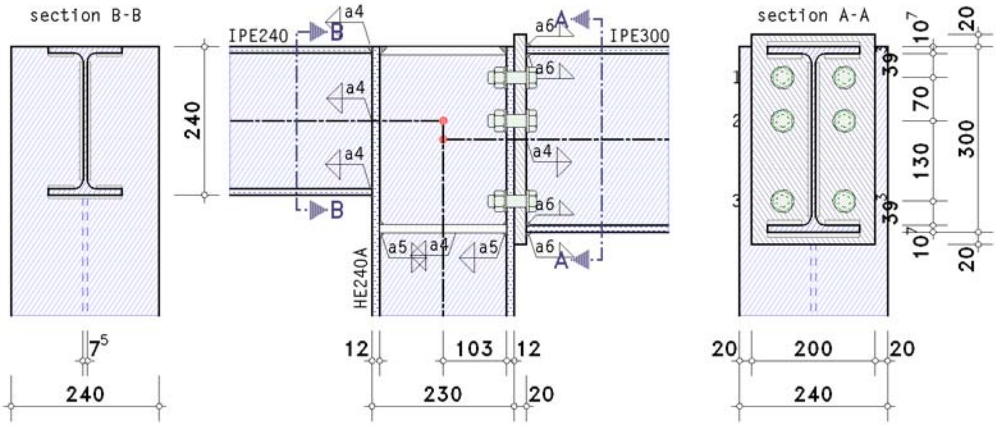
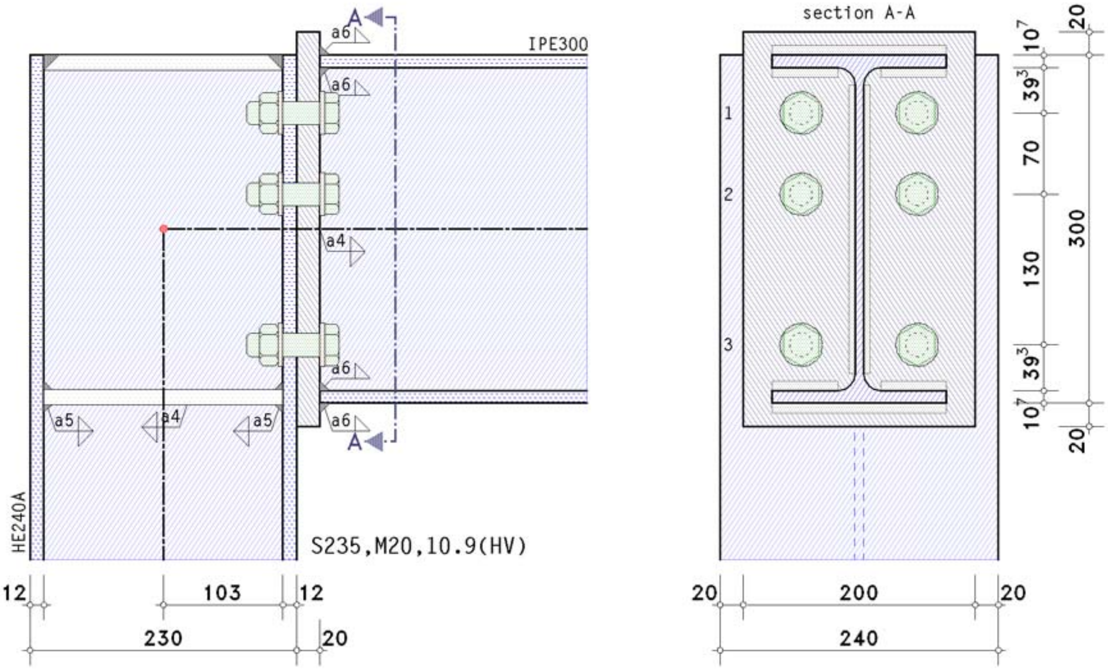
POS. 11: BEISPIEL T-CONNECTION

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

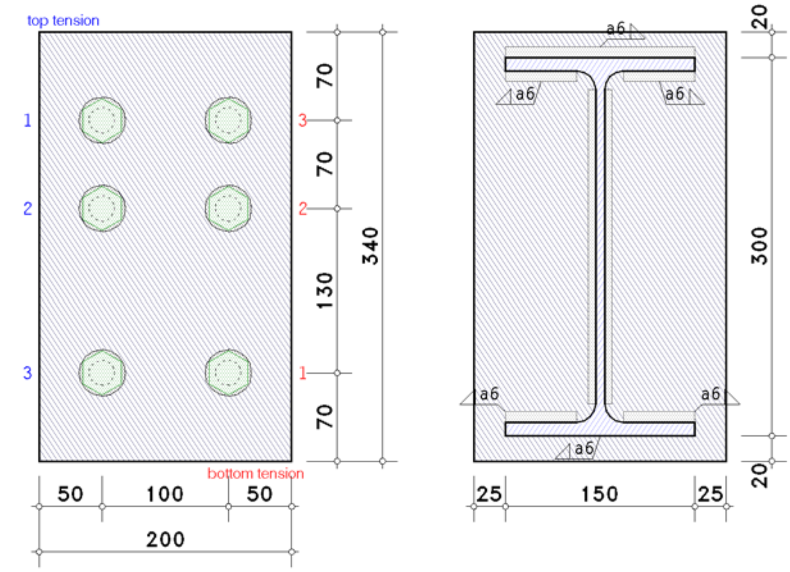
1. input report



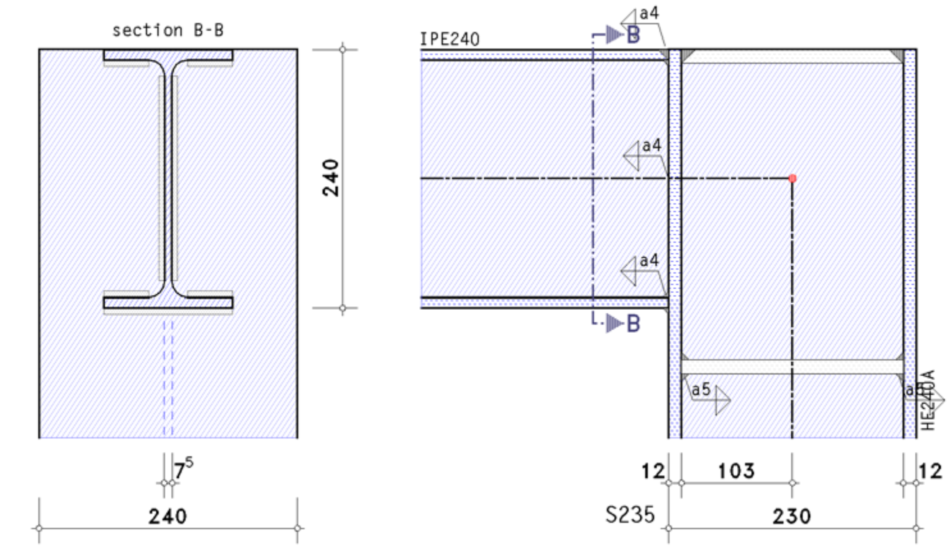
connection right



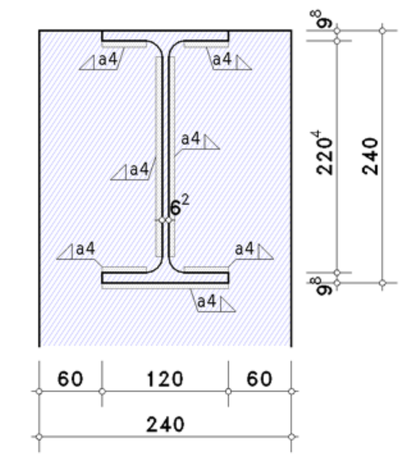
details (section A - A)



connection left



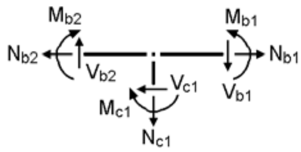
details (section B - B)



According to EC 3-1-8, 5.3 in a double-sided beam-columnn-joint each joint is modelled separately.
centroidal axes of beams do not meet at a point (Δ = 30.0 mm) !!

steel grade
steel grade S235
column parameters
section HE240A
reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 288.1 mm):
thickness t_{st} = 13.0 mm, width b_{st} = 116.3 mm, length l_{st} = 206.0 mm
recess at stiffeners c_{st} = 31.5 mm
welds $a_{st,f}$ = 5.0 mm, $a_{st,w}$ = 4.0 mm

double-sided beam-column joint, right
bolts
bolt class 10.9, bolt size M20
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}^{*} = 0.7 \cdot f_{yb} \cdot A_s$ = 154.3 kN)
shear plane passes through the unthreaded portion of the bolt
beam parameters
section IPE300
verification parameters
bolted end-plate connection:
thickness t_p = 20.0 mm, width b_p = 200.0 mm, length l_p = 340.0 mm
projections $h_{p,o}$ = 20.0 mm, $h_{p,u}$ = 20.0 mm
bolts in connection:
  3 bolt-rows with 2 bolts
  all bolt-rows considered individually
  all bolt-rows for shear transfer (rows 1-3)
  bolt groups generated automatically, considering all groups reg. row 1
  centre distance of the bolts to the lateral edge of the end-plate e_2 = 50.0 mm
  centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 70.0 mm
  centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 70.0 mm
  centre distance of the first bolt-row to the free edge of the column (end row) e_1' = 50.0 mm
  centre distance of the bolt-rows from each other p_{1-2} = 70.0 mm, p_{2-3} = 130.0 mm
welds at the connection point:
  beam flange top: fillet weld, weld thickness a = 6.0 mm
  beam web: fillet weld, weld thickness a = 4.0 mm
  beam flange bottom: fillet weld, weld thickness a = 6.0 mm
double-sided beam-column joint, left
beam parameters
section IPE240
verification parameters
welds at the connection point:
  beam flange top: fillet weld, weld thickness a = 4.0 mm
  beam web: fillet weld, weld thickness a = 4.0 mm
  beam flange bottom: fillet weld, weld thickness a = 4.0 mm
internal forces and moments in the intersection point of system axes
Lk 1: Import LK 1
  $N_{j,b1,Ed}$ = -25.08 kN $M_{j,b1,Ed}$ = -53.71 kNm $V_{j,b1,Ed}$ = 61.01 kN (right)
  $N_{j,b2,Ed}$ = -17.20 kN $M_{j,b2,Ed}$ = -46.12 kNm $V_{j,b2,Ed}$ = -60.20 kN (left)
  $N_{j,c1,Ed}$ = -121.21 kN $M_{j,c1,Ed}$ = -7.58 kNm $V_{j,c1,Ed}$ = -7.59 kN (bottom)



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

notes
no verification for cross-sections.

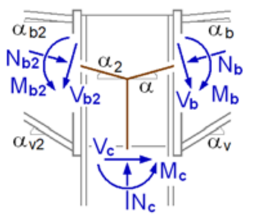
2. Lk 1: Import LK 1

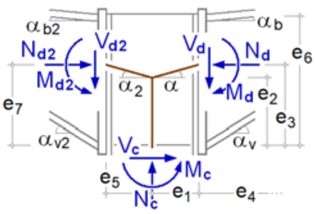
2.1. connection right

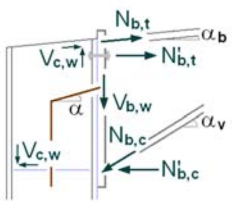
notes
connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.

2.1.1. design values

internal forces at node periphery connection ⊥ zur connection plane partial internal forces and moments







slope angle: $\alpha_b = \alpha = \alpha_v = 0°$
distance: e_1 = 115.0 mm, e_3 = 144.6 mm, e_2 = 144.6 mm, e_5 = 115.0 mm, e_7 = 174.6 mm, e_6 = 288.1 mm

internal forces and moments perpendicular to the connection planes
periphery beam (right)
N_d = 25.08 kN, M_d = 46.69 kNm, V_d = 61.01 kN

periphery beam (left)
N_{d2} = 17.20 kN, M_{d2} = 39.20 kNm, V_{d2} = 60.20 kN
periphery column (bottom)
N_c = 121.21 kN, M_c = 6.48 kNm, V_c = 7.59 kN
partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_{ep}$ = 45.47 kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 144.63 kN, z_b = 289.3 mm, z_{bu} = 144.6 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b$ = 169.71 kN, z_b = 289.3 mm, z_{bo} = 144.6 mm

basic component 1 is not calculated !!

2.1.2. connection capacity

transformation parameter: β_j = 0.141

2.1.2.1. moment resistance

distance of tension-bolt-rows from centre of compression: h_1 = 244.6 mm, h_2 = 174.6 mm, h_3 = 44.6 mm

resistance per bolt-row (tension)
row 1: $F_{tr,Rd}$ = 254.8 kN
row 2: $F_{tr,Rd}$ = 196.0 kN
row 3: $F_{tr,Rd}$ = 255.2 kN
$\Sigma F_{tr,Rd}^*$ = 706.0 kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd}$ = 254.8 kN
row 2: $F_{tr,Rd}$ = 196.0 kN
row 3: $F_{tr,Rd}$ = 59.4 kN
$\Sigma F_{tr,Rd}$ = 510.1 kN
potential failure by basic component 4, 7

resistance of flanges
$\Sigma F_{c,Rd}^*$ = 1020.3 kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r)$ = 99.2 kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^*$ = 706.0 kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^*$ = 1020.3 kN

2.1.2.2. shear/bearing resistance

resistance per bolt-row
row 1: $F_{vr,Rd}$ = 146.0 kN
row 2: $F_{vr,Rd}$ = 169.0 kN
row 3: $F_{vr,Rd}$ = 265.3 kN
$\Sigma F_{vr,Rd}$ = 580.4 kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd}$ = 580.4 kN

2.1.2.3. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd}$ = 674.59 kN
welds: $F_{w,Rd}$ = 413.36 kN
shear resistance of end plate: $V_{ep,Rd} = F_{w,Rd}$ = 413.36 kN

2.1.2.4. total

$M_{j,Rd}$ = 99.2 kNm $N_{j,t,Rd}$ = 706.0 kN $N_{j,c,Rd}$ = 1020.3 kN $V_{j,Rd}$ = 580.4 kN $V_{ep,Rd}$ = 413.4 kN

2.1.3. verifications

2.1.3.1. verification of the connection capacity by means of the component method

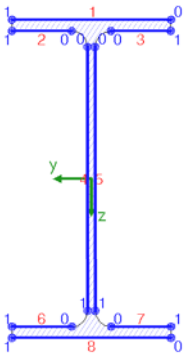
axial force: $N_{b,Ed} = |N_d| = 25.08$ kN $<$ $5\% \cdot N_{pl,Rd} = 63.23$ kN $\Rightarrow$ moment resistance
internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu} = 43.06$ kNm, $z_{bu} = 144.6$ mm
shear force: $V_{Ed} = |V_d| = 61.01$ kN

$M_{Ed}/M_{j,Rd} = 0.434 < 1$ ok
shear/bearing resistance at 43.4% utilization of moment resistance $V_{j,Rd} = 751.8$ kN
$V_{Ed}/V_{j,Rd} = 0.081 < 1$ ok
$V_{Ed}/V_{ep,Rd} = 0.148 < 1$ ok

2.1.3.2. verification of welds at beam section

weld 1: beam flange in tension outer
welds 2,3: beam flange in tension inner
welds 4,5: beam web double-sided
weld 8: beam flange in compression outer
welds 6,7: beam flange in compression inner

calculation section:



weld 1:	$a_w = 6.0$ mm	$l_w = 150.0$ mm
weld 2:	$a_w = 6.0$ mm	$l_w = 56.5$ mm
weld 3:	siehe weld 2	
weld 4:	$a_w = 4.0$ mm	$l_w = 248.6$ mm
weld 5:	siehe weld 4	
weld 6:	$a_w = 6.0$ mm	$l_w = 56.5$ mm
weld 7:	siehe weld 6	
weld 8:	$a_w = 6.0$ mm	$l_w = 150.0$ mm

design values refering to centroid of the section:
$N_{Ed} = -25.08$ kN, $M_{y,Ed} = -46.69$ kNm, $V_{z,Ed} = 61.01$ kN
cross-sectional properties refering to centroid of the line cross-section:
$\Sigma A_w = 51.44$ cm², $A_{w,z} = 19.89$ cm², $\Sigma l_w = 102.3$ cm
$I_{w,y} = 7703.18$ cm⁴, $I_{w,z} = 672.40$ cm⁴, $W_{w,t} = 43.05$ cm³, $\Delta z_w = 0.0$ mm
verifications in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x} = 86.04$ N/mm²		$\Rightarrow U_w = 0.338 < 1$ ok
weld 2,	pt. 0:	$\sigma_{w,x} = 79.55$ N/mm²		$\Rightarrow U_w = 0.313 < 1$ ok
weld 4,	pt. 0:	$\sigma_{w,x} = 70.46$ N/mm²	$\tau_{w,z} = 30.68$ N/mm²	$\Rightarrow U_w = 0.314 < 1$ ok
	pt. 1:	$\sigma_{w,x} = -80.21$ N/mm²	$\tau_{w,z} = 30.68$ N/mm²	$\Rightarrow U_w = 0.348 < 1$ ok
weld 6,	pt. 0:	$\sigma_{w,x} = -89.31$ N/mm²		$\Rightarrow U_w = 0.351 < 1$ ok
weld 8,	pt. 0:	$\sigma_{w,x} = -95.79$ N/mm²		$\Rightarrow U_w = 0.376 < 1$ ok

Result:
weld 8, pt. 0: $\sigma_{w,x} = -95.79$ N/mm²
Max: $\sigma_{1,w,Ed} = 13.55$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm²,
$\sigma_{2,w,Ed} = 6.77$ kN/cm² $<$ $f_{2w,d} = 25.92$ kN/cm² $\Rightarrow$ $U_w = 0.376 < 1$ ok

2.1.3.3. verification of web stiffeners

compression stiffener
$F_{c,Ed} = 241.81$ kN
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 96.0$ kN, $H = F \cdot e_F/e_H = 34.4$ kN
assumption: stiffeners do not buckle: $c/t = 8.9 \cdot \varepsilon \leq 33 \cdot \varepsilon$ $\Rightarrow$ section class $1 \leq 2$ ok
cross-section at flange
compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 258.91$ kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 113.0$ kN
$F_{Ed} = 113.0$ kN $<$ $F_{Rd} = 258.9$ kN $\Rightarrow$ $U = 0.436 < 1$ ok
cross-section at web
shear resistance $V_{Rd} = 363.34$ kN
design value: $F_{Ed} = F = 96.0$ kN
$F_{Ed} = 96.0$ kN $<$ $F_{Rd} = 363.3$ kN $\Rightarrow$ $U = 0.264 < 1$ ok
flange welds
design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 5.66$ kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 2.03$ kN/cm, $b_1 = 84.8$ mm
0% decrease of stress by pressure contact
$\sigma_{1,w,Ed} = 13.33$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm² $\Rightarrow$ $U = 0.370 < 1$ ok
$\sigma_{2,w,Ed} = 11.32$ kN/cm² $<$ $f_{2w,d} = 25.92$ kN/cm² $\Rightarrow$ $U = 0.437 < 1$ ok
web welds
design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 3.36$ kN/cm, $l_1 = 143.0$ mm
0% decrease of stress by pressure contact
$\sigma_{1,w,Ed} = 14.53$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm² $\Rightarrow$ $U = 0.404 < 1$ ok

stiffener in tension
$F_{t,Ed}$ = 216.73 kN
forces per rib
$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 86.0 kN, $H = F \cdot e_F/e_H$ = 30.8 kN
cross-section at flange
tension resistance $N_{t,Rd}$ = 258.91 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 101.3 kN
F_{Ed} = 101.3 kN $<$ F_{Rd} = 258.9 kN $\Rightarrow$ U = 0.391 $<$ 1 ok
cross-section at web
shear resistance V_{Rd} = 363.34 kN
design value: $F_{Ed} = F$ = 86.0 kN
F_{Ed} = 86.0 kN $<$ F_{Rd} = 363.3 kN $\Rightarrow$ U = 0.237 $<$ 1 ok

2.1.3.4. elastic verification of the shear area

column web
requirements concerning stiffeners: s. verification of web stiffeners
requirements concerning shear area: shear buckling: $h_p/t_p = 27.47 \le 72/(\eta \cdot \varepsilon) = 60.00$ ok
internal forces and moments at web (sign definition of statics):
N_1 = -17.20 kN, M_1 = -39.56 kNm, V_1 = -60.20 kN
N_3 = -121.21 kN, M_3 = -6.48 kNm, V_3 = -7.59 kN
N_4 = -25.08 kN, M_4 = -47.05 kNm, V_4 = 61.01 kN
dimensions of the shear area: h_b = 213.5 mm, h_t = 206.0 mm, h_l = 217.0 mm, h_r = 276.1 mm
stresses within the shear area:
τ_b = 8.3 N/mm², τ_t = 8.5 N/mm², τ_l = 48.1 N/mm², τ_r = 36.1 N/mm²
verification of the shear area:
max τ_{Ed} = 48.1 N/mm² $<$ τ_{Rd} = 135.7 N/mm² $\Rightarrow$ U = 0.355 $<$ 1 ok

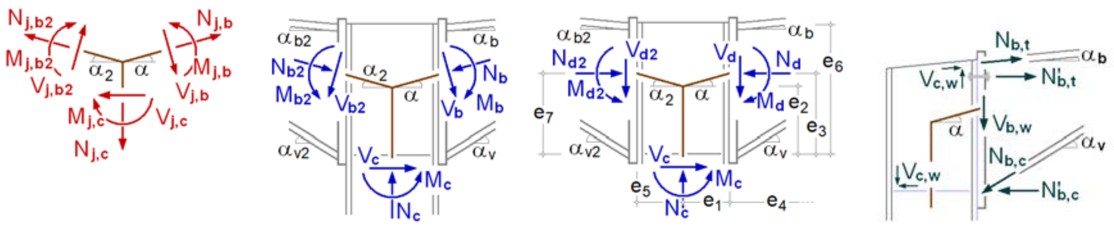
2.1.3.5. verification result

maximum utilization: max U = 0.437 $<$ 1 ok

2.2. connection left

2.2.1. design values

internal forces at node   periphery connection ⊥ zur connection plane   partial internal forces and moments



slope angle: $\alpha_b = \alpha = \alpha_v = 0°$
distance: e_1 = 115.0 mm, e_3 = 174.6 mm, e_2 = 174.6 mm, e_5 = 115.0 mm, e_7 = 144.6 mm, e_6 = 288.1 mm

internal forces and moments perpendicular to the connection planes
periphery beam (right)
N_d = 17.20 kN, M_d = 39.20 kNm, V_d = 60.20 kN
periphery beam (left)
N_{d2} = 25.08 kN, M_{d2} = 46.69 kNm, V_{d2} = 61.01 kN
periphery column (bottom)
N_c = 121.21 kN, M_c = -6.26 kNm, V_c = -7.59 kN
partial internal forces and moments
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M_d/z_b$ = 161.69 kN, z_b = 230.2 mm, z_{bu} = 115.1 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M_d/z_b$ = 178.89 kN, z_b = 230.2 mm, z_{bo} = 115.1 mm

basic component 1 is not calculated !!

2.2.2. connection capacity

transformation parameter: β_j = 0.164

2.2.2.1. moment resistance

distance between tension force and centre of compression: $z = 230.2$ mm

resistance
$F_{Rd} = 297.5$ kN

resistance of flanges (compression)
$\Sigma F_{c,Rd}^{*} = 595.1$ kN

moment resistance
$M_{j,Rd} = F_{Rd} \cdot z = 68.5$ kNm
tension resistance
$N_{j,t,Rd} = F_{t,Rd} = 733.2$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^{*} = 595.1$ kN

2.2.3. verifications

2.2.3.1. verification of the connection capacity by means of the component method

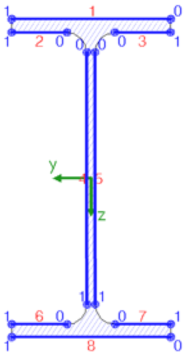
axial force: $N_{b,Ed} = |N_d| = 17.20$ kN $<$ 5%$\cdot N_{pl,Rd} = 45.96$ kN $\Rightarrow$ moment resistance
internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu} = 37.22$ kNm, $z_{bu} = 115.1$ mm
shear force: $V_{Ed} = |V_d| = 60.20$ kN

$M_{Ed}/M_{j,Rd} = 0.543 < 1$ ok

2.2.3.2. verification of welds at beam section

weld 1: beam flange in tension outer
weld 8: beam flange in compression outer
welds 2,3: beam flange in tension inner
welds 4,5: beam web double-sided
welds 6,7: beam flange in compression inner

calculation section:



weld 1:	$a_w = 4.0$ mm	$l_w = 120.0$ mm
weld 2:	$a_w = 4.0$ mm	$l_w = 41.9$ mm
weld 3:	siehe weld 2	
weld 4:	$a_w = 4.0$ mm	$l_w = 190.4$ mm
weld 5:	siehe weld 4	
weld 6:	$a_w = 4.0$ mm	$l_w = 41.9$ mm
weld 7:	siehe weld 6	
weld 8:	$a_w = 4.0$ mm	$l_w = 120.0$ mm

design values refering to centroid of the section:
$N_{Ed} = -17.20$ kN, $M_{y,Ed} = -39.20$ kNm, $V_{z,Ed} = 60.20$ kN
cross-sectional properties refering to centroid of the line cross-section:
$\Sigma A_w = 31.54$ cm², $A_{w,z} = 15.23$ cm², $\Sigma l_w = 78.8$ cm
$I_{w,y} = 2656.70$ cm⁴, $I_{w,z} = 228.70$ cm⁴, $W_{w,t} = 31.29$ cm³, $\Delta z_w = 0.0$ mm
verifications in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x} = 171.61$ N/mm²		$\Rightarrow U_w = 0.674 < 1$ ok
weld 2,	pt. 0:	$\sigma_{w,x} = 157.15$ N/mm²		$\Rightarrow U_w = 0.617 < 1$ ok
weld 4,	pt. 0:	$\sigma_{w,x} = 135.02$ N/mm²	$\tau_{w,z} = 39.52$ N/mm²	$\Rightarrow U_w = 0.563 < 1$ ok
	pt. 1:	$\sigma_{w,x} = -145.93$ N/mm²	$\tau_{w,z} = 39.52$ N/mm²	$\Rightarrow U_w = 0.604 < 1$ ok
weld 6,	pt. 0:	$\sigma_{w,x} = -168.06$ N/mm²		$\Rightarrow U_w = 0.660 < 1$ ok
weld 8,	pt. 0:	$\sigma_{w,x} = -182.52$ N/mm²		$\Rightarrow U_w = 0.717 < 1$ ok

Result:
weld 8, pt. 0: $\sigma_{w,x} = -182.52$ N/mm²
Max: $\sigma_{1,w,Ed} = 25.81$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm²,
$\sigma_{2,w,Ed} = 12.91$ kN/cm² $< f_{2w,d} = 25.92$ kN/cm² $\Rightarrow U_w = 0.717 < 1$ ok

2.2.3.3. verification of web stiffeners

compression stiffener
$F_{c,Ed} = 180.46$ kN
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 71.6$ kN, $H = F \cdot e_F/e_H = 25.7$ kN
assumption: stiffeners do not buckle: $c/t = 8.9 \cdot \varepsilon \leq 33 \cdot \varepsilon \Rightarrow$ section class $1 \leq 2$ ok
cross-section at flange
compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 258.91$ kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 84.3$ kN
$F_{Ed} = 84.3$ kN $< F_{Rd} = 258.9$ kN $\Rightarrow U = 0.326 < 1$ ok

cross-section at web
shear resistance V_{Rd} = 363.34 kN
design value: F_{Ed} = F = 71.6 kN
F_{Ed} = 71.6 kN < F_{Rd} = 363.3 kN ⇒ U = 0.197 < 1 ok
flange welds
design values: $F_{Ed}(\sigma_s)$ = F / (2·b_1) = 4.23 kN/cm, $F_{Ed}(\tau_p)$ = H / (2·b_1) = 1.52 kN/cm, b_1 = 84.8 mm
0% decrease of stress by pressure contact
$\sigma_{1,w,Ed}$ = 9.95 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² ⇒ U = 0.276 < 1 ok
$\sigma_{2,w,Ed}$ = 8.45 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² ⇒ U = 0.326 < 1 ok
web welds
design value: $F_{Ed}(\tau_p)$ = F / (2·l_1) = 2.50 kN/cm, l_1 = 143.0 mm
0% decrease of stress by pressure contact
$\sigma_{1,w,Ed}$ = 10.84 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² ⇒ U = 0.301 < 1 ok
stiffener in tension
$F_{t,Ed}$ = 163.26 kN
forces per rib
F = 0.5·$F_{t,Ed}$ · (b_f-2·r-t_w)/b_f = 64.8 kN, H = F · e_F/e_H = 23.2 kN
cross-section at flange
tension resistance $N_{t,Rd}$ = 258.91 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 76.3 kN
F_{Ed} = 76.3 kN < F_{Rd} = 258.9 kN ⇒ U = 0.295 < 1 ok
cross-section at web
shear resistance V_{Rd} = 363.34 kN
design value: F_{Ed} = F = 64.8 kN
F_{Ed} = 64.8 kN < F_{Rd} = 363.3 kN ⇒ U = 0.178 < 1 ok

2.2.3.4. elastic verification of the shear area

column web
requirements concerning stiffeners: s. verification of web stiffeners
requirements concerning shear area: shear buckling: $h_p/t_p = 27.47 \leq 72/(\eta \cdot \varepsilon) = 60.00$ ok
internal forces and moments at web (sign definition of statics):
N_1 = -25.08 kN, M_1 = -47.05 kNm, V_1 = -61.01 kN
N_3 = -121.21 kN, M_3 = 6.26 kNm, V_3 = 7.59 kN
N_4 = -17.20 kN, M_4 = -39.56 kNm, V_4 = 60.20 kN
dimensions of the shear area: h_b = 213.3 mm, h_t = 206.0 mm, h_l = 276.1 mm, h_r = 217.0 mm
stresses within the shear area:
τ_b = 8.3 N/mm², τ_t = 8.5 N/mm², τ_l = 35.6 N/mm², τ_r = 47.4 N/mm²
verification of the shear area:
max τ_{Ed} = 47.4 N/mm² < τ_{Rd} = 135.7 N/mm² ⇒ U = 0.350 < 1 ok

2.2.3.5. verification result

maximum utilization: max U = 0.717 < 1 ok

3. final result

utilization of the connection

Lk	U_j	equilibrium ΣH	ΣV	ΣM	
--		kN	kN	kNm	
1	0.717*	0.29	0.00	0.00	ok

U_j: utilization of the connection; tolerances of equilibrium 1 kN / 1 kNm
*) maximum utilization

maximum utilization: max U = 0.717 < 1 ok

verification succeeded

4. Regulations

DIN EN 1990, Eurocode 0: Grundlagen der Tragwerksplanung;
Deutsche Fassung EN 1990:2002 + A1:2005 + A1:2005/AC:2010, Ausgabe Dezember 2010
DIN EN 1990/NA, Nationaler Anhang zur DIN EN 1990, Ausgabe Dezember 2010

DIN EN 1993-1-1, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau;
Deutsche Fassung EN 1993-1-1:2005 + AC:2009, Ausgabe Dezember 2010
DIN EN 1993-1-1/A1, Ergänzungen zur DIN EN 1993-1-1, Ausgabe Juli 2014
DIN EN 1993-1-1/NA, Nationaler Anhang zur DIN EN 1993-1-1, Ausgabe September 2017

DIN EN 1993-1-8, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -

Teil 1-8: Bemessung von Anschlüssen;
Deutsche Fassung EN 1993-1-8:2005 + AC:2009, Ausgabe Dezember 2010
DIN EN 1993-1-8/NA, Nationaler Anhang zur DIN EN 1993-1-8, Ausgabe Dezember 2010