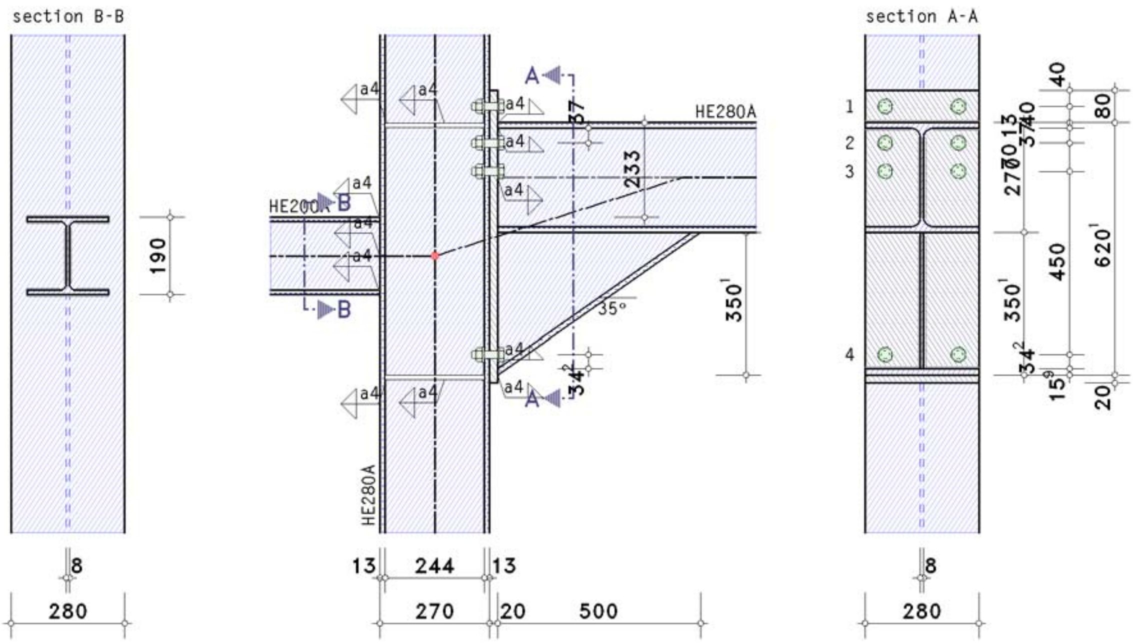
POS. 80: BEIDS. ANSCHL. BSP 1

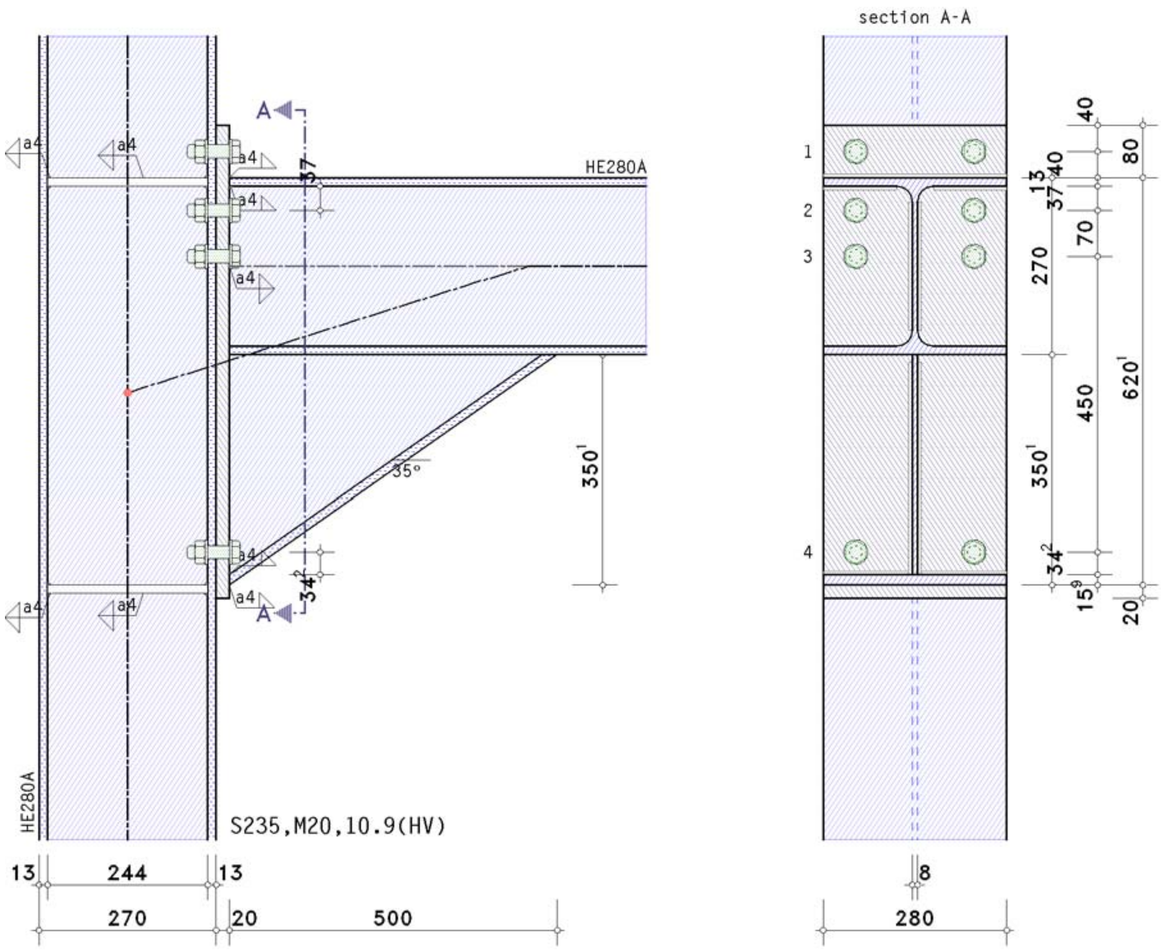
4H-ec3BT version: 4/2013-5l

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

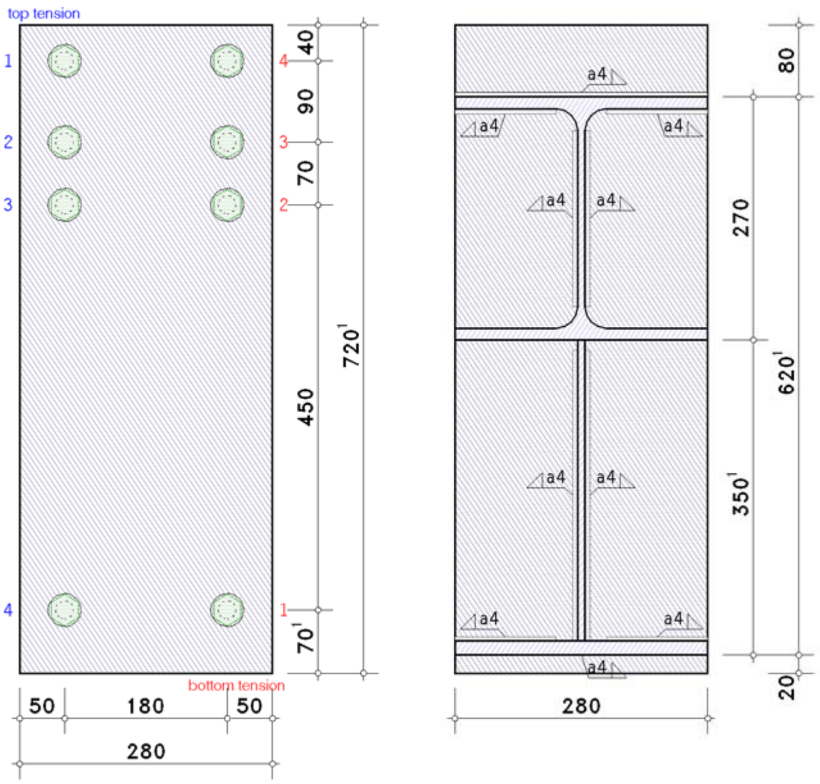
1. input report



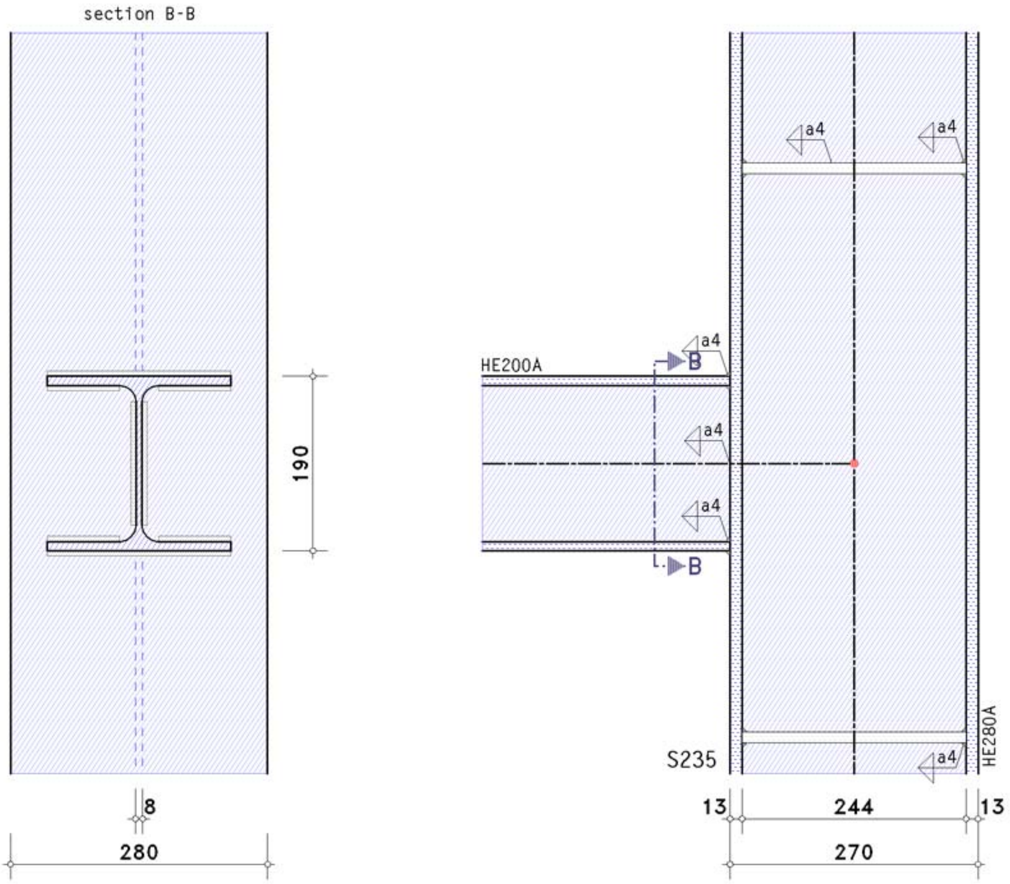
connection right



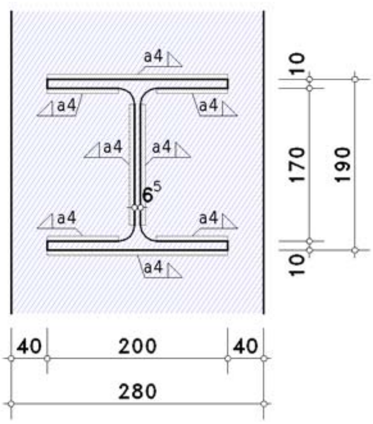
details (section A - A)



connection left

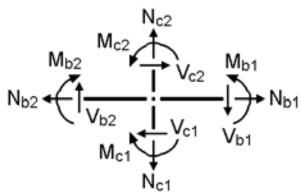


details (section B - B)



According to EC 3-1-8, 5.3 in a double-sided beam-columnn-joint each joint is modelled separately. displacement of top edges of beams (right-left) amounts 233.0 mm.

steel grade
steel grade S235
column parameters
section HE280A
reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 619.7 mm):
thickness t_{st} = 12.0 mm, width b_{st} = 136.0 mm, length l_{st} = 244.0 mm
recess at stiffeners c_{st} = 36.0 mm
welds $a_{st,f}$ = 4.0 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 HE280A
slope angle of haunch about the horizontal axis α_v = 35.00°
haunch length L_v = 500.0 mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b))$ = 350.1 mm
web thickness $t_{w,v}$ = 8.0 mm, flange width, thickness $b_{f,v}$ = 280.0 mm, $t_{f,v}$ = 13.0 mm
total beam depth at the connection point $h_{ges} = h_b + h_v$ = 620.1 mm
verification parameters
bolted end-plate connection:
thickness t_p = 20.0 mm, width b_p = 280.0 mm, length l_p = 720.1 mm
projections $h_{p,o}$ = 80.0 mm, $h_{p,u}$ = 20.0 mm
bolts in connection:
4 bolt-rows with 2 bolts
all bolt-rows considered individually
all bolt-rows for shear transfer (rows 1-4)
bolt groups generated automatically, considering all groups bzgl. 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 = 40.0 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 70.1 mm
centre distance of the bolt-rows from each other p_{1-2} = 90.0 mm, p_{2-3} = 70.0 mm, p_{3-4} = 450.0 mm
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, angle φ = 125°
double-sided beam-column joint, left
beam parameters
section HE200A
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: $N_{j,b1,Ed}$ = 7.25 kN $M_{j,b1,Ed}$ = -8.15 kNm $V_{j,b1,Ed}$ = 12.74 kN (right)
$N_{j,b2,Ed}$ = -5.46 kN $M_{j,b2,Ed}$ = -14.47 kNm $V_{j,b2,Ed}$ = -14.53 kN (left)
$N_{j,c1,Ed}$ = -55.15 kN $M_{j,c1,Ed}$ = 7.33 kNm $V_{j,c1,Ed}$ = 11.43 kN (bottom)
$N_{j,c2,Ed}$ = -26.99 kN $M_{j,c2,Ed}$ = 1.75 kNm $V_{j,c2,Ed}$ = -1.27 kN (top)
Lk 2: $N_{j,b1,Ed}$ = 8.91 kN $M_{j,b1,Ed}$ = -18.74 kNm $V_{j,b1,Ed}$ = 20.03 kN (right)
$N_{j,b2,Ed}$ = 2.61 kN $M_{j,b2,Ed}$ = -7.21 kNm $V_{j,b2,Ed}$ = -11.86 kN (left)
$N_{j,c1,Ed}$ = -67.66 kN $M_{j,c1,Ed}$ = -3.47 kNm $V_{j,c1,Ed}$ = -3.10 kN (bottom)
$N_{j,c2,Ed}$ = -34.56 kN $M_{j,c2,Ed}$ = 7.67 kNm $V_{j,c2,Ed}$ = -9.41 kN (top)

Lk 3: $N_{j,b1,Ed}$ = 9.56 kN $M_{j,b1,Ed}$ = -13.00 kNm $V_{j,b1,Ed}$ = 17.93 kN (right)
$N_{j,b2,Ed}$ = -4.78 kN $M_{j,b2,Ed}$ = -16.34 kNm $V_{j,b2,Ed}$ = -17.60 kN (left)
$N_{j,c1,Ed}$ = -72.69 kN $M_{j,c1,Ed}$ = 6.43 kNm $V_{j,c1,Ed}$ = 10.63 kN (bottom)
$N_{j,c2,Ed}$ = -35.95 kN $M_{j,c2,Ed}$ = 3.74 kNm $V_{j,c2,Ed}$ = -3.71 kN (top)
Lk 4: $N_{j,b1,Ed}$ = 6.60 kN $M_{j,b1,Ed}$ = -13.88 kNm $V_{j,b1,Ed}$ = 14.84 kN (right)
$N_{j,b2,Ed}$ = 1.93 kN $M_{j,b2,Ed}$ = -5.34 kNm $V_{j,b2,Ed}$ = -8.78 kN (left)
$N_{j,c1,Ed}$ = -50.12 kN $M_{j,c1,Ed}$ = -2.57 kNm $V_{j,c1,Ed}$ = -2.30 kN (bottom)
$N_{j,c2,Ed}$ = -25.60 kN $M_{j,c2,Ed}$ = 5.68 kNm $V_{j,c2,Ed}$ = -6.97 kN (top)

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

check of data
connection right:
ok
connection left:
ok
bolts right:
distances between bolt-rows at end-plate

horizontal:	e_2 = 50.0 mm > 1.2·d_0 = 26.4 mm,	e_2 = 50.0 mm < 4·t + 40 mm = 92.0 mm
horizontal:	p_2 = 180.0 mm > 2.4·d_0 = 52.8 mm,	p_2 = 180.0 mm < min(14·t, 200 mm) = 182.0 mm
vertical:	e_1 = 40.0 mm > 1.2·d_0 = 26.4 mm,	e_1 = 40.0 mm < 4·t + 40 mm = 92.0 mm
vertical:	p_1 = 90.0 mm > 2.2·d_0 = 48.4 mm,	p_1 = 90.0 mm < min(14·t, 200 mm) = 182.0 mm
vertical:	p_1 = 70.0 mm > 2.2·d_0 = 48.4 mm,	p_1 = 70.0 mm < min(14·t, 200 mm) = 182.0 mm
vertical:	p_1 = 450.0 mm > 2.2·d_0 = 48.4 mm,	p_1 = 450.0 mm > min(14·t, 200 mm) = 182.0 mm
vertical:	e_1 = 70.1 mm > 1.2·d_0 = 26.4 mm,	e_1 = 70.1 mm < 4·t + 40 mm = 92.0 mm

horizontal distance of bolts from column edge

vertical:	e_1 = 50.0 mm > 1.2·d_0 = 26.4 mm,	e_1 = 50.0 mm < 4·t + 40 mm = 92.0 mm

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

utilizations of each joint (right)

Lk	U_m	U_v	U_{ep}	U_{sb}	U_{ss}	U_{sw}	U
--	---	---	---	---	---	---	---
1	0.037	0.008	0.011	0.022	0.031	0.578	0.578
2	0.079	0.014	0.018	0.047	0.067	0.414	0.414
3	0.058	0.012	0.016	0.034	0.049	0.685	0.685
4	0.059	0.010	0.013	0.035	0.050	0.307	0.307

U_m: utilization due to bending; U_v: utilization due to shear/bearing resistance; U_{pl}: utilization due to shear in end-plate
U_{sb}: utilization due to weld; U_{ss}: utilization due to stiffeners/ribs; U_{sw}: utilization due to shearfeld
U: utilization of each joint; U: utilization of each joint

utilizations of each joint (left)

Lk	U_m	U_{sb}	U_{ss}	U_{sw}	U
--	---	---	---	---	---
1	0.199	0.191	0.140	0.582	0.582
2	0.133	0.086	0.063	0.418	0.418
3	0.224	0.212	0.155	0.691	0.691
4	0.098	0.064	0.047	0.310	0.310

U_m: utilization due to bending; U_{sb}: utilization due to weld; U_{ss}: utilization due to stiffeners/ribs
U_{sw}: utilization due to shearfeld; U: utilization of each joint; U: utilization of each joint

2. final result

utilization of the connection

		Gleichgewicht			
Lk	U_j	ΣH	ΣV	ΣM	
--		kN	kN	kNm	
1	0.582	3.49	3.67	0.75	!!
2	0.418	5.61	4.82	0.39	!!
3	0.691*	4.95	4.92	0.65	!!
4	0.310	4.16	3.57	0.29	!!

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

maximum utilization [Lk 3]: max U = 0.691 < 1 ok

verification succeeded

3. 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

4. Detailed edition of Lk 3 (decisive)

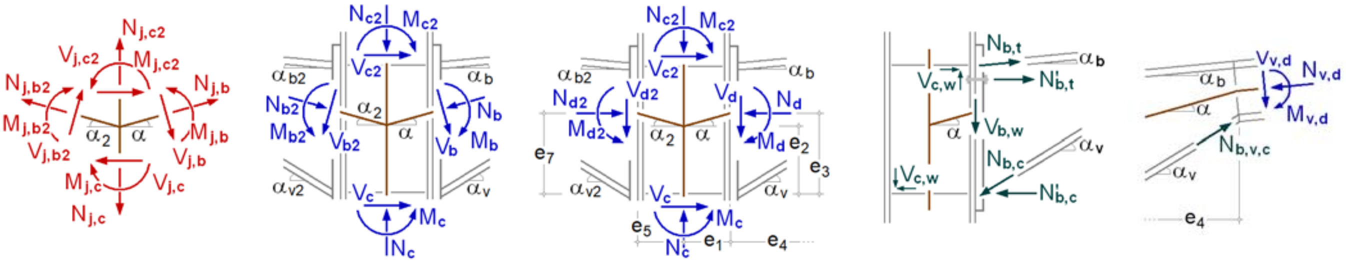
notes
no verification for cross-sections.

4.1. 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.
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. Only at calcualation of end plate the lower beam flange is respected as a stiffener.
no verfication for welds of welded section.

4.1.1. design values

periphery connection⊥ zur connection plane   partial internal forces and moments / haunch beam



slope angle: $\alpha_b = 0.00°$, $\alpha_v = 35.00°$ $\Rightarrow$ $\alpha = (\alpha_b+\alpha_v)/2 = 17.50°$, $\Delta\alpha = \alpha-\alpha_b = 17.50°$
distance: $e_1 = 135.0$ mm, $e_3 = 340.6$ mm, $e_2 = 298.1$ mm, $e_5 = 135.0$ mm, $e_7 = 298.2$ mm, $e_4 = 516.1$ mm

internal forces and moments perpendicular to the connection planes
periphery beam (right)
$N_d = -14.51$ kN, $M_d = 10.47$ kNm, $V_d = 14.23$ kN
periphery haunch-beam
$N_{v,d} = -14.51$ kN, $M_{v,d} = 0.76$ kNm, $V_{v,d} = 14.23$ kN
periphery beam (left)
$N_{d2} = 4.78$ kN, $M_{d2} = 13.85$ kNm, $V_{d2} = -17.60$ kN
periphery column (bottom)
$N_c = 72.69$ kN, $M_c = -3.26$ kNm, $V_c = 10.63$ kN
periphery column (top)
$N_{c2} = 35.95$ kN, $M_{c2} = -2.55$ kNm, $V_{c2} = -3.71$ kN
partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d \cdot t_{ep} \cdot \tan(\alpha) - V_d \cdot t_{ep} = 10.09$ kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b = 23.48$ kN, $z_b = 605.7$ mm, $z_{bu} = 284.8$ mm
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_v) = 10.96$ kN, $z_b = 605.7$ mm, $z_{bo} = 320.8$ mm

basic component 1 is not calculated !!

4.1.2. connection capacity

4.1.2.1. moment resistance

distance of tension-bolt-rows from centre of compression:
$h_1 = 652.2$ mm, $h_2 = 562.2$ mm, $h_3 = 492.2$ mm, $h_4 = 42.2$ mm

resistance per bolt-row
row 1: $F_{tr,Rd} = 217.6$ kN
row 2: $F_{tr,Rd} = 195.2$ kN
row 3: $F_{tr,Rd} = 0.0$ kN
row 4: $F_{tr,Rd} = 0.0$ kN
$\Sigma F_{tr,Rd} = 412.8$ kN
potential failure by basic component 4, 20

resistance of flanges
$\Sigma F_{c,Rd}^* = 1130.6$ kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 251.6$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 749.9$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 1130.6$ kN

4.1.2.2. shear/bearing resistance

resistance per bolt-row
row 1: $F_{vr,Rd} = 168.7$ kN
row 2: $F_{vr,Rd} = 182.4$ kN
row 3: $F_{vr,Rd} = 301.6$ kN
row 4: $F_{vr,Rd} = 301.6$ kN
$\Sigma F_{vr,Rd} = 954.3$ kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd} = 954.3$ kN

4.1.2.3. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd} = 1474.09$ kN
welds: $F_{w,Rd} = 903.27$ kN
shear resistance of end plate: $V_{ep,Rd} = F_{w,Rd} = 903.27$ kN

4.1.2.4. total

$M_{j,Rd} = 251.6$ kNm $N_{j,t,Rd} = 749.9$ kN $N_{j,c,Rd} = 1130.6$ kN $V_{j,Rd} = 903.3$ kN $V_{ep,Rd} = 903.3$ kN

4.1.3. verifications

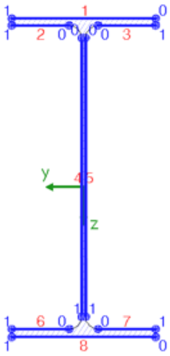
4.1.3.1. verification of the connection capacity by means of the component method

axial force: $N_{b,Ed} = |N_d \cdot \cos(\alpha) + V_d \cdot \sin(\alpha)| = 9.56$ kN $< 5\% \cdot N_{pl,Rd} = 156.37$ kN $\Rightarrow$ moment resistance
internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu} = 14.51$ kNm, $z_{bu} = 278.5$ mm
shear force: $V_{Ed} = |V_d| = 14.23$ kN

$M_{Ed}/M_{j,Rd} = 0.058 < 1$ ok
$V_{Ed}/V_{j,Rd} = 0.016 < 1$ ok
$V_{Ed}/V_{ep,Rd} = 0.016 < 1$ ok

4.1.3.2. 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 = 4.0 mm	l_w = 280.0 mm
weld 2:	a_w = 4.0 mm	l_w = 112.0 mm
weld 3:	siehe weld 2	
weld 4:	a_w = 4.0 mm	l_w = 543.2 mm
weld 5:	siehe weld 4	
weld 6:	a_w = 4.0 mm	l_w = 112.0 mm
weld 7:	siehe weld 6	
weld 8:	a_w = 4.0 mm	l_w = 280.0 mm

design values refering to centroid of the section:

N_{Ed} = 14.51 kN, $M_{y,Ed}$ = -10.47 kNm, $V_{z,Ed}$ = 14.23 kN

cross-sectional properties refering to centroid of the line cross-section:

ΣA_w = 83.78 cm², $A_{w,z}$ = 43.46 cm², Σl_w = 209.4 cm

$I_{w,y}$ = 47881.43 cm⁴, $I_{w,z}$ = 2922.18 cm⁴, $W_{w,t}$ = 106.46 cm³, Δz_w = -18.3 mm

verifications in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x}$ = 8.49 N/mm²		$\Rightarrow U_w = 0.033 < 1$ ok
weld 2,	pt. 0:	$\sigma_{w,x}$ = 8.20 N/mm²		$\Rightarrow U_w = 0.032 < 1$ ok
weld 4,	pt. 0:	$\sigma_{w,x}$ = 7.68 N/mm²	$\tau_{w,z}$ = 3.27 N/mm²	$\Rightarrow U_w = 0.034 < 1$ ok
	pt. 1:	$\sigma_{w,x}$ = -4.20 N/mm²	$\tau_{w,z}$ = 3.27 N/mm²	$\Rightarrow U_w = 0.023 < 1$ ok
weld 6,	pt. 0:	$\sigma_{w,x}$ = -4.72 N/mm²		$\Rightarrow U_w = 0.019 < 1$ ok
weld 8,	pt. 0:	$\sigma_{w,x}$ = -5.07 N/mm²		$\Rightarrow U_w = 0.020 < 1$ ok

Result:

weld 4, pt. 0: $\sigma_{w,x}$ = 7.68 N/mm² $\tau_{w,z}$ = 3.27 N/mm²

Max: $\sigma_{1,w,Ed}$ = 1.22 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm²,

$\sigma_{2,w,Ed}$ = 0.54 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² $\Rightarrow U_w = 0.034 < 1$ ok

4.1.3.3. verification of web stiffeners

compression stiffener

$F_{c,Ed}$ = 10.85 kN

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 4.3 kN, $H = F \cdot e_F/e_H$ = 1.5 kN

assumption: stiffeners do not buckle: $c/t = 11.3 \cdot \varepsilon \le 33 \cdot \varepsilon \Rightarrow$ section class $1 \le 2$ ok

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0}$ = 282.00 kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 5.1 kN

F_{Ed} = 5.1 kN < F_{Rd} = 282.0 kN $\Rightarrow U = 0.018 < 1$ ok

cross-section at web

shear resistance V_{Rd} = 397.26 kN

design value: $F_{Ed} = F$ = 4.3 kN

F_{Ed} = 4.3 kN < F_{Rd} = 397.3 kN $\Rightarrow U = 0.011 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 0.22 kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 0.08 kN/cm, b_1 = 100.0 mm

$\sigma_{1,w,Ed}$ = 0.64 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² $\Rightarrow U = 0.018 < 1$ ok

$\sigma_{2,w,Ed}$ = 0.54 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² $\Rightarrow U = 0.021 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 0.13 kN/cm, l_1 = 172.0 mm

$\sigma_{1,w,Ed}$ = 0.55 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² $\Rightarrow U = 0.015 < 1$ ok

stiffener in tension

$F_{t,Ed}$ = 25.36 kN

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 10.1 kN, $H = F \cdot e_F/e_H$ = 3.6 kN

cross-section at flange

tension resistance $N_{t,Rd}$ = 282.00 kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 11.9 kN

F_{Ed} = 11.9 kN < F_{Rd} = 282.0 kN $\Rightarrow U = 0.042 < 1$ ok

cross-section at web

shear resistance V_{Rd} = 397.26 kN

design value: $F_{Ed} = F$ = 10.1 kN

F_{Ed} = 10.1 kN < F_{Rd} = 397.3 kN $\Rightarrow U = 0.026 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 0.51 kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 0.18 kN/cm, b_1 = 100.0 mm

$\sigma_{1,w,Ed}$ = 1.49 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² $\Rightarrow U = 0.041 < 1$ ok

$\sigma_{2,w,Ed}$ = 1.27 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² $\Rightarrow U = 0.049 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 0.29 kN/cm, l_1 = 172.0 mm

$\sigma_{1,w,Ed}$ = 1.28 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² $\Rightarrow U = 0.035 < 1$ ok

4.1.3.4. elastic verification of the shear area

column web
requirements concerning stiffeners: s. verification of web stiffeners
requirements concerning shear area: shear buckling: $h_p/t_p = 30.50 \le 72/(\eta \cdot \varepsilon) = 60.00$ ok
internal forces and moments at web (sign definition of statics):
$N_1 = -4.78$ kN, $M_1 = -13.96$ kNm, $V_1 = -17.60$ kN
$N_3 = -72.69$ kN, $M_3 = 3.26$ kNm, $V_3 = 10.63$ kN
$N_4 = 14.51$ kN, $M_4 = -10.59$ kNm, $V_4 = 14.23$ kN
$N_2 = -35.95$ kN, $M_2 = 2.55$ kNm, $V_2 = -3.71$ kN
dimensions of the shear area: $h_b = 305.8$ mm, $h_t = 328.4$ mm, $h_l = 164.4$ mm, $h_r = 590.1$ mm
stresses within the shear area:
$\tau_b = 30.3$ N/mm², $\tau_t = 27.7$ N/mm², $\tau_l = 93.0$ N/mm², $\tau_r = 7.1$ N/mm²
verification of the shear area:
max $\tau_{Ed} = 93.0$ N/mm² $< \tau_{Rd} = 135.7$ N/mm² $\Rightarrow$ $U = 0.685 < 1$ ok

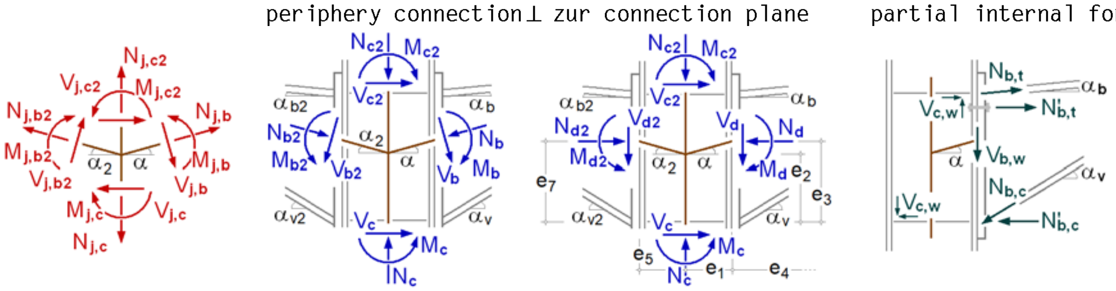
4.1.3.5. verification result

maximum utilization: max $U = 0.685 < 1$ ok

4.2. connection left

4.2.1. design values

periphery connection ⊥ zur connection plane   partial internal forces and moments



slope angle: $\alpha_b = \alpha_v = \alpha = 0°$
$\alpha_{b2} = 0.00°$, $\alpha_{v2} = 35.00°$ $\Rightarrow$ $\alpha_2 = (\alpha_{b2}+\alpha_{v2})/2 = 17.50°$
distance: $e_1 = 135.0$ mm, $e_3 = 298.2$ mm, $e_2 = 298.2$ mm, $e_5 = 135.0$ mm, $e_7 = 340.6$ mm

internal forces and moments perpendicular to the connection planes
periphery beam (right)
$N_d = 4.78$ kN, $M_d = 13.96$ kNm, $V_d = 17.60$ kN
periphery beam (left)
$N_{d2} = -14.51$ kN, $M_{d2} = 10.58$ kNm, $V_{d2} = -14.23$ kN
periphery column (bottom)
$N_c = 72.69$ kN, $M_c = 3.26$ kNm, $V_c = -10.63$ kN
periphery column (top)
$N_{c2} = 35.95$ kN, $M_{c2} = 2.55$ kNm, $V_{c2} = 3.71$ kN
partial internal forces and moments
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M_d/z_b = 75.19$ kN, $z_b = 180.0$ mm, $z_{bu} = 90.0$ mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M_d/z_b = 79.97$ kN, $z_b = 180.0$ mm, $z_{bo} = 90.0$ mm

basic component 1 is not calculated !!

4.2.2. connection capacity

4.2.2.1. moment resistance

distance between tension force and centre of compression: $z = 180.0$ mm

resistance
$F_{Rd} = 335.1$ kN

resistance of flanges
$\Sigma F_{c,Rd}^* = 670.2$ kN

moment resistance
$M_{j,Rd} = F_{Rd} \cdot z = 60.3$ kNm
tension resistance
$N_{j,t,Rd} = F_{t,Rd} = 387.9$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 670.2$ kN

4.2.3. verifications

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

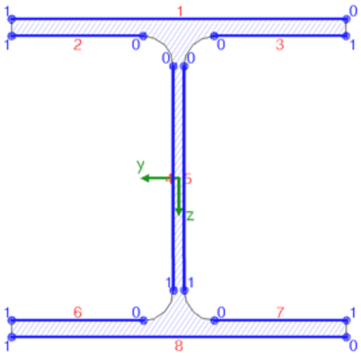
axial force: $N_{b,Ed} = |N_d| = 4.78$ kN $<$ 5%$\cdot N_{pl,Rd} = 63.25$ kN $\Rightarrow$ moment resistance
internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu} = 13.53$ kNm, $z_{bu} = 90.0$ mm
shear force: $V_{Ed} = |V_d| = 17.60$ kN

$M_{Ed}/M_{j,Rd} = 0.224 < 1$ ok

4.2.3.2. 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: $a_w = 4.0$ mm $l_w = 200.0$ mm
weld 2: $a_w = 4.0$ mm $l_w = 78.8$ mm
weld 3: siehe weld 2
weld 4: $a_w = 4.0$ mm $l_w = 134.0$ mm
weld 5: siehe weld 4
weld 6: $a_w = 4.0$ mm $l_w = 78.8$ mm
weld 7: siehe weld 6
weld 8: $a_w = 4.0$ mm $l_w = 200.0$ mm

design values refering to centroid of the section:
$N_{Ed} = -4.78$ kN, $M_{y,Ed} = -13.96$ kNm, $V_{z,Ed} = 17.60$ kN
cross-sectional properties refering to centroid of the line cross-section:
$\Sigma A_w = 39.32$ cm², $A_{w,z} = 10.72$ cm², $\Sigma l_w = 98.3$ cm
$I_{w,y} = 2514.76$ cm⁴, $I_{w,z} = 1062.68$ cm⁴, $W_{w,t} = 43.06$ cm³, $\Delta z_w = 0.0$ mm
verifications in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x} = 51.54$ N/mm²		$\Rightarrow U_w = 0.202 < 1$ ok
weld 2,	pt. 0:	$\sigma_{w,x} = 45.99$ N/mm²		$\Rightarrow U_w = 0.181 < 1$ ok
weld 4,	pt. 0:	$\sigma_{w,x} = 35.99$ N/mm²	$\tau_{w,z} = 16.42$ N/mm²	$\Rightarrow U_w = 0.162 < 1$ ok
	pt. 1:	$\sigma_{w,x} = -38.42$ N/mm²	$\tau_{w,z} = 16.42$ N/mm²	$\Rightarrow U_w = 0.170 < 1$ ok
weld 6,	pt. 0:	$\sigma_{w,x} = -48.42$ N/mm²		$\Rightarrow U_w = 0.190 < 1$ ok
weld 8,	pt. 0:	$\sigma_{w,x} = -53.97$ N/mm²		$\Rightarrow U_w = 0.212 < 1$ ok

Result:
weld 8, pt. 0: $\sigma_{w,x} = -53.97$ N/mm²
Max: $\sigma_{1,w,Ed} = 7.63$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm²,
$\sigma_{2,w,Ed} = 3.82$ kN/cm² $< f_{2w,d} = 25.92$ kN/cm² $\Rightarrow U_w = 0.212 < 1$ ok

4.2.3.3. verification of web stiffeners

compression stiffener
$F_{c,Ed} = 80.61$ kN
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 32.2$ kN, $H = F \cdot e_F/e_H = 11.4$ kN
assumption: stiffeners do not buckle: $c/t = 11.3 \cdot \varepsilon \leq 33 \cdot \varepsilon \Rightarrow$ section class $1 \leq 2$ ok
cross-section at flange
compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 282.00$ kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 37.8$ kN
$F_{Ed} = 37.8$ kN $< F_{Rd} = 282.0$ kN $\Rightarrow U = 0.134 < 1$ ok
cross-section at web
shear resistance $V_{Rd} = 397.26$ kN
design value: $F_{Ed} = F = 32.2$ kN
$F_{Ed} = 32.2$ kN $< F_{Rd} = 397.3$ kN $\Rightarrow U = 0.081 < 1$ ok
flange welds
design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 1.61$ kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 0.57$ kN/cm, $b_1 = 100.0$ mm
$\sigma_{1,w,Ed} = 4.72$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm² $\Rightarrow U = 0.131 < 1$ ok
$\sigma_{2,w,Ed} = 4.03$ kN/cm² $< f_{2w,d} = 25.92$ kN/cm² $\Rightarrow U = 0.155 < 1$ ok
web welds
design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 0.94$ kN/cm, $l_1 = 172.0$ mm
$\sigma_{1,w,Ed} = 4.06$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm² $\Rightarrow U = 0.113 < 1$ ok
stiffener in tension
$F_{t,Ed} = 75.83$ kN
forces per rib
$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 30.3$ kN, $H = F \cdot e_F/e_H = 10.7$ kN
cross-section at flange

tension resistance $N_{t,Rd}$ = 282.00 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 35.5 kN
F_{Ed} = 35.5 kN < F_{Rd} = 282.0 kN ⇒ U = 0.126 < 1 ok
cross-section at web
shear resistance V_{Rd} = 397.26 kN
design value: F_{Ed} = F = 30.3 kN
F_{Ed} = 30.3 kN < F_{Rd} = 397.3 kN ⇒ U = 0.076 < 1 ok
flange welds
design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 1.52 kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 0.53 kN/cm, b_1 = 100.0 mm
$\sigma_{1,w,Ed}$ = 4.44 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² ⇒ U = 0.123 < 1 ok
$\sigma_{2,w,Ed}$ = 3.79 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² ⇒ U = 0.146 < 1 ok
web welds
design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 0.88 kN/cm, l_1 = 172.0 mm
$\sigma_{1,w,Ed}$ = 3.82 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² ⇒ U = 0.106 < 1 ok

4.2.3.4. elastic verification of the shear area

column web
requirements concerning stiffeners: s. verification of web stiffeners
requirements concerning shear area: shear buckling: $h_p/t_p = 30.50 \leq 72/(\eta \cdot \varepsilon) = 60.00$ ok
internal forces and moments at web (sign definition of statics):
N_1 = 14.51 kN, M_1 = -10.71 kNm, V_1 = -14.23 kN
N_3 = -72.69 kN, M_3 = -3.26 kNm, V_3 = -10.63 kN
N_4 = -4.78 kN, M_4 = -14.08 kNm, V_4 = 17.60 kN
N_2 = -35.95 kN, M_2 = -2.55 kNm, V_2 = 3.71 kN
dimensions of the shear area: h_b = 304.1 mm, h_t = 330.3 mm, h_l = 590.1 mm, h_r = 164.4 mm
stresses within the shear area:
τ_b = 30.5 N/mm², τ_t = 28.0 N/mm², τ_l = 7.2 N/mm², τ_r = 93.7 N/mm²
verification of the shear area:
max τ_{Ed} = 93.7 N/mm² < τ_{Rd} = 135.7 N/mm² ⇒ U = 0.691 < 1 ok

4.2.3.5. verification result

maximum utilization: max U = 0.691 < 1 ok