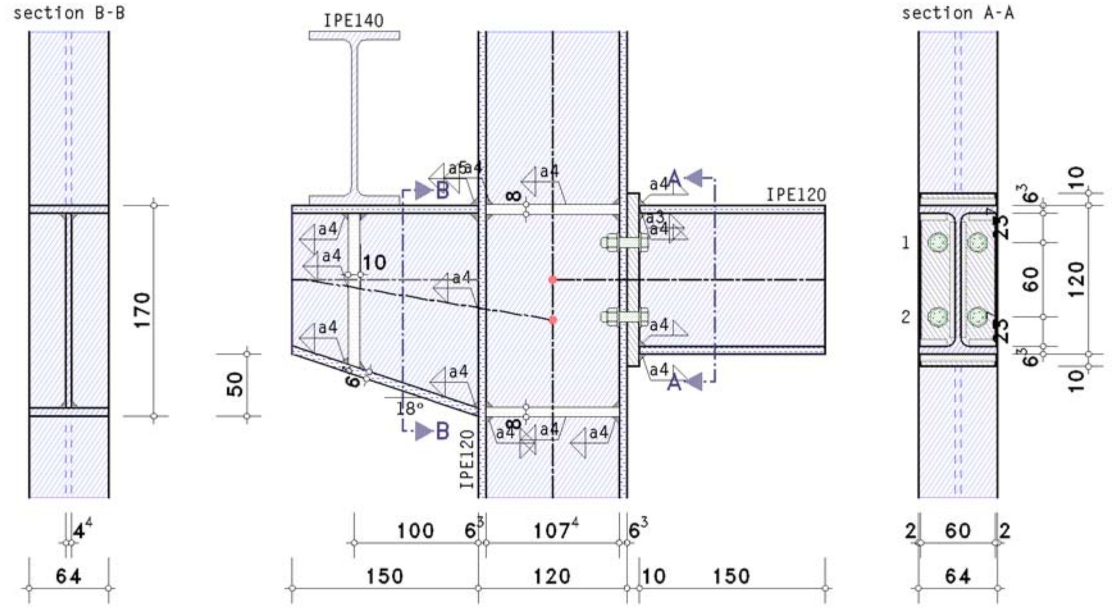
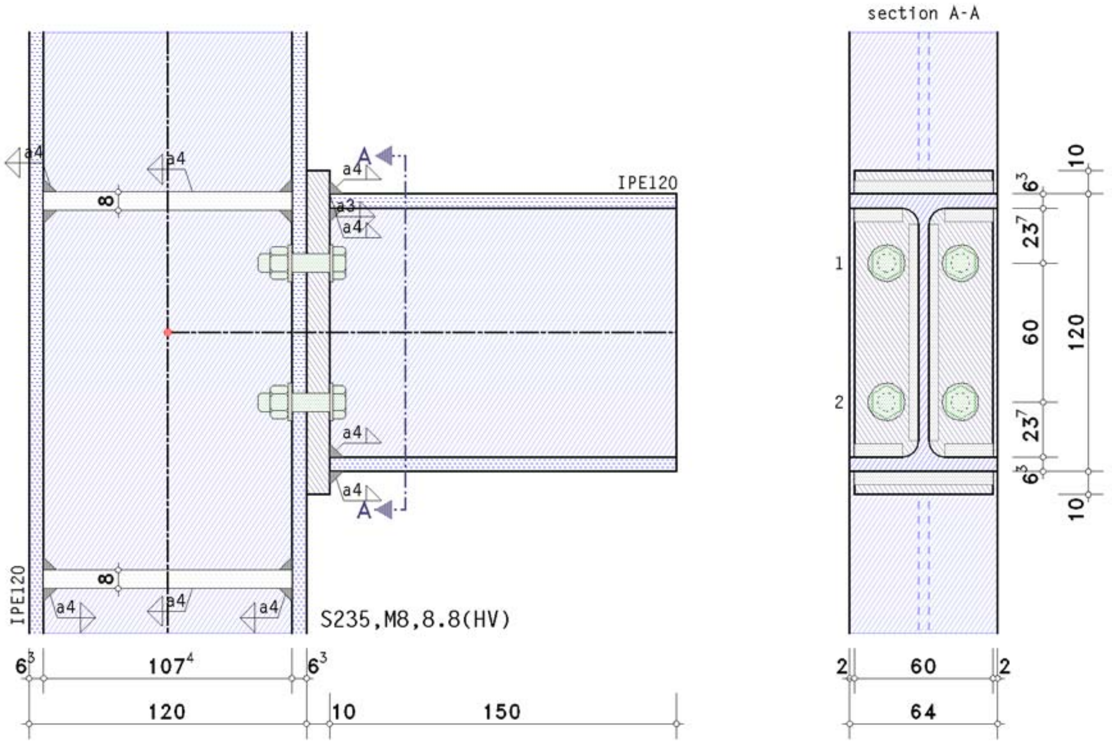
POS. 32: RELI / GESCHWEISST-BOLTED

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

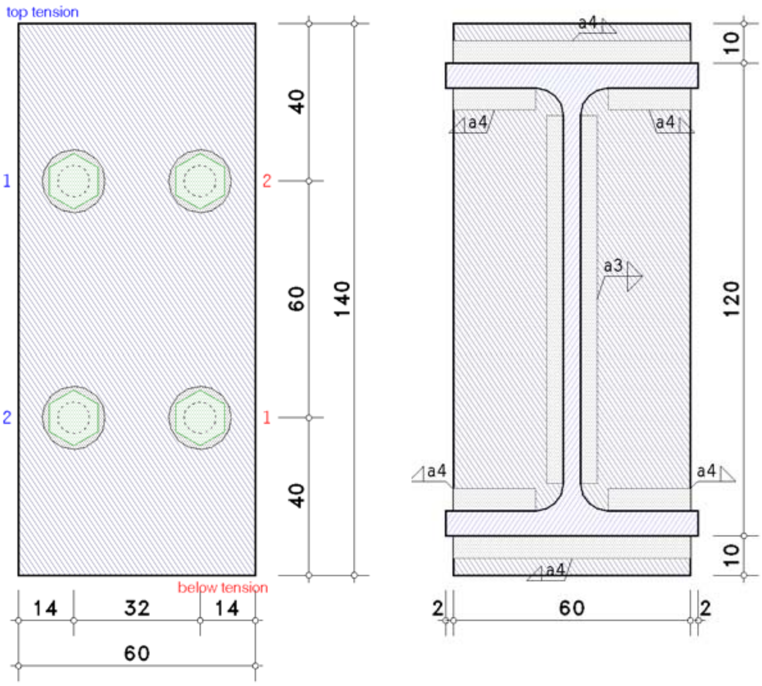
1. input report



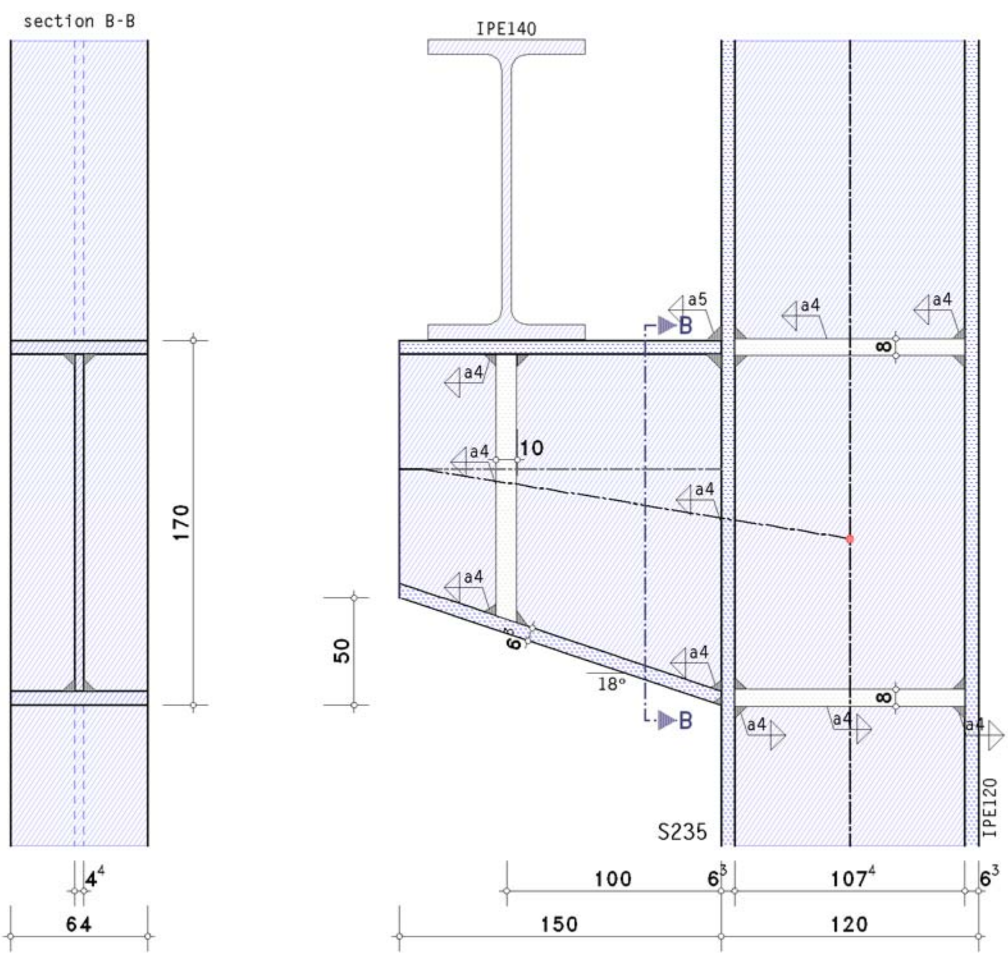
bracket right



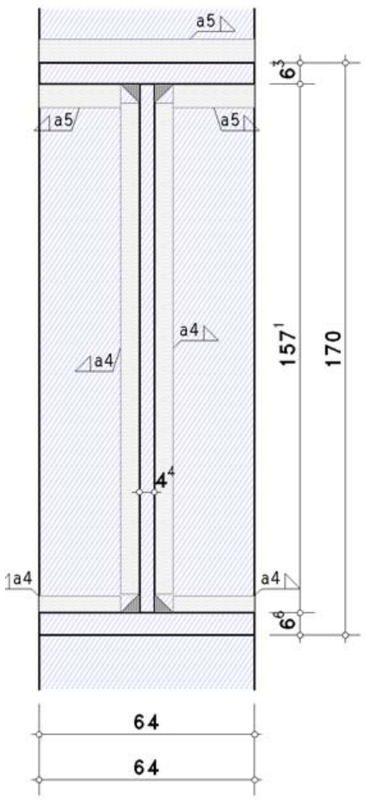
details (section A - A)



bracket left



details (section B - B)



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
double-sided beam-column joint
according to EC 3-1-8, 5.3 each connection side of two-sided beam-column connections is analysed independently of the other. centroidal axes of beams do not meet at a point (Δ = 32.5 mm != 0 !!). EC 3-requirement !!
connection right
beam parameters
section IPE120
bolts
bolt class 8.8, bolt size M8
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}* = 0.7 \cdot f_{yb} \cdot A_s$ = 16.4 kN)
shear plane passes through the unthreaded portion of the bolt
welds
beam flange top: fillet weld, weld thickness a = 4.0 mm
beam web: fillet weld, weld thickness a = 3.0 mm
beam flange below: fillet weld, weld thickness a = 4.0 mm
75% of compressive stress is transfered by contact
parameters of the connection
bolted end-plate connection
end-plate thickness t_p = 10.0 mm, width b_p = 60.0 mm, length l_p = 140.0 mm
projections $h_{p,o}$ = 10.0 mm, $h_{p,u}$ = 10.0 mm
bolts:
2 bolt-rows with 2 bolts
centre distance of the bolts to the lateral edge of the end-plate e_2 = 14.0 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 = 40.0 mm
centre distance of the bolt-rows from each other p_{1-2} = 60.0 mm
distance of transverse loading Δa = 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 λ_σ = 0.300, λ_τ = 0.300
connection left
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 $\alpha_v = 18.43°$
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 $\varphi = 108°$
75% of compressive stress is transfered 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)

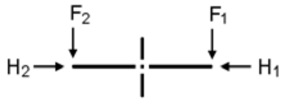
bracket load and internal forces and moments in the intersection point of system axes (ULS)

Lc --	$F_{1,Ed}$ kN	$F_{2,Ed}$ kN
1	40.00	40.00

$F_{1,Ed}$, $F_{2,Ed}$: loading of bracket

bracket load (FLS)

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



$F_{1,Ed}$, $F_{2,Ed}$: 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 right

2.1. table of results (ULS)

2.1.1. utilizations of each joint

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	1.374	0.884	0.640	1.339	1.374	0.618	0.795	0.828	0.575	0.884	1.374*

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 each joint; U: utilization of each joint
*) maximum utilization of each joint

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
check bolt distances to column web ($\Delta e_{wc} = -1.2$ mm ≤ 0) !!

distances between bolts at end-plate

horizontal:	$e_2 = 14.0$ mm $> 1.2 \cdot d_0 = 10.8$ mm,	$e_2 = 14.0$ mm $< 4 \cdot t + 40$ mm $= 65.2$ mm
horizontal:	$p_2 = 32.0$ mm $> 2.4 \cdot d_0 = 21.6$ mm,	$p_2 = 32.0$ mm $< \min(14 \cdot t, 200$ mm$) = 88.2$ mm
top-below:	$e_1 = 40.0$ mm $> 1.2 \cdot d_0 = 10.8$ mm,	$e_1 = 40.0$ mm $< 4 \cdot t + 40$ mm $= 65.2$ mm
top-below:	$p_1 = 60.0$ mm $> 2.2 \cdot d_0 = 19.8$ mm,	$p_1 = 60.0$ mm $< \min(14 \cdot t, 200$ mm$) = 88.2$ mm
top-below:	$e_1 = 40.0$ mm $> 1.2 \cdot d_0 = 10.8$ mm,	$e_1 = 40.0$ mm $< 4 \cdot t + 40$ mm $= 65.2$ mm

bolt distance from column edge

horizontal: e_2 = 16.0 mm > 1.2·d_0 = 10.8 mm, e_2 = 16.0 mm < 4·t + 40 mm = 65.2 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	0.640	1.339	1.374	0.937	0.343	0.618	0.795	1.374*

$U_{\sigma,b}$: 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

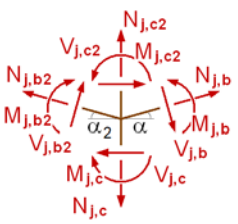
2.2.2. final result

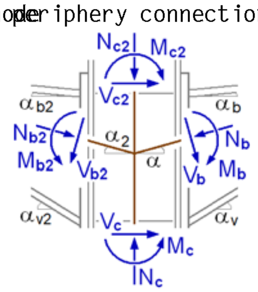
maximum utilization: max U = 1.374 > 1 not ok !!

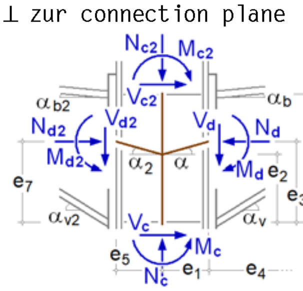
2.2.3. Lc 1 (decisive)

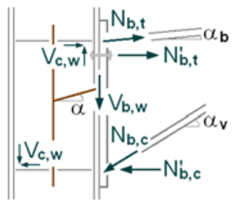
2.2.3.1. design values

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 = 5.60 kNm, V_d = 40.00 kN

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_p$ = 5.20 kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 45.73 kN, z_b = 113.7 mm, z_{bu} = 56.9 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b$ = 45.73 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 = -5.20 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 $U_{c/t}$ = 0.165 < 1 ok
internal compression parts: utilization $U_{c/t}$ = 0.098 < 1 ok

2.2.3.3. connection capacity

transformation parameter: β_j = 1.00

2.2.3.3.1. moment resistance

distance of tension-bolt-rows from centre of compression: h_1 = 86.8 mm, h_2 = 26.9 mm

resistance per bolt-row (tension)
row 1: $F_{tr,Rd}$ = 36.8 kN
row 2: $F_{tr,Rd}$ = 36.8 kN
$\Sigma F_{tr,Rd}^*$ = 73.5 kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd}$ = 36.8 kN
row 2: $F_{tr,Rd}$ = 36.8 kN
$\Sigma F_{tr,Rd}$ = 73.5 kN
potential failure by basic component 4

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

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 4.2$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^{*} = 73.5$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^{*} = 164.5$ kN

2.2.3.3.2. shear/bearing resistance

resistance per bolt-row
row 1: $F_{vr,Rd} = 14.6$ kN
row 2: $F_{vr,Rd} = 14.6$ kN
$\Sigma F_{vr,Rd} = 29.1$ kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd} = 29.1$ kN

2.2.3.3.3. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd} = 126.72$ kN
welds: $F_{w,Rd} = 116.48$ kN
shear resistance of end plate: $V_{ep,Rd} = F_{w,Rd} = 116.48$ kN
shear resistance of column web
$V_{wp,Rd} = 82.2$ kN

2.2.3.3.4. total

$M_{j,Rd} = 4.2$ kNm $N_{j,t,Rd} = 73.5$ kN $N_{j,c,Rd} = 164.5$ kN $V_{j,Rd} = 29.1$ kN $V_{wp,Rd} = 82.2$ kN $V_{ep,Rd} = 116.5$ kN

2.2.3.4. verifications

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

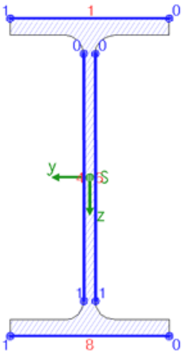
internal moment: $M_{Ed} = M_d = 5.60$ kNm
shear force: $V_{Ed} = |V_d| = 40.00$ kN
shear force: $V_{c,w,Ed} = M_d/z - (V_{c1}-V_{c2})/2 = 77.05$ kN, $z = 72.7$ mm
shear force: $V_{b,w,Ed} = 40.00$ kN

$M_{Ed}/M_{j,Rd} = 1.339 > 1$ not ok !!
$V_{Ed}/V_{j,Rd} = 1.374 > 1$ not ok !!
$V_{c,w,Ed}/V_{wp,Rd} = 0.937 < 1$ ok
$V_{b,w,Ed}/V_{ep,Rd} = 0.343 < 1$ 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 2: effective weld length $l_{eff} = 20.8$ mm < 30 mm $\Rightarrow$ has no effect !!
weld 2: effective weld length $l_{eff} = 20.8$ mm $< 6 \cdot a = 24.0$ mm $\Rightarrow$ has no effect !!
weld 6: effective weld length $l_{eff} = 20.8$ mm < 30 mm $\Rightarrow$ has no effect !!
weld 6: effective weld length $l_{eff} = 20.8$ mm $< 6 \cdot a = 24.0$ mm $\Rightarrow$ has no effect !!
calculation section:



weld 1: $a_w = 4.0$ mm $l_w = 60.0$ mm
weld 4: $a_w = 3.0$ mm $l_w = 93.4$ mm
weld 5: see weld 4
weld 8: $a_w = 4.0$ mm $l_w = 60.0$ mm

design values refering to centroid of the section:
$M_{y,Ed} = -5.60$ kNm, $V_{z,Ed} = 40.00$ kN
cross-sectional properties refering to centroid of the line cross-section:
$\Sigma A_w = 10.40$ cm², $A_{w,z} = 5.60$ cm², $\Sigma l_w = 30.7$ cm
$I_{w,y} = 213.54$ cm⁴, $I_{w,z} = 14.67$ cm⁴, $\Delta z_w = 0.0$ mm
verifications in weld edges:

75% decrease of stress by pressure contact

weld 1, pt. 0: $\sigma_{w,x}$ = 157.35 N/mm² ⇒ U_w = 0.618 < 1 ok

weld 4, pt. 0: $\sigma_{w,x}$ = 122.47 N/mm² $\tau_{w,z}$ = 71.38 N/mm² ⇒ U_w = 0.591 < 1 ok

pt. 1: $\sigma_{w,x}$ = -30.62 N/mm² $\tau_{w,z}$ = 71.38 N/mm² ⇒ U_w = 0.364 < 1 ok

weld 8, pt. 0: $\sigma_{w,x}$ = -39.34 N/mm² ⇒ U_w = 0.155 < 1 ok

Result:

weld 1, pt. 0: $\sigma_{w,x}$ = 157.35 N/mm²

Max: $\sigma_{1,w,Ed}$ = 222.52 N/mm² < $f_{1w,d}$ = 360.00 N/mm²,

$\sigma_{2,w,Ed}$ = 111.26 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U_w = 0.618 < 1 ok

2.2.3.4.3. verification of web stiffeners

compression stiffener (below)

$F_{c,Ed}$ = 77.05 kN

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 27.45 kN, $H = F \cdot e_F/e_H$ = 5.15 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: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 28.86 kN

F_{Ed} = 28.86 kN < F_{Rd} = 36.28 kN ⇒ U = 0.795 < 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 = 27.45 kN

F_{Ed} = 27.45 kN < F_{Rd} = 93.78 kN ⇒ U = 0.293 < 1 ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 711.11 kN/m, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 133.42 kN/m, b_1 = 19.3 mm

75% decrease of stress by pressure contact

$\sigma_{1,w,Ed}$ = 46.73 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.130 < 1 ok

$\sigma_{2,w,Ed}$ = 44.44 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U = 0.171 < 1 ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 158.85 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) !!

$\sigma_{1,w,Ed}$ = 17.20 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.048 < 1 ok

stiffener in tension (top)

$F_{t,Ed}$ = 77.05 kN

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 27.45 kN, $H = F \cdot e_F/e_H$ = 5.15 kN

cross-section at flange

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

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 28.86 kN

F_{Ed} = 28.86 kN < F_{Rd} = 36.28 kN ⇒ U = 0.795 < 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 = 27.45 kN

F_{Ed} = 27.45 kN < F_{Rd} = 93.78 kN ⇒ U = 0.293 < 1 ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 711.11 kN/m, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 133.42 kN/m, b_1 = 19.3 mm

$\sigma_{1,w,Ed}$ = 186.93 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.519 < 1 ok

$\sigma_{2,w,Ed}$ = 177.78 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U = 0.686 < 1 ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 158.85 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) !!

$\sigma_{1,w,Ed}$ = 68.78 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.191 < 1 ok

2.2.3.4.4. verification result

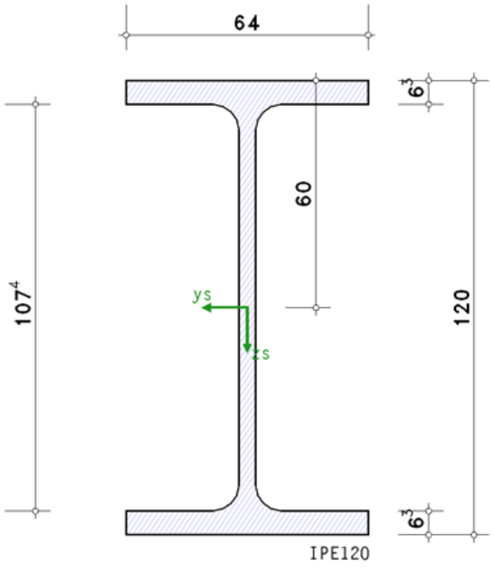
maximum utilization: max U = 1.374 > 1 not ok !!

failure at verification of bending: U = 1.339

failure at verification shear/bearing resistance: U = 1.374

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	$F_{z,Ed}$	V_{Ed}	U_σ	U_b	U_i	U
--	kN	kN	---	---	---	---
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 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
For info: compression $F_{z,Ed,ULS}/(s_s \cdot s_s')$ = 52.91 N/mm²
effective loading length $l_{eff} = s_s + 2 \cdot t_f$ = 42.6 mm
length of local loading:
refering to outer edge of flange $s_s = l_{eff} - 2 \cdot t_f$ = 30.0 mm / to periphery of web $s_w = l_{eff} + 2 \cdot r$ = 56.6 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

compression of single load at first cut of web:
local stresses $\sigma_{oz,Ed}$ = -160.6 N/mm²
$|\sigma_{oz,Ed}|$ = 160.6 N/mm² < σ_{Rd} = 235.0 N/mm² ⇒ U = 0.683 < 1 ok

stresses at first cut of web:
transverse loading $F_{z,Ed}$ = 40.0 kN
shear buckling: $h_p/t_p = 24.41 \leq 72 \cdot \varepsilon/\eta = 60.00$ ok
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 = 194.6 N/mm² < σ_{Rd} = 235.0 N/mm² ⇒ U = 0.828 < 1 ok
utilization at first cut of web max U_σ = 0.828 < 1 ok

maximum utilization: max U = 0.828 < 1 ok

2.3.3.2. buckling of transverse loading (ULS)

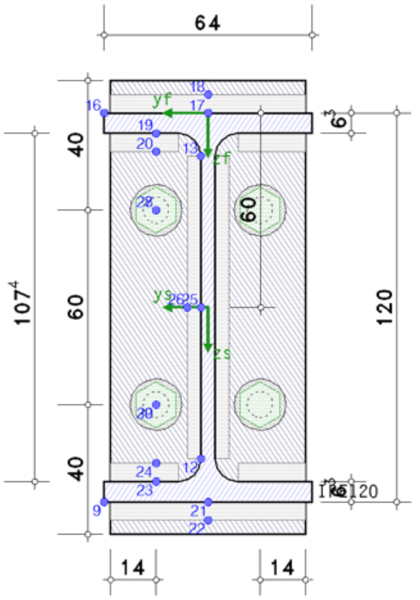
slenderness $\lambda_F = (F_y/F_{cr})^{1/2} = 0.326$, F_y = 76.5 kN
reduction factor χ_F = 1.000
resistance of buckling $F_{z,Rd} = f_y \cdot L_{eff} \cdot t_w/\gamma_{M1}$ = 69.59 kN, $L_{eff} = \chi_F \cdot l_y$ = 74.0 mm, l_y = 74.0 mm
verification: $F_{z,Ed}/F_{z,Rd}$ = 0.575 < 1 ok

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/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	6.4	60.0	0.0	100.0	0.0	am plate (web)	8.2(5)
x 27	16.0	30.0	50.0	100.0	0.0	am plate	8.1(14) 8.1(15)
28	16.0	30.0	90.0	0.0	0.0	am plate	8.1(10)
x 29	16.0	90.0	50.0	100.0	0.0	am plate	8.1(14) 8.1(15)
30	16.0	90.0	90.0	0.0	0.0	am plate	8.1(10)

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

loading

Lc 1: $M_{y,Ed} = -0.70$ kNm, $V_{z,Ed} = 5.00$ kN
Lc 2: $M_{y,Ed} = -2.10$ kNm, $V_{z,Ed} = 15.00$ kN

2.4.1. fatigue design

distances between bolts at end-plate:
horizontal: $e_2 = 14.0$ mm $> 1.5{\cdot}d_0 = 13.5$ mm
horizontal: $p_2 = 32.0$ mm $> 2.5{\cdot}d_0 = 22.5$ mm
vertical: $e_1 = 40.0$ mm $> 1.5{\cdot}d_0 = 13.5$ mm
vertical: $p_1 = 60.0$ mm $> 2.5{\cdot}d_0 = 22.5$ mm
vertical: $e_1 = 40.0$ mm $> 1.5{\cdot}d_0 = 13.5$ mm

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. 27: $y_f = 16.0$ mm, $z_f = 30.0$ mm $\Delta\sigma_{x,Ed} = 128.5$ N/mm² $\Delta\tau_{Ed} = 49.7$ N/mm²

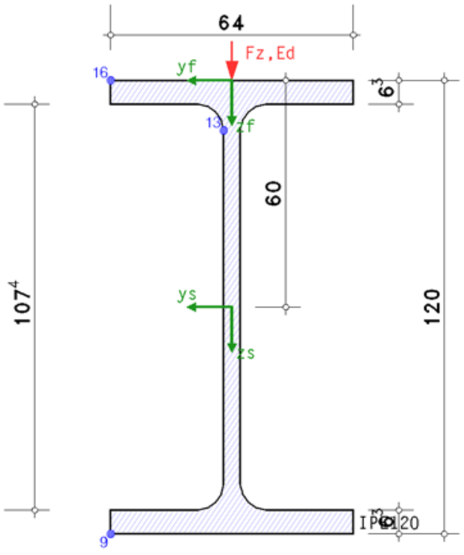
equivalent constand amplitude stress range:
pt. 27: $y_f = 16.0$ mm, $z_f = 30.0$ mm $\Delta\sigma_{x,f} = 38.5$ N/mm² $\Delta\tau_f = 14.9$ N/mm²

verification of notch stresses:
pt. 27: $y = 16.0$ mm, $z = 30.0$ mm $\Delta\sigma_{x,f} = 38.5$ N/mm² $< \Delta\sigma_{x,Rd,f} = 43.5$ N/mm² $\Rightarrow U_{\Delta\sigma x} = 0.887$ ok
$\Delta\tau_f = 14.9$ N/mm² $< \Delta\tau_{Rd,f} = 87.0$ N/mm² $\Rightarrow U_{\Delta\tau} = 0.172$ ok

fatigue design [pt. 27]: max U = 0.887 < 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)

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 $l_{eff} = s_s + 2 \cdot t_f$ = 42.6 mm
... at beam web σ_{oz} = -60.2 N/mm², τ_o = 12.0 N/mm²

stress ranges:
pt. 13: y_f = 2.2 mm, z_f = 13.3 mm $\Delta\tau_{Ed}$ = 42.3 N/mm² $\Delta\sigma_{z,Ed}$ = 60.2 N/mm²

equivalent constand amplitude stress range:
pt. 13: y_f = 2.2 mm, z_f = 13.3 mm $\Delta\tau_f$ = 12.7 N/mm² $\Delta\sigma_{z,f}$ = 18.1 N/mm²

verification of notch stresses:
pt. 13: y = 2.2 mm, z = 13.3 mm $\Delta\tau_f$ = 12.7 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm² ⇒ $U_{\Delta\tau}$ = 0.146 ok
$\Delta\sigma_{z,f}$ = 18.1 N/mm² < $\Delta\sigma_{z,Rd,f}$ = 139.1 N/mm² ⇒ $U_{\Delta\sigma z}$ = 0.130 ok

fatigue design [pt. 13]: max U = 0.146 < 1 ok

3. verifications left

3.1. table of results (ULS)

3.1.1. utilizations of each joint

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	0.434	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
$U_{j,\sigma}$: 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 each joint; U: utilization of each joint
*) maximum utilization of each joint

3.2. connection to column (ULS)

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

notes

no verification of haunch connection to beam.

no verfication for welds of welded section.

check of data

ok

3.2.1. table of results

utilization

Lc	$U_{\sigma,b}$	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*

$U_{\sigma,b}$: 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

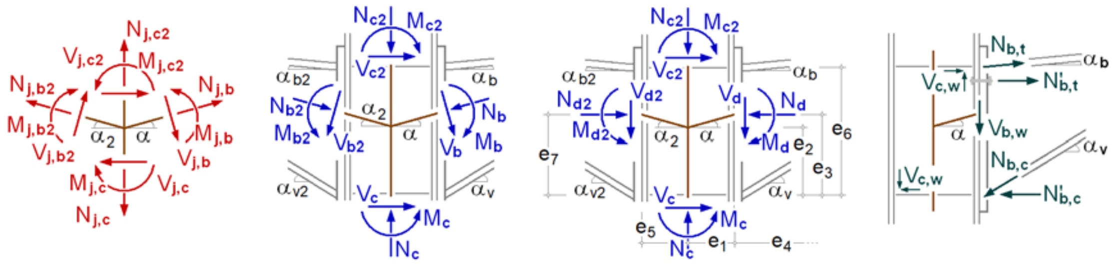
3.2.2. final result

maximum utilization: max U = 0.434 < 1 ok

3.2.3. Lc 1 (decisive)

3.2.3.1. design values

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



slope angle: $\alpha_b = 0.00°$, $\alpha_v = 18.43°$ ⇒ $\alpha = (\alpha_b+\alpha_v)/2 = 9.22°$

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

$N_d = -12.65$ kN, $M_d = 4.00$ kNm, $V_d = 37.95$ kN

partial internal forces and moments

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

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

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

3.2.3.2. resistance of cross-section in the periphery

elastic verification for N = 12.65 kN, $M_y = -4.00$ kNm, $V_z = 37.95$ kN

verification: $\sigma_v = 102.01$ N/mm² < $\sigma_{v,Rd} = 235.00$ N/mm² ⇒ $U_\sigma = 0.434$ < 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

3.2.3.3. connection capacity

transformation parameter: $\beta_j = 1.00$

3.2.3.3.1. moment resistance

distance between tension force and centre of compression: z = 163.5 mm

resistance

$F_{Rd} = 80.6$ kN

resistance of flanges (compression)

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

moment resistance

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

tension resistance
$N_{j,t,Rd} = F_{t,Rd} = 120.3$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^{*} = 161.3$ kN

3.2.3.3.2. shear resistance

shear resistance of column flange
$V_{cf,Rd} = 124.58$ kN
shear resistance of column web
$V_{wp,Rd} = 80.6$ kN

3.2.3.3.3. total

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

3.2.3.4. verifications

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

axial force:	$N_{b,Ed} = \lvert N_d \cdot \cos(\alpha) + V_d \cdot \sin(\alpha) \rvert = 6.41$ kN $<$ 5%$\cdot N_{pl,Rd} = 17.85$ kN $\Rightarrow$ moment resistance
internal moment:	$M_{Ed} = M_d - N_d \cdot z_{bu} = 5.02$ kNm, $z_{bu} = 80.6$ mm
shear force:	$V_{Ed} = \lvert V_d \rvert = 37.95$ kN
shear force:	$V_{c,w,Ed} = M_d/z - (V_{c1}-V_{c2})/2 = 24.46$ kN, $z = 163.5$ mm
shear force:	$V_{b,w,Ed} = 31.93$ kN

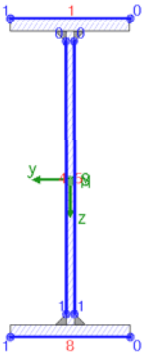
$M_{Ed}/M_{j,Rd} = 0.381 < 1$ ok
$V_{c,w,Ed}/V_{wp,Rd} = 0.303 < 1$ ok
$V_{b,w,Ed}/V_{cf,Rd} = 0.256 < 1$ ok

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

calculation section:



weld 1:	$a_w = 5.0$ mm	$l_w = 64.0$ mm
weld 4:	$a_w = 4.0$ mm	$l_w = 145.7$ 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:
$N_{Ed} = 12.65$ kN, $M_{y,Ed} = -4.00$ kNm, $V_{z,Ed} = 37.95$ kN
cross-sectional properties refering to centroid of the line cross-section:
$\Sigma A_w = 17.42$ cm², $A_{w,z} = 11.66$ cm², $\Sigma l_w = 41.9$ cm
$I_{w,y} = 620.73$ cm⁴, $I_{w,z} = 20.23$ cm⁴, $\Delta z_w = -4.3$ mm
verifications in weld edges:
75% decrease of stress by pressure contact

weld 1,	pt. 0:	$\sigma_{w,x} = 59.95$ N/mm²		$\Rightarrow U_w = 0.236 < 1$ ok
weld 4,	pt. 0:	$\sigma_{w,x} = 52.24$ N/mm²	$\tau_{w,z} = 32.55$ N/mm²	$\Rightarrow U_w = 0.258 < 1$ ok
	pt. 1:	$\sigma_{w,x} = -10.42$ N/mm²	$\tau_{w,z} = 32.55$ N/mm²	$\Rightarrow U_w = 0.162 < 1$ ok
weld 8,	pt. 0:	$\sigma_{w,x} = -12.40$ N/mm²		$\Rightarrow U_w = 0.049 < 1$ ok

Result:
weld 4, pt. 0: $\sigma_{w,x} = 52.24$ N/mm² $\tau_{w,z} = 32.55$ N/mm²
Max: $\sigma_{1,w,Ed} = 92.93$ N/mm² $< f_{1w,d} = 360.00$ N/mm²,
$\sigma_{2,w,Ed} = 36.94$ N/mm² $< f_{2w,d} = 259.20$ N/mm² $\Rightarrow U_w = 0.258 < 1$ ok

verification of deflection forces (directional method)
weld compression flange

compression force $N_{b,c}$ = 4.8 kN
weld angle φ = 108.4°:
$\sigma_{1,w,Ed}$ = 13.74 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.038 < 1 ok
$\sigma_{2,w,Ed}$ = 8.59 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U = 0.033 < 1 ok
weld angle φ = 71.6°:
$\sigma_{1,w,Ed}$ = 16.12 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.045 < 1 ok
$\sigma_{2,w,Ed}$ = 6.19 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U = 0.024 < 1 ok

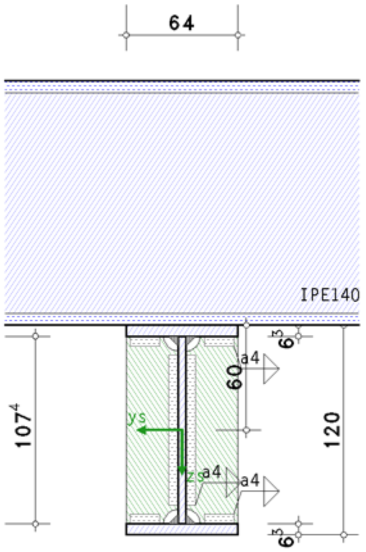
3.2.3.4.3. verification of web stiffeners

compression stiffener (below)
$F_{c,Ed}$ = 18.78 kN
forces per rib
F = 0.5·$F_{c,Ed}$ · (b_f-2·r-t_w)/b_f = 6.69 kN, H = F · e_F/e_H = 1.26 kN
assumption: stiffeners do not buckle: section class 1 ≤ 3 ok
cross-section at flange
compression resistance $N_{c,Rd}$ = (A·f_y) / γ_{M0} = 36.28 kN
design value: F_{Ed} = (F^2 + 3·H^2)$^{1/2}$ = 7.03 kN
F_{Ed} = 7.03 kN < F_{Rd} = 36.28 kN ⇒ U = 0.194 < 1 ok
cross-section at web
shear resistance V_{Rd} = (A_v·f_y) / ($3^{1/2}$·γ_{M0}) = 93.78 kN
design value: F_{Ed} = F = 6.69 kN
F_{Ed} = 6.69 kN < F_{Rd} = 93.78 kN ⇒ U = 0.071 < 1 ok
flange welds
design values: $F_{Ed}(\sigma_s)$ = F / (2·b_1) = 173.32 kN/m, $F_{Ed}(\tau_p)$ = H / (2·b_1) = 32.52 kN/m, b_1 = 19.3 mm
75% decrease of stress by pressure contact
$\sigma_{1,w,Ed}$ = 11.39 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.032 < 1 ok
$\sigma_{2,w,Ed}$ = 10.83 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U = 0.042 < 1 ok
web welds
design value: $F_{Ed}(\tau_p)$ = F / (2·l_1) = 38.72 kN/m, l_1 = 86.4 mm
75% decrease of stress by pressure contact
weld thickness a = 4.0 mm > a_{max} = 0.7·t_{min} = 3.1 mm (welding technology, s. DIN 18800) !!
$\sigma_{1,w,Ed}$ = 4.19 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.012 < 1 ok
stiffener in tension (top)
$F_{t,Ed}$ = 31.43 kN
forces per rib
F = 0.5·$F_{t,Ed}$ · (b_f-2·r-t_w)/b_f = 11.20 kN, H = F · e_F/e_H = 2.10 kN
cross-section at flange
tension resistance $N_{t,Rd}$ = 36.28 kN
design value: F_{Ed} = (F^2 + 3·H^2)$^{1/2}$ = 11.77 kN
F_{Ed} = 11.77 kN < F_{Rd} = 36.28 kN ⇒ U = 0.324 < 1 ok
cross-section at web
shear resistance V_{Rd} = (A_v·f_y) / ($3^{1/2}$·γ_{M0}) = 93.78 kN
design value: F_{Ed} = F = 11.20 kN
F_{Ed} = 11.20 kN < F_{Rd} = 93.78 kN ⇒ U = 0.119 < 1 ok
flange welds
design values: $F_{Ed}(\sigma_s)$ = F / (2·b_1) = 290.06 kN/m, $F_{Ed}(\tau_p)$ = H / (2·b_1) = 54.42 kN/m, b_1 = 19.3 mm
$\sigma_{1,w,Ed}$ = 76.25 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.212 < 1 ok
$\sigma_{2,w,Ed}$ = 72.51 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U = 0.280 < 1 ok
web welds
design value: $F_{Ed}(\tau_p)$ = F / (2·l_1) = 64.79 kN/m, l_1 = 86.4 mm
weld thickness a = 4.0 mm > a_{max} = 0.7·t_{min} = 3.1 mm (welding technology, s. DIN 18800) !!
$\sigma_{1,w,Ed}$ = 28.06 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.078 < 1 ok

3.2.3.4.4. verification result

maximum utilization: max U = 0.434 < 1 ok

3.3. 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.

3.3.1. table of results

Lc	$F_{z,Ed}$	N_{Ed}	V_{Ed}	U_r	U_σ	U
--	kN	kN	kN	---	---	---
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

3.3.2. final result

maximum utilization [Lc 1]: max U = 0.616 < 1 ok

3.3.3. Lc 1 (decisive)

cross-sectional properties: A = 12.79 cm², z_s = 60.0 mm, I_y = 306.31 cm⁴, y_s = 0.0 mm, I_z = 27.60 cm⁴

feed width by the beam $s_s' = 2 \cdot t_f + t_w + 2.828 \cdot a_w$ = 28.3 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 $F_{z,Ed,ULS}/(s_s \cdot s_s')$ = 52.91 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 $s_w = l_{eff}$ = 39.3 mm

3.3.3.1. 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):
<u>forces per rib</u>
$F = 0.5 \cdot F_{c,Ed}$ = 20.00 kN, $H = F \cdot e_F/e_H$ = 3.56 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 !!
<u>cross-section at flange</u>
compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0}$ = 50.09 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 20.93 kN
F_{Ed} = 20.93 kN < F_{Rd} = 50.09 kN ⇒ U = 0.418 < 1 ok
<u>cross-section at web</u>
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0})$ = 122.69 kN
design value: F_{Ed} = F = 20.00 kN
F_{Ed} = 20.00 kN < F_{Rd} = 122.69 kN ⇒ U = 0.163 < 1 ok
<u>flange welds</u>
design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 469.16 kN/m, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 83.62 kN/m, b_1 = 21.3 mm
$\sigma_{1,w,Ed}$ = 122.75 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.341 < 1 ok
$\sigma_{2,w,Ed}$ = 117.29 N/mm² < $f_{2w,d}$ = 259.20 N/mm² ⇒ U = 0.453 < 1 ok
<u>web welds</u>
design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 110.58 kN/m, l_1 = 90.4 mm
weld thickness a = 4.0 mm > $a_{max} = 0.7 \cdot t_{min}$ = 3.1 mm (welding technology, s. DIN 18800) !!

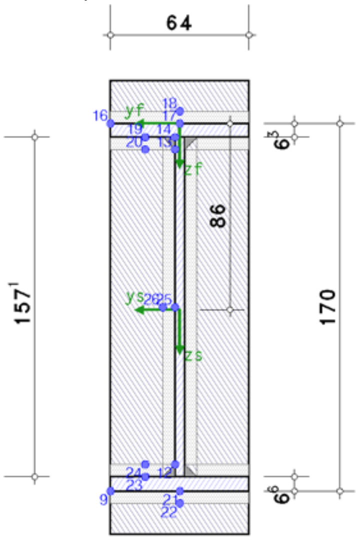
$\sigma_{1,w,Ed}$ = 47.88 N/mm² < $f_{1w,d}$ = 360.00 N/mm² ⇒ U = 0.133 < 1 ok
total: utilization of ribs U_R = 0.453 < 1 ok

stresses at first cut of web:
local transversal stresses $\sigma_{oz,Ed}$ = 0 N/mm²
Lc 1: N_{Ed} = 6.4 kN, $V_{z,Ed}$ = 39.5 kN
stresses $\sigma_{x,Ed}$ = 5.0 N/mm², $\tau_{xz,Ed}$ = 83.6 N/mm²
$|\sigma_{x,Ed}|$ = 5.0 N/mm² < σ_{Rd} = 235.0 N/mm² ⇒ U = 0.021 < 1 ok
$|\tau_{xz,Ed}|$ = 83.6 N/mm² < τ_{Rd} = 135.7 N/mm² ⇒ U = 0.616 < 1 ok
σ_v = 144.8 N/mm² < σ_{Rd} = 235.0 N/mm² ⇒ U = 0.616 < 1 ok
utilization at first cut of web max U_σ = 0.616 < 1 ok

maximum utilization: max U = 0.616 < 1 ok

3.4. 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	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)
° 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)

°: verification of the weld, *: notch stress $\Delta\sigma_{z,Rd}$ increased by one category

loading

Lc 1: $M_{y,Ed}$ = -0.50 kNm, $V_{z,Ed}$ = 5.00 kN
Lc 2: $M_{y,Ed}$ = -1.49 kNm, $V_{z,Ed}$ = 15.00 kN

3.4.1. fatigue design

cross-sectional properties: A = 15.19 cm², z_s = 86.0 mm, I_y = 695.85 cm⁴, y_s = 0.0 mm, I_z = 28.38 cm⁴

stress ranges:
pt. 21: y_f = 0.0 mm, z_f = 170.0 mm $\Delta\sigma_{x,Ed}$ = 13.2 N/mm²

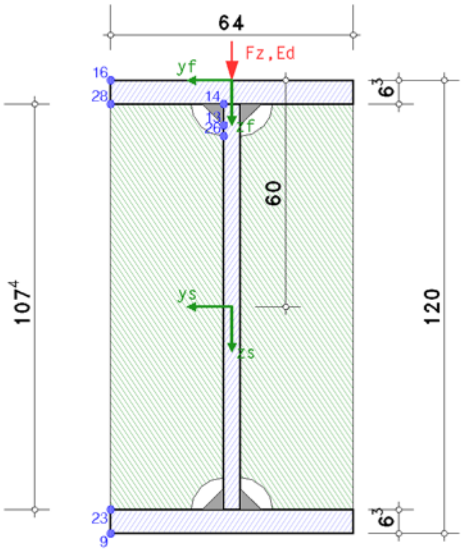
equivalent constand amplitude stress range:
pt. 21: y_f = 0.0 mm, z_f = 170.0 mm $\Delta\sigma_{x,f}$ = 13.2 N/mm²

verification of notch stresses:
pt. 21: y = 0.0 mm, z = 170.0 mm $\Delta\sigma_{x,f}$ = 13.2 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm² ⇒ $U_{\Delta\sigma x}$ = 0.421 ok

fatigue design [pt. 21]: max U = 0.421 < 1 ok

3.5. 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.	yf mm	zf mm	Δσx,Rd N/mm²	ΔτRd N/mm²	Δσz,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(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)

°: verification of the weld, *: notch stress $\Delta\sigma_{z,Rd}$ increased by one category

loading

Lc 1: $M_{y,Ed}$ = 0.01 kNm, $V_{z,Ed}$ = 5.00 kN
Lc 2: $M_{y,Ed}$ = 0.03 kNm, $V_{z,Ed}$ = 15.00 kN
transverse loading on top flange: vertical single load $F_{z,Ed}$ = 15.0 kN

3.5.1. fatigue design

cross-sectional properties: A = 12.79 cm², z_s = 60.0 mm, I_y = 306.31 cm⁴, y_s = 0.0 mm, I_z = 27.60 cm⁴

effective loading length $l_{eff} = s_s + 2 \cdot t_f$ = 39.3 mm
... at beam web σ_{oz} = -86.7 N/mm², τ_o = 17.3 N/mm² / ... at weld σ_{oz} = -47.7 N/mm², τ_o = 9.5 N/mm²

stress ranges:
pt. 13: y_f = 2.2 mm, z_f = 12.0 mm $\Delta\sigma_{x,Ed}$ = 0.3 N/mm² $\Delta\tau_{Ed}$ = 52.7 N/mm² $\Delta\sigma_{z,Ed}$ = 86.7 N/mm²

equivalent constand amplitude stress range:
pt. 13: y_f = 2.2 mm, z_f = 12.0 mm $\Delta\sigma_{x,f}$ = 0.3 N/mm² $\Delta\tau_f$ = 52.7 N/mm² $\Delta\sigma_{z,f}$ = 86.7 N/mm²

verification of notch stresses:
pt. 13: y = 2.2 mm, z = 12.0 mm
$\Delta\sigma_{x,f}$ = 0.3 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 87.0 N/mm² ⇒ $U_{\Delta\sigma x}$ = 0.004 ok
$\Delta\tau_f$ = 52.7 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm² ⇒ $U_{\Delta\tau}$ = 0.606 ok
$\Delta\sigma_{z,f}$ = 86.7 N/mm² > $\Delta\sigma_{z,Rd,f}$ = 34.8 N/mm² ⇒ $U_{\Delta\sigma z}$ = 2.494 not ok !!
interaction $U_i = U_{\Delta\sigma x}^3 + U_{\Delta\sigma z}^3 + U_{\Delta\tau}^5$ = 15.592 > 1 not ok !!

fatigue design [pt. 13]: max U = 2.494 > 1 not ok !!

4. final result

maximum utilization: max U = 2.494 > 1 not ok !!
fatigue of the connection / of the section

resistance not ensured !!

5. 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