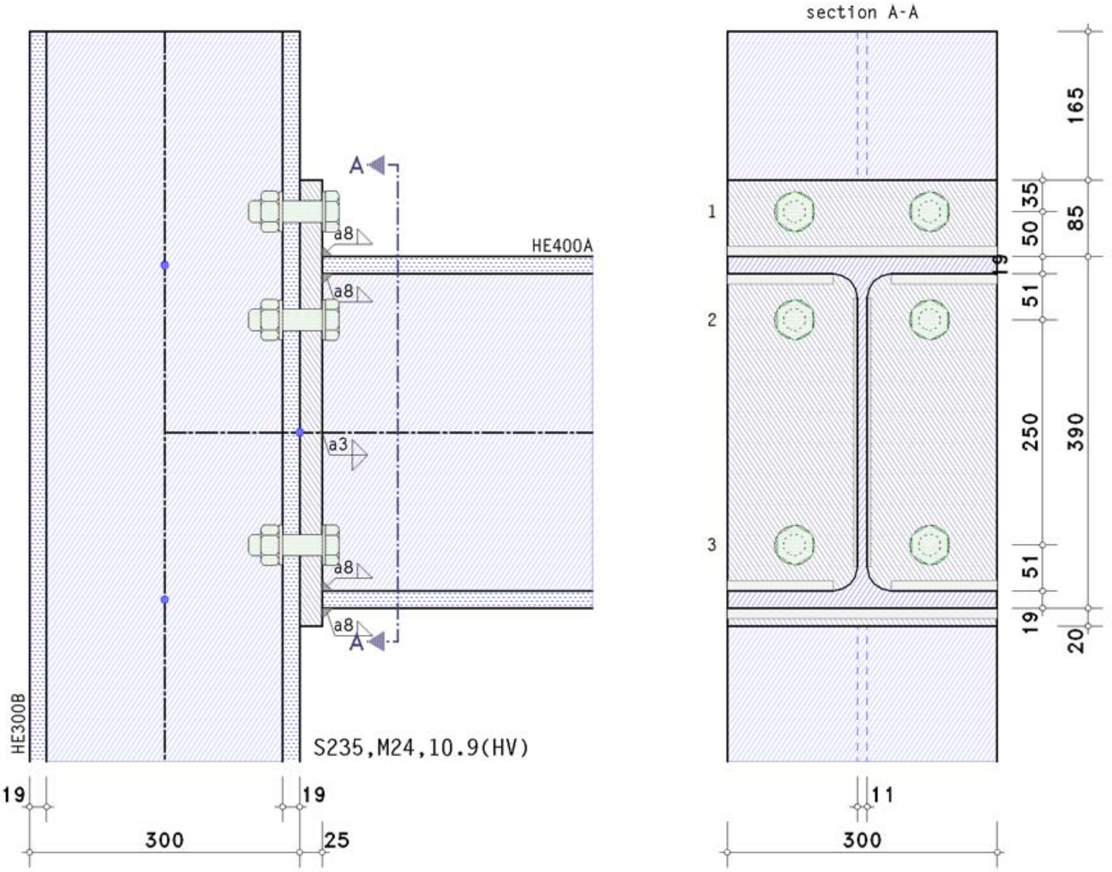
POS. 140: KAHLMEYER 3.6.4.1

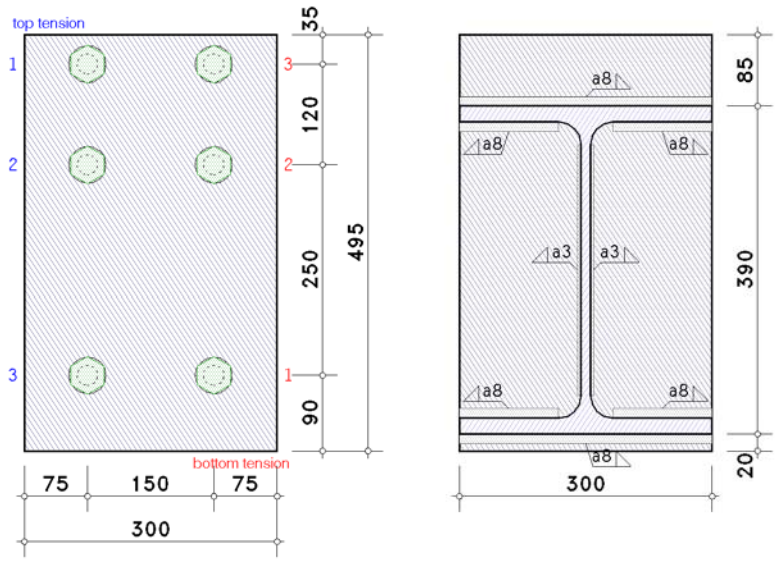
4H-ec3BT version: 4/2013-5I

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

1. input report



details (section A - A)



steel grade

steel grade S235, calculation parameters:

char. yield strength $f_y = 235.0$ N/mm² for $t \leq 40$ mm, $f_y = 215.0$ N/mm² for $t > 40$ mm

char. tensile strength $f_u = 360.0$ N/mm² for $t \leq 40$ mm, $f_u = 360.0$ N/mm² for $t > 40$ mm

correlation value of fillet weld $\beta_w = 0.80$

elastic modulus E = 210000.0 N/mm²

column parameters

section HE300B

design values:

overall depth h = 300.0 mm, web thickness $t_w = 11.0$ mm

flange width b_f = 300.0 mm, flange thickness t_f = 19.0 mm
rolled section, root radius r = 27.0 mm
clear depth of web without root/weld d_w = 208.0 mm
width of one flange side without root/weld c_f = 117.5 mm
centroid distance from top z_s = 150.0 mm
cross-sectional area A = 149.08 cm^2
second moment of area I_y = 25164.07 cm^4
plastic section modulus $W_{pl,y}$ = 1869.00 cm^3
elastic section modulus $W_{el,y} = I_y/z_s$ = 1677.60 cm^3
effective shear area A_{vz} = 47.43 cm^2
torsional moment of inertia I_T = 186.00 cm^4

bolts

bolt class 10.9
calculation parameters:
char. yield strength f_{yb} = 900.0 N/mm^2
char. tensile strength f_{ub} = 1000.0 N/mm^2
bolt size M24
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}^* = 0.7 \cdot f_{yb} \cdot A_s$ = 222.4 kN)
calculation parameters:
shaft diameter d = 24.0 mm, clearance Δd = 2.0 mm $\Rightarrow$ hole diameter d_0 = 26.0 mm
gross cross-section area A = 4.524 cm^2
tensile stress area A_s = 3.530 cm^2
diameter of the bolt head (across flats dimension) d_s = 41.0 mm
diameter of the bolt head (across points dimension) d_e = 45.20 mm
thickness of the bolt head t_k = 15.0 mm
thickness of nut t_m = 20.0 mm
diameter of the plate under the bolt or the nut d_p = 44.0 mm
thickness of the plate under the bolt or the nut t_p = 4.0 mm
washer double-sided
shear plane passes through the unthreaded portion of the bolt

beam parameters

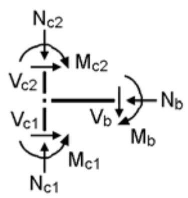
section HE400A
design values:
overall depth h = 390.0 mm, web thickness t_w = 11.0 mm
flange width b_f = 300.0 mm, flange thickness t_f = 19.0 mm
rolled section, root radius r = 27.0 mm
clear depth of web without root/weld d_w = 298.0 mm
width of one flange side without root/weld c_f = 117.5 mm
centroid distance from top z_s = 195.0 mm
cross-sectional area A = 158.98 cm^2
second moment of area I_y = 45067.79 cm^4
plastic section modulus $W_{pl,y}$ = 2562.00 cm^3
elastic section modulus $W_{el,y} = I_y/z_s$ = 2311.17 cm^3
effective shear area A_{vz} = 57.33 cm^2
torsional moment of inertia I_T = 190.00 cm^4

verification parameters

bolted end-plate connection:
thickness t_p = 25.0 mm, width b_p = 300.0 mm, length l_p = 495.0 mm
projections $h_{p,o}$ = 85.0 mm, $h_{p,u}$ = 20.0 mm
bolts in connection:
3 bolt-rows with 2 bolts
all bolt-rows considered individually
tensile edge top: group of bolts with 2 rows (1-2)
tensile edge bottom: no group of bolts
all bolt-rows for shear transfer (rows 1-3)
centre distance of the bolts to the lateral edge of the end-plate e_2 = 75.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 35.0 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 90.0 mm
centre distance of the first bolt-row to the free edge of the column (end row) e_1' = 200.0 mm
centre distance of the bolt-rows from each other $p_{1\text{-}2}$ = 120.0 mm, $p_{2\text{-}3}$ = 250.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 8.0 mm
beam web: fillet weld, weld thickness a = 3.0 mm
beam flange bottom: fillet weld, weld thickness a = 8.0 mm

internal forces and moments at the joint periphery refering to the system axes

Lk 1: $M_{b,Ed}$ = 200.00 kNm $V_{b,Ed}$ = 270.00 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

check of data
ok
distances between bolt-rows at end-plate

horizontal:	e_2 = 75.0 mm > 1.2·d_0 = 31.2 mm,	e_2 = 75.0 mm < 4·t + 40 mm = 116.0 mm
horizontal:	p_2 = 150.0 mm > 2.4·d_0 = 62.4 mm,	p_2 = 150.0 mm < min(14·t, 200 mm) = 200.0 mm
vertical:	e_1 = 35.0 mm > 1.2·d_0 = 31.2 mm,	e_1 = 35.0 mm < 4·t + 40 mm = 116.0 mm
vertical:	e_1 = 200.0 mm > 1.2·d_0 = 31.2 mm,	e_1 = 200.0 mm > 4·t + 40 mm = 116.0 mm !!
vertical:	p_1 = 120.0 mm > 2.2·d_0 = 57.2 mm,	p_1 = 120.0 mm < min(14·t, 200 mm) = 200.0 mm
vertical:	p_1 = 250.0 mm > 2.2·d_0 = 57.2 mm,	p_1 = 250.0 mm > min(14·t, 200 mm) = 200.0 mm !!
vertical:	e_1 = 90.0 mm > 1.2·d_0 = 31.2 mm,	e_1 = 90.0 mm < 4·t + 40 mm = 116.0 mm

horizontal distance of bolts from column edge

vertical:	e_1 = 75.0 mm > 1.2·d_0 = 31.2 mm,	e_1 = 75.0 mm < 4·t + 40 mm = 116.0 mm

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

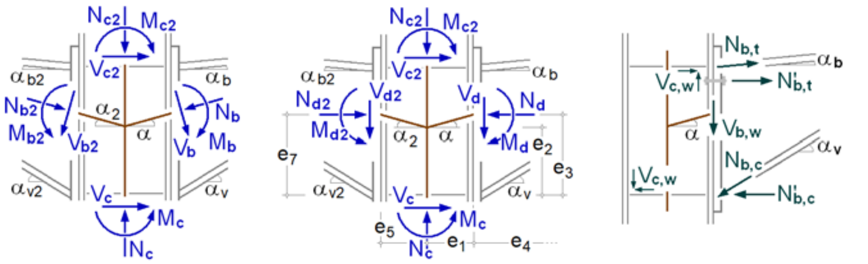
notes
no verification for cross-sections.
no verfication for welds within the connection.

2. Lk 1

notes
the following verification is applied to the connection of a girder to a continuous column.
to dimension a frame corner completely, further verifications are required.
connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.
calculation of T-stub-resistance with the standard method.

2.1. design values

periphery connection ⊥ zur connection plane   partial internal forces and moments



sign definition of EC3:   a positive axial force means compression, a positive bending moment produces tension at the top

slope angle: $\alpha_b = \alpha_v = \alpha = 0°$
distance: e_1 = 150.0 mm, e_3 = 185.5 mm, e_2 = 185.5 mm

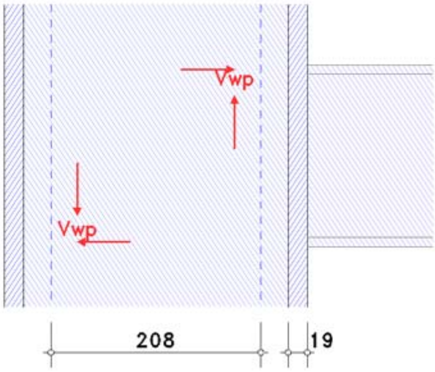
transformation joint values -> design values
M_d = 200.00 kNm, V_d = 270.00 kN
internal forces and moments perpendicular to the connection planes
periphery beam
M_d = 200.00 kNm, V_d = 270.00 kN
calculation of internal forces and moments at periphery column (top)
$N_{c2} = N_c - V_d$ = -270.00 kN
$M_{c2} = M_c + V_c \cdot e_6 - M_d - V_d \cdot e_1 - N_d \cdot (e_6 - e_3)$ = -240.50 kNm
partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_{ep}$ = 193.25 kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 520.89 kN, z_b = 371.0 mm, z_{bu} = 185.5 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b$ = 520.89 kN, z_b = 371.0 mm, z_{bo} = 185.5 mm

2.2. basic components

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

2.2.1. Gk 1: Column web panel in shear

transformation parameter (EC 3-1-8, 5.3(9)) β_j = 1.00 for M_{j1} = 200.00 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} = 18.91 < 69·ε = 69.00, ε = 1.00 ⇒ method applicable

shear area

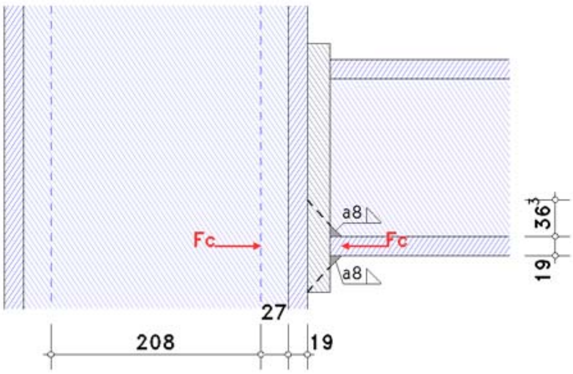
shear area A_v = 47.43 cm²

plastic shear resistance

plastic shear resistance $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_v) / (3^{1/2} \cdot \gamma_{M0})$ = 579.1 kN

2.2.2. Gk 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) β_j = 1.00 for M_{j1} = 200.00 kNm (M_{j2} = 0)



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

effective width

effective width of column web in transverse compression $b_{eff,c} = t_{f,b} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{f,c}+s_c) + s_p$ = 305.3 mm, s_p = 33.7 mm

longitudinal compressive stress in web

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

plate buckling

plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2}$ = 0.714, E = 210000 N/mm²

reduction factor for web buckling ρ = 1

shear area

shear area A_v = 47.43 cm²

interaction with shear stress

reduction factor for interaction with shear stress β = 1 ⇒ $\omega = \omega_1$ = 0.778

with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w / A_v)^2]^{1/2}$ = 0.778

resistance of an unstiffened web in transverse compression

$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0}$ = 614.07 kN, $f_{y,w}$ = 235.0 N/mm²

$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1}$ = 558.25 kN (decisive)

resistance of upper beam flange:

effective width

effective width of column web in transverse compression $b_{eff,c} = t_{f,b} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{f,c}+s_c) + s_p$ = 321.6 mm, s_p = 50.0 mm

longitudinal compressive stress in web

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

plate buckling

plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2}$ = 0.733, E = 210000 N/mm²

reduction factor for web buckling $\rho = (\lambda_p - 0.2) / \lambda_p^2$ = 0.992

shear area

shear area A_v = 47.43 cm²

interaction with shear stress

reduction factor for interaction with shear stress β = 1 ⇒ $\omega = \omega_1$ = 0.762

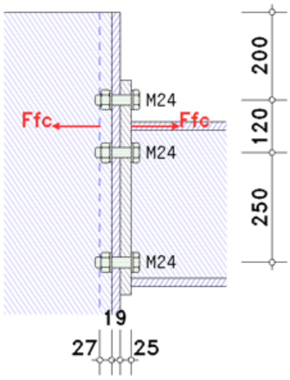
with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w / A_v)^2]^{1/2}$ = 0.762

resistance of an unstiffened web in transverse compression

$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0}$ = 633.32 kN, $f_{y,w}$ = 235.0 N/mm²

$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1}$ = 571.11 kN (decisive)

2.2.3. Gk 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 edge of flange e = 75.0 mm
distance centre-line of the bolt to the stub web m = 47.9 mm

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

(other) end bolt-row

$l_{eff,cp,a} = \min(2 \cdot \pi \cdot m,\ \pi \cdot m + 2 \cdot e_1) = 301.0$ mm
$l_{eff,nc,a} = \min(4 \cdot m + 1.25 \cdot e,\ 2 \cdot m + 0.625 \cdot e + e_1) = 285.4$ mm
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc},\ l_{eff,cp}) = 285.4$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 285.4$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 59.9$ mm, $e_{min} = 75.0$ mm, $m = 47.9$ mm
resisting plastic moments:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 6.05$ kNm, $t_f = 19.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$
design value of tension resistance:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 254.16$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 508.32$ kN, $n_b = 1$
prying forces always appear at preloaded bolts !
calculation with the standard method
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 505.38$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 394.71$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 508.32$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd},\ F_{T,2,Rd},\ F_{T,3,Rd}) = 394.71$ kN

row 2

distance centre-line of the bolt to the edge of flange e = 75.0 mm
distance centre-line of the bolt to the stub web m = 47.9 mm

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

(other) end bolt-row

$l_{eff,cp,a} = 2 \cdot \pi \cdot m = 301.0$ mm ($e_1 = \infty$)
$l_{eff,nc,a} = 4 \cdot m + 1.25 \cdot e = 285.4$ mm ($e_1 = \infty$)
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc},\ l_{eff,cp}) = 285.4$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 285.4$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 59.9$ mm, $e_{min} = 75.0$ mm, $m = 47.9$ mm
resisting plastic moments:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 6.05$ kNm, $t_f = 19.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$
design value of tension resistance:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 254.16$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 508.32$ kN, $n_b = 1$
prying forces always appear at preloaded bolts !
calculation with the standard method
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 505.38$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 394.71$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 508.32$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd},\ F_{T,2,Rd},\ F_{T,3,Rd}) = 394.71$ kN

row 3

distance centre-line of the bolt to the edge of flange e = 75.0 mm
distance centre-line of the bolt to the stub web m = 47.9 mm

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

(other) inner bolt-row

$l_{eff,cp,i} = 2 \cdot \pi \cdot m = 301.0$ mm

$l_{eff,nc,i} = 4 \cdot m + 1.25 \cdot e = 285.4$ mm
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 285.4$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 285.4$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 59.9$ mm, $e_{min} = 75.0$ mm, $m = 47.9$ mm
resisting plastic moments:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 6.05$ kNm, $t_f = 19.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$
design value of tension resistance:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 254.16$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 508.32$ kN, $n_b = 1$
prying forces always appear at preloaded bolts !
calculation with the standard method
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 505.38$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 394.71$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 508.32$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 394.71$ kN

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

$F_{t,fc,Rd,1} = 394.71$ kN, $l_{eff,1} = 285.4$ mm
$F_{t,fc,Rd,2} = 394.71$ kN, $l_{eff,2} = 285.4$ mm
$F_{t,fc,Rd,3} = 394.71$ kN, $l_{eff,3} = 285.4$ mm

equivalent T-stub flange (group of bolt-rows):

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

row 1

distance centre-line of the bolt to the edge of flange e = 75.0 mm
distance centre-line of the bolt to the stub web m = 47.9 mm
distance between bolt-rows p = 120.0 mm
(other) end bolt-row
$l_{eff,cp,a} = \min(\pi \cdot m+p, 2 \cdot e_1+p) = 270.5$ mm
$l_{eff,nc,a} = \min(2 \cdot m+0.625 \cdot e+0.5 \cdot p, e_1+0.5 \cdot p) = 202.7$ mm

row 2

distance centre-line of the bolt to the edge of flange e = 75.0 mm
distance centre-line of the bolt to the stub web m = 47.9 mm
distance between bolt-rows p = 120.0 mm
(other) end bolt-row
$l_{eff,cp,a} = \pi \cdot m+p = 270.5$ mm ($e_1 = \infty$)
$l_{eff,nc,a} = 2 \cdot m+0.625 \cdot e+0.5 \cdot p = 202.7$ mm ($e_1 = \infty$)

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

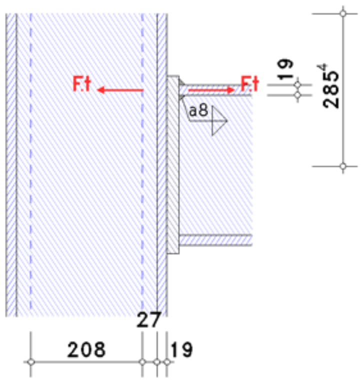
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 405.4$ mm, $\Sigma l_{eff,cp} = 541.0$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 405.4$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 59.9$ mm, $e_{min} = 75.0$ mm, $m = 47.9$ mm
resisting plastic moments:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.60$ kNm, $t_f = 19.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$
design value of tension resistance:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 254.16$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 1016.64$ kN, $n_b = 2$
prying forces always appear at preloaded bolts !
calculation with the standard method
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 717.91$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 724.34$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1016.64$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 717.91$ kN
effective length: $\Sigma l_{eff} = 405.4$ mm, 2 rows

2.2.4. Gk 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.00$ for $M_{j1} = 200.00$ 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 $b_{eff,t}$ = 285.4 mm (l_{eff} from bc 4)

shear area

shear area A_v = 47.43 cm²

interaction with shear stress

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = \omega_1 = 0.798$

with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w / A_v)^2]^{1/2} = 0.798$

resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0}$ = 588.8 kN, $f_{y,wc}$ = 235.0 N/mm²

row 2

effective width

effective width of column web in transverse tension $b_{eff,t}$ = 285.4 mm (l_{eff} from bc 4)

shear area

shear area A_v = 47.43 cm²

interaction with shear stress

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = \omega_1 = 0.798$

with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w / A_v)^2]^{1/2} = 0.798$

resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0}$ = 588.8 kN, $f_{y,wc}$ = 235.0 N/mm²

row 3

effective width

effective width of column web in transverse tension $b_{eff,t}$ = 285.4 mm (l_{eff} from bc 4)

shear area

shear area A_v = 47.43 cm²

interaction with shear stress

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = \omega_1 = 0.798$

with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w / A_v)^2]^{1/2} = 0.798$

resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0}$ = 588.8 kN, $f_{y,wc}$ = 235.0 N/mm²

each group of bolt-rows:

effective width

effective width of column web in transverse tension $b_{eff,t}$ = 405.4 mm (l_{eff} from bc 4)

shear area

shear area A_v = 47.43 cm²

interaction with shear stress

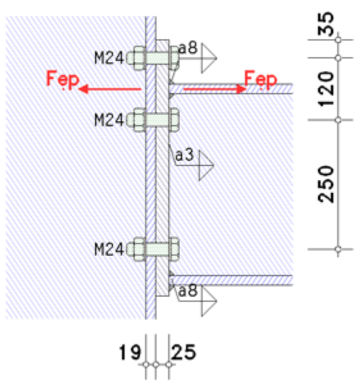
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = \omega_1 = 0.682$

with $\omega_1 = 1 / [1 + 1.3 \cdot (b_{eff} \cdot t_w / A_v)^2]^{1/2} = 0.682$

resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0}$ = 714.8 kN, $f_{y,wc}$ = 235.0 N/mm²

2.2.5. Gk 5: end-plate in bending



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

extended part of end-plate

in the extended part of the end-plate only one bolt-row is considered (n_b = 1).

distance centre-line of the bolt to beam flange m_1 = 40.9 mm

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

$e_x = e_1 = 35.0$ mm, $m_x = m_1 = 40.9$ mm, $w = b_p \cdot 2 \cdot e = 150.0$ mm with $b_p = 300.0$ mm, $e = 75.0$ mm

end bolt-row outside tension flange of beam

$l_{eff,cp,sa} = \min(2 \cdot \pi \cdot m_x,\ \pi \cdot m_x + w,\ \pi \cdot m_x + 2 \cdot e) = 257.3$ mm

$l_{eff,nc,sa} = \min(4 \cdot m_x + 1.25 \cdot e_x,\ e + 2 \cdot m_x + 0.625 \cdot e_x,\ 0.5 \cdot b_p,\ 0.5 \cdot w + 2 \cdot m_x + 0.625 \cdot e_x) = 150.0$ mm

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc},\ l_{eff,cp}) = 150.0$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 150.0$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 35.0$ mm, $e_{min} = 35.0$ mm, $m = 40.9$ mm

resisting plastic moments:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.51$ kNm, $t_f = 25.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 254.16$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 508.32$ kN, $n_b = 1$

prying forces always appear at preloaded bolts !

calculation with the standard method

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 538.02$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 379.29$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 508.32$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd},\ F_{T,2,Rd},\ F_{T,3,Rd}) = 379.29$ kN

resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 610.94$ kN (≥ 379.29 kN, not decisive)

resistance and effective length of end-plate in bending (projection)

$F_{t,ep,Rd,1} = 379.29$ kN, $l_{eff,1} = 150.0$ mm

part of end-plate between beam flanges

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

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

row 2

distance centre-line of the bolt to the stiffener $m_2 = 41.9$ mm

distance centre-line of the bolt to the edge of flange $e = 75.0$ mm

distance centre-line of the bolt to the stub web $m = 66.1$ 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 $\lambda_1 = m / (m+e) = 0.468$, $\lambda_2 = m_2 / (m+e) = 0.297 \Rightarrow \alpha = 6.63$ (calculated)

$l_{eff,cp,si} = 2 \cdot \pi \cdot m = 415.4$ mm

$l_{eff,nc,si} = \alpha \cdot m = 438.6$ mm

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc},\ l_{eff,cp}) = 415.4$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 438.6$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 75.0$ mm, $e_{min} = 75.0$ mm, $m = 66.1$ mm

resisting plastic moments:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 15.25$ kNm, $t_f = 25.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 16.10$ kNm

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 254.16$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 508.32$ kN, $n_b = 1$

prying forces always appear at preloaded bolts !

calculation with the standard method

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 922.84$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 498.42$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 508.32$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd},\ F_{T,2,Rd},\ F_{T,3,Rd}) = 498.42$ kN

resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 634.39$ kN (≥ 498.42 kN, not decisive)

row 3

distance centre-line of the bolt to the stiffener $m_2 = 41.9$ mm

distance centre-line of the bolt to the edge of flange $e = 75.0$ mm

distance centre-line of the bolt to the stub web $m = 66.1$ 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 $\lambda_1 = m / (m+e) = 0.468$, $\lambda_2 = m_2 / (m+e) = 0.297 \Rightarrow \alpha = 6.63$ (calculated)

$l_{eff,cp,si} = 2 \cdot \pi \cdot m = 415.4$ mm

$l_{eff,nc,si} = \alpha \cdot m = 438.6$ mm

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc},\ l_{eff,cp}) = 415.4$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 438.6$ mm

tension resistance of the T-stub flange

$n = \min(e_{min}, 1.25 \cdot m) = 75.0$ mm, $e_{min} = 75.0$ mm, $m = 66.1$ mm

resisting plastic moments:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 15.25$ kNm, $t_f = 25.0$ mm, $f_y = 235.0$ N/mm², $\gamma_{M0} = 1.00$

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 16.10$ kNm

design value of tension resistance:

tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 254.16$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 508.32$ kN, $n_b = 1$

prying forces always appear at preloaded bolts !

calculation with the standard method

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 922.84$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 498.42$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 508.32$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 498.42$ kN

resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm², $f_u = 360.0$ N/mm², $\beta_w = 0.80$

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 634.39$ kN (≥ 498.42 kN, not decisive)

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

$F_{ep,Rd,2} = 498.42$ kN, $l_{eff,2} = 415.4$ mm

$F_{ep,Rd,3} = 498.42$ kN, $l_{eff,3} = 415.4$ mm

2.2.6. Gk 7: beam flange and web in compression

section class of beam ($\varepsilon = 1.00$):

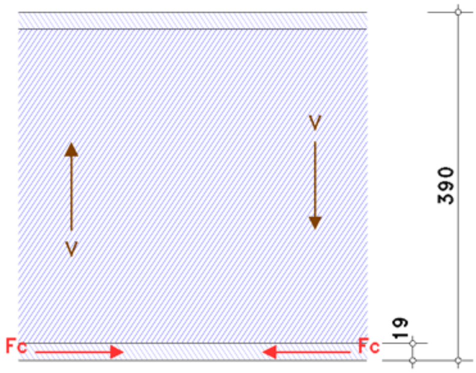
flange bottom: section class for $c/(\varepsilon \cdot t) = 6.18$ (outstand flange): 1

web: section class for $\alpha = 0.50$ and $c/(\varepsilon \cdot t) = 27.09$ (internal compression parts, bending): 1

total: section class: 1

taking into account the moment-shear force-interaction $V_{Ed} = 270.0$ kN

plastic section modulus $W_{pl} = 2562.000$ cm³



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

stress due to bending with shear force

$V_{pl,Rd} = A_v \cdot (f_y/3^{1/2}) / \gamma_{M0} = 777.8$ kN, $A_v = 57.33$ cm²

$V_{Ed} = 270.0$ kN ≤ 388.9 kN $= V_{pl,Rd}/2$ $\Rightarrow$ no effect on the moment resistance !

stress for section class 1

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 602.07$ kNm, $W_{pl} = 2562.00$ cm³

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 1622.83$ kN, $(h - t_f) = 371.0$ mm

resistance of upper beam flange:

stress due to bending with shear force

$V_{pl,Rd} = A_v \cdot (f_y/3^{1/2}) / \gamma_{M0} = 777.8$ kN, $A_v = 57.33$ cm²

$V_{Ed} = 270.0$ kN ≤ 388.9 kN $= V_{pl,Rd}/2$ $\Rightarrow$ no effect on the moment resistance !

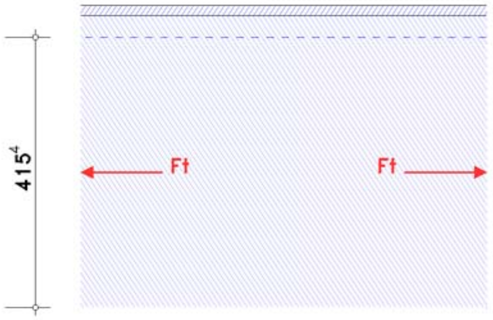
stress for section class 1

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 602.07$ kNm, $W_{pl} = 2562.00$ cm³

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 1622.83$ kN, $(h - t_f) = 371.0$ mm

2.2.7. Gk 8: beam web in tension



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

each individual bolt-row:

row 2

effective width

effective width of the beam web in tension $b_{eff,t,wb}$ = 415.4 mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 1073.7 kN, $f_{y,wb}$ = 235.0 N/mm²

row 3

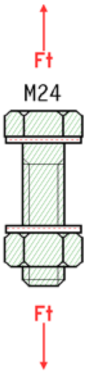
effective width

effective width of the beam web in tension $b_{eff,t,wb}$ = 415.4 mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0}$ = 1073.7 kN, $f_{y,wb}$ = 235.0 N/mm²

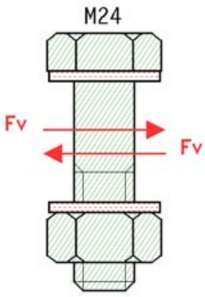
2.2.8. Gk 10: bolts in tension



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

bolt category D:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2}$ = 254.16 kN, k_2 = 0.90, f_{ub} = 1000.0 N/mm²
p. sh. load capicity: $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2}$ = 444.55 kN, d_m = 43.1 mm, t_p = 19.0 mm, f_u = 360.0 N/mm²
tension-/punching shear load capicity for 2 bolts: $\Sigma F_{tp,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd})$ = 508.32 kN

2.2.9. Gk 11: bolts in shear

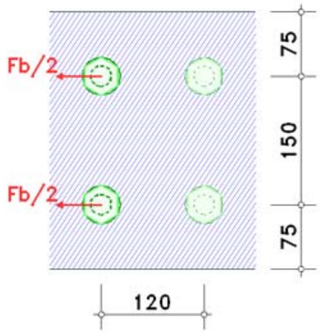


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

bolt category A:
shear plane passes through the unthreaded portion of the bolt: α_v = 0.6, A = 4.52 cm²
shear resistance per shear plane: $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2}$ = 217.15 kN, f_{ub} = 1000.0 N/mm²
shear resistance of 2 bolts (1-shear): $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd}$ = 434.29 kN

2.2.10. Gk 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: $\alpha_{d,a} = e_1/(3 \cdot d_0) = 0.45$ (end bolt)

$\Rightarrow$ $\alpha_b = 0.45$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)

$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 193.85$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 24.0$ mm

bolt 2:

in direction of load transfer: $\alpha_{d,a} = e_1/(3 \cdot d_0) = 0.45$ (end bolt)

$\Rightarrow$ $\alpha_b = 0.45$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)

$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 193.85$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 24.0$ mm

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 387.69$ 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: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)

$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 328.32$ kN, $f_u = 360.0$ N/mm², $t = 19.0$ mm, $d = 24.0$ mm

bolt 2:

in direction of load transfer: $\alpha_b = 1.00$

across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)

$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 328.32$ kN, $f_u = 360.0$ N/mm², $t = 19.0$ mm, $d = 24.0$ mm

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 656.64$ kN

row 2

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

bolt 1:

in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 1.29$ (inner bolt)

$\Rightarrow$ $\alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)

$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 432.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 24.0$ mm

bolt 2:

in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 1.29$ (inner bolt)

$\Rightarrow$ $\alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)

$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 432.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 24.0$ mm

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 864.00$ kN

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

bolt 1:

in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 2.96$ (inner bolt)

$\Rightarrow$ $\alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)

$\Rightarrow$ $k_1 = 2.50$ (smallest value of k_1 or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 328.32$ kN, $f_u = 360.0$ N/mm², $t = 19.0$ mm, $d = 24.0$ mm

bolt 2:

in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 2.96$ (inner bolt)

$\Rightarrow$ $\alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 328.32$ kN, $f_u = 360.0$ N/mm², $t = 19.0$ mm, $d = 24.0$ mm
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 656.64$ kN

row 3

end-plate (for $V_b \geq 0$):
bolt 1:
in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 2.96$ (inner bolt)
$\Rightarrow \alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 432.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 24.0$ mm
bolt 2:
in direction of load transfer: $\alpha_{d,i} = p_1/(3 \cdot d_0) - 1/4 = 2.96$ (inner bolt)
$\Rightarrow \alpha_b = 1.00$ (smallest value of α_d or $f_{ub}/f_u = 2.78$ or 1.0)
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 432.00$ kN, $f_u = 360.0$ N/mm², $t = 25.0$ mm, $d = 24.0$ mm
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 864.00$ 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: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 328.32$ kN, $f_u = 360.0$ N/mm², $t = 19.0$ mm, $d = 24.0$ mm
bolt 2:
in direction of load transfer: $\alpha_b = 1.00$
across to the direction of load transfer: $k_{1,i} = 1.4 \cdot p_2/d_0 - 1.7 = 6.38$ (inner bolt)
across to the direction of load transfer: $k_{1,a} = \min(2.8 \cdot e_2/d_0 - 1.7,\ 1.4 \cdot p_2/d_0 - 1.7) = 6.38$ (end bolt)
$\Rightarrow k_1 = 2.50$ (smallest value of k_1 or 2.5)
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 328.32$ kN, $f_u = 360.0$ N/mm², $t = 19.0$ mm, $d = 24.0$ mm
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 656.64$ kN

bearing resistance (3 rows)

$\Sigma F_{b,Rd,1} = 387.69$ kN
$\Sigma F_{b,Rd,2} = 656.64$ kN
$\Sigma F_{b,Rd,3} = 656.64$ kN

2.3. connection capacity

2.3.1. moment resistance

distance of tension-bolt-rows from centre of compression: $h_1 = 430.5$ mm, $h_2 = 310.5$ mm, $h_3 = 60.5$ 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: $F_{tr,Rd} = 379.3$ kN
row 2: $F_{tr,Rd} = 394.7$ kN
row 3: $F_{tr,Rd} = 394.7$ 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

row 1: $\Sigma F_{tr,Rd} = 0.0$ kN
Gk 3: $\Delta F_{tr,Rd} = F_{t,wc,Rd} - \Sigma F_{tr,Rd} = 714.8$ kN $F_{tr,Rd} = 379.3$ kN $< \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 379.3$ kN
Gk 4: $\Delta F_{tr,Rd} = F_{t,fc,Rd} - \Sigma F_{tr,Rd} = 717.9$ kN $F_{tr,Rd} = 379.3$ kN $< \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 379.3$ kN
row 2: $\Sigma F_{tr,Rd} = 379.3$ kN (row 1)
Gk 3: $\Delta F_{tr,Rd} = F_{t,wc,Rd} - \Sigma F_{tr,Rd} = 335.5$ kN $F_{tr,Rd} = 394.7$ kN $> \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 335.5$ kN
Gk 4: $\Delta F_{tr,Rd} = F_{t,fc,Rd} - \Sigma F_{tr,Rd} = 338.6$ kN $F_{tr,Rd} = 335.5$ kN $< \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 335.5$ kN

resistance per bolt-row (tension)

row 1: $F_{tr,Rd} = 379.3$ kN
row 2: $F_{tr,Rd} = 335.5$ kN
row 3: $F_{tr,Rd} = 394.7$ kN
$\Sigma F_{tr,Rd}^{*} = 1109.5$ 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
Gk 1: $\Delta F_{tr,Rd} = V_{wp,Rd}/\beta_j - \Sigma F_{tr,Rd} = 579.1$ kN $F_{tr,Rd} = 379.3$ kN $< \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 379.3$ kN
Gk 2: $\Delta F_{tr,Rd} = F_{c,w,Rd} - \Sigma F_{tr,Rd} = 558.2$ kN $F_{tr,Rd} = 379.3$ kN $< \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 379.3$ kN
Gk 7: $\Delta F_{tr,Rd} = F_{c,f,Rd} - \Sigma F_{tr,Rd} = 1622.8$ kN $F_{tr,Rd} = 379.3$ kN $< \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 379.3$ kN

row 2: $\Sigma F_{tr,Rd} = 379.3$ kN (row 1)
Gk 1: $\Delta F_{tr,Rd} = V_{wp,Rd}/\beta_j - \Sigma F_{tr,Rd} = 199.8$ kN $F_{tr,Rd} = 335.5$ kN $>$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 199.8$ kN
Gk 2: $\Delta F_{tr,Rd} = F_{c,w,Rd} - \Sigma F_{tr,Rd} = 179.0$ kN $F_{tr,Rd} = 199.8$ kN $>$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 179.0$ kN
Gk 7: $\Delta F_{tr,Rd} = F_{c,f,Rd} - \Sigma F_{tr,Rd} = 1243.5$ kN $F_{tr,Rd} = 179.0$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 179.0$ kN
row 3: $\Sigma F_{tr,Rd} = 558.2$ kN (rows 1 bis 2)
Gk 1: $\Delta F_{tr,Rd} = V_{wp,Rd}/\beta_j - \Sigma F_{tr,Rd} = 20.9$ kN $F_{tr,Rd} = 394.7$ kN $>$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 20.9$ kN
Gk 2: $\Delta F_{tr,Rd} = F_{c,w,Rd} - \Sigma F_{tr,Rd} = 0.0$ kN $F_{tr,Rd} = 20.9$ kN $>$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 0.0$ kN
Gk 7: $\Delta F_{tr,Rd} = F_{c,f,Rd} - \Sigma F_{tr,Rd} = 1064.6$ kN $F_{tr,Rd} = 0.0$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 0.0$ kN

check acc. to EC 3-1-8, 6.2.7.2(9)
decisive basic component: 10
row 1: $F_{tx,Rd} = 379.3$ kN, $h_x = 430.5$ mm $\Rightarrow$ $F_{tx,Rd} \leq \lim F_{tx,Rd} = 482.9$ kN, no deduction
row 2: $F_{tx,Rd} = 179.0$ kN, $h_x = 310.5$ mm $\Rightarrow$ $F_{tx,Rd} \leq \lim F_{tx,Rd} = 482.9$ kN, no deduction

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 379.3$ kN
row 2: $F_{tr,Rd} = 179.0$ kN
row 3: $F_{tr,Rd} = 0.0$ kN
$\Sigma F_{tr,Rd} = 558.2$ kN
potential failure by basic component 2, 3, 4, 5

resistance of flanges (compression)
$\Sigma F_{c,Rd}^{*} = 579.1$ kN

moment resistance regarding the centre of compression
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 218.9$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^{*} = 1109.5$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^{*} = 579.1$ kN

2.3.2. shear/bearing resistance

resistance per bolt-row
decisive basic components: 11, 12
row 1: $F_{vr,Rd} = 387.7$ kN
row 2: $F_{vr,Rd} = 434.3$ kN
row 3: $F_{vr,Rd} = 434.3$ kN

deductions depending on tension force (at 100% utilization of moment resistance)
decisive basic component: 10
row 1: $F_{vr,Rd} = f_{vt} \cdot 387.7$ kN $= 181.1$ kN with $f_{vt} = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd}) = 0.467$
row 2: $F_{vr,Rd} = f_{vt} \cdot 434.3$ kN $= 325.1$ kN with $f_{vt} = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd}) = 0.749$
row 3: $F_{vr,Rd} = f_{vt} \cdot 434.3$ kN $= 434.3$ kN with $f_{vt} = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd}) = 1.000$

resistance per bolt-row
row 1: $F_{vr,Rd} = 181.1$ kN
row 2: $F_{vr,Rd} = 325.1$ kN
row 3: $F_{vr,Rd} = 434.3$ kN
$\Sigma F_{vr,Rd} = 940.4$ kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd} = 940.4$ kN

2.3.3. shear resistance

shear resistance of column web
decisive basic component: 1
$V_{wp,Rd}/\beta_j = 579.1$ kN

2.3.4. total

$M_{j,Rd} = 218.9$ kNm $N_{j,t,Rd} = 1109.5$ kN $N_{j,c,Rd} = 579.1$ kN $V_{j,Rd} = 940.4$ kN $V_{wp,Rd}/\beta_j = 579.1$ kN

2.4. verifications

calculation of internal lever arm z_{eq} s. rotational stiffness

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

internal moment: $M_{Ed} = M_d = 200.00$ kNm
perpend. to connection plane
shear force: $V_{Ed} = |V_d| = 270.00$ kN
parallel to connection plane
shear force: $V_{c,w,Ed} = M_{d,w}/z - (V_{c1}-V_{c2})/2 = 553.66$ kN, $M_{d,w} = 205.1$ kNm, $z = 370.5$ mm
in column web

moment resistance
$M_{Ed}/M_{j,Rd} = 0.914 < 1$ ok
shear/bearing resistance at 100% utilization of moment resistance
$V_{Ed}/V_{j,Rd} = 0.287 < 1$ ok
shear resistance of column web
$V_{c,w,Ed}/(V_{wp,Rd}/\beta_j) = 0.956 < 1$ ok

2.4.2. verification result

maximum utilization: max $U = 0.956 < 1$ ok

2.5. rotational stiffness

stiffness coefficients
equivalent stiffness coefficient for 2 tension-bolt-rows:
effective stiffness coefficient for bolt-row 1:
$k_5 = 0.9 \cdot l_{eff} \cdot t_p^3 / m^3 = 30.72$ mm, $l_{eff} = 150.0$ mm, $m = 40.9$ mm
$k_{10} = 1.6 \cdot A_s / L_b = 8.13$ mm, $L_b = t_{ges} + 2 \cdot t_p + (t_k+t_m)/2 = 69.5$ mm, $t_{ges} = 44.0$ mm
$k_3 = 0.7 \cdot b_{eff,t,wc} \cdot t_{wc} / d_c = 10.56$ mm, $b_{eff,t,wc} = 285.4$ mm
$k_4 = 0.9 \cdot l_{eff} \cdot t_{tc}^3 / m^3 = 16.03$ mm, $l_{eff} = 285.4$ mm, $m = 47.9$ mm
$\Sigma(1/k_{i,1}) = 1/k_3 + 1/k_4 + 1/k_5 + 1/k_{10} = 0.313 \Rightarrow k_{eff,1} = 1 / \Sigma(1/k_{i,1}) = 3.198$ mm
effective stiffness coefficient for bolt-row 2:
$k_5 = 0.9 \cdot l_{eff} \cdot t_p^3 / m^3 = 20.22$ mm, $l_{eff} = 415.4$ mm, $m = 66.1$ mm
$k_{10} = 1.6 \cdot A_s / L_b = 8.13$ mm, $L_b = t_{ges} + 2 \cdot t_p + (t_k+t_m)/2 = 69.5$ mm, $t_{ges} = 44.0$ mm
$k_3 = 0.7 \cdot b_{eff,t,wc} \cdot t_{wc} / d_c = 10.56$ mm, $b_{eff,t,wc} = 285.4$ mm
$k_4 = 0.9 \cdot l_{eff} \cdot t_{tc}^3 / m^3 = 16.03$ mm, $l_{eff} = 285.4$ mm, $m = 47.9$ mm
$\Sigma(1/k_{i,2}) = 1/k_3 + 1/k_4 + 1/k_5 + 1/k_{10} = 0.330 \Rightarrow k_{eff,2} = 1 / \Sigma(1/k_{i,2}) = 3.034$ mm
equivalent internal lever arm: $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r) = 381.7$ mm
$k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq} = 6.075$ mm
stiffness coefficient of basic component 1:
$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z) = 4.86$ mm, $\beta = 1.0$, $z = 370.5$ mm
stiffness coefficient of basic component 2:
$k_2 = 0.7 \cdot b_{eff,c,wc} \cdot t_{wc} / d_c = 11.30$ mm, $b_{eff,c,wc} = 305.3$ mm
sum of stiffness coefficients $\Sigma(1/k_i) = 1/k_1 + 1/k_2 + 1/k_{eq} = 0.459$
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i) = 66722.9$ kNm/rad, $z = z_{eq} = 381.7$ mm
internal moment at the connection point: $M_{j,Ed} = M_{Ed} = 200.00$ kNm
$|M_{j,Ed}| = 200.00$ kNm $>$ 2/3 $M_{j,Rd} = 145.9$ kNm $\Rightarrow \mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi} = 2.343$, $\Psi = 2.7$
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu = 28474.0$ kNm/rad
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd} = 0.402°$

3. final result

maximum utilization:	max $U = 0.956 < 1$ ok
minimum rotational stiffness:	min $S_j = 28.5$ MNm/rad, $S_{j,ini} = 66.7$ MNm/rad, $\varphi_j = 0.402°$

verification succeeded

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4. Selected Design Parameters of the National Annex
DIN EN 1993-1-1 (EC 3)

chapter	value	definition
6.1(1)		partial 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

5. Regulations

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

DIN 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
DIN EN 1993-1-1/A1, Ergänzungen zur DIN EN 1993-1-1, Ausgabe Juli 2014
DIN EN 1993-1-1/NA, Nationaler Anhang zur DIN EN 1993-1-1, Ausgabe September 2017

DIN 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
DIN EN 1993-1-8/NA, Nationaler Anhang zur DIN EN 1993-1-8, Ausgabe Dezember 2010