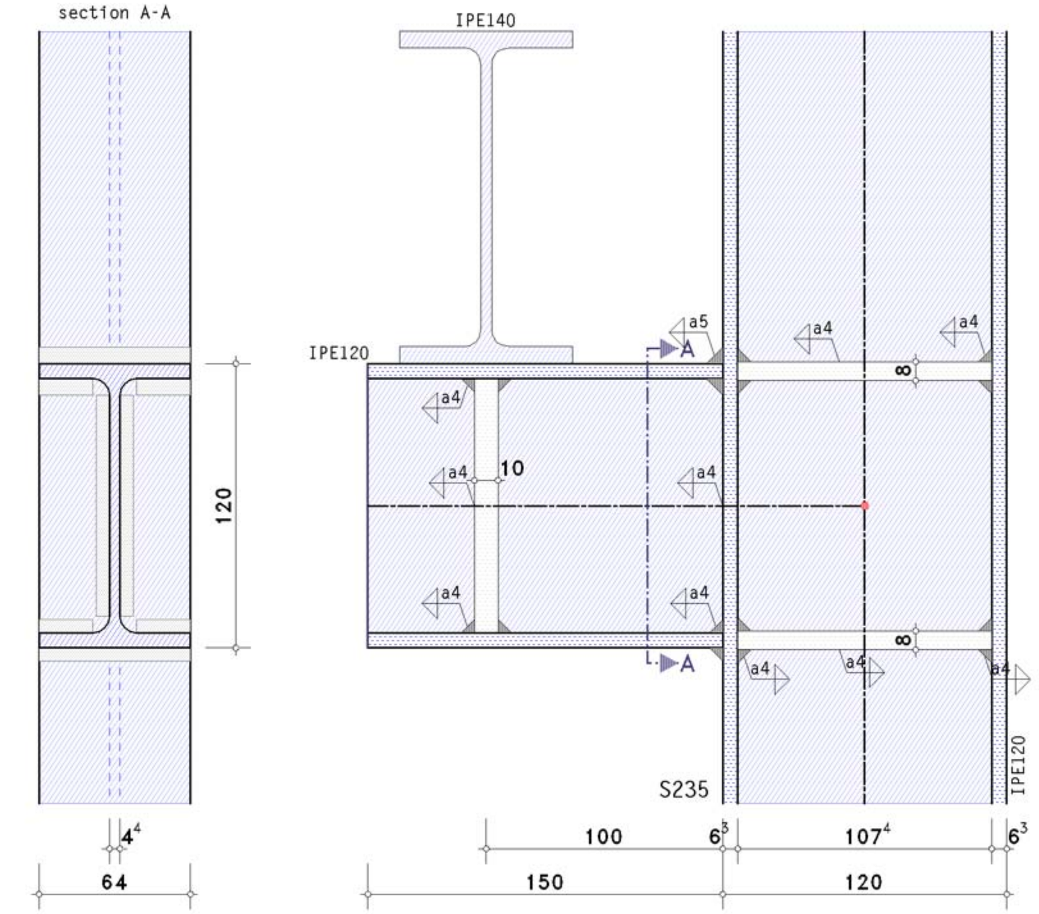
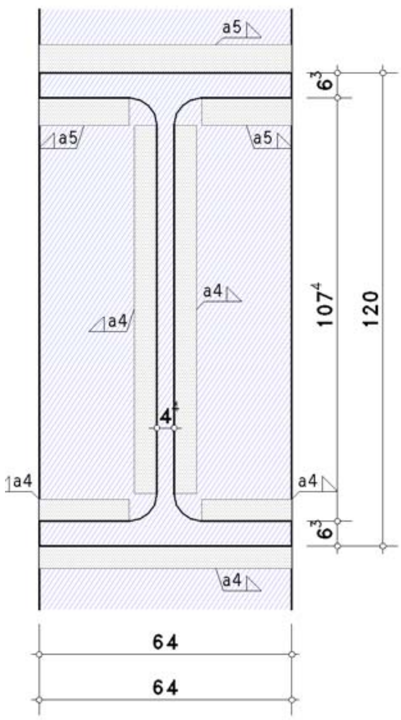
bracket EC 3-1-8 (12.10), NA: Deutschland 4H-EC3SK version: 6/2025-1b

1. input report



details (section A - A)



steel grade
steel grade S235
column parameters
section IPE120

reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 113.7 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

section IPE120
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} = 10.5 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
75% of compressive stress is transfered by contact

parameters of the connection

welded connection
distance of transverse loading Δa = 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 λ_σ = 1.000, λ_τ = 1.000

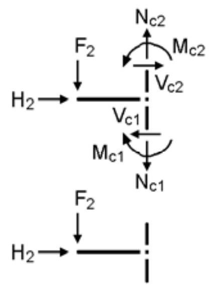
calculation of resistances

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 γ_{M0} = 1.00
resistance of members in stability failure γ_{M1} = 1.10
resistance of bolts, welds, plates in bearing γ_{M2} = 1.25

material safety factor (FLS)

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

2. verifications

2.1. table of results (ULS)

2.1.1. utilizations

Lc	U_j	U_l	$U_{j,b}$	$U_{j,\sigma}$	$U_{j,\tau}$	$U_{j,w}$	$U_{j,r}$	$U_{l,\sigma}$	$U_{l,r}$	U
--	---	---	---	---	---	---	---	---	---	---
1	0.640	0.512	0.640	0.428	0.501	0.369	0.375	0.512	0.356	0.640*

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; $U_{l,r}$: 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

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	0.428	0.428	0.501	0.369	0.375	0.640*

$U_{\sigma,b}$: 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 im column flange; U_{sb}: utilization due to weld; U_{ss}: utilization due to stiffeners/ribs
U: utilization of the connection

*) maximum utilization

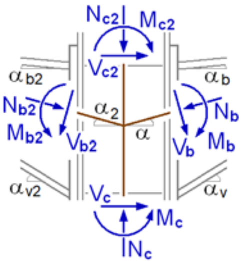
2.2.2. final result

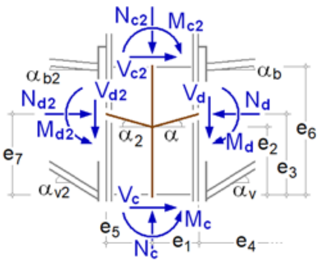
maximum utilization: max U = 0.640 < 1 ok

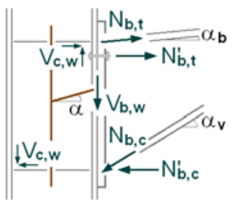
2.2.3. Lc 1 (decisive)

2.2.3.1. design values

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







slope angle: $\alpha_b = \alpha = \alpha_v = 0°$
distance: e_1 = 60.0 mm, e_3 = 56.8 mm, e_2 = 56.8 mm, e_6 = 113.7 mm

internal forces and moments perpendicular to the connection planes
periphery beam
M_d = 4.00 kNm, V_d = 40.00 kN

partial internal forces and moments
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M_d/z_b$ = 35.18 kN, z_b = 113.7 mm, z_{bu} = 56.9 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M_d/z_b$ = 35.18 kN, z_b = 113.7 mm, z_{bo} = 56.9 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, $V_{b,w} = V_d - V_{b,t} - V_{b,c}$ = 40.00 kN

2.2.3.2. resistance of cross-section in the periphery

elastic verification for M_y = -4.00 kNm, V_z = 40.00 kN
verification: σ_v = 150.35 N/mm² < $\sigma_{v,Rd}$ = 235.00 N/mm² ⇒ U_σ = 0.640 < 1 ok
c/t-ratio: outstand flange: utilization $U_{c/t}$ = 0.145 < 1 ok
internal compression parts: utilization $U_{c/t}$ = 0.086 < 1 ok

2.2.3.3. connection capacity

transformation parameter: β_j = 1.00

2.2.3.3.1. moment resistance

distance between tension force and centre of compression: z = 113.7 mm

resistance
F_{Rd} = 82.2 kN

resistance of flanges (compression)
$\Sigma F_{c,Rd}^*$ = 164.5 kN

moment resistance
$M_{j,Rd} = F_{Rd} \cdot z$ = 9.4 kNm
tension resistance
$N_{j,t,Rd} = F_{t,Rd}$ = 120.3 kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^*$ = 164.5 kN

2.2.3.3.2. shear resistance

shear resistance of column flange
$V_{cf,Rd}$ = 79.84 kN
shear resistance of column web
$V_{wp,Rd}$ = 82.2 kN

2.2.3.3.3. total

$M_{j,Rd}$ = 9.4 kNm $N_{j,t,Rd}$ = 120.3 kN $N_{j,c,Rd}$ = 164.5 kN $V_{wp,Rd}$ = 82.2 kN $V_{cf,Rd}$ = 79.8 kN

2.2.3.4. verifications

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

internal moment: $M_{Ed} = M_d$ = 4.00 kNm
shear force: $V_{Ed} = |V_d|$ = 40.00 kN
shear force: $V_{c,w,Ed} = M_d/z - (V_{c1}-V_{c2})/2$ = 35.18 kN, z = 113.7 mm
shear force: $V_{b,w,Ed}$ = 40.00 kN

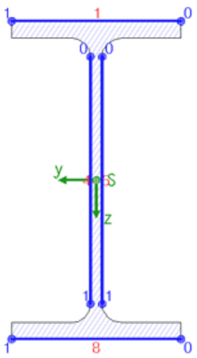
$M_{Ed}/M_{j,Rd} = 0.428 < 1$ ok
$V_{c,w,Ed}/V_{wp,Rd} = 0.428 < 1$ ok
$V_{b,w,Ed}/V_{cf,Rd} = 0.501 < 1$ ok

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 mm > $a_{max} = 0.7 \cdot t_{min}$ = 4.4 mm (welding technology, s. DIN 18800) !!
weld 2: weld thickness a = 5.0 mm > $a_{max} = 0.7 \cdot t_{min}$ = 4.4 mm (welding technology, s. DIN 18800) !!
weld 2: effective weld length l_{eff} = 22.8 mm < 30 mm ⇒ has no effect !!
weld 2: effective weld length l_{eff} = 22.8 mm < 6·a = 30.0 mm ⇒ has no effect !!
weld 4: weld thickness a = 4.0 mm > $a_{max} = 0.7 \cdot t_{min}$ = 3.1 mm (welding technology, s. DIN 18800) !!
weld 6: effective weld length l_{eff} = 22.8 mm < 30 mm ⇒ has no effect !!
weld 6: effective weld length l_{eff} = 22.8 mm < 6·a = 24.0 mm ⇒ has no effect !!

calculation section:



weld 1:	a_w = 5.0 mm	l_w = 64.0 mm
weld 4:	a_w = 4.0 mm	l_w = 93.4 mm
weld 5:	see weld 4	
weld 8:	a_w = 4.0 mm	l_w = 64.0 mm

design values refering to centroid of the section:
$M_{y,Ed}$ = -4.00 kNm, $V_{z,Ed}$ = 40.00 kN
cross-sectional properties refering to centroid of the line cross-section:
ΣA_w = 13.23 cm², $A_{w,z}$ = 7.47 cm², Σl_w = 31.5 cm
$I_{w,y}$ = 260.56 cm⁴, $I_{w,z}$ = 20.02 cm⁴, Δz_w = -2.9 mm
verifications in weld edges:
75% decrease of stress by pressure contact

weld 1,	pt. 0:	$\sigma_{w,x}$ = 87.65 N/mm²		⇒ $U_w = 0.344 < 1$ ok
weld 4,	pt. 0:	$\sigma_{w,x}$ = 67.24 N/mm²	$\tau_{w,z}$ = 53.53 N/mm²	⇒ $U_w = 0.369 < 1$ ok
	pt. 1:	$\sigma_{w,x}$ = -19.04 N/mm²	$\tau_{w,z}$ = 53.53 N/mm²	⇒ $U_w = 0.268 < 1$ ok
weld 8,	pt. 0:	$\sigma_{w,x}$ = -24.14 N/mm²		⇒ $U_w = 0.095 < 1$ ok

Result:
weld 4, pt. 0: $\sigma_{w,x}$ = 67.24 N/mm² $\tau_{w,z}$ = 53.53 N/mm²
Max: $\sigma_{1,w,Ed}$ = 132.81 N/mm² < $f_{1w,d}$ = 360.00 N/mm²,
$\sigma_{2,w,Ed}$ = 47.54 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ $U_w = 0.369 < 1$ ok

2.2.3.4.3. verification of web stiffeners

compression stiffener (below)
$F_{c,Ed}$ = 36.29 kN
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 12.93 kN, $H = F \cdot e_F/e_H$ = 2.43 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 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 13.59 kN
F_{Ed} = 13.59 kN < F_{Rd} = 36.28 kN ⇒ U = 0.375 < 1 ok
cross-section at web
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0})$ = 93.78 kN
design value: $F_{Ed} = F$ = 12.93 kN

$F_{Ed} = 12.93$ kN $<$ $F_{Rd} = 93.78$ kN $\Rightarrow$ U = 0.138 $<$ 1 ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 334.92$ kN/m, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 62.84$ kN/m, $b_1 = 19.3$ mm

75% decrease of stress by pressure contact

$\sigma_{1,w,Ed} = 22.01$ N/mm² $<$ $f_{1w,d} = 360.00$ N/mm² $\Rightarrow$ U = 0.061 $<$ 1 ok

$\sigma_{2,w,Ed} = 20.93$ N/mm² $<$ $f_{2w,d} = 259.20$ N/mm² $\Rightarrow$ U = 0.081 $<$ 1 ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 74.81$ kN/m, $l_1 = 86.4$ mm

75% decrease of stress by pressure contact

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

$\sigma_{1,w,Ed} = 8.10$ N/mm² $<$ $f_{1w,d} = 360.00$ N/mm² $\Rightarrow$ U = 0.022 $<$ 1 ok

stiffener in tension (top)

$F_{t,Ed} = 36.29$ kN

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 12.93$ kN, $H = F \cdot e_F/e_H = 2.43$ kN

cross-section at flange

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

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 13.59$ kN

$F_{Ed} = 13.59$ kN $<$ $F_{Rd} = 36.28$ kN $\Rightarrow$ U = 0.375 $<$ 1 ok

cross-section at web

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

design value: $F_{Ed} = F = 12.93$ kN

$F_{Ed} = 12.93$ kN $<$ $F_{Rd} = 93.78$ kN $\Rightarrow$ U = 0.138 $<$ 1 ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 334.92$ kN/m, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 62.84$ kN/m, $b_1 = 19.3$ mm

$\sigma_{1,w,Ed} = 88.04$ N/mm² $<$ $f_{1w,d} = 360.00$ N/mm² $\Rightarrow$ U = 0.245 $<$ 1 ok

$\sigma_{2,w,Ed} = 83.73$ N/mm² $<$ $f_{2w,d} = 259.20$ N/mm² $\Rightarrow$ U = 0.323 $<$ 1 ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 74.81$ kN/m, $l_1 = 86.4$ mm

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

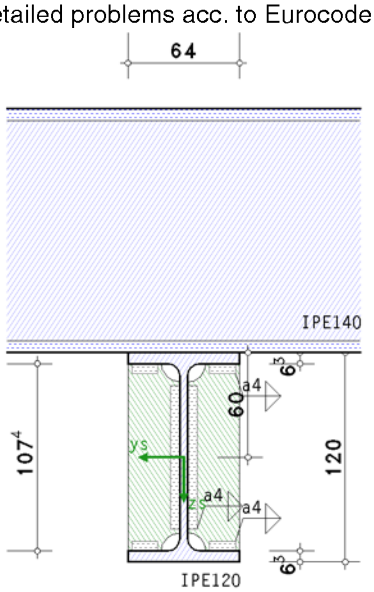
$\sigma_{1,w,Ed} = 32.40$ N/mm² $<$ $f_{1w,d} = 360.00$ N/mm² $\Rightarrow$ U = 0.090 $<$ 1 ok

2.2.3.4.4. verification result

maximum utilization: max U = 0.640 $<$ 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	$F_{z,Ed}$	V_{Ed}	U_r	U_σ	U
--	kN	kN	---	---	---
1	40.00	40.00	0.356	0.512	0.512*

$F_{z,Ed}$: vertical single load by a load girder; 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 = 0.512 < 1 ok

2.3.3. Lc 1 (decisive)

cross-sectional properties: A = 13.21 cm², z_s = 60.0 mm, I_y = 317.76 cm⁴, y_s = 0.0 mm, I_z = 27.67 cm⁴

feed width by the beam $s_s' = 2 \cdot t_f + t_w + 1.172 \cdot r$ = 25.2 mm
feed length of load by the load beam $s_s = 2 \cdot t_f + t_w + 1.172 \cdot r$ = 26.7 mm
For info: compression $F_{z,Ed,ULS}/(s_s \cdot s_s')$ = 59.44 N/mm²
effective loading length $l_{eff} = s_s + 2 \cdot t_f$ = 39.3 mm
length of local loading:
refering to outer edge of flange $s_s = l_{eff} - 2 \cdot t_f$ = 26.7 mm / to periphery of web $s_w = l_{eff} + 2 \cdot r$ = 53.3 mm

2.3.3.1. compression of web (ULS)

permissible stresses: $\sigma_{Rd} = f_y/\gamma_{M0}$ = 235.0 N/mm², $\tau_{Rd} = f_y/(3^{1/2} \cdot \gamma_{M0})$ = 135.7 N/mm²

load transfer with transverse stiffeners (ribs):
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 14.25 kN, $H = F \cdot e_F/e_H$ = 2.67 kN
load component of web $F_{c,Ed} - 2 \cdot F$ = 11.50 kN
assumption: stiffeners do not buckle: section class 1 ≤ 3 ok
note: b_R > 25.8 mm ⇒ end return is not possible, pay attention to corrosion protection !!
cross-section at flange
compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0}$ = 45.36 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 14.98 kN
F_{Ed} = 14.98 kN < F_{Rd} = 45.36 kN ⇒ U = 0.330 < 1 ok
cross-section at web
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0})$ = 117.23 kN
design value: F_{Ed} = F = 14.25 kN
F_{Ed} = 14.25 kN < F_{Rd} = 117.23 kN ⇒ U = 0.122 < 1 ok
flange welds
design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 369.17 kN/m, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 69.26 kN/m, b_1 = 19.3 mm
$\sigma_{1,w,Ed}$ = 97.04 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.270 < 1 ok
$\sigma_{2,w,Ed}$ = 92.29 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U = 0.356 < 1 ok
web welds
design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 82.47 kN/m, l_1 = 86.4 mm
weld thickness a = 4.0 mm > $a_{max} = 0.7 \cdot t_{min}$ = 3.1 mm (welding technology, s. DIN 18800) !!
$\sigma_{1,w,Ed}$ = 35.71 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.099 < 1 ok
total: utilization of ribs U_R = 0.356 < 1 ok

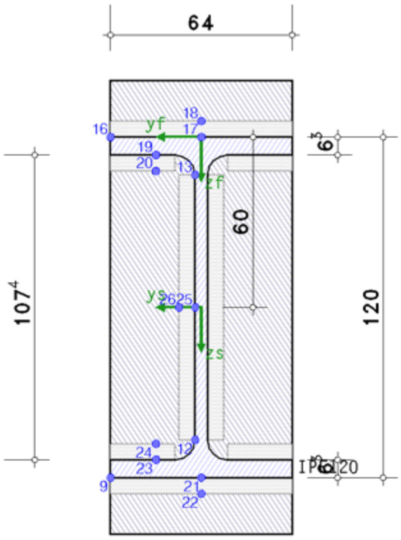
compression of single load at first cut of web:
reduced transverse loading $F_{z,Ed}$ = 11.5 kN
local stresses $\sigma_{oz,Ed}$ = -49.0 N/mm²
$|\sigma_{oz,Ed}|$ = 49.0 N/mm² < σ_{Rd} = 235.0 N/mm² ⇒ U = 0.209 < 1 ok

stresses at first cut of web:
reduced transverse loading $F_{z,Ed}$ = 11.5 kN
Lc 1: $V_{z,Ed}$ = 40.0 kN
stresses $\tau_{xz,Ed}$ = 63.4 N/mm²
$|\tau_{xz,Ed}|$ = 63.4 N/mm² < τ_{Rd} = 135.7 N/mm² ⇒ U = 0.468 < 1 ok
σ_v = 120.3 N/mm² < σ_{Rd} = 235.0 N/mm² ⇒ U = 0.512 < 1 ok
utilization at first cut of web max U_σ = 0.512 < 1 ok

maximum utilization: max U = 0.512 < 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/mm²	$\Delta\tau_{Rd}$ N/mm²	$\Delta\sigma_{z,Rd}$ N/mm²	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	-5.7	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	12.0	80.0	0.0	0.0	am plate (top flange)	8.5(1)
° 21	0.0	120.0	36.0	0.0	0.0	am plate (bottom flange)	8.5(3)
22	0.0	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: $M_{y,Ed}$ = -0.50 kNm, $V_{z,Ed}$ = 5.00 kN

Lc 2: $M_{y,Ed}$ = -1.50 kNm, $V_{z,Ed}$ = 15.00 kN

2.4.1. fatigue design

cross-sectional properties: A = 13.21 cm², z_s = 60.0 mm, I_y = 317.76 cm⁴, y_s = 0.0 mm, I_z = 27.67 cm⁴

stress ranges:
pt. 21: y_f = 0.0 mm, z_f = 120.0 mm $\Delta\sigma_{x,Ed}$ = 20.9 N/mm²

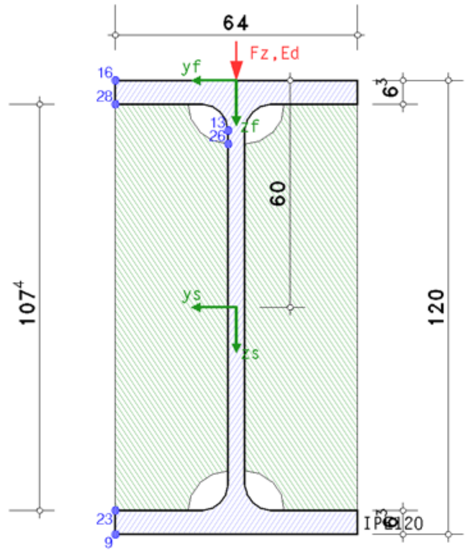
equivalent constand amplitude stress range:
pt. 21: y_f = 0.0 mm, z_f = 120.0 mm $\Delta\sigma_{x,f}$ = 20.9 N/mm²

verification of notch stresses:
pt. 21: y = 0.0 mm, z = 120.0 mm $\Delta\sigma_{x,f}$ = 20.9 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm² ⇒ $U_{\Delta\sigma x}$ = 0.667 ok

fatigue design [pt. 21]: max U = 0.667 < 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/mm²	$\Delta\tau_{Rd}$ N/mm²	$\Delta\sigma_{z,Rd}$ N/mm²	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)
23	32.0	113.7	80.0	0.0	0.0	due to transv.stiff.	8.4(7)
26	2.2	16.8	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)

loading

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

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

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

2.5.1. fatigue design

cross-sectional properties: A = 13.21 cm², z_s = 60.0 mm, I_y = 317.76 cm⁴, y_s = 0.0 mm, I_z = 27.67 cm⁴

effective loading length $l_{eff} = s_s + 2 \cdot t_f$ = 39.3 mm
... at beam web σ_{oz} = -64.0 N/mm², τ_o = 12.8 N/mm²

stress ranges:
pt. 13: y_f = 2.2 mm, z_f = 13.3 mm $\Delta\tau_{Ed}$ = 43.8 N/mm² $\Delta\sigma_{z,Ed}$ = 64.0 N/mm²

equivalent constand amplitude stress range:
pt. 13: y_f = 2.2 mm, z_f = 13.3 mm $\Delta\tau_f$ = 43.8 N/mm² $\Delta\sigma_{z,f}$ = 64.0 N/mm²

verification of notch stresses:
pt. 13: y = 2.2 mm, z = 13.3 mm $\Delta\tau_f$ = 43.8 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm² ⇒ $U_{\Delta\tau}$ = 0.504 ok
$\Delta\sigma_{z,f}$ = 64.0 N/mm² < $\Delta\sigma_{z,Rd,f}$ = 139.1 N/mm² ⇒ $U_{\Delta\sigma z}$ = 0.460 ok

fatigue design [pt. 13]: max U = 0.504 < 1 ok

3. final result

maximum utilization: max U = 0.667 < 1 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