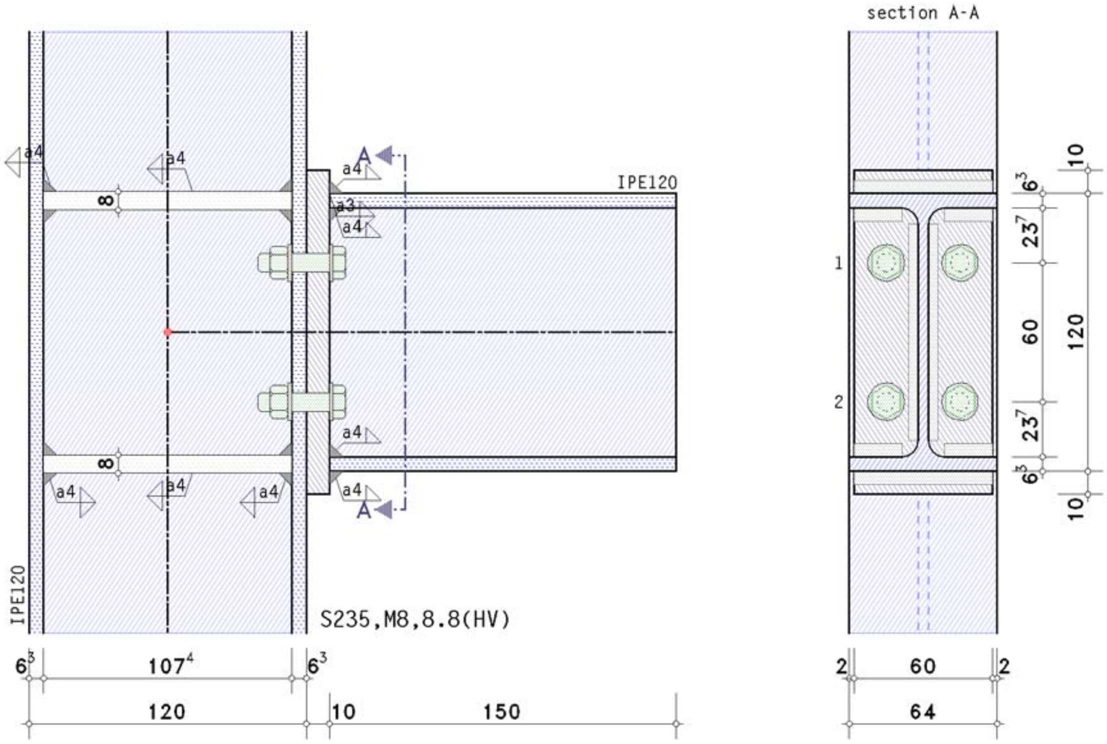
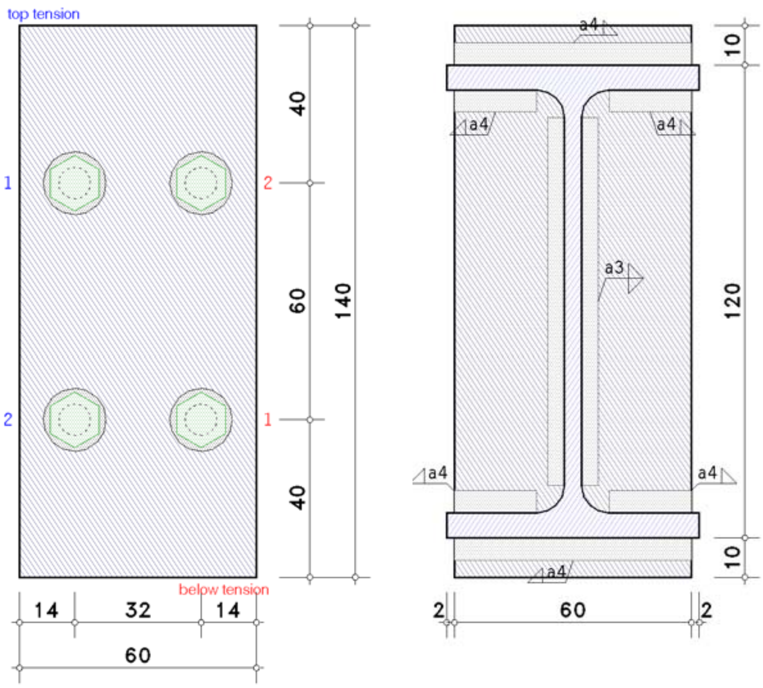
POS. 33: RE / BOLTED

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, calculation parameters:

char. yield strength $f_{y,t\leq 40mm}$ = 235.0 N/mm², $f_{y,t\leq 80mm}$ = 215.0 N/mm², $f_{y,t>80mm}$ = 215.0 N/mm²

char. tensile strength $f_{u,t\leq 40mm}$ = 360.0 N/mm², $f_{u,t\leq 80mm}$ = 360.0 N/mm², $f_{u,t>80mm}$ = 360.0 N/mm²

correlation value of fillet weld β_w = 0.80

elastic modulus E = 210000.0 N/mm²

column parameters

section IPE120

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, gewalzt with r = 7.0 mm

rolled section, root radius r = 7.0 mm

clear depth of web without root/weld d_w = 93.4 mm

width of one flange side without root/weld c_f = 22.8 mm
centroid distance from top z_s = 60.0 mm
cross-sectional area A = 13.21 cm², perimeter U = 47.52 cm
second moments of area I_y = 317.75 cm⁴, I_z = 27.66 cm⁴
plastic section moduli $W_{pl,y}$ = 60.72 cm³, $W_{pl,z}$ = 13.57 cm³
effective shear arean A_{vy} = 8.06 cm², A_{vz} = 6.31 cm²
torsional moment of inertia I_T = 1.81 cm⁴

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
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, gewalzt with r = 7.0 mm
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bolts

bolt class 8.8
calculation parameters:
char. yield strength f_{yb} = 640.0 N/mm²
char. tensile strength f_{ub} = 800.0 N/mm²
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)
normal clearance
calculation parameters:
shaft diameter d = 8.0 mm, clearance Δd = 1.0 mm $\Rightarrow$ hole diameter d_0 = 9.0 mm
gross cross-section area A = 0.503 cm²
tensile stress area A_s = 0.366 cm²
diameter of the bolt head (across flats dimension) d_s = 13.00 mm
diameter of the bolt head (across points dimension) d_e = 14.20 mm
thickness of the bolt head t_k = 5.3 mm
thickness of nut t_m = 7.9 mm
diameter of the plate under the bolt or the nut d_p = 16.0 mm
thickness of the plate under the bolt or the nut t_p = 1.6 mm
washer double-sided
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 fatigue:
damage equivalent stress factors λ_σ = 1.000, λ_τ = 1.000

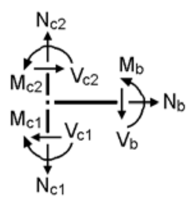
calculation of resistances

verification of bracket-column connection, limit state of resistance (ULS)
fatigue design of the connection and of bracket cross-section, limit state of fatigue (FLS)

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

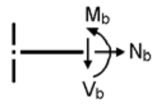
Lc	$M_{j,b1,Ed}$	$V_{j,b1,Ed}$
--	kNm	kN
1	-8.00	40.00

$M_{j,b1,Ed}$, $V_{j,b1,Ed}$: internal forces at node



internal forces and moments in the intersection point of system axes (FLS)

Lc	$M_{j,b1,Ed}$	$V_{j,b1,Ed}$
--	kNm	kN
1	-1.00	5.00
2	-3.00	15.00



$M_{j,b1,Ed}$, $V_{j,b1,Ed}$: internal forces at node

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 ⇒ fatigue strength γ_{Mf} = 1.15

2. verifications

2.1. connection to column (ULS)

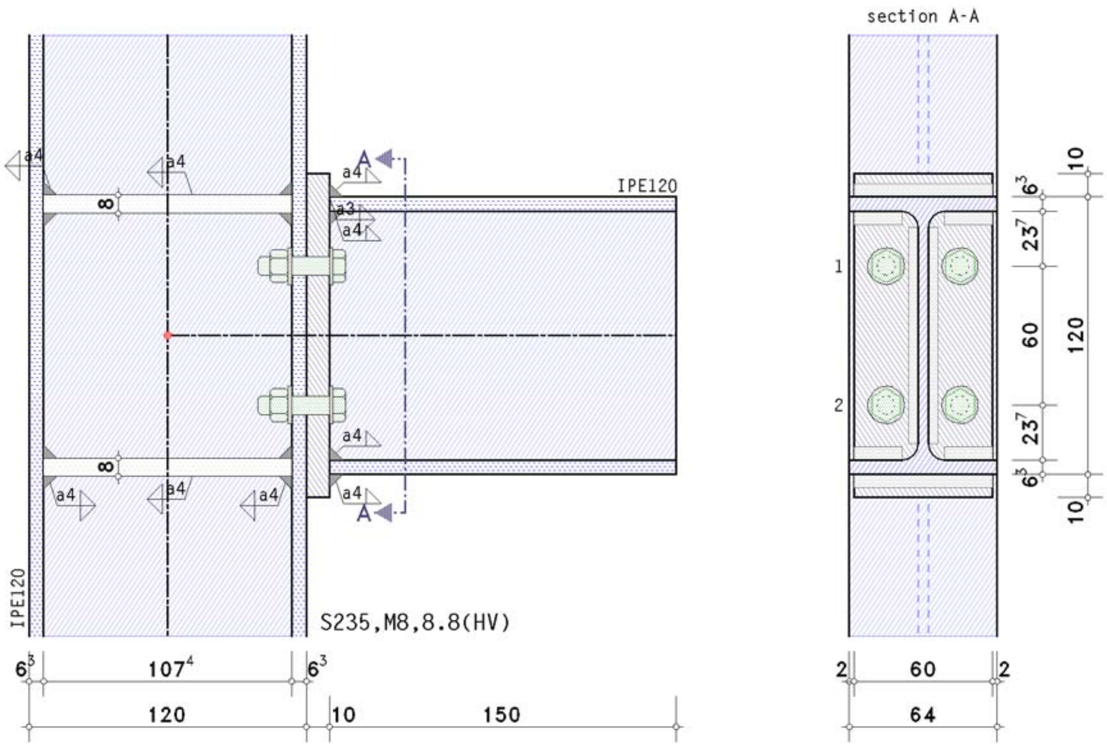
conversion internal forces at node -> bracket loads

moment error $\Delta M_{Ed} = M_{j,b,Ed} + F_{Ed} \cdot \Delta a - H_{Ed} \cdot \Delta h$ with Δa = 200.0 mm, Δh = 60.0 mm

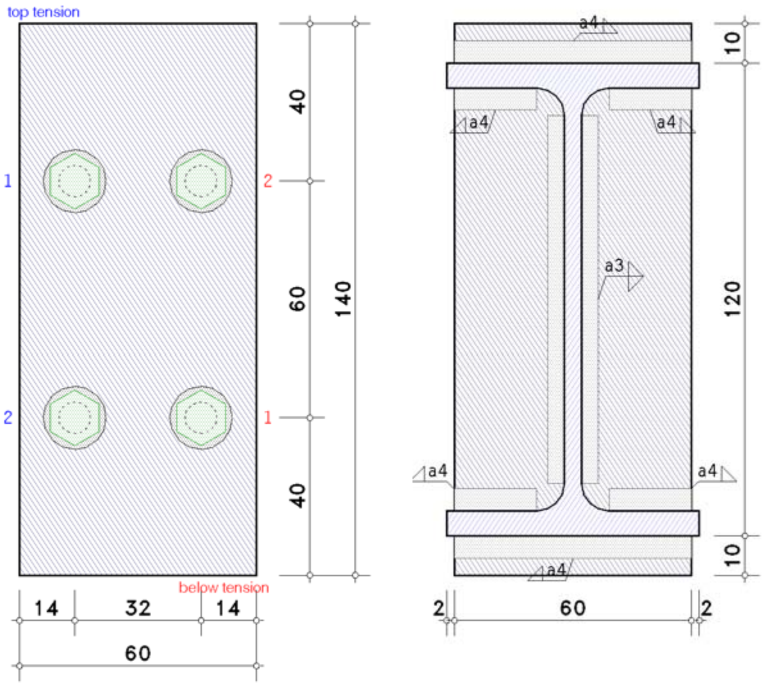
$F_{1,Ed}$ = 40.00 kN

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

2.1.1. input report



details (section A - A)



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correlation value of fillet weld β_w = 0.80

elastic modulus E = 210000.0 N/mm²

column parameters

section IPE120

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, gewalzt with r = 7.0 mm

rolled section, root radius r = 7.0 mm

clear depth of web without root/weld d_w = 93.4 mm

width of one flange side without root/weld c_f = 22.8 mm

centroid distance from top z_s = 60.0 mm

cross-sectional area A = 13.21 cm², perimeter U = 47.52 cm

second moments of area I_y = 317.75 cm⁴, I_z = 27.66 cm⁴

plastic section moduli $W_{pl,y}$ = 60.72 cm³, $W_{pl,z}$ = 13.57 cm³

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torsional moment of inertia I_T = 1.81 cm⁴

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

bolts

bolt class 8.8

calculation parameters:

char. yield strength f_{yb} = 640.0 N/mm²

char. tensile strength f_{ub} = 800.0 N/mm²

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)

normal clearance

calculation parameters:

shaft diameter d = 8.0 mm, clearance Δd = 1.0 mm $\Rightarrow$ hole diameter d_0 = 9.0 mm

gross cross-section area A = 0.503 cm²

tensile stress area A_s = 0.366 cm²

diameter of the bolt head (across flats dimension) d_s = 13.00 mm

diameter of the bolt head (across points dimension) d_e = 14.20 mm

thickness of the bolt head t_k = 5.3 mm

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diameter of the plate under the bolt or the nut d_p = 16.0 mm

thickness of the plate under the bolt or the nut t_p = 1.6 mm

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shear plane passes through the unthreaded portion of the bolt

beam parameters

section IPE120

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, gewalzt with r = 7.0 mm

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width of one flange side without root/weld c_f = 22.8 mm
centroid distance from top z_s = 60.0 mm
cross-sectional area A = 13.21 cm^2, perimeter U = 47.52 cm
second moments of area I_y = 317.75 cm^4, I_z = 27.66 cm^4
plastic section moduli $W_{pl,y}$ = 60.72 cm^3, $W_{pl,z}$ = 13.57 cm^3
effective shear arean A_{vy} = 8.06 cm^2, A_{vz} = 6.31 cm^2
torsional moment of inertia I_T = 1.81 cm^4

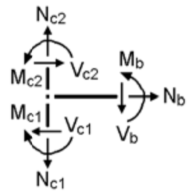
verification parameters

bolted end-plate connection

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 in connection:
2 bolt-rows with 2 bolts
all bolt-rows considered individually
all bolt-rows for shear transfer (rows 1-2)
bolt groups generated automatically, considering all groups reg. row 1
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
welds at the connection point:
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

internal forces and moments in the intersection point of system axes

Lc 1: $M_{j,b,Ed}$ = -8.00 kNm $V_{j,b,Ed}$ = 40.00 kN



partial safety factors for material

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
prestressing of high strength bolts γ_{M7} = 1.10

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 (Δe_{wc} = -1.2 mm $\leq$ 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

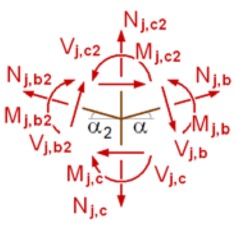
bolt distance from column edge

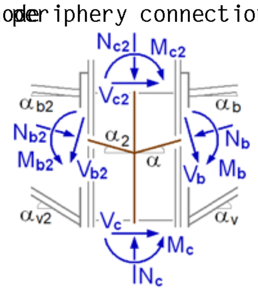
horizontal:	e_2 = 16.0 mm > $1.2 \cdot d_0$ = 10.8 mm,	e_2 = 16.0 mm < $4 \cdot t$ + 40 mm = 65.2 mm

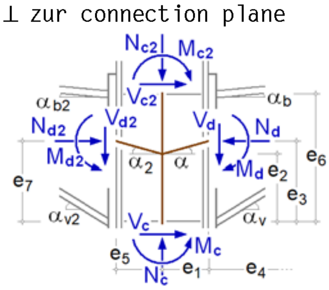
2.1.2. Lc 1

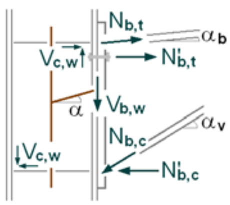
2.1.2.1. design values

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









sign definition of statics: a positive axial force means tension, a positive bending moment produces tension at the bottom
⇒ transformation to EC3: a positive axial force means compression, a positive bending moment produces tension at the top

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 in the intersection point of system axes (input beam)
$M_{j,b,Ed}$ = -8.00 kNm, $V_{j,b,Ed}$ = 40.00 kN

transformation sign convention of statics -> sign convention of EC3

$M_{j,b,Ed}$ = 8.00 kNm, $V_{j,b,Ed}$ = 40.00 kN

transformation node values -> joint values

$M_{b,Ed}$ = 5.60 kNm, $V_{b,Ed}$ = 40.00 kN

transformation joint values -> design values

M_d = 5.60 kNm, V_d = 40.00 kN

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

2.1.2.2. resistance of cross-section in the periphery

plastic verification for M_y = -5.20 kNm, V_z = 40.00 kN

valid normal/shear stress: $\sigma_{x,Rd}$ = 235.0 N/mm², τ_{Rd} = 135.7 N/mm²

top flange: resistance forces $N_{max,O}$ = 94.75 kN, $N_{min,O}$ = -94.75 kN

bottom flange: resistance forces $N_{max,U}$ = 94.75 kN, $N_{min,U}$ = -94.75 kN

web: shear force V_S = 40.00 kN, shearspg. τ_S = 79.96 N/mm² $\Rightarrow$ $U_{\tau,S}$ = 0.589

resistance forces $N_{max,S}$ = 94.98 kN, $N_{min,S}$ = -94.98 kN

main bending: M_y = -5.20 kNm, resistance moments $M_{y,max}$ = 13.47 kNm, $M_{y,min}$ = -13.47 kNm $\Rightarrow$ U_{My} = 0.386

total (possibly due to load increase): max U = 0.592 < 1 ok

c/t-ratio reg. section class 3:

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

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

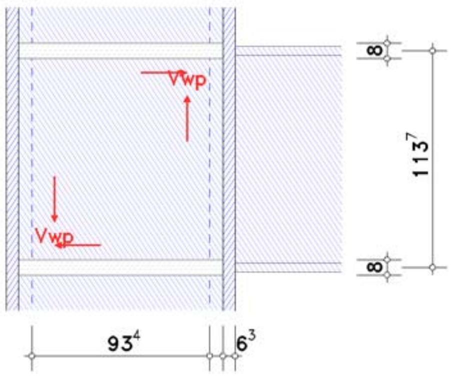
total: utilization $U_{c/t}$ = 0.165 < 1 ok

2.1.2.3. basic components

end-plate joint: basic components: 1, 2, 3, 4, 5, 7, 8, 10, 11, 12

2.1.2.3.1. 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} = 8.00 kNm (M_{j2} = 0)



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

assumption

slenderness of column web d_c/t_{wc} = 21.23 < $69 \cdot \varepsilon$ = 69.00, ε = 1.00 $\Rightarrow$ method applicable

shear area

shear area A_v = 6.31 cm²

plastic shear resistance

plastic shear resistance without stiffeners $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_v) / (3^{1/2} \cdot \gamma_{M0})$ = 77.0 kN

placing of intermediate web stiffeners:

plastic section modulus of a column flange $W_{pl,fc} = b_{fc} \cdot t_{fc}^2/4$ = 0.64 cm³

plastic section modulus of a web stiffener $W_{pl,st} = 2 \cdot b_{st} \cdot t_{st}^2/4$ = 0.95 cm³

plastic moment resistance of column flange $M_{pl,fc,Rd} = (W_{pl,fc} \cdot f_{y,c}) / \gamma_{M0}$ = 0.15 kNm

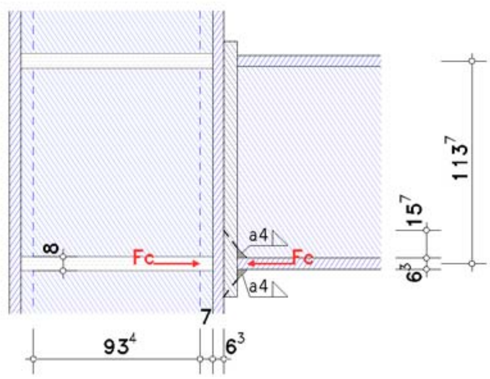
plastic moment resistance of the web stiffener $M_{pl,st,Rd} = (W_{pl,st} \cdot f_{y,st}) / \gamma_{M0}$ = 0.22 kNm

additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/d_{st}$ = 5.3 kN $\leq$ $2 \cdot (M_{pl,fc,Rd}+M_{pl,st,Rd})/d_{st}$ = 6.6 kN ok

plastic shear resistance with transverse stiffeners $V_{wp,Rd}$ = 82.2 kN

2.1.2.3.2. bc 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.00 \le 2$ for $M_{j1} = 8.00$ kNm ($M_{j2} = 0$)



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

resistance without transverse stiffeners

effective width

effective width of column web in transverse compression $b_{eff,c} = t_{f,b} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{f,c}+s_c) + s_p = 98.5$ mm,
$a_p = 4.0$ mm, $s_c = 7.0$ mm, $s_p = 14.3$ mm

longitudinal compressive stress in web

reduction factor $k_w = 1.0$ ($\sigma_{com,Ed} = 0$)

plate buckling

plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.679$, $E = 210000$ N/mm²
reduction factor for web buckling $\rho = 1$

shear area

shear area $A_v = 6.31$ cm²

interaction with shear stress

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = \omega_1 = 0.787$
with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w/A_v)^2]^{1/2} = 0.787$

resistance of web in transverse compression

$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 80.14$ kN, $f_{y,w} = 235.0$ N/mm²
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 72.86$ kN (decisive)

reinforcement of web with transverse stiffeners:

assumption: stiffeners do not buckle:
section class 1 for $\alpha = 1.000$, $c/t = 3.73$ (outstand flange) $< 9.00 \cdot \varepsilon = 9.00 \le 3$ ok

minimum demands of the moment of inertia of stiffeners

length of buckling field (distance of stiffeners) $a = 113.7$ mm
web height between the flanges $h_{wc} = d_c+2 \cdot s_c = 107.4$ mm
moment of inertia of stiffeners $I_{st} = (2 \cdot b_{st}+t_{wc})^3 \cdot t_{st} / 12 = 17.48$ cm⁴
minimum moment of inertia for $a/h_{wc} = 1.06 < 2^{1/2}$: $I_{st,min} = 1.5 \cdot (h_{wc} \cdot t_{wc})^3 / a^2 = 1.22$ cm⁴ $< I_{st}$ ok

requirement concerning stiffeners to aviod lateral torsional buckling

torsional moment of inertia of stiffeners $I_T \approx b_{st} \cdot t_{st}^3 / 3 = 0.51$ cm⁴
polar moment of inertia of stiffeners $I_p = b_{st} \cdot t_{st}^3 / 12 + t_{st} \cdot b_{st}^3 / 12 = 1.89$ cm⁴
$I_T / I_p \approx 0.269 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok

resistance of the stiffened webs with transverse compression

area of stiffeners incl. web $A_{st} = (2 \cdot b_{st}+t_{wc}) \cdot t_{st} = 5.12$ cm²
radius of inertia of stiffeners $i = (I_{st} / A_{st})^{1/2} = 18.5$ mm
buckling length of stiffeners $L_{cr} = h_{wc} = 107.4$ mm
slenderness $\lambda = L_{cr} / (i \cdot \lambda_1) = 0.062$ with $\lambda_1 = \pi \cdot (E_{st}/f_{y,st})^{1/2} = 93.9$
$\lambda \le 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd} = \chi \cdot A_{st} \cdot f_{y,st} / \gamma_{M1} = 109.4$ kN

maximum resistance

$F_{c,w,Rd} = 109.4$ kN (resistance with transverse stiffeners)

resistance of the upper beam flange:

resistance without transverse stiffeners

effective width

effective width of column web in transverse compression $b_{eff,c} = t_{f,b} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{f,c}+s_c) + s_p = 98.5$ mm,
$a_p = 4.0$ mm, $s_c = 7.0$ mm, $s_p = 14.3$ mm

longitudinal compressive stress in web

reduction factor $k_w = 1.0$ ($\sigma_{com,Ed} = 0$)

plate buckling

plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.679$, $E = 210000$ N/mm²
reduction factor for web buckling $\rho = 1$

shear area

shear area $A_v = 6.31$ cm²

interaction with shear stress

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = \omega_1 = 0.787$
with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w/A_v)^2]^{1/2} = 0.787$

resistance of web in transverse compression

$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 80.14$ kN, $f_{y,w} = 235.0$ N/mm²

$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 72.86$ kN (decisive)

reinforcement of web with transverse stiffeners:

assumption: stiffeners do not buckle:

section class 1 for $\alpha = 1.000$, $c/t = 3.73$ (outstand flange) $< 9.00 \cdot \varepsilon = 9.00 \leq 3$ ok

minimum demands of the moment of inertia of stiffeners

length of buckling field (distance of stiffeners) $a = 113.7$ mm

web height between the flanges $h_{wc} = d_c + 2 \cdot s_c = 107.4$ mm

moment of inertia of stiffeners $I_{st} = (2 \cdot b_{st} + t_{wc})^3 \cdot t_{st} / 12 = 17.48$ cm^4

minimum moment of inertia for $a/h_{wc} = 1.06 < 2^{1/2}$: $I_{st,min} = 1.5 \cdot (h_{wc} \cdot t_{wc})^3 / a^2 = 1.22$ cm$^4 < I_{st}$ ok

requirement concerning stiffeners to aviod lateral torsional buckling

torsional moment of inertia of stiffeners $I_T \approx b_{st} \cdot t_{st}^3 / 3 = 0.51$ cm^4

polar moment of inertia of stiffeners $I_p = b_{st} \cdot t_{st}^3 / 12 + t_{st} \cdot b_{st}^3 / 12 = 1.89$ cm^4

$I_T / I_p \approx 0.269 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok

resistance of the stiffened webs with transverse compression

area of stiffeners incl. web $A_{st} = (2 \cdot b_{st} + t_{wc}) \cdot t_{st} = 5.12$ cm^2

radius of inertia of stiffeners $i = (I_{st} / A_{st})^{1/2} = 18.5$ mm

buckling length of stiffeners $L_{cr} = h_{wc} = 107.4$ mm

slenderness $\lambda = L_{cr} / (i \cdot \lambda_1) = 0.062$ with $\lambda_1 = \pi \cdot (E_{st}/f_{y,st})^{1/2} = 93.9$

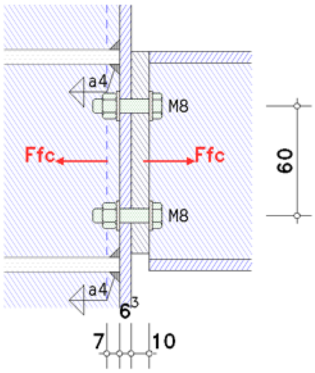
$\lambda \leq 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)

design value of resistance of flexural buckling $F_{c,w,Rd} = \chi \cdot A_{st} \cdot f_{y,st} / \gamma_{M1} = 109.4$ kN

maximum resistance

$F_{c,w,Rd} = 109.4$ kN (resistance with transverse stiffeners)

2.1.2.3.3. bc 4: column flange in bending



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

equivalent T-stub flange (each individual bolt-row):

here: number of bolt-rows $n_b = 1$

row 1

distance centre-line of the bolt to the stiffener $m_2 = 18.3$ mm $> 2 \cdot m = 16.4$ mm $\Rightarrow m_2 = 0$

distance centre-line of the bolt to the edge of flange $e = 16.0$ mm

distance centre-line of the bolt to the stub web $m = 8.2$ mm

effective length of the T-stub flange (column flange)

(other) end bolt-row

$l_{eff,cp,a} = 2 \cdot \pi \cdot m = 51.5$ mm ($e_1 = \infty$)

$l_{eff,nc,a} = 4 \cdot m + 1.25 \cdot e = 52.8$ mm ($e_1 = \infty$)

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 51.5$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 52.8$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 10.3$ mm, $e_{min} = 14.0$ mm, $m = 8.2$ mm

resisting plastic moments:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.12$ kNm, $t_f = 6.3$ mm, $f_y = 235.0$ N/mm^2, $\gamma_{M0} = 1.00$

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.12$ kNm

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 21.08$ kN, $k_2 = 0.90$, $f_{ub} = 800.0$ N/mm^2

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 42.16$ kN, $n_b = 1$

prying forces always appear at preloaded bolts !

calculation with the alternative method

decisive diameter of the bolt $d_w = d_p = 16.00$ mm $\Rightarrow e_w = d_w/4 = 4.0$ mm

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 94.28$ kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 36.77$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 42.16$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 36.77$ kN

row 2

distance centre-line of the bolt to the stiffener $m_2 = 18.3$ mm $> 2 \cdot m = 16.4$ mm $\Rightarrow m_2 = 0$

distance centre-line of the bolt to the edge of flange $e = 16.0$ mm

distance centre-line of the bolt to the stub web $m = 8.2$ mm

effective length of the T-stub flange (column flange)

(other) end bolt-row

$l_{eff,cp,a} = 2\cdot\pi\cdot m = 51.5$ mm ($e_1 = \infty$)

$l_{eff,nc,a} = 4\cdot m+1.25\cdot e = 52.8$ mm ($e_1 = \infty$)

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 51.5$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 52.8$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25\cdot m) = 10.3$ mm, $e_{min} = 14.0$ mm, $m = 8.2$ mm

resisting plastic moments:

in mode 1: $M_{pl,1,Rd} = (0.25\cdot\Sigma l_{eff,1}\cdot t_f^2\cdot f_y) / \gamma_{M0} = 0.12$ kNm, $t_f = 6.3$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$

in mode 2: $M_{pl,2,Rd} = (0.25\cdot\Sigma l_{eff,2}\cdot t_f^2\cdot f_y) / \gamma_{M0} = 0.12$ kNm

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2\cdot f_{ub}\cdot A_s) / \gamma_{M2} = 21.08$ kN, $k_2 = 0.90$, $f_{ub} = 800.0$ N/mm²

in mode 3: $\Sigma F_{t,Rd} = 2\cdot n_b\cdot F_{t,Rd} = 42.16$ kN, $n_b = 1$

prying forces always appear at preloaded bolts !

calculation with the alternative method

decisive diameter of the bolt $d_w = d_p = 16.00$ mm $\Rightarrow$ $e_w = d_w/4 = 4.0$ mm

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8\cdot n-2\cdot e_w)\cdot M_{pl,1,Rd}) / (2\cdot m\cdot n-e_w\cdot(m+n)) = 94.28$ kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2\cdot M_{pl,2,Rd} + n\cdot\Sigma F_{t,Rd}) / (m+n) = 36.77$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 42.16$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 36.77$ kN

resistances and effective lengths of column flange in bending (per bolt-row)

$F_{t,fc,Rd,1} = 36.77$ kN, $l_{eff,1} = 51.5$ mm

$F_{t,fc,Rd,2} = 36.77$ kN, $l_{eff,2} = 51.5$ mm

equivalent T-stub flange (group of bolts 1):

here: number of bolt-rows $n_b = 2$ (between stiffeners)

row 1

distance centre-line of the bolt to the edge of flange $e = 16.0$ mm

distance centre-line of the bolt to the stub web $m = 8.2$ mm

distance between bolt-rows $p = 60.0$ mm

distance centre-line of the bolt to the stiffener $m_2 = 18.3$ mm $> 2\cdot m = 16.4$ mm $\Rightarrow$ $m_2 = 0$

(other) end bolt-row

$l_{eff,cp,a} = \pi\cdot m+p = 85.8$ mm ($e_1 = \infty$)

$l_{eff,nc,a} = 2\cdot m+0.625\cdot e+0.5\cdot p = 56.4$ mm ($e_1 = \infty$)

row 2

distance centre-line of the bolt to the edge of flange $e = 16.0$ mm

distance centre-line of the bolt to the stub web $m = 8.2$ mm

distance between bolt-rows $p = 60.0$ mm

distance centre-line of the bolt to the stiffener $m_2 = 18.3$ mm $> 2\cdot m = 16.4$ mm $\Rightarrow$ $m_2 = 0$

(other) end bolt-row

$l_{eff,cp,a} = \pi\cdot m+p = 85.8$ mm ($e_1 = \infty$)

$l_{eff,nc,a} = 2\cdot m+0.625\cdot e+0.5\cdot p = 56.4$ mm ($e_1 = \infty$)

effective length of the T-stub flange (column flange)

in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 112.8$ mm, $\Sigma l_{eff,cp} = 171.5$ mm

in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 112.8$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25\cdot m) = 10.3$ mm, $e_{min} = 14.0$ mm, $m = 8.2$ mm

resisting plastic moments:

in mode 1+2: $M_{pl,Rd} = (0.25\cdot\Sigma l_{eff}\cdot t_f^2\cdot f_y) / \gamma_{M0} = 0.26$ kNm, $t_f = 6.3$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2\cdot f_{ub}\cdot A_s) / \gamma_{M2} = 21.08$ kN, $k_2 = 0.90$, $f_{ub} = 800.0$ N/mm²

in mode 3: $\Sigma F_{t,Rd} = 2\cdot n_b\cdot F_{t,Rd} = 84.33$ kN, $n_b = 2$

prying forces always appear at preloaded bolts !

calculation with the alternative method

decisive diameter of the bolt $d_w = d_p = 16.00$ mm $\Rightarrow$ $e_w = d_w/4 = 4.0$ mm

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8\cdot n-2\cdot e_w)\cdot M_{pl,1,Rd}) / (2\cdot m\cdot n-e_w\cdot(m+n)) = 206.40$ kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2\cdot M_{pl,2,Rd} + n\cdot\Sigma F_{t,Rd}) / (m+n) = 75.36$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 84.33$ kN

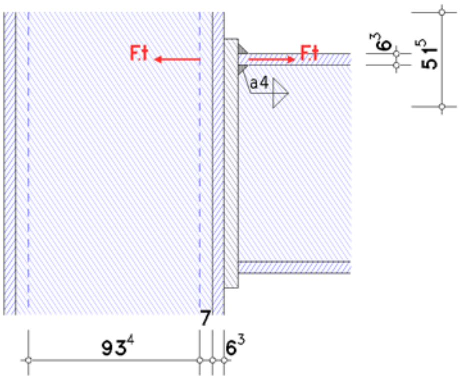
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 75.36$ kN

resistances and effective lengths of column flange in bending (per bolt group):

$F_{ep,Rd,1-2} = 75.36$ kN, $\Sigma l_{eff} = 112.8$ mm, 2 rows

2.1.2.3.4. bc 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 8.00$ kNm ($M_{j2} = 0$)



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

each individual bolt-row:

row 1

effective width

effective width of column web in transverse tension $b_{eff,t} = 51.5$ mm (l_{eff} from bc 4)

shear area

shear area $A_v = 6.31$ cm²

interaction with shear stress

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = \omega_1 = 0.925$

with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w / A_v)^2]^{1/2} = 0.925$

resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 49.3$ kN, $f_{y,wc} = 235.0$ N/mm²

row 2

effective width

effective width of column web in transverse tension $b_{eff,t} = 51.5$ mm (l_{eff} from bc 4)

shear area

shear area $A_v = 6.31$ cm²

interaction with shear stress

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = \omega_1 = 0.925$

with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w / A_v)^2]^{1/2} = 0.925$

resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 49.3$ kN, $f_{y,wc} = 235.0$ N/mm²

group of bolt-rows, group 1:

effective width

effective width of column web in transverse tension $b_{eff,t} = 112.8$ mm (l_{eff} from bc 4)

shear area

shear area $A_v = 6.31$ cm²

interaction with shear stress

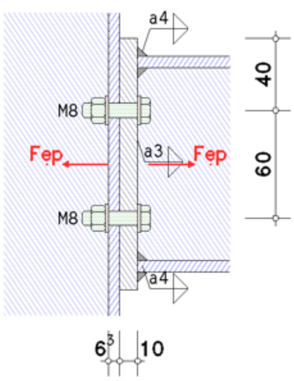
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = \omega_1 = 0.744$

with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w / A_v)^2]^{1/2} = 0.744$

resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 86.8$ kN, $f_{y,wc} = 235.0$ N/mm²

2.1.2.3.5. bc 5: end-plate in bending



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

part of end-plate between beam flanges

equivalent T-stub flange (each individual bolt-row):

here: number of bolt-rows $n_b = 1$

row 1

distance centre-line of the bolt to the stiffener $m_2 = 19.2$ mm

distance centre-line of the bolt to the edge of flange $e = 14.0$ mm

distance centre-line of the bolt to the stub web $m = 10.4$ mm

effective length of the T-stub flange (end-plate)

inner bolt-row outside tension flange of beam coefficient for stiffened column flanges and end-plates:

input values $\lambda_1 = m / (m+e) = 0.426$, $\lambda_2 = m_2 / (m+e) = 0.786 \Rightarrow \alpha = 5.74$ (calculated)

$l_{eff,cp,si} = 2 \cdot \pi \cdot m = 65.4$ mm

$l_{eff,nc,si} = \alpha \cdot m = 59.7$ mm

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 59.7$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 59.7$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 13.0$ mm, $e_{min} = 14.0$ mm, $m = 10.4$ mm

resisting plastic moments:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.35$ kNm, $t_f = 10.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 21.08$ kN, $k_2 = 0.90$, $f_{ub} = 800.0$ N/mm²

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 42.16$ kN, $n_b = 1$

prying forces always appear at preloaded bolts !

calculation with the alternative method

decisive diameter of the bolt $d_w = d_p = 16.00$ mm $\Rightarrow e_w = d_w/4 = 4.0$ mm

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 190.26$ kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 53.38$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 42.16$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 42.16$ kN

shear strength: $f_{vw,d} = (f_u/3^{1/2}) / (\beta_w \cdot \gamma_{M2}) = 207.8$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$

tension resistance of welds: $F_{T,w,Rd} = 2 \cdot f_{vw,d} \cdot a \cdot l_{eff} = 74.44$ kN (≥ 42.16 kN, not decisive)

row 2

distance centre-line of the bolt to the stiffener $m_2 = 19.2$ mm

distance centre-line of the bolt to the edge of flange $e = 14.0$ mm

distance centre-line of the bolt to the stub web $m = 10.4$ mm

effective length of the T-stub flange (end-plate)

inner bolt-row outside tension flange of beam

coefficient for stiffened column flanges and end-plates:

input values $\lambda_1 = m / (m+e) = 0.426$, $\lambda_2 = m_2 / (m+e) = 0.786 \Rightarrow \alpha = 5.74$ (calculated)

$l_{eff,cp,si} = 2 \cdot \pi \cdot m = 65.4$ mm

$l_{eff,nc,si} = \alpha \cdot m = 59.7$ mm

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 59.7$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 59.7$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 13.0$ mm, $e_{min} = 14.0$ mm, $m = 10.4$ mm

resisting plastic moments:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.35$ kNm, $t_f = 10.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 21.08$ kN, $k_2 = 0.90$, $f_{ub} = 800.0$ N/mm²

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 42.16$ kN, $n_b = 1$

prying forces always appear at preloaded bolts !

calculation with the alternative method

decisive diameter of the bolt $d_w = d_p = 16.00$ mm $\Rightarrow e_w = d_w/4 = 4.0$ mm

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 190.26$ kN

mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 53.38$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 42.16$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 42.16$ kN

shear strength: $f_{vw,d} = (f_u/3^{1/2}) / (\beta_w \cdot \gamma_{M2}) = 207.8$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$

tension resistance of welds: $F_{T,w,Rd} = 2 \cdot f_{vw,d} \cdot a \cdot l_{eff} = 74.44$ kN (≥ 42.16 kN, not decisive)

resistances and effective lengths of end-plate in bending (per bolt-row):

$F_{ep,Rd,1} = 42.16$ kN, $l_{eff,1} = 59.7$ mm

$F_{ep,Rd,2} = 42.16$ kN, $l_{eff,2} = 59.7$ mm

equivalent T-stub flange (group of bolts 1):

here: number of bolt-rows $n_b = 2$

row 1

distance centre-line of the bolt to the stiffener $m_2 = 19.2$ mm

distance centre-line of the bolt to the edge of flange $e = 14.0$ mm

distance centre-line of the bolt to the stub web $m = 10.4$ mm

distance between bolt-rows $p = 60.0$ mm

inner bolt-row outside tension flange of beam

coefficient for stiffened column flanges and end-plates:

input values $\lambda_1 = m / (m+e) = 0.426$, $\lambda_2 = m_2 / (m+e) = 0.786 \Rightarrow \alpha = 5.74$ (calculated)

$l_{eff,cp,si} = \pi \cdot m + p = 92.7$ mm

$l_{eff,nc,si} = 0.5 \cdot p + \alpha \cdot m - (2 \cdot m + 0.625 \cdot e) = 60.1$ mm

row 2

distance centre-line of the bolt to the stiffener m_2 = 19.2 mm
distance centre-line of the bolt to the edge of flange e = 14.0 mm
distance centre-line of the bolt to the stub web m = 10.4 mm
distance between bolt-rows p = 60.0 mm
inner bolt-row outside tension flange of beam
coefficient for stiffened column flanges and end-plates:
input values $\lambda_1 = m / (m+e) = 0.426$, $\lambda_2 = m_2 / (m+e) = 0.786$ $\Rightarrow$ $\alpha = 5.74$ (calculated)
$l_{eff,cp,si} = \pi \cdot m + p = 92.7$ mm
$l_{eff,nc,si} = 0.5 \cdot p + \alpha \cdot m - (2 \cdot m + 0.625 \cdot e) = 60.1$ mm

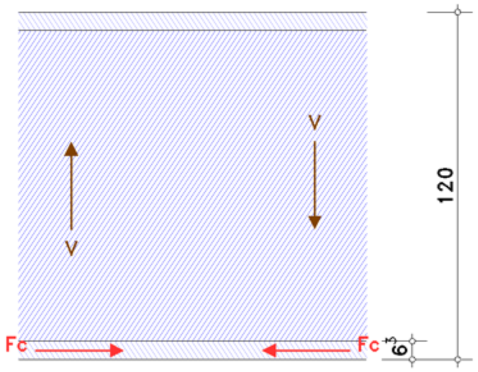
effective length of the T-stub flange (end-plate)
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 120.3$ mm, $\Sigma l_{eff,cp} = 185.4$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 120.3$ mm

tension resistance of the T-stub flange
$n = \min(e_{min}, 1.25 \cdot m) = 13.0$ mm, $e_{min} = 14.0$ mm, m = 10.4 mm
resisting plastic moments:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.71$ kNm, $t_f = 10.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$
design value of tension resistance:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 21.08$ kN, $k_2 = 0.90$, $f_{ub} = 800.0$ N/mm²
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 84.33$ kN, $n_b = 2$
prying forces always appear at preloaded bolts !
calculation with the alternative method
decisive diameter of the bolt $d_w = d_p = 16.00$ mm $\Rightarrow$ $e_w = d_w/4 = 4.0$ mm
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 383.31$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 107.20$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 84.33$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 84.33$ kN
shear strength: $f_{vw,d} = (f_u/3^{1/2}) / (\beta_w \cdot \gamma_{M2}) = 207.8$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$
tension resistance of welds: $F_{T,w,Rd} = 2 \cdot f_{vw,d} \cdot a \cdot l_{eff} = 149.97$ kN ($\geq$ 84.33 kN, not decisive)

resistances and effective lengths of end-plate in bending (per bolt group):
$F_{ep,Rd,1\text{-}2} = 84.33$ kN, $\Sigma l_{eff} = 120.3$ mm, 2 rows

2.1.2.3.6. bc 7: beam flange and web in compression

section class of the beam ($\varepsilon = 1.00$)
coefficient α is determined elastically.
flange below
section class 1 for $\alpha = 1.000$, c/t = 3.62 (outstand flange) < $9.00 \cdot \varepsilon = 9.00$
web
section class 1 for c/t = 21.23 (internal compression parts, bending) < $72.00 \cdot \varepsilon = 72.00$
total: section class 1
taking into account the moment-shear force-interaction $V_{Ed} = 40.0$ kN
plastic section modulus $W_{pl} = 60.718$ cm³



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

stress due to bending with shear force
$V_{pl,Rd} = A_v \cdot (f_y/3^{1/2}) / \gamma_{M0} = 85.6$ kN, $A_v = 6.31$ cm²
$V_{Ed} = 40.0$ kN $\leq$ 42.8 kN $= V_{pl,Rd}/2$ $\Rightarrow$ no effect on the moment resistance !

stress for section class 1
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 14.27$ kNm, $W_{pl} = 60.72$ cm³

resistance of a flange (and web) with compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 125.49$ kN, $(h - t_f) = 113.7$ mm

resistance of the upper beam flange:
plastic section modulus $W_{pl} = 60.718$ cm³

stress due to bending with shear force
$V_{pl,Rd} = A_v \cdot (f_y/3^{1/2}) / \gamma_{M0} = 85.6$ kN, $A_v = 6.31$ cm²
$V_{Ed} = 40.0$ kN $\leq$ 42.8 kN $= V_{pl,Rd}/2$ $\Rightarrow$ no effect on the moment resistance !

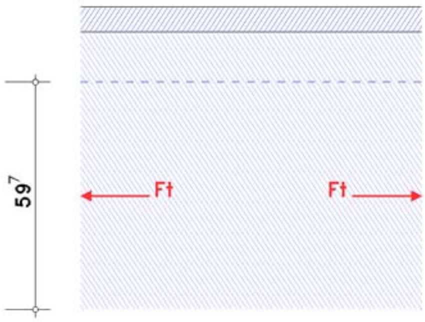
stress for section class 1

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 14.27$ kNm, $W_{pl} = 60.72$ cm^3

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 125.49$ kN, $(h - t_f) = 113.7$ mm

2.1.2.3.7. bc 8: beam web in tension



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

each individual bolt-row:

row 1

effective width

effective width of the beam web in tension $b_{eff,t,wb} = 59.7$ mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 61.7$ kN, $f_{y,wb} = 235.0$ N/mm^2

row 2

effective width

effective width of the beam web in tension $b_{eff,t,wb} = 59.7$ mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 61.7$ kN, $f_{y,wb} = 235.0$ N/mm^2

group of bolt-rows, group 1:

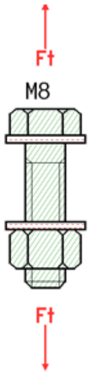
effective width

effective width of the beam web in tension $b_{eff,t,wb} = 120.3$ mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 124.3$ kN, $f_{y,wb} = 235.0$ N/mm^2

2.1.2.3.8. bc 10: bolts in tension

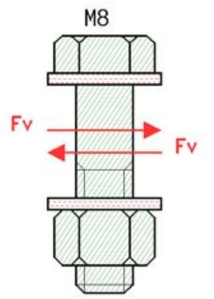


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

bolt category D:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 21.08$ kN, $k_2 = 0.90$, $f_{ub} = 800.0$ N/mm^2
p. sh. load capicity: $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 46.51$ kN, $d_m = 13.6$ mm, $t_p = 6.3$ mm, $f_u = 360.0$ N/mm^2
tension-/punching shear load capicity for 2 bolts: $\Sigma F_{tp,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 42.16$ kN

2.1.2.3.9. bc 11: bolts in shear

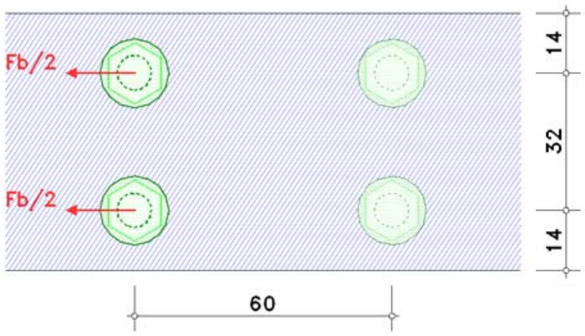
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



bolt category A:
shear plane passes through the unthreaded portion of the bolt: $\alpha_v = 0.6$, $A = 0.50\ cm^2$
shear resistance per shear plane: $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2} = 19.30$ kN, $f_{ub} = 800.0\ N/mm^2$
shear resistance of 2 bolts (1-shear): $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd} = 38.60$ kN

2.1.2.3.10. bc 12: plate with bearing resistance

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



row 1

end-plate (for $V_b \geq 0$):

bolt 1:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 3.28$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.66$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 57.60$ kN, $f_u = 360.0\ N/mm^2$, $t = 10.0$ mm, $d = 8.0$ mm

bolt 2:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 3.28$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.66$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 57.60$ kN, $f_u = 360.0\ N/mm^2$, $t = 10.0$ mm, $d = 8.0$ mm
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 115.20$ kN

column flange (for $V_b \geq 0$):

bolt 1:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 3.28$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.66$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 36.29$ kN, $f_u = 360.0\ N/mm^2$, $t = 6.3$ mm, $d = 8.0$ mm

bolt 2:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 3.28$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.66$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 36.29$ kN, $f_u = 360.0\ N/mm^2$, $t = 6.3$ mm, $d = 8.0$ mm
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 72.58$ kN

row 2

end-plate (for $V_b \geq 0$):

bolt 1:
in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 1.97$ (inner bolt)
$\Rightarrow \alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.22$ or 1.0)
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 3.28$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 2.66$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 57.60$ kN, $f_u = 360.0\ N/mm^2$, $t = 10.0$ mm, $d = 8.0$ mm

bolt 2:
in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0)-1/4 = 1.97$ (inner bolt)
$\Rightarrow$ $\alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.22$ or 1.0)
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0-1.7 = 3.28$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0-1.7,\ 1.4 \cdot p_2/d_0-1.7) = 2.66$ (end bolt)
$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 57.60$ kN, $f_u = 360.0$ N/mm², $t = 10.0$ mm, $d = 8.0$ mm
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 115.20$ kN

column flange (for $V_b \geq 0$):
bolt 1:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0-1.7 = 3.28$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0-1.7,\ 1.4 \cdot p_2/d_0-1.7) = 2.66$ (end bolt)
$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 36.29$ kN, $f_u = 360.0$ N/mm², $t = 6.3$ mm, $d = 8.0$ mm
bolt 2:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0-1.7 = 3.28$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0-1.7,\ 1.4 \cdot p_2/d_0-1.7) = 2.66$ (end bolt)
$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 36.29$ kN, $f_u = 360.0$ N/mm², $t = 6.3$ mm, $d = 8.0$ mm
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 72.58$ kN

bearing resistance (2 rows)
$\Sigma F_{b,Rd,1} = 72.58$ kN
$\Sigma F_{b,Rd,2} = 72.58$ kN

2.1.2.4. connection capacity

transformation parameter: $\beta_j = 1.00$

2.1.2.4.1. moment resistance

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

resistances acc. to EC 3-1-8, 6.2.7.2(6) for bolt-rows considered individually
decisive basic components: 3, 4, 5, 8
row 1: $F_{tr,Rd} = 36.8$ kN
row 2: $F_{tr,Rd} = 36.8$ kN

deductions acc. to EC 3-1-8, 6.2.7.2(8) for bolt-rows as part of a group (column)
decisive basic components: 3, 4
group 1
row 1: $\Sigma F_{tr,Rd} = 0.0$ kN
bc 3: $\Delta F_{tr,Rd} = F_{t,wc,Rd} - \Sigma F_{tr,Rd} = 86.8$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
bc 4: $\Delta F_{tr,Rd} = F_{t,fc,Rd} - \Sigma F_{tr,Rd} = 75.4$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
row 2: $\Sigma F_{tr,Rd} = 36.8$ kN (due to row 1)
bc 3: $\Delta F_{tr,Rd} = F_{t,wc,Rd} - \Sigma F_{tr,Rd} = 50.0$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
bc 4: $\Delta F_{tr,Rd} = F_{t,fc,Rd} - \Sigma F_{tr,Rd} = 38.6$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN

deductions acc. to EC 3-1-8, 6.2.7.2(8) for bolt-rows as part of a group (end-plate)
decisive basic components: 5, 8
group 1
row 1: $\Sigma F_{tr,Rd} = 0.0$ kN
bc 5: $\Delta F_{tr,Rd} = F_{t,ep,Rd} - \Sigma F_{tr,Rd} = 84.3$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
bc 8: $\Delta F_{tr,Rd} = F_{t,wb,Rd} - \Sigma F_{tr,Rd} = 124.3$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
row 2: $\Sigma F_{tr,Rd} = 36.8$ kN (due to row 1)
bc 5: $\Delta F_{tr,Rd} = F_{t,ep,Rd} - \Sigma F_{tr,Rd} = 47.6$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
bc 8: $\Delta F_{tr,Rd} = F_{t,wb,Rd} - \Sigma F_{tr,Rd} = 87.6$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN

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

deductions acc. to EC 3-1-8, 6.2.7.2(7)
decisive basic components: 1, 2, 7
row 1: $\Sigma F_{tr,Rd} = 0.0$ kN
bc 1: $\Delta F_{tr,Rd} = V_{wp,Rd}/\beta_j - \Sigma F_{tr,Rd} = 82.2$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
bc 2: $\Delta F_{tr,Rd} = F_{c,w,Rd} - \Sigma F_{tr,Rd} = 109.4$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
bc 7: $\Delta F_{tr,Rd} = F_{c,f,Rd} - \Sigma F_{tr,Rd} = 125.5$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
row 2: $\Sigma F_{tr,Rd} = 36.8$ kN (row 1)
bc 1: $\Delta F_{tr,Rd} = V_{wp,Rd}/\beta_j - \Sigma F_{tr,Rd} = 45.5$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN
bc 2: $\Delta F_{tr,Rd} = F_{c,w,Rd} - \Sigma F_{tr,Rd} = 72.6$ kN — $F_{tr,Rd} = 36.8$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 36.8$ kN

bc 7: $\Delta F_{tr,Rd} = F_{c,f,Rd} - \Sigma F_{tr,Rd} = 88.7$ kN $\quad F_{tr,Rd} = 36.8$ kN $< \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 36.8$ kN

check acc. to EC 3-1-8, 6.2.7.2(9)
decisive basic component: 10
row 1: $F_{tx,Rd} = 36.8$ kN, $h_x = 86.8$ mm $\Rightarrow F_{tx,Rd} \leq \lim F_{tx,Rd} = 40.1$ kN, no deduction

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

moment resistance regarding the centre of compression
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 4.2$ 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

2.1.2.4.2. shear/bearing resistance

resistance per bolt-row
decisive basic components: 11, 12
row 1: $F_{vr,Rd} = 38.6$ kN
row 2: $F_{vr,Rd} = 38.6$ kN

deductions depending on tension force (at full utilization of moment resistance)
decisive basic component: 10
row 1: $F_{vr,Rd} = f_{vt} \cdot 38.6$ kN $= 14.6$ kN with $f_{vt} = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd}) = 0.377$
row 2: $F_{vr,Rd} = f_{vt} \cdot 38.6$ kN $= 14.6$ kN with $f_{vt} = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd}) = 0.377$

resistance per bolt-row (finally)
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

2.1.2.4.3. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd} = \tau_{Rd} \cdot t_{ep} \cdot l_{eff} = 126.72$ kN, $\tau_{Rd} = 135.7$ N/mm², $t_{ep} = 10.0$ mm, $l_{eff} = d_w = 93.4$ mm
shear strength: $f_{vw,d} = (f_u/3^{1/2}) / (\beta_w \cdot \gamma_{M2}) = 207.8$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$
welds: $F_{w,Rd} = 2 \cdot a \cdot l_{eff} \cdot f_{vw,d} = 116.48$ kN, $a = 3.0$ mm, $l_{eff} = d_w = 93.4$ mm
shear resistance of end plate: $V_{ep,Rd} = F_{w,Rd} = 116.48$ kN
shear resistance of column web
decisive basic component: 1
$V_{wp,Rd} = 82.2$ kN

2.1.2.4.4. 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

2.1.2.5. verifications

internal lever arm for a bolted end-plate joint with 2 tension-bolt-rows:
equivalent internal lever arm:
1: $k_3 = 1.70$ mm, $k_4 = 21.03$ mm, $k_5 = 47.68$ mm, $k_{10} = 2.24$ mm $\Rightarrow k_{eff,1} = 1 / \Sigma(1/k_{i,1}) = 0.907$ mm
2: $k_3 = 1.70$ mm, $k_4 = 21.03$ mm, $k_5 = 47.68$ mm, $k_{10} = 2.24$ mm $\Rightarrow k_{eff,2} = 1 / \Sigma(1/k_{i,2}) = 0.907$ mm
$z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r) = 7493.5 / 103.1 = 72.7$ mm

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

internal moment: $M_{Ed} = M_d = 5.60$ kNm
perpend. to connection plane
shear force: $V_{Ed} = |V_d| = 40.00$ kN
parallel to connection plane
shear force: $V_{c,w,Ed} = M_d/z - (V_{c1}-V_{c2})/2 = 77.05$ kN, $z = 72.7$ mm
in column web
shear force: $V_{b,w,Ed} = 40.00$ kN

in end-plate

moment resistance

$M_{Ed}/M_{j,Rd} = 1.339 > 1$ not ok !!

shear/bearing resistance at 100% utilization of moment resistance

$V_{Ed}/V_{j,Rd} = 1.374 > 1$ not ok !!

shear resistance of column web

$V_{c,w,Ed}/V_{wp,Rd} = 0.937 < 1$ ok

shear resistance of end plate

$V_{b,w,Ed}/V_{ep,Rd} = 0.343 < 1$ ok

2.1.2.5.2. 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

structural limits for welds:

weld 1: plate thickness $t_1 = 10.0$ mm > 4 mm ok
plate thickness $t_2 = 6.3$ mm > 4 mm ok
NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0$ mm > $a_{min} = t_{max}^{1/2} - 0.5 = 2.66$ mm ok
weld thickness $a = 4.0$ mm < $a_{max} = 0.7 \cdot t_{min} = 4.4$ mm (welding technology, s. DIN 18800) ok
weld thickness $a = 4.0$ mm > $a_{min} = 3$ mm ok
effective weld length $l_{eff} = 60.0$ mm > 30 mm ok
effective weld length $l_{eff} = 60.0$ mm > $6 \cdot a = 24.0$ mm ok

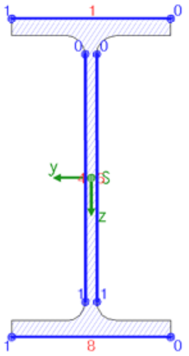
weld 2: plate thickness $t_1 = 10.0$ mm > 4 mm ok
plate thickness $t_2 = 6.3$ mm > 4 mm ok
NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0$ mm > $a_{min} = t_{max}^{1/2} - 0.5 = 2.66$ mm ok
weld thickness $a = 4.0$ mm < $a_{max} = 0.7 \cdot t_{min} = 4.4$ mm (welding technology, s. DIN 18800) ok
weld thickness $a = 4.0$ mm > $a_{min} = 3$ mm ok
effective weld length $l_{eff} = 20.8$ mm < 30 mm $\Rightarrow$ has no effect !!
effective weld length $l_{eff} = 20.8$ mm < $6 \cdot a = 24.0$ mm $\Rightarrow$ has no effect !!

weld 4: plate thickness $t_1 = 10.0$ mm > 4 mm ok
plate thickness $t_2 = 4.4$ mm > 4 mm ok
NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 3.0$ mm > $a_{min} = t_{max}^{1/2} - 0.5 = 2.66$ mm ok
weld thickness $a = 3.0$ mm < $a_{max} = 0.7 \cdot t_{min} = 3.1$ mm (welding technology, s. DIN 18800) ok
weld thickness $a = 3.0$ mm $\geq a_{min} = 3$ mm ok
effective weld length $l_{eff} = 93.4$ mm > 30 mm ok
effective weld length $l_{eff} = 93.4$ mm > $6 \cdot a = 18.0$ mm ok

weld 8: plate thickness $t_1 = 10.0$ mm > 4 mm ok
plate thickness $t_2 = 6.3$ mm > 4 mm ok
NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0$ mm > $a_{min} = t_{max}^{1/2} - 0.5 = 2.66$ mm ok
weld thickness $a = 4.0$ mm < $a_{max} = 0.7 \cdot t_{min} = 4.4$ mm (welding technology, s. DIN 18800) ok
weld thickness $a = 4.0$ mm > $a_{min} = 3$ mm ok
effective weld length $l_{eff} = 60.0$ mm > 30 mm ok
effective weld length $l_{eff} = 60.0$ mm > $6 \cdot a = 24.0$ mm ok

weld 6: plate thickness $t_1 = 10.0$ mm > 4 mm ok
plate thickness $t_2 = 6.3$ mm > 4 mm ok
NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0$ mm > $a_{min} = t_{max}^{1/2} - 0.5 = 2.66$ mm ok
weld thickness $a = 4.0$ mm < $a_{max} = 0.7 \cdot t_{min} = 4.4$ mm (welding technology, s. DIN 18800) ok
weld thickness $a = 4.0$ mm > $a_{min} = 3$ mm ok
effective weld length $l_{eff} = 20.8$ mm < 30 mm $\Rightarrow$ has no effect !!
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

distribution of internal forces and moments:

weld 1: $N_w = 37.76$ kN
weld 4: $M_{y,w} = -0.53$ kNm
weld 5: see weld 4
weld 8: $N_w = -37.76$ kN
from conventionel distribution of shear force: $V_{z,w} = 40.00$ kN

weld edges:

weld 1, pt. 0: y' = -30.0 mm z' = 0.0 mm
weld 4, pt. 0: y' = 2.2 mm z' = 13.3 mm
pt. 1: y' = 2.2 mm z' = 106.7 mm
weld 8, pt. 0: y' = -30.0 mm z' = 120.0 mm

stresses in weld edges:

weld 1, pt. 0: $\sigma_{w,x} = 157.35$ N/mm²
weld 4, pt. 0: $\sigma_{w,x} = 122.47$ N/mm² $\tau_{w,z} = 71.38$ N/mm²
pt. 1: $\sigma_{w,x} = -30.62$ N/mm² $\tau_{w,z} = 71.38$ N/mm² (compressive contact)
weld 5, pt. 0: see weld 4
pt. 1: see weld 4
weld 8, pt. 0: $\sigma_{w,x} = -39.34$ N/mm² (compressive contact)
maximum/minimum stresses $\sigma_{w,x,max} = 157.3$ N/mm² $\sigma_{w,x,min} = -39.3$ N/mm² $\tau_{w,z,max} = 71.4$ N/mm²

verifications in weld edges:

75% decrease of stress by pressure contact

verification of weld 1, pt. 0:

stresses on the design area of the weld ($\alpha = 45°$):
$\sigma_{w,Ed} = \sigma_{w,x} = 157.3$ N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a = 629.39$ kN/m
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 831.38$ kN/m, $f_{vw,d} = 207.85$ N/mm², a = 4.0 mm
$F_{w,Ed} = 629.39$ kN/m < $F_{w,Rd} = 831.38$ kN/m ⇒ U = 0.757 < 1 ok

verification of weld 4, pt. 0:

stresses on the design area of the weld ($\alpha = 45°$):
$\sigma_{w,Ed} = (\sigma_{w,x}^2 + \tau_{w,z}^2)^{1/2} = 141.8$ N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a = 425.25$ kN/m
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 623.54$ kN/m, $f_{vw,d} = 207.85$ N/mm², a = 3.0 mm
$F_{w,Ed} = 425.25$ kN/m < $F_{w,Rd} = 623.54$ kN/m ⇒ U = 0.682 < 1 ok

verification of weld 4, pt. 1:

stresses on the design area of the weld ($\alpha = 45°$):
$\sigma_{w,Ed} = (\sigma_{w,x}^2 + \tau_{w,z}^2)^{1/2} = 77.7$ N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a = 233.00$ kN/m
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 623.54$ kN/m, $f_{vw,d} = 207.85$ N/mm², a = 3.0 mm
$F_{w,Ed} = 233.00$ kN/m < $F_{w,Rd} = 623.54$ kN/m ⇒ U = 0.374 < 1 ok

verification of weld 8, pt. 0:

stresses on the design area of the weld ($\alpha = 45°$):
$\sigma_{w,Ed} = \sigma_{w,x} = 39.3$ N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a = 157.35$ kN/m
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 831.38$ kN/m, $f_{vw,d} = 207.85$ N/mm², a = 4.0 mm
$F_{w,Ed} = 157.35$ kN/m < $F_{w,Rd} = 831.38$ kN/m ⇒ U = 0.189 < 1 ok

Result:

weld 1, pt. 0: $\sigma_{w,x} = 157.35$ N/mm²
Max: $F_{w,Ed} = 629.39$ kN/m < $F_{w,Rd} = 831.38$ kN/m ⇒ $U_w = 0.757$ < 1 ok

2.1.2.5.3. verification of web stiffeners

$N_{R,t} = (-N_d \cdot z_{bu} + M_d) / z = 77.05$ kN, $z = z_{eq} = 72.7$ mm, $z_{bu} = 56.9$ mm
$N_{R,c} = (N_d \cdot z_{bo} + M_d) / z = 77.05$ kN, $z = z_{eq} = 72.7$ mm, $z_{bo} = 15.8$ mm

compression stiffener (below)

$F_{c,Ed} = N_{R,c} = 77.05$ kN
verification of the welds with the simplified method.

dimensions, lever arms, forces per rib

$b_R = b_{st} = 29.8$ mm, $b_1 = b_R - r_R = 19.3$ mm, $e_F = b_R - 0.5 \cdot b_1 = 20.2$ mm with $r_R = 10.5$ mm
three-sided joint of ribs:
$l_R = l_{st} = 107.4$ mm, $l_1 = l_R - 2 \cdot r_R = 86.4$ mm, $e_H = l_R = 107.4$ mm, $t_R = 8.0$ mm
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 27.45$ kN, $H = F \cdot e_F / e_H = 5.15$ kN
assumption: stiffeners do not buckle:
section class 1 for $\alpha = 1.000$, c/t = 3.73 (outstand flange) < $9.00 \cdot \varepsilon = 9.00 \leq 3$ 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 $\Rightarrow$ U = 0.293 $<$ 1 ok

flange welds

fillet weld with a = 4.0 mm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 831.38$ kN/m, $f_{vw,d} = 207.85$ N/mm², a = 4.0 mm
design value: $F_{Ed} = (F^2 + H^2)^{1/2} / (2 \cdot b_1) = 723.51$ kN/m
75% decrease of stress by pressure contact
$F_{Ed} = 180.88$ kN/m $<$ $F_{Rd} = 831.38$ kN/m $\Rightarrow$ U = 0.218 $<$ 1 ok

web welds

fillet weld with a = 4.0 mm
weld thickness a = 4.0 mm > $a_{max} = 0.7 \cdot t_{min} = 3.1$ mm (welding technology, s. DIN 18800) !!
weld thickness a = 4.0 mm > $a_{max} = 0.7 \cdot t_{min} = 3.1$ mm (welding technology, s. DIN 18800) !!
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 831.38$ kN/m, $f_{vw,d} = 207.85$ N/mm², a = 4.0 mm
three-sided joint of ribs:
design value: $F_{Ed} = F / (2 \cdot l_1) = 158.85$ kN/m
75% decrease of stress by pressure contact
$F_{Ed} = 39.71$ kN/m $<$ $F_{Rd} = 831.38$ kN/m $\Rightarrow$ U = 0.048 $<$ 1 ok

stiffener in tension (top)

$F_{t,Ed} = N_{R,t} = 77.05$ kN
verification of the welds with the simplified method.

dimensions, lever arms, forces per rib

$b_R = b_{st} = 29.8$ mm, $b_1 = b_R - r_R = 19.3$ mm, $e_F = b_R - 0.5 \cdot b_1 = 20.2$ mm with $r_R = 10.5$ mm
three-sided joint of ribs:
$l_R = l_{st} = 107.4$ mm, $l_1 = l_R - 2 \cdot r_R = 86.4$ mm, $e_H = l_R = 107.4$ mm, $t_R = 8.0$ mm
$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 27.45$ 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 $\Rightarrow$ 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 $\Rightarrow$ U = 0.293 $<$ 1 ok

flange welds

fillet weld with a = 4.0 mm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 831.38$ kN/m, $f_{vw,d} = 207.85$ N/mm², a = 4.0 mm
design value: $F_{Ed} = (F^2 + H^2)^{1/2} / (2 \cdot b_1) = 723.51$ kN/m
$F_{Ed} = 723.51$ kN/m $<$ $F_{Rd} = 831.38$ kN/m $\Rightarrow$ U = 0.870 $<$ 1 ok

web welds

fillet weld with a = 4.0 mm
weld thickness a = 4.0 mm > $a_{max} = 0.7 \cdot t_{min} = 3.1$ mm (welding technology, s. DIN 18800) !!
weld thickness a = 4.0 mm > $a_{max} = 0.7 \cdot t_{min} = 3.1$ mm (welding technology, s. DIN 18800) !!
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 831.38$ kN/m, $f_{vw,d} = 207.85$ N/mm², a = 4.0 mm
three-sided joint of ribs:
design value: $F_{Ed} = F / (2 \cdot l_1) = 158.85$ kN/m
$F_{Ed} = 158.85$ kN/m $<$ $F_{Rd} = 831.38$ kN/m $\Rightarrow$ U = 0.191 $<$ 1 ok

2.1.2.5.4. 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

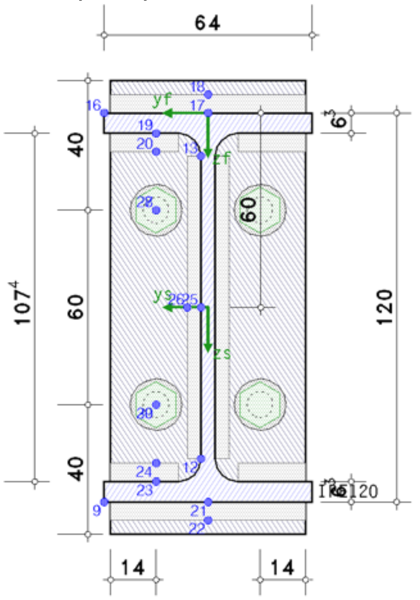
2.1.3. final result

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

2.2. fatigue of the connection bracket-column (FLS)

conversion internal forces at node -> bracket loads
moment error $\Delta M_{Ed} = M_{j,b,Ed} + F_{Ed} \cdot \Delta a - H_{Ed} \cdot \Delta h$ with $\Delta a = 200.0$ mm, $\Delta h = 60.0$ mm
$F_{1,Ed} = 5.00$ kN
$F_{2,Ed} = 15.00$ kN
design values
$M_{y1,Ed} = -0.70$ kNm, $V_{z1,Ed} = 5.00$ kN
$M_{y2,Ed} = -2.10$ kNm, $V_{z2,Ed} = 15.00$ kN
detailed problems acc. to Eurocode 3 EC 3-1-9 (12.10), NA: Deutschland

2.2.1. input report



steel grade

steel grade S235, calculation parameters:

char. yield strength $f_{y,t\leq 40mm}$ = 235.0 N/mm², $f_{y,t\leq 80mm}$ = 215.0 N/mm², $f_{y,t>80mm}$ = 215.0 N/mm²

char. tensile strength $f_{u,t\leq 40mm}$ = 360.0 N/mm², $f_{u,t\leq 80mm}$ = 360.0 N/mm², $f_{u,t>80mm}$ = 360.0 N/mm²

elastic modulus E = 210000.0 N/mm²

cross-section

beam: section IPE120

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, gewalzt with r = 7.0 mm

rolled section, root radius r = 7.0 mm

clear depth of web without root/weld d_w = 93.4 mm

width of one flange side without root/weld c_f = 22.8 mm

centroid distance from top z_s = 60.0 mm

cross-sectional area A = 13.21 cm², perimeter U = 47.52 cm

second moments of area I_y = 317.75 cm⁴, I_z = 27.66 cm⁴

plastic section moduli $W_{pl,y}$ = 60.72 cm³, $W_{pl,z}$ = 13.57 cm³

effective shear arean A_{vy} = 8.06 cm², A_{vz} = 6.31 cm²

torsional moment of inertia I_T = 1.81 cm⁴

parameters

damage equivalent stress factors λ_σ = 1.000, λ_τ = 1.000

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	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

material safety factor

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

2.2.2. 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

stresses of bolts:
N_{Ed} = -0.00 kN, $M_{y,Ed}$ = -0.70 kNm
row 1: Δz_1 = 86.9 mm, $F_{sc,1}$ = 4.70 kN, σ = 64.24 N/mm², τ = 24.87 N/mm²
row 2: Δz_2 = 26.9 mm, $F_{sc,2}$ = 1.45 kN, σ = 19.86 N/mm², τ = 24.87 N/mm²
N_{Ed} = -0.00 kN, $M_{y,Ed}$ = -2.10 kNm
row 1: Δz_1 = 86.9 mm, $F_{sc,1}$ = 14.11 kN, σ = 192.73 N/mm², τ = 74.60 N/mm²
row 2: Δz_2 = 26.9 mm, $F_{sc,2}$ = 4.36 kN, σ = 59.58 N/mm², τ = 74.60 N/mm²
stresses in end-plate (gross section):
N_{Ed} = -0.00 kN, $M_{y,Ed}$ = -0.70 kNm: σ = 8.33 N/mm²
N_{Ed} = -0.00 kN, $M_{y,Ed}$ = -2.10 kNm: σ = 25.00 N/mm²
elastic stresses / stress ranges
$\Delta\sigma_{x,Ed} = \sigma_{x,max} - \sigma_{x,min}$, $\tau_{Ed} = \tau_{xz,max} - \tau_{xz,min}$

pt. 9:	y_f = 32.0 mm, z_f = 120.0 mm	Lc 1:	σ_x = -13.2 N/mm²	
		2:	σ_x = -39.7 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 26.4 N/mm²	
12:	y_f = 2.2 mm, z_f = 106.7 mm	Lc 1:	σ_x = -10.3 N/mm²	τ_{xz} = 9.1 N/mm²
		2:	σ_x = -30.9 N/mm²	τ_{xz} = 27.4 N/mm²
			$\Delta\sigma_{x,Ed}$ = 20.6 N/mm²	$\Delta\tau_{Ed}$ = 18.2 N/mm²
13:	y_f = 2.2 mm, z_f = 13.3 mm	Lc 1:	σ_x = 10.3 N/mm²	τ_{xz} = 9.1 N/mm²
		2:	σ_x = 30.9 N/mm²	τ_{xz} = 27.4 N/mm²
			$\Delta\sigma_{x,Ed}$ = 20.6 N/mm²	$\Delta\tau_{Ed}$ = 18.2 N/mm²
16:	y_f = 32.0 mm, z_f = 0.0 mm	Lc 1:	σ_x = 13.2 N/mm²	
		2:	σ_x = 39.7 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 26.4 N/mm²	
17:	y_f = 0.0 mm, z_f = 0.0 mm	Lc 1:	σ_x = 14.6 N/mm²	
		2:	σ_x = 43.8 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 29.2 N/mm²	
18:	y_f = 0.0 mm, z_f = -5.7 mm	Lc 1:	σ_x = 14.5 N/mm²	
		2:	σ_x = 43.4 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 28.9 N/mm²	
19:	y_f = 16.0 mm, z_f = 6.3 mm	Lc 1:	σ_x = 13.1 N/mm²	
		2:	σ_x = 39.2 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 26.2 N/mm²	
20:	y_f = 16.0 mm, z_f = 12.0 mm	Lc 1:	σ_x = 10.6 N/mm²	
		2:	σ_x = 31.8 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 21.2 N/mm²	
21:	y_f = 0.0 mm, z_f = 120.0 mm	Lc 1:	σ_x = -14.6 N/mm²	
		2:	σ_x = -43.8 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 29.2 N/mm²	
22:	y_f = 0.0 mm, z_f = 125.7 mm	Lc 1:	σ_x = -14.5 N/mm²	
		2:	σ_x = -43.4 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 28.9 N/mm²	
23:	y_f = 16.0 mm, z_f = 113.7 mm	Lc 1:	σ_x = -13.1 N/mm²	
		2:	σ_x = -39.2 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 26.2 N/mm²	
24:	y_f = 16.0 mm, z_f = 108.0 mm	Lc 1:	σ_x = -10.6 N/mm²	
		2:	σ_x = -31.8 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 21.2 N/mm²	
25:	y_f = 2.2 mm, z_f = 60.0 mm	Lc 1:	σ_x = -0.0 N/mm²	τ_{xz} = 9.1 N/mm²
		2:	σ_x = -0.0 N/mm²	τ_{xz} = 27.4 N/mm²
				$\Delta\tau_{Ed}$ = 18.3 N/mm²
26:	y_f = 6.4 mm, z_f = 60.0 mm	Lc 1:		τ_{xz} = 10.9 N/mm²
		2:		τ_{xz} = 32.6 N/mm²
				$\Delta\tau_{Ed}$ = 21.7 N/mm²
27:	y_f = 16.0 mm, z_f = 30.0 mm	Lc 1:	σ_x = 64.2 N/mm²	τ_{xz} = 24.9 N/mm²
		2:	σ_x = 192.7 N/mm²	τ_{xz} = 74.6 N/mm²
			$\Delta\sigma_{x,Ed}$ = 128.5 N/mm²	$\Delta\tau_{Ed}$ = 49.7 N/mm²
28:	y_f = 16.0 mm, z_f = 30.0 mm	Lc 1:	σ_x = 8.3 N/mm²	
		2:	σ_x = 25.0 N/mm²	
			$\Delta\sigma_{x,Ed}$ = 16.7 N/mm²	

29: y_f = 16.0 mm, z_f = 90.0 mm	Lc 1:	σ_x = 19.9 N/mm²	τ_{xz} = 24.9 N/mm²
	2:	σ_x = 59.6 N/mm²	τ_{xz} = 74.6 N/mm²
		$\Delta\sigma_{x,Ed}$ = 39.7 N/mm²	$\Delta\tau_{Ed}$ = 49.7 N/mm²
30: y_f = 16.0 mm, z_f = 90.0 mm	Lc 1:	σ_x = 8.3 N/mm²	
	2:	σ_x = 25.0 N/mm²	
		$\Delta\sigma_{x,Ed}$ = 16.7 N/mm²	

equivalent constand amplitude stress range

$\Delta\sigma_{x,f} = \Delta\sigma_{x,Ed}\cdot\lambda_\sigma$, $\Delta\tau_f = \Delta\tau_{Ed}\cdot\lambda_\tau$

pt. 9: y_f = 32.0 mm, z_f = 120.0 mm	$\Delta\sigma_{x,f}$ = 26.4 N/mm²	
12: y_f = 2.2 mm, z_f = 106.7 mm	$\Delta\sigma_{x,f}$ = 20.6 N/mm²	$\Delta\tau_f$ = 18.2 N/mm²
13: y_f = 2.2 mm, z_f = 13.3 mm	$\Delta\sigma_{x,f}$ = 20.6 N/mm²	$\Delta\tau_f$ = 18.2 N/mm²
16: y_f = 32.0 mm, z_f = 0.0 mm	$\Delta\sigma_{x,f}$ = 26.4 N/mm²	
17: y_f = 0.0 mm, z_f = 0.0 mm	$\Delta\sigma_{x,f}$ = 29.2 N/mm²	
18: y_f = 0.0 mm, z_f = -5.7 mm	$\Delta\sigma_{x,f}$ = 28.9 N/mm²	
19: y_f = 16.0 mm, z_f = 6.3 mm	$\Delta\sigma_{x,f}$ = 26.2 N/mm²	
20: y_f = 16.0 mm, z_f = 12.0 mm	$\Delta\sigma_{x,f}$ = 21.2 N/mm²	
21: y_f = 0.0 mm, z_f = 120.0 mm	$\Delta\sigma_{x,f}$ = 29.2 N/mm²	
22: y_f = 0.0 mm, z_f = 125.7 mm	$\Delta\sigma_{x,f}$ = 28.9 N/mm²	
23: y_f = 16.0 mm, z_f = 113.7 mm	$\Delta\sigma_{x,f}$ = 26.2 N/mm²	
24: y_f = 16.0 mm, z_f = 108.0 mm	$\Delta\sigma_{x,f}$ = 21.2 N/mm²	
25: y_f = 2.2 mm, z_f = 60.0 mm		$\Delta\tau_f$ = 18.3 N/mm²
26: y_f = 6.4 mm, z_f = 60.0 mm		$\Delta\tau_f$ = 21.7 N/mm²
27: y_f = 16.0 mm, z_f = 30.0 mm	$\Delta\sigma_{x,f}$ = 128.5 N/mm²	$\Delta\tau_f$ = 49.7 N/mm²
28: y_f = 16.0 mm, z_f = 30.0 mm	$\Delta\sigma_{x,f}$ = 16.7 N/mm²	
29: y_f = 16.0 mm, z_f = 90.0 mm	$\Delta\sigma_{x,f}$ = 39.7 N/mm²	$\Delta\tau_f$ = 49.7 N/mm²
30: y_f = 16.0 mm, z_f = 90.0 mm	$\Delta\sigma_{x,f}$ = 16.7 N/mm²	

valid notch stresses

$\Delta\sigma_{x,Rd,f} = \Delta\sigma_{x,Rd}/\gamma_{Mf}$, $\Delta\tau_{Rd,f} = \Delta\tau_{Rd}/\gamma_{Mf}$

pt. 9: y_f = 32.0 mm, z_f = 120.0 mm	$\Delta\sigma_{x,Rd,f}$ = 139.1 N/mm²	
12: y_f = 2.2 mm, z_f = 106.7 mm	$\Delta\sigma_{x,Rd,f}$ = 139.1 N/mm²	$\Delta\tau_{Rd,f}$ = 87.0 N/mm²
13: y_f = 2.2 mm, z_f = 13.3 mm	$\Delta\sigma_{x,Rd,f}$ = 139.1 N/mm²	$\Delta\tau_{Rd,f}$ = 87.0 N/mm²
16: y_f = 32.0 mm, z_f = 0.0 mm	$\Delta\sigma_{x,Rd,f}$ = 139.1 N/mm²	
17: y_f = 0.0 mm, z_f = 0.0 mm	$\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm²	
18: y_f = 0.0 mm, z_f = -5.7 mm	$\Delta\sigma_{x,Rd,f}$ = 69.6 N/mm²	
19: y_f = 16.0 mm, z_f = 6.3 mm	$\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm²	
20: y_f = 16.0 mm, z_f = 12.0 mm	$\Delta\sigma_{x,Rd,f}$ = 69.6 N/mm²	
21: y_f = 0.0 mm, z_f = 120.0 mm	$\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm²	
22: y_f = 0.0 mm, z_f = 125.7 mm	$\Delta\sigma_{x,Rd,f}$ = 69.6 N/mm²	
23: y_f = 16.0 mm, z_f = 113.7 mm	$\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm²	
24: y_f = 16.0 mm, z_f = 108.0 mm	$\Delta\sigma_{x,Rd,f}$ = 69.6 N/mm²	
25: y_f = 2.2 mm, z_f = 60.0 mm	$\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm²	$\Delta\tau_{Rd,f}$ = 69.6 N/mm²
26: y_f = 6.4 mm, z_f = 60.0 mm		$\Delta\tau_{Rd,f}$ = 87.0 N/mm²
27: y_f = 16.0 mm, z_f = 30.0 mm	$\Delta\sigma_{x,Rd,f}$ = 43.5 N/mm²	$\Delta\tau_{Rd,f}$ = 87.0 N/mm²
28: y_f = 16.0 mm, z_f = 30.0 mm	$\Delta\sigma_{x,Rd,f}$ = 78.3 N/mm²	
29: y_f = 16.0 mm, z_f = 90.0 mm	$\Delta\sigma_{x,Rd,f}$ = 43.5 N/mm²	$\Delta\tau_{Rd,f}$ = 87.0 N/mm²
30: y_f = 16.0 mm, z_f = 90.0 mm	$\Delta\sigma_{x,Rd,f}$ = 78.3 N/mm²	

verification of notch stresses

pt. 9: y = 32.0 mm, z = 120.0 mm	$\Delta\sigma_{x,f}$ = 26.4 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 139.1 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.190 ok
12: y = 2.2 mm, z = 106.7 mm	$\Delta\sigma_{x,f}$ = 20.6 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 139.1 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.148 ok
	$\Delta\tau_f$ = 18.2 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm²	$\Rightarrow U_{\Delta\tau}$ = 0.210 ok
	interaction $U_i = U_{\Delta\sigma x}^3 + U_{\Delta\tau}^5$ = 0.004 < 1 ok	
13: y = 2.2 mm, z = 13.3 mm	$\Delta\sigma_{x,f}$ = 20.6 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 139.1 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.148 ok
	$\Delta\tau_f$ = 18.2 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm²	$\Rightarrow U_{\Delta\tau}$ = 0.210 ok
	interaction $U_i = U_{\Delta\sigma x}^3 + U_{\Delta\tau}^5$ = 0.004 < 1 ok	
16: y = 32.0 mm, z = 0.0 mm	$\Delta\sigma_{x,f}$ = 26.4 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 139.1 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.190 ok
17: y = 0.0 mm, z = 0.0 mm	$\Delta\sigma_{x,f}$ = 29.2 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.933 ok
18: y = 0.0 mm, z = -5.7 mm	$\Delta\sigma_{x,f}$ = 28.9 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 69.6 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.416 ok
19: y = 16.0 mm, z = 6.3 mm	$\Delta\sigma_{x,f}$ = 26.2 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.835 ok
20: y = 16.0 mm, z = 12.0 mm	$\Delta\sigma_{x,f}$ = 21.2 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 69.6 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.304 ok
21: y = 0.0 mm, z = 120.0 mm	$\Delta\sigma_{x,f}$ = 29.2 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.933 ok
22: y = 0.0 mm, z = 125.7 mm	$\Delta\sigma_{x,f}$ = 28.9 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 69.6 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.416 ok
23: y = 16.0 mm, z = 113.7 mm	$\Delta\sigma_{x,f}$ = 26.2 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 31.3 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.835 ok
24: y = 16.0 mm, z = 108.0 mm	$\Delta\sigma_{x,f}$ = 21.2 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 69.6 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.304 ok
25: y = 2.2 mm, z = 60.0 mm	$\Delta\tau_f$ = 18.3 N/mm² < $\Delta\tau_{Rd,f}$ = 69.6 N/mm²	$\Rightarrow U_{\Delta\tau}$ = 0.263 ok
26: y = 6.4 mm, z = 60.0 mm	$\Delta\tau_f$ = 21.7 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm²	$\Rightarrow U_{\Delta\tau}$ = 0.250 ok
27: y = 16.0 mm, z = 30.0 mm	$\Delta\sigma_{x,f}$ = 128.5 N/mm² > $\Delta\sigma_{x,Rd,f}$ = 43.5 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 2.955 not ok !!
	$\Delta\tau_f$ = 49.7 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm²	$\Rightarrow U_{\Delta\tau}$ = 0.572 ok
28: y = 16.0 mm, z = 30.0 mm	$\Delta\sigma_{x,f}$ = 16.7 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 78.3 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.213 ok
29: y = 16.0 mm, z = 90.0 mm	$\Delta\sigma_{x,f}$ = 39.7 N/mm² < $\Delta\sigma_{x,Rd,f}$ = 43.5 N/mm²	$\Rightarrow U_{\Delta\sigma x}$ = 0.914 ok
	$\Delta\tau_f$ = 49.7 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm²	$\Rightarrow U_{\Delta\tau}$ = 0.572 ok

30: $y = 16.0$ mm, $z = 90.0$ mm $\Delta\sigma_{x,f} = 16.7$ N/mm² < $\Delta\sigma_{x,Rd,f} = 78.3$ N/mm² ⇒ $U_{\Delta\sigma x} = 0.213$ ok

2.2.3. final result

fatigue design [pt. 27]: max $U = 2.955 > 1$ not ok !!

2.3. fatigue of bracket cross-section (FLS)

conversion internal forces at node -> bracket loads

moment error $\Delta M_{Ed} = M_{j,b,Ed}+F_{Ed}\cdot\Delta a-H_{Ed}\cdot\Delta h$ with $\Delta a = 200.0$ mm, $\Delta h = 60.0$ mm

$F_{1,Ed} = 5.00$ kN

$F_{2,Ed} = 15.00$ kN

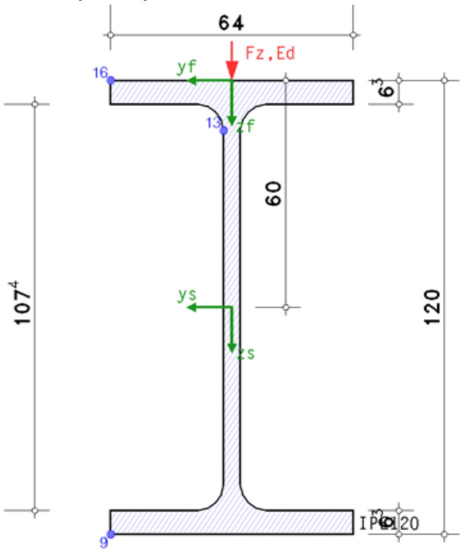
design values

$V_{z1,Ed} = 5.00$ kN

$V_{z2,Ed} = 15.00$ kN

detailed problems acc. to Eurocode 3 EC 3-1-9 (12.10), NA: Deutschland

2.3.1. input report



steel grade

steel grade S235, calculation parameters:

char. yield strength $f_{y,t\leq40mm} = 235.0$ N/mm², $f_{y,t\leq80mm} = 215.0$ N/mm², $f_{y,t>80mm} = 215.0$ N/mm²

char. tensile strength $f_{u,t\leq40mm} = 360.0$ N/mm², $f_{u,t\leq80mm} = 360.0$ N/mm², $f_{u,t>80mm} = 360.0$ N/mm²

elastic modulus $E = 210000.0$ N/mm²

cross-section

beam: section IPE120

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, gewalzt with $r = 7.0$ mm

rolled section, root radius $r = 7.0$ mm

clear depth of web without root/weld $d_w = 93.4$ mm

width of one flange side without root/weld $c_f = 22.8$ mm

centroid distance from top $z_s = 60.0$ mm

cross-sectional area $A = 13.21$ cm², perimeter $U = 47.52$ cm

second moments of area $I_y = 317.75$ cm⁴, $I_z = 27.66$ cm⁴

plastic section moduli $W_{pl,y} = 60.72$ cm³, $W_{pl,z} = 13.57$ cm³

effective shear arean $A_{vy} = 8.06$ cm², $A_{vz} = 6.31$ cm²

torsional moment of inertia $I_T = 1.81$ cm⁴

parameters

damage equivalent stress factors $\lambda_\sigma = 1.000$, $\lambda_\tau = 1.000$

notch class / valid notch stresses at $N = 2\cdot10^6$ 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, loading length s_s = 30.0 mm
material safety factor
design concept: damage tolerance, damage consequence: high ⇒ fatigue strength γ_{Mf} = 1.15

2.3.2. 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, s_s = 30.0 mm, t_f = 6.3 mm
effective loading length refered ...
... 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
local stresses ...
... at beam web σ_{oz} = -60.2 N/mm², τ_o = 12.0 N/mm²

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 = 2.2 mm, z_f = 13.3 mm   Lc 1: τ_{xz} = 9.1 N/mm²
2: τ_{xz} = 27.4 N/mm²
$\Delta\tau_{Ed}$ = 42.3 N/mm² $\Delta\sigma_{z,Ed}$ = 60.2 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 = 2.2 mm, z_f = 13.3 mm $\Delta\tau_f$ = 42.3 N/mm² $\Delta\sigma_{z,f}$ = 60.2 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. 9: y_f = 32.0 mm, z_f = 120.0 mm
13: y_f = 2.2 mm, z_f = 13.3 mm $\Delta\tau_{Rd,f}$ = 87.0 N/mm² $\Delta\sigma_{z,Rd,f}$ = 139.1 N/mm²
16: y_f = 32.0 mm, z_f = 0.0 mm

verification of notch stresses
pt. 13: y = 2.2 mm, z = 13.3 mm $\Delta\tau_f$ = 42.3 N/mm² < $\Delta\tau_{Rd,f}$ = 87.0 N/mm² ⇒ $U_{\Delta\tau}$ = 0.487 ok
$\Delta\sigma_{z,f}$ = 60.2 N/mm² < $\Delta\sigma_{z,Rd,f}$ = 139.1 N/mm² ⇒ $U_{\Delta\sigma z}$ = 0.433 ok

2.3.3. final result

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

3. final result

maximum utilization: max U = 2.955 > 1 not ok !!
fatigue of the connection / of the section

resistance not ensured !!

4. Selected Design Parameters of the National Annex

EN 1993-1-1 (EC 3, Hochbau), NA Deutschland

chapter	value	definition
6.1(1)		partial safety factors for structural steel
	γ_{M0} = 1.00	collapse of cross-section
	γ_{M1} = 1.10	instability
	γ_{M2} = 1.25	fracture cross-sections in tension
		resp. resistance of bolts,
		welds, plates in bearing

EN 1993-1-9 (EC 3, fatigue), NA Deutschland

chapter	value		definition
3(7)	damage consequence		partial safety factor of fatigue
	low	high	
	γ_{Mf} = 1.00	γ_{Mf} = 1.15	damage tolerance
	γ_{Mf} = 1.15	γ_{Mf} = 1.35	failure without notice

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 1993-1-9, 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