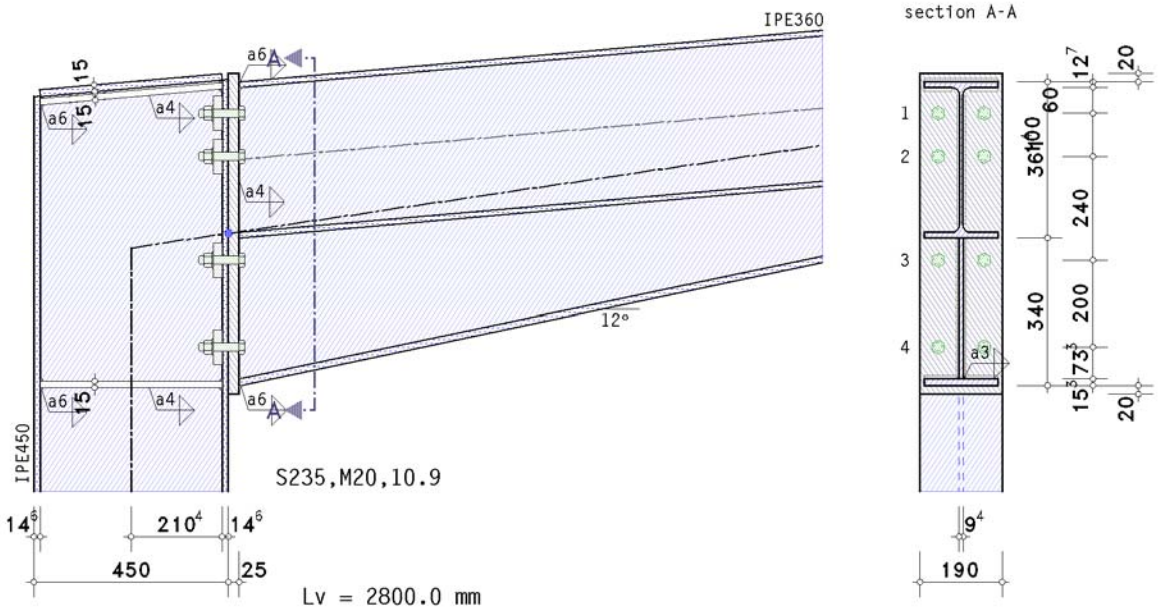
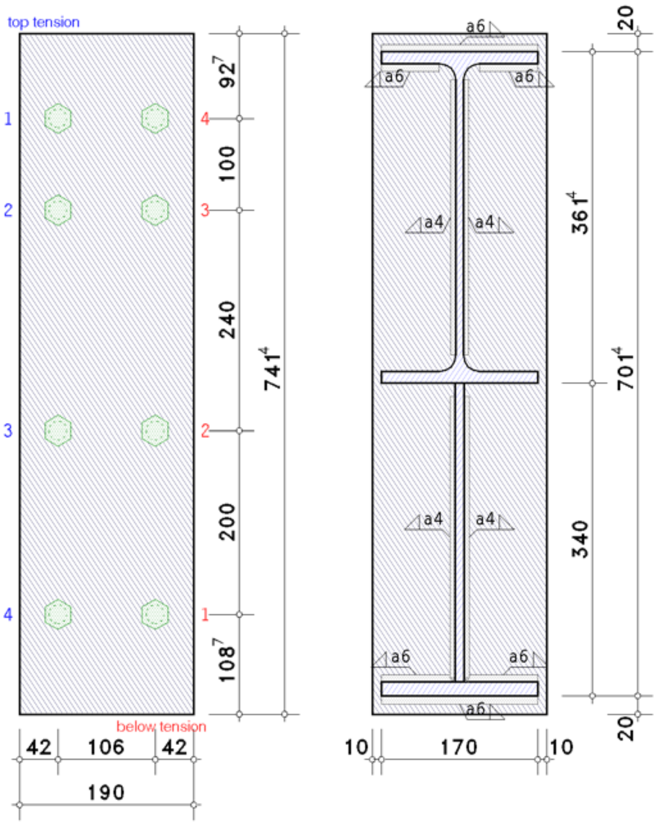
POS. 1: WAGENKNECHT BD.3 4.6.4

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 IPE450

reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 687.9 mm):

thickness t_{st} = 15.0 mm, width b_{st} = 90.3 mm, length l_{st} = 420.8 mm

recess at stiffeners c_{st} = 31.5 mm

welds $a_{st,f}$ = 6.0 mm, $a_{st,w}$ = 4.0 mm

cap plate t_k = 15.0 mm (has no effect)

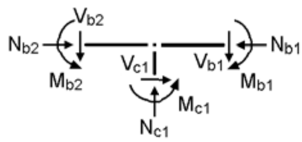
bolts

bolt class 10.9, bolt size M20, normal wrench size

shear plane passes through the unthreaded portion of the bolt

packing (flange reinforcement): thickness t_{bp} = 20.0 mm, width b_{bp} = 80.0 mm, length l_{bp} = 80.0 mm

beam parameters
section IPE360
slope angle of section about the horizontal axis α_b = 5.00° $\Rightarrow$ section depth at the joint loc. $h_b = h/\cos(\alpha_b)$ = 361.4 mm
slope angle of haunch about the horizontal axis α_v = 11.80° $\Rightarrow$ haunch angle about the beam axis $\Delta\alpha_v$ = 6.80°
haunch length L_v = 2800.0 mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b))$ = 340.0 mm
web thickness $t_{w,v}$ = 10.0 mm, flange width, thickness $b_{f,v}$ = 170.0 mm, $t_{f,v}$ = 15.0 mm, weld thickness a_v = 3.0 mm
total beam depth at the connection point $h_{ges} = h_b + h_v$ = 701.4 mm
verification parameters
bolted end-plate connection
thickness t_p = 25.0 mm, width b_p = 190.0 mm, length l_p = 741.4 mm
projections $h_{p,o}$ = 20.0 mm, $h_{p,u}$ = 20.0 mm
bolts in connection:
4 bolt-rows with 2 bolts
of these 2 bolt-rows top in tension (rows 1-2)
and 2 bolt-rows for shear transfer top (rows 3-4)
of these 2 bolt-rows below in tension (rows 3-4)
and 2 bolt-rows for shear transfer below (rows 3-4)
centre distance of the bolts to the lateral edge of the end-plate e_2 = 42.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 92.7 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 108.7 mm
centre distance of the first bolt-row to the free edge of the column (end row) e_1' = 92.7 mm
centre distance of the bolt-rows from each other $p_{1\text{-}2}$ = 100.0 mm, $p_{2\text{-}3}$ = 240.0 mm, $p_{3\text{-}4}$ = 200.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 6.0 mm, angle φ = 85°
beam web: fillet weld, weld thickness a = 4.0 mm
beam flange below: fillet weld, weld thickness a = 6.0 mm, angle φ = 102°
internal forces and moments at the joint periphery refering to the system axes
Lc 1: $N_{b,Ed}$ = 45.20 kN $M_{b,Ed}$ = 250.00 kNm $V_{b,Ed}$ = 84.60 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
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.
welds of the haunch profile are not verified.
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 = 42.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_2 = 42.0 mm < $4 \cdot t + 40$ mm = 98.4 mm
horizontal:	p_2 = 106.0 mm > $2.4 \cdot d_0$ = 52.8 mm,	p_2 = 106.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	e_1 = 92.7 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 92.7 mm < $4 \cdot t + 40$ mm = 98.4 mm
top-below:	e_1 = 92.7 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 92.7 mm < $4 \cdot t + 40$ mm = 98.4 mm
top-below:	p_1 = 100.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 100.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	p_1 = 240.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 240.0 mm > min($14 \cdot t$, 200 mm) = 200.0 mm !!
top-below:	p_1 = 200.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 200.0 mm ≤ min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	e_1 = 108.7 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 108.7 mm > $4 \cdot t + 40$ mm = 98.4 mm !!

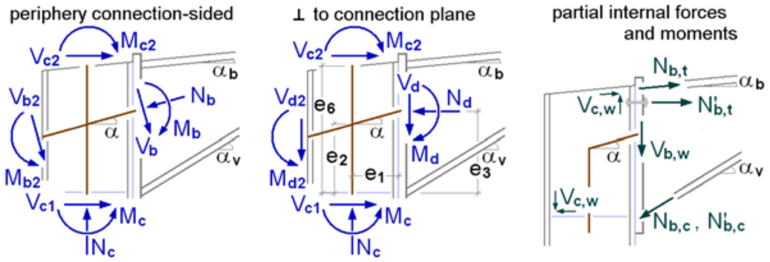
bolt distance from column edge

horizontal:	e_2 = 42.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_2 = 42.0 mm < $4 \cdot t + 40$ mm = 98.4 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: $\alpha_b = 5.00°$, $\alpha_v = 11.80°$ $\Rightarrow$ $\alpha = (\alpha_b+\alpha_v)/2 = 8.40°$
distances: $e_1 = 225.0$ mm, $e_3 = 347.6$ mm, $e_2 = 314.4$ mm, $e_6 = 687.9$ mm

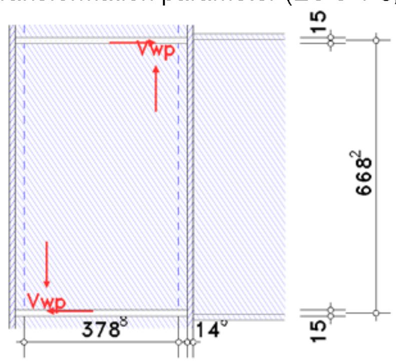
internal forces and moments perpendicular to the connection planes
periphery beam
$N_d = 32.36$ kN, $M_d = 250.00$ kNm, $V_d = 90.30$ kN
periphery column (below, calculated)
$N_c = 90.30$ kN, $M_c = 259.07$ kNm, $V_c = 32.36$ kN

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d \cdot t_p \cdot \tan(\alpha) - V_d \cdot t_p = 247.86$ kNm
$N_{b,t} = (-N_d \cdot z_{bu}/z_b + M'_d/z_b) / \cos(\alpha_b) = 346.23$ kN, $z_b = 687.3$ mm, $z_{bu} = 333.6$ mm
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_v) = 385.42$ kN, $z_b = 687.3$ mm, $z_{bo} = 353.7$ mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = -30.18$ kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v) = 78.82$ kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 41.66$ 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 ($M_{j2} = 0$)

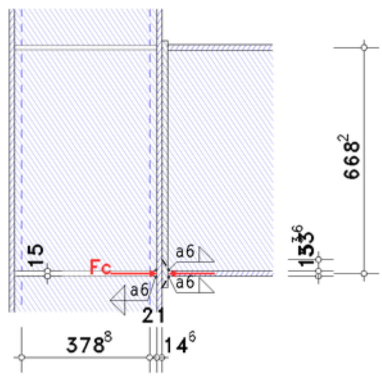


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

slenderness of column web $h_{wc}/t_{wc} = 44.77 < 72 \cdot \varepsilon/\eta = 60.00$ $\Rightarrow$ method applicable
plastic shear resistance without web stiffeners $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_{wp}) / (3^{1/2} \cdot \gamma_{M0}) = 516.52$ kN
proportion of column flange:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/z_{wp} = 15.3$ kN, $z_{wp} = h_r = 621.0$ mm
plastic shear resistance plus proportion of column flange $V_{wp,Rd} = 531.8$ kN
placing of intermediate web stiffeners:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/h_r + 2 \cdot M_{pl,st,Rd})/h_r = 15.33$ kN
plastic shear resistance with transverse stiffeners $V_{wp,Rd} = 547.18$ 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 $\sigma_{com,Ed} = 154.56$ N/mm²



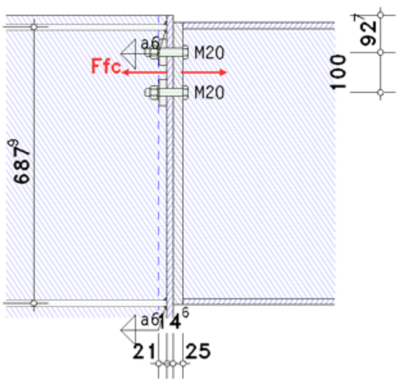
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p = 246.9$ mm
reduction factor $k_w = 1.0$ for $\sigma_{com,Ed} = 154.6$ N/mm² $\leq 0.7 \cdot f_{y,w} = 164.5$ N/mm²
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w{}^2)]^{1/2} = 1.014$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p{}^2 = 0.772$ for $\lambda_p > 0.673$

reduction factor for interaction with shear stress $\beta = 1 \;\Rightarrow\; \omega = 0.848$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 462.44$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 324.55$ kN (decisive)
reinforcement of web with intermediate transverse stiffeners:
assumption: stiffeners do not buckle: section class 1, section class $1 \le 3$ ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 668.2 mm
web height between the flanges $h_{wc} = 420.8$ mm
moment of inertia of stiffeners $I_{st} = 857.38$ cm^4
minimum moment of inertia for $a/h_{wc} = 1.59 \ge 2^{1/2}$: $I_{st,min} = 26.21$ cm^4 $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners $I_T = 10.16$ cm^4
polar moment of inertia of stiffeners $I_p = 94.58$ cm^4
$I_T / I_p \approx 0.107 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web $A_{st} = 28.50$ cm^2
slenderness $\lambda = 0.082$
$\lambda \le 0.2 \;\Rightarrow\;$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd} = 608.9$ kN
maximum resistance:
$F_{c,w,Rd} = 608.9$ kN (resistance with transverse stiffeners)

resistance of the upper beam flange:
effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p = 244.3$ mm
reduction factor $k_w = 1.0$ for $\sigma_{com,Ed} = 154.6$ N/mm^2 $\le 0.7 \cdot f_{y,w} = 164.5$ N/mm^2
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 1.009$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.775$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $\beta = 1 \;\Rightarrow\; \omega = 0.850$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 458.81$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 323.27$ kN (decisive)
reinforcement of web with intermediate transverse stiffeners:
assumption: stiffeners do not buckle: section class 1, section class $1 \le 3$ ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 668.2 mm
web height between the flanges $h_{wc} = 420.8$ mm
moment of inertia of stiffeners $I_{st} = 857.38$ cm^4
minimum moment of inertia for $a/h_{wc} = 1.59 \ge 2^{1/2}$: $I_{st,min} = 26.21$ cm^4 $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners $I_T = 10.16$ cm^4
polar moment of inertia of stiffeners $I_p = 94.58$ cm^4
$I_T / I_p \approx 0.107 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web $A_{st} = 28.50$ cm^2
slenderness $\lambda = 0.082$
$\lambda \le 0.2 \;\Rightarrow\;$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd} = 608.9$ kN
maximum resistance:
$F_{c,w,Rd} = 608.9$ 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: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 178.6$ mm, $l_{eff,cp} = 197.9$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 178.6$ mm
tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 2.24$ kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_y) / \gamma_{M0} = 4.20$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.26$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.51$ kN
$L_b = 78.3$ mm ≤ 121.2 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,2,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd} + 4 \cdot n \cdot M_{bp,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 725.38$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 258.94$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.51$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 258.94$ kN

row 2

effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 178.5$ mm, $l_{eff,cp} = 197.9$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 178.5$ mm

tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 2.24$ kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_y) / \gamma_{M0} = 4.19$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.26$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.51$ kN
$L_b = 78.3$ mm ≤ 121.2 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,2,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd} + 4 \cdot n \cdot M_{bp,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 725.17$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 258.92$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.51$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 258.92$ kN

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

$F_{t,fc,Rd,1} = 258.94$ kN, $l_{eff,1} = 178.6$ mm
$F_{t,fc,Rd,2} = 258.92$ kN, $l_{eff,2} = 178.5$ mm

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

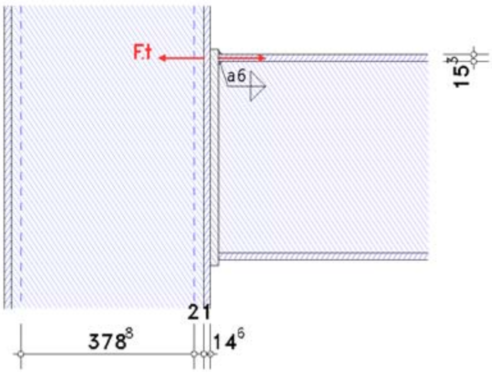
here: number of bolt-rows $n_b = 2$ (between stiffeners)

effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 278.6$ mm, $\Sigma l_{eff,cp} = 397.9$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 278.6$ mm

tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 3.49$ kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_y) / \gamma_{M0} = 6.55$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.26$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 705.02$ kN
$L_b = 78.3$ mm ≤ 155.3 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,2,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd} + 4 \cdot n \cdot M_{bp,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 1131.64$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 490.12$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 705.02$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 490.12$ kN
effective length: $\Sigma l_{eff} = 278.6$ 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 $\beta = 1 \Rightarrow \omega = 0.911$

resistance of an unstiffened column web in transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 359.36$ kN, $b_{eff,t,wc} = 178.6$ mm

row 2

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.911$

resistance of an unstiffened column web in transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 359.27$ kN, $b_{eff,t,wc} = 178.5$ mm

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

$F_{t,wc,Rd,1} = 359.36$ kN, $b_{eff,t,wc} = 178.6$ mm (s. bc 4)

$F_{t,wc,Rd,2} = 359.27$ kN, $b_{eff,t,wc} = 178.5$ mm (s. bc 4)

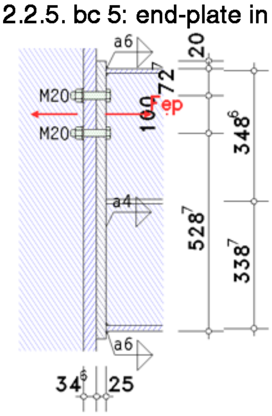
each group of bolt-rows:

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.817$

resistance of an unstiffened column web in transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 502.72$ kN, $b_{eff,t,wc} = 278.6$ 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: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 240.2$ mm, $l_{eff,cp} = 279.4$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 240.2$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.82$ kNm

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.26$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.51$ kN

$L_b = 78.3$ mm > 50.5 mm $= L_b^*$ $\Rightarrow$ no prying forces !

mode 1 and 2: complete yielding of the T-stub flange and possibly coincident bolt failure

$F_{T,1\text{-}2,Rd} = (2 \cdot M_{pl,1,Rd}) / m = 396.58$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.51$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1\text{-}2,Rd}, F_{T,3,Rd}) = 352.51$ 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: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 489.10$ kN (≥ 352.51 kN, not decisive)

row 2

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 230.4$ mm, $l_{eff,cp} = 279.4$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 230.4$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.46$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.26$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.51$ kN
$L_b = 78.3$ mm > 52.6 mm $= L_b^*$ $\Rightarrow$ no prying forces !
mode 1 and 2: complete yielding of the T-stub flange and possibly coincident bolt failure
$F_{T,1\text{-}2,Rd} = (2 \cdot M_{pl,1,Rd}) / m = 380.44$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.51$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1\text{-}2,Rd}, F_{T,3,Rd}) = 352.51$ 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: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 469.20$ kN (≥ 352.51 kN, not decisive)

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

$F_{ep,Rd,1} = 352.51$ kN, $l_{eff,1} = 240.2$ mm
$F_{ep,Rd,2} = 352.51$ kN, $l_{eff,2} = 230.4$ mm

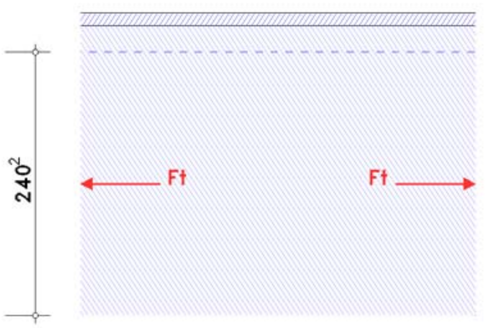
equivalent T-stub flange (group of bolt-rows):

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

effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 275.0$ mm, $\Sigma l_{eff,cp} = 439.7$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 275.0$ mm

tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 10.10$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.26$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 705.02$ kN
$L_b = 78.3$ mm ≤ 88.2 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 1091.86$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 575.94$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 705.02$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 575.94$ 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: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 559.97$ kN
total loading capacity of the T-stub flange: $F_{T,Rd} = F_{T,w,Rd} = 559.97$ kN !!
effective length: $\Sigma l_{eff} = 275.0$ mm, 2 rows

2.2.6. 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 1
effective width $b_{eff,t,wb} = 240.2$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 451.52$ kN

row 2
effective width $b_{eff,t,wb} = 230.4$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 433.15$ kN

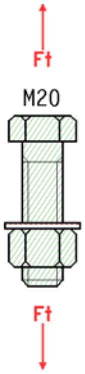
resistance of a beam web in tension (per bolt-row)

$F_{t,wb,Rd,1} = 451.52$ kN, $b_{eff,t,wb} = 240.2$ mm (s. bc 5)
$F_{t,wb,Rd,2} = 433.15$ kN, $b_{eff,t,wb} = 230.4$ mm (s. bc 5)

eine group of bolt-rows decisive:

effective width $b_{eff,t,wb} = 275.0$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 516.95$ kN

2.2.7. bc 10: bolts in tension

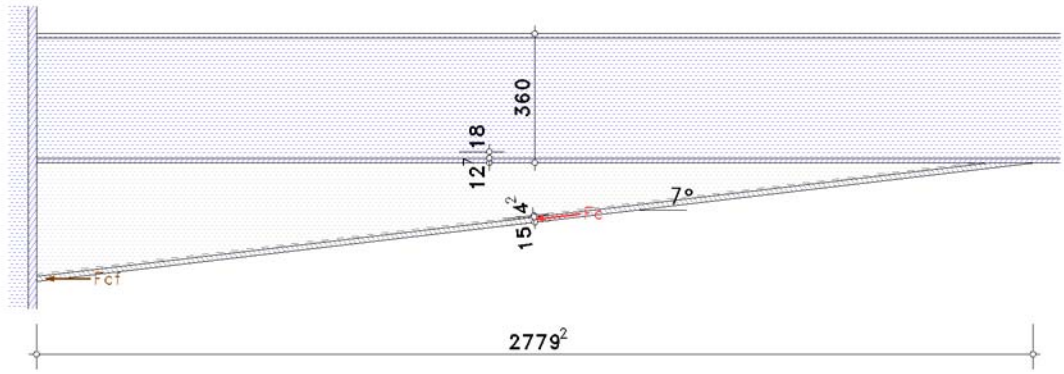


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

tension resistance of one bolt $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.26$ kN, $k_2 = 0.90$
punching shear load capicity of a bolt $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 249.47$ kN, $t_p = 14.6$ mm
tension-/punching shear load capicity for 2 bolts: $\Sigma F_{tp,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 352.51$ kN

2.2.8. bc 20: haunched beam in compression

flange below: section class 1
web: section class 2
total: section class 2
section class of the beam in connection plane: 2
taking into account the moment-shear force-interaction $V_{Ed} = 84.6$ kN
transformation into aspect plane (due to beam slope):
$L_{v,bc20} = L_v/\cos(\alpha_b)-h_b \cdot \tan(\alpha_b) = 2779.2$ mm, $\alpha_{v,bc20} = 6.80°$



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 $h = h_b + h_v = 691.4$ mm
resistance only flange (without web, plastic: section class 2)
haunch flange width $b_{f,v} = 170.0$ mm $<$ max $b_{f,v} = 634.5$ mm ok
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 408.14$ kNm, $W_{pl} = 1736.76$ cm^3
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 603.50$ kN
refering to haunch flange $F_{c,f,Rd}/\cos(\alpha_v) = 607.77$ kN
total loading capacity of a haunched beam in compression
$F_{c,v,Rd} = F_{c,f,Rd} = 607.77$ kN
resistance relative to the connection plane $F_{c,v,Rd} \cdot \cos(\alpha_v) = 594.93$ kN

resistance of the upper beam flange (bc 7):
calculation refering to the plane perpendicular to the beam
stress due to bending with shear force: $V_{Ed} = 84.6$ kN ≤ 361.4 kN $= 0.5 \cdot V_{pl,Rd}/2$ $\Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 552.73$ kNm, $W_{pl} = 2352.06$ cm^3
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 814.40$ kN
resistance relative to the connection plane $F_{c,v,Rd} \cdot \cos(\alpha_b) = 811.30$ 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: $h_1 = 621.0$ mm, $h_2 = 521.0$ 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: $F_{tr,Rd} = 258.9$ kN
row 2: $F_{tr,Rd} = 258.9$ 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: $F_{tr,Rd} = 258.9$ kN
row 2: $F_{tr,Rd} = 231.2$ 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: $F_{tr,Rd} = 258.9$ kN
row 2: $F_{tr,Rd} = 231.2$ kN

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 258.9$ kN
row 2: $F_{tr,Rd} = 231.2$ kN
$\Sigma F_{tr,Rd}^* = 490.1$ kN

deductions acc. to EC 3-1-8, B.3.2.2(7)
decisive basic components: 1, 2, 20
row 1: $F_{tr,Rd} = 258.9$ kN
row 2: $F_{tr,Rd} = 231.2$ kN

check acc. to EC 3-1-8, B.3.2.2(9)
decisive basic component: 10
row 1: $F_{tr,Rd} = 258.9$ kN
row 2: $F_{tr,Rd} = 231.2$ kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 258.9$ kN
row 2: $F_{tr,Rd} = 231.2$ kN
$\Sigma F_{tr,Rd} = 490.1$ kN
potential failure by basic component 4

resistance of flanges (compression)
$\Sigma F_{c,Rd}^* = 1094.4$ kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 281.2$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 490.1$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 1094.4$ kN

2.3.2. shear resistance

shear resistance of column web
decisive basic component: 1
$V_{wp,Rd} = 547.18$ kN

2.3.3. total

$M_{j,Rd} = 281.2$ kNm $N_{j,t,Rd} = 490.1$ kN $N_{j,c,Rd} = 1094.4$ kN $V_{wp,Rd} = 547.2$ kN

2.4. rotational stiffness

stiffness coefficients
equivalent stiffness coefficient for 2 tension-bolt-rows:
1: $k_3 = 3.10$ mm, $k_4 = 16.00$ mm, $k_5 = 38.39$ mm, $k_{10} = 5.00$ mm $\Rightarrow$ $k_{eff,1} = 1 / \Sigma(1/k_{i,1}) = 1.637$ mm
2: $k_3 = 3.10$ mm, $k_4 = 16.00$ mm, $k_5 = 36.83$ mm, $k_{10} = 5.00$ mm $\Rightarrow$ $k_{eff,2} = 1 / \Sigma(1/k_{i,2}) = 1.633$ mm
equivalent internal lever arm $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r) = 575.42$ mm
equivalent stiffness coefficient $k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq} = 3.245$ mm
$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z) = 3.36$ mm
$k_2 = \infty$ (stiffened)
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i) = 114747.6$ kNm/rad, $z = z_{eq} = 575.4$ mm, $\Sigma(1/k_i) = 0.606$ mm^{-1}
$|M_{j,Ed}| = 239.32$ kNm $>$ 2/3 $M_{j,Rd} = 187.49$ kNm $\Rightarrow$ $\mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi} = 1.933$, $\Psi = 2.7$
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu = 59367.8$ kNm/rad
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd} = 0.231°$

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