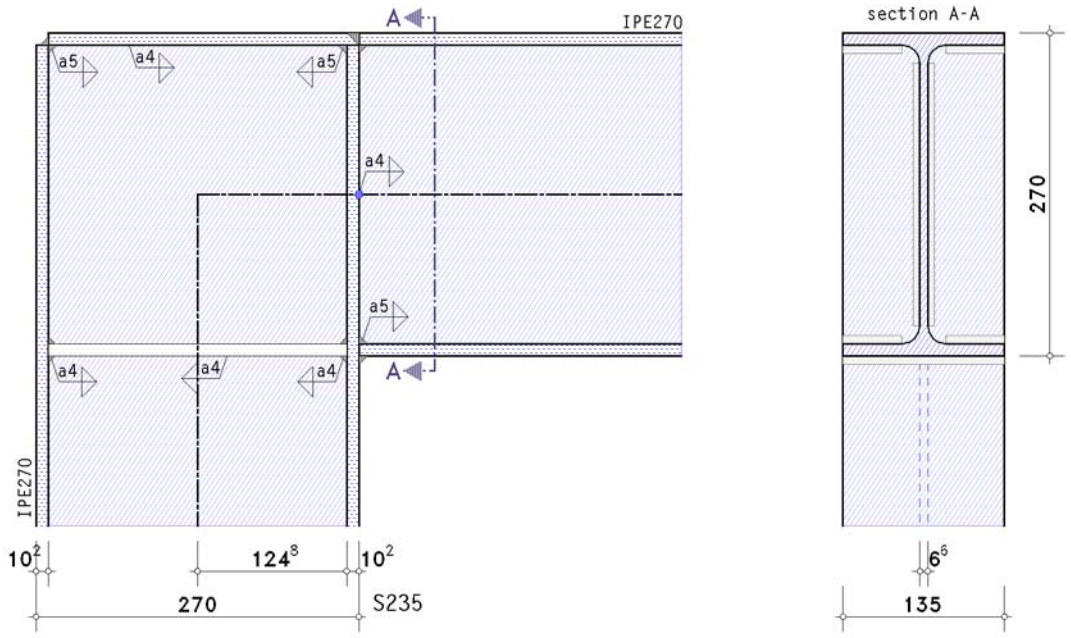
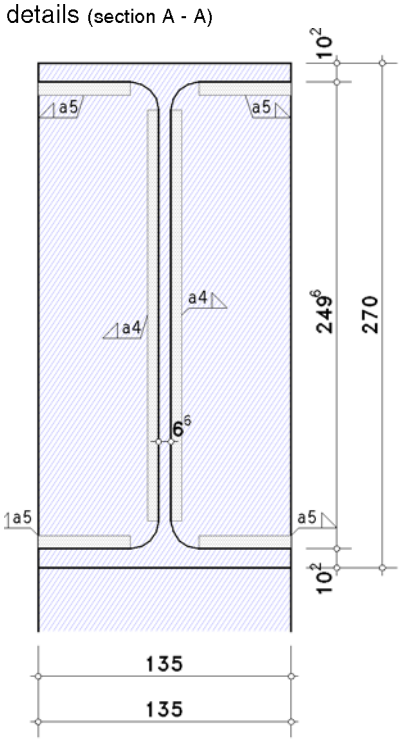
POS. 5: BEISPIEL WELDED

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

1. input report



details (section A - A)

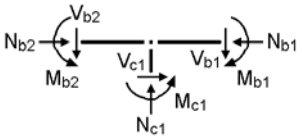


steel grade
steel grade S235
column parameters
section IPE270
reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 259.8 mm):
thickness t_{st} = 10.0 mm, width b_{st} = 64.2 mm, length l_{st} = 249.6 mm
recess at stiffeners c_{st} = 22.5 mm
welds $a_{st,f}$ = 4.0 mm, $a_{st,w}$ = 4.0 mm
beam parameters
section IPE270
verification parameters
welded connection:
tension plate: thickness t_z = 10.2 mm, width b_z = 135.0 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
beam web: fillet weld, weld thickness a = 4.0 mm
beam flange bottom: fillet weld, weld thickness a = 5.0 mm

internal forces and moments at the joint periphery perpendicular to the connection plane

Lk 1: M_d = 60.00 kNm V_d = 120.00 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

check of data

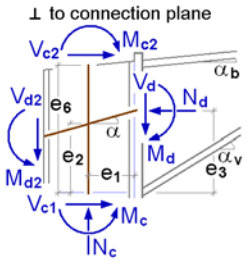
ok

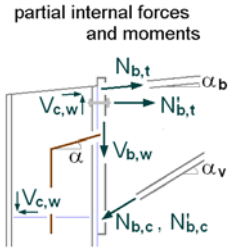
notes

plated structures- and shear buckling is not investigated.

2. Lk 1

2.1. design values





sign definition of 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 = 135.0 mm, e_3 = 129.9 mm, e_2 = 129.9 mm, e_6 = 259.8 mm

internal forces and moments perpendicular to the connection planes

periphery beam
M_d = 60.00 kNm, V_d = 120.00 kN
periphery column (bottom, calculated)
N_c = 120.00 kN, M_c = 76.20 kNm

partial internal forces and moments

$N_{b,t} = -N_d \cdot z_{bu}/z_b + M_d/z_b$ = 230.95 kN, z_b = 259.8 mm, z_{bu} = 129.9 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M_d/z_b$ = 230.95 kN, z_b = 259.8 mm, z_{bo} = 129.9 mm
$V_{b,w} = V_d$ = 120.00 kN

2.2. resistance of cross-section

column bottom

plastic cross-sectional check for N = -120.00 kN, M_y = -76.20 kNm
elastic stresses: max σ_x = 15.15 kN/cm², min σ_x = -20.38 kN/cm², max τ = 0.00 kN/cm²
valid normal/shear stress: zul σ_{Rd} = 23.50 kN/cm², zul τ_{Rd} = 13.57 kN/cm²
top flange: resistance forces $N_{max,O}$ = 323.59 kN, $N_{min,O}$ = -323.59 kN
bottom flange: resistance forces $N_{max,U}$ = 323.59 kN, $N_{min,U}$ = -323.59 kN
web: resistance forces $N_{max,S}$ = 402.95 kN, $N_{min,S}$ = -402.95 kN
main bending: axial force N = -120.00 kN, resistance forces N_{max} = 1050.14 kN, N_{min} = -1050.14 kN ⇒ U_N = 0.114
moment M_y = -76.20 kNm, resistance moments $M_{y,max}$ = 107.92 kNm, $M_{y,min}$ = -107.92 kNm ⇒ U_{My} = 0.706
total (possibly due to load increase): max U = 0.722 < 1 ok
utilizations: resistance U_σ = 0.722 < 1 ok, c/t-ratio $U_{c/t}$ = 0.321 < 1 ok

beam

plastic cross-sectional check for M_y = -60.00 kNm, V_z = 120.00 kN
elastic stresses: max σ_x = 13.99 kN/cm², min σ_x = -13.99 kN/cm², max τ = 7.60 kN/cm²
valid normal/shear stress: zul σ_{Rd} = 23.50 kN/cm², zul τ_{Rd} = 13.57 kN/cm²
top flange: resistance forces $N_{max,O}$ = 323.59 kN, $N_{min,O}$ = -323.59 kN
bottom flange: resistance forces $N_{max,U}$ = 323.59 kN, $N_{min,U}$ = -323.59 kN
web: shear force V_S = 120.00 kN, shear stress τ_S = 7.00 kN/cm² ⇒ $U_{\tau,S}$ = 0.516
resistance forces $N_{max,S}$ = 345.21 kN, $N_{min,S}$ = -345.21 kN
main bending: moment M_y = -60.00 kNm, resistance moments $M_{y,max}$ = 106.49 kNm, $M_{y,min}$ = -106.49 kNm ⇒ U_{My} = 0.563
total (possibly due to load increase): max U = 0.613 < 1 ok
utilizations: resistance U_σ = 0.613 < 1 ok, c/t-ratio $U_{c/t}$ = 0.265 < 1 ok

welded beam connection: basic components: 1, 2, 3, 7

2.3. connection capacity

transformation parameter: $\beta_j = 1.000$

2.3.1. moment resistance

distance between tension force and centre of compression: $z = 259.8$ mm

resistance (tension)
decisive basic components: 3
$F_{t,Rd} = 317.3$ kN

deductions analogical EC 3-1-8, 6.2.7.2(7)
decisive basic components: 1, 2, 7
Gk 1: $F_{Rd} = 317.3$ kN $>$ $V_{wp,Rd}/\beta_j = 282.5$ kN $\Rightarrow$ $F_{Rd} = 282.5$ kN
Gk 2: $F_{Rd} = 282.5$ kN $<$ $F_{c,w,Rd} = 288.4$ kN $\Rightarrow$ $F_{Rd} = 282.5$ kN
Gk 7: $F_{Rd} = 282.5$ kN $<$ $F_{c,f,Rd} = 437.8$ kN $\Rightarrow$ $F_{Rd} = 282.5$ kN

resistance (finally)
$F_{Rd} = 282.5$ kN

resistance of flanges (compression)
$\Sigma F_{c,Rd}^{*} = 565.0$ kN

moment resistance regarding the centre of compression
$M_{j,Rd} = F_{Rd} \cdot z = 73.4$ kNm
tension resistance
$N_{j,t,Rd} = F_{t,Rd} = 317.3$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^{*} = 565.0$ kN

2.3.2. shear resistance

shear resistance of column web
decisive basic component: 1
$V_{wp,Rd}/\beta_j = 282.5$ kN

2.3.3. total

$M_{j,Rd} = 73.4$ kNm $N_{j,t,Rd} = 317.3$ kN $N_{j,c,Rd} = 565.0$ kN $V_{wp,Rd}/\beta_j = 282.5$ kN

2.4. verifications

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

internal moment: $M_{Ed} = M_d = 60.00$ kNm
perpend. to connection plane
shear force: $V_{Ed} = |V_d| = 120.00$ kN
parallel to connection plane
shear force: $V_{c,w,Ed} = (M_{d1,w}-M_{d2,w})/z - V_c/2 = 235.66$ kN, $M_{d1,w} = 61.2$ kNm, $M_{d2,w} = 0.0$ kNm, $z = 259.8$ mm
in column web

moment resistance
$M_{Ed}/M_{j,Rd} = 0.818 < 1$ ok
shear resistance of column web
$V_{c,w,Ed}/(V_{wp,Rd}/\beta_j) = 0.834 < 1$ ok

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

shear force in column web:
$V_{c,w,Ed} = (M_{d1,w}-M_{d2,w})/z - V_c/2 = 235.66$ kN, $M_{d1,w} = 61.2$ kNm, $M_{d2,w} = 0.0$ kNm
$z = 259.8$ mm

Gk 1: $F_{Rd} = V_{wp,Rd}/\beta_j = 282.5$ kN, $F_{Ed} = |V_{c,w,Ed}| = 235.66$ kN
$F_{Ed} = 235.7$ kN $<$ $F_{Rd} = 282.5$ kN $\Rightarrow$ $U = 0.834 < 1$ ok
Gk 2: $F_{Rd} = F_{c,w,Rd} = 288.4$ kN, $F_{Ed} = N_{b,c} = 230.95$ kN
$F_{Ed} = 230.9$ kN $<$ $F_{Rd} = 288.4$ kN $\Rightarrow$ $U = 0.801 < 1$ ok
Gk 3: $F_{Rd} = F_{t,wc,Rd} = 317.3$ kN, $F_{Ed} = N_{b,t} = 230.95$ kN
$F_{Ed} = 230.9$ kN $<$ $F_{Rd} = 317.3$ kN $\Rightarrow$ $U = 0.728 < 1$ ok
Gk 7: flange: $F_{Rd} = F_{c,f,Rd} = 437.8$ kN, $F_{Ed} = N_{b,c} = 230.95$ kN
$F_{Ed} = 230.9$ kN $<$ $F_{Rd} = 437.8$ kN $\Rightarrow$ $U = 0.528 < 1$ ok

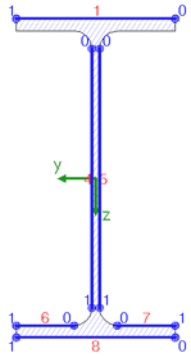
utilization partial internal forces and moments $U_{Gk} = 0.834 < 1$ ok

2.4.3. 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 = 10.2 mm > $a_{max} = 0.7 \cdot t_{min}$ = 7.1 mm !!

calculation section:



weld 1:	a_w = 10.2 mm	l_w = 135.0 mm
weld 4:	a_w = 4.0 mm	l_w = 219.6 mm
weld 5:	siehe weld 4	
weld 6:	a_w = 5.0 mm	l_w = 49.2 mm
weld 7:	siehe weld 6	
weld 8:	a_w = 5.0 mm	l_w = 135.0 mm

design values refering to centroid of the section:

$M_{y,Ed}$ = -60.00 kNm, $V_{z,Ed}$ = 120.00 kN

cross-sectional properties refering to centroid of the line cross-section:

ΣA_w = 43.01 cm², $A_{w,z}$ = 17.57 cm², Σl_w = 80.8 cm
$I_{w,y}$ = 5186.17 cm⁴, $I_{w,z}$ = 414.03 cm⁴, Δz_w = -7.8 mm

stresses in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x}$ = 147.21 N/mm²	
weld 4,	pt. 0:	$\sigma_{w,x}$ = 118.05 N/mm²	$\tau_{w,z}$ = 68.31 N/mm²
	pt. 1:	$\sigma_{w,x}$ = -136.01 N/mm²	$\tau_{w,z}$ = 68.31 N/mm²
weld 5,	pt. 0:	siehe weld 4	
	pt. 1:	siehe weld 4	
weld 6,	pt. 0:	$\sigma_{w,x}$ = -153.36 N/mm²	
weld 7,	pt. 0:	siehe weld 6	
	pt. 1:	siehe weld 6	
weld 8,	pt. 0:	$\sigma_{w,x}$ = -165.16 N/mm²	

verifications in weld edges:

verification of weld 1, pt. 0:

stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 147.2 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 15.02 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 21.20 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 10.2 mm
$F_{w,Ed}$ = 15.02 kN/cm < $F_{w,Rd}$ = 21.20 kN/cm ⇒ U = 0.708 < 1 ok

verification of weld 4, pt. 0:

stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = (\sigma_{w,x}^2 + \tau_{w,z}^2)^{1/2}$ = 136.4 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 5.46 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 8.31 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 4.0 mm
$F_{w,Ed}$ = 5.46 kN/cm < $F_{w,Rd}$ = 8.31 kN/cm ⇒ U = 0.656 < 1 ok

verification of weld 4, pt. 1:

stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = (\sigma_{w,x}^2 + \tau_{w,z}^2)^{1/2}$ = 152.2 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 6.09 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 8.31 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 4.0 mm
$F_{w,Ed}$ = 6.09 kN/cm < $F_{w,Rd}$ = 8.31 kN/cm ⇒ U = 0.732 < 1 ok

verification of weld 6, pt. 0:

stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 153.4 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 7.67 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 10.39 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 5.0 mm
$F_{w,Ed}$ = 7.67 kN/cm < $F_{w,Rd}$ = 10.39 kN/cm ⇒ U = 0.738 < 1 ok

verification of weld 8, pt. 0:

stresses on the design area of the weld (α = 45°):
$\sigma_{w,Ed} = \sigma_{w,x}$ = 165.2 N/mm²
resultant weld force $F_{w,Ed} = \sigma_{w,Ed} \cdot a$ = 8.26 kN/cm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a$ = 10.39 kN/cm, $f_{vw,d}$ = 207.85 N/mm², a = 5.0 mm
$F_{w,Ed}$ = 8.26 kN/cm < $F_{w,Rd}$ = 10.39 kN/cm ⇒ U = 0.795 < 1 ok

Result:

weld 8, pt. 0: $\sigma_{w,x}$ = -165.16 N/mm²
Max: $F_{w,Ed}$ = 8.26 kN/cm < $F_{w,Rd}$ = 10.39 kN/cm ⇒ U_w = 0.795 < 1 ok

2.4.4. verification of web stiffeners / tension plate

$N_{R,t} = (-N_d \cdot z_{bu} + M_d + V_d \cdot \Delta s) / z = 233.30$ kN, $z = z_b = 259.8$ mm, $z_{bu} = 129.9$ mm, $\Delta s = t_{fc}/2 = 5.1$ mm
$N_{R,c} = (N_d \cdot z_{bo} + M_d + V_d \cdot \Delta s) / z = 233.30$ kN, $z = z_b = 259.8$ mm, $z_{bo} = 129.9$ mm, $\Delta s = t_{fc}/2 = 5.1$ mm
compression stiffener
$F_{c,Ed} = N_{R,c} = 233.30$ kN
verification of the welds with the simplified method.
dimensions, lever arms, forces per rib
$b_R = b_{st} = 64.2$ mm, $b_1 = b_R - r_R = 41.7$ mm, $e_F = b_R - 0.5 \cdot b_1 = 43.4$ mm with $r_R = 22.5$ mm
$l_R = l_{st} = 249.6$ mm, $l_1 = l_R - 2 \cdot r_R = 204.6$ mm, $e_H = l_R = 249.6$ mm, $t_R = 10.0$ mm
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 85.0$ kN, $H = F \cdot e_F/e_H = 14.8$ kN
assumption: stiffeners do not buckle (verification method 'elastic-plastic' ⇒ max. section class 2)
c/t-ratio $c/t = 6.42 \le 33.00 = 33 \cdot \varepsilon$, $\varepsilon = (235/f_y)^{1/2} = 1.00$ ⇒ section class $1 \le 2$ ⇒ assumption succeeded !!
cross-section at flange
compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 98.00$ kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 88.8$ kN
$F_{Ed} = 88.8$ kN < $F_{Rd} = 98.0$ kN ⇒ $U = 0.906 < 1$ ok
cross-section at web
shear resistance $V_{Rd} = 338.65$ kN
design value: $F_{Ed} = F = 85.0$ kN
$F_{Ed} = 85.0$ kN < $F_{Rd} = 338.7$ kN ⇒ $U = 0.251 < 1$ ok
flange welds
fillet weld with a = 4.0 mm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 8.31$ kN/cm, $f_{vw,d} = 207.85$ N/mm², $a = 4.0$ mm
design value: $F_{Ed} = (F^2 + H^2)^{1/2} / (2 \cdot b_1) = 10.35$ kN/cm
0% decrease of stress by pressure contact
$F_{Ed} = 10.35$ kN/cm > $F_{Rd} = 8.31$ kN/cm ⇒ $U = 1.245 > 1$ fault !!
web welds
fillet weld with a = 4.0 mm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 8.31$ kN/cm, $f_{vw,d} = 207.85$ N/mm², $a = 4.0$ mm
design value: $F_{Ed} = F / (2 \cdot l_1) = 2.08$ kN/cm
0% decrease of stress by pressure contact
$F_{Ed} = 2.08$ kN/cm < $F_{Rd} = 8.31$ kN/cm ⇒ $U = 0.250 < 1$ ok
tension plate
$F_{t,Ed} = N_{R,t} = 233.30$ kN
verification of the welds with the simplified method.
dimensions
$t_{pk} = 10.2$ mm, $b_{pk} = 135.0$ mm, $l_{pk} = 259.8$ mm
cross-section
resistance of plate in tension:
$N_{pl,Rd} = (A \cdot f_y) / \gamma_{M0} = 323.60$ kN
$N_{u,Rd} = (0.9 \cdot A_{net} \cdot f_u) / \gamma_{M2} = 356.92$ kN
tension resistance $N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = 323.60$ kN
design value: $F_{Ed} = 233.3$ kN
$F_{Ed} = 233.3$ kN < $F_{Rd} = 323.6$ kN ⇒ $U = 0.721 < 1$ ok
weld flange
fillet weld with a = 5.0 mm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 10.39$ kN/cm, $f_{vw,d} = 207.85$ N/mm², $a = 5.0$ mm
$F = F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 170.1$ kN
design value: $F_{Ed} = 0.5 \cdot F / b_z = 6.30$ kN/cm, $b_z = 135.0$ mm
$F_{Ed} = 6.30$ kN/cm < $F_{Rd} = 10.39$ kN/cm ⇒ $U = 0.606 < 1$ ok
web welds
fillet weld with a = 4.0 mm
resistance of a weld: $F_{w,Rd} = f_{vw,d} \cdot a = 8.31$ kN/cm, $f_{vw,d} = 207.85$ N/mm², $a = 4.0$ mm
design value: $F_{Ed} = F / (2 \cdot d_w) = 3.87$ kN/cm, $F = 170.1$ kN, $d_w = 219.6$ mm
$F_{Ed} = 3.87$ kN/cm < $F_{Rd} = 8.31$ kN/cm ⇒ $U = 0.466 < 1$ ok

2.4.5. verification result

maximum utilization: max $U = 1.245 > 1$ fault !!
failure at verification of web stiffeners: U = 1.245

2.5. rotational stiffness

stiffness coefficients
stiffness coefficient of basic component 1:
$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z) = 3.24$ mm, $A_{vc} = 22.14$ cm², $\beta = 1.0$, $z = 259.8$ mm
stiffness coefficient of basic component 2:
$k_2 = \infty$ (stiffened)
stiffness coefficient of basic component 3:
$k_3 = \infty$ (stiffened)
sum of stiffness coefficients $\Sigma(1/k_i) = 1/k_1 + 1/k_2 + 1/k_3 = 0.309$

rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i) = 45897.0$ kNm/rad, $z = 259.8$ mm
internal moments at the connection point reg. centre of compression: $M_{j,Ed} = 60.00$ kNm, $M_{j,Rd} = 73.39$ kNm
$|M_{j,Ed}| = 60.00$ kNm $>$ $2/3\ M_{j,Rd} = 48.9$ kNm $\Rightarrow$ $\mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi} = 1.735$, $\Psi = 2.7$
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu = 26458.0$ kNm/rad
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd} = 0.130°$

3. final result

maximum utilization:	max U = 1.245 > 1 fault !!
minimum rotational stiffness:	min $S_j = 26.5$ MNm/rad, $S_{j,ini} = 45.9$ MNm/rad, $\varphi_j = 0.130°$

resistance not ensured !!