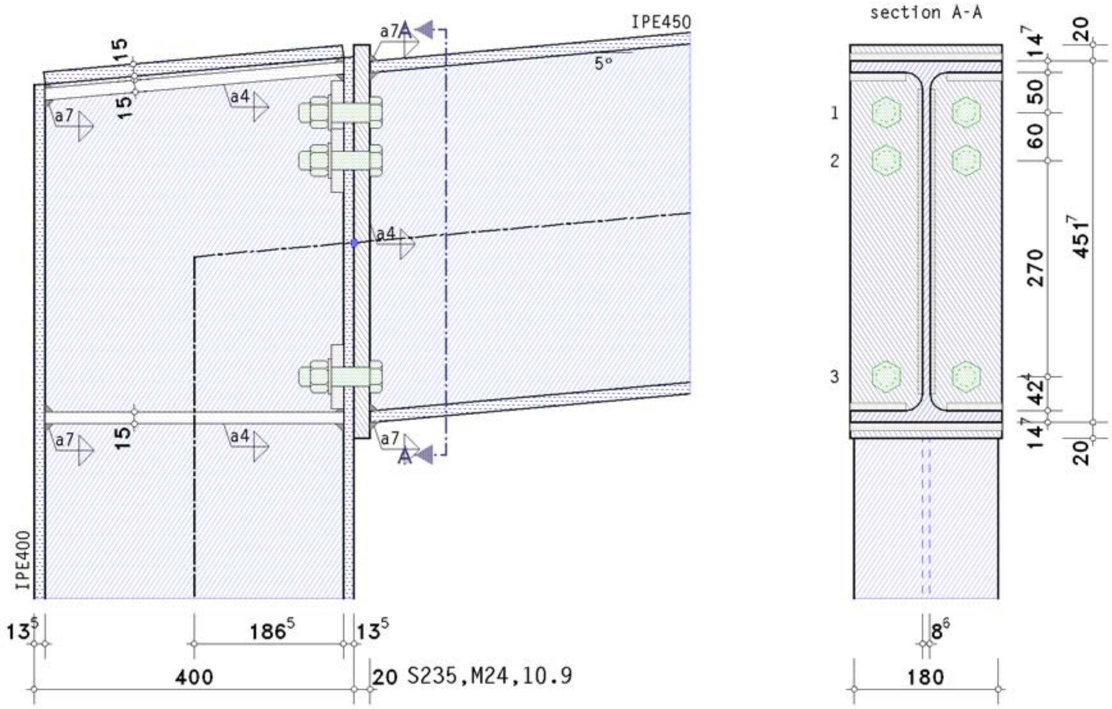
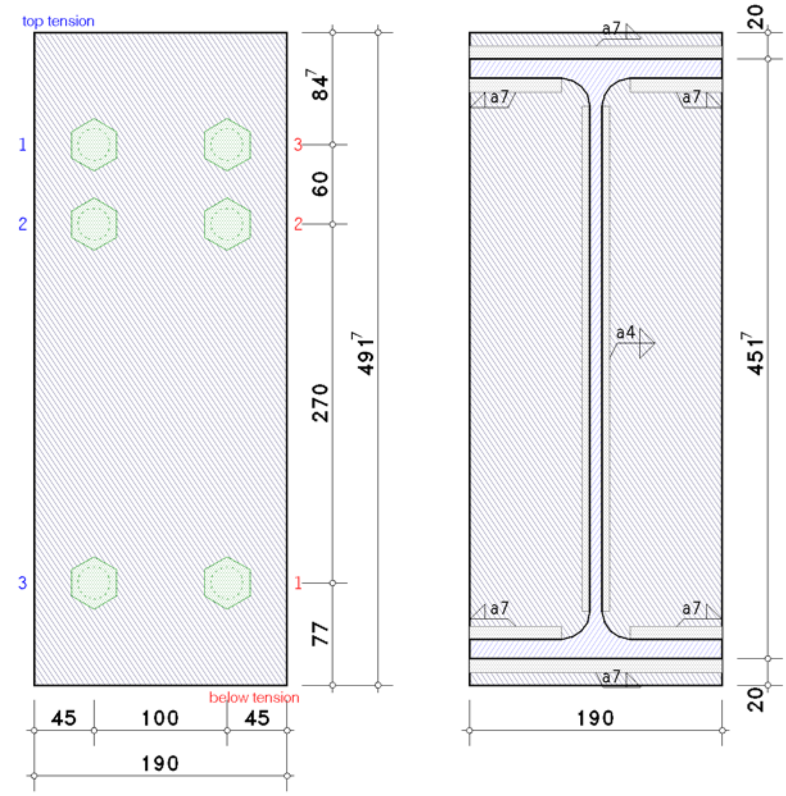
POS. 2: WAGENKNECHT BD.3 4.6.5

frame corner EC 3-1-8 (04.25), NA: Deutschland 4H-EC3RE version: 2/2020-2q

1. input report



details (section A - A)



steel grade

steel grade S235

column parameters

section IPE400

reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 435.7 mm):

thickness t_{st} = 15.0 mm, width b_{st} = 85.7 mm, length l_{st} = 373.0 mm

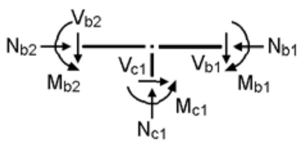
recess at stiffeners c_{st} = 31.5 mm

welds $a_{st,f}$ = 7.0 mm, $a_{st,w}$ = 4.0 mm

cap plate t_k = 15.0 mm (has no effect)

bolts

bolt class 10.9, bolt size M24, normal wrench size
shear plane passes through the unthreaded portion of the bolt
packing (flange reinforcement): thickness t_{bp} = 15.0 mm, width b_{bp} = 80.0 mm, length l_{bp} = 80.0 mm
beam parameters
section IPE450
slope angle of section about the horizontal axis α_b = 5.00° ⇒ section depth at the joint loc. $h_b = h/\cos(\alpha_b)$ = 451.7 mm
verification parameters
bolted end-plate connection
thickness t_p = 20.0 mm, width b_p = 190.0 mm, length l_p = 491.7 mm
projections $h_{p,o}$ = 20.0 mm, $h_{p,u}$ = 20.0 mm
bolts in connection:
3 bolt-rows with 2 bolts
of these 2 bolt-rows top in tension (rows 1-2)
and 1 bolt-row for shear transfer top (row 3)
of these 1 bolt-row below in tension (row 3)
and 2 bolt-rows for shear transfer below (rows 2-3)
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 = 84.7 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 77.0 mm
centre distance of the first bolt-row to the free edge of the column (end row) e_1' = 84.7 mm
centre distance of the bolt-rows from each other p_{1-2} = 60.0 mm, p_{2-3} = 270.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 7.0 mm, angle φ = 85°
beam web: fillet weld, weld thickness a = 4.0 mm
beam flange below: fillet weld, weld thickness a = 7.0 mm, angle φ = 95°
internal forces and moments at the joint periphery refering to the system axes
Lc 1: $N_{b,Ed}$ = 57.30 kN $M_{b,Ed}$ = 250.00 kNm $V_{b,Ed}$ = 77.00 kN
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.
local resistance of weld is not respected.
no verification of web stiffeners.
plated structures- and shear buckling is not investigated.
the cap plate has no effect.
local shear resistance of end plate is not respected.

check of data
ok
distances between bolts at end-plate

horizontal:	e_2 = 45.0 mm > 1.2·d_0 = 31.2 mm,	e_2 = 45.0 mm < 4·t + 40 mm = 94.0 mm
horizontal:	p_2 = 100.0 mm > 2.4·d_0 = 62.4 mm,	p_2 = 100.0 mm < min(14·t, 200 mm) = 189.0 mm
top-below:	e_1 = 84.7 mm > 1.2·d_0 = 31.2 mm,	e_1 = 84.7 mm < 4·t + 40 mm = 94.0 mm
top-below:	e_1 = 84.7 mm > 1.2·d_0 = 31.2 mm,	e_1 = 84.7 mm < 4·t + 40 mm = 94.0 mm
top-below:	p_1 = 60.0 mm > 2.2·d_0 = 57.2 mm,	p_1 = 60.0 mm < min(14·t, 200 mm) = 189.0 mm
top-below:	p_1 = 270.0 mm > 2.2·d_0 = 57.2 mm,	p_1 = 270.0 mm > min(14·t, 200 mm) = 189.0 mm !!
top-below:	e_1 = 77.0 mm > 1.2·d_0 = 31.2 mm,	e_1 = 77.0 mm < 4·t + 40 mm = 94.0 mm

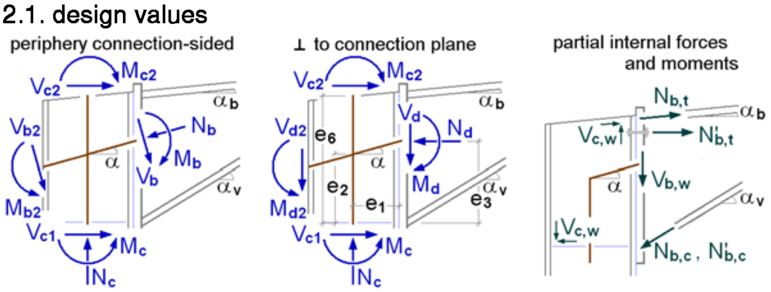
bolt distance from column edge

horizontal:	e_2 = 40.0 mm > 1.2·d_0 = 31.2 mm,	e_2 = 40.0 mm < 4·t + 40 mm = 94.0 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



slope angle: $\alpha_b = \alpha_v = \alpha$ = 5.00°
distances: e_1 = 200.0 mm, e_3 = 218.5 mm, e_2 = 201.0 mm, e_6 = 435.7 mm

internal forces and moments perpendicular to the connection planes
periphery beam

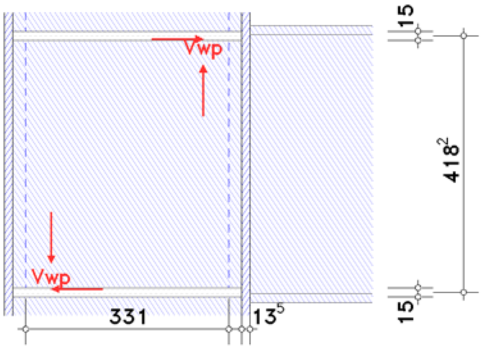
N_d = 50.37 kN, M_d = 250.00 kNm, V_d = 81.70 kN
periphery column (below, calculated)
N_c = 81.70 kN, M_c = 255.33 kNm, V_c = 50.37 kN

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d \cdot t_p \cdot \tan(\alpha) - V_d \cdot t_p$ = 248.45 kNm
$N_{b,t} = (-N_d \cdot z_{bu}/z_b + M'_d/z_b) / \cos(\alpha_b)$ = 545.35 kN, z_b = 437.1 mm, z_{bu} = 218.5 mm
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_b)$ = 595.92 kN, z_b = 437.1 mm, z_{bo} = 218.5 mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b)$ = -47.53 kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_b)$ = 51.94 kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c}$ = 77.29 kN

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} = 250.00 kNm (M_{j2} = 0)

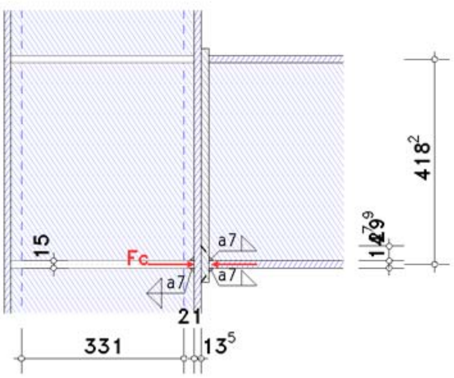


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

slenderness of column web h_{wc}/t_{wc} = 43.37 < $72 \cdot \varepsilon/\eta$ = 60.00 ⇒ method applicable
plastic shear resistance without web stiffeners $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_{wp}) / (3^{1/2} \cdot \gamma_{M0})$ = 420.06 kN
proportion of column flange:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/z_{wp}$ = 20.3 kN, $z_{wp} = h_r$ = 379.7 mm
plastic shear resistance plus proportion of column flange $V_{wp,Rd}$ = 440.4 kN
placing of intermediate web stiffeners:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/h_r + 2 \cdot M_{pl,st,Rd})/h_r$ = 20.30 kN
plastic shear resistance with transverse stiffeners $V_{wp,Rd}$ = 460.66 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} = 250.00 kNm (M_{j2} = 0)
longitudinal compressive stress in column web $\sigma_{com,Ed}$ = 192.39 N/mm²



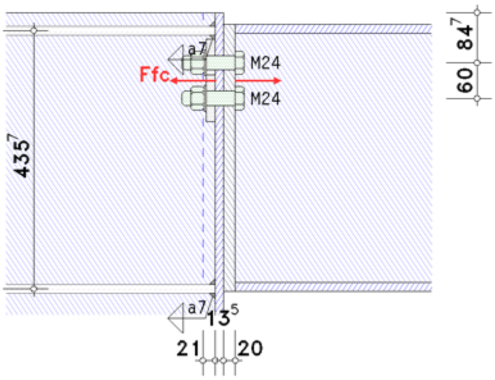
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p$ = 237.1 mm
reduction factor $k_w = 1.7 - \sigma_{com,Ed}/f_{y,w}$ = 0.881
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2}$ = 1.016
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2$ = 0.771 for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $\beta = 1$ ⇒ ω = 0.829
resistance of an unstiffened web in transverse compression:
  $F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0}$ = 349.88 kN
  $F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1}$ = 245.35 kN (decisive)
reinforcement of web with intermediate transverse stiffeners:
assumption: stiffeners do not buckle: section class 1, section class 1 ≤ 3 ok
minimum demands of the moment of inertia of stiffeners:
  length of buckling field (distance of stiffeners) a = 418.2 mm
  web height between the flanges h_{wc} = 373.0 mm
  moment of inertia of stiffeners I_{st} = 729.00 cm⁴
  minimum moment of inertia for $a/h_{wc} = 1.12 < 2^{1/2}$: $I_{st,min}$ = 28.31 cm⁴ < I_{st} ok
requirement concerning stiffeners to aviod lateral torsional buckling:
  torsional moment of inertia of stiffeners I_T = 9.64 cm⁴
  polar moment of inertia of stiffeners I_p = 81.09 cm⁴

I_T / $I_p \approx 0.119 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web A_{st} = 27.00 cm²
slenderness $\lambda = 0.076$
$\lambda \leq 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd}$ = 576.8 kN
maximum resistance:
$F_{c,w,Rd}$ = 576.8 kN (resistance with transverse stiffeners)

resistance of the upper beam flange:
effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p$ = 237.1 mm
reduction factor $k_w = 1.7 - \sigma_{com,Ed}/f_{y,w} = 0.881$
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 1.016$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.771$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.829$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0}$ = 349.88 kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1}$ = 245.35 kN (decisive)
reinforcement of web with intermediate transverse stiffeners:
assumption: stiffeners do not buckle: section class 1, section class 1 ≤ 3 ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 418.2 mm
web height between the flanges h_{wc} = 373.0 mm
moment of inertia of stiffeners I_{st} = 729.00 cm⁴
minimum moment of inertia for $a/h_{wc} = 1.12 < 2^{1/2}$: $I_{st,min}$ = 28.31 cm⁴ < I_{st} ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners I_T = 9.64 cm⁴
polar moment of inertia of stiffeners I_p = 81.09 cm⁴
I_T / $I_p \approx 0.119 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web A_{st} = 27.00 cm²
slenderness $\lambda = 0.076$
$\lambda \leq 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd}$ = 576.8 kN
maximum resistance:
$F_{c,w,Rd}$ = 576.8 kN (resistance with transverse stiffeners)

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
row 1
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 170.3 mm, $l_{eff,cp}$ = 181.6 mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 170.3 mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 1.82 kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_y) / \gamma_{M0}$ = 2.25 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 = 71.2 mm ≤ 178.7 mm = L_b^* $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,2,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd} + 4 \cdot n \cdot M_{bp,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n))$ = 591.56 kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 338.08 kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 507.60 kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 338.08 kN
row 2
effective length of the T-stub flange (column flange):

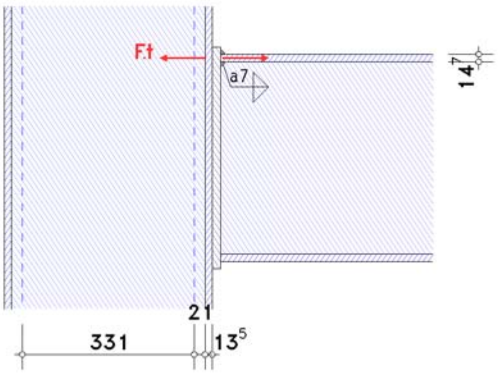
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 165.6$ mm, $l_{eff,cp} = 181.6$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 165.6$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 1.77$ kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_y) / \gamma_{M0} = 2.19$ 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
$L_b = 71.2$ mm ≤ 183.8 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,2,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd} + 4 \cdot n \cdot M_{bp,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 575.29$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 336.54$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 336.54$ kN

resistances and effective lengths of column flange in bending (per bolt-row)
$F_{t,fc,Rd,1} = 338.08$ kN, $l_{eff,1} = 170.3$ mm
$F_{t,fc,Rd,2} = 336.54$ kN, $l_{eff,2} = 165.6$ mm

equivalent T-stub flange (group of bolt-rows):
here: number of bolt-rows $n_b = 2$ (between stiffeners)
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 230.3$ mm, $\Sigma l_{eff,cp} = 301.6$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 230.3$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 2.47$ kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_y) / \gamma_{M0} = 3.04$ 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
$L_b = 71.2$ mm ≤ 264.3 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,2,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd} + 4 \cdot n \cdot M_{bp,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 800.00$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 639.84$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1015.20$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 639.84$ kN
effective length: $\Sigma l_{eff} = 230.3$ mm, 2 rows

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 $M_{j1} = 250.00$ kNm ($M_{j2} = 0$)



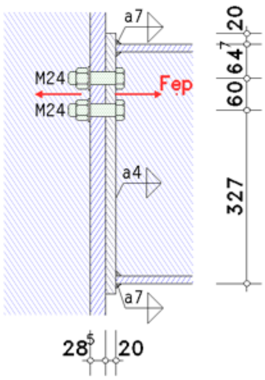
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

each individual bolt-row:
row 1
reduction factor for interaction with shear stress $\beta = 1$ $\Rightarrow$ $\omega = 0.900$
resistance of an unstiffened column web in transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 309.60$ kN, $b_{eff,t,wc} = 170.3$ mm
row 2
reduction factor for interaction with shear stress $\beta = 1$ $\Rightarrow$ $\omega = 0.904$
resistance of an unstiffened column web in transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 302.65$ kN, $b_{eff,t,wc} = 165.6$ mm

resistance of a column web with transverse tension (per bolt-row)
$F_{t,wc,Rd,1} = 309.60$ kN, $b_{eff,t,wc} = 170.3$ mm (s. bc 4)
$F_{t,wc,Rd,2} = 302.65$ kN, $b_{eff,t,wc} = 165.6$ mm (s. bc 4)

each group of bolt-rows:
reduction factor for interaction with shear stress $\beta = 1$ $\Rightarrow$ $\omega = 0.836$
resistance of an unstiffened column web in transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 389.07$ kN, $b_{eff,t,wc} = 230.3$ mm

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 $n_b = 1$

row 1

effective length of the T-stub flange (end-plate):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 236.6$ mm, $l_{eff,cp} = 256.2$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 236.6$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.56$ 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

$L_b = 71.2$ mm ≤ 111.1 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 698.20$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 389.06$ kN

mode 3: bolt failure

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

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 389.06$ kN

resistance of a fillet weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm²

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 481.90$ kN (≥ 389.06 kN, not decisive)

row 2

effective length of the T-stub flange (end-plate):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 219.3$ mm, $l_{eff,cp} = 256.2$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 219.3$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.15$ 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

$L_b = 71.2$ mm ≤ 119.8 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 647.20$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 379.00$ kN

mode 3: bolt failure

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

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 379.00$ kN

resistance of a fillet weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm²

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 446.70$ kN (≥ 379.00 kN, not decisive)

resistances and effective lengths of end-plate in bending (per bolt-row):

$F_{ep,Rd,1} = 389.06$ kN, $l_{eff,1} = 236.6$ mm

$F_{ep,Rd,2} = 379.00$ kN, $l_{eff,2} = 219.3$ mm

equivalent T-stub flange (group of bolt-rows):

here: number of bolt-rows $n_b = 2$ (R1+R2)

effective length of the T-stub flange (end-plate):

in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 217.0$ mm, $\Sigma l_{eff,cp} = 308.1$ mm

in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 217.0$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.10$ 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

$L_b = 71.2$ mm ≤ 242.3 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 640.15$ kN

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

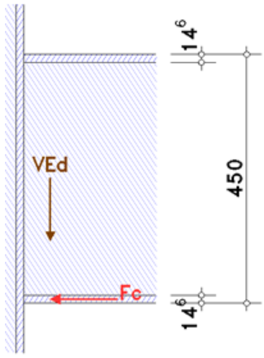
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 628.97$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1015.20$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 628.97$ kN
resistance of a fillet weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm²
tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 441.83$ kN
total loading capacity of the T-stub flange: $F_{T,Rd} = F_{T,w,Rd} = 441.83$ kN !!
effective length: $\Sigma l_{eff} = 217.0$ mm, 2 rows

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
taking into account the moment-shear force-interaction $V_{Ed} = 77.0$ kN

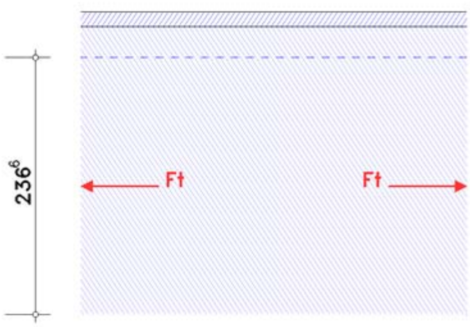


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

stress due to bending with shear force: $V_{Ed} = 77.0 \text{ kN} \leq 344.9 \text{ kN} = 0.5 \cdot V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 399.88$ kNm, $W_{pl} = 1701.60$ cm³
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 918.41$ kN
refering to aspect plane $F_{c,f,Rd} \cdot \cos(\alpha_b) = 914.92$ kN

resistance of the upper beam flange:
stress due to bending with shear force: $V_{Ed} = 77.0 \text{ kN} \leq 344.9 \text{ kN} = 0.5 \cdot V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 399.88$ kNm, $W_{pl} = 1701.60$ cm³
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 918.41$ kN
refering to aspect plane $F_{c,f,Rd} \cdot \cos(\alpha_b) = 914.92$ kN

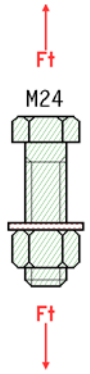
2.2.7. bc 8: beam web in tension



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

each individual bolt-row:
row 1
effective width $b_{eff,t,wb} = 236.6$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 522.72$ kN
row 2
effective width $b_{eff,t,wb} = 219.3$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 484.54$ kN
resistance of a beam web in tension (per bolt-row)
$F_{t,wb,Rd,1} = 522.72$ kN, $b_{eff,t,wb} = 236.6$ mm (s. bc 5)
$F_{t,wb,Rd,2} = 484.54$ kN, $b_{eff,t,wb} = 219.3$ mm (s. bc 5)
eine group of bolt-rows decisive:
effective width $b_{eff,t,wb} = 217.0$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 479.26$ kN

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 $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 276.84$ kN, $t_p = 13.5$ 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: $\beta_j = 1.00$

2.3.1. moment resistance

distance of tension-bolt-rows from centre of compression: $h_1 = 379.7$ mm, $h_2 = 319.7$ 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: $F_{tr,Rd} = 309.6$ kN
row 2: $F_{tr,Rd} = 302.7$ 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
row 1: $F_{tr,Rd} = 309.6$ kN
row 2: $F_{tr,Rd} = 79.5$ 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
row 1: $F_{tr,Rd} = 309.6$ kN
row 2: $F_{tr,Rd} = 79.5$ kN

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 309.6$ kN
row 2: $F_{tr,Rd} = 79.5$ kN
$\Sigma F_{tr,Rd}^* = 389.1$ kN

deductions acc. to EC 3-1-8, B.3.2.2(7)
decisive basic components: 1, 2, 7
row 1: $F_{tr,Rd} = 309.6$ kN
row 2: $F_{tr,Rd} = 79.5$ kN

check acc. to EC 3-1-8, B.3.2.2(9)
decisive basic component: 10
row 1: $F_{tr,Rd} = 309.6$ kN
row 2: $F_{tr,Rd} = 79.5$ kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 309.6$ kN
row 2: $F_{tr,Rd} = 79.5$ kN
$\Sigma F_{tr,Rd} = 389.1$ kN
potential failure by basic component 3

resistance of flanges (compression)
$\Sigma F_{c,Rd}^* = 921.3$ kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 143.0$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 389.1$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 921.3$ kN

2.3.2. shear resistance

shear resistance of column web
decisive basic component: 1
$V_{wp,Rd}$ = 460.66 kN

2.3.3. total

$M_{j,Rd}$ = 143.0 kNm $N_{j,t,Rd}$ = 389.1 kN $N_{j,c,Rd}$ = 921.3 kN $V_{wp,Rd}$ = 460.7 kN

2.4. rotational stiffness

stiffness coefficients
equivalent stiffness coefficient for 2 tension-bolt-rows:
1: k_3 = 3.10 mm, k_4 = 15.62 mm, k_5 = 25.13 mm, k_{10} = 7.93 mm $\Rightarrow$ $k_{eff,1} = 1 / \Sigma(1/k_{i,1})$ = 1.809 mm
2: k_3 = 3.01 mm, k_4 = 15.19 mm, k_5 = 23.30 mm, k_{10} = 7.93 mm $\Rightarrow$ $k_{eff,2} = 1 / \Sigma(1/k_{i,2})$ = 1.764 mm
equivalent internal lever arm $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r)$ = 352.64 mm
equivalent stiffness coefficient $k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq}$ = 3.547 mm
$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z)$ = 4.60 mm
$k_2 = \infty$ (stiffened)
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i)$ = 52300.5 kNm/rad, $z = z_{eq}$ = 352.6 mm, $\Sigma(1/k_i)$ = 0.499 mm^{-1}
$|M_{j,Ed}|$ = 239.08 kNm > 2/3 $M_{j,Rd}$ = 95.30 kNm $\Rightarrow$ $\mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi}$ = 11.980, Ψ = 2.7
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu$ = 4365.6 kNm/rad
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd}$ = 3.138°

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 Anhang zur EN 1993-1-1, Ausgabe April 2026

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 Anhang zur EN 1993-1-8, Ausgabe April 2026

EN 1993-1-5, Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-5: Plattenförmige Bauteile;
Deutsche Fassung EN 1993-1-5:2006 + AC:2009 + A1:2017 + A2:2019, Ausgabe Oktober 2019
EN 1993-1-5/NA, Nationaler Anhang zur EN 1993-1-5, Ausgabe Dezember 2010