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

1. input report

section B-B

section A-A

connection right
details (section A - A)

connection left

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

**Steel grade**

Steel grade S235

**Column parameters**

Section HE280A

Reinforcement of the section with transverse stiffeners (web stiffeners, \(d_{st} = 619.7 \text{ mm}\)):

- Thickness of web stiffeners, \(t_{st} = 12.0 \text{ mm}\),
- Width of web stiffeners, \(b_{st} = 136.0 \text{ mm}\),
- Length of web stiffeners, \(l_{st} = 244.0 \text{ mm}\),
- Recess at stiffeners \(c_{st} = 36.0 \text{ mm}\),
- Welds at stiffeners  \(a_{st,t} = 4.0 \text{ mm}, a_{st,w} = 4.0 \text{ mm}\)

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

**Bolts**

- Bolt class 10.9, bolt size M20
- Large wrench size (high strength bolt), preloaded (for info: preloading \(F_p_c^* = 0.7 f_{yd} A_s = 154.3 \text{ kN}\))
- Shear plane passes through the unthreaded portion of the bolt

**Beam parameters**

Section HE280A

Slope angle of haunch about the horizontal axis \(\alpha_V = 35.0^\circ\)

- Haunch length \(L_V = 500.0 \text{ mm}\), haunch depth at the connection point \(h_V = L_V (\tan(\alpha_V) - \tan(\alpha_B)) = 350.1 \text{ mm}\)
- Web thickness \(t_{w_V} = 8.0 \text{ mm}\), flange width, thickness \(b_{w_V} = 280.0 \text{ mm}, t_{f_V} = 13.0 \text{ mm}\)
- Total beam depth at the connection point \(h_{BA} = h_b + h_v = 620.1 \text{ mm}\)

**Verification parameters**

Bolted end-plate connection:

- Thickness \(t_p = 20.0 \text{ mm}\), width \(b_p = 280.0 \text{ mm}, \text{ length } l_p = 720.1 \text{ mm}\)
- Projections \(h_{P,c} = 80.0 \text{ mm}, h_{p,u} = 20.0 \text{ mm}\)

Bolts in connection:

- 4 bolt-rows with 2 bolts
- All bolt-rows considered individually
- All bolt-rows for shear transfer (rows 1-4)
- Bolt groups generated automatically, considering all groups bzgl. row 1
- Centre distance of the bolts to the lateral edge of the end-plate \(e_2 = 50.0 \text{ mm}\)
- Centre distance of the first bolt-row to the upper edge of the end-plate (end row) \(e_0 = 40.0 \text{ mm}\)
- Centre distance of the last bolt-row to the bottom edge of the end-plate (end row) \(e_3 = 70.0 \text{ mm}\)
- Centre distance of the bolt-rows from each other \(p_{1,2} = 90.0 \text{ mm}, p_{2,3} = 70.0 \text{ mm}, p_{3,4} = 450.0 \text{ mm}\)

Welds at the connection point:

- Beam flange top: fillet weld, weld thickness \(a = 4.0 \text{ mm}\)
- Beam web: fillet weld, weld thickness \(a = 4.0 \text{ mm}\)
- Beam flange bottom: fillet weld, weld thickness \(a = 4.0 \text{ mm}\), angle \(\varphi = 125^\circ\)

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

**Beam parameters**

Section HE200A

**Verification parameters**

Welds at the connection point:

- Beam flange top: fillet weld, weld thickness \(a = 4.0 \text{ mm}\)
- Beam web: fillet weld, weld thickness \(a = 4.0 \text{ mm}\)
- Beam flange bottom: fillet weld, weld thickness \(a = 4.0 \text{ mm}\)

**Internal forces and moments in the intersection point of system axes**

Lk 1:

\[
\begin{align*}
N_{b1,Ed} &= 7.25 \text{ kN} & M_{b1,Ed} &= -8.15 \text{ kNm} & V_{b1,Ed} &= 12.74 \text{ kN (right)} \\
N_{b2,Ed} &= -5.46 \text{ kN} & M_{b2,Ed} &= -14.47 \text{ kNm} & V_{b2,Ed} &= -14.53 \text{ kN (left)} \\
N_{c1,Ed} &= -55.15 \text{ kN} & M_{c1,Ed} &= 7.33 \text{ kNm} & V_{c1,Ed} &= 11.43 \text{ kN (bottom)} \\
N_{c2,Ed} &= -26.99 \text{ kN} & M_{c2,Ed} &= 1.75 \text{ kNm} & V_{c2,Ed} &= -1.27 \text{ kN (top)}
\end{align*}
\]

Lk 2:

\[
\begin{align*}
N_{b1,Ed} &= 8.91 \text{ kN} & M_{b1,Ed} &= -18.74 \text{ kNm} & V_{b1,Ed} &= 20.03 \text{ kN (right)} \\
N_{b2,Ed} &= 2.61 \text{ kN} & M_{b2,Ed} &= -7.21 \text{ kNm} & V_{b2,Ed} &= -11.86 \text{ kN (left)} \\
N_{c1,Ed} &= -67.66 \text{ kN} & M_{c1,Ed} &= -3.47 \text{ kNm} & V_{c1,Ed} &= -3.10 \text{ kN (bottom)} \\
N_{c2,Ed} &= -34.56 \text{ kN} & M_{c2,Ed} &= 7.67 \text{ kNm} & V_{c2,Ed} &= -9.41 \text{ kN (top)}
\end{align*}
\]
Lk 3: $N_{b1,Ed} = 9.56 \text{ kN}$; $M_{b1,Ed} = -13.00 \text{ kNm}$; $V_{b1,Ed} = 17.93 \text{ kN}$ (right)
$N_{b2,Ed} = -4.78 \text{ kN}$; $M_{b2,Ed} = -18.34 \text{ kNm}$; $V_{b2,Ed} = -17.80 \text{ kN}$ (left)
$N_{c1,Ed} = -72.69 \text{ kN}$; $M_{c1,Ed} = 6.43 \text{ kNm}$; $V_{c1,Ed} = 10.63 \text{ kN}$ (bottom)
$N_{c2,Ed} = -35.95 \text{ kN}$; $M_{c2,Ed} = 3.74 \text{ kNm}$; $V_{c2,Ed} = -3.71 \text{ kN}$ (top)

Lk 4: $N_{b1,Ed} = 6.60 \text{ kN}$; $M_{b1,Ed} = -13.88 \text{ kNm}$; $V_{b1,Ed} = 14.84 \text{ kN}$ (right)
$N_{b2,Ed} = 1.99 \text{ kN}$; $M_{b2,Ed} = -5.34 \text{ kNm}$; $V_{b2,Ed} = -8.78 \text{ kN}$ (left)
$N_{c1,Ed} = -50.12 \text{ kN}$; $M_{c1,Ed} = -2.57 \text{ kNm}$; $V_{c1,Ed} = -2.30 \text{ kN}$ (bottom)
$N_{c2,Ed} = -25.60 \text{ kN}$; $M_{c2,Ed} = 5.68 \text{ kNm}$; $V_{c2,Ed} = -6.97 \text{ kN}$ (top)

**Partial safety factors for material**

- Resistance of cross-sections $\gamma_M = 1.00$
- Resistance of members in stability failure $\gamma_M = 1.10$
- Resistance of bolts, welds, plates in bearing $\gamma_M = 1.25$

**Check of data**

Connection right:
- ok

Connection left:
- ok

**Bolts right:**

- **Distances between bolt-rows at end-plate**
  - Horizontal: $e_2 = 50.0 \text{ mm} > 1.2 \cdot d_0 = 26.4 \text{ mm}$, $e_2 = 50.0 \text{ mm} < 4t + 40 \text{ mm} = 92.0 \text{ mm}$
  - Vertical: $e_1 = 40.0 \text{ mm} > 1.2 \cdot d_0 = 26.4 \text{ mm}$, $e_1 = 40.0 \text{ mm} < 4t + 40 \text{ mm} = 92.0 \text{ mm}$

- **Horizontal distance of bolts from column edge**
  - Vertical: $e_1 = 70.1 \text{ mm} > 1.2 \cdot d_0 = 26.4 \text{ mm}$, $e_1 = 70.1 \text{ mm} < 4t + 40 \text{ mm} = 92.0 \text{ mm}$

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

**Utilizations of each joint (right)**

<table>
<thead>
<tr>
<th>Lk</th>
<th>$U_\text{m}$</th>
<th>$U_\text{v}$</th>
<th>$U_{\text{ep}}$</th>
<th>$U_{\text{sb}}$</th>
<th>$U_{\text{ss}}$</th>
<th>$U_{\text{sw}}$</th>
<th>$U$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.037</td>
<td>0.008</td>
<td>0.011</td>
<td>0.022</td>
<td>0.031</td>
<td>0.578</td>
<td>0.578</td>
</tr>
<tr>
<td>2</td>
<td>0.079</td>
<td>0.014</td>
<td>0.018</td>
<td>0.047</td>
<td>0.067</td>
<td>0.414</td>
<td>0.414</td>
</tr>
<tr>
<td>3</td>
<td>0.058</td>
<td>0.012</td>
<td>0.016</td>
<td>0.034</td>
<td>0.049</td>
<td>0.685</td>
<td>0.685</td>
</tr>
<tr>
<td>4</td>
<td>0.059</td>
<td>0.010</td>
<td>0.013</td>
<td>0.035</td>
<td>0.050</td>
<td>0.307</td>
<td>0.307</td>
</tr>
</tbody>
</table>

$U_{\text{m}}$: utilization due to bending; $U_{\text{v}}$: utilization due to shear/bearing resistance; $U_{\text{ep}}$: utilization due to shear in end-plate

**Utilizations of each joint (left)**

<table>
<thead>
<tr>
<th>Lk</th>
<th>$U_\text{m}$</th>
<th>$U_{\text{sb}}$</th>
<th>$U_{\text{ss}}$</th>
<th>$U_{\text{sw}}$</th>
<th>$U$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.199</td>
<td>0.191</td>
<td>0.140</td>
<td>0.582</td>
<td>0.582</td>
</tr>
<tr>
<td>2</td>
<td>0.133</td>
<td>0.086</td>
<td>0.063</td>
<td>0.418</td>
<td>0.418</td>
</tr>
<tr>
<td>3</td>
<td>0.224</td>
<td>0.212</td>
<td>0.155</td>
<td>0.691</td>
<td>0.691</td>
</tr>
<tr>
<td>4</td>
<td>0.098</td>
<td>0.064</td>
<td>0.047</td>
<td>0.310</td>
<td>0.310</td>
</tr>
</tbody>
</table>

$U_{\text{m}}$: utilization due to bending; $U_{\text{sb}}$: utilization due to weld; $U_{\text{ss}}$: utilization due to stiffeners/ribs

**2. Final result**

**Utilization of the connection**

<table>
<thead>
<tr>
<th>Lk</th>
<th>$U_j$</th>
<th>$\Sigma H$</th>
<th>$\Sigma V$</th>
<th>$\Sigma M$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.582</td>
<td>3.49</td>
<td>3.67</td>
<td>0.75</td>
</tr>
<tr>
<td>2</td>
<td>0.418</td>
<td>5.61</td>
<td>4.82</td>
<td>0.39</td>
</tr>
<tr>
<td>3</td>
<td>0.691</td>
<td>4.95</td>
<td>4.92</td>
<td>0.65</td>
</tr>
<tr>
<td>4</td>
<td>0.310</td>
<td>4.16</td>
<td>3.57</td>
<td>0.29</td>
</tr>
</tbody>
</table>

$U_j$: utilization of the connection; $\Sigma H$, $\Sigma V$, $\Sigma M$: tolerances of equilibrium $1 \text{ kN} / 1 \text{ kNm}$

*) Maximum utilization

Maximum utilization [Lk 3]: $\max U = 0.691 < 1$ ok

**Verification succeeded**
3. Regulations

DIN EN 1990, Eurocode 0: Grundlagen der Tragwerksplanung;
DIN EN 1990/NA, Nationaler Anhang zur DIN EN 1990, Ausgabe Dezember 2010

DIN EN 1993-1-1, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau;
DIN EN 1993-1-1/NA, Nationaler Anhang zur DIN EN 1993-1-1, Ausgabe September 2017

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

4. Detailed edition of Lk 3 (decisive)

notes
no verification for cross-sections.

4.1. connection right

notes
connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.
In haunched beams the bottom flange of the rolled section is not considered. A fictive
welded section is shaped from the top beam flange, the beam web and the haunch flange.
Only at calculation of end plate the lower beam flange is respected as a stiffener.
no verification for welds of welded section.

4.1.1. design values

periphery connection\periphery plane\partial internal forces for haunched beams

slope angle: \( \alpha_b = 0.00^\circ, \alpha_v = 35.00^\circ \Rightarrow \alpha = (\alpha_b + \alpha_v)/2 = 17.50^\circ \), \( \Delta \alpha = \alpha - \alpha_b = 17.50^\circ \)
distance: \( e_1 = 135.0 \text{ mm}, e_3 = 340.6 \text{ mm}, e_2 = 286.1 \text{ mm}, e_5 = 135.0 \text{ mm}, e_7 = 286.2 \text{ mm}, e_4 = 516.1 \text{ mm} \)

internal forces and moments perpendicular to the connection planes
periphery beam (right)
\( N_d = -14.51 \text{ kN}, M_d = 10.47 \text{ kNm}, V_d = 14.23 \text{ kN} \)
periphery haunch-beam
\( N_{d_h} = -14.51 \text{ kN}, M_{d_h} = 0.76 \text{ kNm}, V_{d_h} = 14.23 \text{ kN} \)
periphery beam (left)
\( N_{d_2} = 4.78 \text{ kN}, M_{d_2} = 13.85 \text{ kNm}, V_{d_2} = -17.60 \text{ kN} \)
periphery column (bottom)
\( N_c = 72.69 \text{ kN}, M_c = -3.26 \text{ kNm}, V_c = 10.63 \text{ kN} \)
periphery column (top)
\( N_{d_2} = 35.95 \text{ kN}, M_{d_2} = 2.55 \text{ kNm}, V_{d_2} = -3.71 \text{ kN} \)

partial internal forces and moments
internal forces and moments in the periphery end-plate beam: \( M_d = M_d + N_{d_2} \tan(\alpha) \cdot V_{d_2} \tan(\alpha) = 10.09 \text{ kNm} \)
\( N_{b_1} = -N_d z_{bu}/z_b + M_d^2/z_b = 23.48 \text{ kN}, z_b = 605.7 \text{ mm}, z_{bu} = 284.8 \text{ mm} \)
\( N_{b_2} = (N_d z_{bo}/z_b + M_d z_{bo}) / \cos(\alpha) = 10.96 \text{ kN}, z_b = 605.7 \text{ mm}, z_{bo} = 320.8 \text{ mm} \)

basic component 1 is not calculated !!
4.1.2. connection capacity

4.1.2.1. moment resistance

distance of tension-bolt-rows from centre of compression:
   \( h_1 = 652.2 \text{ mm}, \ h_2 = 562.2 \text{ mm}, \ h_3 = 492.2 \text{ mm}, \ h_4 = 42.2 \text{ mm} \)

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

resistance of flanges
   \( \Sigma F_{c,Rd} = 1130.6 \text{ kN} \)

moment resistance
   \( M_{ld,Rd} = \Sigma (F_{tr,Rd} \cdot h) = 251.8 \text{ kNm} \)

tension resistance
   \( N_{ld,Rd} = \Sigma F_{c,Rd} = 749.9 \text{ kN} \)

compression resistance
   \( N_{ldc,Rd} = \Sigma F_{c,Rd} = 1130.6 \text{ kN} \)

4.1.2.2. shear/bearing resistance

resistance per bolt-row
row 1:  \( V_{tr,Rd} = 168.7 \text{ kN} \)
row 2:  \( V_{tr,Rd} = 182.4 \text{ kN} \)
row 3:  \( V_{tr,Rd} = 301.6 \text{ kN} \)
row 4:  \( V_{tr,Rd} = 301.6 \text{ kN} \)
   \( \Sigma V_{tr,Rd} = 954.3 \text{ kN} \)

shear/bearing resistance
   \( V_{ld,Rd} = \Sigma V_{tr,Rd} = 954.3 \text{ kN} \)

4.1.2.3. shear resistance

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

shear resistance of end plate:  \( V_{ep,Rd} = F_{w,Rd} = 903.27 \text{ kN} \)

4.1.2.4. total

   \( M_{ld,Rd} = 251.6 \text{ kNm} \)
   \( N_{ld,Rd} = 749.9 \text{ kN} \)
   \( N_{ldc,Rd} = 1130.6 \text{ kN} \)
   \( V_{ld,Rd} = 903.3 \text{ kN} \)
   \( V_{ep,Rd} = 903.3 \text{ kN} \)

4.1.3. verifications

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

axial force:  \( N_{ld,Rd} = |N_d \cdot \cos(x) + V_d \cdot \sin(x)| = 9.56 \text{ kN} < 5\% N_{pl,Rd} = 156.37 \text{ kN} \Rightarrow \text{moment resistance} \)

internal moment:  \( M_{ed} = M_0 - N_d z_{bu} = 14.51 \text{ kNm}, \ z_{bu} = 279.5 \text{ mm} \)

shear force:  \( V_{ed} = |V_d| = 14.23 \text{ kN} \)

   \( M_{ed}/M_{ld,Rd} = 0.058 < 1 \text{ ok} \)
   \( V_{ed}/V_{ld,Rd} = 0.016 < 1 \text{ ok} \)
   \( V_{ed}/V_{ep,Rd} = 0.016 < 1 \text{ ok} \)

4.1.3.2. verification of welds at beam section

weld 1: beam flange in tension outer

weld 8: beam flange in compression outer

calculation section:

welds 2,3: beam flange in tension inner
welds 4,5: beam web double-sided
welds 6,7: beam flange in compression inner
design values referring to centroid of the section:
\[ N_{Ed} = 14.51 \text{kN}, \quad M_{y,Ed} = -10.47 \text{kNm}, \quad V_{z,Ed} = 14.23 \text{kN} \]
cross-sectional properties referring to centroid of the line cross-section:
\[ \Sigma A_w = 83.78 \text{cm}^2, \quad A_{x,z} = 43.46 \text{cm}^2, \quad \Sigma_w = 209.4 \text{cm} \]
\[ I_{w,y} = 47881.43 \text{cm}^4, \quad I_{w,z} = 2922.18 \text{cm}^4, \quad W_{w,t} = 106.46 \text{cm}^3, \quad \Delta z_w = -18.3 \text{mm} \]

verifications in weld edges:

- **weld 1, pt. 0:** \( \sigma_{w,x} = 8.49 \text{N/mm}^2 \)
  \( \tau_{w,z} = 3.27 \text{N/mm}^2 \)
  \( U_w = 0.033 < 1 \text{ ok} \)

- **weld 2, pt. 0:** \( \sigma_{w,x} = 8.20 \text{N/mm}^2 \)
  \( \tau_{w,z} = 3.27 \text{N/mm}^2 \)
  \( U_w = 0.032 < 1 \text{ ok} \)

- **weld 4, pt. 0:** \( \sigma_{w,x} = 7.68 \text{N/mm}^2 \)
  \( \tau_{w,z} = 3.27 \text{N/mm}^2 \)
  \( U_w = 0.034 < 1 \text{ ok} \)

- **weld 1, pt. 0:** \( \sigma_{w,x} = 4.20 \text{N/mm}^2 \)
  \( \tau_{w,z} = 3.27 \text{N/mm}^2 \)
  \( U_w = 0.023 < 1 \text{ ok} \)

- **weld 6, pt. 0:** \( \sigma_{w,x} = -4.72 \text{N/mm}^2 \)
  \( \tau_{w,z} = 3.27 \text{N/mm}^2 \)
  \( U_w = 0.019 < 1 \text{ ok} \)

- **weld 8, pt. 0:** \( \sigma_{w,x} = -5.07 \text{N/mm}^2 \)
  \( \tau_{w,z} = 3.27 \text{N/mm}^2 \)
  \( U_w = 0.020 < 1 \text{ ok} \)

**Result:**

- **weld 4, pt. 0:** \( \sigma_{w,x} = 7.68 \text{N/mm}^2 \)
  \( \tau_{w,z} = 3.27 \text{N/mm}^2 \)
  \( U_w = 0.034 < 1 \text{ ok} \)

Max: \( \sigma_{1,w,Ed} = 1.22 \text{kN/cm}^2 < f_{w,d} = 36.00 \text{kN/cm}^2 \)
\( \sigma_{2,w,Ed} = 0.54 \text{kN/cm}^2 < f_{w,d} = 25.92 \text{kN/cm}^2 \)

4.1.3.3. verification of web stiffeners

**compression stiffener**

\( F_{c,Ed} = 10.85 \text{kN} \)

forces per rib

\( F = 0.5 \cdot F_{c,Ed} \cdot (b_r-2 \cdot r_{tw})/b_t = 4.3 \text{kN}, \quad H = F \cdot \varphi / \varphi_H = 1.5 \text{kN} \)

assumption: stiffeners do not buckle: \( \varphi = 11.3 \leq \varphi < 33 \Rightarrow \text{section class } 1 \leq 2 \text{ ok} \)

cross-section at flange

**compression resistance** \( N_{c,Rd} = (A_f \cdot f) / \varphi_{Rd} = 282.00 \text{kN} \)

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

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

cross-section at web

**shear resistance** \( V_{Rd} = 397.26 \text{kN} \)

design value: \( F_{Ed} = F = 4.3 \text{kN} \)

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

**flange welds**

design values:

\( F_{Ed}(\sigma) = F / (2 \cdot b_1) = 0.22 \text{kN/cm}, \quad F_{Ed}(\tau) = H / (2 \cdot b_1) = 0.08 \text{kN/cm} \)
\( b_1 = 100.0 \text{ mm} \)

\( \sigma_{1,w,Ed} = 0.64 \text{kN/cm}^2 < f_{w,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.018 < 1 \text{ ok} \)
\( \sigma_{2,w,Ed} = 0.54 \text{kN/cm}^2 < f_{w,d} = 25.92 \text{kN/cm}^2 \Rightarrow U = 0.021 < 1 \text{ ok} \)

**web welds**

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

\( \sigma_{1,w,Ed} = 0.55 \text{kN/cm}^2 < f_{w,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.015 < 1 \text{ ok} \)

**stiffener in tension**

\( F_{l,Ed} = 25.36 \text{kN} \)

forces per rib

\( F = 0.5 \cdot F_{l,Ed} \cdot (b_r-2 \cdot r_{tw})/b_t = 10.1 \text{kN}, \quad H = F \cdot \varphi / \varphi_H = 3.6 \text{kN} \)

cross-section at flange

**tension resistance** \( N_{t,Rd} = 282.00 \text{kN} \)

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

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

cross-section at web

**shear resistance** \( V_{Rd} = 397.26 \text{kN} \)

design value: \( F_{Ed} = F = 10.1 \text{kN} \)

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

**flange welds**

design values:

\( F_{Ed}(\sigma) = F / (2 \cdot b_1) = 0.51 \text{kN/cm}, \quad F_{Ed}(\tau) = H / (2 \cdot b_1) = 0.18 \text{kN/cm} \)
\( b_1 = 100.0 \text{ mm} \)

\( \sigma_{1,w,Ed} = 1.49 \text{kN/cm}^2 < f_{w,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.041 < 1 \text{ ok} \)
\( \sigma_{2,w,Ed} = 1.27 \text{kN/cm}^2 < f_{w,d} = 25.92 \text{kN/cm}^2 \Rightarrow U = 0.049 < 1 \text{ ok} \)

**web welds**

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

\( \sigma_{1,w,Ed} = 1.28 \text{kN/cm}^2 < f_{w,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.035 < 1 \text{ ok} \)
4.1.3.4. elastic verification of the shear area

column web
requirements concerning stiffeners: s. verification of web stiffeners
requirements concerning shear area: shear buckling: $hp/2b = 30.50 \leq 72/(\pi \sqrt{2}) = 60.00$ ok
internal forces and moments at web (sign definition of statics):
$N_1 = -4.78$ kN, $M_1 = -13.96$ kNm, $V_1 = -17.60$ kN
$N_3 = -72.69$ kN, $M_3 = 3.26$ kNm, $V_3 = 10.63$ kN
$N_4 = 14.51$ kN, $M_4 = -10.59$ kNm, $V_4 = 14.23$ kN
$N_2 = -35.95$ kN, $M_2 = 2.55$ kNm, $V_2 = -3.71$ kN
dimensions of the shear area: $h_b = 305.8$ mm, $h_t = 328.4$ mm, $h_l = 164.4$ mm, $h_r = 590.1$ mm
stresses within the shear area:
$t_b = 30.3$ N/mm², $t_l = 27.7$ N/mm², $t_t = 93.0$ N/mm², $t_r = 7.1$ N/mm²
verification of the shear area:
$max \, t_{Ed} = 93.0$ N/mm² $< t_{rd} = 135.7$ N/mm² $\Rightarrow$ $U = 0.685 < 1$ ok

4.1.3.5. verification result
maximum utilization: max $U = 0.685 < 1$ ok

4.2. connection left

4.2.1. design values

periphery connection zur connection plane partial internal forces and moments

slope angle: $\alpha_b = \alpha_l = \alpha_c = 0^\circ$
$\alpha_{b2} = 0.00^\circ$, $\alpha_{l2} = 35.00^\circ$ $\Rightarrow$ $\alpha_2 = (\alpha_{b2} + \alpha_{l2})/2 = 17.50^\circ$
distance: $e_1 = 135.0$ mm, $e_3 = 298.2$ mm, $e_2 = 298.2$ mm, $e_5 = 135.0$ mm, $e_7 = 340.6$ mm

internal forces and moments perpendicular to the connection planes
periphery beam (right)
$N_d = 4.78$ kN, $M_d = 13.96$ kNm, $V_d = 17.60$ kN
periphery beam (left)
$N_{d2} = -14.51$ kN, $M_{d2} = 10.58$ kNm, $V_{d2} = -14.23$ kN
periphery column (bottom)
$N_c = 72.69$ kN, $M_c = 3.26$ kNm, $V_c = -10.63$ kN
periphery column (top)
$N_{c2} = 35.95$ kN, $M_{c2} = 2.55$ kNm, $V_{c2} = 3.71$ kN
partial internal forces and moments
$N_{b1} = -N_d z_{bu}/2 + M_d z_{b} = 75.19$ kN, $z_b = 180.0$ mm, $z_{bu} = 90.0$ mm
$N_{b2} = N_d z_{bo}/2 + M_d z_{b} = 79.97$ kN, $z_b = 180.0$ mm, $z_{bo} = 90.0$ mm

basic component 1 is not calculated !!

4.2.2. connection capacity

4.2.2.1. moment resistance
distance between tension force and centre of compression: $z = 180.0$ mm

resistance
$F_{Ra} = 335.1$ kN

resistance of flanges
$\Sigma F_{c,Ra} = 670.2$ kN

moment resistance
$M_{LRd} = F_{Rd} \cdot z = 60.3$ kNm
tension resistance
$N_{L,Rd} = F_{L,Rd} = 387.9$ kN
compression resistance
$N_{L,C,Rd} = \Sigma F_{c,Rd} = 670.2$ kN
4.2.3. verifications

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

axial force: \( N_{b,Ed} = N_{Ed} = 4.78 \text{kN} < 5\% N_{\text{pl,Rd}} = 63.25 \text{kN} \Rightarrow \) moment resistance

internal moment: \( M_{Ed} = M_{a} - N_{Ed} t_{bw} = 13.59 \text{kNm} \), \( t_{bw} = 90.0 \text{mm} \)

shear force: \( V_{Ed} = |V_{d}| = 17.60 \text{kN} \)

\( M_{Ed}/M_{j,Rd} = 0.224 < 1 \text{ ok} \)

4.2.3.2. verification of welds at beam section

weld 1: beam flange in tension outer

welds 2,3: beam flange in tension inner

weld 4,6: beam web double-sided

welds 6,7: beam flange in compression inner

weld 8: beam flange in compression outer

calculation section:

\[
\begin{align*}
\text{weld 1: } & a_w = 4.0 \text{ mm} \quad l_w = 200.0 \text{ mm} \\
\text{weld 2: } & a_w = 4.0 \text{ mm} \quad l_w = 78.8 \text{ mm} \\
\text{weld 3: } & \text{siehe weld 2} \\
\text{weld 4: } & a_w = 4.0 \text{ mm} \quad l_w = 134.0 \text{ mm} \\
\text{weld 5: } & \text{siehe weld 4} \\
\text{weld 6: } & a_w = 4.0 \text{ mm} \quad l_w = 78.8 \text{ mm} \\
\text{weld 7: } & \text{siehe weld 6} \\
\text{weld 8: } & a_w = 4.0 \text{ mm} \quad l_w = 200.0 \text{ mm}
\end{align*}
\]

design values referring to centroid of the section:

\( N_{Ed} = -4.78 \text{kN} \), \( M_{x,Ed} = -13.96 \text{kNm} \), \( V_{z,Ed} = 17.60 \text{kN} \)

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

\( \Sigma A_w = 39.32 \text{ cm}^2 \), \( A_{w,z} = 10.72 \text{ cm}^2 \), \( \Sigma l_w = 99.3 \text{ cm} \)

\( I_{w,y} = 2514.76 \text{ cm}^4 \), \( I_{w,z} = 1062.68 \text{ cm}^4 \), \( W_{w,t} = 43.06 \text{ cm}^3 \), \( \Delta z_w = 0.0 \text{ mm} \)

verifications in weld edges:

weld 1, pt. 0: \( \sigma_{w,x} = 51.54 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.202 < 1 \text{ ok} \)

weld 2, pt. 0: \( \sigma_{w,x} = 45.99 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.181 < 1 \text{ ok} \)

weld 4, pt. 0: \( \sigma_{w,x} = 35.99 \text{ N/mm}^2 \quad \tau_{w,z} = 16.42 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.162 < 1 \text{ ok} \)

weld 1, pt. 1: \( \sigma_{w,x} = -38.42 \text{ N/mm}^2 \quad \tau_{w,z} = 16.42 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.170 < 1 \text{ ok} \)

weld 6, pt. 0: \( \sigma_{w,x} = -48.42 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.190 < 1 \text{ ok} \)

weld 8, pt. 0: \( \sigma_{w,x} = -53.97 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.212 < 1 \text{ ok} \)

Result:

weld 8, pt. 0: \( \sigma_{w,x} = -53.97 \text{ N/mm}^2 \quad \Rightarrow U_w = 0.212 < 1 \text{ ok} \)

Max: \( \sigma_{1,Ed} = 7.63 \text{kN/cm}^2 < f_{w,d} = 36.00 \text{kN/cm}^2 \)

\( \sigma_{2,Ed} = 3.82 \text{kN/cm}^2 < f_{w,d} = 25.92 \text{kN/cm}^2 \Rightarrow U_w = 0.212 < 1 \text{ ok} \)

4.2.3.3. verification of web stiffeners

compression stiffener

\( F_{C,Ed} = 60.61 \text{kN} \)

forces per rib

\( F = 0.5 F_{C,Ed} \cdot \left( b_r - t_{w} / b_t \right) = 32.2 \text{kN} \), \( H = F = 85.45 \text{kN} \)

assumption: stiffeners do not buckle: \( \alpha_1 = 11.3 \% < 33 \% \Rightarrow \text{section class } 1 \leq 2 \text{ ok} \)

cross-section at flange

compression resistance \( N_{w,Ed} = (A_{w})/\gamma_{M0} = 282.0 \text{kN} \)

design value: \( F_{Ed} = (f_{e1} + 3 \lambda_0)^{1/2} = 37.8 \text{kN} \)

\( F_{Ed} = 37.8 \text{kN} < F_{rd} = 282.0 \text{kN} \Rightarrow U = 0.134 < 1 \text{ ok} \)

cross-section at web

shear resistance \( V_{rd} = 397.26 \text{kN} \)

design value: \( F_{Ed} = F = 32.2 \text{kN} \)

\( F_{Ed} = 32.2 \text{kN} < F_{rd} = 397.3 \text{kN} \Rightarrow U = 0.081 < 1 \text{ ok} \)

flange welds

design values: \( F_{Ed}(\sigma_{1}) = F / (2b_1) = 1.81 \text{kN/cm} \), \( F_{Ed}(\sigma_{2}) = H / (2b_1) = 0.57 \text{kN/cm} \), \( b_1 = 100.0 \text{mm} \)

\( \sigma_{1,Ed} = 4.72 \text{kN/cm}^2 < f_{w,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.131 < 1 \text{ ok} \)

\( \sigma_{2,Ed} = 4.03 \text{kN/cm}^2 < f_{w,d} = 25.92 \text{kN/cm}^2 \Rightarrow U = 0.155 < 1 \text{ ok} \)

web welds

design value: \( F_{Ed}(\sigma_{1}) = F / (2b_1) = 1.84 \text{kN/cm} \), \( b_1 = 172.0 \text{mm} \)

\( \sigma_{1,Ed} = 4.06 \text{kN/cm}^2 < f_{w,d} = 36.00 \text{kN/cm}^2 \Rightarrow U = 0.113 < 1 \text{ ok} \)

stiffener in tension

\( F_{L,Ed} = 75.83 \text{kN} \)

forces per rib

\( F = 0.5 F_{L,Ed} \cdot (b_r - t_w / b_t) = 30.3 \text{kN} \), \( H = F = 85.45 \text{kN} \)

cross-section at flange
tension resistance $N_t, R_d = 282.00 \text{ kN}$
design value: $F_{Ed} = (F^2 + 3H^0)^{1/2} = 35.5 \text{ kN}$
$F_{Ed} = 35.5 \text{ kN} < F_{Rd} = 282.0 \text{ kN} \Rightarrow U = 0.126 < 1 \text{ ok}$
cross-section at web
shear resistance $V_{Rd} = 397.26 \text{ kN}$
design value: $F_{Ed} = F = 30.3 \text{ kN}$
$F_{Ed} = 30.3 \text{ kN} < F_{Rd} = 397.3 \text{ kN} \Rightarrow U = 0.076 < 1 \text{ ok}$
flange welds
design values: $F_{Ed}(\sigma_0) = F / (2 \cdot b_1) = 1.52 \text{ kN/cm}$, $F_{Ed}(\sigma_p) = H / (2 \cdot b_1) = 0.53 \text{ kN/cm}$, $b_1 = 100.0 \text{ mm}$
$\sigma_{1w,Ed} = 4.44 \text{ kN/cm}^2 < f_{w,d} = 36.00 \text{ kN/cm}^2 \Rightarrow U = 0.123 < 1 \text{ ok}$
$\sigma_{2w,Ed} = 3.79 \text{ kN/cm}^2 < f_{w,d} = 25.92 \text{ kN/cm}^2 \Rightarrow U = 0.146 < 1 \text{ ok}$
web welds
design value: $F_{Ed}(\sigma_p) = F / (2 \cdot l_1) = 0.88 \text{ kN/cm}$, $l_1 = 172.0 \text{ mm}$
$\sigma_{1w,Ed} = 3.82 \text{ kN/cm}^2 < f_{w,d} = 36.00 \text{ kN/cm}^2 \Rightarrow U = 0.106 < 1 \text{ ok}$

4.2.3.4. elastic verification of the shear area

column web
requirements concerning stiffeners: s. verification of web stiffeners
requirements concerning shear area: shear buckling: $h_p / h = 30.50 \leq 72 / (1 \cdot e) = 60.00 \text{ ok}$
internal forces and moments at web (sign definition of statics):
$N_1 = 14.51 \text{ kN}$, $M_1 = -10.71 \text{ kNm}$, $V_1 = -14.23 \text{ kN}$
$N_3 = 72.89 \text{ kN}$, $M_3 = -3.26 \text{ kNm}$, $V_3 = -10.63 \text{ kN}$
$N_4 = -4.78 \text{ kN}$, $M_4 = -14.08 \text{ kNm}$, $V_4 = 17.60 \text{ kN}$
$N_2 = -35.95 \text{ kN}$, $M_2 = -2.55 \text{ kNm}$, $V_2 = 3.71 \text{ kN}$
dimensions of the shear area: $h_b = 304.1 \text{ mm}$, $h_t = 330.3 \text{ mm}$, $h_l = 590.1 \text{ mm}$, $h_r = 164.4 \text{ mm}$
pressures within the shear area:
$\sigma_b = 30.9 \text{ N/mm}^2$, $\sigma_t = 28.0 \text{ N/mm}^2$, $\sigma_l = 7.2 \text{ N/mm}^2$, $\sigma_r = 93.7 \text{ N/mm}^2$
verification of the shear area:
$max \tau_{Ed} = 93.7 \text{ N/mm}^2 < \tau_{Rd} = 135.7 \text{ N/mm}^2 \Rightarrow U = 0.691 < 1 \text{ ok}$

4.2.3.5. verification result
maximum utilization: $\max U = 0.691 < 1 \text{ ok}$