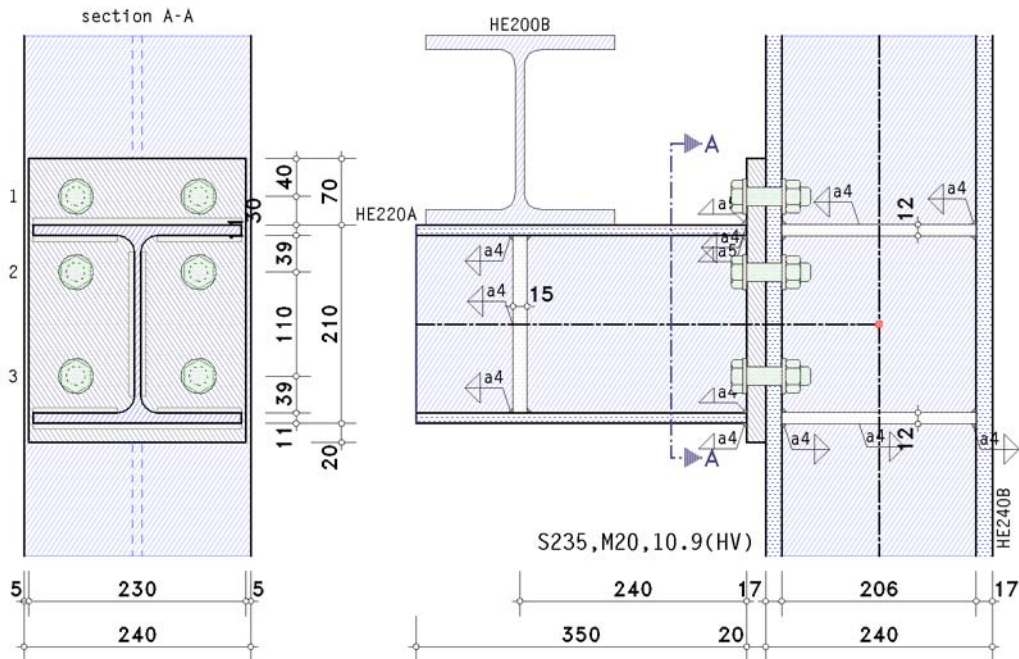
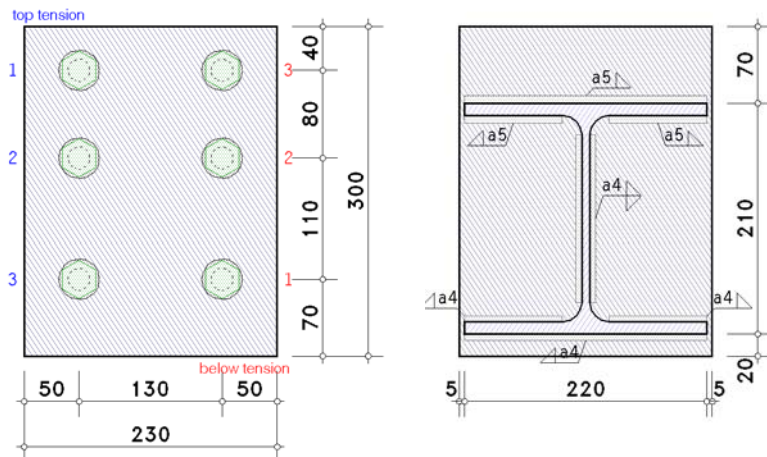


1. input report



details (section A - A)



steel grade

steel grade S235

column parameters

section HE240B

reinforcement of the section with transverse stiffeners (web stiffeners, $d_{st} = 199.0$ mm):

thickness $t_{st} = 12.0$ mm, width $b_{st} = 115.0$ mm, length $l_{st} = 206.0$ mm

recess at stiffeners $c_{st} = 31.5$ mm

welds $a_{st,f} = 4.0$ mm, $a_{st,w} = 4.0$ mm

beam parameters

section HE220A

reinforcement of the section with transverse stiffeners:

thickness $t_{st} = 15.0$ mm, width $b_{st} = 106.5$ mm, length $l_{st} = 188.0$ mm

recess at stiffeners $c_{st} = 27.0$ mm

welds $a_{st,f} = 4.0$ mm, $a_{st,w} = 4.0$ mm

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

welds

beam flange top: fillet weld, weld thickness $a = 5.0$ mm

beam web: fillet weld, weld thickness $a = 4.0$ mm

beam flange below: fillet weld, weld thickness $a = 4.0$ mm

75% of compressive stress is transferred by contact

parameters of the connection

bolted end-plate connection

end-plate thickness $t_p = 20.0$ mm, width $b_p = 230.0$ mm, length $l_p = 300.0$ mm

projections $h_{p,o} = 70.0$ mm, $h_{p,u} = 20.0$ mm

bolts:

3 bolt-rows with 2 bolts

centre distance of the bolts to the lateral edge of the end-plate $e_2 = 50.0$ mm

centre distance of the first bolt-row to the upper edge of the end-plate (end row) $e_o = 40.0$ mm

centre distance of the last bolt-row to the bottom edge of the end-plate (end row) $e_u = 70.0$ mm

centre distance of the bolt-rows from each other $p_{1-2} = 80.0$ mm, $p_{2-3} = 110.0$ mm

distance of transverse loading $\Delta a = 240.0$ mm

verification of local loading:

transverse loading on top flange of bracket by a load girder (crossing of girders) section HE200B

verification of fatigue:

damage equivalent stress factors $\lambda_\sigma = 1.000$, $\lambda_\tau = 1.000$

calculation of resistances

verification of bracket-column connection, limit state of resistance (ULS)

verification of local loading, crossing of girders, limit state of resistance (ULS)

fatigue design of the connection and of bracket cross-section, limit state of fatigue (FLS)

bracket load and internal forces and moments in the intersection point of system axes (ULS)

Lc	F _{2,Ed} kN	H _{2,Ed} kN
--	--	--
1	150.00	50.00

F_{2,Ed}, H_{2,Ed}: loading of bracket

bracket load (FLS)

Lc	F _{2,Ed} kN	H _{2,Ed} kN
--	--	--
1	80.00	---
2	110.00	40.00

F_{2,Ed}, H_{2,Ed}: loading of bracket

partial safety factors for material (ULS)

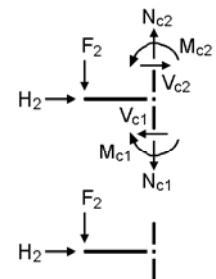
resistance of cross-sections $\gamma_{M0} = 1.00$

resistance of members in stability failure $\gamma_{M1} = 1.10$

resistance of bolts, welds, plates in bearing $\gamma_{M2} = 1.25$

material safety factor (FLS)

design concept: damage tolerance, damage consequence: high $\Rightarrow$ fatigue strength $\gamma_{Mf} = 1.15$



2. verifications

2.1. table of results (ULS)

2.1.1. utilizations

Lc	U _j	U _l	U _{j,b}	U _{j,σ}	U _{j,τ}	U _{j,w}	U _{j,r}	U _{l,σ}	U _{l,r}	U
--	---	---	---	---	---	---	---	---	---	---
1	0.829	0.551	0.829	0.301	0.593	0.625	0.393	0.551	0.366	0.829*

U_j: res. utilization due to beam-column-connection; U_l: res. utilization due to local loading; U_{j,b}: utilization of cross-section beam

U_{j,σ}: utilization due to bending; U_{j,τ}: utilization due to shear force; U_{j,w}: utilization due to weld

U_{j,r}: utilization due to stiffeners/ribs; U_{l,σ}: utilization due to stresses at first cut of web; U_{l,r}: utilization due to ribs

U: utilization of the connection

*) maximum utilization

2.2. connection to column (ULS)

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

note

connection is verified due to EC 3-1-8 regardless of preloading.

however, connections may be constructed with prestressed high strength bolts.

check of data

ok

distances between bolts at end-plate

horizontal: $e_2 = 50.0$ mm $> 1.2 \cdot d_0 = 26.4$ mm,

horizontal: $p_2 = 130.0$ mm $> 2.4 \cdot d_0 = 52.8$ mm,

top-below: $e_1 = 40.0$ mm $> 1.2 \cdot d_0 = 26.4$ mm,

top-below: $p_1 = 80.0$ mm $> 2.2 \cdot d_0 = 48.4$ mm,

top-below: $p_1 = 110.0$ mm $> 2.2 \cdot d_0 = 48.4$ mm,

top-below: $e_1 = 70.0$ mm $> 1.2 \cdot d_0 = 26.4$ mm,

bolt distance from column edge

horizontal: $e_2 = 55.0$ mm $> 1.2 \cdot d_0 = 26.4$ mm,

$e_2 = 50.0$ mm $< 4 \cdot t + 40$ mm = 108.0 mm

$p_2 = 130.0$ mm $< \min(14 \cdot t, 200$ mm) = 200.0 mm

$e_1 = 40.0$ mm $< 4 \cdot t + 40$ mm = 108.0 mm

$p_1 = 80.0$ mm $< \min(14 \cdot t, 200$ mm) = 200.0 mm

$p_1 = 110.0$ mm $< \min(14 \cdot t, 200$ mm) = 200.0 mm

$e_1 = 70.0$ mm $< 4 \cdot t + 40$ mm = 108.0 mm

$e_2 = 55.0$ mm $< 4 \cdot t + 40$ mm = 108.0 mm

2.2.1. table of results

utilization

Lc	$U_{\sigma,b}$	U_m	U_v	U_{wp}	U_{ep}	U_{sb}	U_{ss}	U
1	0.829	0.301	0.183	0.380	0.593	0.625	0.393	0.829*

$U_{\sigma,b}$: stress utilization at the beam; U_m : utilization due to bending; U_v : utilization due to shear/bearing resistance
 U_{wp} : utilization due to shear in column web; U_{ep} : utilization due to shear in end-plate; U_{sb} : utilization due to weld
 U_{ss} : utilization due to stiffeners/ribs; U: utilization of the connection
 *) maximum utilization

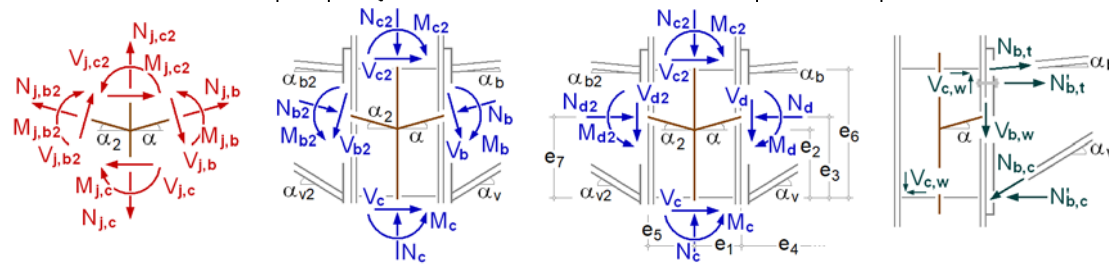
2.2.2. final result

maximum utilization: $\max U = 0.829 < 1$ ok

2.2.3. Lc 1 (decisive)

2.2.3.1. design values

internal forces at node periphery connection zur connection plane partial internal forces and moments



slope angle: $\alpha_b = \alpha = \alpha_v = 0^\circ$

distance: $e_1 = 120.0$ mm, $e_3 = 99.5$ mm, $e_2 = 99.5$ mm, $e_6 = 199.0$ mm

internal forces and moments perpendicular to the connection planes

periphery beam

$N_d = 50.00$ kN, $M_d = 33.75$ kNm, $V_d = 150.00$ kN

partial internal forces and moments

internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_p = 30.75$ kNm

$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b = 129.52$ kN, $z_b = 199.0$ mm, $z_{bu} = 99.5$ mm

$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b = 179.52$ kN, $z_b = 199.0$ mm, $z_{bo} = 99.5$ mm

$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = 0.00$ kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v) = 0.00$ kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 150.00$ kN

2.2.3.2. resistance of cross-section in the periphery

elastic verification for $N = -50.00$ kN, $M_y = -30.75$ kNm, $V_z = 150.00$ kN

verification: $\sigma_v = 194.83$ N/mm² < $\sigma_{v,Rd} = 235.00$ N/mm² $\Rightarrow U_\sigma = 0.829 < 1$ ok

c/t-ratio: outstand flange: utilization $U_{c/t} = 0.306 < 1$ ok

internal compression parts: utilization $U_{c/t} = 0.106 < 1$ ok

2.2.3.3. connection capacity

transformation parameter: $\beta_j = 1.00$

2.2.3.3.1. moment resistance

distance of tension-bolt-rows from centre of compression: $h_1 = 234.5$ mm, $h_2 = 154.5$ mm, $h_3 = 44.5$ mm

resistance per bolt-row (tension)

row 1: $F_{tr,Rd} = 292.7$ kN

row 2: $F_{tr,Rd} = 292.5$ kN

row 3: $F_{tr,Rd} = 244.6$ kN

$\Sigma F_{tr,Rd}^* = 829.8$ kN

resistance per bolt-row (bending)

row 1: $F_{tr,Rd} = 292.7$ kN

row 2: $F_{tr,Rd} = 173.5$ kN

row 3: $F_{tr,Rd} = 0.0$ kN

$\Sigma F_{tr,Rd} = 466.2$ kN

potential failure by basic component 1, 4, 5

resistance of flanges

$\Sigma F_{c,Rd}^* = 932.5$ kN

moment resistance

$$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 95.5 \text{ kNm}$$

tension resistance

$$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 829.8 \text{ kN}$$

compression resistance

$$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 932.5 \text{ kN}$$

2.2.3.3.2. shear/bearing resistance**resistance per bolt-row**

$$\text{row 1: } F_{vr,Rd} = 122.8 \text{ kN}$$

$$\text{row 2: } F_{vr,Rd} = 195.7 \text{ kN}$$

$$\text{row 3: } F_{vr,Rd} = 301.6 \text{ kN}$$

$$\Sigma F_{vr,Rd} = 620.1 \text{ kN}$$

shear/bearing resistance

$$V_{j,Rd} = \Sigma F_{vr,Rd} = 620.1 \text{ kN}$$

2.2.3.3.3. shear resistance**shear resistance of end plate**

$$\text{end-plate: } V_{ep,Rd} = 412.46 \text{ kN}$$

$$\text{welds: } F_{w,Rd} = 252.74 \text{ kN}$$

$$\text{shear resistance of end plate: } V_{ep,Rd} = F_{w,Rd} = 252.74 \text{ kN}$$

shear resistance of column web

$$V_{wp,Rd} = 466.2 \text{ kN}$$

2.2.3.3.4. total

$$M_{j,Rd} = 95.5 \text{ kNm} \quad N_{j,t,Rd} = 829.8 \text{ kN} \quad N_{j,c,Rd} = 932.5 \text{ kN} \quad V_{j,Rd} = 620.1 \text{ kN} \quad V_{wp,Rd} = 466.2 \text{ kN} \quad V_{ep,Rd} = 252.7 \text{ kN}$$

2.2.3.4. verifications**2.2.3.4.1. verification of the connection capacity by means of the component method**

$$\text{axial force: } N_{b,Ed} = |N_d| = 50.00 \text{ kN} < 5\% \cdot N_{pl,Rd} = 75.60 \text{ kN} \Rightarrow \text{moment resistance}$$

$$\text{internal moment: } M_{Ed} = M_d - N_d \cdot z_{bu} = 28.77 \text{ kNm}, \quad z_{bu} = 99.5 \text{ mm}$$

$$\text{shear force: } V_{Ed} = |V_d| = 150.00 \text{ kN}$$

$$\text{shear force: } V_{c,w,Ed} = M_d/z - (V_{c1} - V_{c2})/2 = 177.02 \text{ kN}, \quad z = 190.7 \text{ mm}$$

$$\text{shear force: } V_{b,w,Ed} = 150.00 \text{ kN}$$

$$M_{Ed}/M_{j,Rd} = 0.301 < 1 \quad \text{ok}$$

$$V_{Ed}/V_{j,Rd} = 0.242 < 1 \quad \text{ok}$$

$$V_{c,w,Ed}/V_{wp,Rd} = 0.380 < 1 \quad \text{ok}$$

$$V_{b,w,Ed}/V_{ep,Rd} = 0.593 < 1 \quad \text{ok}$$

2.2.3.4.2. verification of welds at beam section

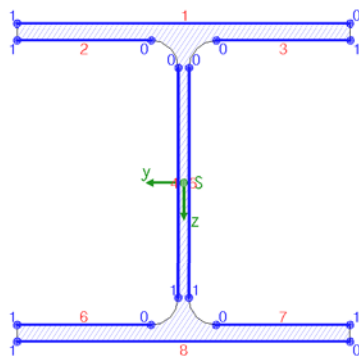
weld 1: beam flange in tension outer

welds 2,3: beam flange in tension inner

weld 8: beam flange in compression outer

welds 4,5: beam web double-sided

welds 6,7: beam flange in compression inner

calculation section:

$$\text{weld 1: } a_w = 5.0 \text{ mm} \quad l_w = 220.0 \text{ mm}$$

$$\text{weld 2: } a_w = 5.0 \text{ mm} \quad l_w = 88.5 \text{ mm}$$

$$\text{weld 3: see weld 2}$$

$$\text{weld 4: } a_w = 4.0 \text{ mm} \quad l_w = 152.0 \text{ mm}$$

$$\text{weld 5: see weld 4}$$

$$\text{weld 6: } a_w = 4.0 \text{ mm} \quad l_w = 88.5 \text{ mm}$$

$$\text{weld 7: see weld 6}$$

$$\text{weld 8: } a_w = 4.0 \text{ mm} \quad l_w = 220.0 \text{ mm}$$

design values referring to centroid of the section:

$$N_{Ed} = -50.00 \text{ kN}, \quad M_{y,Ed} = -33.75 \text{ kNm}, \quad V_{z,Ed} = 150.00 \text{ kN}$$

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

$$\Sigma A_w = 47.89 \text{ cm}^2, \quad A_{w,z} = 12.16 \text{ cm}^2, \quad \Sigma l_w = 109.8 \text{ cm}$$

$$I_{w,y} = 3791.67 \text{ cm}^4, \quad I_{w,z} = 1592.73 \text{ cm}^4, \quad \Delta z_w = -8.3 \text{ mm}$$

verifications in weld edges:

75% decrease of stress by pressure contact

$$\text{weld 1, pt. 0: } \sigma_{w,x} = 75.63 \text{ N/mm}^2$$

$$\Rightarrow U_w = 0.297 < 1 \quad \text{ok}$$

weld 2, pt. 0: $\sigma_{w,x} = 65.84 \text{ N/mm}^2 \Rightarrow U_w = 0.259 < 1$ ok
weld 4, pt. 0: $\sigma_{w,x} = 49.82 \text{ N/mm}^2$ $\tau_{w,z} = 123.36 \text{ N/mm}^2 \Rightarrow U_w = 0.625 < 1$ ok
pt. 1: $\sigma_{w,x} = -21.37 \text{ N/mm}^2$ $\tau_{w,z} = 123.36 \text{ N/mm}^2 \Rightarrow U_w = 0.599 < 1$ ok
weld 6, pt. 0: $\sigma_{w,x} = -25.37 \text{ N/mm}^2 \Rightarrow U_w = 0.100 < 1$ ok
weld 8, pt. 0: $\sigma_{w,x} = -27.82 \text{ N/mm}^2 \Rightarrow U_w = 0.109 < 1$ ok
Result:
weld 4, pt. 0: $\sigma_{w,x} = 49.82 \text{ N/mm}^2$ $\tau_{w,z} = 123.36 \text{ N/mm}^2$
Max: $\sigma_{1,w,Ed} = 224.98 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2$,
 $\sigma_{2,w,Ed} = 35.23 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U_w = 0.625 < 1$ ok

2.2.3.4.3. verification of web stiffeners

compression stiffener (below)

$F_{c,Ed} = 200.93 \text{ kN}$

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w) / b_f = 78.70 \text{ kN}$, $H = F \cdot e_F / e_H = 27.98 \text{ kN}$

assumption: stiffeners do not buckle: section class 2 ≤ 3 ok

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 235.47 \text{ kN}$

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 92.42 \text{ kN}$

$F_{Ed} = 92.42 \text{ kN} < F_{Rd} = 235.47 \text{ kN} \Rightarrow U = 0.393 < 1$ ok

cross-section at web

shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0}) = 232.82 \text{ kN}$

design value: $F_{Ed} = F = 78.70 \text{ kN}$

$F_{Ed} = 78.70 \text{ kN} < F_{Rd} = 232.82 \text{ kN} \Rightarrow U = 0.338 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 471.23 \text{ kN/m}$, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 167.56 \text{ kN/m}$, $b_1 = 83.5 \text{ mm}$

75% decrease of stress by pressure contact

$\sigma_{1,w,Ed} = 34.59 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.096 < 1$ ok

$\sigma_{2,w,Ed} = 29.45 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U = 0.114 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 275.16 \text{ kN/m}$, $l_1 = 143.0 \text{ mm}$

75% decrease of stress by pressure contact

$\sigma_{1,w,Ed} = 29.79 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.083 < 1$ ok

stiffener in tension (top)

$F_{t,Ed} = 150.93 \text{ kN}$

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w) / b_f = 59.11 \text{ kN}$, $H = F \cdot e_F / e_H = 21.02 \text{ kN}$

cross-section at flange

tension resistance $N_{t,Rd} = 235.47 \text{ kN}$

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 69.42 \text{ kN}$

$F_{Ed} = 69.42 \text{ kN} < F_{Rd} = 235.47 \text{ kN} \Rightarrow U = 0.295 < 1$ ok

cross-section at web

shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0}) = 232.82 \text{ kN}$

design value: $F_{Ed} = F = 59.11 \text{ kN}$

$F_{Ed} = 59.11 \text{ kN} < F_{Rd} = 232.82 \text{ kN} \Rightarrow U = 0.254 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 353.97 \text{ kN/m}$, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 125.87 \text{ kN/m}$, $b_1 = 83.5 \text{ mm}$

$\sigma_{1,w,Ed} = 103.93 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.289 < 1$ ok

$\sigma_{2,w,Ed} = 88.49 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U = 0.341 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 206.69 \text{ kN/m}$, $l_1 = 143.0 \text{ mm}$

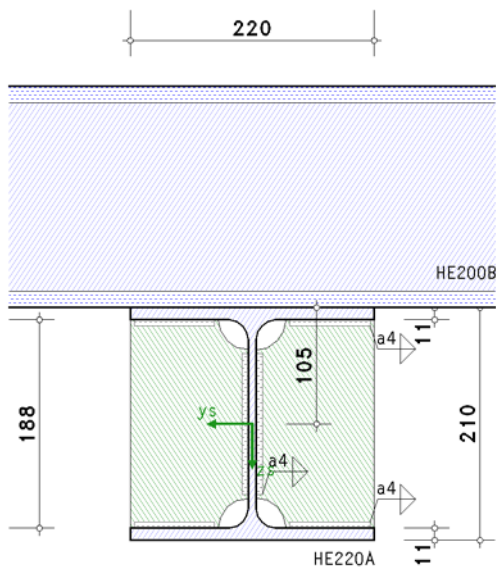
$\sigma_{1,w,Ed} = 89.50 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.249 < 1$ ok

2.2.3.4.4. verification result

maximum utilization: max $U = 0.829 < 1$ ok

2.3. local loading into bracket (ULS)

detailed problems acc. to Eurocode 3 EC 3-1-5 (10.19), NA: Deutschland



assumption: flange induced web buckling is excluded.
 assumption: plated structures-/shear buckling is excluded.
 assumption: transverse stiffeners serve as rigid support of the plated panel.
 assumption: local buckling of stiffeners is excluded.

2.3.1. table of results

Lc	$F_{z,Ed}$ kN	N_{Ed} kN	M_{Ed} kNm	V_{Ed} kN	U_r	U_σ	U
---	---	---	---	---	---	---	---
1	150.00	-50.00	5.25	150.00	0.366	0.551	0.551*

$F_{z,Ed}$: vertical single load by a load girder; N_{Ed}, M_{Ed}, V_{Ed} : design values in cross-section; U_r : utilization due to ribs
 U_σ : utilization due to stresses at first cut of web; U : utilization due to local loading

*) maximum utilization

2.3.2. final result

maximum utilization [Lc 1]: $\max U = 0.551 < 1$ ok

2.3.3. Lc 1 (decisive)

cross-sectional properties: $A = 64.34 \text{ cm}^2$, $z_s = 105.0 \text{ mm}$, $I_y = 5409.75 \text{ cm}^4$, $y_s = 0.0 \text{ mm}$, $I_z = 1954.56 \text{ cm}^4$

feed width by the beam $s_s' = 2 \cdot t_f + t_w + 1.172 \cdot r = 50.1 \text{ mm}$

feed length of load by the load beam $s_s = 2 \cdot t_f + t_w + 1.172 \cdot r = 60.1 \text{ mm}$

For info: compression $F_{z,Ed,ULS}/(s_s \cdot s_s') = 49.84 \text{ N/mm}^2$

effective loading length $l_{eff} = s_s + 2 \cdot t_f = 82.1 \text{ mm}$

length of local loading:

referring to outer edge of flange $s_s = l_{eff} - 2 \cdot t_f = 60.1 \text{ mm}$ / to periphery of web $s_w = l_{eff} + 2 \cdot r = 118.1 \text{ mm}$

2.3.3.1. compression of web (ULS)

permissible stresses: $\sigma_{Rd} = f_y / \gamma_{M0} = 235.0 \text{ N/mm}^2$, $\tau_{Rd} = f_y / (3^{1/2} \cdot \gamma_{M0}) = 135.7 \text{ N/mm}^2$

load transfer with transverse stiffeners (ribs):

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w) / b_f = 60.34 \text{ kN}$, $H = F \cdot e_F / e_H = 21.42 \text{ kN}$

load component of web $F_{c,Ed} - 2 \cdot F = 29.32 \text{ kN}$

assumption: stiffeners do not buckle: section class $1 \leq 3$ ok

note: $b_R > 102.5 \text{ mm} \Rightarrow$ end return is not possible, pay attention to corrosion protection !!

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 280.24 \text{ kN}$

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 70.84 \text{ kN}$

$F_{Ed} = 70.84 \text{ kN} < F_{Rd} = 280.24 \text{ kN} \Rightarrow U = 0.253 < 1$ ok

cross-section at web

shear resistance $V_{Rd} = (A_w \cdot f_y) / (3^{1/2} \cdot \gamma_{M0}) = 272.71 \text{ kN}$

design value: $F_{Ed} = F = 60.34 \text{ kN}$

$F_{Ed} = 60.34 \text{ kN} < F_{Rd} = 272.71 \text{ kN} \Rightarrow U = 0.221 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 379.50 \text{ kN/m}$, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 134.74 \text{ kN/m}$, $b_1 = 79.5 \text{ mm}$

$\sigma_{1,w,Ed} = 111.38 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.309 < 1$ ok

$\sigma_{2,w,Ed} = 94.88 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U = 0.366 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 225.15 \text{ kN/m}$, $l_1 = 134.0 \text{ mm}$

total: utilization of ribs $U_R = 0.366 < 1$ **ok**

compression of single load at first cut of web:

reduced transverse loading $F_{z,Ed} = 29.3 \text{ kN}$

local stresses $\sigma_{0z,Ed} = -35.5 \text{ N/mm}^2$

$$|\sigma_{OZ,Ed}| = 35.5 \text{ N/mm}^2 < \sigma_{Rd} = 235.0 \text{ N/mm}^2 \Rightarrow U = 0.151 < 1 \text{ ok}$$

stresses at first cut of web:

reduced transverse loading $F_{z,Ed} = 29.3 \text{ kN}$

Lc 1: $N_{Ed} = -50.0 \text{ kN}$, $M_{y,Ed} = 5.3 \text{ kNm}$, $V_{z,Ed} = 150.0 \text{ kN}$

stresses $\sigma_{x,Ed} = -15.1 \text{ N/mm}^2$, $\tau_{xz,Ed} = 72.6 \text{ N/mm}^2$

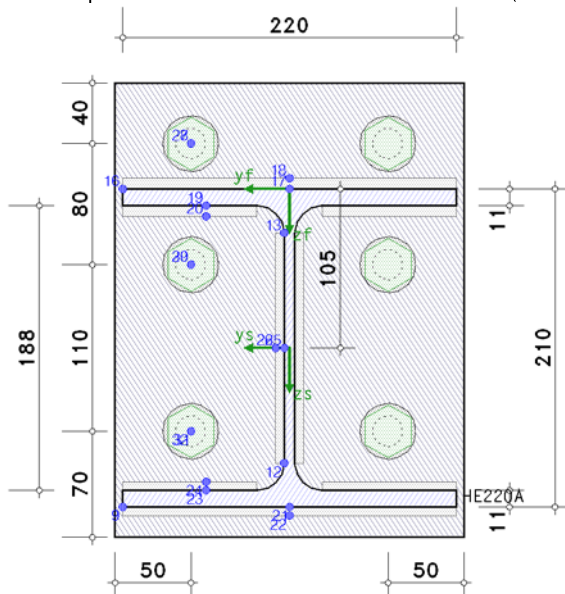
$$|\sigma_{x,Ed}| = 15.1 \text{ N/mm}^2 < \sigma_{Rd} = 235.0 \text{ N/mm}^2 \Rightarrow U = 0.064 < 1 \text{ ok}$$
$$|\tau_{xz,Ed}| = 72.6 \text{ N/mm}^2 < \tau_{Rd} = 135.7 \text{ N/mm}^2 \Rightarrow U = 0.535 < 1 \text{ ok}$$
$$\sigma_v = 129.4 \text{ N/mm}^2 < \sigma_{Rd} = 235.0 \text{ N/mm}^2 \Rightarrow U = 0.551 < 1 \text{ ok}$$

utilization at first cut of web $\max U_{\sigma} = 0.551 < 1$ ok

maximum utilization: $\max U = 0.551 < 1$ ok

2.4. fatigue of the connection bracket-column (FLS)

detailed problems acc. to Eurocode 3 EC 3-1-9 (12.10), NA: Deutschland



notch class / valid notch stresses at $N = 2 \cdot 10^6$ cycles:

Pt.	yf mm	Zf mm	$\Delta\sigma_x, Rd$ N/mm ²	$\Delta\tau Rd$ N/mm ²	$\Delta\sigma_z, Rd$ N/mm ²	notch point	EC 3-1-9, tab.
9	110.0	210.0	160.0	0.0	0.0	at bottom flange	8.1(2)
12	3.5	181.0	160.0	100.0	0.0	at beam web	8.1(2) 8.1(6)
13	3.5	29.0	160.0	100.0	160.0	at beam web	8.1(2) 8.1(6) 8.10(1)
16	110.0	0.0	160.0	0.0	0.0	at top flange	8.1(2)
° 17	0.0	0.0	36.0	0.0	0.0	am plate (top flange)	8.5(3)
18	0.0	-7.1	80.0	0.0	0.0	am plate (top flange)	8.5(1)
° 19	55.0	11.0	36.0	0.0	0.0	am plate (top flange)	8.5(3)
20	55.0	18.1	80.0	0.0	0.0	am plate (top flange)	8.5(1)
° 21	0.0	210.0	36.0	0.0	0.0	am plate (bottom flange)	8.5(3)
22	0.0	215.7	80.0	0.0	0.0	am plate (bottom flange)	8.5(1)
° 23	55.0	199.0	36.0	0.0	0.0	am plate (bottom flange)	8.5(3)
24	55.0	193.3	80.0	0.0	0.0	am plate (bottom flange)	8.5(1)
° 25	3.5	105.0	36.0	80.0	0.0	am plate (web)	8.5(3) 8.5(8)
26	9.2	105.0	0.0	100.0	0.0	am plate (web)	8.2(5)
x 27	65.0	-30.0	50.0	100.0	0.0	am plate	8.1(14) 8.1(15)
28	65.0	-30.0	90.0	0.0	0.0	am plate	8.1(10)
x 29	65.0	50.0	50.0	100.0	0.0	am plate	8.1(14) 8.1(15)
30	65.0	50.0	90.0	0.0	0.0	am plate	8.1(10)
x 31	65.0	160.0	50.0	100.0	0.0	am plate	8.1(14) 8.1(15)
32	65.0	160.0	90.0	0.0	0.0	am plate	8.1(10)

°: verification of the weld, x: verification of bolt

loading

Lc 1: $N_{Ed} = -0.00 \text{ kN}$, $M_{y,Ed} = -20.80 \text{ kNm}$, $V_{z,Ed} = 80.00 \text{ kN}$
 Lc 2: $N_{Ed} = -40.00 \text{ kN}$, $M_{y,Ed} = -24.40 \text{ kNm}$, $V_{z,Ed} = 110.00 \text{ kN}$

2.4.1. fatigue design

distances between bolts at end-plate:

horizontal: $e_2 = 50.0 \text{ mm} > 1.5 \cdot d_0 = 33.0 \text{ mm}$
 horizontal: $p_2 = 130.0 \text{ mm} > 2.5 \cdot d_0 = 55.0 \text{ mm}$
 vertical: $e_1 = 40.0 \text{ mm} > 1.5 \cdot d_0 = 33.0 \text{ mm}$
 vertical: $p_1 = 80.0 \text{ mm} > 2.5 \cdot d_0 = 55.0 \text{ mm}$
 vertical: $p_1 = 110.0 \text{ mm} > 2.5 \cdot d_0 = 55.0 \text{ mm}$
 vertical: $e_1 = 70.0 \text{ mm} > 1.5 \cdot d_0 = 33.0 \text{ mm}$

cross-sectional properties: $A = 64.34 \text{ cm}^2$, $z_s = 105.0 \text{ mm}$, $I_y = 5409.75 \text{ cm}^4$, $y_s = 0.0 \text{ mm}$, $I_z = 1954.56 \text{ cm}^4$

stress ranges:

pt. 21: $y_f = 0.0 \text{ mm}$, $z_f = 210.0 \text{ mm}$ $\Delta\sigma_{x,Ed} = 22.6 \text{ N/mm}^2$

equivalent constant amplitude stress range:

pt. 21: $y_f = 0.0 \text{ mm}$, $z_f = 210.0 \text{ mm}$ $\Delta\sigma_{x,f} = 22.6 \text{ N/mm}^2$

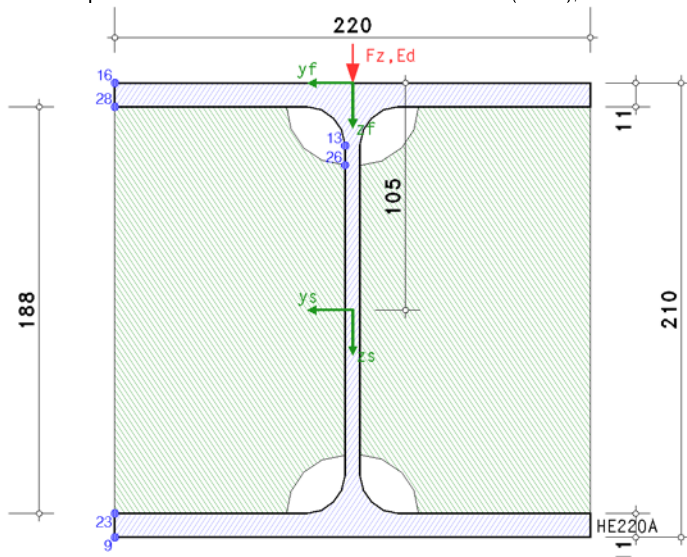
verification of notch stresses:

pt. 21: $y = 0.0 \text{ mm}$, $z = 210.0 \text{ mm}$ $\Delta\sigma_{x,f} = 22.6 \text{ N/mm}^2 < \Delta\sigma_{x,Rd,f} = 31.3 \text{ N/mm}^2 \Rightarrow U_{\Delta\sigma x} = 0.721$ ok

fatigue design [pt. 21]: $\max U = 0.721 < 1$ ok

2.5. fatigue of bracket cross-section (FLS)

detailed problems acc. to Eurocode 3 EC 3-1-9 (12.10), NA: Deutschland



notch class / valid notch stresses at $N = 2 \cdot 10^6$ cycles:

Pt.	y_f mm	z_f mm	$\Delta\sigma_{x,Rd}$ N/mm ²	$\Delta\tau_{Rd}$ N/mm ²	$\Delta\sigma_{z,Rd}$ N/mm ²	notch point	EC 3-1-9, tab.
9	110.0	210.0	160.0	0.0	0.0	at bottom flange	8.1(2)
13	3.5	29.0	160.0	100.0	160.0	at beam web	8.1(2) 8.1(6) 8.10(1)
16	110.0	0.0	160.0	0.0	0.0	at top flange	8.1(2)
23	110.0	199.0	80.0	0.0	0.0	due to transv.stiff.	8.4(7)
26	3.5	38.0	0.0	100.0	0.0	due to transv.stiff.	8.1(6)
28	110.0	11.0	80.0	0.0	0.0	due to transv.stiff.	8.4(7)

loading

Lc 1: $N_{Ed} = -0.00 \text{ kN}$, $M_{y,Ed} = 0.00 \text{ kNm}$, $V_{z,Ed} = 80.00 \text{ kN}$
 Lc 2: $N_{Ed} = -40.00 \text{ kN}$, $M_{y,Ed} = 4.20 \text{ kNm}$, $V_{z,Ed} = 110.00 \text{ kN}$
 transverse loading on top flange: vertical single load $F_{z,Ed} = 110.0 \text{ kN}$

2.5.1. fatigue design

cross-sectional properties: $A = 64.34 \text{ cm}^2$, $z_s = 105.0 \text{ mm}$, $I_y = 5409.75 \text{ cm}^4$, $y_s = 0.0 \text{ mm}$, $I_z = 1954.56 \text{ cm}^4$

effective loading length $l_{eff} = s_s + 2 \cdot t_f = 82.1 \text{ mm}$

... at beam web $\sigma_{oz} = -133.1 \text{ N/mm}^2$, $\tau_o = 26.6 \text{ N/mm}^2$

stress ranges:

pt. 13: $y_f = 3.5 \text{ mm}$, $z_f = 29.0 \text{ mm}$ $\Delta\sigma_{x,Ed} = 12.1 \text{ N/mm}^2$ $\Delta\tau_{Ed} = 74.1 \text{ N/mm}^2$ $\Delta\sigma_{z,Ed} = 133.1 \text{ N/mm}^2$

equivalent constant amplitude stress range:

pt. 13: $y_f = 3.5 \text{ mm}$, $z_f = 29.0 \text{ mm}$

$\Delta\sigma_{x,f} = 12.1 \text{ N/mm}^2$

$\Delta\tau_f = 74.1 \text{ N/mm}^2$

$\Delta\sigma_{z,f} = 133.1 \text{ N/mm}^2$

verification of notch stresses:

pt. 13: $y = 3.5 \text{ mm}$, $z = 29.0 \text{ mm}$

$\Delta\sigma_{x,f} = 12.1 \text{ N/mm}^2 < \Delta\sigma_{x,Rd,f} = 139.1 \text{ N/mm}^2 \Rightarrow U_{\Delta\sigma x} = 0.087 \text{ ok}$

$\Delta\tau_f = 74.1 \text{ N/mm}^2 < \Delta\tau_{Rd,f} = 87.0 \text{ N/mm}^2 \Rightarrow U_{\Delta\tau} = 0.852 \text{ ok}$

$\Delta\sigma_{z,f} = 133.1 \text{ N/mm}^2 < \Delta\sigma_{z,Rd,f} = 139.1 \text{ N/mm}^2 \Rightarrow U_{\Delta\sigma z} = 0.956 \text{ ok}$

interaction $U_i = U_{\Delta\sigma x}^3 + U_{\Delta\sigma z}^3 + U_{\Delta\tau}^5 = 1.324 > 1 \text{ not ok !!}$

fatigue design [pt. 13]:

$\max U = 0.956 < 1 \text{ ok}$

3. final result

maximum utilization:

$\max U = 0.956 < 1 \text{ ok}$

fatigue of the connection / of the section

verification succeeded

4. Regulations

EN 1990, Eurocode 0: Grundlagen der Tragwerksplanung;

Deutsche Fassung EN 1990:2002 + A1:2005 + A1:2005/AC:2010, Ausgabe Dezember 2010

EN 1990/NA, Nationaler Anhang zur EN 1990, Ausgabe Dezember 2010

EN 1993-1-1, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -

Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau;

Deutsche Fassung EN 1993-1-1:2005 + AC:2009, Ausgabe Dezember 2010

EN 1993-1-1/A1, Ergänzungen zur EN 1993-1-1, Ausgabe Juli 2014

EN 1993-1-1/NA, Nationaler Anhang zur EN 1993-1-1, Ausgabe Dezember 2018

EN 1993-1-8, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -

Teil 1-8: Bemessung von Anschlüssen;

Deutsche Fassung EN 1993-1-8:2005 + AC:2009, Ausgabe Dezember 2010

EN 1993-1-8/NA, Nationaler Anhang zur EN 1993-1-8, Ausgabe Dezember 2010

EN 1993-1-5, Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-5: Plattenförmige Bauteile;

Deutsche Fassung EN 1993-1-5:2006 + AC:2009 + A1:2017 + A2:2019, Ausgabe Oktober 2019

EN 1993-1-5/NA, Nationaler Anhang zur EN 1993-1-5, Ausgabe Dezember 2010

EN 1993-1-9, Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-9: Ermüdung;

Deutsche Fassung EN 1993-1-9:2006 + AC:2009, Ausgabe Dezember 2010

EN 1993-1-9/NA, Nationaler Anhang zur EN 1993-1-9, Ausgabe Dezember 2010