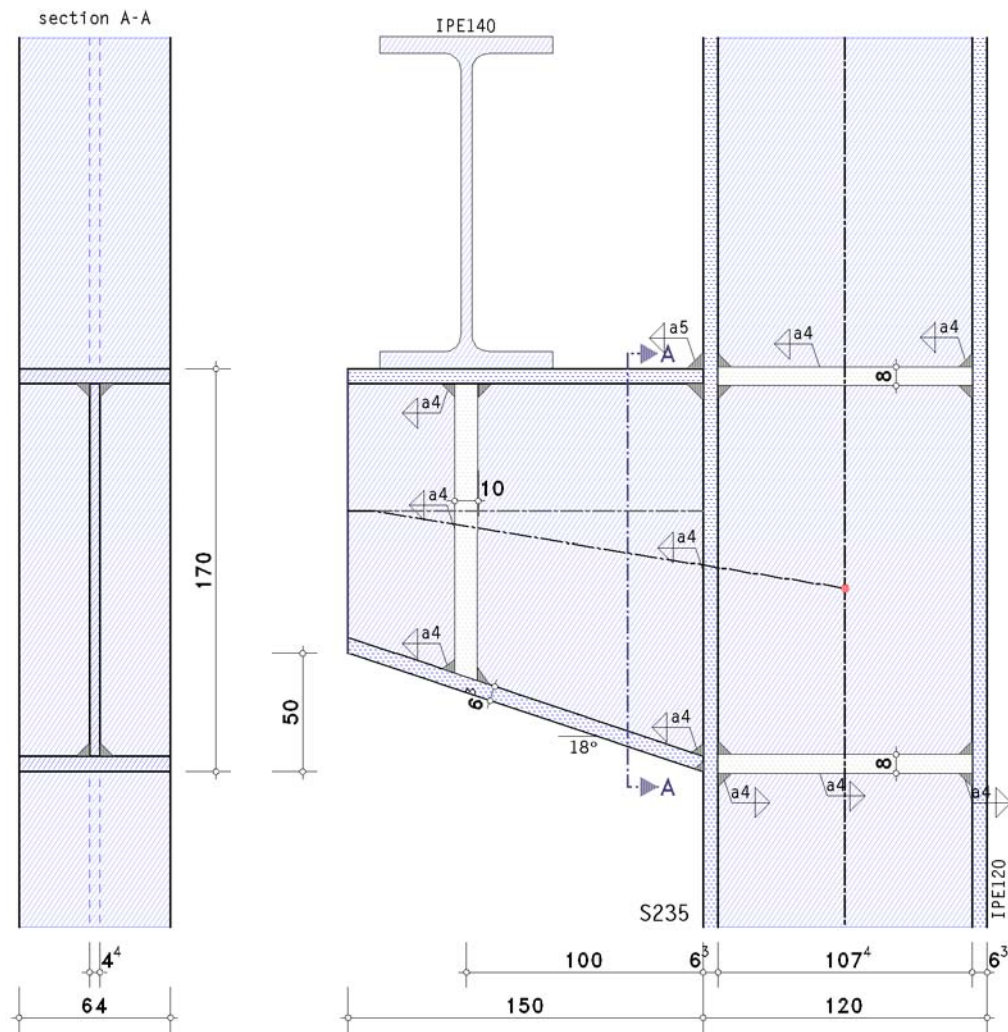
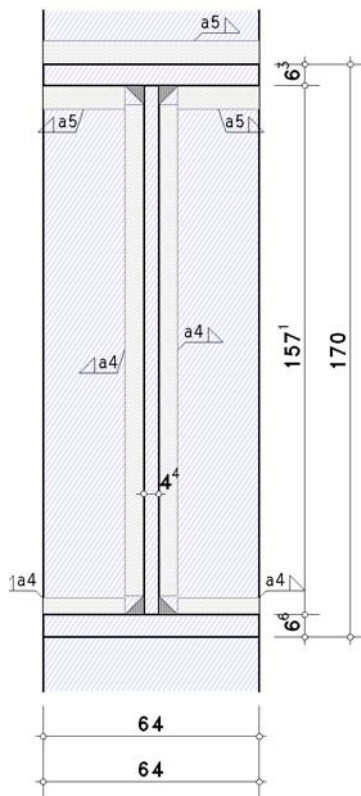


1. input report





steel grade

steel grade S235

column parameters

section IPE120

reinforcement of the section with transverse stiffeners (web stiffeners, $d_{st} = 163.5$ mm):

thickness $t_{st} = 8.0$ mm, width $b_{st} = 29.8$ mm, length $l_{st} = 107.4$ mm

recess at stiffeners $c_{st} = 10.5$ mm

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

beam parameters

parameter (I-section):

overall depth $h = 120.0$ mm, web thickness $t_w = 4.4$ mm

flange width $b_f = 64.0$ mm, flange thickness $t_f = 6.3$ mm

fillet weld $a = 4.0$ mm

welded section, thickness of fillet weld $a = 4.0$ mm

haunch: slope angle towards the horizontal $\alpha_v = 18.43^\circ$

haunch length $L_v = 150.0$ mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b)) = 50.0$ mm

total beam depth at the connection point $h_{ges} = h_b + h_v = 170.0$ mm

reinforcement of the section with transverse stiffeners:

thickness $t_{st} = 10.0$ mm, width $b_{st} = 29.8$ mm, length $l_{st} = 107.4$ mm

recess at stiffeners $c_{st} = 8.5$ mm

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

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, angle $\varphi = 108^\circ$

75% of compressive stress is transferred by contact

parameters of the connection

welded connection

distance of transverse loading $\Delta a = 100.0$ mm

verification of local loading:

transverse loading on top flange of bracket by a load girder (crossing of girders) section IPE140

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
--	--
1	40.00

F_{2,Ed}: loading of bracket

bracket load (FLS)

Lc	F _{2,Ed} kN
--	--
1	5.00
2	15.00

F_{2,Ed}: loading of bracket

partial safety factors for material (ULS)

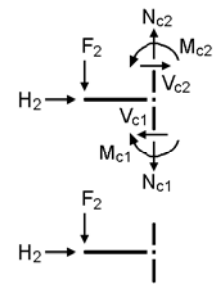
resistance of cross-sections $\gamma_{M0} = 1.00$

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	0.434	0.616	0.434	0.381	0.303	0.258	0.324	0.616	0.453	0.616*

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

notes

no verification of haunch connection to beam.

no verification for welds of welded section.

check of data

ok

2.2.1. table of results

utilization

Lc	U _{σ,b}	U _m	U _{wp}	U _{cf}	U _{sb}	U _{ss}	U
--	--	--	--	--	--	--	--
1	0.434	0.381	0.303	0.256	0.258	0.324	0.434*

U_{σ,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

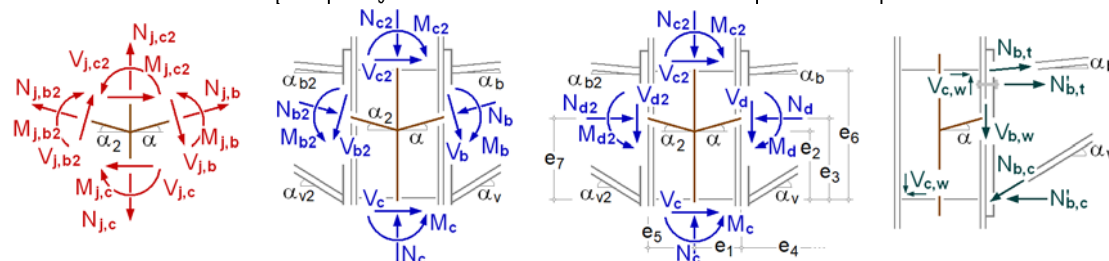
maximum utilization: max U = 0.434 < 1 ok

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: $\alpha_b = 0.00^\circ$, $\alpha_v = 18.43^\circ \Rightarrow \alpha = (\alpha_b + \alpha_v)/2 = 9.22^\circ$

distance: $e_1 = 60.0$ mm, $e_3 = 83.9$ mm, $e_2 = 74.2$ mm, $e_6 = 163.5$ mm

internal forces and moments perpendicular to the connection planes

periphery beam

$N_d = -12.65 \text{ kN}$, $M_d = 4.00 \text{ kNm}$, $V_d = 37.95 \text{ kN}$

partial internal forces and moments

$N_{b,t} = -N_d \cdot z_{bu}/z_b + M_d/z_b = 30.70 \text{ kN}$, $z_b = 163.5 \text{ mm}$, $z_{bu} = 80.6 \text{ mm}$

$N_{b,c} = (N_d \cdot z_{bo}/z_b + M_d/z_b) / \cos(\alpha_v) = 19.02 \text{ kN}$, $z_b = 163.5 \text{ mm}$, $z_{bo} = 82.9 \text{ mm}$

$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = 0.00 \text{ kN}$, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v) = 6.02 \text{ kN}$, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 31.93 \text{ kN}$

2.2.3.2. resistance of cross-section in the periphery

elastic verification for $N = 12.65 \text{ kN}$, $M_y = -4.00 \text{ kNm}$, $V_z = 37.95 \text{ kN}$

verification: $\sigma_v = 102.01 \text{ N/mm}^2 < \sigma_{v,Rd} = 235.00 \text{ N/mm}^2 \Rightarrow U_\sigma = 0.434 < 1$ **ok**

c/t-ratio: outstand flange: utilization $U_{c/t} = 0.112 < 1$ **ok**

internal compression parts: utilization $U_{c/t} = 0.110 < 1$ **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: $z = 163.5 \text{ mm}$

resistance

$F_{Rd} = 80.6 \text{ kN}$

resistance of flanges (compression)

$\Sigma F_{c,Rd}^* = 161.3 \text{ kN}$

moment resistance

$M_{j,Rd} = F_{Rd} \cdot z = 13.2 \text{ kNm}$

tension resistance

$N_{j,t,Rd} = F_{t,Rd} = 120.3 \text{ kN}$

compression resistance

$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 161.3 \text{ kN}$

2.2.3.3.2. shear resistance

shear resistance of column flange

$V_{cf,Rd} = 124.58 \text{ kN}$

shear resistance of column web

$V_{wp,Rd} = 80.6 \text{ kN}$

2.2.3.3.3. total

$M_{j,Rd} = 13.2 \text{ kNm}$ $N_{j,t,Rd} = 120.3 \text{ kN}$ $N_{j,c,Rd} = 161.3 \text{ kN}$ $V_{wp,Rd} = 80.6 \text{ kN}$ $V_{cf,Rd} = 124.6 \text{ kN}$

2.2.3.4. verifications

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

axial force: $N_{b,Ed} = |N_d \cdot \cos(\alpha) + V_d \cdot \sin(\alpha)| = 6.41 \text{ kN} < 5\% \cdot N_{pl,Rd} = 17.85 \text{ kN} \Rightarrow$ moment resistance

internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu} = 5.02 \text{ kNm}$, $z_{bu} = 80.6 \text{ mm}$

shear force: $V_{Ed} = |V_d| = 37.95 \text{ kN}$

shear force: $V_{c,w,Ed} = M_d/z - (V_{c1} - V_{c2})/2 = 24.46 \text{ kN}$, $z = 163.5 \text{ mm}$

shear force: $V_{b,w,Ed} = 31.93 \text{ kN}$

$M_{Ed}/M_{j,Rd} = 0.381 < 1$ **ok**

$V_{c,w,Ed}/V_{wp,Rd} = 0.303 < 1$ **ok**

$V_{b,w,Ed}/V_{cf,Rd} = 0.256 < 1$ **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

weld 8: beam flange in compression outer

welds 4,5: beam web double-sided

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) **!!**

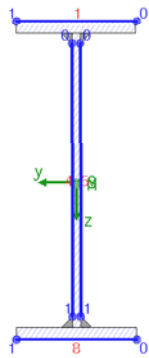
weld 2: effective weld length $l_{\text{eff}} = 24.1 \text{ mm} < 30 \text{ mm} \Rightarrow$ has no effect **!!**

weld 2: effective weld length $l_{\text{eff}} = 24.1 \text{ mm} < 6 \cdot a = 30.0 \text{ mm} \Rightarrow$ 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 $l_{\text{eff}} = 24.1 \text{ mm} < 30 \text{ mm} \Rightarrow$ has no effect **!!**

calculation section:



weld 1:	$a_w = 5.0 \text{ mm}$	$l_w = 64.0 \text{ mm}$
weld 4:	$a_w = 4.0 \text{ mm}$	$l_w = 145.7 \text{ mm}$
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:

$N_{Ed} = 12.65 \text{ kN}$, $M_{y,Ed} = -4.00 \text{ kNm}$, $V_{z,Ed} = 37.95 \text{ kN}$

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

$\Sigma A_w = 17.42 \text{ cm}^2$, $A_{w,z} = 11.66 \text{ cm}^2$, $\Sigma l_w = 41.9 \text{ cm}$
 $I_{w,y} = 620.73 \text{ cm}^4$, $I_{w,z} = 20.23 \text{ cm}^4$, $\Delta z_w = -4.3 \text{ mm}$

verifications in weld edges:

75% decrease of stress by pressure contact

weld 1,	pt. 0:	$\sigma_{w,x} = 59.95 \text{ N/mm}^2$		$\Rightarrow U_w = 0.236 < 1$	ok
weld 4,	pt. 0:	$\sigma_{w,x} = 52.24 \text{ N/mm}^2$	$\tau_{w,z} = 32.55 \text{ N/mm}^2$	$\Rightarrow U_w = 0.258 < 1$	ok
	pt. 1:	$\sigma_{w,x} = -10.42 \text{ N/mm}^2$	$\tau_{w,z} = 32.55 \text{ N/mm}^2$	$\Rightarrow U_w = 0.162 < 1$	ok
weld 8,	pt. 0:	$\sigma_{w,x} = -12.40 \text{ N/mm}^2$		$\Rightarrow U_w = 0.049 < 1$	ok

Result:

weld 4,	pt. 0:	$\sigma_{w,x} = 52.24 \text{ N/mm}^2$	$\tau_{w,z} = 32.55 \text{ N/mm}^2$	
Max:		$\sigma_{1,w,Ed} = 92.93 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2$,		
		$\sigma_{2,w,Ed} = 36.94 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U_w = 0.258 < 1$		ok

verification of deflection forces (directional method)

welds compression flange

compression force $N_{b,c} = 4.8 \text{ kN}$

weld angle $\varphi = 108.4^\circ$:

$\sigma_{1,w,Ed} = 13.74 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.038 < 1$ ok

$\sigma_{2,w,Ed} = 8.59 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U = 0.033 < 1$ ok

weld angle $\varphi = 71.6^\circ$:

$\sigma_{1,w,Ed} = 16.12 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.045 < 1$ ok

$\sigma_{2,w,Ed} = 6.19 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U = 0.024 < 1$ ok

2.2.3.4.3. verification of web stiffeners

compression stiffener (below)

$F_{c,Ed} = 18.78 \text{ kN}$

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w) / b_f = 6.69 \text{ kN}$, $H = F \cdot e_f / e_H = 1.26 \text{ kN}$

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

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 36.28 \text{ kN}$

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 7.03 \text{ kN}$

$F_{Ed} = 7.03 \text{ kN} < F_{Rd} = 36.28 \text{ kN} \Rightarrow U = 0.194 < 1$ ok

cross-section at web

shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0}) = 93.78 \text{ kN}$

design value: $F_{Ed} = F = 6.69 \text{ kN}$

$F_{Ed} = 6.69 \text{ kN} < F_{Rd} = 93.78 \text{ kN} \Rightarrow U = 0.071 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 173.32 \text{ kN/m}$, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 32.52 \text{ kN/m}$, $b_1 = 19.3 \text{ mm}$

75% decrease of stress by pressure contact

$\sigma_{1,w,Ed} = 11.39 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.032 < 1$ ok

$\sigma_{2,w,Ed} = 10.83 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U = 0.042 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 38.72 \text{ kN/m}$, $l_1 = 86.4 \text{ mm}$

75% decrease of stress by pressure contact

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

$\sigma_{1,w,Ed} = 4.19 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.012 < 1$ ok

stiffener in tension (top)

$F_{t,Ed} = 31.43 \text{ kN}$

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w) / b_f = 11.20 \text{ kN}$, $H = F \cdot e_f / e_H = 2.10 \text{ kN}$

cross-section at flange

tension resistance $N_{t,Rd} = 36.28 \text{ kN}$

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 11.77 \text{ kN}$

$F_{Ed} = 11.77 \text{ kN} < F_{Rd} = 36.28 \text{ kN} \Rightarrow U = 0.324 < 1$ ok

cross-section at web

shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0}) = 93.78 \text{ kN}$

design value: $F_{Ed} = F = 11.20 \text{ kN}$

$F_{Ed} = 11.20 \text{ kN} < F_{Rd} = 93.78 \text{ kN} \Rightarrow U = 0.119 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 290.06 \text{ kN/m}$, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 54.42 \text{ kN/m}$, $b_1 = 19.3 \text{ mm}$

$\sigma_{1,w,Ed} = 76.25 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.212 < 1$ ok

$\sigma_{2,w,Ed} = 72.51 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U = 0.280 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 64.79 \text{ kN/m}$, $l_1 = 86.4 \text{ mm}$

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

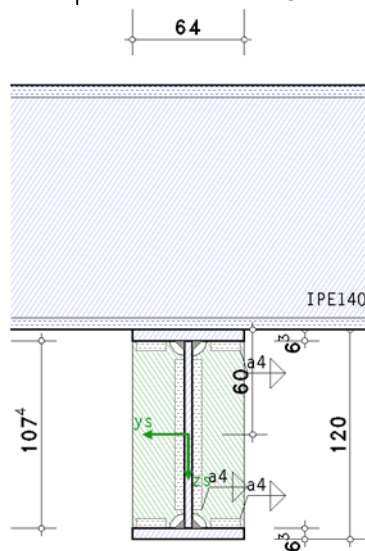
$\sigma_{1,w,Ed} = 28.06 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.078 < 1$ ok

2.2.3.4.4. verification result

maximum utilization: $\max U = 0.434 < 1$ 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	V_{Ed} kN	U_r ---	U_σ ---	U ---
1	40.00	6.41	39.48	0.453	0.616	0.616*

$F_{Z,Ed}$: vertical single load by a load girder; N_{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]: $\max U = 0.616 < 1$ ok

2.3.3. Lc 1 (decisive)

cross-sectional properties: $A = 12.79 \text{ cm}^2$, $z_s = 60.0 \text{ mm}$, $I_y = 306.31 \text{ cm}^4$, $y_s = 0.0 \text{ mm}$, $I_z = 27.60 \text{ cm}^4$

feed width by the beam $s_s' = 2 \cdot t_f + t_w + 2.828 \cdot a_w = 28.3 \text{ mm}$

feed length of load by the load beam $s_s = 2 \cdot t_f + t_w + 1.172 \cdot r = 26.7 \text{ mm}$

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

effective loading length $l_{eff} = s_s + 2 \cdot t_f = 39.3 \text{ mm}$

length of local loading:

referring to outer edge of flange $s_s = l_{eff} - 2 \cdot t_f = 26.7 \text{ mm}$ / to periphery of web $s_w = l_{eff} = 39.3 \text{ mm}$

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

$$F = 0.5 \cdot F_{c,Ed} = 20.00 \text{ kN}, \quad H = F \cdot e_F / e_H = 3.56 \text{ kN}$$

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

note: $b_R > 25.8 \text{ mm} \Rightarrow$ end return is not possible, pay attention to corrosion protection !!

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 50.09 \text{ kN}$

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 20.93 \text{ kN}$

$$F_{Ed} = 20.93 \text{ kN} < F_{Rd} = 50.09 \text{ kN} \Rightarrow U = 0.418 < 1 \text{ ok}$$

cross-section at web

shear resistance $V_{Rd} = (A_v \cdot f_y) / (31^{1/2} \cdot \gamma_{M0}) = 122.69 \text{ kN}$

design value: $F_{Ed} = F = 20.00 \text{ kN}$

$$F_{Ed} = 20.00 \text{ kN} < F_{Rd} = 122.69 \text{ kN} \Rightarrow U = 0.163 < 1 \text{ ok}$$

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 469.16 \text{ kN/m}$, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 83.62 \text{ kN/m}$, $b_1 = 21.3 \text{ mm}$

$$\sigma_{1,w,Ed} = 122.75 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.341 < 1 \text{ ok}$$

$$\sigma_{2,w,Ed} = 117.29 \text{ N/mm}^2 < f_{2w,d} = 259.20 \text{ N/mm}^2 \Rightarrow U = 0.453 < 1 \text{ ok}$$

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 110.58 \text{ kN/m}$, $l_1 = 90.4 \text{ mm}$

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

$$\sigma_{1,w,Ed} = 47.88 \text{ N/mm}^2 < f_{1w,d} = 360.00 \text{ N/mm}^2 \Rightarrow U = 0.133 < 1 \text{ ok}$$

total: utilization of ribs $U_R = 0.453 < 1$ ok

stresses at first cut of web:

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

Lc 1: $N_{Ed} = 6.4 \text{ kN}$, $V_{z,Ed} = 39.5 \text{ kN}$

stresses $\sigma_{x,Ed} = 5.0 \text{ N/mm}^2$, $\tau_{xz,Ed} = 83.6 \text{ N/mm}^2$

$$|\sigma_{x,Ed}| = 5.0 \text{ N/mm}^2 < \sigma_{Rd} = 235.0 \text{ N/mm}^2 \Rightarrow U = 0.021 < 1 \text{ ok}$$

$$|\tau_{xz,Ed}| = 83,6 \text{ N/mm}^2 < \tau_{Rd} = 135,7 \text{ N/mm}^2 \Rightarrow U = 0,616 < 1 \text{ ok}$$

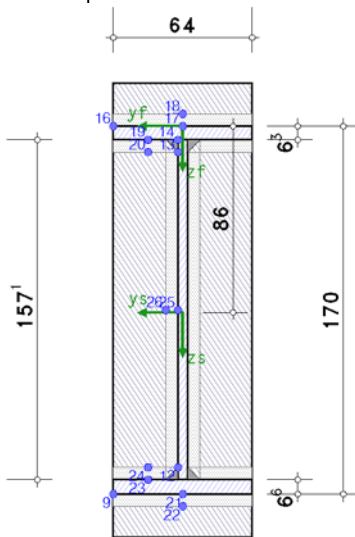
$$\sigma_v = 144.8 \text{ N/mm}^2 < \sigma_{Rd} = 235.0 \text{ N/mm}^2 \Rightarrow U = 0.616 < 1 \text{ ok}$$

utilization at first cut of web $\max U_{\sigma} = 0.616 < 1$ ok

maximum utilization: $\max U = 0.616 < 1$ 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.	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	170.0	160.0	0.0	0.0	at bottom flange	8.1(1)
12	2.2	157.7	100.0	100.0	0.0	at beam web	8.2(7) 8.1(6)
13	2.2	12.0	100.0	100.0	40.0	at beam web	8.2(7) 8.1(6) 8.10(4)*
° 14	2.2	6.3	0.0	80.0	0.0	at beam web	8.5(8)
16	32.0	0.0	160.0	0.0	0.0	at top flange	8.1(1)
° 17	0.0	0.0	36.0	0.0	0.0	am plate (top flange)	8.5(3)
18	0.0	-5.7	80.0	0.0	0.0	am plate (top flange)	8.5(1)

Pt.	yf mm	Zf mm	$\Delta\sigma_x, R_d$ N/mm ²	$\Delta\tau_{Rd}$ N/mm ²	$\Delta\sigma_z, R_d$ N/mm ²	notch point	EC 3-1-9, tab.
° 19	16.0	6.3	36.0	0.0	0.0	am plate (top flange)	8.5(3)
20	16.0	12.0	80.0	0.0	0.0	am plate (top flange)	8.5(1)
° 21	0.0	170.0	36.0	0.0	0.0	am plate (bottom flange)	8.5(3)
22	0.0	175.7	80.0	0.0	0.0	am plate (bottom flange)	8.5(1)
° 23	16.0	163.4	36.0	0.0	0.0	am plate (bottom flange)	8.5(3)
24	16.0	157.7	80.0	0.0	0.0	am plate (bottom flange)	8.5(1)
° 25	2.2	85.0	36.0	80.0	0.0	am plate (web)	8.5(3) 8.5(8)
26	7.9	85.0	0.0	100.0	0.0	am plate (web)	8.2(5)

The diagram shows a T-beam cross-section with the following dimensions and features:

- Overall Width:** 64
- Overall Height:** 107
- Flange Thickness:** 6
- Web Height:** 60
- Web Width:** 14
- Top Flange:** Shaded with diagonal lines. Points 16, 28, and 14 are marked on its top edge. Point 28 is at the top-left corner of the web.
- Bottom Flange:** Shaded with diagonal lines. Points 23 and 9 are marked on its bottom edge. Point 23 is at the bottom-left corner of the web.
- Web:** Shaded with diagonal lines. Point 28 is at the top-left corner, and point 9 is at the bottom-left corner.
- Coordinate Systems:**
 - y^f and z^f are centered at the top of the flange.
 - y^s and z^s are centered at the bottom of the web.
- Applied Forces:** A red arrow labeled F_z, E_d points downwards at the top center of the flange.

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	32.0	120.0	160.0	0.0	0.0	at bottom flange	8.1(1)
13	2.2	12.0	100.0	100.0	40.0	at beam web	8.2(7) 8.1(6) 8.10(4)*
14	2.2	6.3	0.0	80.0	0.0	at beam web	8.5(8)
16	32.0	0.0	160.0	0.0	0.0	at top flange	8.1(1)
23	32.0	113.7	80.0	0.0	0.0	due to transv.stiff.	8.4(7)
26	2.2	14.8	0.0	100.0	0.0	due to transv.stiff.	8.1(6)
28	32.0	6.3	80.0	0.0	0.0	due to transv.stiff.	8.4(7)

Lc 1: $M_{y,Ed} = 0.01 \text{ kNm}$, $V_{z,Ed} = 5.00 \text{ kN}$

Lc 2: $M_{y,Ed} = 0.03 \text{ kNm}$, $V_{z,Ed} = 15.00 \text{ kN}$

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

2.5.1. fatigue design

cross-sectional properties: $A = 12.79 \text{ cm}^2$, $z_s = 60.0 \text{ mm}$, $I_y = 306.31 \text{ cm}^4$, $y_s = 0.0 \text{ mm}$, $I_z = 27.60 \text{ cm}^4$

effective loading length $l_{eff} = s_s + 2 \cdot t_f = 39.3 \text{ mm}$

... at beam web $\sigma_{oz} = -86.7 \text{ N/mm}^2$, $\tau_o = 17.3 \text{ N/mm}^2$ / ... at weld $\sigma_{oz} = -47.7 \text{ N/mm}^2$, $\tau_o = 9.5 \text{ N/mm}^2$

stress ranges:

pt. 13: $y_f = 2.2 \text{ mm}$, $z_f = 12.0 \text{ mm}$ $\Delta\sigma_{x,Ed} = 0.3 \text{ N/mm}^2$ $\Delta\tau_{Ed} = 52.7 \text{ N/mm}^2$ $\Delta\sigma_{z,Ed} = 86.7 \text{ N/mm}^2$

equivalent constant amplitude stress range:

pt. 13: $y_f = 2.2 \text{ mm}$, $z_f = 12.0 \text{ mm}$ $\Delta\sigma_{x,f} = 0.3 \text{ N/mm}^2$ $\Delta\tau_f = 52.7 \text{ N/mm}^2$ $\Delta\sigma_{z,f} = 86.7 \text{ N/mm}^2$

verification of notch stresses:

pt. 13: $y = 2.2 \text{ mm}$, $z = 12.0 \text{ mm}$

$\Delta\sigma_{x,f} = 0.3 \text{ N/mm}^2 < \Delta\sigma_{x,Rd,f} = 87.0 \text{ N/mm}^2 \Rightarrow U_{\Delta\sigma_x} = 0.004 \text{ ok}$

$\Delta\tau_f = 52.7 \text{ N/mm}^2 < \Delta\tau_{Rd,f} = 87.0 \text{ N/mm}^2 \Rightarrow U_{\Delta\tau} = 0.606 \text{ ok}$

$\Delta\sigma_{z,f} = 86.7 \text{ N/mm}^2 > \Delta\sigma_{z,Rd,f} = 34.8 \text{ N/mm}^2 \Rightarrow U_{\Delta\sigma_z} = 2.494 \text{ not ok !!}$

interaction $U_i = U_{\Delta\sigma_x}^3 + U_{\Delta\sigma_z}^3 + U_{\Delta\tau}^5 = 15.592 > 1 \text{ not ok !!}$

fatigue design [pt. 13]: max $U = 2.494 > 1 \text{ not ok !!}$

3. final result

maximum utilization:

max $U = 2.494 > 1 \text{ not ok !!}$

fatigue of the connection / of the section

resistance not ensured !!

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