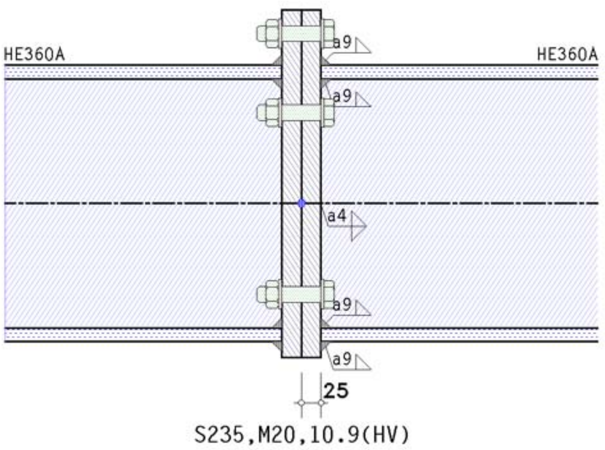
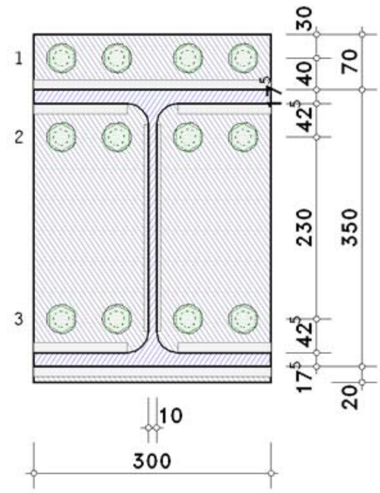
POS. 77: WAGENKNECHT BD.3, BSP. 7.6

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

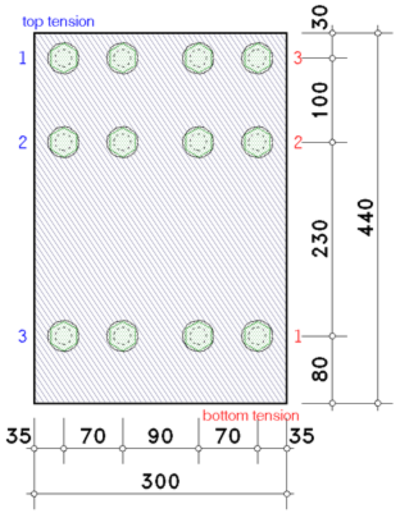
Rigid beam splice EC 3-1-8 (12.10), NA: Deutschland

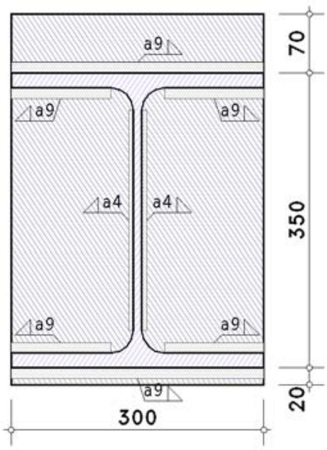
1. input report





details (section A - A)



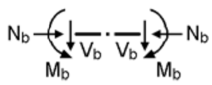


steel grade
steel grade S235
bolts
bolt class 10.9, bolt size M20
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}* = 0.7 \cdot f_{yb} \cdot A_s$ = 154.3 kN)
shear plane passes through the unthreaded portion of the bolt
beam parameters
section HE360A
verification parameters
bolted end-plate connection:
thickness t_p = 25.0 mm, width b_p = 300.0 mm, length l_p = 440.0 mm
projections $h_{p,o}$ = 70.0 mm, $h_{p,u}$ = 20.0 mm
bolts in connection:
  3 bolt-rows with 4 bolts
  row 1: 4 bolts, row 2: 4 bolts, row 3: 4 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 bottom in tension (row 3)
    and 2 bolt-rows for shear transfer bottom (rows 2-3)
  calculation method (4 bolts per row) acc. to Wagenknecht, Stahlbau-Praxis acc. to EC 3, Bd.3
  centre distance between outer and inner bolt w_2 = 70.0 mm
  centre distance of the bolts to the lateral edge of the end-plate e_2 = 35.0 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 = 80.0 mm
  centre distance of the bolt-rows from each other $p_{1\text{-}2}$ = 100.0 mm, $p_{2\text{-}3}$ = 230.0 mm

welds at the connection point:
beam flange top: fillet weld, weld thickness a = 9.0 mm
beam web: fillet weld, weld thickness a = 4.0 mm
beam flange bottom: fillet weld, weld thickness a = 9.0 mm

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

Lk 1: $M_{b,Ed}$ = 320.00 kNm $V_{b,Ed}$ = 180.00 kN
Lk 2: $M_{b,Ed}$ = -120.00 kNm



check of data

ok

distances between bolt-rows at end-plate

horizontal:	e_2 = 35.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_2 = 35.0 mm < $4 \cdot t + 40$ mm = 140.0 mm
horizontal:	p_2 = 70.0 mm > $2.4 \cdot d_0$ = 52.8 mm,	p_2 = 70.0 mm < min(14·t, 200 mm) = 200.0 mm
horizontal:	p_2 = 90.0 mm > $2.4 \cdot d_0$ = 52.8 mm,	p_2 = 90.0 mm < min(14·t, 200 mm) = 200.0 mm
vertical:	e_1 = 30.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 30.0 mm < $4 \cdot t + 40$ mm = 140.0 mm
vertical:	p_1 = 100.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 100.0 mm < min(14·t, 200 mm) = 200.0 mm
vertical:	p_1 = 230.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 230.0 mm > min(14·t, 200 mm) = 200.0 mm !!
vertical:	e_1 = 80.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 80.0 mm < $4 \cdot t + 40$ mm = 140.0 mm

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

notes

no verification for cross-sections.

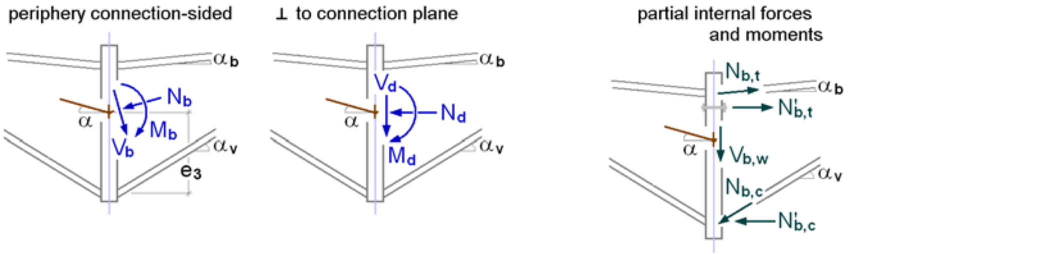
2. Lk 1

notes

connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.
no consideration of bolt groups in joints with 4 bolts per row.

2.1. design values

periphery connection ⊥ zur connection plane   partial internal forces and moments



sign definition of EC3: a positive axial force means compression, a positive bending moment produces tension at the top

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

transformation joint values -> design values
M_d = 320.00 kNm, V_d = 180.00 kN

internal forces and moments perpendicular to the connection planes

periphery beam
M_d = 320.00 kNm, V_d = 180.00 kN

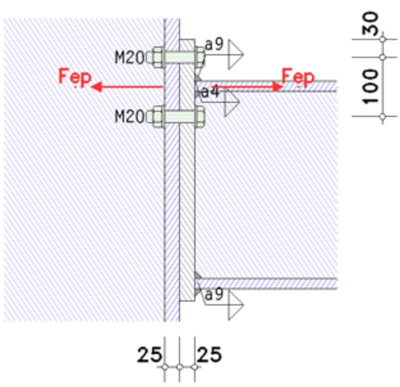
partial internal forces and moments

internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_{ep}$ = 315.50 kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 948.87 kN, z_b = 332.5 mm, z_{bu} = 166.3 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b$ = 948.87 kN, z_b = 332.5 mm, z_{bo} = 166.3 mm

2.2. basic components

beam splice w. end-plate: basic components: 5, 7, 8, 10, 11, 12

2.2.1. Gk 5: end-plate in bending



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

connections with 4 bolts per bolt-row are not treated in EC 3-1-8.
verification according to Wagenknecht, Stahlbau Praxis nach EC 3, Bd.3.

extended part of end-plate

in projecting part of end plate only one bolt-row ($n_b = 1$) is considered (4 bolts per row).

distance centre-line of the bolt to beam flange $m_1 = 29.8$ mm

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

$e_x = 30.0$ mm, $m_x = 29.8$ mm, $w_3 = 35.0$ mm, $w_2 = 70.0$ mm, $w_1 = b_p \text{-} 2 \cdot (w_2 + w_3) = 90.0$ mm with $b_p = 300.0$ mm, $b_{st} = 300.0$ mm

end bolt-row outside tension flange of beam / of stiffener in tension

$l_{eff,cp,1} = 2 \cdot (\pi \cdot m_x + w_2) = 327.3$ mm

$l_{eff,cp,2} = \pi \cdot m_x + 2 \cdot w_2 + w_1 = 323.7$ mm

$l_{eff,cp,3} = \pi \cdot m_x + 2 \cdot (w_2 + w_3) = 303.7$ mm

$l_{eff,cp,4} = 4 \cdot \pi \cdot m_x = 374.7$ mm

$l_{eff,cp,sa} = \min(l_{eff,cp,1},\ l_{eff,cp,2},\ l_{eff,cp,3},\ l_{eff,cp,4}) = 303.7$ mm

$l_{eff,nc,1} = 4 \cdot m_x + 1.25 \cdot e_x + w_2 = 226.8$ mm

$l_{eff,nc,2} = 2 \cdot m_x + 0.625 \cdot e_x + w_2 + 0.5 \cdot w_1 = 193.4$ mm

$l_{eff,nc,3} = 2 \cdot m_x + 0.625 \cdot e_x + w_2 + w_3 = 183.4$ mm

$l_{eff,nc,4} = 8 \cdot m_x + 2.5 \cdot e_x = 313.5$ mm

$l_{eff,nc,5} = w_2 + w_3 + 0.5 \cdot w_1 = 150.0$ mm

$l_{eff,nc,sa} = \min(l_{eff,nc,1},\ l_{eff,nc,2},\ l_{eff,nc,3},\ l_{eff,nc,4},\ l_{eff,nc,5}) = 150.0$ mm

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc},\ l_{eff,cp}) = 150.0$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 150.0$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 30.0$ mm, $e_{min} = 30.0$ mm, $m = 29.8$ mm

resisting plastic moments:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.51$ kNm, $t_f = 25.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.40$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 4 \cdot n_b \cdot F_{t,Rd} = 705.60$ kN, $n_b = 1$

prying forces always appear at preloaded bolts !

calculation with the standard method

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 738.87$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 538.03$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 705.60$ kN, $F_{T,4,Rd} = 2 \cdot M_{pl,1,Rd} / m = 369.43$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd},\ F_{T,2,Rd},\ F_{T,3,Rd}) = 538.03$ kN

shear strength: $f_{vw,d} = (f_u / 3^{1/2}) / (\beta_w \cdot \gamma_{M2}) = 207.8$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$

tension resistance of welds: $F_{T,w,Rd} = 2 \cdot f_{vw,d} \cdot a \cdot l_{eff} = 561.18$ kN (≥ 538.03 kN, not decisive)

resistance and effective length of end-plate in bending (projection)

$F_{t,ep,Rd,1} = 538.03$ kN, $l_{eff,1} = 150.0$ 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 (4 bolts per row)

distance centre-line of the bolt to the stiffener $m_2 = 32.3$ mm

distance centre-line of the bolt to the edge of flange $e = 35.0$ mm

distance centre-line of the bolt to the stub web $m = 35.5$ mm

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

inner bolt-row outside tension flange of beam / of stiffener in tension

$l_{eff,cp,1} = 4 \cdot \pi \cdot m = 445.8$ mm

$l_{eff,cp,2} = 2 \cdot (\pi \cdot m + w_2) = 362.9$ mm

$l_{eff,cp,3} = \pi \cdot m + 2 \cdot (w_2 + w_3) = 321.4$ mm

$l_{eff,cp,si} = \min(l_{eff,cp,1},\ l_{eff,cp,2},\ l_{eff,cp,3}) = 321.4$ mm

distance of inner bolt axis to the edge of flange $e = 105.0$ mm

coefficient for stiffened column flanges and end-plates:

input values $\lambda_1 = m / (m+e) = 0.253$, $\lambda_2 = m_2 / (m+e) = 0.230 \Rightarrow \alpha = 7.37$ (s. figure 6.11)

$l_{eff,nc,1} = \alpha \cdot m = 261.4$ mm

$l_{eff,nc,2} = 4 \cdot m + 1.25 \cdot (w_2 + w_3) = 273.1$ mm

$l_{eff,nc,si} = \min(l_{eff,nc,1},\ l_{eff,nc,2}) = 261.4$ mm

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc},\ l_{eff,cp}) = 261.4$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 261.4$ mm

tension resistance of the T-stub flange

$m = 35.5$ mm, $n = \min(w + e_{min}, 1.25 \cdot m) = 44.3$ mm

$n_1 = w = 70.0$ mm, $n_2 = \min(e_{min}, 1.25 \cdot m + n_1) = 35.0$ mm, $e_{min} = 35.0$ mm

resisting plastic moments:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 9.60$ kNm, $t_f = 25.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.40$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 4 \cdot n_b \cdot F_{t,Rd} = 705.60$ kN, $n_b = 1$

prying forces always appear at preloaded bolts !

calculation with the standard method

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 1082.20$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + (n_1+n_2) \cdot 0.25 \cdot \Sigma F_{t,Rd} \cdot (3.6 - 1.6 \cdot n_1/(n_1+n_2))) / (m+n_1+n_2) = 470.67$ kN

mode 3: bolt failure

$F_{T,3,Rd} = 0.9 \cdot \Sigma F_{t,Rd} = 635.04$ kN, $F_{T,4,Rd} = (3.6 \cdot M_{pl,1,Rd}) / (1.8 \cdot m + 0.8 \cdot n_1) = 288.28$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 470.67$ kN

shear strength: $f_{vw,d} = (f_u/3^{1/2}) / (\beta_w \cdot \gamma_{M2}) = 207.8$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$

tension resistance of welds: $F_{T,w,Rd} = 2 \cdot f_{vw,d} \cdot a \cdot l_{eff} = 434.62$ kN, a = 4.0 mm, $l_{eff} = 261.4$ mm

total loading capacity of the T-stub flange: $F_{T,Rd} = F_{T,w,Rd} = 434.62$ kN

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

$F_{ep,Rd,2} = 434.62$ kN, $l_{eff,2} = 261.4$ mm

2.2.2. Gk 7: beam flange and web in compression

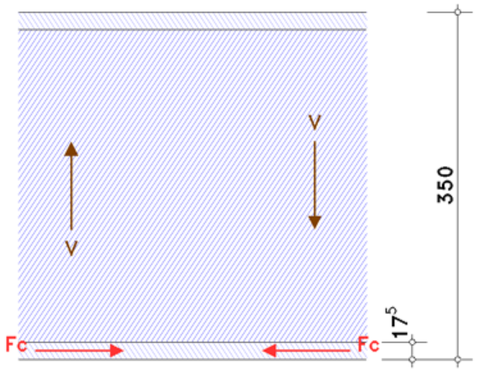
section class of beam ($\varepsilon = 1.00$):

flange bottom: section class for $c/(\varepsilon \cdot t) = 6.74$ (outstand flange): 1

web: section class for $\alpha = 0.50$ and $c/(\varepsilon \cdot t) = 26.10$ (internal compression parts, bending): 1

total: section class: 1

taking into account the moment-shear force-interaction $V_{Ed} = 180.0$ kN



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

stress due to bending with shear force

$V_{pl,Rd} = A_v \cdot (f_y/3^{1/2}) / \gamma_{M0} = 664.2$ kN, $A_v = 48.96$ cm²

$V_{Ed} = 180.0$ kN ≤ 332.1 kN $= V_{pl,Rd}/2 \Rightarrow$ no effect on the moment resistance !

stress for section class 1

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 490.68$ kNm, $W_{pl} = 2088.00$ cm³

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 1475.73$ kN, $(h - t_f) = 332.5$ mm

resistance of upper beam flange:

stress due to bending with shear force

$V_{pl,Rd} = A_v \cdot (f_y/3^{1/2}) / \gamma_{M0} = 664.2$ kN, $A_v = 48.96$ cm²

$V_{Ed} = 180.0$ kN ≤ 332.1 kN $= V_{pl,Rd}/2 \Rightarrow$ no effect on the moment resistance !

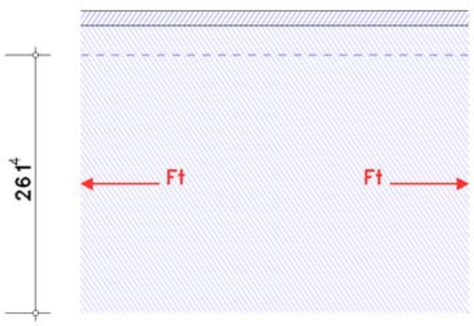
stress for section class 1

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 490.68$ kNm, $W_{pl} = 2088.00$ cm³

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 1475.73$ kN, $(h - t_f) = 332.5$ mm

2.2.3. Gk 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

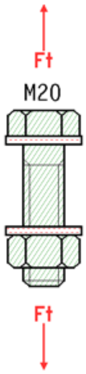
effective width of the beam web in tension $b_{eff,t,wb} = 261.4$ mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 614.2$ kN, $f_{y,wb} = 235.0$ N/mm²

2.2.4. Gk 10: bolts in tension

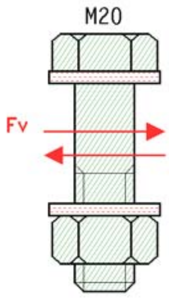
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



bolt category D:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.40$ kN, $k_2 = 0.90$, $f_{ub} = 1000.0$ N/mm²
p. sh. load capicity: $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 454.85$ kN, $d_m = 33.5$ mm, $t_p = 25.0$ mm, $f_u = 360.0$ N/mm²
tension-/punching shear load capicity for 4 bolts: $\Sigma F_{tp,Rd} = 4 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 705.60$ kN

2.2.5. Gk 11: bolts in shear

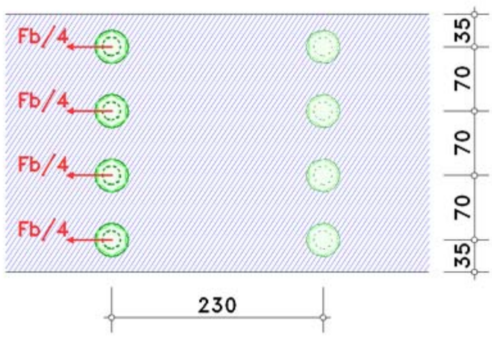
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



bolt category A:
shear plane passes through the unthreaded portion of the bolt: $\alpha_v = 0.6$, $A = 3.14$ cm²
shear resistance per shear plane: $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2} = 150.80$ kN, $f_{ub} = 1000.0$ N/mm²
shear resistance of 4 bolts (1-shear): $\Sigma F_{v,Rd} = 4 \cdot F_{v,Rd} = 603.19$ kN

2.2.6. Gk 12: plate with bearing resistance

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



row 3

bolt 1:
in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 3.23$ (inner bolt)
$\Rightarrow \alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.75$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm

bolt 2:
in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 3.23$ (inner bolt)
$\Rightarrow \alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm

bolt 3:
in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 3.23$ (inner bolt)
$\Rightarrow \alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bolt 4:
in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0)-1/4 = 3.23$ (inner bolt)
$\Rightarrow$ $\alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0-1.7 = 2.75$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0-1.7,\ 1.4 \cdot p_2/d_0-1.7) = 2.75$ (end bolt)
$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bearing resistance of 1x4 bolts: $\Sigma F_{b,Rd} = 1440.00$ kN

2.3. connection capacity

2.3.1. moment resistance

distance of tension-bolt-rows from centre of compression: $h_1 = 381.3$ mm, $h_2 = 281.3$ mm

resistances acc. to EC 3-1-8, 6.2.7.2(6) for bolt-rows considered individually
decisive basic components: 5, 8
row 1: $F_{tr,Rd} = 538.0$ kN
row 2: $F_{tr,Rd} = 434.6$ kN

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 538.0$ kN
row 2: $F_{tr,Rd} = 434.6$ kN
$\Sigma F_{tr,Rd}^* = 972.6$ kN

deductions acc. to EC 3-1-8, 6.2.7.2(7)
decisive basic component: 7
row 1: $\Sigma F_{tr,Rd} = 0.0$ kN
Gk 7:$\Delta F_{tr,Rd} = F_{c,f,Rd} - \Sigma F_{tr,Rd} = 1475.7$ kN$F_{tr,Rd} = 538.0$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 538.0$ kN
row 2: $\Sigma F_{tr,Rd} = 538.0$ kN (row 1)
Gk 7:$\Delta F_{tr,Rd} = F_{c,f,Rd} - \Sigma F_{tr,Rd} = 937.7$ kN$F_{tr,Rd} = 434.6$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 434.6$ kN

check acc. to EC 3-1-8, 6.2.7.2(9)
decisive basic component: 10
row 1: $F_{tx,Rd} = 538.0$ kN, $h_x = 381.3$ mm $\Rightarrow$ $F_{tx,Rd} \leq \lim F_{tx,Rd} = 670.3$ kN, no deduction

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 538.0$ kN
row 2: $F_{tr,Rd} = 434.6$ kN
$\Sigma F_{tr,Rd} = 972.6$ kN
potential failure by basic component 5

resistance of flanges (compression)
$\Sigma F_{c,Rd}^* = 2951.5$ kN

moment resistance regarding the centre of compression
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 327.4$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 972.6$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 2951.5$ kN

2.3.2. shear/bearing resistance

resistance per bolt-row
decisive basic components: 11, 12
row 3: $F_{vr,Rd} = 603.2$ kN

deductions depending on tension force (at 100% utilization of moment resistance)
decisive basic component: 10
row 3: $F_{vr,Rd} = f_{vt} \cdot 603.2$ kN $= 603.2$ kN with $f_{vt} = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd}) = 1.000$

resistance per bolt-row
row 3: $F_{vr,Rd} = 603.2$ kN
$\Sigma F_{vr,Rd} = 603.2$ kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd} = 603.2$ kN

2.3.3. total

$M_{j,Rd}$ = 327.4 kNm $N_{j,t,Rd}$ = 972.6 kN $N_{j,c,Rd}$ = 2951.5 kN $V_{j,Rd}$ = 603.2 kN

2.4. verifications

calculation of internal lever arm z_{eq} s. rotational stiffness

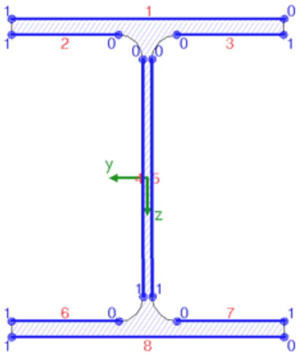
2.4.1. verification of the connection capacity by means of the component method

internal moment: $M_{Ed} = M_d$ = 320.00 kNm
perpend. to connection plane
shear force: $V_{Ed} = |V_d|$ = 180.00 kN
parallel to connection plane

moment resistance
$M_{Ed}/M_{j,Rd} = 0.978 < 1$ ok
shear/bearing resistance at 100% utilization of moment resistance
$V_{Ed}/V_{j,Rd} = 0.298 < 1$ ok

2.4.2. 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
weld 4: NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0 \text{ mm} < a_{min} = t_{max}^{1/2} - 0.5 = 4.50$ mm !!
weld 5: NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0 \text{ mm} < a_{min} = t_{max}^{1/2} - 0.5 = 4.50$ mm !!
calculation section:



weld 1:	a_w = 9.0 mm	l_w = 300.0 mm
weld 2:	a_w = 9.0 mm	l_w = 118.0 mm
weld 3:	siehe weld 2	
weld 4:	a_w = 4.0 mm	l_w = 261.0 mm
weld 5:	siehe weld 4	
weld 6:	a_w = 9.0 mm	l_w = 118.0 mm
weld 7:	siehe weld 6	
weld 8:	a_w = 9.0 mm	l_w = 300.0 mm

design values refering to centroid of the section:
$M_{y,Ed}$ = -320.00 kNm, $V_{z,Ed}$ = 180.00 kN

cross-sectional properties refering to centroid of the line cross-section:
ΣA_w = 117.36 cm², $A_{w,z}$ = 20.88 cm², Σl_w = 159.4 cm
$I_{w,y}$ = 28260.50 cm⁴, $I_{w,z}$ = 8065.90 cm⁴, $W_{w,t}$ = 114.21 cm³, Δz_w = 0.0 mm

distribution of internal forces and moments:

weld 1:	N_w = 535.02 kN
weld 2:	N_w = 189.40 kN
weld 3:	siehe weld 2
weld 4:	$M_{y,w}$ = -6.71 kNm
weld 5:	siehe weld 4
weld 6:	N_w = -189.40 kN
weld 7:	siehe weld 6
weld 8:	N_w = -535.02 kN

from conventionel distribution of shear force: $V_{z,w}$ = 180.00 kN

stresses in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x}$ = 198.16 N/mm²	
weld 2,	pt. 0:	$\sigma_{w,x}$ = 178.34 N/mm²	
weld 3,	pt. 0:	siehe weld 2	
	pt. 1:	siehe weld 2	
weld 4,	pt. 0:	$\sigma_{w,x}$ = 147.77 N/mm²	$\tau_{w,z}$ = 86.21 N/mm²
	pt. 1:	$\sigma_{w,x}$ = -147.77 N/mm²	$\tau_{w,z}$ = 86.21 N/mm²
weld 5,	pt. 0:	siehe weld 4	
	pt. 1:	siehe weld 4	
weld 6,	pt. 0:	$\sigma_{w,x}$ = -178.34 N/mm²	
weld 7,	pt. 0:	siehe weld 6	
	pt. 1:	siehe weld 6	
weld 8,	pt. 0:	$\sigma_{w,x}$ = -198.16 N/mm²	

verifications in weld edges:

verification of weld 1, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 198.2 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 17.83 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 18.71 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 9.0 mm
$F_{w,Ed}$ = 17.83 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm $\Rightarrow$ U = 0.953 < 1 ok

verification of weld 2, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 178.3 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 16.05 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 18.71 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 9.0 mm
$F_{w,Ed}$ = 16.05 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm $\Rightarrow$ U = 0.858 < 1 ok

verification of weld 4, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = (\sigma_{w,x}^2 + \tau_{w,z}^2)^{1/2}$ = 171.1 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 6.84 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 8.31 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 4.0 mm
$F_{w,Ed}$ = 6.84 kN/cm < $F_{w,Rd}$ = 8.31 kN/cm $\Rightarrow$ U = 0.823 < 1 ok

verification of weld 4, pt. 1:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = (\sigma_{w,x}^2 + \tau_{w,z}^2)^{1/2}$ = 171.1 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 6.84 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 8.31 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 4.0 mm
$F_{w,Ed}$ = 6.84 kN/cm < $F_{w,Rd}$ = 8.31 kN/cm $\Rightarrow$ U = 0.823 < 1 ok

verification of weld 6, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 178.3 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 16.05 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 18.71 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 9.0 mm
$F_{w,Ed}$ = 16.05 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm $\Rightarrow$ U = 0.858 < 1 ok

verification of weld 8, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 198.2 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 17.83 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 18.71 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 9.0 mm
$F_{w,Ed}$ = 17.83 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm $\Rightarrow$ U = 0.953 < 1 ok

Result:
weld 1, pt. 0: $\sigma_{w,x}$ = 198.16 N/mm²
Max: $F_{w,Ed}$ = 17.83 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm $\Rightarrow$ U_w = 0.953 < 1 ok

2.4.3. verification result

maximum utilization: max U = 0.978 < 1 ok

2.5. rotational stiffness

stiffness coefficients
equivalent stiffness coefficient for 2 tension-bolt-rows:
effective stiffness coefficient for bolt-row 1 (4 bolts):
$k_5 = 0.9 \cdot l_{eff} \cdot t_p^3 / m^3$ = 79.57 mm, l_{eff} = 150.0 mm, m = 29.8 mm
$k_{10} = 1.6 \cdot A_s / L_b$ = 5.43 mm, $L_b = t_{ges} + 2 \cdot t_p + (t_k + t_m)/2$ = 72.3 mm, t_{ges} = 50.0 mm
$\Sigma(1/k_{i,1}) = 1/k_5 + 1/k_5 + 1/(2 \cdot k_{10})$ = 0.117 $\Rightarrow$ $k_{eff,1} = 1 / \Sigma(1/k_{i,1})$ = 8.526 mm
effective stiffness coefficient for bolt-row 2 (4 bolts):
$k_5 = 0.9 \cdot l_{eff} \cdot t_p^3 / m^3$ = 82.34 mm, l_{eff} = 261.4 mm, m = 35.5 mm
$k_{10} = 1.6 \cdot A_s / L_b$ = 5.43 mm, $L_b = t_{ges} + 2 \cdot t_p + (t_k + t_m)/2$ = 72.3 mm, t_{ges} = 50.0 mm
$\Sigma(1/k_{i,2}) = 1/k_5 + 1/k_5 + 1/(2 \cdot k_{10})$ = 0.116 $\Rightarrow$ $k_{eff,2} = 1 / \Sigma(1/k_{i,2})$ = 8.588 mm
equivalent internal lever arm: $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r)$ = 338.6 mm
$k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq}$ = 16.732 mm
sum of stiffness coefficients $\Sigma(1/k_i) = 1/k_{eq}$ = 0.060
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i)$ = 402890.9 kNm/rad, $z = z_{eq}$ = 338.6 mm
internal moment at the connection point: $M_{j,Ed} = M_{Ed}$ = 320.00 kNm
$|M_{j,Ed}|$ = 320.00 kNm > 2/3 $M_{j,Rd}$ = 218.2 kNm $\Rightarrow$ $\mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi}$ = 2.810, Ψ = 2.7
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu$ = 143352.4 kNm/rad
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd}$ = 0.128°

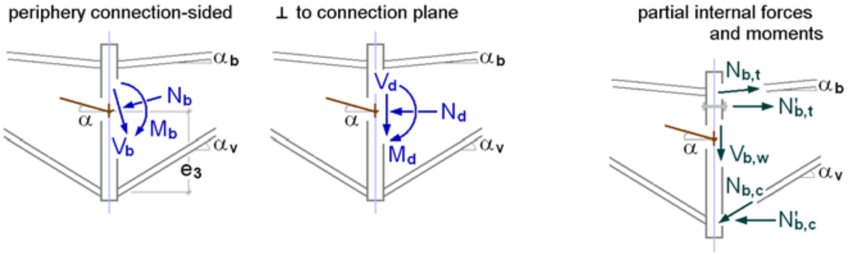
3. Lk 2

notes

connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.
no consideration of bolt groups in joints with 4 bolts per row.

3.1. design values

periphery connection ⊥ zur connection plane   partial internal forces and moments



sign definition of EC3:   a positive axial force means compression, a positive bending moment produces tension at the top

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

transformation joint values -> design values

M_d = -120.00 kNm

internal forces and moments perpendicular to the connection planes

periphery beam

M_d = -120.00 kNm

negative internal moment M_d $\Rightarrow$ mirrored model

M_d = 120.00 kNm

partial internal forces and moments refering to the mirrored model

internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_{ep}$ = 120.00 kNm

$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 360.90 kN, z_b = 332.5 mm, z_{bu} = 166.3 mm

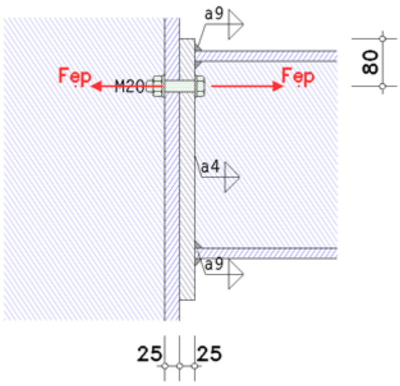
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b$ = 360.90 kN, z_b = 332.5 mm, z_{bo} = 166.3 mm

bending: bolt-rows below centre of compression are ignored !!

3.2. basic components

beam splice w. end-plate: basic components: 5, 7, 8, 10, 11, 12

3.2.1. Gk 5: end-plate in bending



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

connections with 4 bolts per bolt-row are not treated in EC 3-1-8.
verification according to Wagenknecht, Stahlbau Praxis nach EC 3, Bd.3.

part of end-plate between beam flanges

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

here: number of bolt-rows n_b = 1

distance centre-line of the bolt to the stiffener m_2 = 32.3 mm

distance centre-line of the bolt to the edge of flange e = 35.0 mm

distance centre-line of the bolt to the stub web m = 35.5 mm

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

inner bolt-row outside tension flange of beam / of stiffener in tension

$l_{eff,cp,1} = 4 \cdot \pi \cdot m$ = 445.8 mm

$l_{eff,cp,2} = 2 \cdot (\pi \cdot m + w_2)$ = 362.9 mm

$l_{eff,cp,3} = \pi \cdot m + 2 \cdot (w_2 + w_3)$ = 321.4 mm

$l_{eff,cp,si} = \min(l_{eff,cp,1}, l_{eff,cp,2}, l_{eff,cp,3})$ = 321.4 mm

distance of inner bolt axis to the edge of flange e = 105.0 mm

coefficient for stiffened column flanges and end-plates:

input values $\lambda_1 = m / (m+e)$ = 0.253, $\lambda_2 = m_2 / (m+e)$ = 0.230 $\Rightarrow$ α = 7.37 (s. figure 6.11)

$l_{eff,nc,1} = \alpha \cdot m$ = 261.4 mm

$l_{eff,nc,2} = 4 \cdot m + 1.25 \cdot (w_2 + w_3)$ = 273.1 mm

$l_{eff,nc,si} = \min(l_{eff,nc,1},\ l_{eff,nc,2}) = 261.4$ mm
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc},\ l_{eff,cp}) = 261.4$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 261.4$ mm

tension resistance of the T-stub flange

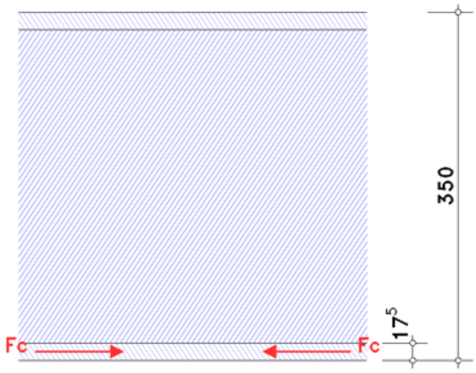
$m = 35.5$ mm, $n = \min(w+e_{min}, 1.25 \cdot m) = 44.3$ mm
$n_1 = w = 70.0$ mm, $n_2 = \min(e_{min}, 1.25 \cdot m+n_1) = 35.0$ mm, $e_{min} = 35.0$ mm
resisting plastic moments:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 9.60$ kNm, $t_f = 25.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$
design value of tension resistance:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.40$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 4 \cdot n_b \cdot F_{t,Rd} = 705.60$ kN, $n_b = 1$
prying forces always appear at preloaded bolts !
calculation with the standard method
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 1082.20$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + (n_1+n_2) \cdot 0.25 \cdot \Sigma F_{t,Rd} \cdot (3.6 - 1.6 \cdot n_1/(n_1+n_2))) / (m+n_1+n_2) = 470.67$ kN
mode 3: bolt failure
$F_{T,3,Rd} = 0.9 \cdot \Sigma F_{t,Rd} = 635.04$ kN, $F_{T,4,Rd} = (3.6 \cdot M_{pl,1,Rd}) / (1.8 \cdot m + 0.8 \cdot n_1) = 288.28$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd},\ F_{T,2,Rd},\ F_{T,3,Rd}) = 470.67$ kN
shear strength: $f_{vw,d} = (f_u/3^{1/2}) / (\beta_w \cdot \gamma_{M2}) = 207.8$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$
tension resistance of welds: $F_{T,w,Rd} = 2 \cdot f_{vw,d} \cdot a \cdot l_{eff} = 434.62$ kN, $a = 4.0$ mm, $l_{eff} = 261.4$ mm
total loading capacity of the T-stub flange: $F_{T,Rd} = F_{T,w,Rd} = 434.62$ kN

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

$F_{ep,Rd,1} = 434.62$ kN, $l_{eff,1} = 261.4$ mm

3.2.2. Gk 7: beam flange and web in compression

section class of beam ($\varepsilon = 1.00$):
flange bottom: section class for $c/(\varepsilon \cdot t) = 6.74$ (outstand flange): 1
web: section class for $\alpha = 0.50$ and $c/(\varepsilon \cdot t) = 26.10$ (internal compression parts, bending): 1
total: section class: 1



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

stress for section class 1

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 490.68$ kNm, $W_{pl} = 2088.00$ cm³

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 1475.73$ kN, $(h - t_f) = 332.5$ mm

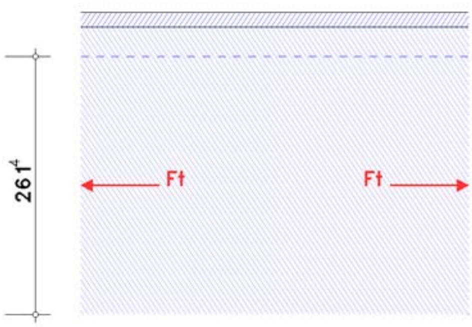
resistance of upper beam flange:
stress for section class 1

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 490.68$ kNm, $W_{pl} = 2088.00$ cm³

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 1475.73$ kN, $(h - t_f) = 332.5$ mm

3.2.3. Gk 8: beam web in tension



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

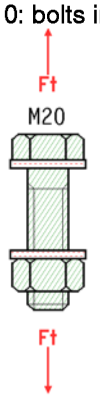
each individual bolt-row:
effective width

effective width of the beam web in tension $b_{eff,t,wb} = 261.4$ mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 614.2$ kN, $f_{y,wb} = 235.0$ N/mm²

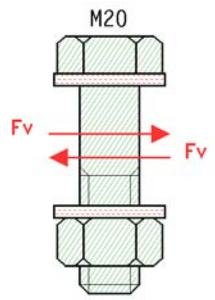
3.2.4. Gk 10: bolts in tension



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

bolt category D:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 176.40$ kN, $k_2 = 0.90$, $f_{ub} = 1000.0$ N/mm²
p. sh. load capicity: $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 454.85$ kN, $d_m = 33.5$ mm, $t_p = 25.0$ mm, $f_u = 360.0$ N/mm²
tension-/punching shear load capicity for 4 bolts: $\Sigma F_{tp,Rd} = 4 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 705.60$ kN

3.2.5. Gk 11: bolts in shear

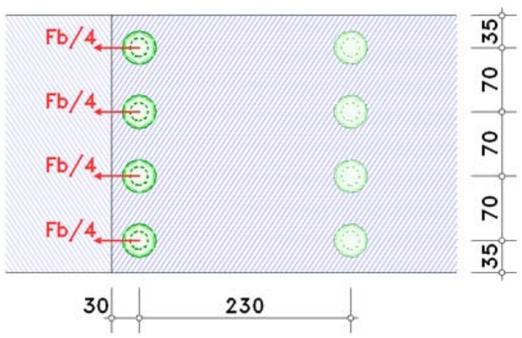


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

bolt category A:
shear plane passes through the unthreaded portion of the bolt: $\alpha_v = 0.6$, $A = 3.14$ cm²
shear resistance per shear plane: $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2} = 150.80$ kN, $f_{ub} = 1000.0$ N/mm²
shear resistance of 4 bolts (1-shear): $\Sigma F_{v,Rd} = 4 \cdot F_{v,Rd} = 603.19$ kN

3.2.6. Gk 12: plate with bearing resistance

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



row 2

bolt 1:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.75$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bolt 2:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bolt 3:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)

$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bolt 4:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.75$ (end bolt)
$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bearing resistance of 1x4 bolts: $\Sigma F_{b,Rd} = 1440.00$ kN

row 3

bolt 1:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.75$ (end bolt)
$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bolt 2:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bolt 3:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bolt 4:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 2.75$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.75$ (end bolt)
$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 360.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 20.0$ mm
bearing resistance of 1x4 bolts: $\Sigma F_{b,Rd} = 1440.00$ kN

bearing resistance (2 rows)

$\Sigma F_{b,Rd,2} = 1440.00$ kN
$\Sigma F_{b,Rd,3} = 1440.00$ kN

3.3. connection capacity

3.3.1. moment resistance

distance of tension-bolt-row from centre of compression: $h_1 = 281.3$ mm

resistances acc. to EC 3-1-8, 6.2.7.2(6) for bolt-rows considered individually
decisive basic components: 5, 8
row 1: $F_{tr,Rd} = 434.6$ kN

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 434.6$ kN
$\Sigma F_{tr,Rd}^* = 434.6$ kN

deductions acc. to EC 3-1-8, 6.2.7.2(7)
decisive basic component: 7
row 1: $\Sigma F_{tr,Rd} = 0.0$ kN
Gk 7:$\Delta F_{tr,Rd} = F_{c,f,Rd} - \Sigma F_{tr,Rd} = 1475.7$ kN$F_{tr,Rd} = 434.6$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 434.6$ kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 434.6$ kN
$\Sigma F_{tr,Rd} = 434.6$ kN
potential failure by basic component 5

resistance of flanges (compression)
$\Sigma F_{c,Rd}^* = 2951.5$ kN

moment resistance regarding the centre of compression
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 122.2$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 434.6$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 2951.5$ kN

3.3.2. shear/bearing resistance

resistance per bolt-row
decisive basic components: 11, 12
row 2: $F_{vr,Rd}$ = 603.2 kN
row 3: $F_{vr,Rd}$ = 603.2 kN

deductions depending on tension force (at 100% utilization of moment resistance)
decisive basic component: 10
row 2: $F_{vr,Rd} = f_{vt} \cdot 603.2$ kN = 603.2 kN with $f_{vt} = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd}) = 1.000$
row 3: $F_{vr,Rd} = f_{vt} \cdot 603.2$ kN = 603.2 kN with $f_{vt} = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd}) = 1.000$

resistance per bolt-row
row 2: $F_{vr,Rd}$ = 603.2 kN
row 3: $F_{vr,Rd}$ = 603.2 kN
$\Sigma F_{vr,Rd}$ = 1206.4 kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd}$ = 1206.4 kN

3.3.3. total

$M_{j,Rd}$ = 122.2 kNm $N_{j,t,Rd}$ = 434.6 kN $N_{j,c,Rd}$ = 2951.5 kN $V_{j,Rd}$ = 1206.4 kN

3.4. verifications

calculation of internal lever arm z_{eq} s. rotational stiffness

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

internal moment: $M_{Ed} = M_d$ = 120.00 kNm
perpend. to connection plane

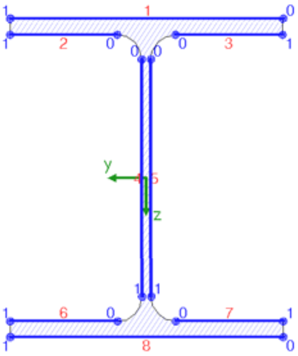
moment resistance
$M_{Ed}/M_{j,Rd} = 0.982 < 1$ ok

3.4.2. 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

weld 4: NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0$ mm $< a_{min} = t_{max}^{1/2} - 0.5 = 4.50$ mm !!
weld 5: NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0$ mm $< a_{min} = t_{max}^{1/2} - 0.5 = 4.50$ mm !!

calculation section:



weld 1:	a_w = 9.0 mm	l_w = 300.0 mm
weld 2:	a_w = 9.0 mm	l_w = 118.0 mm
weld 3:	siehe weld 2	
weld 4:	a_w = 4.0 mm	l_w = 261.0 mm
weld 5:	siehe weld 4	
weld 6:	a_w = 9.0 mm	l_w = 118.0 mm
weld 7:	siehe weld 6	
weld 8:	a_w = 9.0 mm	l_w = 300.0 mm

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

cross-sectional properties refering to centroid of the line cross-section:
ΣA_w = 117.36 cm², $A_{w,z}$ = 20.88 cm², Σl_w = 159.4 cm
$I_{w,y}$ = 28260.50 cm⁴, $I_{w,z}$ = 8065.90 cm⁴, $W_{w,t}$ = 114.21 cm³, Δz_w = 0.0 mm

distribution of internal forces and moments:
weld 1: N_w = 200.63 kN
weld 2: N_w = 71.02 kN
weld 3: siehe weld 2
weld 4: $M_{y,w}$ = -2.52 kNm
weld 5: siehe weld 4
weld 6: N_w = -71.02 kN
weld 7: siehe weld 6
weld 8: N_w = -200.63 kN

stresses in weld edges:

weld 1, pt. 0: $\sigma_{w,x}$ = 74.31 N/mm²
weld 2, pt. 0: $\sigma_{w,x}$ = 66.88 N/mm²
weld 3, pt. 0: siehe weld 2
pt. 1: siehe weld 2
weld 4, pt. 0: $\sigma_{w,x}$ = 55.41 N/mm²
pt. 1: $\sigma_{w,x}$ = -55.41 N/mm²
weld 5, pt. 0: siehe weld 4
pt. 1: siehe weld 4
weld 6, pt. 0: $\sigma_{w,x}$ = -66.88 N/mm²
weld 7, pt. 0: siehe weld 6
pt. 1: siehe weld 6
weld 8, pt. 0: $\sigma_{w,x}$ = -74.31 N/mm²

verifications in weld edges:

verification of weld 1, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 74.3 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 6.69 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 18.71 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 9.0 mm
$F_{w,Ed}$ = 6.69 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm ⇒ U = 0.358 < 1 ok

verification of weld 2, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 66.9 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 6.02 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 18.71 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 9.0 mm
$F_{w,Ed}$ = 6.02 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm ⇒ U = 0.322 < 1 ok

verification of weld 4, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 55.4 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 2.22 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 8.31 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 4.0 mm
$F_{w,Ed}$ = 2.22 kN/cm < $F_{w,Rd}$ = 8.31 kN/cm ⇒ U = 0.267 < 1 ok

verification of weld 4, pt. 1:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 55.4 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 2.22 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 8.31 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 4.0 mm
$F_{w,Ed}$ = 2.22 kN/cm < $F_{w,Rd}$ = 8.31 kN/cm ⇒ U = 0.267 < 1 ok

verification of weld 6, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 66.9 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 6.02 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 18.71 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 9.0 mm
$F_{w,Ed}$ = 6.02 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm ⇒ U = 0.322 < 1 ok

verification of weld 8, pt. 0:
stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 74.3 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 6.69 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 18.71 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 9.0 mm
$F_{w,Ed}$ = 6.69 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm ⇒ U = 0.358 < 1 ok

Result:
weld 1, pt. 0: $\sigma_{w,x}$ = 74.31 N/mm²
Max: $F_{w,Ed}$ = 6.69 kN/cm < $F_{w,Rd}$ = 18.71 kN/cm ⇒ U_w = 0.358 < 1 ok

3.4.3. verification result

maximum utilization: max U = 0.982 < 1 ok

3.5. rotational stiffness

stiffness coefficients
stiffness coefficient of basic component 5:
$k_5 = 0.9 \cdot l_{eff} \cdot t_p^3 / m^3$ = 82.34 mm, l_{eff} = 261.4 mm, m = 35.5 mm (bolt-row 1)
stiffness coefficient of basic component 10:
$k_{10} = 1.6 \cdot A_s / L_b$ = 5.43 mm, $L_b = t_{ges} + 2 \cdot t_p + (t_k + t_m)/2$ = 72.3 mm, t_{ges} = 50.0 mm
sum of stiffness coefficients $\Sigma(1/k_i) = 1/k_5 + 1/k_5 + 1/(2 \cdot k_{10})$ = 0.116
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i)$ = 142652.2 kNm/rad, z = 281.3 mm
internal moment at the connection point: $M_{j,Ed} = M_{Ed}$ = 120.00 kNm
$|M_{j,Ed}|$ = 120.00 kNm > 2/3 $M_{j,Rd}$ = 81.5 kNm ⇒ $\mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi}$ = 2.843, Ψ = 2.7
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu$ = 50174.7 kNm/rad

rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd} = 0.137°$

4. final result

utilization/rotation of the connection

Lk	$S_{j,ini}$	S_j	φ_j	U_j	Gleichgewicht ΣH	ΣV	ΣM	
--	MNm/rad	MNm/rad	°		kN	kN	kNm	
1	402.9	143.4	0.128	0.978	0.00	180.00	320.00	!!
2	142.7	50.2	0.137	0.982*	0.00	0.00	120.00	!!

$S_{j,ini}$: initial rotational stiffness; S_j: rotational stiffness; φ_j: rotation; U_j: utilization of the connection; tolerances of equilibrium 1 kN / 1 kNm
*) maximum utilization

maximum utilization [Lk 2]: max U = 0.982 < 1 ok
minimum rotational stiffness: min S_j = 50.2 MNm/rad, $S_{j,ini}$ = 142.7 MNm/rad, φ_j = 0.137°

verification succeeded