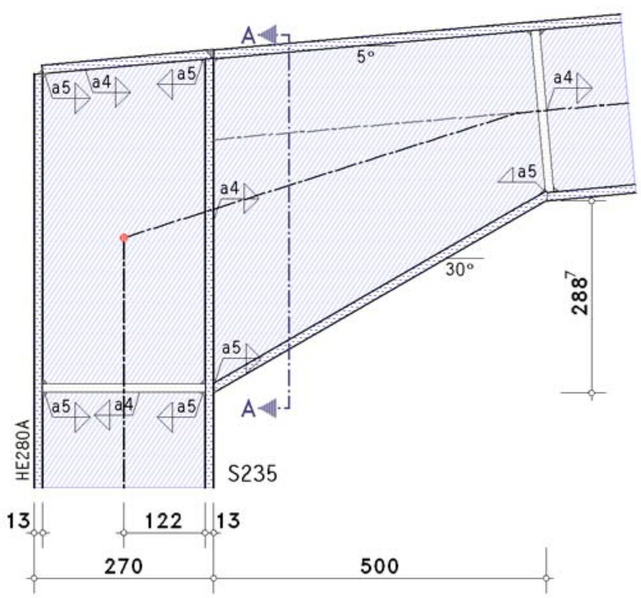
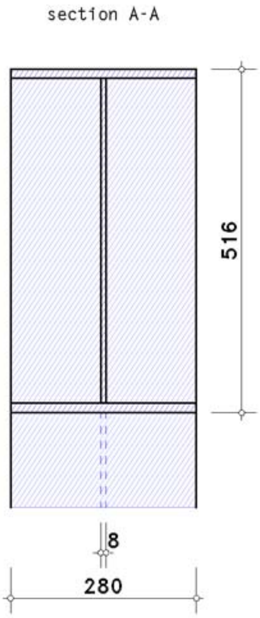
POS. 9: BEISPIEL BUCKLING/SHEARFELD

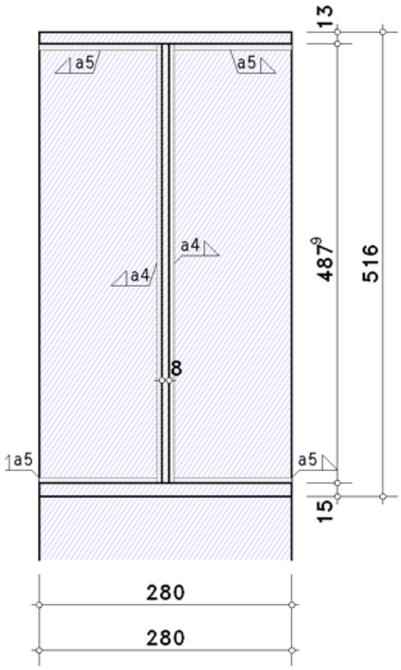
frame corner EC 3-1-8 (12.10), NA: Deutschland

1. input report





details (section A - A)



steel grade
steel grade S235
column parameters
section HE280A
reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 500.8 mm):
thickness t_{st} = 13.0 mm, width b_{st} = 136.0 mm, length l_{st} = 244.0 mm
recess at stiffeners c_{st} = 36.0 mm
welds $a_{st,f}$ = 5.0 mm, $a_{st,w}$ = 4.0 mm
beam parameters
parameter (I-section):
overall depth h = 270.0 mm, web thickness t_w = 8.0 mm
flange width b_f = 280.0 mm, flange thickness t_f = 13.0 mm
slope angle of section about the horizontal axis α_b = 5.00° $\Rightarrow$ section depth at the joint loc. $h_b = h/\cos(\alpha_b)$ = 271.0 mm
slope angle of haunch about the horizontal axis α_v = 30.00° $\Rightarrow$ haunch angle about the beam axis $\Delta\alpha_v$ = 25.00°
haunch length L_v = 500.0 mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b))$ = 244.9 mm
total beam depth at the connection point $h_{ges} = h_b + h_v$ = 516.0 mm
reinforcement of the section with transverse stiffeners:

thickness t_{st} = 13.0 mm, width b_{st} = 136.0 mm, length l_{st} = 244.0 mm
welds $a_{st,f}$ = 5.0 mm, $a_{st,w}$ = 4.0 mm

verification parameters

welded connection:

tension plate: thickness t_z = 13.0 mm, width b_z = 280.0 mm

welds $a_{z,f}$ = 5.0 mm, $a_{z,w}$ = 4.0 mm

welds at the connection point:

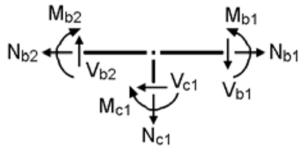
beam flange top: fillet weld, weld thickness a = 5.0 mm, angle φ = 85°

beam web: fillet weld, weld thickness a = 4.0 mm

beam flange bottom: fillet weld, weld thickness a = 5.0 mm, angle φ = 120°

internal forces and moments in the intersection point of system axes

Lk 1: $N_{j,b,Ed}$ = -800.00 kN $M_{j,b,Ed}$ = -100.00 kNm
$N_{j,c1,Ed}$ = -240.56 kN $M_{j,c1,Ed}$ = -100.00 kNm $V_{j,c1,Ed}$ = -762.97 kN (calculated)



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

check of data

ok

notes

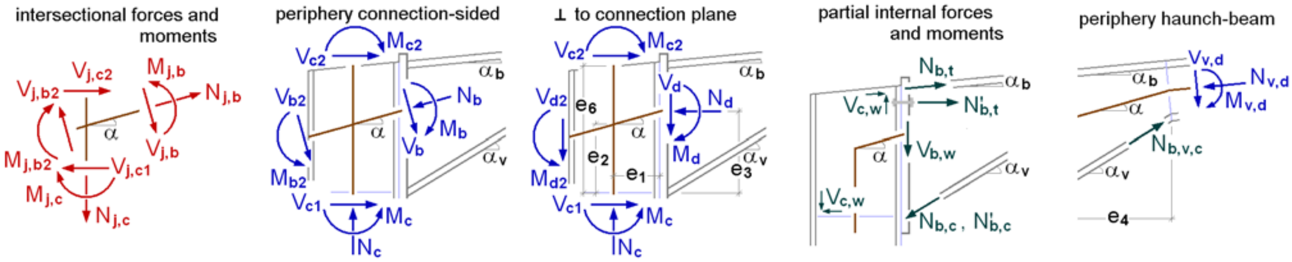
no verification for cross-sections.

2. Lk 1

notes

no verfication for welds of welded section.

2.1. design values



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: α_b = 5.00°, α_v = 30.00° ⇒ $\alpha = (\alpha_b+\alpha_v)/2$ = 17.50°, $\Delta\alpha = \alpha-\alpha_b$ = 12.50°
distance: e_1 = 135.0 mm, e_3 = 268.5 mm, e_2 = 225.9 mm, e_6 = 500.8 mm, e_4 = 496.1 mm

transformation sign convention of statics -> EC3-cos
$N_{j,b,Ed}$ = 800.00 kN, $M_{j,b,Ed}$ = 100.00 kNm
transformation node values -> joint values
$N_{b,Ed}$ = 800.00 kN, $M_{b,Ed}$ = 100.00 kNm
transformation joint values -> design values
N_d = 762.97 kN, M_d = 100.00 kNm, V_d = 240.56 kN
internal forces and moments at periphery haunch-beam
$N_{v,d}$ = 781.04 kN, $M_{v,d}$ = 100.00 kNm, $V_{v,d}$ = 173.15 kN

internal forces and moments perpendicular to the connection planes

periphery beam
N_d = 762.97 kN, M_d = 100.00 kNm, V_d = 240.56 kN
periphery haunch-beam
$N_{v,d}$ = 781.04 kN, $M_{v,d}$ = 100.00 kNm, $V_{v,d}$ = 173.15 kN
periphery column (bottom, calculated)
N_c = 240.56 kN, M_c = -72.39 kNm, V_c = 762.97 kN

$|N_{b,Ed}|$ = 800.00 kN > 5%$\cdot N_{pl,Rd}$ = 138.18 kN ⇒ verification with partial internal forces and moments
with $N_{pl,Rd} = A_b \cdot f_{yb} / \gamma_{M0}$ = 2763.65 kN

partial internal forces and moments

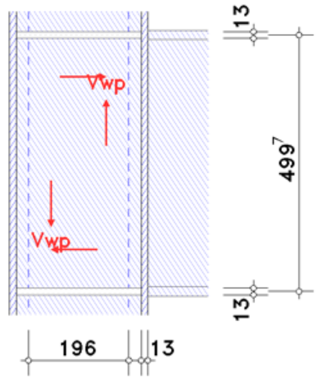
$N_{b,t} = (-N_d \cdot z_{bu}/z_b + M_d/z_b) / \cos(\alpha_b)$ = -165.32 kN, z_b = 501.9 mm, z_{bu} = 239.4 mm < 0 (compression connection)
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M_d/z_b) / \cos(\alpha_v)$ = 690.84 kN, z_b = 501.9 mm, z_{bo} = 262.5 mm
$V_{b,w} = V_d + N_{b,c} \cdot \sin(\alpha_v) - N_{b,t} \cdot \sin(\alpha_b)$ = 600.39 kN

2.2. basic components

welded beam connection: basic components: 1, 2, 3, 20

2.2.1. Gk 1: Column web panel in shear

transformation parameter (EC 3-1-8, 5.3(9)) β_j = 1.000 for M_{j1} = 100.00 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} = 24.50 < $69\cdot\varepsilon$ = 69.00, ε = 1.00 ⇒ method applicable

shear area

shear area A_v = 31.74 cm²

plastic shear resistance

plastic shear resistance without stiffeners $V_{wp,Rd} = (0.9\cdot f_{y,w}\cdot A_v) / (3^{1/2}\cdot\gamma_{M0})$ = 387.6 kN

placing of intermediate web stiffeners:

plastic section modulus of a column flange $W_{pl,fc} = b_{fc}\cdot t_{fc}^2/4$ = 11.83 cm³

plastic section modulus of a web stiffener $W_{pl,st} = 2\cdot b_{st}\cdot t_{st}^2/4$ = 11.49 cm³

plastic moment resistance of column flange $M_{pl,fc,Rd} = (W_{pl,fc}\cdot f_{y,c}) / \gamma_{M0}$ = 2.78 kNm

plastic moment resistance of the web stiffener $M_{pl,st,Rd} = (W_{pl,st}\cdot f_{y,st}) / \gamma_{M0}$ = 2.70 kNm

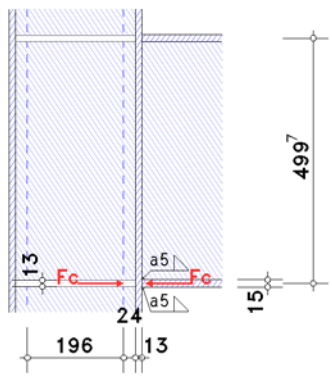
additional resistance $V_{wp,add,Rd} = 4\cdot M_{pl,fc,Rd}/d_{st}$ = 22.3 kN

$V_{wp,add,Rd} > 2\cdot(M_{pl,fc,Rd}+M_{pl,st,Rd})/d_{st}$ = 21.9 kN ⇒ $V_{wp,add,Rd}$ = 21.9 kN, d_{st} = 499.7 mm

plastic shear resistance with transverse stiffeners $V_{wp,Rd}$ = 409.6 kN

2.2.2. Gk 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) β_j = 1.000 for M_{j1} = 100.00 kNm (M_{j2} = 0)



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

reinforcement of web with transverse stiffeners:

assumption: stiffeners do not buckle (verification method 'elastic-elastic' ⇒ max. section class 3)

c/t-ratio c/t = 10.46 ≤ 33.00 = $33\cdot\varepsilon$, $\varepsilon = (235/f_y)^{1/2}$ = 1.00 ⇒ section class 1 ≤ 3 ⇒ assumption succeeded !!

minimum demands of the moment of inertia of stiffeners

length of buckling field (distance of stiffeners) a = 499.7 mm

web height between the flanges $h_{wc} = d_c+2\cdot s_c$ = 244.0 mm

moment of inertia of stiffeners $I_{st} = (2\cdot b_{st}+t_{wc})^3\cdot t_{st} / 12$ = 2378.13 cm⁴

minimum moment of inertia for a/h_{wc} = 2.05 ≥ $2^{1/2}$: $I_{st,min} = 0.75\cdot h_{wc}\cdot t_{wc}^3$ = 9.37 cm⁴ < I_{st} ok

requirement concerning stiffeners to aviod lateral torsional buckling

torsional moment of inertia of stiffeners $I_T \approx b_{st}\cdot t_{st}^3 / 3$ = 9.96 cm⁴

polar moment of inertia of stiffeners $I_p = b_{st}\cdot t_{st}^3 / 12 + t_{st}\cdot b_{st}^3 / 12$ = 275.00 cm⁴

$I_T / I_p \approx 0.036 > 0.006 = 5.3\cdot f_{y,st}/E_{st}$ ok

resistance of stiffened webs with transverse compression

area of stiffeners incl. web $A_{st} = (2\cdot b_{st}+t_{wc})\cdot t_{st}$ = 36.40 cm²

radius of inertia of stiffeners $i = (I_{st} / A_{st})^{1/2}$ = 80.8 mm

buckling length of stiffeners $L_{cr} = h_{wc}$ = 244.0 mm

slenderness $\lambda = L_{cr} / (i\cdot\lambda_1)$ = 0.032 with $\lambda_1 = \pi\cdot(E_{st}/f_{y,st})^{1/2}$ = 93.9

$\lambda \le 0.2$ ⇒ no deduction (χ = 1.0)

design value of resistance of flexural buckling $F_{c,w,Rd} = \chi\cdot A_{st}\cdot f_{y,st} / \gamma_{M1}$ = 777.6 kN

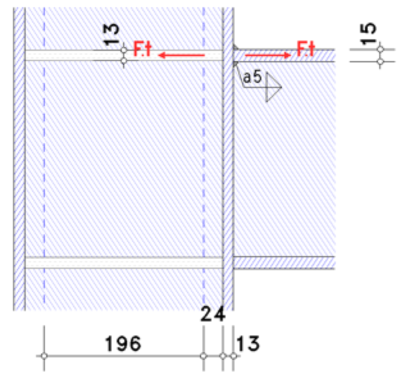
resistance of upper beam flange:

reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle (verification method 'elastic-elastic' ⇒ max. section class 3)
c/t-ratio $c/t = 10.46 \le 33.00 = 33 \cdot \varepsilon$, $\varepsilon = (235/f_y)^{1/2} = 1.00$ ⇒ section class $1 \le 3$ ⇒ assumption succeeded !!
minimum demands of the moment of inertia of stiffeners
length of buckling field (distance of stiffeners) $a = 499.7$ mm
web height between the flanges $h_{wc} = d_c + 2 \cdot s_c = 244.0$ mm
moment of inertia of stiffeners $I_{st} = (2 \cdot b_{st} + t_{wc})^3 \cdot t_{st} / 12 = 2378.13$ cm^4
minimum moment of inertia for $a/h_{wc} = 2.05 \ge 2^{1/2}$: $I_{st,min} = 0.75 \cdot h_{wc} \cdot t_{wc}^3 = 9.37$ cm^4 $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling
torsional moment of inertia of stiffeners $I_T \approx b_{st} \cdot t_{st}^3 / 3 = 9.96$ cm^4
polar moment of inertia of stiffeners $I_p = b_{st} \cdot t_{st}^3 / 12 + t_{st} \cdot b_{st}^3 / 12 = 275.00$ cm^4
$I_T / I_p \approx 0.036 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of stiffened webs with transverse compression
area of stiffeners incl. web $A_{st} = (2 \cdot b_{st} + t_{wc}) \cdot t_{st} = 36.40$ cm^2
radius of inertia of stiffeners $i = (I_{st} / A_{st})^{1/2} = 80.8$ mm
buckling length of stiffeners $L_{cr} = h_{wc} = 244.0$ mm
slenderness $\lambda = L_{cr} / (i \cdot \lambda_1) = 0.032$ with $\lambda_1 = \pi \cdot (E_{st}/f_{y,st})^{1/2} = 93.9$
$\lambda \le 0.2$ ⇒ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd} = \chi \cdot A_{st} \cdot f_{y,st} / \gamma_{M1} = 777.6$ kN

2.2.3. Gk 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.000$ for $M_{j1} = 100.00$ kNm ($M_{j2} = 0$)

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

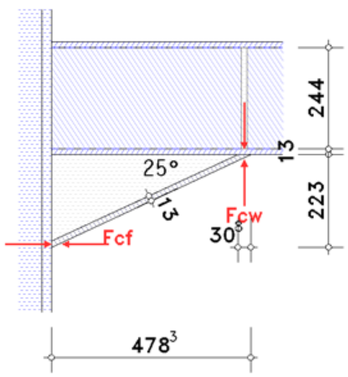


reinforcement of web with transverse stiffeners:
area of stiffeners incl. web $A_{st} = (2 \cdot b_{st} + t_{wc}) \cdot t_{st} = 36.40$ cm^2
resistance of a column web with transverse tension $F_{t,wc,Rd} = A_{st} \cdot f_{y,st} / \gamma_{M0} = 855.4$ kN, $f_{y,st} = 235.0$ N/mm^2

2.2.4. Gk 20: haunched beam in compression

longitudinal compressive stress in the beamsteg $\sigma_{com,Ed} = N_{v,d}/A_b + M_{v,d}/W_{w,b} = 178.45$ N/mm^2
$N_{v,d} = 781.0$ kN, $A_b = 92.3$ cm^2, $M_{v,d} = 100.0$ kNm, $W_{w,b} = I_{y,b}/z_{w,b} = 1065.5$ cm^3, $z_{w,b} = 122.0$ mm
section class of the beam in connection plane ($\varepsilon = 1.00$):
flange top: section class for $c/(\varepsilon \cdot t) = 10.42$ (outstand flange): 3
flange bottom: section class for $c/(\varepsilon \cdot t) = 9.06$ (outstand flange): 2
web: section class for $c/(\varepsilon \cdot t) = 60.99$ (internal compression parts): 4
total: section class: 4
transformation into aspect plane (due to beam slope): $L_{v,Gk20} = L_v \cdot \cos \alpha_b = 478.3$ mm, $\alpha_{v,Gk20} = 25.00°$

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



assumption: haunch flange no hazard of buckling
c/t-ratio $10 \cdot \varepsilon = 10.00 < c/t = 10.46 \le 14.00 = 14 \cdot \varepsilon$, $\varepsilon = (235/f_y)^{1/2} = 1.00$ ⇒ section class $3 \le 3$ ok

connection haunch-column: (basic component 7: beam flange and web in compression)
resistance only of compression flange (without web, elastic ⇒ section class 3)
haunch flange width $b_{f,v} = 280.0$ mm $<$ max $b_{f,v} = 42 \cdot t_{f,v} \cdot \varepsilon = 546.0$ mm ($\varepsilon = 1.00$) ok
beam height incl. haunch $h = h_b + h_v = 493.0$ mm, thickness of flanges $t_{fo} = 13.0$ mm, $t_{f,v} = 13.0$ mm
section modulus $W_{el,v} = 1747.307$ cm^3
stress for section class 3

resistance $M_{c,Rd} = M_{el,Rd} = (W_{el,min} \cdot f_y) / \gamma_{M0} = 410.62$ kNm, $W_{el,min} = 1747.31$ cm^3

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 855.40$ kN, $(h - t_f) = 480.0$ mm

refering to aspect plane $F_{c,f,Rd} \cdot \cos \alpha_v = 775.26$ kN

connection haunch-beam: **(basic component 2: Column web in transverse compression)**

reinforcement of web with transverse stiffeners:

assumption: stiffeners do not buckle (verification method 'elastic-elastic' $\Rightarrow$ max. section class 3)

c/t-ratio $c/t = 10.46 \leq 33.00 = 33 \cdot \varepsilon$, $\varepsilon = (235/f_y)^{1/2} = 1.00$ $\Rightarrow$ section class $1 \leq 3$ $\Rightarrow$ assumption succeeded !!

minimum demands of the moment of inertia of stiffeners

length of buckling field (distance of stiffeners) a = 478.3 mm

web height between the flanges $h_{wc} = d_c + 2 \cdot s_c = 244.0$ mm

moment of inertia of stiffeners $I_{st} = (2 \cdot b_{st} + t_{wc})^3 \cdot t_{st} / 12 = 2378.13$ cm^4

minimum moment of inertia for $a/h_{wc} = 1.96 \geq 2^{1/2}$: $I_{st,min} = 0.75 \cdot h_{wc} \cdot t_{wc}^3 = 9.37$ cm^4 $< I_{st}$ ok

requirement concerning stiffeners to aviod lateral torsional buckling

torsional moment of inertia of stiffeners $I_T \approx b_{st} \cdot t_{st}^3 / 3 = 9.96$ cm^4

polar moment of inertia of stiffeners $I_p = b_{st} \cdot t_{st}^3 / 12 + t_{st} \cdot b_{st}^3 / 12 = 275.00$ cm^4

$I_T / I_p \approx 0.036 > 0.006 = 5.3 \cdot f_{y,st} / E_{st}$ ok

resistance of stiffened webs with transverse compression

area of stiffeners incl. web $A_{st} = (2 \cdot b_{st} + t_{wc}) \cdot t_{st} = 36.40$ cm^2

radius of inertia of stiffeners $i = (I_{st} / A_{st})^{1/2} = 80.8$ mm

buckling length of stiffeners $L_{cr} = h_{wc} = 244.0$ mm

slenderness $\lambda = L_{cr} / (i \cdot \lambda_1) = 0.032$ 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 $F_{c,w,Rd} = \chi \cdot A_{st} \cdot f_{y,st} / \gamma_{M1} = 777.6$ kN

refering to aspect plane $F_{c,w,Rd} \cdot \cot \alpha_v = 1667.65$ kN

total loading capacity of a haunched beam in compression

$F_{c,v,Rd} = \min(F_{c,f,Rd} \cdot \cos \alpha_v, F_{c,w,Rd} \cdot \cot \alpha_v) = 775.26$ kN

retransformation (due to beam slope) $F_{c,v,Rd} \cdot \cos \alpha_v / \cos \alpha_{v,Gk20} = 740.80$ kN

resistance of upper beam flange (Gk 7):

stress for section class 4

resistance $M_{c,Rd} = (W_{eff,min} \cdot f_y) / \gamma_{M0} = 498.61$ kNm, $W_{eff,min} = 2121.76$ cm^3

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 993.39$ kN, $(h - t_f) = 501.9$ mm

2.3. shear resistance

shear resistance of column web

decisive basic component: 1

$V_{wp,Rd}/\beta_j = 409.6$ kN

2.4. verifications

2.4.1. verification of the connection capacity with partial internal forces and moments

shear force in column web:

$V_{c,w,Ed} = (M_{d1,w} - M_{d2,w})/z - V_c/2 = -182.26$ kN, $M_{d1,w} = 100.0$ kNm, $M_{d2,w} = 0.0$ kNm

$z = 501.9$ mm

Gk 1: $F_{Rd} = V_{wp,Rd}/\beta_j = 409.6$ kN, $F_{Ed} = |V_{c,w,Ed}| = 182.26$ kN

$F_{Ed} = 182.3$ kN $<$ $F_{Rd} = 409.6$ kN $\Rightarrow$ $U = 0.445$ $<$ 1 ok

Gk 2: $F_{Rd} = F_{c,w,Rd} = 777.6$ kN, $F_{Ed} = N_{b,c} = 690.84$ kN

$F_{Ed} = 690.8$ kN $<$ $F_{Rd} = 777.6$ kN $\Rightarrow$ $U = 0.888$ $<$ 1 ok

Gk 3: $F_{Rd} = F_{t,wc,Rd} = 855.4$ kN, $F_{Ed} = N_{b,t} = -165.32$ kN $\leq$ 0 **no verification**

Gk 20: $F_{Rd} = F_{c,v,Rd} = 740.8$ kN, $F_{Ed} = N_{b,c} \cdot \cos(\alpha_v) = 598.29$ kN

$F_{Ed} = 598.3$ kN $<$ $F_{Rd} = 740.8$ kN $\Rightarrow$ $U = 0.808$ $<$ 1 ok

utilization partial internal forces and moments $U_{Gk} = 0.888$ $<$ 1 ok

2.4.2. verification of welds at beam section

weld 1: beam flange in tension outer

weld 8: beam flange in compression outer

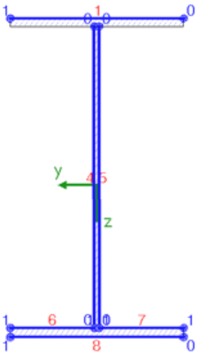
welds 2,3: beam flange in tension inner

welds 4,5: beam web double-sided

welds 6,7: beam flange in compression inner

weld 1: weld thickness a = 13.0 mm $> a_{max} = 0.7 \cdot t_{min} = 9.1$ mm !!

calculation section:



weld 1: $a_w = 13.0$ mm $l_w = 280.0$ mm
weld 4: $a_w = 4.0$ mm $l_w = 487.9$ mm
weld 5: siehe weld 4
weld 6: $a_w = 5.0$ mm $l_w = 136.0$ mm
weld 7: siehe weld 6
weld 8: $a_w = 5.0$ mm $l_w = 280.0$ mm

design values refering to centroid of the section:
$N_{Ed} = -762.97$ kN, $M_{y,Ed} = -100.00$ kNm, $V_{z,Ed} = 240.56$ kN

cross-sectional properties refering to centroid of the line cross-section:
$\Sigma A_w = 103.17$ cm², $A_{w,z} = 39.03$ cm², $\Sigma l_w = 180.8$ cm
$I_{w,y} = 48778.31$ cm⁴, $I_{w,z} = 4222.77$ cm⁴, $\Delta z_w = -35.8$ mm

distribution of internal forces and moments:
weld 1: $N_w = -95.47$ kN
weld 4: $N_w = -153.82$ kN $M_{y,w} = -7.94$ kNm
weld 5: siehe weld 4
weld 6: $N_w = -87.60$ kN
weld 7: siehe weld 6
weld 8: $N_w = -184.67$ kN
from conventionel distribution of shear force: $V_{z,w} = 240.56$ kN

stresses in weld edges:
weld 1, pt. 0: $\sigma_{w,x} = -26.13$ N/mm²
weld 4, pt. 0: $\sigma_{w,x} = -28.80$ N/mm² $\tau_{w,z} = 61.63$ N/mm²
pt. 1: $\sigma_{w,x} = -128.83$ N/mm² $\tau_{w,z} = 61.63$ N/mm²
weld 5, pt. 0: siehe weld 4
pt. 1: siehe weld 4
weld 6, pt. 0: $\sigma_{w,x} = -128.83$ N/mm²
weld 7, pt. 0: siehe weld 6
pt. 1: siehe weld 6
weld 8, pt. 0: $\sigma_{w,x} = -131.90$ N/mm²

verifications in weld edges:
verification of weld 1, pt. 0:
stresses on the design area of the weld ($\alpha = 45°$, $\sigma_w = \sigma_{w,x}$):
$\sigma_s = \sigma_w \cdot \cos(\alpha) = -18.5$ N/mm²
$\tau_s = \sigma_w \cdot \sin(\alpha) = -18.5$ N/mm²
$\sigma_{1,w,Ed} = (\sigma_s^2 + 3 \cdot (\tau_s^2 + \tau_p^2))^{1/2} = 3.70$ kN/cm²
resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 36.00$ kN/cm²
$\sigma_{1,w,Ed} = 3.70$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm² $\Rightarrow$ U = 0.103 $<$ 1 ok
$\sigma_{2,w,Ed} = |\sigma_s| = 1.85$ kN/cm²
resistance of a weld (req.2): $f_{2w,d} = 0.9 \cdot f_u / \gamma_{M2} = 25.92$ kN/cm²
$\sigma_{2,w,Ed} = 1.85$ kN/cm² $<$ $f_{2w,d} = 25.92$ kN/cm² $\Rightarrow$ U = 0.071 $<$ 1 ok
verification of weld 4, pt. 0:
stresses on the design area of the weld ($\alpha = 45°$, $\sigma_w = \sigma_{w,x}$):
$\sigma_s = \sigma_w \cdot \cos(\alpha) = -20.4$ N/mm²
$\tau_s = \sigma_w \cdot \sin(\alpha) = -20.4$ N/mm²
$\tau_p = \tau_{w,z} = 61.6$ N/mm²
$\sigma_{1,w,Ed} = (\sigma_s^2 + 3 \cdot (\tau_s^2 + \tau_p^2))^{1/2} = 11.43$ kN/cm²
resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 36.00$ kN/cm²
$\sigma_{1,w,Ed} = 11.43$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm² $\Rightarrow$ U = 0.317 $<$ 1 ok
$\sigma_{2,w,Ed} = |\sigma_s| = 2.04$ kN/cm²
resistance of a weld (req.2): $f_{2w,d} = 0.9 \cdot f_u / \gamma_{M2} = 25.92$ kN/cm²
$\sigma_{2,w,Ed} = 2.04$ kN/cm² $<$ $f_{2w,d} = 25.92$ kN/cm² $\Rightarrow$ U = 0.079 $<$ 1 ok
verification of weld 4, pt. 1:
stresses on the design area of the weld ($\alpha = 45°$, $\sigma_w = \sigma_{w,x}$):
$\sigma_s = \sigma_w \cdot \cos(\alpha) = -91.1$ N/mm²
$\tau_s = \sigma_w \cdot \sin(\alpha) = -91.1$ N/mm²
$\tau_p = \tau_{w,z} = 61.6$ N/mm²
$\sigma_{1,w,Ed} = (\sigma_s^2 + 3 \cdot (\tau_s^2 + \tau_p^2))^{1/2} = 21.12$ kN/cm²
resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 36.00$ kN/cm²
$\sigma_{1,w,Ed} = 21.12$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm² $\Rightarrow$ U = 0.587 $<$ 1 ok
$\sigma_{2,w,Ed} = |\sigma_s| = 9.11$ kN/cm²
resistance of a weld (req.2): $f_{2w,d} = 0.9 \cdot f_u / \gamma_{M2} = 25.92$ kN/cm²

$\sigma_{2,w,Ed}$ = 9.11 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² ⇒ U = 0.351 < 1 ok

verification of weld 6, pt. 0:

stresses on the design area of the weld (α = 45°, $\sigma_w = \sigma_{w,x}$):

$\sigma_s = \sigma_w \cdot \cos(\alpha)$ = -91.1 N/mm²

$\tau_s = \sigma_w \cdot \sin(\alpha)$ = -91.1 N/mm²

$\sigma_{1,w,Ed} = (\sigma_s^2 + 3 \cdot (\tau_s^2 + \tau_p^2))^{1/2}$ = 18.22 kN/cm²

resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2})$ = 36.00 kN/cm²

$\sigma_{1,w,Ed}$ = 18.22 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² ⇒ U = 0.506 < 1 ok

$\sigma_{2,w,Ed} = |\sigma_s|$ = 9.11 kN/cm²

resistance of a weld (req.2): $f_{2w,d} = 0.9 \cdot f_u / \gamma_{M2}$ = 25.92 kN/cm²

$\sigma_{2,w,Ed}$ = 9.11 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² ⇒ U = 0.351 < 1 ok

verification of weld 8, pt. 0:

stresses on the design area of the weld (α = 45°, $\sigma_w = \sigma_{w,x}$):

$\sigma_s = \sigma_w \cdot \cos(\alpha)$ = -93.3 N/mm²

$\tau_s = \sigma_w \cdot \sin(\alpha)$ = -93.3 N/mm²

$\sigma_{1,w,Ed} = (\sigma_s^2 + 3 \cdot (\tau_s^2 + \tau_p^2))^{1/2}$ = 18.65 kN/cm²

resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2})$ = 36.00 kN/cm²

$\sigma_{1,w,Ed}$ = 18.65 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² ⇒ U = 0.518 < 1 ok

$\sigma_{2,w,Ed} = |\sigma_s|$ = 9.33 kN/cm²

resistance of a weld (req.2): $f_{2w,d} = 0.9 \cdot f_u / \gamma_{M2}$ = 25.92 kN/cm²

$\sigma_{2,w,Ed}$ = 9.33 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² ⇒ U = 0.360 < 1 ok

Result:

weld 4, pt. 1: $\sigma_{w,x}$ = -128.83 N/mm² $\tau_{w,z}$ = 61.63 N/mm²

Max: $\sigma_{1,w,Ed}$ = 21.12 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm²,

$\sigma_{2,w,Ed}$ = 9.11 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² ⇒ U_w = 0.587 < 1 ok

2.4.3. verification of web stiffeners / tension plate

design values

$N_{R,t} = (-N_d \cdot z_{bu} + M_d + V_d \cdot \Delta s) / (z \cdot \cos(\alpha_b))$ = -162.37 kN, $z = z_b - \Delta s \cdot \tan(\alpha_b)$ = 501.4 mm, z_{bu} = 239.4 mm, $\Delta s = t_{fc}/2$ = 6.5 mm < 0

$N_{R,c} = (N_d \cdot z_{bo} + M_d + V_d \cdot \Delta s) / z$ = 601.40 kN, $z = z_b$ = 501.9 mm, z_{bo} = 262.5 mm, $\Delta s = t_{fc}/2$ = 6.5 mm

$N_{R,b} = N'_{b,c} \cdot \sin(\Delta\alpha_v)/\cos(\alpha_v)$ = 291.96 kN

column

compression stiffener

$F_{c,Ed} = N_{R,c}$ = 601.40 kN

verification of the welds with the directional method.

dimensions, lever arms, forces per rib

$b_R = b_{st}$ = 136.0 mm, $b_1 = b_R - r_R$ = 100.0 mm, $e_F = b_R - 0.5 \cdot b_1$ = 86.0 mm with r_R = 36.0 mm

$l_R = l_{st}$ = 244.0 mm, $l_1 = l_R - 2 \cdot r_R$ = 172.0 mm, $e_H = l_R$ = 244.0 mm, t_R = 13.0 mm

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 240.6 kN, $H = F \cdot e_F/e_H$ = 84.8 kN

assumption: stiffeners do not buckle (verification method 'elastic-elastic' ⇒ max. section class 3)

c/t-ratio c/t = 10.46 ≤ 33.00 = 33·ε, $\varepsilon = (235/f_y)^{1/2}$ = 1.00 ⇒ section class 1 ≤ 3 ⇒ assumption succeeded !!

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0}$ = 305.50 kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 281.8 kN

F_{Ed} = 281.8 kN < F_{Rd} = 305.5 kN ⇒ U = 0.923 < 1 ok

cross-section at web

shear resistance V_{Rd} = 430.37 kN

design value: $F_{Ed} = F$ = 240.6 kN

F_{Ed} = 240.6 kN < F_{Rd} = 430.4 kN ⇒ U = 0.559 < 1 ok

flange welds

fillet weld with a = 5.0 mm

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 12.03 kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 4.24 kN/cm

0% decrease of stress by pressure contact

stresses on the design area of the weld: σ_s = 24.06 kN/cm² τ_p = 8.48 kN/cm²

$\sigma_{1,w,Ed} = (\sigma_s^2 + 3 \cdot (\tau_s^2 + \tau_p^2))^{1/2}$ = 28.18 kN/cm²

resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2})$ = 36.00 kN/cm²

$\sigma_{1,w,Ed}$ = 28.18 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² ⇒ U = 0.783 < 1 ok

$\sigma_{2,w,Ed} = |\sigma_s|$ = 24.06 kN/cm²

resistance of a weld (req.2): $f_{2w,d} = 0.9 \cdot f_u / \gamma_{M2}$ = 25.92 kN/cm²

$\sigma_{2,w,Ed}$ = 24.06 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² ⇒ U = 0.928 < 1 ok

web welds

fillet weld with a = 4.0 mm

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 6.99 kN/cm

0% decrease of stress by pressure contact

stresses on the design area of the weld: τ_p = 17.48 kN/cm²

$\sigma_{1,w,Ed} = (\sigma_s^2 + 3 \cdot (\tau_s^2 + \tau_p^2))^{1/2}$ = 30.28 kN/cm²

resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2})$ = 36.00 kN/cm²

$\sigma_{1,w,Ed}$ = 30.28 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² ⇒ U = 0.841 < 1 ok

beam

$F_{c,Ed} = N_{R,b}$ = 291.96 kN

verification of the welds with the directional method.
dimensions, lever arms, forces per rib

b_R = 136.0 mm (maximum width), $b_1 = b_R - r_R$ = 136.0 mm, $e_F = b_R - 0.5 \cdot b_1$ = 68.0 mm with r_R = 0.0 mm
$l_R = h - 2 \cdot t_f$ = 244.0 mm, $l_1 = l_R - 2 \cdot r_R$ = 244.0 mm, $e_H = l_R$ = 244.0 mm, t_R = 13.0 mm
$F = 0.5 \cdot F_{c,Ed}$ = 146.0 kN, $H = F \cdot e_F / e_H$ = 40.7 kN
assumption: stiffeners do not buckle (verification method 'elastic-elastic' $\Rightarrow$ max. section class 3)
c/t-ratio $c/t = 10.46 \leq 33.00 = 33 \cdot \varepsilon$, $\varepsilon = (235/f_y)^{1/2} = 1.00$ $\Rightarrow$ section class $1 \leq 3$ $\Rightarrow$ assumption succeeded !!

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0}$ = 415.48 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 162.1 kN
F_{Ed} = 162.1 kN < F_{Rd} = 415.5 kN $\Rightarrow$ U = 0.390 < 1 ok

cross-section at web

shear resistance V_{Rd} = 430.37 kN
design value: $F_{Ed} = F$ = 146.0 kN
F_{Ed} = 146.0 kN < F_{Rd} = 430.4 kN $\Rightarrow$ U = 0.339 < 1 ok

flange welds

fillet weld with a = 5.0 mm
design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1)$ = 5.37 kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1)$ = 1.50 kN/cm
0% decrease of stress by pressure contact
stresses on the design area of the weld: σ_s = 10.73 kN/cm² τ_p = 2.99 kN/cm²
$\sigma_{1,w,Ed} = (\sigma_s^2 + 3 \cdot (\tau_s^2 + \tau_p^2))^{1/2}$ = 11.92 kN/cm²
resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2})$ = 36.00 kN/cm²
$\sigma_{1,w,Ed}$ = 11.92 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² $\Rightarrow$ U = 0.331 < 1 ok
$\sigma_{2,w,Ed} = |\sigma_s|$ = 10.73 kN/cm²
resistance of a weld (req.2): $f_{2w,d} = 0.9 \cdot f_u / \gamma_{M2}$ = 25.92 kN/cm²
$\sigma_{2,w,Ed}$ = 10.73 kN/cm² < $f_{2w,d}$ = 25.92 kN/cm² $\Rightarrow$ U = 0.414 < 1 ok

web welds

fillet weld with a = 4.0 mm
design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1)$ = 2.99 kN/cm
0% decrease of stress by pressure contact
stresses on the design area of the weld: τ_p = 7.48 kN/cm²
$\sigma_{1,w,Ed} = (\sigma_s^2 + 3 \cdot (\tau_s^2 + \tau_p^2))^{1/2}$ = 12.95 kN/cm²
resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2})$ = 36.00 kN/cm²
$\sigma_{1,w,Ed}$ = 12.95 kN/cm² < $f_{1w,d}$ = 36.00 kN/cm² $\Rightarrow$ U = 0.360 < 1 ok

2.4.4. elastic verification of the shear area

column web as an ideal shear area

requirements concerning stiffeners: s. verification of web stiffeners
requirements concerning shear area: s. verification of buckling resistance
internal forces and moments at web (sign definition of statics):
$N_3 = -N_c$ = -240.56 kN, $M_3 = -M_c$ = 72.39 kNm, $V_3 = -V_c$ = -762.97 kN
$N_4 = -N_d$ = -762.97 kN, $M_4 = -(M_d + (V_d - N_d \cdot \tan(\alpha)) \cdot t_{fc}/2)$ = -100.00 kNm, $V_4 = V_d$ = 240.56 kN
dimensions of the joint area: l_b = 257.0 mm, l_t = 258.0 mm, l_l = 478.9 mm, l_r = 501.4 mm
joint forces at shear area:
F_{b4} = -580.94 kN, F_{t4} = -182.03 kN, V_4 = 240.56 kN
F_{r3} = 161.39 kN, F_{l3} = -401.95 kN, V_{b3} = -762.97 kN
internal forces and moments of the edge stiffeners:
N_b = -580.94 kN, N_t = -182.73 kN, N_l = -401.95 kN, N_r = 177.31 kN
forces in the shear area:
T_b = 182.03 kN, T_t = 182.73 kN, T_l = 401.95 kN, T_r = 417.88 kN
dimensions of the shear area (at periphery of stiffeners):
h_b = 244.0 mm, h_t = 244.9 mm, h_l = 465.9 mm, h_r = 488.3 mm
stresses within the shear area:
τ_b = 93.3 N/mm², τ_t = 93.3 N/mm², τ_l = 107.9 N/mm², τ_r = 107.0 N/mm²
verification of the shear area:
max τ_{Ed} = 107.9 N/mm² < τ_{Rd} = 135.7 N/mm² $\Rightarrow$ U = 0.795 < 1 ok

beam web as an ideal shear area

internal forces and moments at shear area (sign definition of statics):
$N_4 = -N_d$ = -762.97 kN, $M_4 = -(M_d + V_d \cdot t_{fc}/2)$ = -100.00 kNm, $V_4 = V_d$ = 240.56 kN
$N_5 = -N_{v,d}$ = -781.04 kN, $M_5 = -M_{v,d}$ = -100.00 kNm, $V_5 = V_{v,d}$ = 173.15 kN
dimensions of the joint area: l_b = 584.9 mm, l_t = 486.0 mm, l_l = 512.6 mm, l_r = 257.0 mm
joint forces at shear area:
F_{b4} = -576.56 kN, F_{t4} = -186.41 kN, F_{b5} = -779.62 kN, F_{t5} = -1.41 kN
internal forces and moments of the edge stiffeners:
N_b = 194.46 kN, N_t = -185.71 kN, N_l = 349.19 kN, N_r = -363.54 kN
forces in the shear area:
T_b = -194.46 kN, T_t = -185.71 kN, T_l = -108.62 kN, T_r = -190.39 kN
dimensions of the shear area (at periphery of stiffeners):
h_b = 577.4 mm, h_t = 479.4 mm, h_l = 487.9 mm, h_r = 244.0 mm
stresses within the shear area:
τ_b = 42.1 N/mm², τ_t = 48.4 N/mm², τ_l = 27.8 N/mm², τ_r = 97.5 N/mm²

verification of the shear area:
max τ_{Ed} = 97.5 N/mm² < τ_{Rd} = 135.7 N/mm² ⇒ U = 0.719 < 1 ok

2.4.5. verification of buckling resistance

column

requirements concerning stiffeners: s. verification of web stiffeners
verification method 'elastic-elastic' ⇒ valid section class 3
plate buckling: section class of the section 1 ≤ 3 ok
shear buckling: h_p/t_p = 30.50 ≤ 72/(η·ε) = 60.00, h_p = 244.0 mm, t_p = 8.0 mm ok

beam

requirements concerning stiffeners: s. verification of web stiffeners
verification method 'elastic-elastic' ⇒ valid section class 3
plate buckling: section class of the section 4 > 3 ⇒ particular verification is required !!
shear buckling: h_p/t_p = 60.99 > 72/(η·ε) = 60.00, h_p = 487.9 mm, t_p = 8.0 mm ⇒ particular verification is required !!

method of effective cross-sectional areas

assumption: shear distortions will be ignored.

<u>plate buckling</u>

flange bottom:

pressure plate: a = 576.0 mm, b = 136.0 mm, t = 15.0 mm, σ_1 = 106.8 N/mm², σ_2 = 106.8 N/mm²
one-side supported plated panel: stress ratio $\Psi = \sigma_2/\sigma_1$ = 1.000 ⇒ buckling factor k_σ = 0.43
critical buckling stress $\sigma_{cr,p} = k_\sigma \cdot \sigma_E$ = 997.4 N/mm² with $\sigma_E = (\pi^2 \cdot E \cdot t^2)/(12 \cdot (1-\mu) \cdot b^2)$ = 2312.3 N/mm²
buckling slenderness ratio $\lambda_p = (f_y/\sigma_{cr,p})^{1/2}$ = 0.485
reduction factor for λ_p < 0.748: ρ = 1
effective width $b_{c,eff} = \rho \cdot b$ = 136.0 mm

flange top:

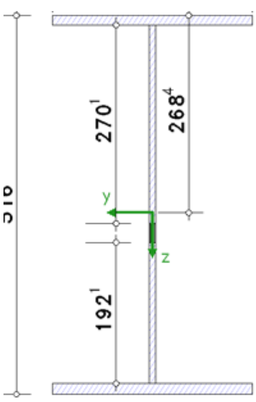
pressure plate: a = 576.0 mm, b = 136.0 mm, t = 13.0 mm, σ_1 = 18.9 N/mm², σ_2 = 18.9 N/mm²
one-side supported plated panel: stress ratio $\Psi = \sigma_2/\sigma_1$ = 1.000 ⇒ buckling factor k_σ = 0.43
critical buckling stress $\sigma_{cr,p} = k_\sigma \cdot \sigma_E$ = 753.8 N/mm² with $\sigma_E = (\pi^2 \cdot E \cdot t^2)/(12 \cdot (1-\mu) \cdot b^2)$ = 1747.5 N/mm²
buckling slenderness ratio $\lambda_p = (f_y/\sigma_{cr,p})^{1/2}$ = 0.558
reduction factor for λ_p < 0.748: ρ = 1
effective width $b_{c,eff} = \rho \cdot b$ = 136.0 mm

web:

pressure plate: a = 576.0 mm, b = 487.9 mm, t = 8.0 mm, σ_1 = 107.8 N/mm², σ_2 = 20.2 N/mm²
two-side supported plated panel: stress ratio $\Psi = \sigma_2/\sigma_1$ = 0.188 ⇒ buckling factor k_σ = 6.63
critical buckling stress $\sigma_{cr,p} = k_\sigma \cdot \sigma_E$ = 338.1 N/mm² with $\sigma_E = (\pi^2 \cdot E \cdot t^2)/(12 \cdot (1-\mu) \cdot b^2)$ = 51.0 N/mm²
buckling slenderness ratio $\lambda_p = (f_y/\sigma_{cr,p})^{1/2}$ = 0.834
reduction factor for $\lambda_p > 0.5 + (0.085-0.055 \cdot \Psi)^{1/2}$ = 0.773: $\rho = (\lambda_p - 0.055 \cdot (3+\Psi))/\lambda_p^2$ = 0.947 ≤ 1
effective width $b_{c,eff} = \rho \cdot b$ = 462.2 mm (b_{e1} = 192.1 mm, b_{e2} = 270.1 mm)
web buckling owing to the flange:
h_w/t_w = 60.99 < $(k \cdot E)/(f_y \cdot (A_w/A_{fc})^{1/2})$ = 262.12 ok with k = 0.30 for section class 4,
h_w = 487.9 mm, A_w = 39.03 cm², A_{fc} = 40.83 cm²

limit loads refering to the reduced cross-section:

centroid distance from top $z_{s,eff}$ = 268.4 mm
cross-sectional area A_{eff} = 113.30 cm²
second moment of area $I_{y,eff}$ = 55661.10 cm⁴
elastic section modulus $W_{y,eff}$ = 2248.73 cm³
load capacities $N_{Rd} = (f_y \cdot A_{eff}) / \gamma_{M0}$ = 2662.49 kN
$M_{Rd} = (f_y \cdot W_{eff}) / \gamma_{M0}$ = 528.45 kNm



verification: $N_{Ed}/N_{Rd}+M_{Ed}/M_{Rd}$ = 0.287 + 0.189 = 0.476 < 1 ok

<u>shear buckling</u>

contribution of the web:

buckling factor of shear for a/h_w = 1.18 > 1: $k_\tau = 5.34 + 4/(a/h_w)^2$ = 8.21, a = 576.0 mm, h_w = 487.9 mm
critical buckling stress of shear $\tau_{cr,p} = k_\tau \cdot \sigma_E$ = 418.9 N/mm² with $\sigma_E = (\pi^2 \cdot E \cdot t^2)/(12 \cdot (1-\mu) \cdot b^2)$ = 51.0 N/mm²
modified slenderness $\lambda_w = 0.76 \cdot (f_{yw}/\tau_{cr,p})$ = 0.569, f_{yw} = 235.0 N/mm²
reduction factor for $\lambda_w < 0.83/\eta$ = 0.692: $\chi_w = 1.200 = \eta$
resistance $V_{bw,Rd} = (\chi_w \cdot f_{yw} \cdot h_w \cdot t_w) / (3^{1/2} \cdot \gamma_{M1})$ = 577.72 kN

contribution of the flanges:

resisting moment solely from the effective flange areas $M_{f,Rd} = M_{f,k}/\gamma_{M0}$ = 430.99 kNm
with $M_{f,k} = \min(A_{f1},A_{f2}) \cdot (h_w+(t_{f1}+t_{f2})/2) \cdot f_y$ = 430.99 kNm, $A_{f1} = b_{f1} \cdot t_{f1}$ = 36.54 cm², $A_{f2} = b_{f2} \cdot t_{f2}$ = 42.03 cm²
N_{Ed} > 0: $f_{N,f} = 1 - N_{Ed}/((A_{f1}+A_{f2}) \cdot f_y/\gamma_{M0})$ = 0.587 < 1 ⇒ $M_{f,Rd} = M_{f,Rd} \cdot f_{N,f}$ = 252.90 kNm
M_{Ed} = 100.00 kNm < $M_{f,Rd}$ = 252.90 kNm:
flange 1: t_f = 13.0 mm, b_f = 280.0 mm ≤ $2 \cdot 15 \cdot t_f \cdot \varepsilon$ = 399.5 mm, a = 576.0 mm
$c = a \cdot (0.25+(1.6 \cdot b_f \cdot t_f^2 \cdot f_{yf})/(t_w \cdot h_w^2 \cdot f_{yw}))$ = 167.1 mm
flange 2: t_f = 15.0 mm, b_f = 280.0 mm ≤ $2 \cdot 15 \cdot t_f \cdot \varepsilon$ = 458.3 mm, a = 576.0 mm
$c = a \cdot (0.25+(1.6 \cdot b_f \cdot t_f^2 \cdot f_{yf})/(t_w \cdot h_w^2 \cdot f_{yw}))$ = 174.5 mm

resistance $V_{bf,Rd} = (b_f \cdot t_f^2 \cdot f_{yf})/(c \cdot \gamma_{M1}) \cdot (1-(M_{Ed}/M_{f,Rd})^2) = 51.43$ kN (flange 1 maßgeb.)
design value of resistance $V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} = 629.2$ kN $>$ lim $V_{b,Rd}$ $\Rightarrow$ $V_{b,Rd} = 577.7$ kN
with lim $V_{b,Rd} = (\eta \cdot f_{yw} \cdot h_w \cdot t_w)/(3^{1/2} \cdot \gamma_{M1}) = 577.72$ kN
verification: $V_{Ed}/V_{b,Rd} = 0.416 < 1$ ok
interaction not required, da utilization due to shear buckling of the web $\eta_3 = V_{Ed}/V_{bw,Rd} = 0.416 \leq 0.5$

2.4.6. verification result

maximum utilization: max $U = 0.928 < 1$ ok

3. final result

maximum utilization: max $U = 0.928 < 1$ ok

verification succeeded