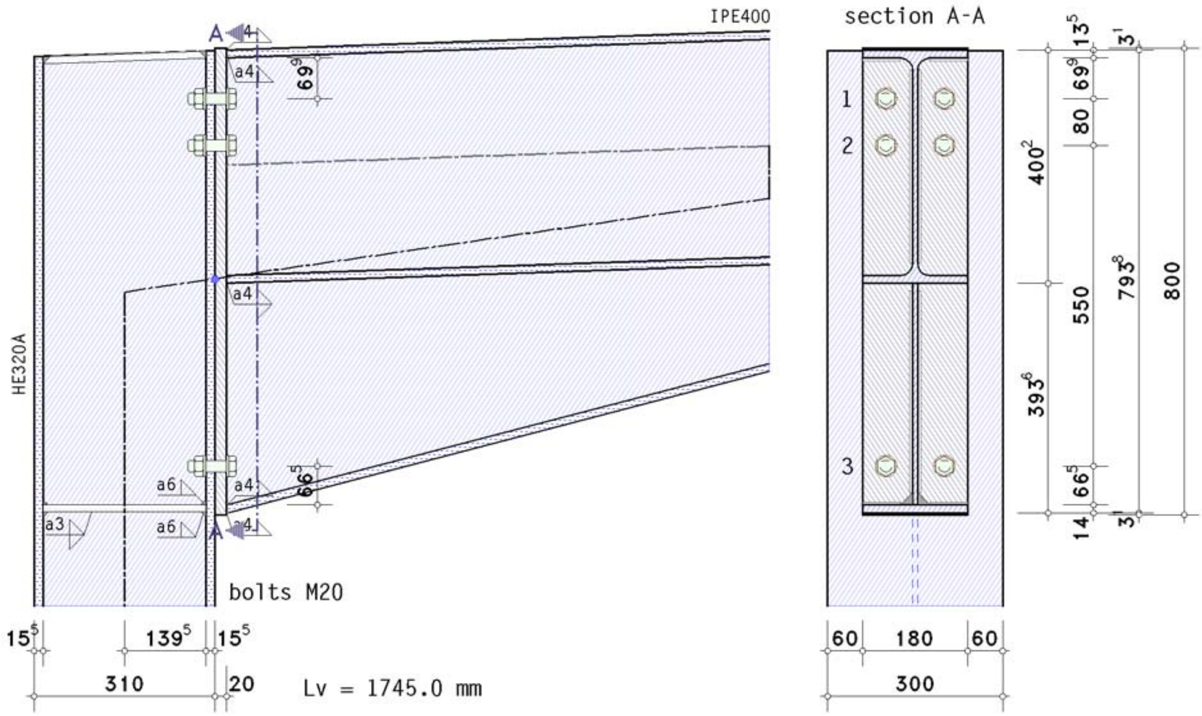
POS. 6: KINDMANN/KRÜGER 11.5.11

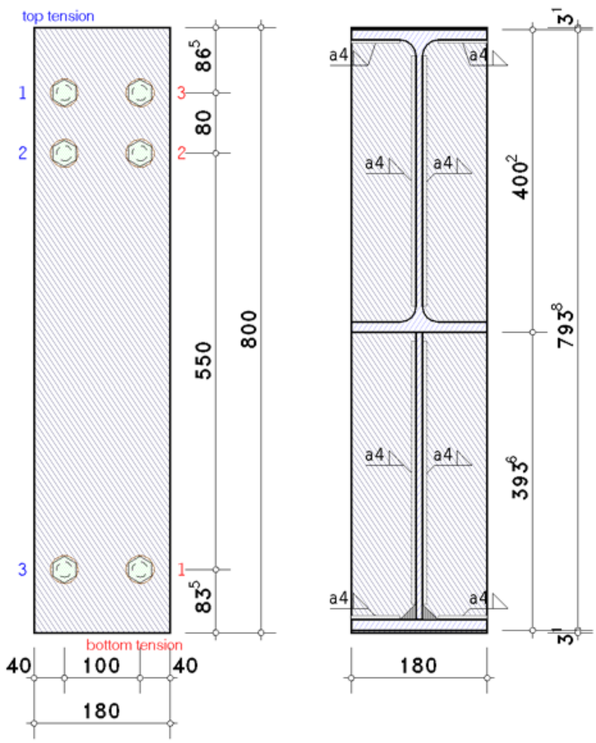
4H-EC3RE version: 5/2014-1x

frame corner

EC 3-1-8 (12.10), NA: Deutschland



details



steel grade
steel grade S 235
column parameters
section HE320A
reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 778.9 mm):
  thickness t_{st} = 12.0 mm, width b_{st} = 100.0 mm
  recess at stiffeners c_{st} = 30.0 mm
  welds $a_{st,f}$ = 6.0 mm, $a_{st,w}$ = 3.0 mm

beam parameters
section IPE400
section angle of inclination about the horizontal axis α_b = 2.00° ⇒ section depth at the joint loc. $h_b = h/\cos(\alpha_b)$ = 400.2 mm
haunch angle of inclination about the horizontal axis α_v = 14.60° ⇒ 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
total beam depth at the connection point $h_{ges} = h_b + h_v$ = 793.8 mm
bolts
bolt: bolt class 10.9, bolt size M20
large width across flats (high strength bolt), slip resistant connection (prestressed)
shear plane passes through the unthreaded portion of the bolt
class of friction surfaces A (slip factor μ = 0.50)
verification parameters
bolted end-plate joint:
thickness t_p = 20.0 mm, length l_p = 800.0 mm, width b_p = 180.0 mm
projections $h_{p,o}$ = 3.1 mm, $h_{p,u}$ = 3.1 mm
bolts at the connection point:
3 bolt-row(s) with 2 bolts each
of these 2 bolt-rows top (M^+) in tension (rows 1-2)
and 1 bolt-row for shear transfer at tension top (row 3)
of these 1 bolt-row bottom (M^-) in tension (row 3)
and 2 bolt-rows for shear transfer at tension bottom (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' = 82.9 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 bottom: fillet weld, weld thickness a = 4.0 mm, angle φ = 105°
internal forces and moments in the periphery of the connection refering to the system axes
Lk 1: $N_{b1,Ed}$ = 37.70 kN $M_{b1,Ed}$ = 291.00 kNm $V_{b1,Ed}$ = 94.30 kN

Component method

notes
high strength bolts have to be controlled prestressed, bolt category E (tension), C (shear).
In haunched beams the bottom flange of the rolled section is not considered. A fictive welded section is shaped from the top beam flange, the beam web and the haunch flange.
there are several basic components selected which perhaps do not ensure the total loading capacity of the joint.
no verification of haunch connection to beam.
no verfication for welds within the connection.

distances between bolt-rows at end-plate

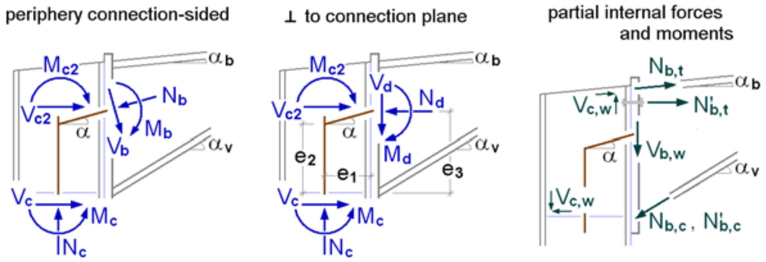
edge dist.:	e_2 = 40.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_2 = 40.0 mm < $4 \cdot t_{min}$ + 40 mm = 102.0 mm
pitch:	p_2 = 100.0 mm > $2.4 \cdot d_0$ = 52.8 mm,	p_2 = 100.0 mm < min($14 \cdot t_{min}$, 200 mm) = 200.0 mm
edge dist.:	e_1 = 86.5 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 86.5 mm < $4 \cdot t_1$ + 40 mm = 120.0 mm
edge dist.:	e_1 = 82.9 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 82.9 mm < $4 \cdot t_2$ + 40 mm = 102.0 mm
pitch:	p_1 = 80.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 80.0 mm < min($14 \cdot t_{min}$, 200 mm) = 200.0 mm
pitch:	p_1 = 550.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 550.0 mm > min($14 \cdot t_{min}$, 200 mm) = 200.0 mm !!
edge dist.:	e_1 = 83.5 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 83.5 mm < $4 \cdot t_1$ + 40 mm = 120.0 mm

horizontal distance of bolts from column edge

edge dist.:	e_2 = 100.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_2 = 100.0 mm < $4 \cdot t_{min}$ + 40 mm = 102.0 mm

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

design values



angle of inclination: $\alpha_b = 2.0°$, $\alpha_v = 14.6°$ $\Rightarrow$ $\alpha = (\alpha_b+\alpha_v)/2 = 8.3°$

periphery column (bottom)

$N_{c,Ed} = N_{b,Ed}\cdot\sin(\alpha) + V_{b,Ed}\cdot\cos(\alpha) = 98.75$ kN

$M_{c,Ed} = M_{b,Ed} - V_{c,Ed}\cdot e_3 + N_{c,Ed}\cdot e_1 = 297.07$ kNm, $e_1 = 155.0$ mm, $e_3 = 389.8$ mm

$V_{c,Ed} = N_{b,Ed}\cdot\cos(\alpha) - V_{b,Ed}\cdot\sin(\alpha) = 23.69$ kN

internal forces and moments perpendicular to the connection plane

$N_d = N_{b,Ed}\cdot\cos(\alpha) - V_{b,Ed}\cdot\sin(\alpha) = 23.69$ kN

$M_d = M_{b,Ed} = 291.00$ kNm

$V_d = N_{b,Ed}\cdot\sin(\alpha) + V_{b,Ed}\cdot\cos(\alpha) = 98.75$ kN

partial internal forces and moments

internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d\cdot t_{ep}\cdot\tan(\alpha) - V_d\cdot t_{ep} = 289.09$ kN

$N_{b,t} = (-N_d\cdot z_{bu}/z_b + M'_d/z_b) / \cos(\alpha_b) = 359.03$ kN, $z_b = 780.1$ mm, $z_{bu} = 387.5$ mm

$N_{b,c} = (N_d\cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_v) = 395.26$ kN, $z_b = 780.1$ mm, $z_{bo} = 392.6$ mm

$V_{b,w} = V_d + N_{b,c}\cdot\sin(\alpha_v) - N_{b,t}\cdot\sin(\alpha_b) = 185.86$ kN

resistance of cross section

column

plastic cross-sectional check for $M_{Ed} = -297.07$ kNm, $N_{Ed} = -98.75$ kN, $V_{Ed} = 23.69$ kN

elastic stresses: max $\sigma_x = 19.28$ kN/cm², min $\sigma_x = -20.88$ kN/cm², max $\tau = 0.93$ kN/cm², max $\sigma_v = 20.88$ kN/cm²

plastic design resistance moment: $M_{pl,N,Q} = 366.42$ kNm

utilizations: design resistance $U_\sigma = 0.814 < 1$ ok., c/t-ratio $U_{c/t} = 0.511 < 1$ ok.

beam

plastic cross-sectional check for $M_{Ed} = -289.09$ kNm, $N_{Ed} = -23.69$ kN, $V_{Ed} = 98.75$ kN

elastic stresses: max $\sigma_x = 10.54$ kN/cm², min $\sigma_x = -10.82$ kN/cm², max $\tau = 1.70$ kN/cm², max $\sigma_v = 10.83$ kN/cm²

plastic design resistance moment: $M_{pl,N,Q} = 747.93$ kNm

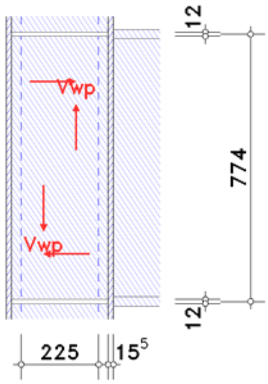
utilizations: design resistance $U_\sigma = 0.393 < 1$ ok., c/t-ratio $U_{c/t} = 0.486 < 1$ ok.

basic components

end-plate joint: selected basic component(s): 1, 2, 4, 5

basic component 1: Column web panel in shear

transformation parameter (table 5.4) $\beta = 1.0$



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

slenderness of column web $d_c/t_{wc} = 25.00 < 69\cdot\varepsilon = 69.00$ $\Rightarrow$ method applicable

plastic shear resistance without stiffeners $V_{wp,Rd} = (0.9\cdot f_{y,wc}\cdot A_{vc}) / (3^{1/2}\cdot\gamma_{M0}) = 502.3$ kN

placing of intermediate web stiffeners:

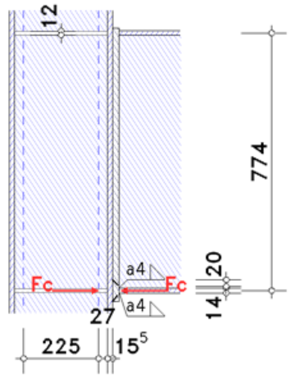
additional design resistance $V_{wp,add,Rd} = 4\cdot M_{pl,fc,Rd}/d_{st} = 21.9$ kN

$V_{wp,add,Rd} > 2\cdot(M_{pl,fc,Rd}+M_{pl,st,Rd})/d_{st} = 15.3$ kN $\Rightarrow$ $V_{wp,add,Rd} = 15.3$ kN

plastic shear resistance with transverse stiffeners $V_{wp,Rd} = 517.6$ kN

basic component 2: column web in transverse compression

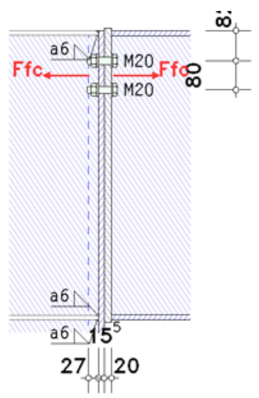
transformation parameter (table 5.4) $\beta = 1.0$
maximum longitudinal compressive stress in column web $\sigma_{com,Ed} = 153.71$ N/mm²



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

reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: $c/t = 8.3 \cdot \varepsilon \leq 33 \cdot \varepsilon \Rightarrow$ section class $1 \leq 2$ ok.
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) $a = 774.0$ 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 $a/h_{wc} = 2.77 \geq 2^{1/2}$: $I_{st,min} = 15.25$ 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.
design resistance of 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

basic component 4: column flange in bending



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

equivalent T-stub flange (each bolt-row decisive):
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: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 193.2$ mm
design 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: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 2.73$ kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.80$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 354.74$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 297.40$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.80$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 297.40$ 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

design 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

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.80$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 354.74$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 311.80$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.80$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 311.80$ kN

design resistance of column flange in bending (per bolt-row)

$F_{fc,Rd,1} = 297.4$ kN, $l_{eff,1} = 150.2$ mm

$F_{fc,Rd,2} = 311.8$ kN, $l_{eff,1} = 150.2$ mm

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

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

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

in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 273.2$ mm, $\Sigma l_{eff,cp} = 310.2$ mm

in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 273.2$ mm

design tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 3.86$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 705.60$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 645.28$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 535.40$ kN

mode 3: bolt failure

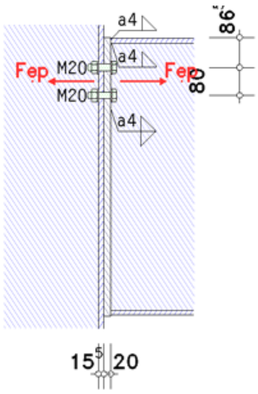
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 705.60$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 535.40$ kN

design resistance of column flange in bending (group of bolts, 2 rows)

$F_{fc,Rd} = 535.4$ kN, $\Sigma l_{eff,1} = 273.2$ mm

<u>basic component 5: end-plate in bending</u>



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 bolt-row decisive):

here: number of bolt rows $n_b = 1$

row 1

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 218.8$ mm, $l_{eff,cp} = 258.7$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 218.8$ mm

design tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.14$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.80$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 499.41$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 300.51$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.80$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 300.51$ kN

design resistance of the weld: $F_{w,Rd} = 181.87$ kN per side

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 design 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

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.80$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 298.16$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.80$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 298.16$ kN

design resistance of the weld: $F_{w,Rd} = 178.50$ kN per side

design resistances of end-plate in bending (per bolt-row):

$F_{ep,Rd,1} = 300.5$ kN, $l_{eff,1} = 218.8$ mm

$F_{ep,Rd,2} = 298.2$ kN, $l_{eff,1} = 214.7$ mm

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

here: number of bolt rows $n_b = 2$

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

in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 298.8$ mm, $\Sigma l_{eff,cp} = 418.7$ mm

in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 298.8$ mm

design tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 7.02$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 705.60$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 682.04$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 520.67$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 705.60$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 520.67$ kN

design resistance of the weld: $F_{w,Rd} = 248.38$ kN per side

total loading capacity of the T-stub flange: $F_{T,Rd} = 2 \cdot F_{w,Rd} = 496.76$ kN

design resistance of end-plate in bending (group of bolts, 2 row(s))

$F_{t,ep,Rd} = 496.8$ kN, $\Sigma l_{eff,1} = 298.8$ mm

verifications

equivalent lever arm for 2 bolt-rows: $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r) = 665.0$ mm

verification of the connection design capacity with partial internal forces and moments

shear force in column web panel:

$V_{wp,Ed} = (M_{d1,w} - M_{d2,w})/z - V_c/2 = 427.98$ kN, $M_{d,w} = 292.5$ kNm , $z = z_{eq} = 665.0$ mm

tension force in the bolt rows:

$N'_{b,t} = (-N_d \cdot z_{bu} + M_d)/z = 423.90$ kN, $z = z_{eq} = 665.0$ mm, $z_{bu} = 384.6$ mm

compression force refering to the tension force in the bolt rows:

$N'_{b,c} = (N_d \cdot z_{bo} + M_d)/z = 447.59$ kN, $z = z_{eq} = 665.0$ mm, $z_{bo} = 280.4$ mm

Gk 1: $F_{Rd} = V_{wp,Rd}/\beta = 517.6$ kN, $F_{Ed} = |V_{wp,Ed}| = 427.98$ kN
$F_{Ed} = 428.0$ kN $<$ $F_{Rd} = 517.6$ kN $\Rightarrow$ $U = 0.827 < 1$ ok.

Gk 2: $F_{Rd} = F_{c,w,Rd} = 535.8$ kN, $F_{Ed} = N'_{b,c} = 447.59$ kN
$F_{Ed} = 447.6$ kN $<$ $F_{Rd} = 535.8$ kN $\Rightarrow$ $U = 0.835 < 1$ ok.

Gk 4: $F_{Rd} = \min(\Sigma F_{t,fc,Rd,i}, F_{t,fc,Rd}) = 535.4$ kN, $F_{Ed} = N'_{b,t} = 423.90$ kN
$F_{Ed} = 423.9$ kN $<$ $F_{Rd} = 535.4$ kN $\Rightarrow$ $U = 0.792 < 1$ ok.

Gk 5: $F_{Rd} = \min(\Sigma F_{t,ep,Rd,i}, F_{t,ep,Rd}) = 496.8$ kN, $F_{Ed} = N'_{b,t} = 423.90$ kN
$F_{Ed} = 423.9$ kN $<$ $F_{Rd} = 496.8$ kN $\Rightarrow$ $U = 0.853 < 1$ ok.

total utilization $U_{Gk} = 0.853 < 1$ ok.

verification of web stiffeners (ribs)

column

compression stiffener

$F_{c,Ed} = 447.59$ kN

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 176.8$ kN, $H = F \cdot e_F/e_H = 41.2$ kN

assumption: stiffeners do not buckle: $c/t = 8.3 \cdot \varepsilon \leq 33 \cdot \varepsilon$ $\Rightarrow$ section class $1 \leq 2$ ok.

cross section at flange

compression resistance: $F_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 197.40$ kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 190.7$ kN

$F_{Ed} = 190.7$ kN $<$ $F_{Rd} = 197.4$ kN $\Rightarrow$ $U = 0.966 < 1$ ok.

cross section at web

shear resistance: $V_{p,Rd} = (f_y \cdot A_v) / (3^{1/2} \cdot \gamma_{M0}) = 454.25$ kN

design value: $F_{Ed} = F = 176.8$ kN

$F_{Ed} = 176.8$ kN $<$ $F_{Rd} = 454.2$ kN $\Rightarrow$ U = 0.389 $<$ 1 **ok.**

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 12.63$ kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 2.94$ kN/cm, $b_1 = 70.0$ mm

$\sigma_{1,w,Ed} = 22.70$ kN/cm² $<$ $f_{1,w,Rd} = 36.00$ kN/cm² $\Rightarrow$ utilization U = 0.630 $<$ 1 **ok.**

$\sigma_{2,w,Ed} = 21.05$ kN/cm² $<$ $f_{2,w,Rd} = 25.92$ kN/cm² $\Rightarrow$ utilization U = 0.812 $<$ 1 **ok.**

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 4.04$ kN/cm, $l_1 = 219.0$ mm

$\sigma_{1,w,Ed} = 23.30$ kN/cm² $<$ $f_{1,w,Rd} = 36.00$ kN/cm² $\Rightarrow$ utilization U = 0.647 $<$ 1 **ok.**

stiffener in tension

$F_{t,Ed} = 423.90$ kN

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 167.4$ kN, $H = F \cdot e_F/e_H = 39.0$ kN

cross section at flange

tension resistance: $F_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = 197.40$ kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 180.6$ kN

$F_{Ed} = 180.6$ kN $<$ $F_{Rd} = 197.4$ kN $\Rightarrow$ U = 0.915 $<$ 1 **ok.**

cross section at web

shear resistance: $V_{p,Rd} = (f_y \cdot A_v) / (3^{1/2} \cdot \gamma_{M0}) = 454.25$ kN

design value: $F_{Ed} = F = 167.4$ kN

$F_{Ed} = 167.4$ kN $<$ $F_{Rd} = 454.2$ kN $\Rightarrow$ U = 0.369 $<$ 1 **ok.**

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 11.96$ kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 2.79$ kN/cm, $b_1 = 70.0$ mm

$\sigma_{1,w,Ed} = 21.50$ kN/cm² $<$ $f_{1,w,Rd} = 36.00$ kN/cm² $\Rightarrow$ utilization U = 0.597 $<$ 1 **ok.**

$\sigma_{2,w,Ed} = 19.93$ kN/cm² $<$ $f_{2,w,Rd} = 25.92$ kN/cm² $\Rightarrow$ utilization U = 0.769 $<$ 1 **ok.**

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 3.82$ kN/cm, $l_1 = 219.0$ mm

$\sigma_{1,w,Ed} = 22.07$ kN/cm² $<$ $f_{1,w,Rd} = 36.00$ kN/cm² $\Rightarrow$ utilization U = 0.613 $<$ 1 **ok.**

verification result

maximum utilization: max U = 0.966 $<$ 1 **ok.**

Final result

maximum utilization: max U = 0.966 $<$ 1 **ok.**

verification succeeded