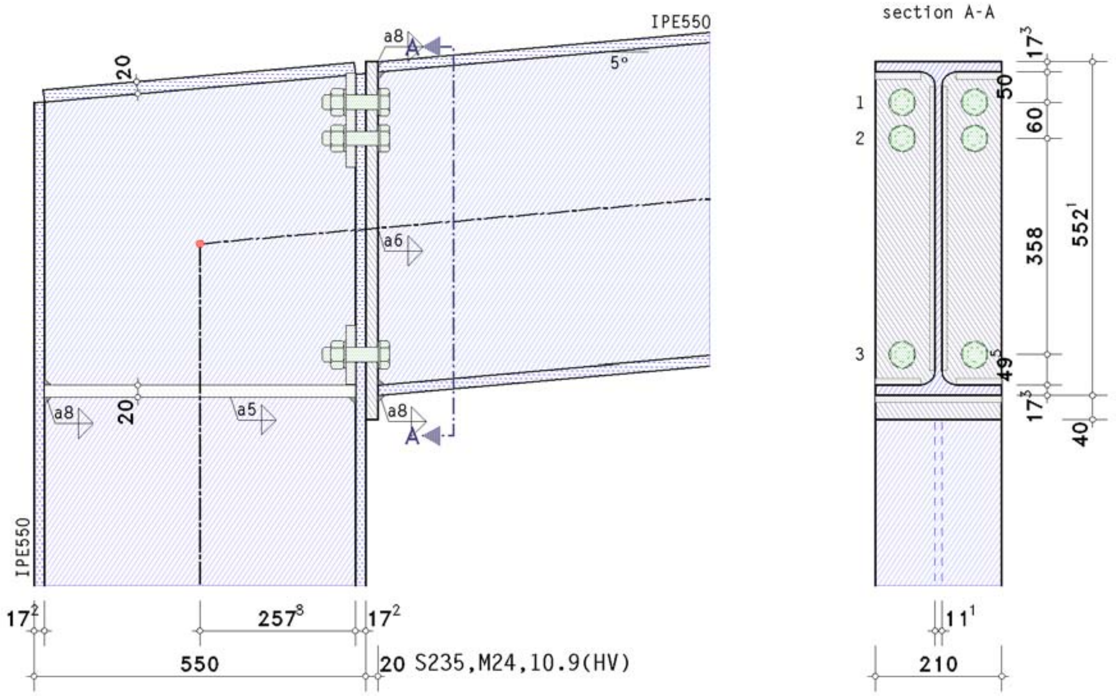
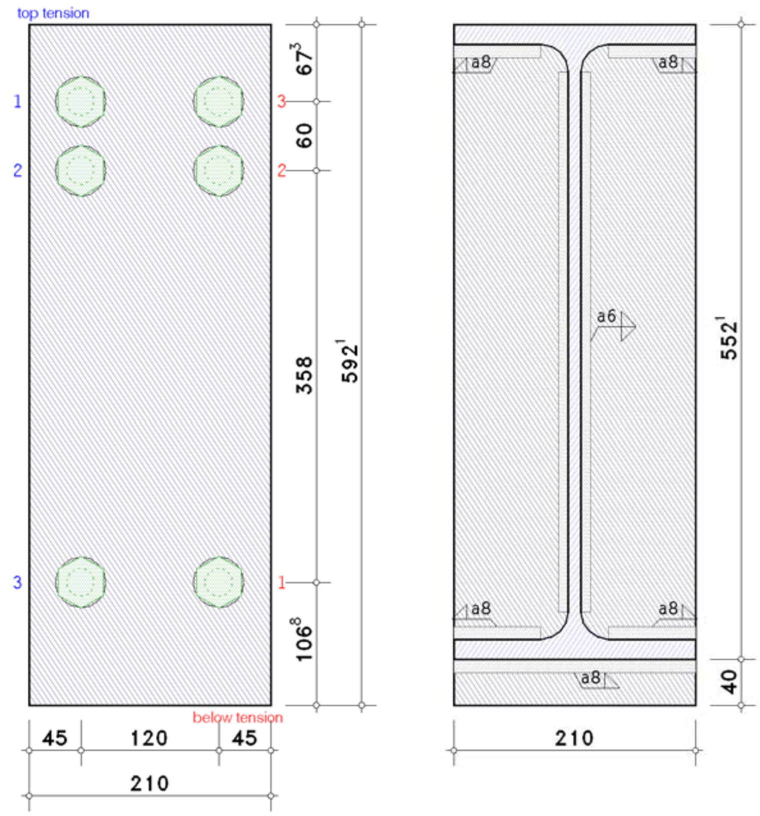
frame corner EC 3-1-8 (04.25), NA: Deutschland 4H-EC3RE version: 2/2020-2q

1. input report



details (section A - A)



steel grade

steel grade S235

column parameters

section IPE550

reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 523.6 mm):

thickness t_{st} = 20.0 mm, width b_{st} = 95.0 mm, length l_{st} = 515.6 mm

recess at stiffeners c_{st} = 36.0 mm

welds $a_{st,f}$ = 8.0 mm, $a_{st,w}$ = 5.0 mm

cap plate t_k = 20.0 mm (structurally effective)

bolts

bolt class 10.9, bolt size M24
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}^* = 0.7 \cdot f_{yb} \cdot A_s$ = 222.1 kN)
shear plane passes through the unthreaded portion of the bolt
packing (flange reinforcement): thickness t_{bp} = 15.0 mm, width b_{bp} = 96.0 mm, length l_{bp} = 96.0 mm

beam parameters

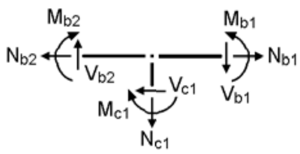
section IPE550
slope angle of section about the horizontal axis α_b = 5.00° $\Rightarrow$ section depth at the joint loc. $h_b = h/\cos(\alpha_b)$ = 552.1 mm

verification parameters

bolted end-plate connection

thickness t_p = 20.0 mm, width b_p = 210.0 mm, length l_p = 592.1 mm
projections $h_{p,o}$ = 0.0 mm, $h_{p,u}$ = 40.0 mm
bolts in connection:
3 bolt-rows with 2 bolts
of these 2 bolt-rows top in tension (rows 1-2)
and 1 bolt-row for shear transfer top (row 3)
of these 1 bolt-row below in tension (row 3)
and 2 bolt-rows for shear transfer below (rows 2-3)
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 = 45.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 67.3 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 106.8 mm
centre distance of the first bolt-row to the free edge of the column (end row) e_1' = 67.3 mm
centre distance of the bolt-rows from each other $p_{1\text{-}2}$ = 60.0 mm, $p_{2\text{-}3}$ = 358.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 8.0 mm, angle φ = 85°
beam web: fillet weld, weld thickness a = 6.0 mm
beam flange below: fillet weld, weld thickness a = 8.0 mm, angle φ = 95°

internal forces and moments in the intersection point of system axes



Lc 1: Ek 4 (right frame corner)
$N_{j,b,Ed}$ = -58.30 kN $M_{j,b,Ed}$ = -292.00 kNm $V_{j,b,Ed}$ = 103.30 kN
$N_{j,c1,Ed}$ = -107.99 kN $M_{j,c1,Ed}$ = -292.00 kNm $V_{j,c1,Ed}$ = -49.07 kN (calculated)
Lc 2: Ek 5 (left frame corner)
$N_{j,b,Ed}$ = -3.00 kN $M_{j,b,Ed}$ = 52.20 kNm $V_{j,b,Ed}$ = -3.80 kN
$N_{j,c1,Ed}$ = 3.52 kN $M_{j,c1,Ed}$ = 52.20 kNm $V_{j,c1,Ed}$ = -3.32 kN (calculated)
Lc 3: Ek 2 (right frame corner)
$N_{j,b,Ed}$ = -61.00 kN $M_{j,b,Ed}$ = -281.70 kNm $V_{j,b,Ed}$ = 117.00 kN
$N_{j,c1,Ed}$ = -121.87 kN $M_{j,c1,Ed}$ = -281.70 kNm $V_{j,c1,Ed}$ = -50.57 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
prestressing of high strength bolts γ_{M7} = 1.10

notes

connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.
no verification for cross-sections.
welds are not regarded by calculation the T-stub resistance.
local resistance of weld is not respected.
no verification of web stiffeners.
plated structures- and shear buckling is not investigated.
calculation of T-stub-resistance with the standard method.

check of data

ok

distances between bolts at end-plate

horizontal:	e_2 = 45.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_2 = 45.0 mm < $4 \cdot t$ + 40 mm = 108.8 mm
horizontal:	p_2 = 120.0 mm > $2.4 \cdot d_0$ = 62.4 mm,	p_2 = 120.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	e_1 = 67.3 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 67.3 mm < $4 \cdot t$ + 40 mm = 108.8 mm
top-below:	e_1 = 67.3 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 67.3 mm < $4 \cdot t$ + 40 mm = 108.8 mm
top-below:	p_1 = 60.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 60.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
top-below:	p_1 = 358.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 358.0 mm > min($14 \cdot t$, 200 mm) = 200.0 mm !!
top-below:	e_1 = 106.8 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 106.8 mm < $4 \cdot t$ + 40 mm = 108.8 mm

bolt distance from column edge

horizontal:	e_2 = 45.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_2 = 45.0 mm < $4 \cdot t$ + 40 mm = 108.8 mm

maximum values for spacings and edge distances should be in order to avoid local buckling and to prevent corrosion.

2. table of results

utilization/rotation

Lc	U_m	U_v	U_{wp}	U_{bw}	U_{ep}	U_{sb}	U	$S_{j,ini}$	S_j	φ_j
--	---	---	---	---	---	---	---	MNm/rad	MNm/rad	°
1	1.139	0.249	0.696	0.147	0.081	0.716	1.139*	117.3	27.6	0.520
2	0.282	0.004	0.131	0.005	0.003	0.136	0.282	85.4	85.4	0.034
3	1.073	0.281	0.656	0.166	0.092	0.680	1.073!	117.3	32.4	0.417

U_m: utilization due to bending; U_v: utilization due to shear/bearing resistance; U_{wp}: utilization due to shear in column web
U_{bw}: utilization due to shear in beam web; U_{ep}: utilization due to shear in end-plate; U_{sb}: utilization due to weld
U: utilization of the connection; $S_{j,ini}$: initial rotational stiffness; S_j: rotational stiffness
φ_j: rotation
*) maximum utilization

3. final result

maximum utilization [Lc 1]: max U = 1.139 > 1 not ok !!
minimum rotational stiffness: min S_j = 27.6 MNm/rad, $S_{j,ini}$ = 117.3 MNm/rad, φ_j = 0.520°

resistance not ensured !!

4. Regulations

EN 1990, Eurocode 0: Grundlagen der Tragwerksplanung;
Deutsche Fassung EN 1990:2002 + A1:2005 + A1:2005/AC:2010, Ausgabe Dezember 2010
EN 1990/NA, Nationaler Anhang zur EN 1990, Ausgabe Dezember 2010

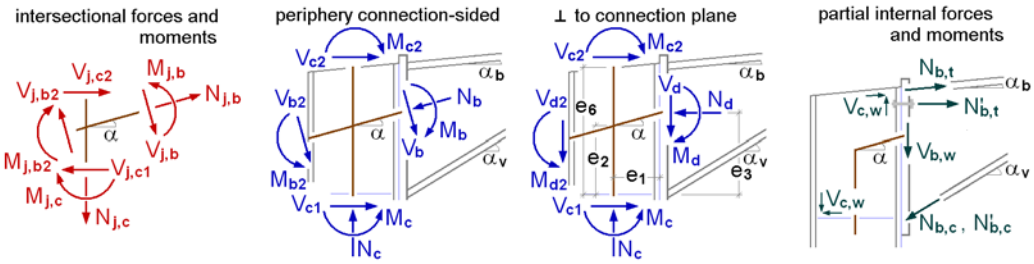
EN 1993-1-1, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau;
Deutsche Fassung EN 1993-1-1:2022, Ausgabe April 2025
EN 1993-1-1/NA, Nationaler Anhang zur EN 1993-1-1, Ausgabe April 2026

EN 1993-1-8, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-8: Bemessung von Anschlüssen; Deutsche Fassung EN 1993-1-8:2024, Ausgabe April 2025
EN 1993-1-8/NA, Nationaler Anhang zur EN 1993-1-8, Ausgabe April 2026

EN 1993-1-5, Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-5: Plattenförmige Bauteile;
Deutsche Fassung EN 1993-1-5:2006 + AC:2009 + A1:2017 + A2:2019, Ausgabe Oktober 2019
EN 1993-1-5/NA, Nationaler Anhang zur EN 1993-1-5, Ausgabe Dezember 2010

5. Lc 1 (decisive)

5.1. design values



slope angle: $\alpha_b = \alpha_v = \alpha = 5.00°$
distances: e_1 = 275.0 mm, e_3 = 267.4 mm, e_2 = 243.4 mm, e_6 = 523.6 mm

internal forces and moments perpendicular to the connection planes
periphery beam
N_d = 49.07 kN, M_d = 263.48 kNm, V_d = 107.99 kN
periphery column (below, calculated)
N_c = 107.99 kN, M_c = 280.06 kNm, V_c = 49.07 kN

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d \cdot t_p \cdot \tan(\alpha) - V_d \cdot t_p$ = 261.41 kNm
$N_{b,t} = (-N_d \cdot z_{bu}/z_b + M'_d/z_b) / \cos(\alpha_b)$ = 466.00 kN, z_b = 534.8 mm, z_{bu} = 267.4 mm
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_b)$ = 515.27 kN, z_b = 534.8 mm, z_{bo} = 267.4 mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b)$ = -40.61 kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_b)$ = 44.91 kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c}$ = 103.69 kN

5.2. connection capacity

transformation parameter: $\beta_j = 1.00$

5.2.1. moment resistance

distance of tension-bolt-rows from centre of compression: $h_1 = 476.2$ mm, $h_2 = 416.2$ mm

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 376.9$ kN
row 2: $F_{tr,Rd} = 97.0$ kN
$\Sigma F_{tr,Rd}^* = 473.9$ kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 376.9$ kN
row 2: $F_{tr,Rd} = 97.0$ kN
$\Sigma F_{tr,Rd} = 473.9$ kN
potential failure by basic component 3, 4, 5

resistance of flanges (compression)
$\Sigma F_{c,Rd}^* = 1613.6$ kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 219.9$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 473.9$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 1613.6$ kN

5.2.2. shear/bearing resistance

resistance per bolt-row (shear/bearing resistance)
row 3: $F_{vr,Rd} = 434.3$ kN
$\Sigma F_{vr,Rd} = 434.3$ kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd} = 434.3$ kN

5.2.3. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd} = 1274.20$ kN

shear resistance of the beam web
beam web: $V_{bw,Rd} = 707.18$ kN

shear resistance of column web
$V_{wp,Rd} = 806.80$ kN

5.2.4. total

$M_{j,Rd} = 219.9$ kNm $N_{j,t,Rd} = 473.9$ kN $N_{j,c,Rd} = 1613.6$ kN $V_{j,Rd} = 434.3$ kN $V_{wp,Rd} = 806.8$ kN $V_{ep,Rd} = 1274.2$ kN
$V_{bw,Rd} = 707.2$ kN

5.3. verifications

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

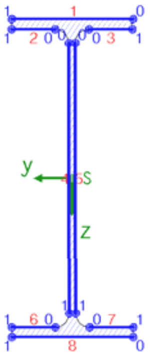
axial force: $N_{b,Ed} = |N_d \cdot \cos(\alpha) + V_d \cdot \sin(\alpha)| = 58.30$ kN $<$ $5\% \cdot N_{pl,Rd} = 158.52$ kN $\Rightarrow$ moment resistance
internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu} = 250.45$ kNm, $z_{bu} = 265.7$ mm
shear force: $V_{Ed} = |V_d| = 107.99$ kN
shear force: $V_{c,w,Ed} = (M_{d1} - M_{d2})/z - V_c/2 = 561.51$ kN, $z = 449.6$ mm
shear force: $V_{b,w,Ed} = 103.69$ kN

$M_{Ed}/M_{j,Rd} = 1.139 > 1$ **not ok !!**
$V_{Ed}/V_{j,Rd} = 0.249 < 1$ ok
$V_{c,w,Ed}/V_{wp,Rd} = 0.696 < 1$ ok
$V_{b,w,Ed}/V_{bw,Rd} = 0.147 < 1$ ok
$V_{b,w,Ed}/V_{ep,Rd} = 0.081 < 1$ ok

5.3.2. 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

calculation section:



weld 1:	a_w = 8.0 mm	l_w = 210.0 mm
weld 2:	a_w = 8.0 mm	l_w = 75.4 mm
weld 3:	see weld 2	
weld 4:	a_w = 6.0 mm	l_w = 469.6 mm
weld 5:	see weld 4	
weld 6:	a_w = 8.0 mm	l_w = 75.4 mm
weld 7:	see weld 6	
weld 8:	a_w = 8.0 mm	l_w = 210.0 mm

design values refering to centroid of the section:
N_{Ed} = -49.07 kN, $M_{y,Ed}$ = -263.48 kNm, $V_{z,Ed}$ = 107.99 kN
cross-sectional properties refering to centroid of the line cross-section:
ΣA_w = 114.09 cm², $A_{w,z}$ = 56.35 cm², Σl_w = 166.1 cm
$I_{w,y}$ = 52127.42 cm⁴, $I_{w,z}$ = 2459.43 cm⁴, Δz_w = 0.0 mm
verifications in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x}$ = 135.23 N/mm²		$\Rightarrow U_w = 0.531 < 1$ ok
weld 2,	pt. 0:	$\sigma_{w,x}$ = 126.50 N/mm²		$\Rightarrow U_w = 0.497 < 1$ ok
weld 4,	pt. 0:	$\sigma_{w,x}$ = 114.37 N/mm²	$\tau_{w,z}$ = 19.16 N/mm²	$\Rightarrow U_w = 0.459 < 1$ ok
	pt. 1:	$\sigma_{w,x}$ = -122.98 N/mm²	$\tau_{w,z}$ = 19.16 N/mm²	$\Rightarrow U_w = 0.492 < 1$ ok
weld 6,	pt. 0:	$\sigma_{w,x}$ = -135.11 N/mm²		$\Rightarrow U_w = 0.531 < 1$ ok
weld 8,	pt. 0:	$\sigma_{w,x}$ = -143.83 N/mm²		$\Rightarrow U_w = 0.565 < 1$ ok

Result:
weld 8, pt. 0: $\sigma_{w,x}$ = -143.83 N/mm²
Max: $\sigma_{1,w,Ed}$ = 203.41 N/mm² $<$ $f_{1w,d}$ = 360.00 N/mm²,
$\sigma_{2,w,Ed}$ = 101.71 N/mm² $<$ $f_{2w,d}$ = 259.20 N/mm² $\Rightarrow U_w = 0.565 < 1$ ok

verification of deflection forces (directional method)
welds tension flange
tension force $N_{b,t}$ = 466.0 kN
weld angle φ = 85.0°:
$\sigma_{1,w,Ed}$ = 233.2 N/mm² $<$ $f_{1w,d}$ = 360.0 N/mm² $\Rightarrow U = 0.648 < 1$ ok
$\sigma_{2,w,Ed}$ = 109.0 N/mm² $<$ $f_{2w,d}$ = 259.2 N/mm² $\Rightarrow U = 0.421 < 1$ ok
weld angle φ = 95.0°:
$\sigma_{1,w,Ed}$ = 223.2 N/mm² $<$ $f_{1w,d}$ = 360.0 N/mm² $\Rightarrow U = 0.620 < 1$ ok
$\sigma_{2,w,Ed}$ = 119.0 N/mm² $<$ $f_{2w,d}$ = 259.2 N/mm² $\Rightarrow U = 0.459 < 1$ ok
welds compression flange
compression force $N_{b,c}$ = 515.3 kN
weld angle φ = 95.0°:
$\sigma_{1,w,Ed}$ = 246.8 N/mm² $<$ $f_{1w,d}$ = 360.0 N/mm² $\Rightarrow U = 0.686 < 1$ ok
$\sigma_{2,w,Ed}$ = 131.6 N/mm² $<$ $f_{2w,d}$ = 259.2 N/mm² $\Rightarrow U = 0.508 < 1$ ok
weld angle φ = 85.0°:
$\sigma_{1,w,Ed}$ = 257.8 N/mm² $<$ $f_{1w,d}$ = 360.0 N/mm² $\Rightarrow U = 0.716 < 1$ ok
$\sigma_{2,w,Ed}$ = 120.6 N/mm² $<$ $f_{2w,d}$ = 259.2 N/mm² $\Rightarrow U = 0.465 < 1$ ok

5.3.3. verification result

maximum utilization: max U = 1.139 $>$ 1 **not ok !!**
failure at verification of bending: U = 1.139

5.4. rotational stiffness

stiffness coefficients
equivalent stiffness coefficient for 2 tension-bolt-rows:
1: k_3 = 3.60 mm, k_4 = 22.64 mm, k_5 = 18.20 mm $\Rightarrow k_{eff,1} = 1 / \Sigma(1/k_{i,1})$ = 2.653 mm
2: k_3 = 3.28 mm, k_4 = 20.62 mm, k_5 = 16.42 mm $\Rightarrow k_{eff,2} = 1 / \Sigma(1/k_{i,2})$ = 2.413 mm
equivalent internal lever arm $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r)$ = 449.60 mm
equivalent stiffness coefficient $k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq}$ = 5.043 mm
$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z)$ = 6.11 mm
$k_2 = \infty$ (stiffened)
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i)$ = 117308.2 kNm/rad, $z = z_{eq}$ = 449.6 mm, $\Sigma(1/k_i)$ = 0.362 mm⁻¹
$|N_{b,Ed}|$ = 58.30 kN $<$ 5%·$N_{pl,Rd}$ = 158.52 kN ok
$|M_{j,Ed}|$ = 250.45 kNm $>$ 2/3 $M_{j,Rd}$ = 146.57 kNm $\Rightarrow \mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi}$ = 4.248, Ψ = 2.7
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu$ = 27612.7 kNm/rad

rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd} = 0.520°$