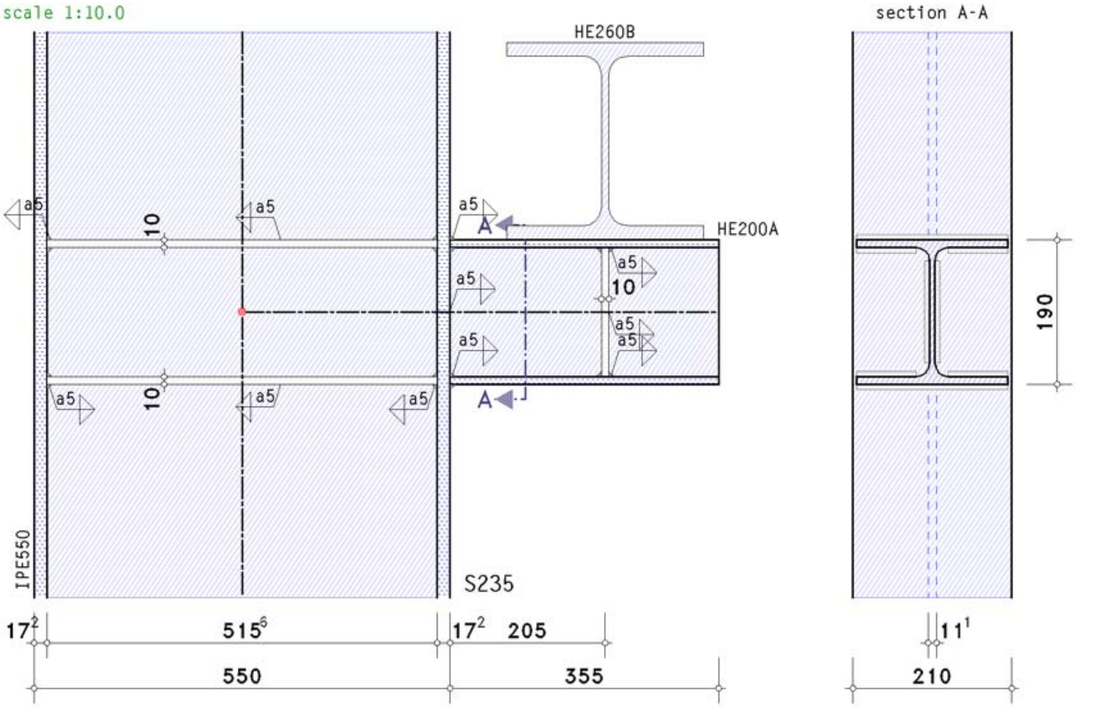
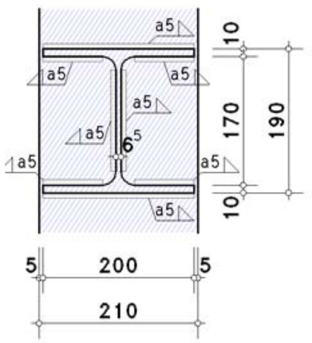
bracket EC 3-1-8 (12.10), NA: Deutschland 4H-EC3SK version: 6/2025-1b

1. input report



details (section A - A)



steel grade
steel grade S235
column parameters
section IPE550
reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 180.0 mm):
 thickness t_{st} = 10.0 mm, width b_{st} = 99.5 mm, length l_{st} = 515.6 mm
 recess at stiffeners c_{st} = 36.0 mm
 welds $a_{st,f}$ = 5.0 mm, $a_{st,w}$ = 5.0 mm
beam parameters
section HE200A
reinforcement of the section with transverse stiffeners:
 thickness t_{st} = 10.0 mm, width b_{st} = 96.8 mm, length l_{st} = 170.0 mm
 recess at stiffeners c_{st} = 27.0 mm
 welds $a_{st,f}$ = 5.0 mm, $a_{st,w}$ = 5.0 mm
welds
 beam flange top: fillet weld, weld thickness a = 5.0 mm
 beam web: fillet weld, weld thickness a = 5.0 mm
 beam flange below: fillet weld, weld thickness a = 5.0 mm
parameters of the connection
welded connection
distance of transverse loading Δa = 205.0 mm
verification of local loading:
 transverse loading on top flange of bracket by a load girder (crossing of girders) section HE260B
verification of fatigue:

damage equivalent stress factors $\lambda_\sigma = 0.315$, $\lambda_\tau = 0.500$

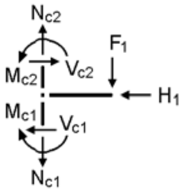
calculation of resistances

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)

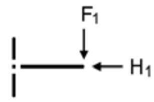
bracket load and internal forces and moments in the intersection point of system axes (ULS)



Lc	$F_{1,Ed}$
--	kN
1	89.00
2	61.50

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

bracket load (FLS)



Lc	$F_{1,Ed}$
--	kN
1	---
2	56.91

$F_{1,Ed}$: 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	$U_{j,b}$	$U_{j,\sigma}$	$U_{j,\tau}$	$U_{j,w}$	$U_{j,r}$	$U_{l,\sigma}$	$U_{l,r}$	U
--	---	---	---	---	---	---	---	---	---	---
1	0.586	0.375	0.586	0.236	0.285	0.370	0.290	0.375	0.253	0.586*
2	0.405	0.259	0.405	0.163	0.197	0.256	0.201	0.259	0.175	0.405

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	0.586	0.236	0.108	0.285	0.370	0.290	0.586*
2	0.405	0.163	0.075	0.197	0.256	0.201	0.405

$U_{\sigma,b}$: 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

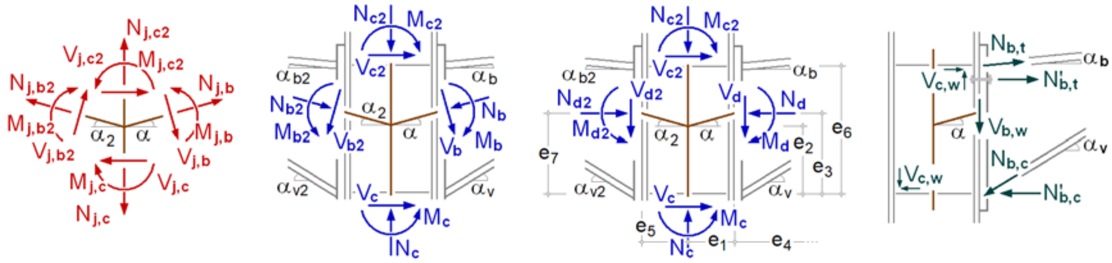
2.2.2. final result

maximum utilization [Lc 1]: max U = 0.586 < 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: $e_1 = 275.0$ mm, $e_3 = 90.0$ mm, $e_2 = 90.0$ mm, $e_6 = 180.0$ mm

internal forces and moments perpendicular to the connection planes

periphery beam

$M_d = 18.24$ kNm, $V_d = 89.00$ kN

partial internal forces and moments

$N_{b,t} = -N_d \cdot z_{bu}/z_b + M_d/z_b = 101.36$ kN, $z_b = 180.0$ mm, $z_{bu} = 90.0$ mm

$N_{b,c} = N_d \cdot z_{bo}/z_b + M_d/z_b = 101.36$ kN, $z_b = 180.0$ mm, $z_{bo} = 90.0$ 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, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 89.00$ kN

2.2.3.2. resistance of cross-section in the periphery

elastic verification for $M_y = -18.24$ kNm, $V_z = 89.00$ kN

verification: $\sigma_v = 137.61$ N/mm² $<$ $\sigma_{v,Rd} = 235.00$ N/mm² $\Rightarrow$ $U_\sigma = 0.586 < 1$ ok

c/t-ratio: outstand flange: utilization $U_{c/t} = 0.249 < 1$ ok

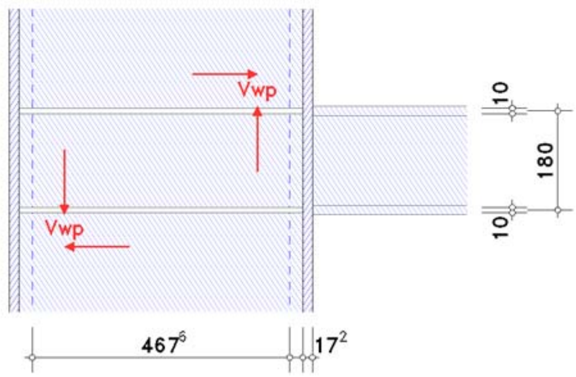
internal compression parts: utilization $U_{c/t} = 0.063 < 1$ ok

2.2.3.3. basic components

2.2.3.3.1. bc 1: Column web panel in shear

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 42.72$ kNm ($M_{j2} = 0$)

Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



slenderness of column web $d_c/t_{wc} = 42.13 < 69 \cdot \varepsilon = 69.00$ $\Rightarrow$ method applicable

plastic shear resistance without stiffeners $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_v) / (3^{1/2} \cdot \gamma_{M0}) = 883.4$ kN

placing of intermediate web stiffeners:

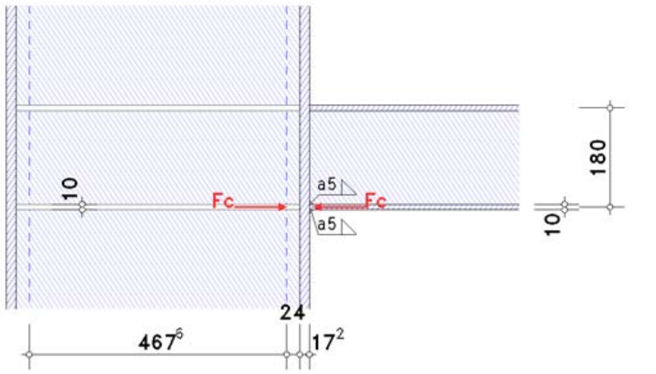
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/d_{st} = 81.1$ kN

$V_{wp,add,Rd} > 2 \cdot (M_{pl,fc,Rd} + M_{pl,st,Rd})/d_{st} = 53.5$ kN $\Rightarrow$ $V_{wp,add,Rd} = 53.5$ kN

plastic shear resistance with transverse stiffeners $V_{wp,Rd} = 936.9$ kN

2.2.3.3.2. bc 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 42.72$ kNm ($M_{j2} = 0$)



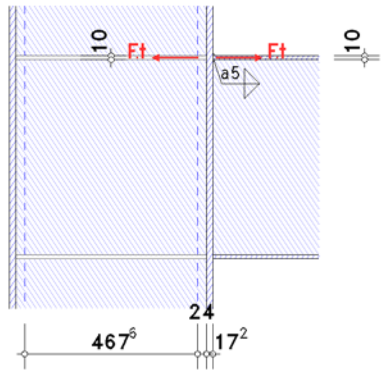
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

resistance without transverse stiffeners:
effective width of column web in transverse compression $b_{eff,c} = t_{f,b} + 2 \cdot 2^{1/2} \cdot a_b + 5 \cdot (t_{f,c} + s_c) = 230.1$ mm
reduction factor $k_w = 1.0$ ($\sigma_{com,Ed} = 0$)
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.921$
reduction factor for web buckling $\rho = (\lambda_p - 0.2) / \lambda_p^2 = 0.850$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.928$
resistance of web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 556.88$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 430.18$ kN (decisive)
reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: section class $2 \leq 3$ ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 180.0 mm
web height between the flanges $h_{wc} = 515.6$ mm
moment of inertia of stiffeners $I_{st} = 771.75$ cm^4
minimum moment of inertia for $a/h_{wc} = 0.35 < 2^{1/2}$: $I_{st,min} = 867.87$ cm$^4 > I_{st}$ not ok !!
$\Rightarrow$ transverse stiffeners no effect !

resistance of the upper beam flange:
resistance without transverse stiffeners:
effective width of column web in transverse compression $b_{eff,c} = t_{f,b} + 2 \cdot 2^{1/2} \cdot a_b + 5 \cdot (t_{f,c} + s_c) = 230.1$ mm
reduction factor $k_w = 1.0$ ($\sigma_{com,Ed} = 0$)
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.921$
reduction factor for web buckling $\rho = (\lambda_p - 0.2) / \lambda_p^2 = 0.850$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.928$
resistance of web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 556.88$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 430.18$ kN (decisive)
reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: section class $2 \leq 3$ ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 180.0 mm
web height between the flanges $h_{wc} = 515.6$ mm
moment of inertia of stiffeners $I_{st} = 771.75$ cm^4
minimum moment of inertia for $a/h_{wc} = 0.35 < 2^{1/2}$: $I_{st,min} = 867.87$ cm$^4 > I_{st}$ not ok !!
$\Rightarrow$ transverse stiffeners no effect !

2.2.3.3.3. bc 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 42.72$ kNm ($M_{j2} = 0$)

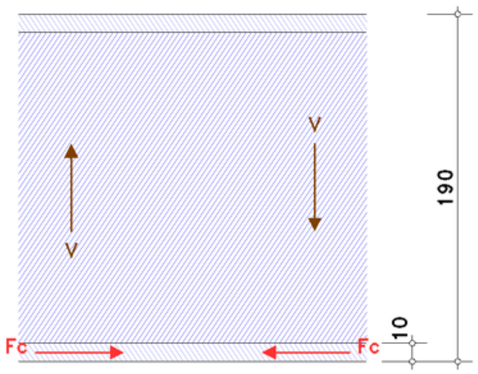


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

reinforcement of web with transverse stiffeners:
area of stiffeners incl. web $A_{st} = (2 \cdot b_{st}+t_{wc}) \cdot t_{st} = 21.00$ cm²
resistance of a column web with transverse tension $F_{t,wc,Rd} = A_{st} \cdot f_{y,st} / \gamma_{M0} = 493.5$ kN

2.2.3.3.4. bc 7: beam flange and web in compression

flange below: section class 1
web: section class 1
total: section class 1
section class of the beam: 1
taking into account the moment-shear force-interaction $V_{Ed} = 89.0$ kN



Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

stress due to bending with shear force: $V_{Ed} = 89.0\text{ kN} \leq 122.7\text{ kN} = V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 100.92$ kNm, $W_{pl} = 429.45$ cm³
resistance of a flange (and web) with compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 560.67$ kN

resistance of the upper beam flange:
stress due to bending with shear force: $V_{Ed} = 89.0\text{ kN} \leq 122.7\text{ kN} = V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 100.92$ kNm, $W_{pl} = 429.45$ cm³
resistance of a flange (and web) with compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 560.67$ kN

2.2.3.4. connection capacity

transformation parameter: $\beta_j = 1.00$

2.2.3.4.1. moment resistance

distance between tension force and centre of compression: $z = 180.0$ mm

resistance
$F_{Rd} = 430.2$ kN

resistance of flanges (compression)
$\Sigma F_{c,Rd}^* = 860.4$ kN

moment resistance
$M_{j,Rd} = F_{Rd} \cdot z = 77.4$ kNm
tension resistance
$N_{j,t,Rd} = F_{t,Rd} = 493.5$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 860.4$ kN

2.2.3.4.2. shear resistance

shear resistance of column flange
$V_{cf,Rd}$ = 312.71 kN
shear resistance of column web
decisive basic component: 1
$V_{wp,Rd}$ = 936.9 kN

2.2.3.4.3. total

$M_{j,Rd}$ = 77.4 kNm $N_{j,t,Rd}$ = 493.5 kN $N_{j,c,Rd}$ = 860.4 kN $V_{wp,Rd}$ = 936.9 kN $V_{cf,Rd}$ = 312.7 kN

2.2.3.5. verifications

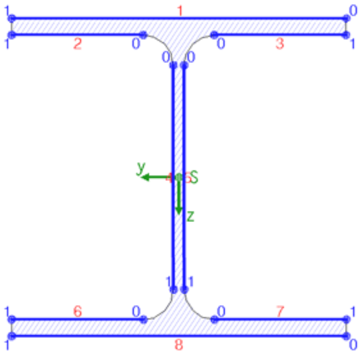
2.2.3.5.1. verification of the connection capacity by means of the component method

internal moment: $M_{Ed} = M_d$ = 18.24 kNm
shear force: $V_{Ed} = |V_d|$ = 89.00 kN
shear force: $V_{c,w,Ed} = M_d/z - (V_{c1}-V_{c2})/2$ = 101.36 kN, z = 180.0 mm
shear force: $V_{b,w,Ed}$ = 89.00 kN

$M_{Ed}/M_{j,Rd}$ = 0.236 < 1 ok
$V_{c,w,Ed}/V_{wp,Rd}$ = 0.108 < 1 ok
$V_{b,w,Ed}/V_{cf,Rd}$ = 0.285 < 1 ok

2.2.3.5.2. 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 4: weld thickness a = 5.0 mm > $a_{max} = 0.7 \cdot t_{min}$ = 4.5 mm (welding technology, s. DIN 18800) !!
calculation section:



weld 1: a_w = 5.0 mm l_w = 200.0 mm
weld 2: a_w = 5.0 mm l_w = 78.8 mm
weld 3: see weld 2
weld 4: a_w = 5.0 mm l_w = 134.0 mm
weld 5: see weld 4
weld 6: a_w = 5.0 mm l_w = 78.8 mm
weld 7: see weld 6
weld 8: a_w = 5.0 mm l_w = 200.0 mm

design values refering to centroid of the section:
$M_{y,Ed}$ = -18.24 kNm, $V_{z,Ed}$ = 89.00 kN
cross-sectional properties refering to centroid of the line cross-section:
ΣA_w = 49.15 cm², $A_{w,z}$ = 13.40 cm², Σl_w = 98.3 cm
$I_{w,y}$ = 3143.45 cm⁴, $I_{w,z}$ = 1328.35 cm⁴, Δz_w = 0.0 mm
distribution of internal forces and moments:
weld 1: N_w = 55.14 kN
weld 2: N_w = 19.43 kN
weld 3: see weld 2
weld 4: $M_{y,w}$ = -0.58 kNm
weld 5: see weld 4
weld 6: N_w = -19.43 kN
weld 7: see weld 6
weld 8: N_w = -55.14 kN
from conventionel distribution of shear force: $V_{z,w}$ = 89.00 kN
verifications in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x}$ = 55.14 N/mm²		⇒ U_w = 0.265 < 1 ok
weld 2,	pt. 0:	$\sigma_{w,x}$ = 49.34 N/mm²		⇒ U_w = 0.237 < 1 ok
weld 4,	pt. 0:	$\sigma_{w,x}$ = 38.89 N/mm²	$\tau_{w,z}$ = 66.42 N/mm²	⇒ U_w = 0.370 < 1 ok
	pt. 1:	$\sigma_{w,x}$ = -38.89 N/mm²	$\tau_{w,z}$ = 66.42 N/mm²	⇒ U_w = 0.370 < 1 ok
weld 6,	pt. 0:	$\sigma_{w,x}$ = -49.34 N/mm²		⇒ U_w = 0.237 < 1 ok
weld 8,	pt. 0:	$\sigma_{w,x}$ = -55.14 N/mm²		⇒ U_w = 0.265 < 1 ok

Result:
weld 4, pt. 0: $\sigma_{w,x}$ = 38.89 N/mm² $\tau_{w,z}$ = 66.42 N/mm²
Max: $F_{w,Ed}$ = 384.82 kN/m < $F_{w,Rd}$ = 1039.23 kN/m ⇒ U_w = 0.370 < 1 ok

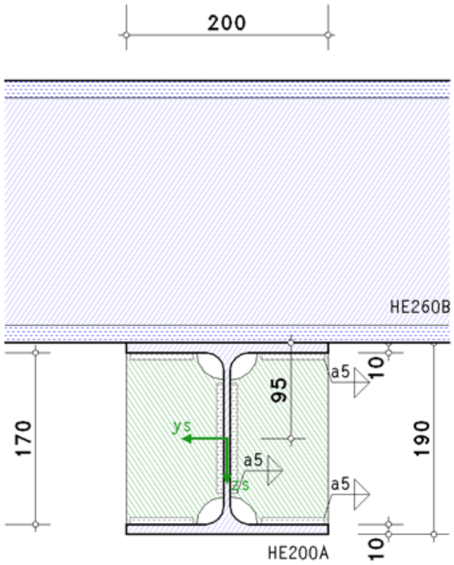
2.2.3.5.3. verification of web stiffeners

compression stiffener (below)
$F_{c,Ed}$ = 105.61 kN
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 37.95 kN, $H = F \cdot e_F/e_H$ = 4.98 kN
assumption: stiffeners do not buckle: section class 2 ≤ 3 ok
cross-section at flange
compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0}$ = 149.11 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 38.91 kN
F_{Ed} = 38.91 kN < F_{Rd} = 149.11 kN ⇒ U = 0.261 < 1 ok
cross-section at web
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0})$ = 601.86 kN
design value: $F_{Ed} = F$ = 37.95 kN
F_{Ed} = 37.95 kN < F_{Rd} = 601.86 kN ⇒ U = 0.063 < 1 ok
flange welds
resistance of a weld: $F_{w,Rd}$ = 1039.23 kN/m
design value: $F_{Ed} = (F^2 + H^2)^{1/2} / (2 \cdot b_1)$ = 301.59 kN/m, b_1 = 63.4 mm
F_{Ed} = 301.59 kN/m < F_{Rd} = 1039.23 kN/m ⇒ U = 0.290 < 1 ok
web welds
resistance of a weld: $F_{w,Rd}$ = 1039.23 kN/m
three-sided joint of ribs:
design value: $F_{Ed} = F / (2 \cdot l_1)$ = 42.77 kN/m, l_1 = 443.6 mm
F_{Ed} = 42.77 kN/m < F_{Rd} = 1039.23 kN/m ⇒ U = 0.041 < 1 ok
stiffener in tension (top)
$F_{t,Ed}$ = 105.61 kN
forces per rib
$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 37.95 kN, $H = F \cdot e_F/e_H$ = 4.98 kN
cross-section at flange
tension resistance $N_{t,Rd}$ = 149.11 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 38.91 kN
F_{Ed} = 38.91 kN < F_{Rd} = 149.11 kN ⇒ U = 0.261 < 1 ok
cross-section at web
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0})$ = 601.86 kN
design value: $F_{Ed} = F$ = 37.95 kN
F_{Ed} = 37.95 kN < F_{Rd} = 601.86 kN ⇒ U = 0.063 < 1 ok
flange welds
resistance of a weld: $F_{w,Rd}$ = 1039.23 kN/m
design value: $F_{Ed} = (F^2 + H^2)^{1/2} / (2 \cdot b_1)$ = 301.59 kN/m, b_1 = 63.4 mm
F_{Ed} = 301.59 kN/m < F_{Rd} = 1039.23 kN/m ⇒ U = 0.290 < 1 ok
web welds
resistance of a weld: $F_{w,Rd}$ = 1039.23 kN/m
three-sided joint of ribs:
design value: $F_{Ed} = F / (2 \cdot l_1)$ = 42.77 kN/m, l_1 = 443.6 mm
F_{Ed} = 42.77 kN/m < F_{Rd} = 1039.23 kN/m ⇒ U = 0.041 < 1 ok

2.2.3.5.4. verification result

maximum utilization: max U = 0.586 < 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	89.00	89.00	0.253	0.375	0.375*
2	61.50	61.50	0.175	0.259	0.259

$F_{z,Ed}$: 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 = 0.375 < 1 ok

2.3.3. Lc 1 (decisive)

cross-sectional properties: $A = 53.83\ cm^2$, $z_s = 95.0$ mm, $I_y = 3692.20\ cm^4$, $y_s = 0.0$ mm, $I_z = 1335.51\ cm^4$

feed width by the beam $s_s' = 2 \cdot t_f + t_w + 1.172 \cdot r = 47.6$ mm
feed length of load by the load beam $s_s = 2 \cdot t_f + t_w + 1.172 \cdot r = 73.1$ mm
For info: compression $F_{z,Ed,ULS}/(s_s \cdot s_s') = 25.58\ N/mm^2$
effective loading length $l_{eff} = s_s + 2 \cdot t_f = 93.1$ mm
length of local loading:
refering to outer edge of flange $s_s = l_{eff} - 2 \cdot t_f = 73.1$ mm / to periphery of web $s_w = l_{eff} + 2 \cdot r = 129.1$ mm

2.3.3.1. compression of web (ULS)

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

load transfer with transverse stiffeners (ribs):
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 35.04$ kN, $H = F \cdot e_F/e_H = 12.75$ kN
load component of web $F_{c,Ed} - 2 \cdot F = 18.91$ kN
assumption: stiffeners do not buckle: section class $2 \leq 3$ ok
note: $b_R > 91.8$ mm $\Rightarrow$ end return is not possible, pay attention to corrosion protection !!
cross-section at flange
compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 163.91$ kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 41.43$ kN
$F_{Ed} = 41.43$ kN $<$ $F_{Rd} = 163.91$ kN $\Rightarrow$ U = 0.253 < 1 ok
cross-section at web
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0}) = 157.39$ kN
design value: $F_{Ed} = F = 35.04$ kN
$F_{Ed} = 35.04$ kN $<$ $F_{Rd} = 157.39$ kN $\Rightarrow$ U = 0.223 < 1 ok
flange welds
design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 251.21$ kN/m, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 91.43$ kN/m, $b_1 = 69.8$ mm
$\sigma_{1,w,Ed} = 59.39\ N/mm^2 < f_{1w,d} = 360.00\ N/mm^2 \Rightarrow U = 0.165 < 1$ ok
$\sigma_{2,w,Ed} = 50.24\ N/mm^2 < f_{2w,d} = 259.20\ N/mm^2 \Rightarrow U = 0.194 < 1$ ok
web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 151.05$ kN/m, $l_1 = 116.0$ mm
weld thickness a = 5.0 mm > $a_{max} = 0.7 \cdot t_{min} = 4.5$ mm (welding technology, s. DIN 18800) !!
$\sigma_{1,w,Ed} = 52.33$ N/mm² < $f_{1w,d} = 360.00$ N/mm² ⇒ U = 0.145 < 1 ok
total: utilization of ribs $U_R = 0.253$ < 1 ok

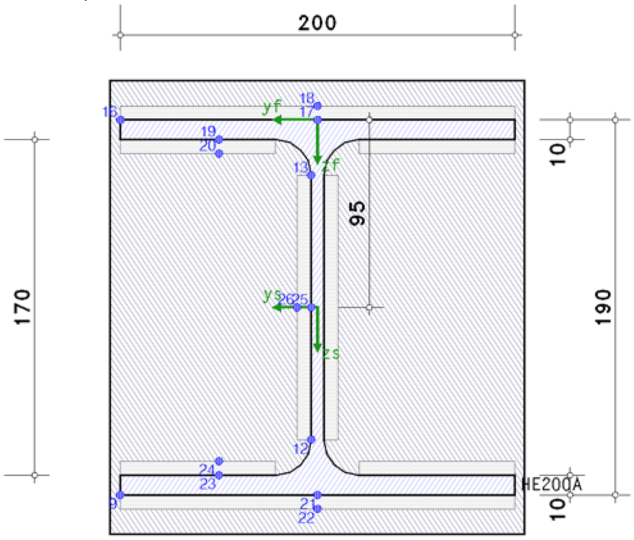
compression of single load at first cut of web:
reduced transverse loading $F_{z,Ed} = 18.9$ kN
local stresses $\sigma_{oz,Ed} = -22.5$ N/mm²
$|\sigma_{oz,Ed}| = 22.5$ N/mm² < $\sigma_{Rd} = 235.0$ N/mm² ⇒ U = 0.096 < 1 ok

stresses at first cut of web:
reduced transverse loading $F_{z,Ed} = 18.9$ kN
Lc 1: $V_{z,Ed} = 89.0$ kN
stresses $\tau_{xz,Ed} = 49.2$ N/mm²
$|\tau_{xz,Ed}| = 49.2$ N/mm² < $\tau_{Rd} = 135.7$ N/mm² ⇒ U = 0.363 < 1 ok
$\sigma_v = 88.2$ N/mm² < $\sigma_{Rd} = 235.0$ N/mm² ⇒ U = 0.375 < 1 ok
utilization at first cut of web max $U_\sigma = 0.375$ < 1 ok

maximum utilization: max U = 0.375 < 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/mm²	$\Delta\tau_{Rd}$ N/mm²	$\Delta\sigma_{z,Rd}$ N/mm²	notch point	EC 3-1-9, tab.
9	100.0	190.0	160.0	0.0	0.0	at bottom flange	8.1(2)
12	3.2	162.0	160.0	100.0	0.0	at beam web	8.1(2) 8.1(6)
13	3.2	28.0	160.0	100.0	160.0	at beam web	8.1(2) 8.1(6) 8.10(1)
16	100.0	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	50.0	10.0	36.0	0.0	0.0	am plate (top flange)	8.5(3)
20	50.0	17.1	80.0	0.0	0.0	am plate (top flange)	8.5(1)
° 21	0.0	190.0	36.0	0.0	0.0	am plate (bottom flange)	8.5(3)
22	0.0	197.1	80.0	0.0	0.0	am plate (bottom flange)	8.5(1)
° 23	50.0	180.0	36.0	0.0	0.0	am plate (bottom flange)	8.5(3)
24	50.0	172.9	80.0	0.0	0.0	am plate (bottom flange)	8.5(1)
° 25	3.2	95.0	36.0	80.0	0.0	am plate (web)	8.5(3) 8.5(8)
26	10.3	95.0	0.0	100.0	0.0	am plate (web)	8.2(5)

°: verification of the weld

loading

Lc 1: $M_{y,Ed} = 0.00$ kNm, $V_{z,Ed} = 0.00$ kN
Lc 2: $M_{y,Ed} = -11.67$ kNm, $V_{z,Ed} = 56.91$ kN

2.4.1. fatigue design

cross-sectional properties: $A = 53.83\ cm^2$, $z_s = 95.0\ mm$, $I_y = 3692.20\ cm^4$, $y_s = 0.0\ mm$, $I_z = 1335.51\ cm^4$

elastic stresses / stress ranges:
$\Delta\sigma_{x,Ed} = \sigma_{x,max} - \sigma_{x,min}$, $\tau_{Ed} = \tau_{xz,max} - \tau_{xz,min}$
pt. 21: $y_f = 0.0\ mm$, $z_f = 190.0\ mm$ Lc 1: $\sigma_x = 0.0\ N/mm^2$
2: $\sigma_x = -38.1\ N/mm^2$
$\Delta\sigma_{x,Ed} = 38.1\ N/mm^2$

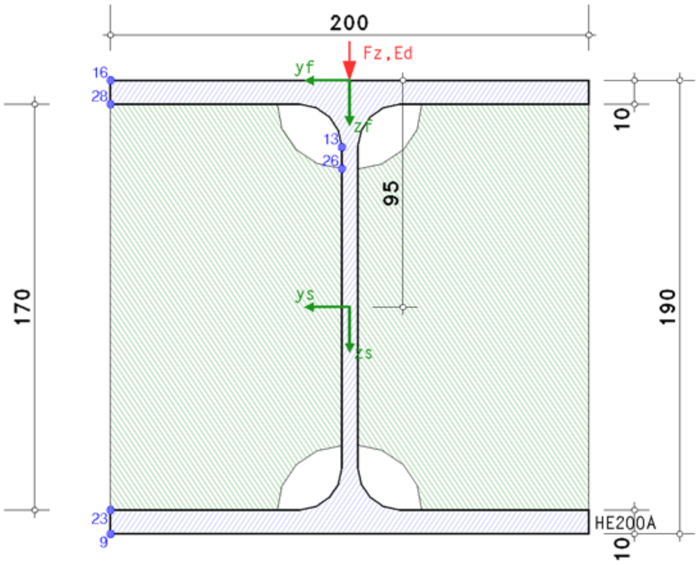
equivalent constand amplitude stress range:
$\Delta\sigma_{x,f} = \Delta\sigma_{x,Ed} \cdot \lambda_\sigma$, $\Delta\tau_f = \Delta\tau_{Ed} \cdot \lambda_\tau$
pt. 21: $y_f = 0.0\ mm$, $z_f = 190.0\ mm$ $\Delta\sigma_{x,f} = 12.0\ N/mm^2$

valid notch stresses:
$\Delta\sigma_{x,Rd,f} = \Delta\sigma_{x,Rd}/\gamma_{Mf}$, $\Delta\tau_{Rd,f} = \Delta\tau_{Rd}/\gamma_{Mf}$
pt. 21: $y_f = 0.0\ mm$, $z_f = 190.0\ mm$ $\Delta\sigma_{x,Rd,f} = 31.3\ N/mm^2$

verification of notch stresses:
pt. 21: $y = 0.0\ mm$, $z = 190.0\ mm$ $\Delta\sigma_{x,f} = 12.0\ N/mm^2 < \Delta\sigma_{x,Rd,f} = 31.3\ N/mm^2 \Rightarrow U_{\Delta\sigma x} = 0.384$ ok

fatigue design [pt. 21]: max $U = 0.384 < 1$ 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	100.0	190.0	160.0	0.0	0.0	at bottom flange	8.1(2)
13	3.2	28.0	160.0	100.0	160.0	at beam web	8.1(2) 8.1(6) 8.10(1)
16	100.0	0.0	160.0	0.0	0.0	at top flange	8.1(2)
23	100.0	180.0	80.0	0.0	0.0	due to transv.stiff.	8.4(7)
26	3.2	37.0	0.0	100.0	0.0	due to transv.stiff.	8.1(6)
28	100.0	10.0	80.0	0.0	0.0	due to transv.stiff.	8.4(7)

loading
Lc 1: $V_{z,Ed} = 0.00\ kN$
Lc 2: $V_{z,Ed} = 56.91\ kN$
transverse loading on top flange: vertical single load $F_{z,Ed} = 56.9\ kN$

2.5.1. fatigue design

cross-sectional properties: $A = 53.83\ cm^2$, $z_s = 95.0\ mm$, $I_y = 3692.20\ cm^4$, $y_s = 0.0\ mm$, $I_z = 1335.51\ cm^4$

effective loading length $l_{eff} = s_s + 2 \cdot t_f = 93.1\ mm$
effective loading length refered ...
... to outer edge of flange $s_s = l_{eff} - 2 \cdot t_f = 73.1\ mm$ / ... to periphery of web $s_w = l_{eff} + 2 \cdot r = 129.1\ mm$
local stresses ...
... at beam web $\sigma_{oz} = -67.8\ N/mm^2$, $\tau_o = 13.6\ N/mm^2$

elastic stresses / stress ranges:
$\Delta\sigma_{x,Ed} = \sigma_{x,max} - \sigma_{x,min}$, $\tau_{Ed} = \tau_{xz,max} - \tau_{xz,min} + 2 \cdot \tau_o$, $\Delta\sigma_{z,Ed} = -\sigma_{oz}$
pt. 13: $y_f = 3.2\ mm$, $z_f = 28.0\ mm$ Lc 1: $\tau_{xz} = 0.0\ N/mm^2$

2: τ_{xz} = 47.2 N/mm²
$\Delta\tau_{Ed}$ = 74.3 N/mm² $\Delta\sigma_{z,Ed}$ = 67.8 N/mm²

equivalent constand amplitude stress range:
$\Delta\sigma_{x,f} = \Delta\sigma_{x,Ed}\cdot\lambda_\sigma$, $\Delta\tau_f = \Delta\tau_{Ed}\cdot\lambda_\tau$, $\Delta\sigma_{z,f} = \Delta\sigma_{z,Ed}\cdot\lambda_\sigma$
pt. 13: y_f = 3.2 mm, z_f = 28.0 mm $\Delta\tau_f$ = 37.2 N/mm² $\Delta\sigma_{z,f}$ = 21.4 N/mm²

valid notch stresses:
$\Delta\sigma_{x,Rd,f} = \Delta\sigma_{x,Rd}/\gamma_{Mf}$, $\Delta\tau_{Rd,f} = \Delta\tau_{Rd}/\gamma_{Mf}$, $\Delta\sigma_{z,Rd,f} = \Delta\sigma_{z,Rd}/\gamma_{Mf}$
pt. 13: y_f = 3.2 mm, z_f = 28.0 mm $\Delta\tau_{Rd,f}$ = 87.0 N/mm² $\Delta\sigma_{z,Rd,f}$ = 139.1 N/mm²

verification of notch stresses:
pt. 13: y = 3.2 mm, z = 28.0 mm $\Delta\tau_f$ = 37.2 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm² ⇒ $U_{\Delta\tau}$ = 0.427 ok
$\Delta\sigma_{z,f}$ = 21.4 N/mm² < $\Delta\sigma_{z,Rd,f}$ = 139.1 N/mm² ⇒ $U_{\Delta\sigma z}$ = 0.154 ok

fatigue design [pt. 13]: max U = 0.427 < 1 ok

3. final result

maximum utilization: max U = 0.586 < 1 ok
connection to column

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