1. input report

web stiffener (single-sided), $t = 7.0$ mm

connection right

web stiffener (single-sided), $t = 7.0$ mm
According to EC 3-1-8, 5.3 in a double-sided beam-column joint each joint is modelled separately.

**Steel grade**

Steel grade S235

**Column parameters**

**Section** IPE180

Reinforcement of the section by 1 supplementary web plate(s):  
- thickness $t_w = 7.0$ mm, width $b_w = 146.0$ mm  
- weld thickness $a_w = 5.0$ mm

Reinforcement of the section with transverse stiffeners (web stiffeners, $d_{st} = 172.0$ mm):  
- thickness $t_{st} = 8.0$ mm, width $b_{st} = 42.9$ mm, length $l_{st} = 164.0$ mm  
- recess at stiffeners $c_{st} = 13.5$ mm  
- welds $a_{st,f} = 3.0$ mm, $a_{st,w} = 3.0$ mm

**Double-sided beam-column joint, right**

- **Bolts**
  - Bolt class 8.8, bolt size M12
  - Large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,c^*} = 0.7 f_{yb} A_d = 37.8$ kN)
  - Shear plane passes through the unthreaded portion of the bolt

**Beam parameters**

- Section IPE180

**Verification parameters**
bolted end-plate connection:
thickness $t_p = 10.0\, \text{mm}$, width $b_p = 90.0\, \text{mm}$, length $l_p = 240.0\, \text{mm}$
projections $h_p, o = 50.0\, \text{mm}$, $h_p, u = 10.0\, \text{mm}$
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 bgzl. row 1
verification with the Component method: MNV-interaction acc. to Cerfontaine (in Jaspart/Weynand)
centre distance of the bolts to the lateral edge of the end-plate $a_2 = 20.0\, \text{mm}$
centre distance of the first bolt-row to the upper edge of the end-plate (end row) $e_a = 20.0\, \text{mm}$
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) $e_a = 50.0\, \text{mm}$
centre distance of the bolt-rows from each other $p_{1-2} = 70.0\, \text{mm}$, $p_{2-3} = 100.0\, \text{mm}$
welds at the connection point:
beam flange top: fillet weld, weld thickness $a = 3.0\, \text{mm}$
beam web: fillet weld, weld thickness $a = 3.0\, \text{mm}$
beam flange bottom: fillet weld, weld thickness $a = 3.0\, \text{mm}$
double-sided beam-column joint, left
bolts
class 8.8, bolt size M12
large wrench size (high strength bolt), preloaded (for into: preloading $F_{P,c} = 0.7 I_{Wb} \cdot A_g = 37.8\, \text{kN}$)
shave plane passes through the unthreaded portion of the bolt
beam parameters
section IP180
verification parameters
bolted end-plate connection:
thickness $t_p = 8.0\, \text{mm}$, width $b_p = 90.0\, \text{mm}$, length $l_p = 180.0\, \text{mm}$
projections $h_p, o = 0.0\, \text{mm}$, $h_p, u = 0.0\, \text{mm}$
bolts in connection:
2 bolt-rows with 2 bolts
all bolt-rows considered individually
all bolt-rows for shear transfer (rows 1-2)
bolt groups generated automatically, considering all groups bgzl. row 1
verification with the Component method: MNV-interaction acc. to Cerfontaine (in Jaspart/Weynand)
centre distance of the bolts to the lateral edge of the end-plate $a_2 = 20.0\, \text{mm}$
centre distance of the first bolt-row to the upper edge of the end-plate (end row) $e_a = 40.0\, \text{mm}$
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) $e_a = 40.0\, \text{mm}$
centre distance of the bolt-rows from each other $p_{1-2} = 100.0\, \text{mm}$
welds at the connection point:
beam flange top: fillet weld, weld thickness $a = 3.0\, \text{mm}$
beam web: fillet weld, weld thickness $a = 3.0\, \text{mm}$
beam flange bottom: fillet weld, weld thickness $a = 3.0\, \text{mm}$
internal forces and moments in the intersection point of system axes referring to the non-haunched axis
intenal forces and moments by sign definition of statics

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partial safety factors for material
resistance of cross-sections $\gamma_{MO} = 1.00$
resistance of members in stability failure $\gamma_{M1} = 1.10$
resistance of bolts, welds, plates in bearing $\gamma_{M2} = 1.25$
prestress of high strength bolts $\gamma_{M7} = 1.10$

check of data
connection right:
ok
connection left:
ok
bolts right:
distances between bolt-rows at end-plate
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U_MN: stress utilization at the beam; U_Wc: stress utilization at the column; U_MNV: utilization due to MNV-interaction
U_Wc: utilization due to shear in column web; U_SBP: utilization due to shear in end-plate; U_Sb: utilization due to weld
U_Ss: utilization due to stiffeners/ribs; U: utilization of each joint; U: utilization of each joint

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U_MN: stress utilization at the beam; U_Wc: stress utilization at the column; U_MNV: utilization due to MNV-interaction
U_Wc: utilization due to shear in column web; U_SBP: utilization due to shear in end-plate; U_Sb: utilization due to weld
U_Ss: utilization due to stiffeners/ribs; U: utilization of each joint; U: utilization of each joint

2. Final result
utilization/rotation of the connection
## 3. Detailed edition of Lk 13 (decisive)

### notes

- No verification for welds of supplementary web plates.

#### 3.1. connection right

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

#### 3.1.1. design values

Knotenschnittgrößen periphery connection I zur connection plane

<table>
<thead>
<tr>
<th>Lk</th>
<th>$S_{J, \text{int}}$</th>
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<th>$\varphi_{J}$</th>
<th>$u_{J}$</th>
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$S_{J, \text{int}}$: initial rotational stiffness; $S_{J}$: rotational stiffness; $\varphi_{J}$: rotation; $u_{J}$: utilization of the connection; tolerances of equilibrium $1 \text{kN} / 1 \text{MNm}$

* maximum utilization

### maximum utilization [Lk 13]:

$\text{max } u_{J} = 0.982 < 1 \text{ ok}$

### minimum rotational stiffness (right):

$\text{min } S_{J} = 3.5 \text{ MNm/rad}, \hspace{1cm} S_{J, \text{ini}} = 3.5 \text{ MNm/rad}, \hspace{1cm} \varphi_{J} = 0.086^\circ$

### minimum rotational stiffness (left):

$\text{min } S_{J} = 3.3 \text{ MNm/rad}, \hspace{1cm} S_{J, \text{ini}} = 3.3 \text{ MNm/rad}, \hspace{1cm} \varphi_{J} = 0.045^\circ$

### verification succeeded

#### slope angle:

$\alpha_{b} = \alpha_{u} = \alpha = 0^\circ$

#### distance:

$e_{1} = 90.0 \text{ mm}, \hspace{1cm} e_{3} = 86.0 \text{ mm}, \hspace{1cm} e_{2} = 86.0 \text{ mm}, \hspace{1cm} e_{5} = 90.0 \text{ mm}, \hspace{1cm} e_{7} = 86.0 \text{ mm}$

### internal forces and moments perpendicular to the connection planes

#### periphery beam (right)

- $N_{A} = -0.88 \text{ kN}$, $M_{D} = -5.26 \text{ kNm}$, $V_{A} = -1.57 \text{ kN}$
- $N_{A2} = 7.31 \text{ kN}$, $M_{D2} = 9.31 \text{ kNm}$, $V_{A2} = -6.05 \text{ kN}$
- $N_{C} = 7.09 \text{ kN}$, $M_{C} = -9.87 \text{ kNm}$, $V_{C} = 13.65 \text{ kN}$

#### periphery column (bottom)

- $N_{C2} = 2.61 \text{ kN}$, $M_{D2} = 3.75 \text{ kNm}$, $V_{C2} = 5.46 \text{ kN}$

#### periphery column (top)

- $N_{A2} = -0.88 \text{ kN}$, $M_{D2} = 5.26 \text{ kNm}$, $V_{A2} = 1.57 \text{ kN}$
- $N_{A2} = 7.31 \text{ kN}$, $M_{D2} = -9.31 \text{ kNm}$, $V_{A2} = 6.05 \text{ kN}$
- $N_{C} = 7.09 \text{ kN}$, $M_{C} = 9.87 \text{ kNm}$, $V_{C} = 5.46 \text{ kN}$

#### partial internal forces and moments referring to the mirrored model

- Internal forces and moments in the periphery end-plate-beam: $M_{d} = M_{D} - V_{A2, \text{ep}} = 5.25 \text{ kNm}$
- $N_{b1, e} = -N_{b2, e} z_{b} / z_{b} + M_{d} w_{2} z_{b} = 30.93 \text{ kN}$, $z_{b} = 172.0 \text{ mm}, z_{b2} = 86.0 \text{ mm}$
\[ N_{0,c} = Nd_{ztbo}/Z_b + M_d/Z_b = 30.06 \text{ kN}, \quad z_b = 172.0 \text{ mm}, \quad z_{bo} = 86.0 \text{ mm} \]

### 3.1.2. Resistance of cross-section

**Column bottom**
- Plastic cross-sectional check for: \( N = -2.61 \text{ kN}, M_y = -3.75 \text{ kNm}, V_z = -5.46 \text{ kN} \)
- Top flange: resistance forces \( N_{\text{max},0} = 171.08 \text{ kN}, N_{\text{min},0} = -171.08 \text{ kN} \)
- Bottom flange: resistance forces \( N_{\text{max},u} = 171.08 \text{ kN}, N_{\text{min},u} = -171.08 \text{ kN} \)
- Web: shear force \( V_{S} = -5.46 \text{ kN}, \text{ shear stress } \tau_S = 0.60 \text{ kN/cm}^2 \Rightarrow U_{c,S} = 0.044 \)
- Resistance forces \( N_{\text{max},s} = 214.02 \text{ kN}, N_{\text{min},s} = -214.02 \text{ kN} \)
- Main bending: axial force \( N = -2.61 \text{ kN}, \) resistance forces \( N_{\text{max}} = 556.18 \text{ kN}, N_{\text{min}} = -556.18 \text{ kN} \Rightarrow U_N = 0.005 \)
- Moment \( M_y = -3.75 \text{ kNm}, \) resistance moments \( M_{y,\text{max}} = 38.63 \text{kNm}, M_{y,\text{min}} = -38.63 \text{kNm} \Rightarrow U_{My} = 0.097 \)
- Total (possibly due to load increase): max \( U = 0.100 < 1 \text{ ok} \)
- Utilizations: \( U_{c} = 0.100 < 1 \text{ ok}, \) c/t-ratio \( U_{ct} = 0.101 < 1 \text{ ok} \)

**Column top**
- Plastic cross-sectional check for: \( N = -7.09 \text{ kN}, M_y = 9.87 \text{ kNm}, V_z = -13.65 \text{ kN} \)
- Top flange: resistance forces \( N_{\text{max},0} = 171.08 \text{ kN}, N_{\text{min},0} = -171.08 \text{ kN} \)
- Bottom flange: resistance forces \( N_{\text{max},u} = 171.08 \text{ kN}, N_{\text{min},u} = -171.08 \text{ kN} \)
- Web: shear force \( V_{S} = -13.65 \text{ kN}, \text{ shear stress } \tau_S = 1.50 \text{ kN/cm}^2 \Rightarrow U_{c,S} = 0.110 \)
- Resistance forces \( N_{\text{max},s} = 212.92 \text{ kN}, N_{\text{min},s} = -212.92 \text{ kN} \)
- Main bending: axial force \( N = -7.09 \text{ kN}, \) resistance forces \( N_{\text{max}} = 555.08 \text{ kN}, N_{\text{min}} = -555.08 \text{ kN} \Rightarrow U_N = 0.013 \)
- Moment \( M_y = 9.87 \text{ kNm}, \) resistance moments \( M_{y,\text{max}} = 38.57 \text{kNm}, M_{y,\text{min}} = -38.57 \text{kNm} \Rightarrow U_{My} = 0.256 \)
- Total (possibly due to load increase): max \( U = 0.263 < 1 \text{ ok} \)
- Utilizations: \( U_{c} = 0.263 < 1 \text{ ok}, \) c/t-ratio \( U_{ct} = 0.164 < 1 \text{ ok} \)

**Beam**
- Plastic cross-sectional check for: \( N = 0.88 \text{ kN}, M_y = -5.25 \text{ kNm}, V_z = 1.57 \text{ kN} \)
- Top flange: resistance forces \( N_{\text{max},0} = 171.08 \text{ kN}, N_{\text{min},0} = -171.08 \text{ kN} \)
- Bottom flange: resistance forces \( N_{\text{max},u} = 171.08 \text{ kN}, N_{\text{min},u} = -171.08 \text{ kN} \)
- Web: shear force \( V_{S} = 1.57 \text{ kN}, \text{ shear stress } \tau_S = 0.17 \text{ kN/cm}^2 \Rightarrow U_{c,S} = 0.013 \)
- Resistance forces \( N_{\text{max},s} = 212.21 \text{ kN}, N_{\text{min},s} = -212.21 \text{ kN} \)
- Main bending: axial force \( N = 0.88 \text{ kN}, \) resistance forces \( N_{\text{max}} = 556.37 \text{ kN}, N_{\text{min}} = -556.37 \text{ kN} \Rightarrow U_N = 0.002 \)
- Moment \( M_y = -5.25 \text{ kNm}, \) resistance moments \( M_{y,\text{max}} = 38.64 \text{kNm}, M_{y,\text{min}} = -38.64 \text{kNm} \Rightarrow U_{My} = 0.136 \)
- Total (possibly due to load increase): max \( U = 0.136 < 1 \text{ ok} \)
- Utilizations: \( U_{c} = 0.136 < 1 \text{ ok}, \) c/t-ratio \( U_{ct} = 0.117 < 1 \text{ ok} \)

### 3.1.3. Basic Components

#### 3.1.3.1. Gk 1: Column web panel in shear

**Transformation parameter (EC 3-1-8, 5.3(9))**

\[ \beta_1 = |1 - M_2/M_1| = 2.00 \]

For \( M_1 = -5.40 \text{ kNm}, M_2 = 9.86 \text{ kNm} \)

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

[Diagram of web panel in shear]

- Web thickness incl. reinforcement \( t_{wc} = 10.6 \text{ mm} \)
- Slenderness of column web \( d_{w}/t_{wc} = 13.77 < 69.69 = 69.00 \Rightarrow \text{method applicable} \)
- Shear area with reinforcement \( A_v = 18.99 \text{ cm}^2 \)
- Plastic shear resistance without stiffeners \( V_{wp,Rd} = (0.9f_{y,w}A_v) / (31/2\gamma_M) = 231.9 \text{ kN} \)
- Placing of intermediate web stiffeners:
  - Additional resistance \( V_{wp,add,Rd} = 4M_{pl,tc,Rd}/d_{st} = 8.0 \text{ kN} \)
  - \( V_{wp,add,Rd} > 2(M_{pl,tc,Rd}+M_{pl,ct,Rd})/d_{st} = 7.7 \text{ kN} \Rightarrow V_{wp,add,Rd} = 7.7 \text{ kN} \)
- Plastic shear resistance with transverse stiffeners \( V_{wp,Rd} = 239.6 \text{ kN} \)
3.1.3.2. Gk 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) β = |1 - M2/M1| = 2.00 for M1 = -5.40 kNm, M2 = 9.86 kNm
longitudinal compressive stress in column web σcom,Ed = 21.88 N/mm²

reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: a/t = 5.4 t ≤ 33 → section class 1 ≤ 2 ok
minimum demands of the moment of inertia of stiffeners:
  length of buckling field (distance of stiffeners) a = 172.0 mm
  web height between the flanges hwc = 164.0 mm
  moment of inertia of stiffeners ist = 50.24 cm²
  minimum moment of inertia for a/hwc = 1.05 < 2¹/²: ist, min = 3.33 cm² < ist ok
requirement concerning stiffeners to avoid lateral torsional buckling:
  torsional moment of inertia of stiffeners Itr = 0.73 cm²
  Itr / Iₚ = 0.135 > 0.006 = 5.3 fᵧₛEd / Eₚ ok
resistance of stiffened webs with transverse compression:
  area of stiffeners incl. web Ast = 7.28 cm²
  slenderness λ = 0.066
  λ ≤ 0.2 ⇒ no deduction (γ = 1.0)
  design value of resistance of flexural buckling Fc,W,Rd = 155.5 kN

resistance of upper beam flange:
reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: a/t = 5.4 t ≤ 33 → section class 1 ≤ 2 ok
minimum demands of the moment of inertia of stiffeners:
  length of buckling field (distance of stiffeners) a = 172.0 mm
  web height between the flanges hwc = 164.0 mm
  moment of inertia of stiffeners ist = 50.24 cm²
  minimum moment of inertia for a/hwc = 1.05 < 2¹/²: ist, min = 3.33 cm² < ist ok
requirement concerning stiffeners to avoid lateral torsional buckling:
  torsional moment of inertia of stiffeners Itr = 0.73 cm²
  Itr / Iₚ = 0.135 > 0.006 = 5.3 fᵧₛEd / Eₚ ok
resistance of stiffened webs with transverse compression:
  area of stiffeners incl. web Ast = 7.28 cm²
  slenderness λ = 0.066
  λ ≤ 0.2 ⇒ no deduction (γ = 1.0)
  design value of resistance of flexural buckling Fc,W,Rd = 155.5 kN

3.1.3.3. Gk 4: column flange in bending

equivalent T-stub flange (each individual bolt-row):
  here: number of bolt-rows nₛ = 1
row 1
effective length of the T-stub flange (column flange):
  in mode 1: Sₑᵣ₁ = lₑᵣ₁ = min(lₑᵣₑ, lₑᵣₑ) = 87.0 mm, lₑᵣₑ = 95.2 mm
  in mode 2: Sₑᵣ₂ = lₑᵣ₂ = lₑᵣₑ = 87.0 mm

Only the essential sizes are sketched to scale.
The connection geometry is only hinted.
tension resistance of the T-stub flange:
in mode 1+2: \[ M_{p,Rd} = (0.25 \Xi_{eff} t^2 f_y) / \gamma_{MO} = 0.33 \text{kNm} \]
in mode 3: \[ \Xi_{Fr,Rd} = 2 n_b F_{r,Rd} = 97.11 \text{kN} \]
mode 1: complete yielding of the T-stub flange
\[ F_{t,1,Rd} = ((8 n-2 \omega v) M_{p,1,Rd}) / (2 m-n-\omega v (m+n)) = 123.51 \text{kN} \]
mode 2: bolt failure simultaneously with yielding of the T-stub flange
\[ F_{t,2,Rd} = (2 M_{p,2,Rd} + n \Xi_{Fr,Rd}) / (m+n) = 73.14 \text{kN} \]
mode 3: bolt failure
\[ F_{t,3,Rd} = \Xi_{Fr,Rd} = 97.11 \text{kN} \]
tension resistance of the T-stub flange: \[ F_{t,Rd} = \min(F_{t,1,Rd}, F_{t,2,Rd}, F_{t,3,Rd}) = 73.14 \text{kN} \]

row 2
effective length of the T-stub flange (column flange):
in mode 1: \[ \Xi_{eff,1} = \min (\Xi_{eff,nc}, \Xi_{eff,cp}) = 87.0 \text{mm}, \quad \Xi_{eff,cp} = 95.2 \text{mm} \]
in mode 2: \[ \Xi_{eff,2} = \min (\Xi_{eff,nc}, \Xi_{eff,cp}) = 87.0 \text{mm} \]
tension resistance of the T-stub flange:
in mode 1+2: \[ M_{p,Rd} = (0.25 \Xi_{eff} t^2 f_y) / \gamma_{MO} = 0.33 \text{kNm} \]
in mode 3: \[ \Xi_{Fr,Rd} = 2 n_b F_{r,Rd} = 97.11 \text{kN} \]
mode 1: complete yielding of the T-stub flange
\[ F_{t,1,Rd} = ((8 n-2 \omega v) M_{p,1,Rd}) / (2 m-n-\omega v (m+n)) = 123.51 \text{kN} \]
mode 2: bolt failure simultaneously with yielding of the T-stub flange
\[ F_{t,2,Rd} = (2 M_{p,2,Rd} + n \Xi_{Fr,Rd}) / (m+n) = 73.14 \text{kN} \]
mode 3: bolt failure
\[ F_{t,3,Rd} = \Xi_{Fr,Rd} = 97.11 \text{kN} \]
tension resistance of the T-stub flange: \[ F_{t,Rd} = \min(F_{t,1,Rd}, F_{t,2,Rd}, F_{t,3,Rd}) = 73.14 \text{kN} \]
resistances and effective lengths of column flange in bending (per bolt-row):
\[ F_{t,Rd,1} = 73.14 \text{kN}, \quad \Xi_{eff,1} = 87.0 \text{mm} \]
\[ F_{t,Rd,2} = 73.14 \text{kN}, \quad \Xi_{eff,2} = 87.0 \text{mm} \]
equivalent T-stub flange (group of bolts 1):
here: number of bolt-rows \( n_b = 2 \) (between stiffeners)
effective length of the T-stub flange (column flange):
in mode 1: \[ \Xi_{eff,1} = \min (\Xi_{eff,nc}, \Xi_{eff,cp}) = 187.7 \text{mm}, \quad \Xi_{eff,cp} = 295.2 \text{mm} \]
in mode 2: \[ \Xi_{eff,2} = \min (\Xi_{eff,nc}, \Xi_{eff,cp}) = 187.7 \text{mm} \]
tension resistance of the T-stub flange:
in mode 1+2: \[ M_{p,Rd} = (0.25 \Xi_{eff} t^2 f_y) / \gamma_{MO} = 0.71 \text{kNm} \]
in mode 3: \[ \Xi_{Fr,Rd} = 2 n_b F_{r,Rd} = 194.23 \text{kN} \]
mode 1: complete yielding of the T-stub flange
\[ F_{t,1,Rd} = ((8 n-2 \omega v) M_{p,1,Rd}) / (2 m-n-\omega v (m+n)) = 266.59 \text{kN} \]
mode 2: bolt failure simultaneously with yielding of the T-stub flange
\[ F_{t,2,Rd} = (2 M_{p,2,Rd} + n \Xi_{Fr,Rd}) / (m+n) = 149.31 \text{kN} \]
mode 3: bolt failure
\[ F_{t,3,Rd} = \Xi_{Fr,Rd} = 194.23 \text{kN} \]
tension resistance of the T-stub flange: \[ F_{t,Rd} = \min(F_{t,1,Rd}, F_{t,2,Rd}, F_{t,3,Rd}) = 149.31 \text{kN} \]
resistances and effective lengths of column flange in bending (per bolt group):
\[ F_{\phi,Rd,1,2} = 149.31 \text{kN}, \quad \Xi_{eff} = 187.7 \text{mm}, \quad 2 \text{ rows} \]

3.1.3.4. Gk 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3[9]) \( \beta_1 = 11 \cdot M_{p2}/M_{p1} = 2.00 \) for \( M_{p1} = -5.40 \text{kNm}, \quad M_{p2} = 9.86 \text{kNm} \)

\[ \begin{array}{c}
\text{F1} \\
\text{146} \\
\text{8} \\
\end{array} \]

Each individual bolt-row:
row 1
effective width \( b_{eff,1} = 87.0 \text{mm} \) (left from bc 4)
reinforcement of column web with 1 supplementary web plate:
fillet weld with \( a_2 \geq t_w/2 \) \( = 4.9 \text{mm} \): effective web thickness \( t_{w,eff} = t_{w,c} + 0.4 \cdot t_a = 7.4 \text{mm} \) for S235
reduction factor for interaction with shear stress \( \beta = 2 \Rightarrow \sigma = 0.790 \)
resistance of a column web with transverse tension
\[ F_{t,w,Rd} = \sigma \cdot (b_{eff,1} t_{w,eff}) / \gamma_{MO} = 119.9 \text{kN} \]
row 2
effective width \( b_{eff,2} = 87.0 \text{mm} \) (left from bc 4)
reinforcement of column web with 1 supplementary web plate:
fillet weld with \( t_a \geq 10/21\) = 4.9 mm: effective web thickness \( t_{w,eff} = t_{wc} + 0.4t_a = 7.4 \text{ mm} \) for S235
reduction factor for interaction with shear stress \( \beta = 2 \Rightarrow \varphi = 0.790 \)
resistance of a column web with transverse tension
\[
F_{t,wc,Rd} = \varphi \cdot \left( b_{eff,t} \cdot t_{wc,Rd} \right) / \gamma_{M0} = 119.9 \text{ kN}
\]

**group of bolt-rows, group 1:**
effective width \( b_{eff,t} = 187.7 \text{ mm} \) (left from bc 4)
reinforcement of column web with 1 supplementary web plate:
fillet weld with \( t_a \geq 10/21\) = 4.9 mm: effective web thickness \( t_{w,eff} = t_{wc} + 0.4t_a = 7.4 \text{ mm} \) for S235
reduction factor for interaction with shear stress \( \beta = 2 \Rightarrow \varphi = 0.513 \)
resistance of a column web with transverse tension
\[
F_{t,wc,Rd} = \varphi \cdot \left( b_{eff,t} \cdot t_{wc,Rd} \right) / \gamma_{M0} = 168.0 \text{ kN}
\]

### 3.1.3.5. Gk 5: end-plate in bending

**part of end-plate between beam flanges**

**equivalent T-stub flange (each individual bolt-row):**
- here: number of bolt-rows \( n_0 = 1 \)
- **row 1**
  - effective length of the T-stub flange (end-plate):
    - in mode 1: \( \Sigma_{left,1} = left,1 = \min(left,nc, left,cp) = 102.8 \text{ mm}, \quad left,cp = 119.1 \text{ mm} \)
    - in mode 2: \( \Sigma_{left,2} = left,2 = left,nc = 102.8 \text{ mm} \)
  - tension resistance of the T-stub flange:
    - in mode 1: \( M_{pl,Rd} = (0.25 \cdot \Sigma_{left} \cdot t^2 / l_y) / \gamma_{M0} = 0.60 \text{ kNm} \)
    - in mode 3: \( \Sigma F_{t,Rd} = 2 - nb, F_{t,Rd} = 97.11 \text{ kN} \)
    - mode 1: complete yielding of the T-stub flange
    - mode 2: bolt failure simultaneously with yielding of the T-stub flange
    - mode 3: bolt failure
  - tension resistance of the T-stub flange:
    - \( F_{t,Rd} = \min(F_{t,1,Rd}, F_{t,2,Rd}, F_{t,3,Rd}) = 80.87 \text{ kN} \)
    - resistance of a weld (req.1): \( f_{w,d} = f_u / (\gamma_{w}\gamma_{M2}) = 360.0 \text{ N/mm}^2 \)
    - tension resistance of welds: \( F_{w,Rd} = 21^1/2 \cdot f_{w,d} \cdot a_{left} = 157.04 \text{ kN} (\geq 80.87 \text{ kN, not decisive}) \)
- **row 2**
  - effective length of the T-stub flange (end-plate):
    - in mode 1: \( \Sigma_{left,1} = left,1 = \min(left,nc, left,cp) = 102.8 \text{ mm}, \quad left,cp = 119.1 \text{ mm} \)
    - in mode 2: \( \Sigma_{left,2} = left,2 = left,nc = 102.8 \text{ mm} \)
  - tension resistance of the T-stub flange:
    - in mode 1: \( M_{pl,Rd} = (0.25 \cdot \Sigma_{left} \cdot t^2 / l_y) / \gamma_{M0} = 0.60 \text{ kNm} \)
    - in mode 3: \( \Sigma F_{t,Rd} = 2 - nb, F_{t,Rd} = 97.11 \text{ kN} \)
    - mode 1: complete yielding of the T-stub flange
    - mode 2: bolt failure simultaneously with yielding of the T-stub flange
    - mode 3: bolt failure
  - tension resistance of the T-stub flange:
    - \( F_{t,Rd} = \min(F_{t,1,Rd}, F_{t,2,Rd}, F_{t,3,Rd}) = 80.87 \text{ kN} \)
    - resistance of a weld (req.1): \( f_{w,d} = f_u / (\gamma_{w}\gamma_{M2}) = 360.0 \text{ N/mm}^2 \)
    - tension resistance of welds: \( F_{w,Rd} = 21^1/2 \cdot f_{w,d} \cdot a_{left} = 157.04 \text{ kN} (\geq 80.87 \text{ kN, not decisive}) \)

**resistances and effective lengths of end-plate in bending (per bolt-row):**
- \( F_{ep,Rd,1} = 80.87 \text{ kN}, \quad left,1 = 102.8 \text{ mm} \)
- \( F_{ep,Rd,2} = 80.87 \text{ kN}, \quad left,2 = 102.8 \text{ mm} \)

**equivalent T-stub flange (group of bolts 1):**
- here: number of bolt-rows \( n_0 = 2 \)
- effective length of the T-stub flange (end-plate):
  - in mode 1: \( \Sigma_{left,1} = \min(\Sigma_{left,nc}, \Sigma_{left,cp}) = 204.8 \text{ mm}, \quad left,cp = 319.1 \text{ mm} \)
  - in mode 2: \( \Sigma_{left,2} = \Sigma_{left,nc} = 204.8 \text{ mm} \)
- tension resistance of the T-stub flange:
in mode 1+2: \[ M_{\text{PL,Rd}} = \left( 0.25 \cdot \sum \text{eff} \cdot t^2 \cdot f_y \right) / \gamma_M = 1.20 \text{ kNm} \]
in mode 3: \[ \sum F_{\text{T,Rd}} = 2 \cdot n_b \cdot F_{\text{T,Rd}} = 194.23 \text{ kN} \]
mode 1: complete yielding of the T-stub flange
\[ F_{T,1,Rd} = \left( (8 \cdot n \cdot 2 \cdot a_w) \cdot M_{\text{pl,1,Rd}} / (2 \cdot m \cdot n \cdot a_w \cdot (m+n)) \right) = 340.54 \text{ kN} \]
mode 2: bolt failure simultaneously with yielding of the T-stub flange
\[ F_{T,2,Rd} = \left( 2 \cdot M_{\text{pl,2,Rd}} \right) / (m+n) = 161.49 \text{ kN} \]
mode 3: bolt failure
\[ F_{T,3,Rd} = \sum F_{\text{T,Rd}} = 194.23 \text{ kN} \]
tension resistance of the T-stub flange: \[ F_{T,Rd} = \min( F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd} ) = 161.49 \text{ kN} \]
resistance of a weld (req.1): \[ f_{w,d} = f_u / (\gamma_w \cdot \gamma_M) = 360.0 \text{ N/mm}^2 \]
tension resistance of welds: \[ F_{w,Rd} = 2 \cdot \sum f_{w,d} \cdot a_{\text{eff}} = 312.83 \text{ kN} \geq 161.49 \text{ kN, not decisive} \]

resistances and effective lengths of end-plate in bending (per bolt group):
\[ F_{\text{ep,Rd,1-2}} = 161.49 \text{ kN}, \quad \sum \text{eff} = 204.8 \text{ mm, 2 rows} \]

3.1.3.6. Gk 7: beam flange and web in compression

flange bottom: section class for \( c/(c+t) = 4.23: 1 \)
web: section class for \( \alpha = 0.49 \) and \( c/(c+t) = 27.55: 1 \)
section class of beam: 1

Taking into account the moment-shear force-interaction \( V_{Ed} = 1.6 \text{ kN} \)

\[ \text{stress due to bending with shear force: } V_{Ed} = 1.6 \text{ kN} \leq 76.3 \text{ kN} = V_{pl,Rd} / 2 \Rightarrow \text{no effect} \]
resistance \( M_\text{c,Rd} = M_{\text{pl,Rd}} = (W_{pl} \cdot f_y) / \gamma_M = 39.01 \text{ kNm}, \ W_{pl} = 166.00 \text{ cm}^3 \)
resistance of a flange (and web) with compression
\[ F_{c,t,Rd} = M_{\text{c,Rd}} / (h - t) = 226.80 \text{ kN} \]

resistance of upper beam flange:

stress due to bending with shear force: \( V_{Ed} = 1.6 \text{ kN} \leq 76.3 \text{ kN} = V_{pl,Rd} / 2 \Rightarrow \text{no effect} \)
resistance \( M_\text{c,Rd} = M_{\text{pl,Rd}} = (W_{pl} \cdot f_y) / \gamma_M = 39.01 \text{ kNm}, \ W_{pl} = 166.00 \text{ cm}^3 \)
resistance of a flange (and web) with compression
\[ F_{c,t,Rd} = M_{\text{c,Rd}} / (h - t) = 226.80 \text{ kN} \]

3.1.3.7. Gk 8: beam web in tension

\[ \text{each individual bolt-row:} \]
row 1
effective width \( b_{\text{eff,t,wb}} = 102.8 \text{ mm (left from bc 5)} \)
resistance of a beam web in tension
\[ F_{t,w,Rd} = b_{\text{eff,t,wb}} \cdot f_{w,t,wb} \cdot f_y / \gamma_M = 128.1 \text{ kN} \]
row 2
effective width \( b_{\text{eff,t,wb}} = 102.8 \text{ mm (left from bc 5)} \)
resistance of a beam web in tension
\[ F_{t,w,Rd} = b_{\text{eff,t,wb}} \cdot f_{w,t,wb} \cdot f_y / \gamma_M = 128.1 \text{ kN} \]

group of bolt-rows, group 1:
effective width \( b_{\text{eff,t,wb}} = 204.8 \text{ mm (left from bc 5)} \)
resistance of a beam web in tension
3.1.3.8. Gk 10: bolts in tension

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

\[ F_{L,wb,Rd} = \frac{b_{eff,wb}}{f_{y,wb}} \gamma_{M0} = 255.1 \text{ kN} \]

Tension resistance of one bolt:

\[ F_{L,Rd} = \frac{C_k f_{ub} A_b}{\gamma_{M2}} = 48.56 \text{ kN}, \quad C_k = 0.90 \]

Punching shear load capacity:

\[ F_{p,Rd} = \left( 0.6 \pi d_t^3 f_{ub} \right) = 99.69 \text{ kN}, \quad d_t = 8.0 \text{ mm} \]

Tension-/punching shear load capacity for 2 bolts:

\[ \Sigma F_{p,Rd} = 2 \cdot \min(F_{L,Rd}, F_{p,Rd}) = 97.11 \text{ kN} \]

3.1.3.9. Gk 11: bolts in shear

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

Shear resistance per shear plane:

\[ F_{V,Rd} = \alpha_{v} f_{ub} A_b \gamma_{M2} = 43.43 \text{ kN}, \quad \alpha_{v} = 0.60 \]

Shear resistance of 2 bolts (1-shear):

\[ \Sigma F_{V,Rd} = 2 F_{V,Rd} = 86.86 \text{ kN} \]

3.1.3.10. Gk 12: plate with bearing resistance

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

Row 1

End-plate:

- Bolt 1: Bearing resistance:
  \[ F_{b,Rd} = \frac{(k_1 \cdot c_b \cdot f_{ub} \cdot d_1)}{\gamma_{M2}} = 86.40 \text{ kN}, \quad k_1 = 2.50, \quad c_b = 1.00 \]

- Bolt 2: Bearing resistance:
  \[ F_{b,Rd} = \frac{(k_1 \cdot c_b \cdot f_{ub} \cdot d_1)}{\gamma_{M2}} = 86.40 \text{ kN}, \quad k_1 = 2.50, \quad c_b = 1.00 \]

Bearing resistance of 1x2 bolts:

\[ \Sigma F_{b,Rd} = 172.80 \text{ kN} \]

Column flanges:

- Bolt 1: Bearing resistance:
  \[ F_{b,Rd} = \frac{(k_1 \cdot c_b \cdot f_{ub} \cdot d_1)}{\gamma_{M2}} = 69.12 \text{ kN}, \quad k_1 = 2.50, \quad c_b = 1.00 \]

- Bolt 2: Bearing resistance:
  \[ F_{b,Rd} = \frac{(k_1 \cdot c_b \cdot f_{ub} \cdot d_1)}{\gamma_{M2}} = 69.12 \text{ kN}, \quad k_1 = 2.50, \quad c_b = 1.00 \]

Bearing resistance of 1x2 bolts:

\[ \Sigma F_{b,Rd} = 138.24 \text{ kN} \]

Row 2

End-plate:

- Bolt 1: Bearing resistance:
  \[ F_{b,Rd} = \frac{(k_1 \cdot c_b \cdot f_{ub} \cdot d_1)}{\gamma_{M2}} = 86.40 \text{ kN}, \quad k_1 = 2.50, \quad c_b = 1.00 \]

- Bolt 2: Bearing resistance:
  \[ F_{b,Rd} = \frac{(k_1 \cdot c_b \cdot f_{ub} \cdot d_1)}{\gamma_{M2}} = 86.40 \text{ kN}, \quad k_1 = 2.50, \quad c_b = 1.00 \]

Bearing resistance of 1x2 bolts:

\[ \Sigma F_{b,Rd} = 172.80 \text{ kN} \]

Column flange:

- Bolt 1: Bearing resistance:
  \[ F_{b,Rd} = \frac{(k_1 \cdot c_b \cdot f_{ub} \cdot d_1)}{\gamma_{M2}} = 69.12 \text{ kN}, \quad k_1 = 2.50, \quad c_b = 1.00 \]

- Bolt 2: Bearing resistance:
  \[ F_{b,Rd} = \frac{(k_1 \cdot c_b \cdot f_{ub} \cdot d_1)}{\gamma_{M2}} = 69.12 \text{ kN}, \quad k_1 = 2.50, \quad c_b = 1.00 \]

Bearing resistance of 1x2 bolts:

\[ \Sigma F_{b,Rd} = 138.24 \text{ kN} \]

Row 3

End-plate:
bolt 1: bearing resistance \( F_{b,Rd} = (k_1 \cdot \alphaB \cdot f_u \cdot d_{l}) / \gamma_M = 86.40 \text{ kN} \), \( k_1 = 2.50 \), \( \alphaB = 1.00 \)

bolt 2: bearing resistance \( F_{b,Rd} = (k_1 \cdot \alphaB \cdot f_u \cdot d_{l}) / \gamma_M = 86.40 \text{ kN} \), \( k_1 = 2.50 \), \( \alphaB = 1.00 \)

bearing resistance of 1x2 bolts: \( \Sigma F_{b,Rd} = 172.80 \text{ kN} \)

column flange:

bolt 1: bearing resistance \( F_{b,Rd} = (k_1 \cdot \alphaB \cdot f_u \cdot d_{l}) / \gamma_M = 69.12 \text{ kN} \), \( k_1 = 2.50 \), \( \alphaB = 1.00 \)

bolt 2: bearing resistance \( F_{b,Rd} = (k_1 \cdot \alphaB \cdot f_u \cdot d_{l}) / \gamma_M = 69.12 \text{ kN} \), \( k_1 = 2.50 \), \( \alphaB = 1.00 \)

bearing resistance of 1x2 bolts: \( \Sigma F_{b,Rd} = 138.24 \text{ kN} \)

**bearing resistance (3 rows)**

\( \Sigma F_{b,Rd,1} = 138.24 \text{ kN} \)
\( \Sigma F_{b,Rd,2} = 138.24 \text{ kN} \)
\( \Sigma F_{b,Rd,3} = 138.24 \text{ kN} \)

### 3.1.4. connection capacity

#### 3.1.4.1. moment resistance

distance of tension-bolt-rows from centre of compression: \( h_1 = 136.0 \text{ mm} \), \( h_2 = 36.0 \text{ mm} \)

**resistance per bolt-row (MNV-interaction)**

row 1: \( F_{r,Rd} = 73.1 \text{ kN} \)

row 2: \( F_{r,Rd} = 48.6 \text{ kN} \)

**resistance of flanges (MNV-interaction)**

bottom: \( F_{c,Rd} = 119.8 \text{ kN} \)

**moment resistance (MNV-interaction)**

\( M_{r,Rd} = \sum (F_{r,Rd} \cdot h) = 11.7 \text{ kNm} \)

**shear force resistance (MNV-interaction)**

\( V_{r,Rd} = 3.4 \text{ kN} \)

### 3.1.4.2. shear resistance

**shear resistance of end plate**

end-plate: \( V_{e,Rd} = 198.09 \text{ kN} \)

welds: \( F_{w,Rd} = 182.07 \text{ kN} \)

shear resistance of end plate: \( V_{e,Rd} = F_{w,Rd} = 182.07 \text{ kN} \)

**shear resistance of column web**

\( V_{w,Rd} = 119.8 \text{ kN} \)

### 3.1.4.3. total

\( V_{w,Rd} = 119.8 \text{ kN} \quad V_{e,Rd} = 182.1 \text{ kN} \)

### 3.1.5. verifications

#### 3.1.5.1. verification of the connection capacity by means of the component method

\( U_{MNV} = 0.456 < 1 \quad \text{ok} \)

\( V_{c,Ed}(V_{w,Rd}; h) = 0.982 < 1 \quad \text{ok} \)

\( V_{Ed}V_{e,Rd} = 0.009 < 1 \quad \text{ok} \)

### 3.1.5.2. 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 6,7: beam flange in compression inner

**calculation section:**

weld 1: \( a_w = 3.0 \text{ mm} \quad l_w = 90.0 \text{ mm} \)

weld 2: \( a_w = 3.0 \text{ mm} \quad l_w = 33.4 \text{ mm} \)

weld 3: siehe weld 2

weld 4: \( a_w = 3.0 \text{ mm} \quad l_w = 146.0 \text{ mm} \)

weld 5: siehe weld 4

weld 6: \( a_w = 3.0 \text{ mm} \quad l_w = 33.4 \text{ mm} \)

weld 7: siehe weld 6

weld 8: \( a_w = 3.0 \text{ mm} \quad l_w = 90.0 \text{ mm} \)

**design values referring to centroid of the section:**

\( N_{Ed} = 0.88 \text{ kN} \), \( M_{y,Ed} = -5.26 \text{ kNm} \), \( V_{z,Ed} = 1.57 \text{ kN} \)
cross-sectional properties referring to centroid of the line cross-section:
\[
\Sigma_{Ax} = 18.16 \text{ cm}^2, \quad A_{xw} = 8.76 \text{ cm}^2, \quad \Sigma_{w} = 60.5 \text{ cm}
\]
\[
l_{w} = 862.10 \text{ cm}, \quad l_{wz} = 72.88 \text{ cm}, \quad W_{w1} = 14.27 \text{ cm}^2, \quad \Delta x_w = 0.0 \text{ mm}
\]
distribution of internal forces and moments:
weld 1: \[N_w = 14.96 \text{ kN}\]
weld 2: \[N_w = 5.05 \text{ kN}\]
weld 3: siehe weld 2
cross-sectional properties referring to centroid of the line cross-section:
weld 4: \[N_w = 0.21 \text{ kN} \quad M_{y, w} = -0.47 \text{ kNm}\]
weld 5: siehe weld 4
cross-sectional properties referring to centroid of the line cross-section:
weld 6: \[N_w = -4.96 \text{ kN}\]
weld 7: siehe weld 6
cross-sectional properties referring to centroid of the line cross-section:
weld 8: \[N_w = -14.70 \text{ kN}\]
from conventional distribution of shear force: \[V_{z, w} = 1.57 \text{ kN}\]
verifications in weld edges:
weld 1, pt. 0: \[\sigma_{w, x} = 55.40 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.218 < 1 \text{ ok}\]
weld 2, pt. 0: \[\sigma_{w, x} = 50.52 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.198 < 1 \text{ ok}\]
weld 4, pt. 0: \[\sigma_{w, x} = 45.03 \text{ N/mm}^2 \quad \tau_{w, z} = 1.79 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.177 < 1 \text{ ok}\]
  pt. 1: \[\sigma_{w, x} = 44.06 \text{ N/mm}^2 \quad \tau_{w, z} = 1.79 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.173 < 1 \text{ ok}\]
weld 6, pt. 0: \[\sigma_{w, x} = 49.56 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.195 < 1 \text{ ok}\]
weld 8, pt. 0: \[\sigma_{w, x} = 54.44 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.214 < 1 \text{ ok}\]

Result:
weld 1, pt. 0: \[\sigma_{w, x} = 55.40 \text{ N/mm}^2 \]
Max: \[c_1, w, Ed = 7.84 \text{ kN/cm}^2 \quad < f_{w, d} = 36.00 \text{ kN/cm}^2,\]
\[c_2, w, Ed = 3.92 \text{ kN/cm}^2 \quad < f_{w, d} = 25.92 \text{ kN/cm}^2 \quad \Rightarrow U_w = 0.218 < 1 \text{ ok}\]

3.1.5.3. verification of web stiffeners

composition stiffener
\[F_{Ed, Ed} = 45.50 \text{ kN}\]
forces per rib
\[F = 0.5 \cdot F_{Ed, Ed} \cdot (b_r - 2 \cdot t_w) / b_f = 16.9 \text{ kN}, \quad H = F \cdot \sigma_{w} / \sigma_{w} = 2.9 \text{ kN}\]
assumption: stiffeners do not buckle: \[c_t = 5.4 \pm 3.3 \quad \Rightarrow \text{ section class } 1 \leq 2 \quad \text{ ok}\]
cross-section at flange
compression resistance \[N_{x, Ed, Rd} = (A_{x, Ed} / \gamma_{M0}) = 55.18 \text{ kN}\]
design value: \[F_{Ed} = (F^2 + 3 \cdot H)^{1/2} = 17.7 \text{ kN}\]
\[F_{Ed} = 17.7 \text{ kN} \quad < f_{Ed} = 55.2 \text{ kN} \quad \Rightarrow U = 0.320 < 1 \text{ ok}\]
cross-section at web
shear resistance \[V_{Ed} = 178.01 \text{ kN}\]
design value: \[F_{Ed} = F = 16.9 \text{ kN}\]
\[F_{Ed} = 16.9 \text{ kN} \quad < f_{Ed} = 178.0 \text{ kN} \quad \Rightarrow U = 0.095 < 1 \text{ ok}\]
flange welds
design values: \[F_{Ed} = F / (2 \cdot b_1) = 2.88 \text{ kN/cm}, \quad F_{Ed} = H / (2 \cdot b_1) = 0.50 \text{ kN/cm}, \quad b_1 = 29.4 \text{ mm}\]
\[c_1, w, Ed = 10.03 \text{ kN/cm}^2 \quad < f_{w, d} = 36.00 \text{ kN/cm}^2 \quad \Rightarrow U = 0.279 < 1 \text{ ok}\]
\[c_2, w, Ed = 9.61 \text{ kN/cm}^2 \quad < f_{w, d} = 25.92 \text{ kN/cm}^2 \quad \Rightarrow U = 0.371 < 1 \text{ ok}\]
web welds
design value: \[F_{Ed} = F / (2 \cdot h_1) = 0.62 \text{ kN/cm}, \quad h_1 = 137.0 \text{ mm}\]
\[c_1, w, Ed = 3.57 \text{ kN/cm}^2 \quad < f_{w, d} = 36.00 \text{ kN/cm}^2 \quad \Rightarrow U = 0.099 < 1 \text{ ok}\]
stiffener in tension
\[F_{Ed} = 46.37 \text{ kN}\]
forces per rib
\[F = 0.5 \cdot F_{Ed} \cdot (b_r - 2 \cdot t_w) / b_f = 17.2 \text{ kN}, \quad H = F \cdot \sigma_{w} / \sigma_{w} = 3.0 \text{ kN}\]
cross-section at flange
tension resistance \[N_{x, Ed, Rd} = 55.18 \text{ kN}\]
design value: \[F_{Ed} = (F^2 + 3 \cdot H)^{1/2} = 18.0 \text{ kN}\]
\[F_{Ed} = 18.0 \text{ kN} \quad < f_{Ed} = 55.2 \text{ kN} \quad \Rightarrow U = 0.326 < 1 \text{ ok}\]
cross-section at web
shear resistance \[V_{Ed} = 178.01 \text{ kN}\]
design value: \[F_{Ed} = F = 17.2 \text{ kN}\]
\[F_{Ed} = 17.2 \text{ kN} \quad < f_{Ed} = 178.0 \text{ kN} \quad \Rightarrow U = 0.097 < 1 \text{ ok}\]
flange welds
design values: \[F_{Ed} = F / (2 \cdot b_1) = 2.94 \text{ kN/cm}, \quad F_{Ed} = H / (2 \cdot b_1) = 0.50 \text{ kN/cm}, \quad b_1 = 29.4 \text{ mm}\]
\[c_1, w, Ed = 10.22 \text{ kN/cm}^2 \quad < f_{w, d} = 36.00 \text{ kN/cm}^2 \quad \Rightarrow U = 0.284 < 1 \text{ ok}\]
\[c_2, w, Ed = 9.80 \text{ kN/cm}^2 \quad < f_{w, d} = 25.92 \text{ kN/cm}^2 \quad \Rightarrow U = 0.378 < 1 \text{ ok}\]
web welds
design value: \[F_{Ed} = F / (2 \cdot h_1) = 0.63 \text{ kN/cm}, \quad h_1 = 137.0 \text{ mm}\]
\[c_1, w, Ed = 3.63 \text{ kN/cm}^2 \quad < f_{w, d} = 36.00 \text{ kN/cm}^2 \quad \Rightarrow U = 0.101 < 1 \text{ ok}\]
3.1.5.4. verification result
maximum utilization: max U = 0.982 < 1  ok

3.1.6. rotational stiffness

ingility coefficients

equivalent stiffness coefficient for 2 tension-bolt-rows:
1: \[ k_1 = 3.09 \text{ mm}, \ k_4 = 11.52 \text{ mm}, \ k_5 = 13.59 \text{ mm}, \ k_{10} = 4.12 \text{ mm} \Rightarrow k_{\text{eff,1}} = 1 / (1/k_1, 1) = 1.377 \text{ mm} \]
2: \[ k_2 = 3.09 \text{ mm}, \ k_4 = 11.52 \text{ mm}, \ k_5 = 13.59 \text{ mm}, \ k_{10} = 4.12 \text{ mm} \Rightarrow k_{\text{eff,2}} = 1 / (1/k_1, 2) = 1.377 \text{ mm} \]
\[ k_{eq} = \Sigma (k_{\text{eff,1}} / z_{eq}) = 2.058 \text{ mm}, \ z_{eq} = \Sigma (k_{\text{eff,1}} / z_{eq}) / (k_{\text{eff,1}} / z_{eq}) = 115.1 \text{ mm} \]
\[ k_1 = 0.38 A_{\text{ve}} / (\beta z) = 3.14 \text{ mm} \]
\[ k_2 = \infty \text{ (stiffened)} \]

rotational stiffness

initial rotational stiffness: \[ S_j, \text{ini} = (E z^2) / (\Sigma 1/k) = 3454.4 \text{ kNm/rad}, \ z = z_{eq} = 115.1 \text{ mm}, \ (1/k) = 0.805 \text{ mm}^{-1} \]
\[ N_a, \text{Ed} = 0.88 \text{ kN} < 5\% N_{a, \text{Ed}} = 28.14 \text{ kN} \text{ ok} \]
rotational stiffness: \[ S_j, \text{rd} = S_j, \text{ini} / \mu = 3454.4 \text{ kNm/rad}, \ \mu = 1 \]
rotation: \[ \varrho_{ld, \text{Ed}} = M_{ld, \text{Ed}} / S_j, \text{rd} = 0.089 \text{°} \]

3.2. connection left

notes

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

3.2.1. design values

Knotenschnittgrößen periphery connection L zur connection plane partial internal forces and moments

slope angle: \[ \alpha_b = \alpha_V = \alpha = 0^\circ \]
distance: \[ e_1 = 90.0 \text{ mm}, \ e_3 = 86.0 \text{ mm}, \ e_2 = 86.0 \text{ mm}, \ e_5 = 90.0 \text{ mm}, \ e_7 = 86.0 \text{ mm} \]

internal forces and moments perpendicular to the connection planes

periphery beam (right)
\[ N_a = 7.31 \text{ kN}, \ M_d = 9.31 \text{kNm}, \ V_d = 6.05 \text{kN} \]
periphery beam (left)
\[ N_a = 7.31 \text{ kN}, \ M_d = 9.31 \text{kNm}, \ V_d = 6.05 \text{kN} \]
periphery column (bottom)
\[ N_c = 7.09 \text{ kN}, \ M_c = 8.97 \text{kNm}, \ V_c = -13.65 \text{kN} \]
periphery column (top)
\[ N_c = 7.09 \text{ kN}, \ M_c = 8.97 \text{kNm}, \ V_c = -13.65 \text{kN} \]

partial internal forces and moments

internal forces and moments in the periphery end-plate-beam: \[ M_d = M_d - V_a \text{tep} = 9.27 \text{kNm} \]
\[ N_{b,t} = -N_d z_{bd}/zb + M_d z_{b} = 50.22 \text{kN}, \ z_b = 172.0 \text{ mm}, \ z_{bd} = 86.0 \text{ mm} \]
\[ N_{b,c} = N_d z_{bd}/zb + M_d z_{b} = 57.53 \text{kN}, \ z_b = 172.0 \text{ mm}, \ z_{bd} = 86.0 \text{ mm} \]

3.2.2. resistance of cross-section

column bottom

plastic cross-sectional check for \[ N = -7.09 \text{kN}, \ M_y = -9.87 \text{kNm}, \ V_z = -13.65 \text{kN} \]
valid normal/shear stress: \[ z_{ul} = 23.50 \text{kN}/cm^2, \ z_{ul, \text{rd}} = 13.57 \text{kN/cm}^2 \]
top flange: resistance forces \[ N_{\text{max},o} = 171.08 \text{kN}, \ N_{\text{min},o} = -171.08 \text{kN} \]
bottom flange: resistance forces \[ N_{\text{max},u} = 171.08 \text{kN}, \ N_{\text{min},u} = -171.08 \text{kN} \]
web: shear force \[ V_s = -13.65 \text{kN}, \ shear \text{ stress} \[ t_s = 1.50 \text{kN/cm}^2 \Rightarrow u_{rs} = 0.110 \]
resistance forces \[ N_{\text{max},s} = 212.92 \text{kN}, \ N_{\text{min},s} = -212.92 \text{kN} \]
main bending: axial force \[ N = -7.09 \text{kN}, \ resistance forces \[ N_{\text{max}} = 555.08 \text{kN}, \ N_{\text{min}} = -555.08 \text{kN} \Rightarrow U_n = 0.013 \]
moment \[ M_y = -9.87 \text{kNm}, \ resistance \text{ moments} \[ M_{\text{max}} = 38.57 \text{kNm}, \ M_{\text{min}} = -38.57 \text{kNm} \Rightarrow U_{my} = 0.256 \]
total (possibly due to load increase): max U = 0.263 < 1  ok
utilizations: \[ u_{as} = 0.263 < 1 \text{ ok}, \ c/t-ratio \[ u_{ct} = 0.164 < 1 \text{ ok} \]

column top

plastic cross-sectional check for \[ N = -2.61 \text{kN}, \ M_y = 3.75 \text{kNm}, \ V_z = -5.46 \text{kN} \]
valid normal/shear stress: \[ z_{ul} = 23.50 \text{kN/cm}^2, \ z_{ul, \text{rd}} = 13.57 \text{kN/cm}^2 \]
top flange: resistance forces \[ N_{\text{max},o} = 171.08 \text{kN}, \ N_{\text{min},o} = -171.08 \text{kN} \]
bottom flange: resistance forces \[ N_{\text{max},u} = 171.08 \text{kN}, \ N_{\text{min},u} = -171.08 \text{kN} \]
web: shear force $V_{S} = -5.46$ kN, shear stress $\tau_{S} = 0.60$ kN/cm² $\Rightarrow U_{r,S} = 0.044$
resistance forces $N_{\text{max},s} = 214.02$ kN, $N_{\text{min},s} = -214.02$ kN
main bending: axial force $N = -2.61$ kN, resistance forces $N_{\text{max}} = 556.18$ kN, $N_{\text{min}} = -556.18$ kN $\Rightarrow U_{N} = 0.005$
moment $M_{y} = 3.75$ kNm, resistance moments $M_{y,\text{max}} = 38.63$ kNm, $M_{y,\text{min}} = -38.63$ kNm $\Rightarrow U_{M_y} = 0.097$
total (possibly due to load increase): max $U = 0.100 < 1$ ok
utilizations: resistance $U_{c} = 0.100 < 1$ ok, c/t-ratio $U_{c,t} = 0.101 < 1$ ok
beam
plastic cross-sectional check for $N = -7.31$ kN, $M_{y} = -9.27$ kNm, $V_{2} = 6.05$ kN
valid normal/shear stress: $zul \sigma_{Rd} = 23.50$ kN/cm², $zul \tau_{Rd} = 13.57$ kN/cm²
top flange: resistance forces $N_{\text{max},o} = 171.08$ kN, $N_{\text{min},o} = -171.08$ kN
bottom flange: resistance forces $N_{\text{max},u} = 171.08$ kN, $N_{\text{min},u} = -171.08$ kN
web: shear force $V_{S} = 6.05$ kN, shear stress $\tau_{S} = 0.68$ kN/cm² $\Rightarrow U_{r,S} = 0.049$
resistance forces $N_{\text{max},s} = 213.97$ kN, $N_{\text{min},s} = -213.97$ kN
main bending: axial force $N = -7.31$ kN, resistance forces $N_{\text{max}} = 556.13$ kN, $N_{\text{min}} = -556.13$ kN $\Rightarrow U_{N} = 0.013$
moment $M_{y} = -9.27$ kNm, resistance moments $M_{y,\text{max}} = 38.62$ kNm, $M_{y,\text{min}} = -38.62$ kNm $\Rightarrow U_{M_y} = 0.240$
total (possibly due to load increase): max $U = 0.243 < 1$ ok
utilizations: resistance $U_{c} = 0.243 < 1$ ok, c/t-ratio $U_{c,t} = 0.160 < 1$ ok

3.2.3. basic components
3.2.3.1. Gk 1: Column web panel in shear
transformation parameter (EC 3-1-8, 5.3(9)): $\beta_{1} = 11 - |M_{2}|/|M_{1}| = 1.55$ for $M_{1} = 9.86$ kNm, $M_{2} = -5.40$ kNm


web thickness incl. reinforcement $t_{wc} = 10.6$ mm
slenderness of column web $d_{c}/t_{wc} = 13.77 < 69.6 = 69.00 \Rightarrow$ method applicable
shear area with reinforcement $A_{v} = 18.99$ cm²
plastic shear resistance without stiffeners $V_{wp,Rd} = (0.9 f_{y,w} A_{v}) / (3^{1/2} \gamma_{Mo}) = 231.9$ kN
placing of intermediate web stiffeners:
additional resistance $V_{wp,add,Rd} = 4 M_{pl,tc,Rd}/d_{st} = 8.0$ kN
$V_{wp,add,Rd} > 2 \cdot (M_{pl,tc,Rd} + M_{pl,st,Rd})/d_{st} = 7.7$ kN $\Rightarrow V_{wp,add,Rd} = 7.7$ kN
plastic shear resistance with transverse stiffeners $V_{wp,Rd} = 239.6$ kN

3.2.3.2. Gk 2: column web in transverse compression
transformation parameter (EC 3-1-8, 5.3(9)): $\beta_{1} = 11 - |M_{2}|/|M_{1}| = 1.55$ for $M_{1} = 9.86$ kNm, $M_{2} = -5.40$ kNm
longitudinal compressive stress in column web $\sigma_{com,Ed} = 57.65$ N/mm²
reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: $c/t = 5.4 < 33 < \infty \Rightarrow$ section class 1 ≤ 2 ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) $a = 172.0$ mm
web height between the flanges $h_{wc} = 184.0$ mm
moment of inertia of stiffeners $I_{st} = 50.24$ cm⁴
minimum moment of inertia for $a/h_{wc} = 1.05 < 21/2$: $I_{st,min} = 3.33$ cm⁴ < $I_{st}$ ok
requirement concerning stiffeners to avoid lateral torsional buckling:
torsional moment of inertia of stiffeners $I_{tr} = 0.73$ cm⁴
polar moment of inertia of stiffeners $I_{p} = 5.43$ cm⁴
\[ \frac{\text{It}}{I_p} = 0.135 > 0.006 = 5.3 \frac{f_{yw}}{E_d} \text{ ok} \]

resistance of stiffened webs with transverse compression:
- area of stiffeners incl. web \( A_{st} = 7.28 \text{ cm}^2 \)
- slenderness \( \lambda = 0.066 \)
- \( \lambda \leq 0.2 \Rightarrow \) no deduction (\( \gamma_c = 1.0 \))
- design value of resistance of flexural buckling \( F_{C_{w,Rd}} = 155.5 \text{ kN} \)

**resistance of upper beam flange:**
- reinforcement of web with transverse stiffeners:
  - assumption: stiffeners do not buckle: \( \sigma_t = 5.4 \leq 33 < \) section class 1 \( \leq 2 \text{ ok} \)
  - minimum demands of the moment of inertia of stiffeners:
    - length of buckling field (distance of stiffeners) \( a = 172.0 \text{ mm} \)
    - web height between the flanges \( h_{wc} = 164.0 \text{ mm} \)
    - moment of inertia of stiffeners \( I_{st} = 50.24 \text{ cm}^4 \)
    - requirement concerning stiffeners to avoid lateral torsional buckling:
      - torsional moment of inertia of stiffeners \( It = 0.73 \text{ cm}^4 \)
      - polar moment of inertia of stiffeners \( I_p = 5.43 \text{ cm}^4 \)
  - \( \frac{I_t}{I_p} = 0.135 > 0.006 = 5.3 \frac{f_{yw}}{E_d} \text{ ok} \)

resistance of stiffened webs with transverse compression:
- area of stiffeners incl. web \( A_{st} = 7.28 \text{ cm}^2 \)
- slenderness \( \lambda = 0.066 \)
- \( \lambda \leq 0.2 \Rightarrow \) no deduction (\( \gamma_c = 1.0 \))
- design value of resistance of flexural buckling \( F_{C_{w,Rd}} = 155.5 \text{ kN} \)

### 3.2.3.3. Gk 4: column flange in bending

**equivalent T-stub flange (each individual bolt-row):**
- here: number of bolt-rows \( n_b = 1 \)

**row 1**
- effective length of the T-stub flange (column flange):
  - in mode 1: \( S_{left,1} = left,1 = min(left,nc, left,cp) = 87.0 \text{ mm} \), \( left,cp = 95.2 \text{ mm} \)
  - in mode 2: \( S_{left,2} = left,2 = left,nc = 87.0 \text{ mm} \)
- tension resistance of the T-stub flange:
  - in mode 1+2: \( M_{pl,Rd} = \frac{(0.25) S_{left} f_l t_f l_f}{\gamma_{Mo}} = 0.33 \text{ kNm} \)
  - in mode 3: \( S_{Fl,Rd} = 2n_b F_{t,Rd} = 97.11 \text{ kN} \)
  - mode 1: complete yielding of the T-stub flange
    - \( F_{t,1,Rd} = \frac{(8 - 2 - 2\omega_v) M_{pl,1,Rd}}{(2 - m - n - 2\omega_v - (m + n))} = 123.51 \text{ kN} \)
  - mode 2: bolt failure simultaneously with yielding of the T-stub flange
    - \( F_{t,2,Rd} = (2 M_{pl,2,Rd} + n_\omega S_{Fl,Rd}) / (m + n) = 73.14 \text{ kN} \)
  - mode 3: bolt failure
    - \( F_{t,3,Rd} = \Sigma F_{n,Rd} = 97.11 \text{ kN} \)
- tension resistance of the T-stub flange:
  - \( F_{t,Rd} = min(F_{t,1,Rd, f_{t,2,Rd}, f_{t,3,Rd}}) = 73.14 \text{ kN} \)

**row 2**
- effective length of the T-stub flange (column flange):
  - in mode 1: \( S_{left,1} = left,1 = min(left,nc, left,cp) = 87.0 \text{ mm} \), \( left,cp = 95.2 \text{ mm} \)
  - in mode 2: \( S_{left,2} = left,2 = left,nc = 87.0 \text{ mm} \)
- tension resistance of the T-stub flange:
  - in mode 1+2: \( M_{pl,Rd} = \frac{(0.25) S_{left} f_l t_f l_f}{\gamma_{Mo}} = 0.33 \text{ kNm} \)
  - in mode 3: \( S_{Fl,Rd} = 2n_b F_{t,Rd} = 97.11 \text{ kN} \)
  - mode 1: complete yielding of the T-stub flange
    - \( F_{t,1,Rd} = \frac{(8 - 2 - 2\omega_v) M_{pl,1,Rd}}{(2 - m - n - 2\omega_v - (m + n))} = 123.51 \text{ kN} \)
  - mode 2: bolt failure simultaneously with yielding of the T-stub flange
    - \( F_{t,2,Rd} = (2 M_{pl,2,Rd} + n_\omega S_{Fl,Rd}) / (m + n) = 73.14 \text{ kN} \)
  - mode 3: bolt failure
    - \( F_{t,3,Rd} = \Sigma F_{n,Rd} = 97.11 \text{ kN} \)
- tension resistance of the T-stub flange:
  - \( F_{t,Rd} = min(F_{t,1,Rd, f_{t,2,Rd}, f_{t,3,Rd}}) = 73.14 \text{ kN} \)

**resistances and effective lengths of column flange in bending (per bolt-row)**
- \( F_{Fl,Rd,1} = 73.14 \text{ kN} \), \( left,1 = 87.0 \text{ mm} \)
\[ F_{t,1,Rd} = 73.14 \text{kN}, \quad l_{\text{eff},2} = 87.0 \text{mm} \]

equivalent T-stub flange (group of bolts 1):

\[
\text{effective length of the T-stub flange (column flange)}:
\begin{align*}
\text{mode 1: } & \quad \ell_{\text{eff},1} = \min(\ell_{\text{eff},nc}, \ell_{\text{eff},cp}) = 187.7 \text{ mm}, \quad \ell_{\text{eff},cp} = 295.2 \text{ mm} \\
\text{mode 2: } & \quad \ell_{\text{eff},2} = \ell_{\text{eff},nc} = 187.7 \text{ mm}
\end{align*}
\]

tension resistance of the T-stub flange:

\[
\begin{align*}
\text{mode 1: } & \quad M_{pl,1,Rd} = (0.25 \ell_{\text{eff},2} f_y) / \gamma_{M0} = 0.71 \text{kNm} \\
\text{mode 2: } & \quad b \text{ bolt failure simultaneously with yielding of the T-stub flange} \\
\text{mode 3: } & \quad F_{t,1,Rd} = 2 \frac{n_b}{n_b} F_{t,Rd} = 194.23 \text{kN}
\end{align*}
\]

3.2.3.4. Gk 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3.9) \(\beta_1 = 11 - M_2/M_1\) = 1.55 for \(M_1 = 9.86 \text{kNm}, M_2 = -5.40 \text{kNm}\)

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

each individual bolt-row:

row 1

effective width \(b_{\text{eff},1} = 87.0 \text{ mm} \) (\(l_{\text{eff}}\) from bc 4)

reinforcement of column web with 1 supplementary web plate:

fillt weld with \(a_1 \geq t_w/2^{1/2} = 4.9 \text{ mm} \): effective web thickness \(t_{w,\text{eff}} = t_{w} + 0.4 t_a = 7.4 \text{ mm} \) for S235

reduction factor for interaction with shear stress \(1 < \beta < 2 \Rightarrow \omega = 0.855\)

resistance of a column web with transverse tension

\[ F_{t,wc,Rd} = \omega \cdot (b_{\text{eff},1} t_{w,wc} f_y) / \gamma_{M0} = 129.6 \text{kN} \]

column web, group 1:

effective width \(b_{\text{eff},1} = 187.7 \text{ mm} \) (\(l_{\text{eff}}\) from bc 4)

reinforcement of column web with 1 supplementary web plate:

fillt weld with \(a_2 \geq t_w/2^{1/2} = 4.9 \text{ mm} \): effective web thickness \(t_{w,\text{eff}} = t_{w} + 0.4 t_a = 7.4 \text{ mm} \) for S235

reduction factor for interaction with shear stress \(1 < \beta < 2 \Rightarrow \omega = 0.628\)

resistance of a column web with transverse tension

\[ F_{t,wc,Rd} = \omega \cdot (b_{\text{eff},1} t_{w,wc} f_y) / \gamma_{M0} = 205.5 \text{kN} \]

3.2.3.5. Gk 5: end-plate in bending
part of end-plate between beam flanges

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

**row 1**

1. effective length of the T-stub flange (end-plate):
   - in mode 1: \(a_{\text{eff},1} = a_{\text{eff},1} = \min(a_{\text{eff},\text{nc}}, a_{\text{eff},\text{cp}}) = 102.8\ mm, \ a_{\text{eff},\text{cp}} = 119.1\ mm\)
   - in mode 2: \(a_{\text{eff},2} = a_{\text{eff},2} = a_{\text{eff},\text{nc}} = 102.8\ mm\)

2. tension resistance of the T-stub flange:
   - in mode 1 + 2: \(M_{p,\text{rd}} = (0.25 \times \alpha_{\text{eff}} f_{y}) / \gamma_{M0} = 0.39\ kNm\)
   - in mode 3: \(\Sigma F_{\text{tr},\text{rd}} = 2 n_{b} F_{\text{te},\text{rd}} = 97.11\ kN\)
   - mode 1: complete yielding of the T-stub flange
   \[F_{\text{rd},1} = ((8 \times n_{b}) M_{p,\text{rd}}) / ((2 \times n_{b} \times \omega_{b} (m+n)) = 109.09\ kN\]
   - mode 2: bolt failure simultaneously with yielding of the T-stub flange
   \[F_{\text{rd},2} = (2 M_{p,\text{rd}} + n \times \Sigma F_{\text{tr},\text{rd}}) / (m+n) = 69.71\ kN\]
   - mode 3: bolt failure
   \[F_{\text{rd},3} = \Sigma F_{\text{tr},\text{rd}} = 97.11\ kN\]

3. tension resistance of the T-stub flange: \(F_{T,\text{rd}} = \min(F_{T,1,\text{rd}}, F_{T,2,\text{rd}}, F_{T,3,\text{rd}}) = 69.71\ kN\)

4. resistance of a weld (req. 1): \(f_{w,d} = f_{u} / (f_{w} \gamma M2) = 360.0\ N/mm^{2}\)

5. tension resistance of the welds: \(F_{T,w,\text{rd}} = 21^{1/2} f_{w,d} a_{\text{eff}} = 157.04\ kN (\geq 69.71\ kN, \text{not decisive})\)

**row 2**

1. effective length of the T-stub flange (end-plate):
   - in mode 1: \(a_{\text{eff},1} = a_{\text{eff},1} = \min(a_{\text{eff},\text{nc}}, a_{\text{eff},\cp}) = 102.8\ mm, \ a_{\text{eff},\cp} = 119.1\ mm\)
   - in mode 2: \(a_{\text{eff},2} = a_{\text{eff},2} = a_{\text{eff},\text{nc}} = 102.8\ mm\)

2. tension resistance of the T-stub flange:
   - in mode 1 + 2: \(M_{p,\text{rd}} = (0.25 \times \alpha_{\text{eff}} f_{y}) / \gamma_{M0} = 0.39\ kNm\)
   - in mode 3: \(\Sigma F_{\text{tr},\text{rd}} = 2 n_{b} F_{\text{te},\text{rd}} = 97.11\ kN\)
   - mode 1: complete yielding of the T-stub flange
   \[F_{\text{rd},1} = ((8 \times n_{b}) M_{p,\text{rd}}) / ((2 \times n_{b} \times \omega_{b} (m+n)) = 109.09\ kN\]
   - mode 2: bolt failure simultaneously with yielding of the T-stub flange
   \[F_{\text{rd},2} = (2 M_{p,\text{rd}} + n \times \Sigma F_{\text{tr},\text{rd}}) / (m+n) = 69.71\ kN\]
   - mode 3: bolt failure
   \[F_{\text{rd},3} = \Sigma F_{\text{tr},\text{rd}} = 97.11\ kN\]

3. tension resistance of the T-stub flange: \(F_{T,\text{rd}} = \min(F_{T,1,\text{rd}}, F_{T,2,\text{rd}}, F_{T,3,\text{rd}}) = 69.71\ kN\)

4. resistance of a weld (req. 1): \(f_{w,d} = f_{u} / (f_{w} \gamma M2) = 360.0\ N/mm^{2}\)

5. tension resistance of the welds: \(F_{T,w,\text{rd}} = 21^{1/2} f_{w,d} a_{\text{eff}} = 157.04\ kN (\geq 69.71\ kN, \text{not decisive})\)

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

\(F_{\text{ep},\text{rd},1} = 69.71\ kN, \ a_{\text{eff},1} = 102.8\ mm\)

\(F_{\text{ep},\text{rd},2} = 69.71\ kN, \ a_{\text{eff},2} = 102.8\ mm\)

**equivalent T-stub flange (group of bolts 1):**

**row 1**

1. effective length of the T-stub flange (end-plate):
   - in mode 1: \(a_{\text{eff},1} = a_{\text{eff},1} = \min(a_{\text{eff},\text{nc}}, a_{\text{eff},\cp}) = 204.8\ mm, \ a_{\text{eff},\cp} = 319.1\ mm\)
   - in mode 2: \(a_{\text{eff},2} = a_{\text{eff},2} = a_{\text{eff},\text{nc}} = 204.8\ mm\)

2. tension resistance of the T-stub flange:
   - in mode 1 + 2: \(M_{p,\text{rd}} = (0.25 \times \alpha_{\text{eff}} f_{y}) / \gamma_{M0} = 0.77\ kNm\)
   - in mode 3: \(\Sigma F_{\text{tr},\text{rd}} = 2 n_{b} F_{\text{te},\text{rd}} = 194.23\ kN\)
   - mode 1: complete yielding of the T-stub flange
   \[F_{\text{rd},1} = ((8 \times n_{b}) M_{p,\text{rd}}) / ((2 \times n_{b} \times \omega_{b} (m+n)) = 217.30\ kN\]
   - mode 2: bolt failure simultaneously with yielding of the T-stub flange
   \[F_{\text{rd},2} = (2 M_{p,\text{rd}} + n \times \Sigma F_{\text{tr},\text{rd}}) / (m+n) = 139.25\ kN\]
   - mode 3: bolt failure
   \[F_{\text{rd},3} = \Sigma F_{\text{tr},\text{rd}} = 194.23\ kN\]

3. tension resistance of the T-stub flange: \(F_{T,\text{rd}} = \min(F_{T,1,\text{rd}}, F_{T,2,\text{rd}}, F_{T,3,\text{rd}}) = 139.25\ kN\)

4. resistance of a weld (req. 1): \(f_{w,d} = f_{u} / (f_{w} \gamma M2) = 360.0\ N/mm^{2}\)

5. tension resistance of the welds: \(F_{T,w,\text{rd}} = 21^{1/2} f_{w,d} a_{\text{eff}} = 312.83\ kN (\geq 139.25\ kN, \text{not decisive})\)

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

\(F_{\text{ep},\text{rd},1-2} = 139.25\ kN, \ a_{\text{eff},1-2} = 204.8\ mm, 2\ rows\)
3.2.3.6. Gk 7: beam flange and web in compression

flange bottom: section class for $d/(c+t) = 4.23: 1$
web: section class for $a = 0.53$ and $d/(c+t) = 27.55: 1$
section class of beam: 1

taking into account the moment-shear force-interaction $V_{Ed} = 6.1\, \text{kN}$

stress due to bending with shear force: $V_{Ed} = 6.1\, \text{kN} \leq 76.3\, \text{kN} = V_{Pl,Rd}/2 \Rightarrow \text{no effect}$
resistance $M_{c, Rd} = M_{Pl,Rd} = (W_{Pl,fy}) / \gamma_{Mo} = 39.01\, \text{kNm}, \ W_{Pl} = 166.00\, \text{cm}^3$

resistance of a flange (and web) with compression
$F_{c,t,Rd} = M_{c,Rd} / (h - t) = 226.80\, \text{kN}$

resistance of upper beam flange:
stress due to bending with shear force: $V_{Ed} = 6.1\, \text{kN} \leq 76.3\, \text{kN} = V_{Pl,Rd}/2 \Rightarrow \text{no effect}$
resistance $M_{c, Rd} = M_{Pl,Rd} = (W_{Pl,fy}) / \gamma_{Mo} = 39.01\, \text{kNm}, \ W_{Pl} = 166.00\, \text{cm}^3$

resistance of a flange (and web) with compression
$F_{c,t,Rd} = M_{c,Rd} / (h - t) = 226.80\, \text{kN}$

3.2.3.7. Gk 8: beam web in tension

each individual bolt-row:
row 1
effective width $b_{eff,t, wb} = 102.8\, \text{mm}$ (left from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t, wb} f_{t,wb,wb} / \gamma_{Mo} = 128.1\, \text{kN}$
row 2
effective width $b_{eff,t, wb} = 102.8\, \text{mm}$ (left from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t, wb} f_{t,wb,wb} / \gamma_{Mo} = 128.1\, \text{kN}$

group of bolt-rows, group 1:
effective width $b_{eff,t, wb} = 204.8\, \text{mm}$ (left from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t, wb} f_{t,wb,wb} / \gamma_{Mo} = 255.1\, \text{kN}$

3.2.3.8. Gk 10: bolts in tension
tension resistance of one bolt \( F_{t,Rd} = \frac{(k_2 \cdot f_{ub} \cdot A_d)}{\gamma M_2} \), \( k_2 = 0.90 \)
punching shear load capacity \( B_{p,Rd} = \frac{0.6 \cdot \pi \cdot d_m \cdot f_p}{\gamma M_2} \), \( t_p = 8.0 \ mm \)
tension-punching shear load capacity for 2 bolts: \( \Sigma F_{p,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 97.11 \ kN \)

3.2.3.9. Gk 11: bolts in shear

shear resistance per shear plane \( F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma M_2} \), \( \alpha_v = 0.60 \)
shear resistance of 2 bolts (1-shear): \( \Sigma F_{v,Rd} = 2 \cdot F_{v,Rd} = 86.86 \ kN \)

3.2.3.10. Gk 12: plate with bearing resistance

row 1
end-plate:
- bolt 1: bearing resistance \( F_{b,Rd} = \frac{(k_1 \cdot \alpha_b f_{ub} \cdot d_1)}{\gamma M_2} \), \( k_1 = 2.50, \alpha_b = 1.00 \)
- bolt 2: bearing resistance \( F_{b,Rd} = \frac{(k_1 \cdot \alpha_b f_{ub} \cdot d_1)}{\gamma M_2} \), \( k_1 = 2.50, \alpha_b = 1.00 \)

bearing resistance of 1x2 bolts: \( \Sigma F_{b,Rd} = 138.24 \ kN \)

column flange:
- bolt 1: bearing resistance \( F_{b,Rd} = \frac{(k_1 \cdot \alpha_b f_{ub} \cdot d_1)}{\gamma M_2} \), \( k_1 = 2.50, \alpha_b = 1.00 \)
- bolt 2: bearing resistance \( F_{b,Rd} = \frac{(k_1 \cdot \alpha_b f_{ub} \cdot d_1)}{\gamma M_2} \), \( k_1 = 2.50, \alpha_b = 1.00 \)

bearing resistance of 1x2 bolts: \( \Sigma F_{b,Rd} = 138.24 \ kN \)

row 2
end-plate:
- bolt 1: bearing resistance \( F_{b,Rd} = \frac{(k_1 \cdot \alpha_b f_{ub} \cdot d_1)}{\gamma M_2} \), \( k_1 = 2.50, \alpha_b = 1.00 \)
- bolt 2: bearing resistance \( F_{b,Rd} = \frac{(k_1 \cdot \alpha_b f_{ub} \cdot d_1)}{\gamma M_2} \), \( k_1 = 2.50, \alpha_b = 1.00 \)

bearing resistance of 1x2 bolts: \( \Sigma F_{b,Rd} = 138.24 \ kN \)

bearing resistance (2 rows):
- \( \Sigma F_{b,Rd,1} = 138.24 \ kN \)
- \( \Sigma F_{b,Rd,2} = 138.24 \ kN \)
3.2.4. connection capacity

3.2.4.1. moment resistance

distance of tension-bolt-rows from centre of compression: \( h_1 = 136.0 \text{ mm}, \ h_2 = 36.0 \text{ mm} \)

**resistance per bolt-row (MNV-interaction)**
row 1: \( F_{Tr, Rd} = 69.7 \text{ kN} \)
row 2: \( F_{Tr, Rd} = 69.5 \text{ kN} \)

**resistance of flanges (MNV-interaction)**
bottom: \( F_{c, Rd} = 149.3 \text{ kN} \)

**moment resistance (MNV-interaction)**
\( M_{b, Rd} = \sqrt{F_{Tr, Rd} h_1} = 12.0 \text{ kNm} \)

**shear force resistance (MNV-interaction)**
\( V_{i, Rd} = 8.3 \text{ kN} \)

3.2.4.2. shear resistance

**shear resistance of end plate**
end-plate: \( V_{ep, Rd} = 158.47 \text{ kN} \)
welds: \( F_{W, Rd} = 182.07 \text{ kN} \ (\geq 158.47 \text{ kN}, \text{ not decisive}) \)

**shear resistance of column web**
\( V_{wp, Rd}/[\beta] = 154.8 \text{ kN} \)

3.2.4.3. total
\( V_{wp, Rd}/[\beta] = 154.8 \text{ kN} \quad V_{ep, Rd} = 158.5 \text{ kN} \)

3.2.5. verifications

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

\( U_{MNV} = 0.725 < 1 \quad \text{ok} \)
\( V_{c, Ed}/V_{wp, Rd}/[\beta] = 0.760 < 1 \quad \text{ok} \)
\( V_{Ed}/V_{ep, Rd} = 0.038 < 1 \quad \text{ok} \)

3.2.5.2. verification of welds at beam section

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

calculation section:

<table>
<thead>
<tr>
<th>Weld</th>
<th>( \delta_w )</th>
<th>( l_w )</th>
</tr>
</thead>
<tbody>
<tr>
<td>weld 1</td>
<td>3.0 mm</td>
<td>90.0 mm</td>
</tr>
<tr>
<td>weld 2</td>
<td>3.0 mm</td>
<td>33.4 mm</td>
</tr>
<tr>
<td>weld 3</td>
<td>siehe weld 2</td>
<td></td>
</tr>
<tr>
<td>weld 4</td>
<td>3.0 mm</td>
<td>146.0 mm</td>
</tr>
<tr>
<td>weld 5</td>
<td>siehe weld 4</td>
<td></td>
</tr>
<tr>
<td>weld 6</td>
<td>3.0 mm</td>
<td>33.4 mm</td>
</tr>
<tr>
<td>weld 7</td>
<td>siehe weld 6</td>
<td></td>
</tr>
<tr>
<td>weld 8</td>
<td>3.0 mm</td>
<td>90.0 mm</td>
</tr>
</tbody>
</table>

design values referring to centroid of the section:
\( N_{Ed} = -7.31 \text{ kN}, \ M_{y, Ed} = -9.31 \text{ kNm}, \ V_{x, Ed} = 6.05 \text{ kN} \)

cross-sectional properties referring to centroid of the line cross-section:
\( \Sigma A_w = 18.16 \text{ cm}^2, \ A_{w,z} = 8.76 \text{ cm}^2, \ I_{w,y} = 60.5 \text{ cm} \)
\( I_{w,x} = 86.12 \text{ cm}^4, \ I_{w,z} = 72.88 \text{ cm}^4, \ W_{w,1} = 14.27 \text{ cm}^2, \ \Delta x = 0.0 \text{ mm} \)

distribution of internal forces and moments:

weld 1: \( N_w = 25.17 \text{ kN} \)
weld 2: \( N_w = 8.46 \text{ kN} \)
weld 3: siehe weld 2
weld 4: \( N_w = -1.76 \text{ kN} \quad M_{y,w} = -0.84 \text{ kNm} \)
weld 5: siehe weld 4
weld 6: \( N_w = -9.27 \text{ kN} \)
weld 7: siehe weld 6
weld 8: \( N_w = -27.34 \text{ kN} \)

from conventional distribution of shear force: \( V_{x,w} = 6.05 \text{ kN} \)
3.2.5.3. verification of web stiffeners

compression stiffener

\[ F_{c,Ed} = 82.79 \text{kN} \]

forces per rib

\[ F = 0.5 \cdot F_{c,Ed} \cdot (\text{br} - 2 \cdot r_{tw})/d = 30.8 \text{kN}, \quad H = F \cdot \varepsilon/\varepsilon_H = 5.3 \text{kN} \]

assumption: stiffeners do not buckle: \( c/t = 5.4 < 33 \% \Rightarrow \) section class 1 ≤ 2 ok

cross-section at flange

compression resistance \( N_{c,Ed} = (A_f y_f) / \gamma_{MO} = 55.18 \text{kN} \)

design value: \( F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 32.1 \text{kN} \)

\[ F_{Ed} = 32.1 \text{kN} < F_{Rd} = 55.2 \text{kN} \Rightarrow U = 0.582 < 1 \text{ ok} \]

cross-section at web

shear resistance \( V_{Rd} = 178.01 \text{kN} \)

design value: \( F_{Ed} = F = 30.8 \text{kN} \)

\[ F_{Ed} = 30.8 \text{kN} < F_{Rd} = 178.0 \text{kN} \Rightarrow U = 0.179 < 1 \text{ ok} \]

flange welds

design values: \( F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 5.25 \text{kN/cm}^2, \quad F_{Ed}(\sigma_p) = H / (2 \cdot b_1) = 0.90 \text{kN/cm}, \quad b_1 = 29.4 \text{mm} \)

\[ \sigma_{1,Ed} = 18.25 \text{kN/cm}^2 < f_{tw,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.507 < 1 \text{ ok} \]

\[ \sigma_{2,Ed} = 17.49 \text{kN/cm}^2 < f_{tw,d} = 25.92 \text{kN/cm}^2 \Rightarrow U = 0.675 < 1 \text{ ok} \]

web welds

design value: \( F_{Ed}(\sigma_p) = F / (2 \cdot h_1) = 1.12 \text{kN/cm}, \quad h_1 = 137.0 \text{mm} \)

\[ \sigma_{1,Ed} = 6.49 \text{kN/cm}^2 < f_{tw,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.180 < 1 \text{ ok} \]

stiffener in tension

\[ F_{Ed} = 75.48 \text{kN} \]

forces per rib

\[ F = 0.5 \cdot F_{Ed} \cdot (\text{br} - 2 \cdot r_{tw})/d = 28.1 \text{kN}, \quad H = F \cdot \varepsilon/\varepsilon_H = 4.8 \text{kN} \]

cross-section at flange

tension resistance \( N_{s,Ed} = 55.18 \text{kN} \)

design value: \( F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 29.3 \text{kN} \)

\[ F_{Ed} = 29.3 \text{kN} < F_{Rd} = 55.2 \text{kN} \Rightarrow U = 0.531 < 1 \text{ ok} \]

cross-section at web

shear resistance \( V_{Rd} = 178.01 \text{kN} \)

design value: \( F_{Ed} = F = 28.1 \text{kN} \)

\[ F_{Ed} = 28.1 \text{kN} < F_{Rd} = 178.0 \text{kN} \Rightarrow U = 0.158 < 1 \text{ ok} \]

flange welds

design values: \( F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 4.78 \text{kN/cm}, \quad F_{Ed}(\sigma_p) = H / (2 \cdot b_1) = 0.82 \text{kN/cm}, \quad b_1 = 29.4 \text{mm} \)

\[ \sigma_{1,Ed} = 16.63 \text{kN/cm}^2 < f_{tw,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.462 < 1 \text{ ok} \]

\[ \sigma_{2,Ed} = 15.94 \text{kN/cm}^2 < f_{tw,d} = 25.92 \text{kN/cm}^2 \Rightarrow U = 0.615 < 1 \text{ ok} \]

web welds

design value: \( F_{Ed}(\sigma_p) = F / (2 \cdot h_1) = 1.02 \text{kN/cm}, \quad h_1 = 137.0 \text{mm} \)

\[ \sigma_{1,Ed} = 5.92 \text{kN/cm}^2 < f_{tw,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.164 < 1 \text{ ok} \]

3.2.5.4. verification result

maximum utilization: \( \text{max } U = 0.760 < 1 \text{ ok} \)

3.2.6. rotational stiffness

stiffness coefficients

equivalent stiffness coefficient for 2 tension-bolt-rows:

\[ \begin{align*}
1: & \quad k_s = 3.09 \text{ mm}, \quad k_b = 11.52 \text{ mm}, \quad k_s = 6.96 \text{ mm}, \quad k_b = 4.39 \text{ mm} \Rightarrow k_{eff,1} = 1 / \Sigma(1/k_i,1) = 1.279 \text{ mm} \\
2: & \quad k_s = 3.09 \text{ mm}, \quad k_b = 11.52 \text{ mm}, \quad k_s = 6.96 \text{ mm}, \quad k_b = 4.39 \text{ mm} \Rightarrow k_{eff,2} = 1 / \Sigma(1/k_i,2) = 1.279 \text{ mm} \\
\end{align*} \]

\[ k_{eqx} = \Sigma(k_{eff,r,hr}) / z_{eq} = 1.912 \text{ mm}, \quad z_{eq} = \Sigma(k_{eff,r,hr}) / \Sigma(k_{eff,r,hr}) = 115.1 \text{ mm} \]

\[ k_1 = 0.38 \text{ Avc} / (B \cdot z) = 4.05 \text{ mm} \]

\[ k_2 = \text{(stiffened)} \]

rotational stiffness

initial rotational stiffness: \( S_{s,i,init} = (E \cdot z^2) / \Sigma(1/k_i) = 36118.8 \text{kNm/rad}, \quad z = z_{eq} = 115.1 \text{ mm}, \quad \Sigma(1/k_i) = 0.770 \text{ mm}^{-1} \)

\( N_{s,Ed} = 7.31 \text{kN} < 5\% N_{p,L,Rd} = 28.14 \text{kN} \text{ ok} \)

rotational stiffness: \( S_{s,Rd} = S_{s,i,init} / \mu = 36118.8 \text{kNm/rad}, \quad \mu = 1 \)

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rotation: \( \phi_{Ed} = \frac{M_{Ed}}{S_{Ed}} = 0.138^\circ \)