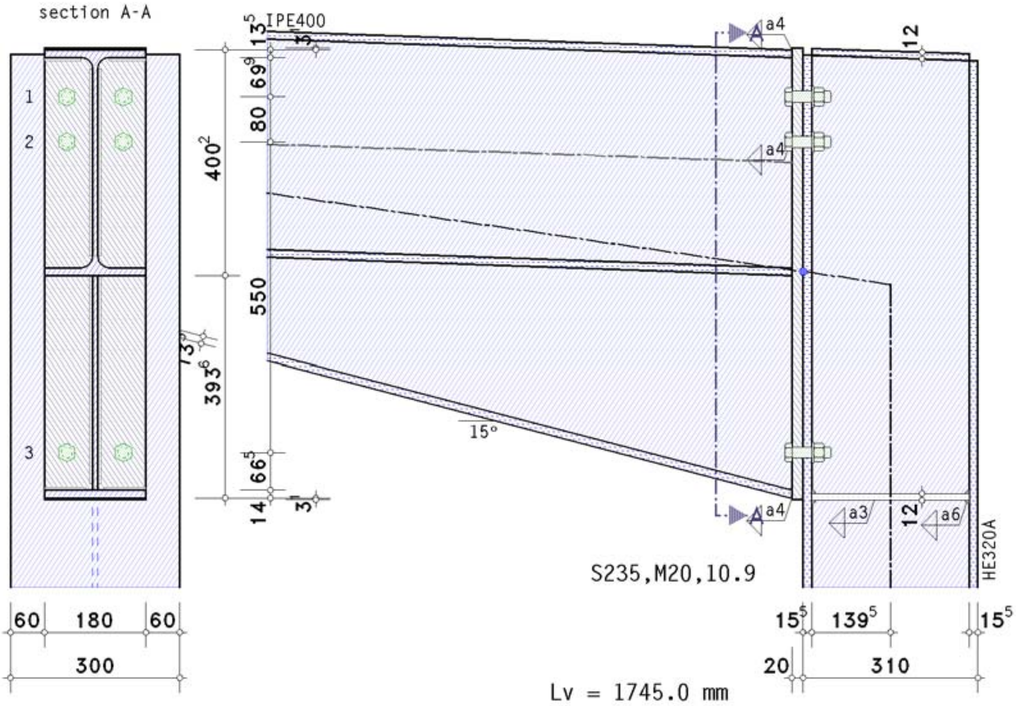
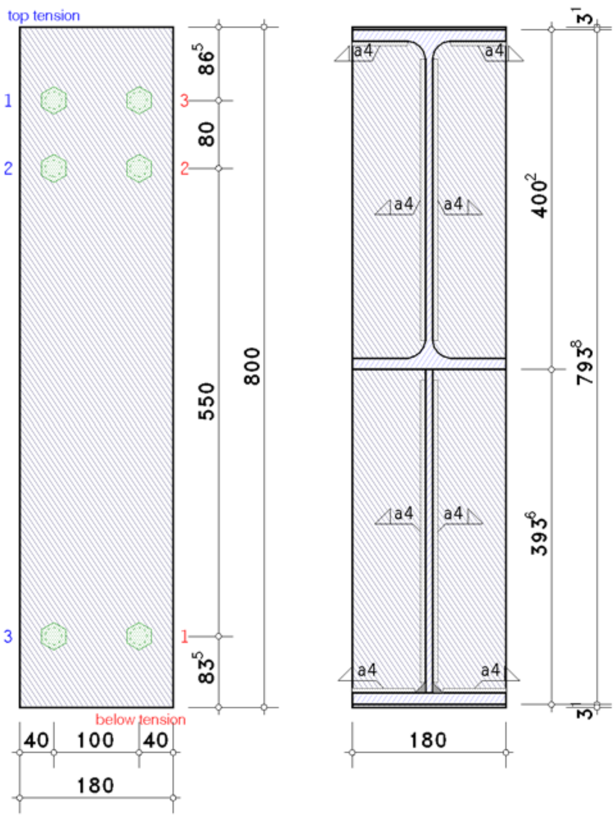
frame corner EC 3-1-8 (04.25), NA: Deutschland 4H-EC3RE version: 6/2026-1b

1. input report



details (section A - A)



steel grade
steel grade S235
column parameters
section HE320A
reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 782.6 mm):
thickness t_{st} = 12.0 mm, width b_{st} = 100.0 mm, length l_{st} = 279.0 mm
recess at stiffeners c_{st} = 30.0 mm
welds $a_{st,f}$ = 6.0 mm, $a_{st,w}$ = 3.0 mm
cap plate t_k = 12.0 mm (structurally effective)
bolts

bolt class 10.9, bolt size M20, normal wrench size
shear plane passes through the unthreaded portion of the bolt

beam parameters

section IPE400

slope angle of section about the horizontal axis α_b = 2.00° $\Rightarrow$ section depth at the joint loc. $h_b = h/\cos(\alpha_b)$ = 400.2 mm

slope angle of haunch about the horizontal axis α_v = 14.60° $\Rightarrow$ haunch angle about the beam axis $\Delta\alpha_v$ = 12.60°

haunch length L_v = 1745.0 mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b))$ = 393.6 mm

web thickness $t_{w,v}$ = 8.6 mm, flange width, thickness $b_{f,v}$ = 180.0 mm, $t_{f,v}$ = 13.5 mm, weld thickness a_v = 8.6 mm

total beam depth at the connection point $h_{ges} = h_b + h_v$ = 793.8 mm

verification parameters

bolted end-plate connection

thickness t_p = 20.0 mm, width b_p = 180.0 mm, length l_p = 800.0 mm

projections $h_{p,o}$ = 3.1 mm, $h_{p,u}$ = 3.1 mm

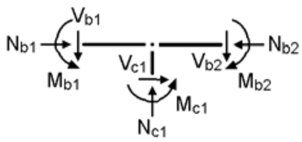
bolts in connection:
3 bolt-rows with 2 bolts
of these 2 bolt-rows top in tension (rows 1-2)
and 1 bolt-row for shear transfer top (row 3)
of these 1 bolt-row below in tension (row 3)
and 2 bolt-rows for shear transfer below (rows 2-3)
centre distance of the bolts to the lateral edge of the end-plate e_2 = 40.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 86.5 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 83.5 mm
centre distance of the first bolt-row to the free edge of the column (end row) e_1' = 86.5 mm
centre distance of the bolt-rows from each other $p_{1\text{-}2}$ = 80.0 mm, $p_{2\text{-}3}$ = 550.0 mm

welds at the connection point:
beam flange top: fillet weld, weld thickness a = 4.0 mm, angle φ = 88°
beam web: fillet weld, weld thickness a = 4.0 mm
beam flange below: fillet weld, weld thickness a = 4.0 mm, angle φ = 105°

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

Lc 1: $N_{b,Ed}$ = 37.70 kN $M_{b,Ed}$ = 291.00 kNm $V_{b,Ed}$ = 94.30 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

certain basic components have been deselected,
that may not ensure the total load-bearing capacity of the joint !

a left-hand joint is transformed to a right-hand joint. All notations and messages apply to this calculation model.

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.
welds are not regarded by calculation the T-stub resistance.
local resistance of weld is not respected.
plated structures- and shear buckling is not investigated.
calculation of T-stub-resistance with the standard method.
local shear resistance of end plate is not respected.

check of data

ok

distances between bolts at end-plate

horizontal:	e_2 = 40.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_2 = 40.0 mm < $4 \cdot t$ + 40 mm = 102.0 mm
horizontal:	p_2 = 100.0 mm > $2.4 \cdot d_0$ = 52.8 mm,	p_2 = 100.0 mm < min(14·t, 200 mm) = 200.0 mm
top-below:	e_1 = 86.5 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 86.5 mm < $4 \cdot t$ + 40 mm = 102.0 mm
top-below:	e_1 = 86.5 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 86.5 mm < $4 \cdot t$ + 40 mm = 102.0 mm
top-below:	p_1 = 80.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 80.0 mm < min(14·t, 200 mm) = 200.0 mm
top-below:	p_1 = 550.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 550.0 mm > min(14·t, 200 mm) = 200.0 mm !!
top-below:	e_1 = 83.5 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 83.5 mm < $4 \cdot t$ + 40 mm = 102.0 mm

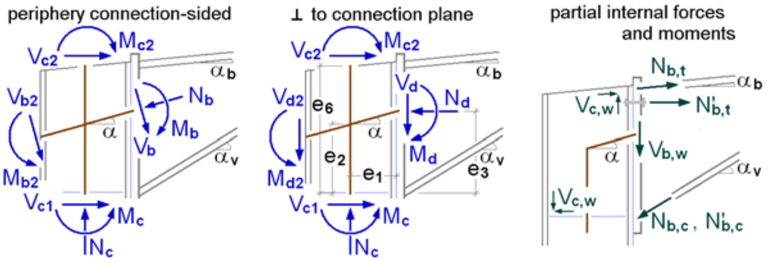
bolt distance from column edge

horizontal:	e_2 = 100.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_2 = 100.0 mm < $4 \cdot t$ + 40 mm = 102.0 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: $\alpha_b = 2.00°$, $\alpha_v = 14.60°$ $\Rightarrow$ $\alpha = (\alpha_b+\alpha_v)/2 = 8.30°$
distances: $e_1 = 155.0$ mm, $e_3 = 398.6$ mm, $e_2 = 376.0$ mm, $e_6 = 782.6$ mm

internal forces and moments perpendicular to the connection planes
periphery beam
$N_d = 23.69$ kN, $M_d = 291.00$ kNm, $V_d = 98.75$ kN
periphery column (below, calculated)
$N_c = 98.75$ kN, $M_c = 296.86$ kNm, $V_c = 23.69$ kN

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d \cdot t_p \cdot \tan(\alpha) - V_d \cdot t_p = 289.09$ kNm
$N_{b,t} = (-N_d \cdot z_{bu}/z_b + M'_d/z_b) / \cos(\alpha_b) = 358.91$ kN, $z_b = 780.1$ mm, $z_{bu} = 391.5$ mm
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_v) = 395.14$ kN, $z_b = 780.1$ mm, $z_{bo} = 388.6$ mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = -12.53$ kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v) = 99.60$ kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 11.68$ kN

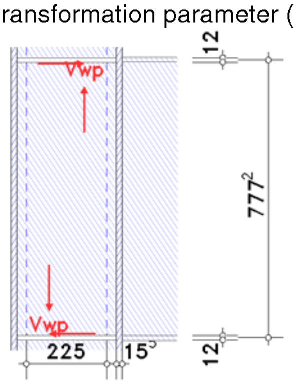
2.2. resistance of cross-section in the periphery

c/t-utilization reg. section class 2
flange below: section class 1, utilization $U_{c/t} = 0.527$
web: section class 3, utilization $U_{c/t} = 1.073$
total: section class 3, c/t-utilization $U_{c/t} = 1.073 > 1$ not ok !!
plastic verification for $N_{Ed} = -23.69$ kN, $M_{y,Ed} = -289.09$ kNm, $V_{z,Ed} = 98.75$ kN
section class of the section 3 > 2 $\Rightarrow$ particular verification is required !!
verification not möglich !!
cross-section cannot be verified using plastic methods !!
elastic verification for $N = -23.69$ kN, $M_y = -289.09$ kNm, $V_z = 98.75$ kN
verification: $\sigma_v = 108.30$ N/mm² $< \sigma_{v,Rd} = 235.00$ N/mm² $\Rightarrow$ $U_\sigma = 0.461 < 1$ ok
c/t-ratio: outstand flange: utilization $U_{c/t} = 0.270 < 1$ ok
internal compression parts: utilization $U_{c/t} = 0.486 < 1$ ok

2.3. basic components

2.3.1. bc 1: Column web panel in shear

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 291.00$ kNm ($M_{j2} = 0$)

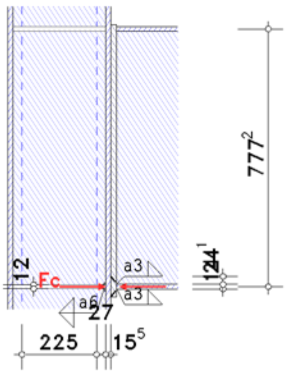


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

slenderness of column web $h_{wc}/t_{wc} = 31.00 < 72 \cdot \varepsilon/\eta = 60.00$ $\Rightarrow$ method applicable
plastic shear resistance without web stiffeners $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_{wp}) / (3^{1/2} \cdot \gamma_{M0}) = 340.69$ kN
proportion of column flange:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/z_{wp} = 24.1$ kN, $z_{wp} = h_r = 703.5$ mm
plastic shear resistance plus proportion of column flange $V_{wp,Rd} = 364.8$ kN
placing of intermediate web stiffeners:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/h_r + 2 \cdot M_{pl,st,Rd})/h_r = 24.08$ kN
plastic shear resistance with transverse stiffeners $V_{wp,Rd} = 388.84$ kN

2.3.2. bc 2: column web in transverse compression

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 291.00$ kNm ($M_{j2} = 0$)
longitudinal compressive stress in column web $\sigma_{com,Ed} = 153.61$ N/mm²

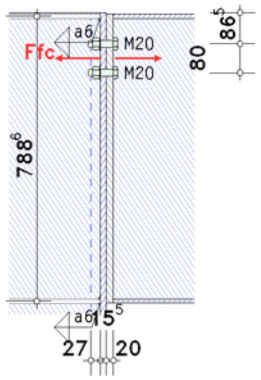


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p = 253.6$ mm
reduction factor $k_w = 1.0$ for $\sigma_{com,Ed} = 153.6$ N/mm² $\leq 0.7 \cdot f_{y,w} = 164.5$ N/mm²
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.828$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.887$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.731$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 392.25$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 316.35$ 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 = 777.2 mm
web height between the flanges $h_{wc} = 279.0$ mm
moment of inertia of stiffeners $I_{st} = 912.93$ cm⁴
minimum moment of inertia for $a/h_{wc} = 2.79 \geq 2^{1/2}$: $I_{st,min} = 15.25$ cm⁴ $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners $I_T = 5.76$ cm⁴
polar moment of inertia of stiffeners $I_p = 101.44$ cm⁴
$I_T / I_p \approx 0.057 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web $A_{st} = 25.08$ cm²
slenderness $\lambda = 0.049$
$\lambda \leq 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd} = 535.8$ kN
maximum resistance:
$F_{c,w,Rd} = 535.8$ kN (resistance with transverse stiffeners)

resistance of the upper beam flange:
effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p = 253.5$ mm
reduction factor $k_w = 1.0$ for $\sigma_{com,Ed} = 153.6$ N/mm² $\leq 0.7 \cdot f_{y,w} = 164.5$ N/mm²
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.827$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.887$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.731$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 392.18$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 316.33$ 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 = 777.2 mm
web height between the flanges $h_{wc} = 279.0$ mm
moment of inertia of stiffeners $I_{st} = 912.93$ cm⁴
minimum moment of inertia for $a/h_{wc} = 2.79 \geq 2^{1/2}$: $I_{st,min} = 15.25$ cm⁴ $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners $I_T = 5.76$ cm⁴
polar moment of inertia of stiffeners $I_p = 101.44$ cm⁴
$I_T / I_p \approx 0.057 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of the stiffened webs with transverse compression:
area of stiffeners incl. web $A_{st} = 25.08$ cm²
slenderness $\lambda = 0.049$
$\lambda \leq 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd} = 535.8$ kN
maximum resistance:
$F_{c,w,Rd} = 535.8$ kN (resistance with transverse stiffeners)

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: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 150.2$ mm, $l_{eff,cp} = 150.2$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 196.8$ mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 2.12$ kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 2.78$ 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

$L_b = 54.3$ 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

$F_{T,1\text{-}2,Rd} = (2 \cdot M_{pl,1,Rd}) / m = 177.37$ kN

mode 3: bolt failure

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

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1\text{-}2,Rd}, F_{T,3,Rd}) = 177.37$ kN

row 2

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 150.2$ mm, $l_{eff,cp} = 150.2$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 220.6$ mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 2.12$ kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 3.11$ 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

$L_b = 54.3$ 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

$F_{T,1\text{-}2,Rd} = (2 \cdot M_{pl,1,Rd}) / m = 177.37$ kN

mode 3: bolt failure

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

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1\text{-}2,Rd}, F_{T,3,Rd}) = 177.37$ kN

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

$F_{t,fc,Rd,1} = 177.37$ kN, $l_{eff,1} = 150.2$ mm

$F_{t,fc,Rd,2} = 177.37$ kN, $l_{eff,2} = 150.2$ 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: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 276.8$ mm, $\Sigma l_{eff,cp} = 310.2$ mm

in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 276.8$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 3.91$ 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

$L_b = 54.3$ mm ≤ 57.1 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 653.88$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 536.99$ kN

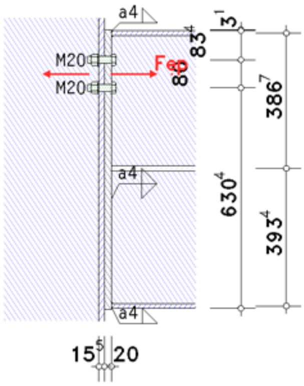
mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 705.02$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 536.99$ kN

effective length: $\Sigma l_{eff} = 276.8$ mm, 2 rows

2.3.4. 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: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 214.7$ mm, $l_{eff,cp} = 258.7$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 214.7$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.05$ 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
$L_b = 54.3$ mm ≤ 87.6 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 490.15$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 298.02$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.51$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 298.02$ kN

row 2
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 214.7$ mm, $l_{eff,cp} = 258.7$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 214.7$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.05$ 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
$L_b = 54.3$ mm ≤ 87.6 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 490.15$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 298.02$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.51$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 298.02$ kN

resistances and effective lengths of end-plate in bending (per bolt-row):
$F_{ep,Rd,1} = 298.02$ kN, $l_{eff,1} = 214.7$ mm
$F_{ep,Rd,2} = 298.02$ kN, $l_{eff,2} = 214.7$ 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: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 227.3$ mm, $\Sigma l_{eff,cp} = 369.4$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 227.3$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.34$ 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
$L_b = 54.3$ mm ≤ 165.4 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 519.03$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 479.05$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 705.02$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 479.05$ kN
effective length: $\Sigma l_{eff} = 227.3$ mm, 2 rows

2.4. shear resistance

shear resistance of column web
$V_{wp,Rd}$ = 388.8 kN

2.5. verifications

2.5.1. verification of the connection capacity with partial internal forces and moments

shear force in column web:
$V_{c,w,Ed} = (M_{d1}-M_{d2})/z - V_c/2$ = 425.17 kN, z = 665.9 mm
tension force in the bolt-rows:
$N'_{b,t} = (-N_d \cdot z_{bu} + M_d) / z$ = 423.19 kN, $z = z_{eq}$ = 665.9 mm, z_{bu} = 388.6 mm
compression force refering to the tension force in the bolt-rows:
$N'_{b,c} = (N_d \cdot z_{bo} + M_d) / z$ = 446.88 kN, $z = z_{eq}$ = 665.9 mm, z_{bo} = 277.3 mm

bc 1: $F_{Rd} = V_{wp,Rd}$ = 388.8 kN, $F_{Ed} = |V_{c,w,Ed}|$ = 425.17 kN
F_{Ed} = 425.2 kN > F_{Rd} = 388.8 kN ⇒ U = 1.093 > 1 not ok !!
bc 2: $F_{Rd} = F_{c,w,Rd}$ = 535.8 kN, $F_{Ed} = N'_{b,c}$ = 446.88 kN
F_{Ed} = 446.9 kN < F_{Rd} = 535.8 kN ⇒ U = 0.834 < 1 ok
$F_{Rd} = F_{c,w,Rd}$ = 535.8 kN, $F_{Ed} = -N'_{b,t}$ = -423.19 kN ≤ 0 **no verification**
bc 4: $F_{Rd} = \min(\Sigma F_{t,fc,Rd,i}, F_{t,fc,Rd})$ = 353.5 kN, $F_{Ed} = N'_{b,t}$ = 423.19 kN
F_{Ed} = 423.2 kN > F_{Rd} = 353.5 kN ⇒ U = 1.197 > 1 not ok !!
bc 5: $F_{Rd} = \min(\Sigma F_{t,ep,Rd,i}, F_{t,ep,Rd})$ = 484.3 kN, $F_{Ed} = N'_{b,t}$ = 423.19 kN
F_{Ed} = 423.2 kN < F_{Rd} = 484.3 kN ⇒ U = 0.874 < 1 ok

utilization partial internal forces and moments U_{bc} = 1.197 > 1 not ok !!

2.5.1.1. shear resistance

shear force in column web:
$V_{c,w,Ed} = (M_{d1}-M_{d2})/z - V_c/2$ = 425.17 kN, z = 665.9 mm

$V_{c,w,Ed}/V_{wp,Rd}$ = 1.093 > 1 not ok !!

2.5.2. verification of web stiffeners

compression stiffener (below)
$F_{c,Ed}$ = 425.30 kN
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 167.99 kN, $H = F \cdot e_F/e_H$ = 39.14 kN
assumption: stiffeners do not buckle: section class 1, section class 1 ≤ 3 ok
cross-section at flange
compression resistance $N_{c,Rd}$ = 197.40 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 181.15 kN
F_{Ed} = 181.15 kN < F_{Rd} = 197.40 kN ⇒ U = 0.918 < 1 ok
cross-section at web
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0})$ = 356.56 kN
design value: $F_{Ed} = F$ = 167.99 kN
F_{Ed} = 167.99 kN < F_{Rd} = 356.56 kN ⇒ U = 0.471 < 1 ok
flange welds
design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 1199.94 kN/m, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 279.56 kN/m, b_1 = 70.0 mm
$\sigma_{1,w,Ed}$ = 215.7 N/mm² < $f_{1w,d}$ = 360.0 N/mm² ⇒ U = 0.599 < 1 ok
$\sigma_{2,w,Ed}$ = 200.0 N/mm² < $f_{2w,d}$ = 259.2 N/mm² ⇒ U = 0.772 < 1 ok
web welds
design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 383.54 kN/m, l_1 = 219.0 mm
$\sigma_{1,w,Ed}$ = 221.4 N/mm² < $f_{1w,d}$ = 360.0 N/mm² ⇒ U = 0.615 < 1 ok
stiffener in tension (top)
$F_{t,Ed}$ = 401.85 kN
cross-section
tension resistance $N_{t,Rd}$ = 846.00 kN
F_{Ed} = 401.85 kN < F_{Rd} = 846.00 kN ⇒ U = 0.475 < 1 ok

2.5.3. verification result

maximum utilization: max U = 1.197 > 1 not ok !!
failure at verification for shear in column web: U = 1.093
failure at verification with partial internal forces and moments: U = 1.197

3. final result

maximum utilization: max U = 1.197 > 1 **not ok !!**

resistance not ensured !!

total loading capacity of the connection may not be guaranteed (s. notes) !

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 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