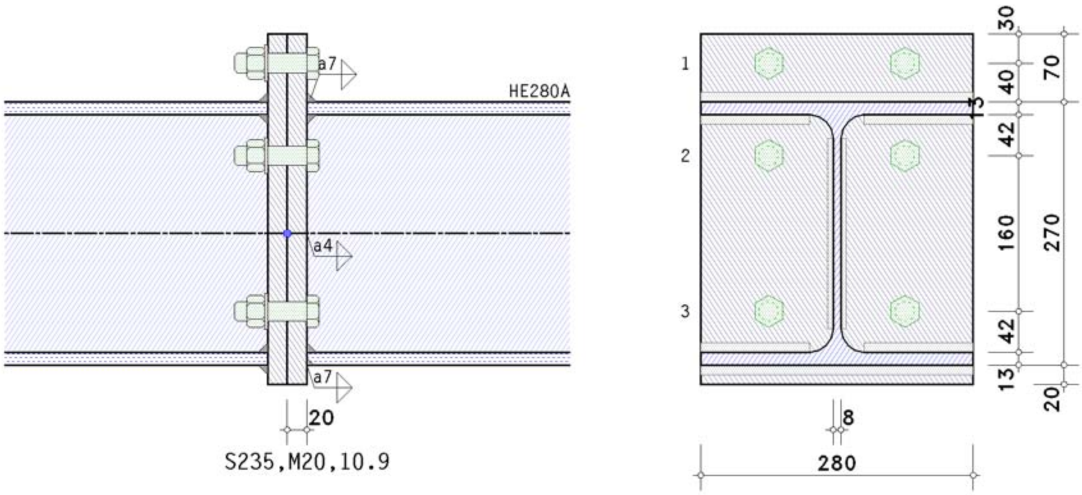
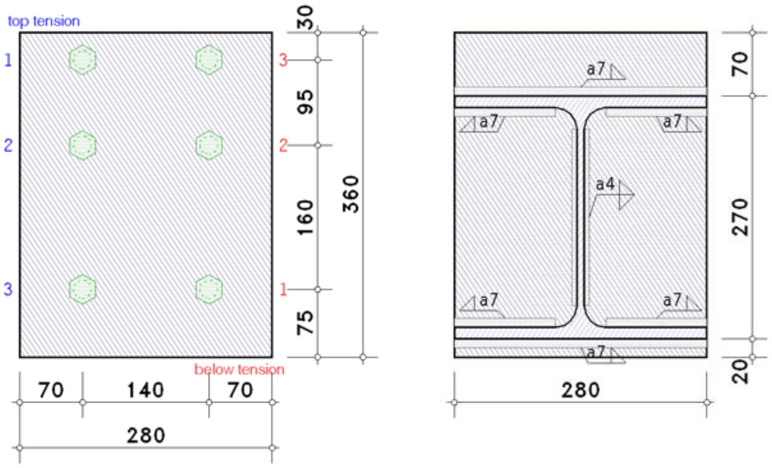
POS. 7: WAGENKNECHT BD.2 6.8.2

Rigid beam splice EC 3-1-8 (04.25), NA: Deutschland 4H-EC3BT version: 10/2019-2w

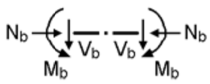
1. input report



details (section A - A)



steel grade
steel grade S235
bolts
bolt class 10.9, bolt size M20, normal wrench size
shear plane passes through the unthreaded portion of the bolt
beam parameters
section HE280A
verification parameters
<u>bolted end-plate connection</u>
thickness t_p = 20.0 mm, width b_p = 280.0 mm, length l_p = 360.0 mm
projections $h_{p,o}$ = 70.0 mm, $h_{p,u}$ = 20.0 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 = 70.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 30.0 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 75.0 mm
centre distance of the bolt-rows from each other p_{1-2} = 95.0 mm, p_{2-3} = 160.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 7.0 mm
beam web: fillet weld, weld thickness a = 4.0 mm
beam flange below: fillet weld, weld thickness a = 7.0 mm



internal forces and moments at the joint periphery perpendicular to the connection plane

Lc 1: M_d = 145.00 kNm V_d = 120.00 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
no verification for cross-sections.
welds are not checked.
calculation of T-stub-resistance with the standard method.

check of data

ok

distances between bolts at end-plate

horizontal:	e_2 = 70.0 mm > 1.2·d_0 = 26.4 mm,	e_2 = 70.0 mm < 4·t + 40 mm = 120.0 mm
horizontal:	p_2 = 140.0 mm > 2.4·d_0 = 52.8 mm,	p_2 = 140.0 mm < min(14·t, 200 mm) = 200.0 mm
top-below:	e_1 = 30.0 mm > 1.2·d_0 = 26.4 mm,	e_1 = 30.0 mm < 4·t + 40 mm = 120.0 mm
top-below:	p_1 = 95.0 mm > 2.2·d_0 = 48.4 mm,	p_1 = 95.0 mm < min(14·t, 200 mm) = 200.0 mm
top-below:	p_1 = 160.0 mm > 2.2·d_0 = 48.4 mm,	p_1 = 160.0 mm < min(14·t, 200 mm) = 200.0 mm
top-below:	e_1 = 75.0 mm > 1.2·d_0 = 26.4 mm,	e_1 = 75.0 mm < 4·t + 40 mm = 120.0 mm

2. table of results

utilization

Lc	U_m	U_v	U_{sb}	U
--	---	---	---	---
1	0.940	0.398	0.624	0.940*

U_m: utilization due to bending; U_V: utilization due to shear/bearing resistance; U_{sb}: utilization due to weld
U: utilization of the connection
*) maximum utilization

3. final result

maximum utilization: max U = 0.940 < 1 ok

verification succeeded

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

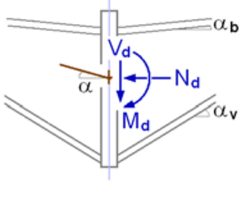
EN 1993-1-1, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau;
Deutsche Fassung EN 1993-1-1:2022, Ausgabe April 2025
EN 1993-1-1/NA, Nationaler appendix zur EN 1993-1-1, Ausgabe Oktober 2022

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 appendix zur EN 1993-1-8, Ausgabe November 2020

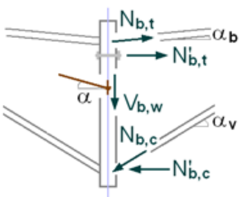
5. Lc 1 (decisive)

5.1. design values

⊥ to connection plane



partial internal forces and moments



slope angle: $\alpha_b = \alpha = \alpha_v = 0°$

internal forces and moments perpendicular to the connection planes

periphery beam

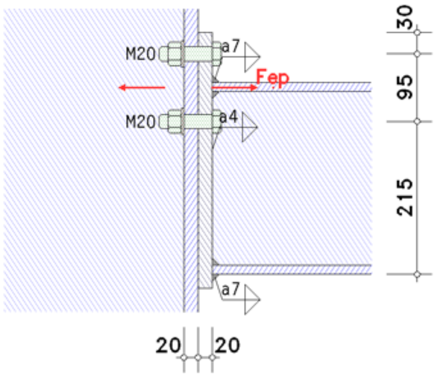
$M_d = 145.00$ kNm, $V_d = 120.00$ kN

partial internal forces and moments

internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_p = 142.60$ kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b = 554.86$ kN, $z_b = 257.0$ mm, $z_{bu} = 128.5$ mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b = 554.86$ kN, $z_b = 257.0$ mm, $z_{bo} = 128.5$ mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = 0.00$ kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v) = 0.00$ kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 120.00$ kN

5.2. basic components

5.2.1. bc 5: end-plate in bending



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

extended part of end-plate

in the extended part of the end-plate only one bolt-row is considered ($n_b = 1$).

effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 140.0$ mm, $l_{eff,cp} = 201.6$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 140.0$ mm

tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 3.29$ 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
$L_b = 58.8$ mm ≤ 63.5 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 410.22$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 276.34$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.51$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 276.34$ 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: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 498.93$ kN (≥ 276.34 kN, not decisive)

resistance and effective length of end-plate in bending (projection)
$F_{t,ep,Rd,1} = 276.34$ kN, $l_{eff,1} = 140.0$ mm

part of end-plate between beam flanges

equivalent T-stub flange (each individual bolt-row):

here: number of bolt-rows $n_b = 1$

row 2

effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 386.3$ mm, $l_{eff,cp} = 386.3$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 419.1$ mm

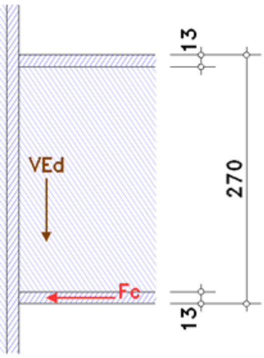
tension resistance of the T-stub flange:
in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 9.08$ kNm
in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 9.85$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.26$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.51$ kN
$L_b = 58.8$ mm ≤ 162.0 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 590.62$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 337.51$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.51$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 337.51$ 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: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 786.60$ kN (≥ 337.51 kN, not decisive)

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

$F_{ep,Rd,2} = 337.51$ kN, $l_{eff,2} = 386.3$ mm

5.2.2. bc 7: beam flange and web in compression

flange below: section class 1
web: section class 1
total: section class 1
section class of the beam: 1
taking into account the moment-shear force-interaction V_{Ed} = 120.0 kN

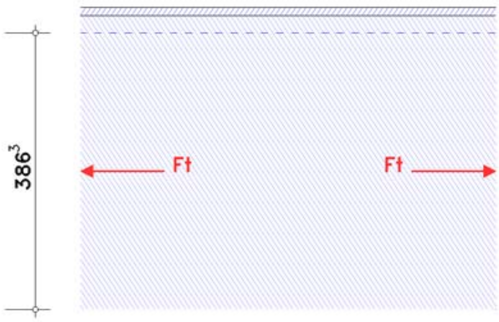


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

stress due to bending with shear force: $V_{Ed} = 120.0\ \text{kN} \leq 215.3\ \text{kN} = 0.5 \cdot V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0}$ = 261.35 kNm, W_{pl} = 1112.13 cm^3
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f)$ = 1016.93 kN

resistance of the upper beam flange:
stress due to bending with shear force: $V_{Ed} = 120.0\ \text{kN} \leq 215.3\ \text{kN} = 0.5 \cdot V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0}$ = 261.35 kNm, W_{pl} = 1112.13 cm^3
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f)$ = 1016.93 kN

5.2.3. bc 8: beam web in tension



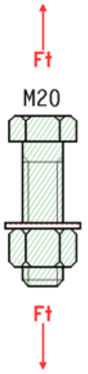
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

each individual bolt-row:
row 2
effective width $b_{eff,t,wb}$ = 386.3 mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 726.16 kN

resistance of a beam web in tension (per bolt-row)
$F_{t,wb,Rd,2}$ = 726.16 kN, $b_{eff,t,wb}$ = 386.3 mm (s. bc 5)

5.2.4. bc 10: bolts in tension

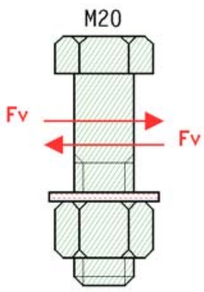
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



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 $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 341.73$ kN, $t_p = 20.0$ 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

5.2.5. bc 11: bolts in shear

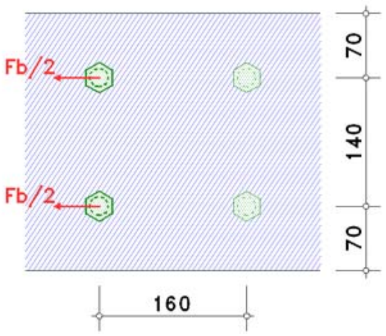
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



shear resistance $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2} = 150.80$ kN, $\alpha_v = 0.60$
shear resistance of 2 bolts (1-shear): $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd} = 301.59$ kN

5.2.6. bc 12: plate with bearing resistance

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



row 3
  bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 345.60$ kN, $k_m = 1.00$, $\alpha_b = 3.00$
  bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 345.60$ kN, $k_m = 1.00$, $\alpha_b = 3.00$
  bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 691.20$ kN

5.3. connection capacity

5.3.1. moment resistance

distance of tension-bolt-rows from centre of compression: $h_1 = 303.5$ mm, $h_2 = 208.5$ mm

resistances acc. to EC 3-1-8, B.3.2.2(6) for bolt-rows considered individually
decisive basic components: 5, 8
row 1: $F_{tr,Rd} = 276.3$ kN
row 2: $F_{tr,Rd} = 337.5$ kN

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 276.3$ kN
row 2: $F_{tr,Rd} = 337.5$ kN
$\Sigma F_{tr,Rd}^{*} = 613.9$ kN

deductions acc. to EC 3-1-8, B.3.2.2(7)
decisive basic component: 7

row 1: $F_{tr,Rd}$ = 276.3 kN
row 2: $F_{tr,Rd}$ = 337.5 kN

check acc. to EC 3-1-8, B.3.2.2(9)
decisive basic component: 10
row 1: $F_{tr,Rd}$ = 276.3 kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd}$ = 276.3 kN
row 2: $F_{tr,Rd}$ = 337.5 kN
$\Sigma F_{tr,Rd}$ = 613.9 kN
potential failure by basic component 5

resistance of flanges (compression)
$\Sigma F_{c,Rd}^{*}$ = 2033.9 kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r)$ = 154.2 kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^{*}$ = 613.9 kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^{*}$ = 2033.9 kN

5.3.2. shear/bearing resistance

resistance per bolt-row
decisive basic components: 11, 12
row 3: $F_{vr,Rd}$ = 301.6 kN

deductions depending on tension force (at full utilization of moment resistance)
decisive basic component: 10
row 3: $F_{vr,Rd}$ = 301.6 kN

resistance per bolt-row (shear/bearing resistance)
row 3: $F_{vr,Rd}$ = 301.6 kN
$\Sigma F_{vr,Rd}$ = 301.6 kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd}$ = 301.6 kN

5.3.3. total

$M_{j,Rd}$ = 154.2 kNm $N_{j,t,Rd}$ = 613.9 kN $N_{j,c,Rd}$ = 2033.9 kN $V_{j,Rd}$ = 301.6 kN

5.4. verifications

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

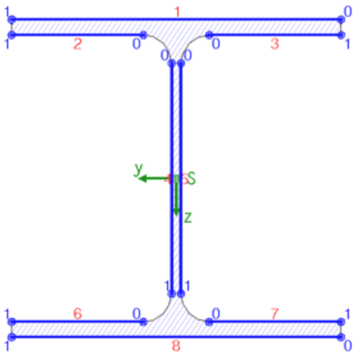
internal moment: $M_{Ed} = M_d$ = 145.00 kNm
shear force: $V_{Ed} = |V_d|$ = 120.00 kN

$M_{Ed}/M_{j,Rd}$ = 0.940 < 1 ok
$V_{Ed}/V_{j,Rd}$ = 0.398 < 1 ok

5.4.2. 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 = 7.0 mm	l_w = 280.0 mm
weld 2:	a_w = 7.0 mm	l_w = 112.0 mm
weld 3:	see weld 2	
weld 4:	a_w = 4.0 mm	l_w = 196.0 mm
weld 5:	see weld 4	
weld 6:	a_w = 7.0 mm	l_w = 112.0 mm
weld 7:	see weld 6	
weld 8:	a_w = 7.0 mm	l_w = 280.0 mm

design values refering to centroid of the section:
$M_{y,Ed}$ = -145.00 kNm, $V_{z,Ed}$ = 120.00 kN

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

ΣA_w = 86.24 cm², $A_{w,z}$ = 15.68 cm², Σl_w = 140.0 cm

$I_{w,y}$ = 12313.79 cm⁴, $I_{w,z}$ = 5104.15 cm⁴, Δz_w = 0.0 mm

distribution of internal forces and moments:

weld 1: N_w = 311.58 kN
weld 2: N_w = 112.63 kN
weld 3: see weld 2
weld 4: $M_{y,w}$ = -2.96 kNm
weld 5: see weld 4
weld 6: N_w = -112.63 kN
weld 7: see weld 6
weld 8: N_w = -311.58 kN

from conventionel distribution of shear force: $V_{z,w}$ = 120.00 kN

verifications in weld edges:

weld 1, pt. 0: $\sigma_{w,x}$ = 158.97 N/mm² ⇒ U_w = 0.624 < 1 ok
weld 2, pt. 0: $\sigma_{w,x}$ = 143.66 N/mm² ⇒ U_w = 0.564 < 1 ok
weld 4, pt. 0: $\sigma_{w,x}$ = 115.40 N/mm² $\tau_{w,z}$ = 76.53 N/mm² ⇒ U_w = 0.584 < 1 ok
pt. 1: $\sigma_{w,x}$ = -115.40 N/mm² $\tau_{w,z}$ = 76.53 N/mm² ⇒ U_w = 0.584 < 1 ok
weld 6, pt. 0: $\sigma_{w,x}$ = -143.66 N/mm² ⇒ U_w = 0.564 < 1 ok
weld 8, pt. 0: $\sigma_{w,x}$ = -158.97 N/mm² ⇒ U_w = 0.624 < 1 ok

Result:

weld 1, pt. 0: $\sigma_{w,x}$ = 158.97 N/mm²

Max: $\sigma_{1,w,Ed}$ = 224.81 N/mm² < $f_{1w,d}$ = 360.00 N/mm²,
$\sigma_{2,w,Ed}$ = 112.41 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U_w = 0.624 < 1 ok

5.4.3. verification result

maximum utilization: max U = 0.940 < 1 ok