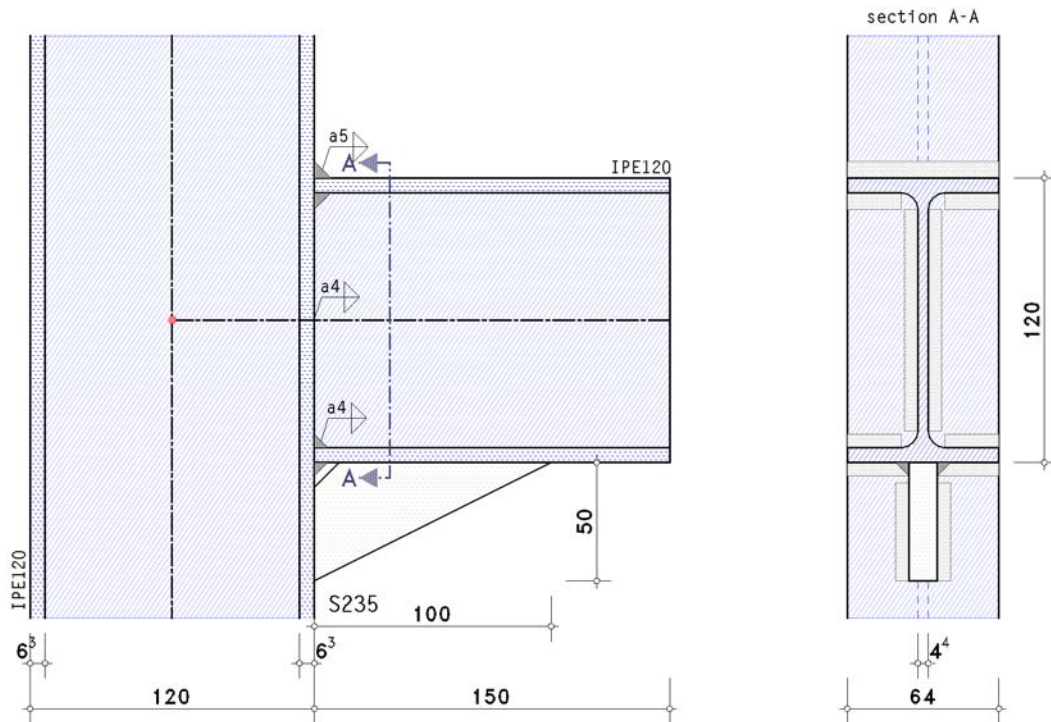
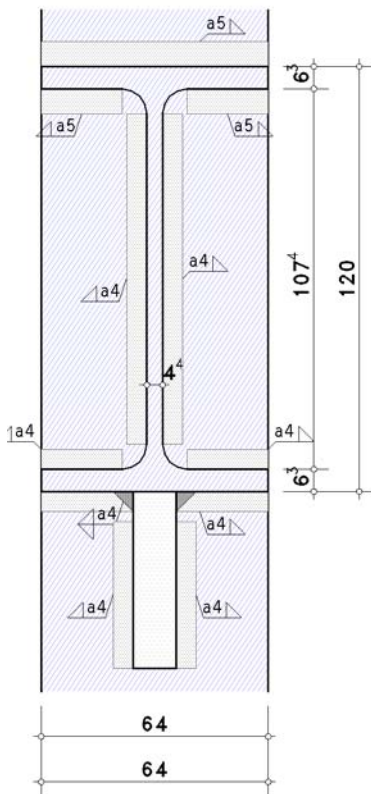


1. input report



details (section A - A)



steel grade

steel grade S235

column parameters

section IPE120

beam parameters

section IPE120

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

triangular rib (below)

thickness $t_R = 12.0$ mm, width $b_R = 100.0$ mm, length $l_R = 50.0$ mm
 welds an column and beam: fillet weld, weld thickness $a_R = 4.0$ mm

parameters of the connection

welded connection

distance of transverse loading $\Delta a = 130.0$ mm

verification of local loading:

transverse loading on top flange of bracket with loading length $s_s = 30.0$ mm

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	$F_{1,Ed}$
--	kN
1	40.00

$F_{1,Ed}$: loading of bracket

bracket load (FLS)

Lc	$F_{1,Ed}$
--	kN
1	5.00
2	15.00

$F_{1,Ed}$: loading of bracket

partial safety factors for material (ULS)

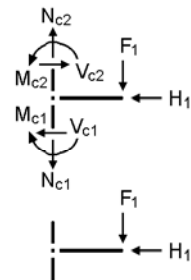
resistance of cross-sections $\gamma_{M0} = 1.00$

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	$U_{j,b}$	$U_{j,\sigma}$	$U_{j,\tau}$	$U_{j,w}$	$U_{j,r}$	$U_{l,\sigma}$	$U_{l,b}$	$U_{l,i}$	U
--	---	---	---	---	---	---	---	---	---	---	---
1	0.696	0.884	0.640	0.696	0.594	0.448	0.147	0.828	0.575	0.884	0.884*

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

$U_{j,\sigma}$: utilization due to bending; $U_{j,\tau}$: utilization due to shear force; $U_{j,w}$: utilization due to weld

$U_{j,r}$: utilization due to stiffeners/ribs; $U_{l,\sigma}$: utilization due to stresses at first cut of web; $U_{l,b}$: utilization due to buckling of transverse loading

$U_{l,i}$: utilization due to interaction; U : utilization of the connection

*) maximum utilization

2.2. connection to column (ULS)

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

check of data

ok

2.2.1. table of results

utilization

Lc	$U_{\sigma,b}$	U_m	U_{wp}	U_{cf}	U_{sb}	U_{ss}	U
--	---	---	---	---	---	---	---
1	0.640	0.696	0.594	0.501	0.448	0.147	0.696*

$U_{\sigma,b}$: stress utilization at the beam; U_m : utilization due to bending; U_{wp} : utilization due to shear in column web

U_{cf} : utilization due to shear in column flange; U_{sb} : utilization due to weld; U_{ss} : utilization due to stiffeners/ribs

U : utilization of the connection

*) maximum utilization

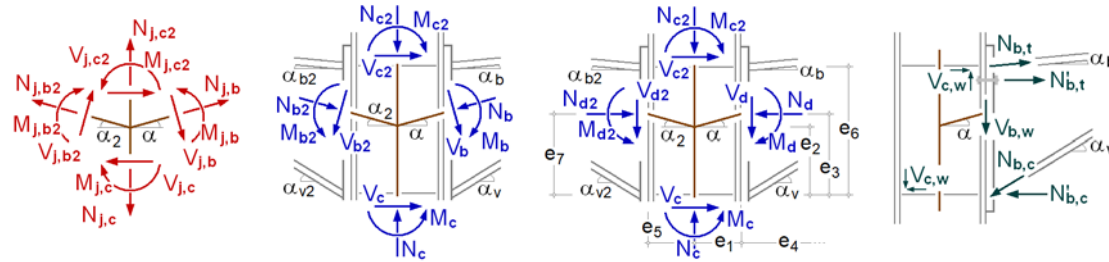
2.2.2. final result

maximum utilization: max $U = 0.696 < 1$ ok

2.2.3. Lc 1 (decisive)

2.2.3.1. design values

internal forces at node periphery connection $\perp$ zur connection plane partial internal forces and moments



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

distance: $e_1 = 60.0 \text{ mm}$, $e_3 = 56.8 \text{ mm}$, $e_2 = 56.8 \text{ mm}$, $e_6 = 113.7 \text{ mm}$

internal forces and moments perpendicular to the connection planes

periphery beam

$M_d = 5.20 \text{ kNm}$, $V_d = 40.00 \text{ kN}$

partial internal forces and moments

$N_{b,t} = -N_d \cdot z_{bu}/z_b + M_d/z_b = 45.73 \text{ kN}$, $z_b = 113.7 \text{ mm}$, $z_{bu} = 56.9 \text{ mm}$

$N_{b,c} = N_d \cdot z_{bo}/z_b + M_d/z_b = 45.73 \text{ kN}$, $z_b = 113.7 \text{ mm}$, $z_{bo} = 56.9 \text{ mm}$

$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = 0.00 \text{ kN}$, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v) = 0.00 \text{ kN}$, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 40.00 \text{ kN}$

2.2.3.2. resistance of cross-section in the periphery

elastic verification for $M_y = -5.20 \text{ kNm}$, $V_z = 40.00 \text{ kN}$

verification: $\sigma_v = 150.35 \text{ N/mm}^2 < \sigma_{v,Rd} = 235.00 \text{ N/mm}^2 \Rightarrow U_\sigma = 0.640 < 1$ ok

c/t-ratio: outstand flange: utilization $U_{c/t} = 0.165 < 1$ ok

internal compression parts: utilization $U_{c/t} = 0.098 < 1$ ok

2.2.3.3. connection capacity

transformation parameter: $\beta_j = 1.00$

2.2.3.3.1. moment resistance

distance between tension force and centre of compression: $z = 113.7 \text{ mm}$

resistance

$F_{Rd} = 65.7 \text{ kN}$

resistance of flanges (compression)

$\Sigma F_{c,Rd}^* = 131.4 \text{ kN}$

moment resistance

$M_{j,Rd} = F_{Rd} \cdot z = 7.5 \text{ kNm}$

tension resistance

$N_{j,t,Rd} = F_{t,Rd} = 73.9 \text{ kN}$

compression resistance

$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 131.4 \text{ kN}$

2.2.3.3.2. shear resistance

shear resistance of column flange

$V_{cf,Rd} = 79.84 \text{ kN}$

shear resistance of column web

$V_{wp,Rd} = 77.0 \text{ kN}$

2.2.3.3.3. total

$M_{j,Rd} = 7.5 \text{ kNm}$ $N_{j,t,Rd} = 73.9 \text{ kN}$ $N_{j,c,Rd} = 131.4 \text{ kN}$ $V_{wp,Rd} = 77.0 \text{ kN}$ $V_{cf,Rd} = 79.8 \text{ kN}$

2.2.3.4. verifications

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

internal moment: $M_{Ed} = M_d = 5.20 \text{ kNm}$

shear force: $V_{Ed} = |V_d| - \Delta V_R = 32.00 \text{ kN}$, $\Delta V_R = 8.00 \text{ kN}$: shear forceanteil der triangular rib

shear force: $V_{c,w,Ed} = M_d/z - (V_{c1} - V_{c2})/2 = 45.73 \text{ kN}$, $z = 113.7 \text{ mm}$

shear force: $V_{b,w,Ed} = 40.00 \text{ kN}$

$M_{Ed}/M_{j,Rd} = 0.696 < 1$ ok

$V_{c,w,Ed}/V_{wp,Rd} = 0.594 < 1$ ok

$V_{b,w,Ed}/V_{cf,Rd} = 0.501 < 1$ 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

welds 4,5: beam web double-sided

weld 8: beam flange in compression outer

welds 6,7: beam flange in compression inner

weld 1: weld thickness $a = 5.0 \text{ mm} > a_{\max} = 0.7 \cdot t_{\min} = 4.4 \text{ mm}$ (welding technology, s. DIN 18800) !!

weld 2: weld thickness $a = 5.0 \text{ mm} > a_{\max} = 0.7 \cdot t_{\min} = 4.4 \text{ mm}$ (welding technology, s. DIN 18800) !!

weld 2: effective weld length $l_{\text{eff}} = 22.8 \text{ mm} < 30 \text{ mm} \Rightarrow$ has no effect !!

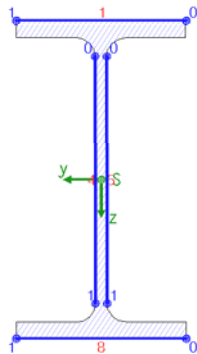
weld 2: effective weld length $l_{\text{eff}} = 22.8 \text{ mm} < 6 \cdot a = 30.0 \text{ mm} \Rightarrow$ has no effect !!

weld 4: weld thickness $a = 4.0 \text{ mm} > a_{\max} = 0.7 \cdot t_{\min} = 3.1 \text{ mm}$ (welding technology, s. DIN 18800) !!

weld 6: effective weld length $l_{\text{eff}} = 22.8 \text{ mm} < 30 \text{ mm} \Rightarrow$ has no effect !!

weld 6: effective weld length $l_{\text{eff}} = 22.8 \text{ mm} < 6 \cdot a = 24.0 \text{ mm} \Rightarrow$ has no effect !!

calculation section:



weld 1: $a_w = 5.0 \text{ mm}$ $l_w = 64.0 \text{ mm}$

weld 4: $a_w = 4.0 \text{ mm}$ $l_w = 93.4 \text{ mm}$

weld 5: see weld 4

weld 8: $a_w = 4.0 \text{ mm}$ $l_w = 64.0 \text{ mm}$

design values referring to centroid of the section:

$M_{y,Ed} = -5.20 \text{ kNm}$, $V_{z,Ed} = 40.00 \text{ kN}$

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

$\Sigma A_w = 13.23 \text{ cm}^2$, $A_{w,z} = 7.47 \text{ cm}^2$, $\Sigma l_w = 31.5 \text{ cm}$

$I_{w,y} = 260.56 \text{ cm}^4$, $I_{w,z} = 20.02 \text{ cm}^4$, $\Delta z_w = -2.9 \text{ mm}$

verifications in weld edges:

75% decrease of stress by pressure contact

weld 1, pt. 0: $\sigma_{w,x} = 113.95 \text{ N/mm}^2 \Rightarrow U_w = 0.448 < 1$ ok

weld 4, pt. 0: $\sigma_{w,x} = 87.41 \text{ N/mm}^2$ $\tau_{w,z} = 53.53 \text{ N/mm}^2 \Rightarrow U_w = 0.429 < 1$ ok

pt. 1: $\sigma_{w,x} = -24.75 \text{ N/mm}^2$ $\tau_{w,z} = 53.53 \text{ N/mm}^2 \Rightarrow U_w = 0.275 < 1$ ok

weld 8, pt. 0: $\sigma_{w,x} = -31.38 \text{ N/mm}^2 \Rightarrow U_w = 0.123 < 1$ ok

Result:

weld 1, pt. 0: $\sigma_{w,x} = 113.95 \text{ N/mm}^2$

Max: $\sigma_{1,w,Ed} = 161.15 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2$,

$\sigma_{2,w,Ed} = 80.57 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U_w = 0.448 < 1$ ok

2.2.3.4.3. verification of web stiffeners

triangular rib (compression)

$F_{c,Ed} = 9.37 \text{ kN}$

forces

$F = F_{c,Ed} = 9.37 \text{ kN}$, $H = F \cdot e_F/e_H = 3.94 \text{ kN}$

assumption: stiffeners do not buckle: section class 1 ≤ 3 ok

cross-section at the column

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 117.07 \text{ kN}$

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 11.59 \text{ kN}$

$F_{Ed} = 11.59 \text{ kN} < F_{Rd} = 117.07 \text{ kN} \Rightarrow U = 0.099 < 1$ ok

cross-section at the beam

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 258.07 \text{ kN}$

design value: $F_{Ed} = (H^2 + 3 \cdot F^2)^{1/2} = 16.70 \text{ kN}$

$F_{Ed} = 16.70 \text{ kN} < F_{Rd} = 258.07 \text{ kN} \Rightarrow U = 0.065 < 1$ ok

welds at the column

resistance of a weld: $F_{w,Rd} = 831.38 \text{ kN/m}$

design value: $F_{Ed} = (F^2 + H^2)^{1/2} / (2 \cdot b_1) = 122.42 \text{ kN/m}$, $b_1 = 41.5 \text{ mm}$

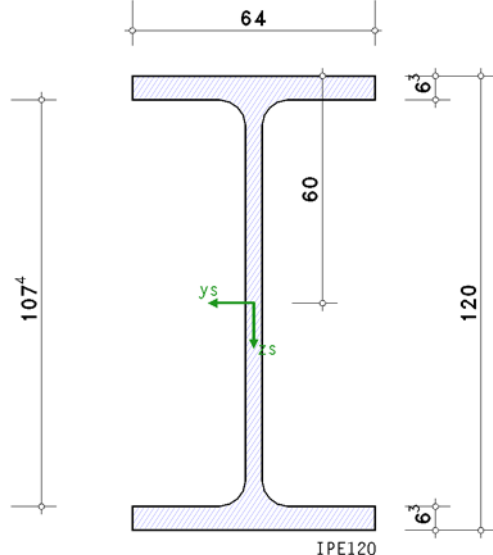
$F_{Ed} = 122.42 \text{ kN/m} < F_{Rd} = 831.38 \text{ kN/m} \Rightarrow U = 0.147 < 1$ **ok**
 welds at the beam
 resistance of a weld: $F_{w,Rd} = 831.38 \text{ kN/m}$
 two-sided joint of ribs:
 design value: $F_{Ed} = (F^2 + H^2)^{1/2} / (2 \cdot l_1) = 55.53 \text{ kN/m}$, $l_1 = 91.5 \text{ mm}$
 $F_{Ed} = 55.53 \text{ kN/m} < F_{Rd} = 831.38 \text{ kN/m} \Rightarrow U = 0.067 < 1$ **ok**

2.2.3.4.4. verification result

maximum utilization: $\max U = 0.696 < 1$ **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.

2.3.1. table of results

Lc	$F_{z,Ed}$ kN	V_{Ed} kN	U_σ	U_b	U_i	U
---	---	---	---	---	---	---
1	40.00	40.00	0.828	0.575	0.884	0.884*

$F_{z,Ed}$: vertical single load by a load girder; V_{Ed} : design values in cross-section; U_σ : utilization due to stresses at first cut of web
 U_b : utilization due to buckling of transverse loading; U_i : utilization due to interaction; U : utilization due to local loading

*) maximum utilization

2.3.2. final result

maximum utilization [Lc 1]: $\max U = 0.884 < 1$ **ok**

2.3.3. Lc 1 (decisive)

cross-sectional properties: $A = 13.21 \text{ cm}^2$, $z_s = 60.0 \text{ mm}$, $I_y = 317.76 \text{ cm}^4$, $y_s = 0.0 \text{ mm}$, $I_z = 27.67 \text{ cm}^4$

feed width by the beam $s_s' = 2 \cdot t_f + t_w + 1.172 \cdot r = 25.2 \text{ mm}$

For info: compression $F_{z,Ed,ULS} / (s_s \cdot s_s') = 52.91 \text{ N/mm}^2$

effective loading length $l_{eff} = s_s + 2 \cdot t_f = 42.6 \text{ mm}$

length of local loading:

referring to outer edge of flange $s_s = l_{eff} - 2 \cdot t_f = 30.0 \text{ mm}$ / to periphery of web $s_w = l_{eff} + 2 \cdot r = 56.6 \text{ mm}$

2.3.3.1. compression of web (ULS)

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

compression of single load at first cut of web:

local stresses $\sigma_{\sigma z,Ed} = -160.6 \text{ N/mm}^2$

$|\sigma_{\sigma z,Ed}| = 160.6 \text{ N/mm}^2 < \sigma_{Rd} = 235.0 \text{ N/mm}^2 \Rightarrow U = 0.683 < 1$ **ok**

stresses at first cut of web:

transverse loading $F_{z,Ed} = 40.0 \text{ kN}$

shear buckling: $h_p / t_p = 24.41 \leq 72 \cdot \epsilon / \eta = 60.00$ **ok**

Lc 1: $V_{z,Ed} = 40.0 \text{ kN}$

stresses $\tau_{xz,Ed} = 63.4 \text{ N/mm}^2$

$|\tau_{xz,Ed}| = 63.4 \text{ N/mm}^2 < \tau_{Rd} = 135.7 \text{ N/mm}^2 \Rightarrow U = 0.468 < 1$ ok
 $\sigma_V = 194.6 \text{ N/mm}^2 < \sigma_{Rd} = 235.0 \text{ N/mm}^2 \Rightarrow U = 0.828 < 1$ ok
 utilization at first cut of web $\max U_\sigma = 0.828 < 1$ ok

maximum utilization: $\max U = 0.828 < 1$ ok

2.3.3.2. buckling of transverse loading (ULS)

slenderness $\lambda_F = (F_y/F_{cr})^{1/2} = 0.326$, $F_y = 76.5 \text{ kN}$

reduction factor $\chi_F = 1.000$

resistance of buckling $F_{z,Rd} = f_y \cdot L_{eff} \cdot t_w / \gamma_{M1} = 69.59 \text{ kN}$, $L_{eff} = \chi_F \cdot l_y = 74.0 \text{ mm}$, $l_y = 74.0 \text{ mm}$

verification: $F_{z,Ed}/F_{z,Rd} = 0.575 < 1$ ok

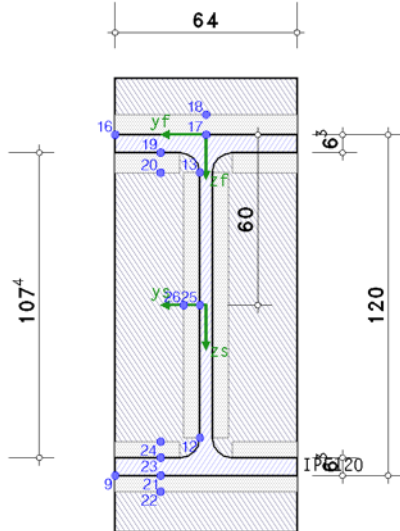
interaction (without plated structures-/shear buckling):

transverse loading and equivalent stress $(\eta_2 + 0.8 \cdot \eta_1) / 1.4 = 0.884 < 1$ ok

with $\eta_2 = F_{z,Ed}/F_{z,Rd} = 0.575$, $\eta_1 = \max U_\sigma = 0.828$

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.	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	32.0	120.0	160.0	0.0	0.0	at bottom flange	8.1(2)
12	2.2	106.7	160.0	100.0	0.0	at beam web	8.1(2) 8.1(6)
13	2.2	13.3	160.0	100.0	160.0	at beam web	8.1(2) 8.1(6) 8.10(1)
16	32.0	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	16.0	6.3	36.0	0.0	0.0	am plate (top flange)	8.5(3)
20	16.0	13.4	80.0	0.0	0.0	am plate (top flange)	8.5(1)
° 21	16.0	120.0	36.0	0.0	0.0	am plate (bottom flange)	8.5(3)
22	16.0	125.7	80.0	0.0	0.0	am plate (bottom flange)	8.5(1)
° 23	16.0	113.7	36.0	0.0	0.0	am plate (bottom flange)	8.5(3)
24	16.0	108.0	80.0	0.0	0.0	am plate (bottom flange)	8.5(1)
° 25	2.2	60.0	36.0	80.0	0.0	am plate (web)	8.5(3) 8.5(8)
26	7.9	60.0	0.0	100.0	0.0	am plate (web)	8.2(5)

°: verification of the weld

loading

Lc 1: $M_{y,Ed} = -0.65 \text{ kNm}$, $V_{z,Ed} = 5.00 \text{ kN}$

Lc 2: $M_{y,Ed} = -1.95 \text{ kNm}$, $V_{z,Ed} = 15.00 \text{ kN}$

2.4.1. fatigue design

cross-sectional properties: $A = 13.21 \text{ cm}^2$, $z_s = 60.0 \text{ mm}$, $I_y = 317.76 \text{ cm}^4$, $y_s = 0.0 \text{ mm}$, $I_z = 27.67 \text{ cm}^4$

stress ranges:

pt. 21: $y_f = 16.0 \text{ mm}$, $z_f = 120.0 \text{ mm}$ $\Delta\sigma_{x,Ed} = 27.1 \text{ N/mm}^2$

equivalent constant amplitude stress range:

pt. 21: $y_f = 16.0 \text{ mm}$, $z_f = 120.0 \text{ mm}$ $\Delta\sigma_{x,f} = 27.1 \text{ N/mm}^2$

verification of notch stresses:

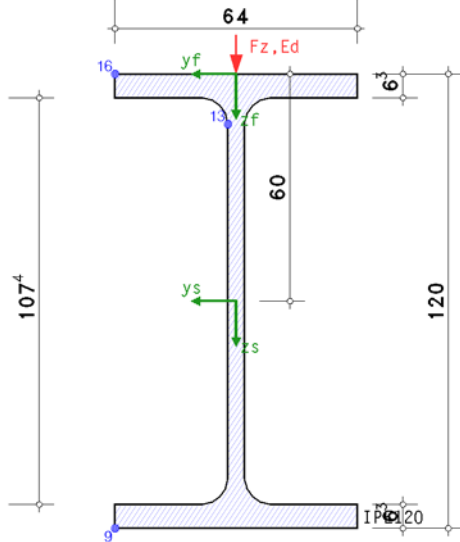


pt. 21: $y = 16.0 \text{ mm}$, $z = 120.0 \text{ mm}$ $\Delta\sigma_{x,f} = 27.1 \text{ N/mm}^2 < \Delta\sigma_{x,Rd,f} = 31.3 \text{ N/mm}^2 \Rightarrow U_{\Delta\sigma x} = 0.867$ **ok**

fatigue design [pt. 21]: $\max U = 0.867 < 1$ **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	32.0	120.0	160.0	0.0	0.0	at bottom flange	8.1(2)
13	2.2	13.3	160.0	100.0	160.0	at beam web	8.1(2) 8.1(6) 8.10(1)
16	32.0	0.0	160.0	0.0	0.0	at top flange	8.1(2)

loading

Lc 1: $V_{z,Ed} = 5.00 \text{ kN}$

Lc 2: $V_{z,Ed} = 15.00 \text{ kN}$

transverse loading on top flange: vertical single load $F_{z,Ed} = 15.0 \text{ kN}$

2.5.1. fatigue design

cross-sectional properties: $A = 13.21 \text{ cm}^2$, $z_s = 60.0 \text{ mm}$, $I_y = 317.76 \text{ cm}^4$, $y_s = 0.0 \text{ mm}$, $I_z = 27.67 \text{ cm}^4$

effective loading length $l_{eff} = s_s + 2 \cdot t_f = 42.6 \text{ mm}$

... at beam web $\sigma_{oz} = -60.2 \text{ N/mm}^2$, $\tau_o = 12.0 \text{ N/mm}^2$

stress ranges:

pt. 13: $y_f = 2.2 \text{ mm}$, $z_f = 13.3 \text{ mm}$ $\Delta\tau_{Ed} = 42.3 \text{ N/mm}^2$ $\Delta\sigma_{z,Ed} = 60.2 \text{ N/mm}^2$

equivalent constant amplitude stress range:

pt. 13: $y_f = 2.2 \text{ mm}$, $z_f = 13.3 \text{ mm}$ $\Delta\tau_f = 42.3 \text{ N/mm}^2$ $\Delta\sigma_{z,f} = 60.2 \text{ N/mm}^2$

verification of notch stresses:

pt. 13: $y = 2.2 \text{ mm}$, $z = 13.3 \text{ mm}$ $\Delta\tau_f = 42.3 \text{ N/mm}^2 < \Delta\tau_{Rd,f} = 87.0 \text{ N/mm}^2 \Rightarrow U_{\Delta\tau} = 0.487$ **ok**
 $\Delta\sigma_{z,f} = 60.2 \text{ N/mm}^2 < \Delta\sigma_{z,Rd,f} = 139.1 \text{ N/mm}^2 \Rightarrow U_{\Delta\sigma z} = 0.433$ **ok**

fatigue design [pt. 13]: $\max U = 0.487 < 1$ **ok**

3. final result

maximum utilization: $\max U = 0.884 < 1$ **ok**
 local loading into bracket

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