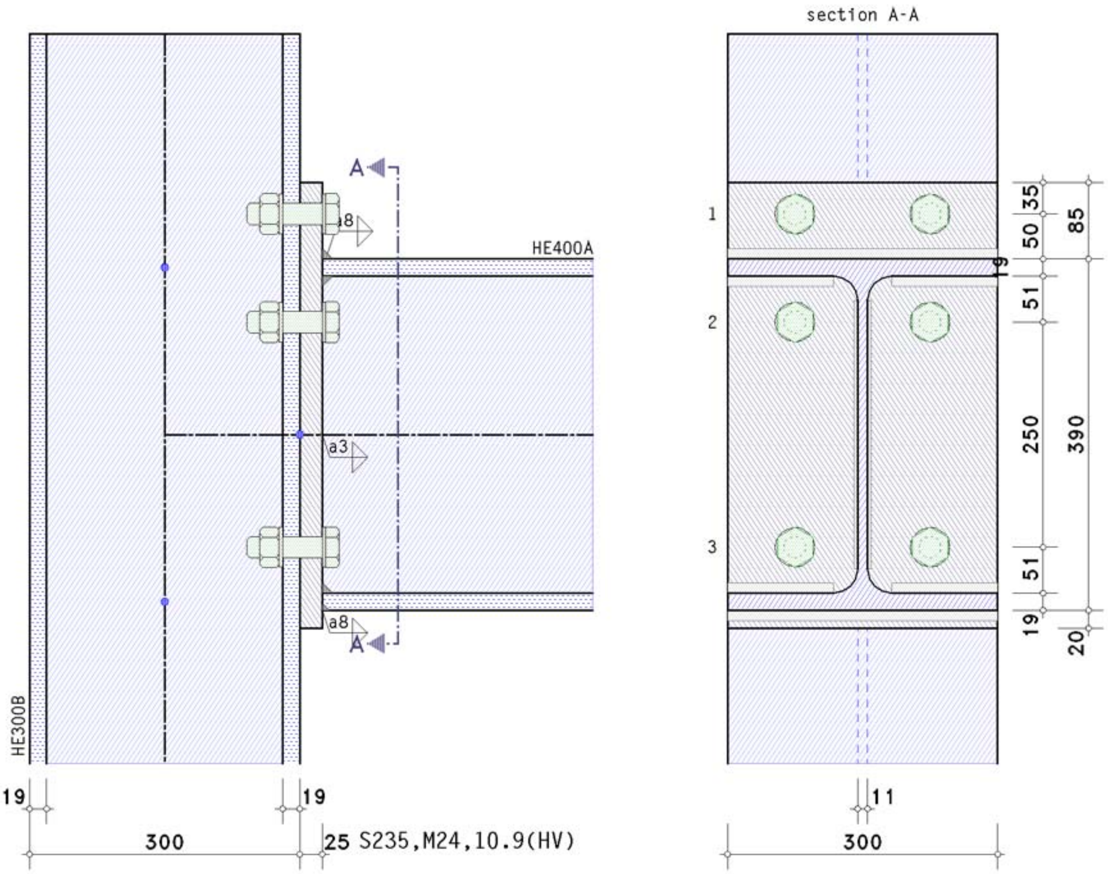
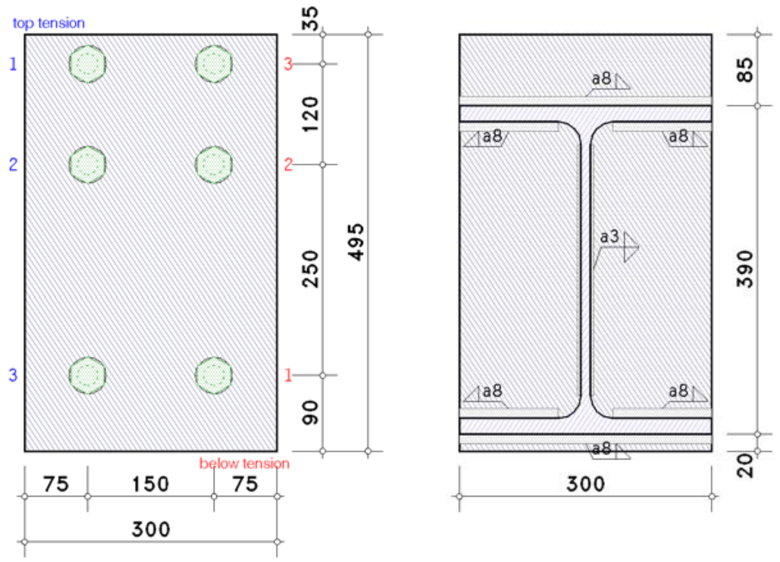
POS. 1: KAHLMEYER 3.6.4.1

Rigid beam connection EC 3-1-8 (04.25), NA: Deutschland 4H-EC3BT version: 10/2019-2w

1. input report



details (section A - A)



steel grade

steel grade S235

column parameters

section HE300B

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

beam parameters

section HE400A

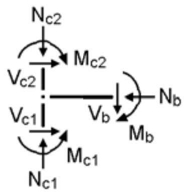
verification parameters

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
of these 2 bolt-rows top in tension (rows 1-2)
and 3 bolt-rows for shear transfer top (rows 1-3)
of these 1 bolt-row below in tension (row 3)
and 1 bolt-row for shear transfer below (row 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_{1-2} = 120.0 mm, p_{2-3} = 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 below: fillet weld, weld thickness a = 8.0 mm

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



Lc 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

notes
the following verification is applied to the connection of a girder to a continuous column.
Zur vollständigen Bemessung eines columnnendes sind ggf. weitere verifications required !
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.

check of data

ok

distances between bolts at end-plate

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

bolt distance from column edge

horizontal:	e_2 = 75.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_2 = 75.0 mm < $4 \cdot t$ + 40 mm = 116.0 mm

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

2. table of results

utilization/rotation

Lc	U_m	U_v	U_{wp}	U_{ep}	U	$S_{j,ini}$	S_j	φ_j
--	---	---	---	---	---	MNm/rad	MNm/rad	°
1	1.073	0.263	1.113	0.727	1.113*	77.8	21.5	0.533

U_m: utilization due to bending; U_v: utilization due to shear/bearing resistance; U_{wp}: utilization due to shear in column web
U_{ep}: utilization due to shear in end-plate; 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:	max U = 1.113 > 1 not ok !!
minimum rotational stiffness:	min S_j = 21.5 MNm/rad, $S_{j,ini}$ = 77.8 MNm/rad, φ_j = 0.533°

resistance not ensured !!

total loading capacity of the connection may not be guaranteed (s. notes) !

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;

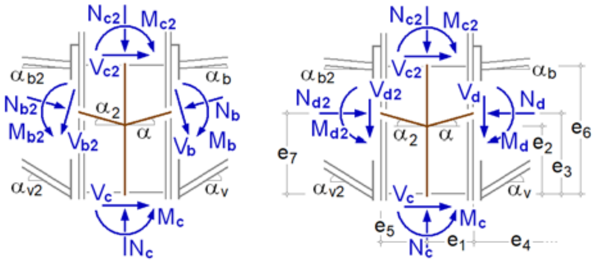
Deutsche Fassung EN 1993-1-1:2022, Ausgabe April 2025
EN 1993-1-1/NA, Nationaler appendix zur EN 1993-1-1, Ausgabe Oktober 2022

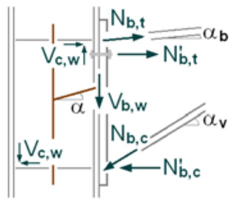
EN 1993-1-8, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-8: Bemessung von Anschlüssen;
Deutsche Fassung EN 1993-1-8:2024, Ausgabe April 2025
EN 1993-1-8/NA, Nationaler appendix zur EN 1993-1-8, Ausgabe November 2020

5. Lc 1 (decisive)

5.1. design values

`periphery connection ⊥ zur connection plane`   `partial internal forces and moments`





slope angle: $\alpha_b = \alpha = \alpha_v = 0°$
distance: $e_1 = 150.0$ mm, $e_3 = 185.5$ mm, $e_2 = 185.5$ mm, $e_6 = 371.0$ mm

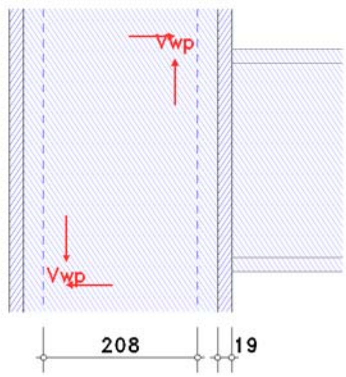
internal forces and moments perpendicular to the connection planes
periphery beam
$M_d = 200.00$ kNm, $V_d = 270.00$ kN

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_p = 193.25$ 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
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = 0.00$ kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v) = 0.00$ kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 270.00$ kN

5.2. basic components

5.2.1. bc 1: Column web panel in shear

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = 1.00 \leq 2$ for $M_{j1} = 200.00$ kNm ($M_{j2} = 0$)



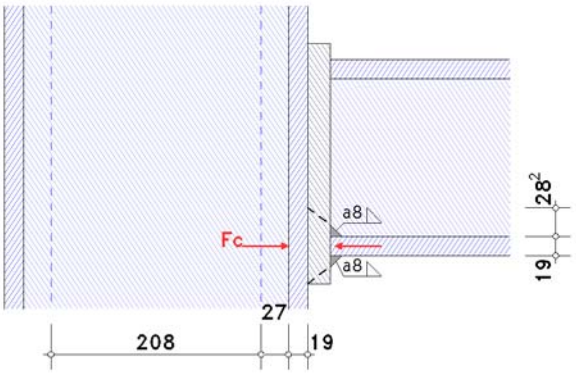
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

slenderness of column web $h_{wc}/t_{wc} = 23.82 < 72 \cdot \varepsilon/\eta = 60.00 \Rightarrow$ method applicable
plastic shear resistance $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_{wp}) / (3^{1/2} \cdot \gamma_{M0}) = 402.96$ kN
Beitrag of column flange:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/z_{wp} = 82.0$ kN, $z_{wp} = h_r = 310.5$ mm
plastic shear resistance plus Beitrag of column flange $V_{wp,Rd} = 484.9$ kN

5.2.2. bc 2: column web in transverse compression

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = 1.00 \le 2$ for $M_{j1} = 200.00$ kNm ($M_{j2} = 0$)

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

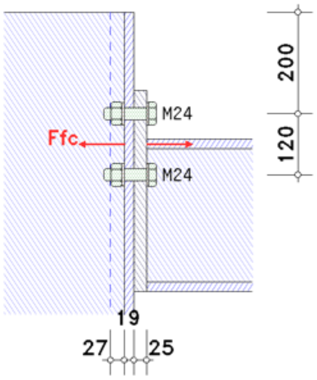


effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p = 305.3$ mm
reduction factor $k_w = 1.0$ ($\sigma_{com,Ed} = 0$)
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.714$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.969$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.653$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 515.23$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 453.79$ kN (decisive)

resistance of the upper beam flange:
effective width of web in transverse compression $b_{eff,c} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_p + 5 \cdot (t_{fc}+s_c) + s_p = 321.6$ mm
reduction factor $k_w = 1.0$ ($\sigma_{com,Ed} = 0$)
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.733$
reduction factor for web buckling $\rho = (\lambda_p - 0.22) / \lambda_p^2 = 0.955$ for $\lambda_p > 0.673$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.633$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 526.44$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 456.92$ kN (decisive)

5.2.3. bc 4: column flange in bending

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



equivalent T-stub flange (each individual bolt-row):
here: number of bolt-rows $n_b = 1$
row 1
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 285.4$ mm, $l_{eff,cp} = 301.0$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 285.4$ mm
tension resistance of the T-stub flange:
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
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 607.79$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 394.31$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 394.31$ kN
row 2
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 285.4$ mm, $l_{eff,cp} = 301.0$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 285.4$ mm
tension resistance of the T-stub flange:
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

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 607.79$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 394.31$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 394.31$ kN

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

$F_{t,fc,Rd,1} = 394.31$ kN, $l_{eff,1} = 285.4$ mm
$F_{t,fc,Rd,2} = 394.31$ kN, $l_{eff,2} = 285.4$ mm

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

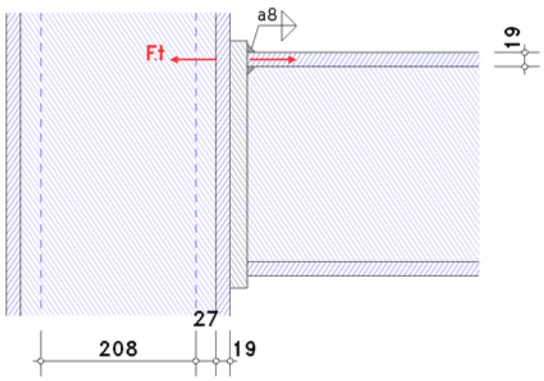
here: number of bolt-rows $n_b = 2$

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:
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
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 1015.20$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 863.38$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 723.54$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 1015.20$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 723.54$ kN
effective length: $\Sigma l_{eff} = 405.4$ mm, 2 rows

5.2.4. bc 3: column web in transverse tension

transformation parameter (EC 3-1-8, 7.2.3(4)) $\beta_j = 1.00 \leq 2$ 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
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.678$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 500.03$ kN, $b_{eff,t,wc} = 285.4$ mm

row 2
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.678$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 500.03$ kN, $b_{eff,t,wc} = 285.4$ mm

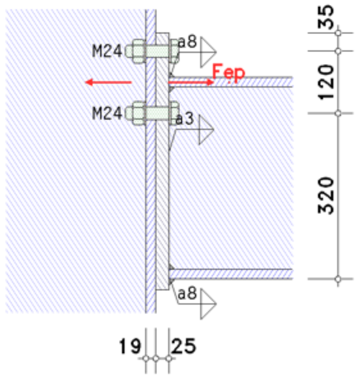
resistance of a column web with transverse tension (per bolt-row)

$F_{t,wc,Rd,1} = 500.03$ kN, $b_{eff,t,wc} = 285.4$ mm (s. bc 4)
$F_{t,wc,Rd,2} = 500.03$ kN, $b_{eff,t,wc} = 285.4$ mm (s. bc 4)

each group of bolt-rows:

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.544$
resistance eines unstiffeneden column webs with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t,wc} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 570.51$ kN, $b_{eff,t,wc} = 405.4$ mm

5.2.5. bc 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).

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 150.0$ mm, $l_{eff,cp} = 257.3$ mm

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

tension resistance of the T-stub flange:

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

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 699.67$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 378.96$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 378.96$ kN

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

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 610.94$ kN ($\geq$ 378.96 kN, not decisive)

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

$F_{t,ep,Rd,1} = 378.96$ 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

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

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

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 434.0$ mm

tension resistance of the T-stub flange:

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

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 15.94$ kNm

$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 507.60$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 1053.99$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 495.69$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 507.60$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 495.69$ kN

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

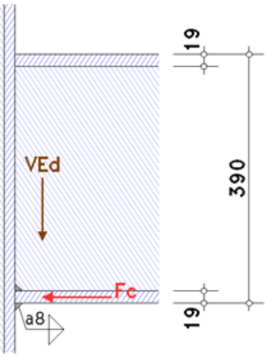
tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 634.39$ kN ($\geq$ 495.69 kN, not decisive)

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

$F_{ep,Rd,2} = 495.69$ kN, $l_{eff,2} = 415.4$ mm

5.2.6. bc 7: beam flange and web in compression

flange below: section class 1
web: section class 1
total: section class 1
section class of the beam: 1
taking into account the moment-shear force-interaction V_{Ed} = 270.0 kN

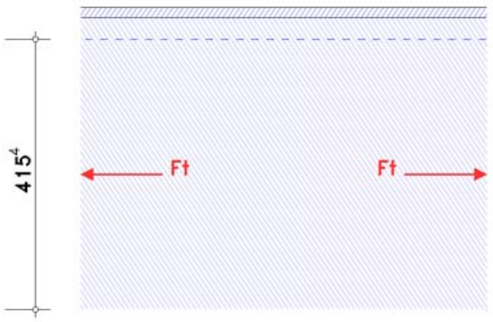


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

stress due to bending with shear force: $V_{Ed} = 270.0$ kN ≤ 388.9 kN $= 0.5 \cdot V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 601.97$ kNm, $W_{pl} = 2561.57$ cm^3
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 1622.56$ kN
resistance of welds
compression force $N_{c,Ed} = 1622.6$ kN
$\sigma_{1,w,Ed} = 478.0$ N/mm^2 $>$ $f_{1w,d} = 360.0$ N/mm^2 $\Rightarrow$ U = 1.328 $>$ 1
$\sigma_{2,w,Ed} = 239.0$ N/mm^2 $<$ $f_{2w,d} = 259.2$ N/mm^2 $\Rightarrow$ U = 0.922 $<$ 1
$\Rightarrow$ $F_{c,f,Rd}$ / max U = 1221.88 kN

resistance of the upper beam flange:
stress due to bending with shear force: $V_{Ed} = 270.0$ kN ≤ 388.9 kN $= 0.5 \cdot V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 601.97$ kNm, $W_{pl} = 2561.57$ cm^3
resistance of flange and web in compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 1622.56$ kN
resistance of welds
compression force $N_{c,Ed} = 1622.6$ kN
$\sigma_{1,w,Ed} = 478.0$ N/mm^2 $>$ $f_{1w,d} = 360.0$ N/mm^2 $\Rightarrow$ U = 1.328 $>$ 1
$\sigma_{2,w,Ed} = 239.0$ N/mm^2 $<$ $f_{2w,d} = 259.2$ N/mm^2 $\Rightarrow$ U = 0.922 $<$ 1
$\Rightarrow$ $F_{c,f,Rd}$ / max U = 1221.88 kN

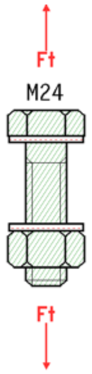
5.2.7. bc 8: beam web in tension



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

each individual bolt-row:
row 2
effective width $b_{eff,t,wb} = 415.4$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 1073.69$ kN
resistance of a fillet weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm^2
tension resistance of welds: $F_{t,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot b_{eff,t} = 634.39$ kN
total loading capacity of plate: $F_{t,wb,Rd} = F_{t,w,Rd} = 634.39$ kN
resistance of a beam web in tension (per bolt-row)
$F_{t,wb,Rd,2} = 634.39$ kN, $b_{eff,t,wb} = 415.4$ mm (s. bc 5)

5.2.8. bc 10: bolts in tension

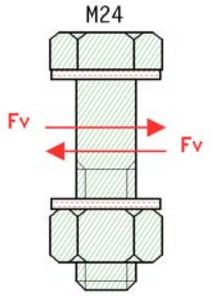


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

tension resistance of one bolt $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 253.80$ kN, $k_2 = 0.90$
punching shear load capicity of a bolt $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 444.55$ kN, $t_p = 19.0$ mm
tension-/punching shear load capicity for 2 bolts: $\Sigma F_{tp,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 507.60$ kN

5.2.9. bc 11: bolts in shear

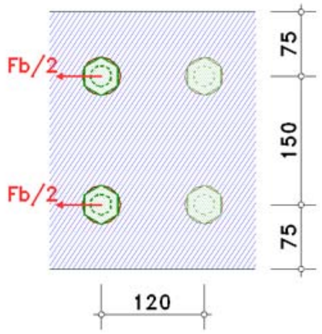
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



shear resistance $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2} = 217.15$ kN, $\alpha_v = 0.60$
shear resistance of 2 bolts (1-shear): $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd} = 434.29$ kN

5.2.10. bc 12: plate with bearing resistance

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



row 1

end-plate:

bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 232.62$ kN, $k_m = 1.00$, $\alpha_b = 1.35$
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 232.62$ kN, $k_m = 1.00$, $\alpha_b = 1.35$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 465.23$ kN

shear block of groups of bolts

shearversagen in Kombination in tensionversagen of plate

failure type 1:
tension-shear resistance $V_{eff,Rd} = (A_{nt} \cdot f_u + \min(A_{gv} \cdot f_y, A_{nv} \cdot f_u)/3^{1/2}) / \gamma_{M2} = 984.25$ kN
failure type 2:
tension-shear resistance $V_{eff,Rd} = (A_{nt} \cdot f_u + \min(A_{gv} \cdot f_y, A_{nv} \cdot f_u)/3^{1/2}) / \gamma_{M2} = 984.25$ kN
failure type 3:
tension-shear resistance $V_{eff,Rd} = (0.5 \cdot A_{nt} \cdot f_u + \min(A_{gv} \cdot f_y, A_{nv} \cdot f_u)/3^{1/2}) / \gamma_{M2} = 714.25$ kN
failure type 4:
tension resistance (without shearanteil) $V_{eff,Rd} = (A_{nt} \cdot f_u) / \gamma_{M2} = 1785.60$ kN
resistance due to shear block: min $V_{eff,Rd} = 714.3$ kN

total

bearing resistance incl. shear block: $\min(\Sigma F_{b,Rd}, \min V_{eff,Rd}) = 465.2$ kN

column flange:

bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 393.98$ kN, $k_m = 1.00$, $\alpha_b = 3.00$
bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 393.98$ kN, $k_m = 1.00$, $\alpha_b = 3.00$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 787.97$ kN

row 2

end-plate:

bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 518.40 kN, k_m = 1.00, α_b = 3.00

bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 518.40 kN, k_m = 1.00, α_b = 3.00

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 1036.80 kN

column flange:

bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 393.98 kN, k_m = 1.00, α_b = 3.00

bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 393.98 kN, k_m = 1.00, α_b = 3.00

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 787.97 kN

row 3

end-plate:

bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 518.40 kN, k_m = 1.00, α_b = 3.00

bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 518.40 kN, k_m = 1.00, α_b = 3.00

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 1036.80 kN

column flange:

bolt 1: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 393.98 kN, k_m = 1.00, α_b = 3.00

bolt 2: bearing resistance $F_{b,Rd} = (k_m \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2}$ = 393.98 kN, k_m = 1.00, α_b = 3.00

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd}$ = 787.97 kN

bearing resistance (3 rows)

$\Sigma F_{b,Rd,1}$ = 465.23 kN

$\Sigma F_{b,Rd,2}$ = 787.97 kN

$\Sigma F_{b,Rd,3}$ = 787.97 kN

5.3. connection capacity

transformation parameter: β_j = 1.00

5.3.1. moment resistance

distance of tension-bolt-rows from centre of compression: h_1 = 430.5 mm, h_2 = 310.5 mm

resistances acc. to EC 3-1-8, B.3.2.2(6) for bolt-rows considered individually

decisive basic components: 3, 4, 5, 8

row 1: $F_{tr,Rd}$ = 379.0 kN

row 2: $F_{tr,Rd}$ = 394.3 kN

deductions acc. to EC 3-1-8, B.3.2.2(8) for bolt-rows as part of a group (column)

decisive basic components: 3, 4

row 1: $F_{tr,Rd}$ = 379.0 kN

row 2: $F_{tr,Rd}$ = 191.5 kN

resistance per bolt-row (tension)

row 1: $F_{tr,Rd}$ = 379.0 kN

row 2: $F_{tr,Rd}$ = 191.5 kN

$\Sigma F_{tr,Rd}^*$ = 570.5 kN

deductions acc. to EC 3-1-8, B.3.2.2(7)

decisive basic components: 1, 2, 7

row 1: $F_{tr,Rd}$ = 379.0 kN

row 2: $F_{tr,Rd}$ = 74.8 kN

check acc. to EC 3-1-8, B.3.2.2(9)

decisive basic component: 10

row 1: $F_{tr,Rd}$ = 379.0 kN

resistance per bolt-row (bending)

row 1: $F_{tr,Rd}$ = 379.0 kN

row 2: $F_{tr,Rd}$ = 74.8 kN

$\Sigma F_{tr,Rd}$ = 453.8 kN

potential failure by basic component 2, 3, 4, 5

resistance of flanges (compression)

$\Sigma F_{c,Rd}^*$ = 907.6 kN

moment resistance

$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r)$ = 186.4 kNm

tension resistance

$N_{j,t,Rd} = \Sigma F_{tr,Rd}^*$ = 570.5 kN

compression resistance

$N_{j,c,Rd} = \Sigma F_{c,Rd}^*$ = 907.6 kN

5.3.2. shear/bearing resistance

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

deductions depending on tension force (at full utilization of moment resistance)
decisive basic component: 10
row 1: $F_{vr,Rd}$ = 202.7 kN
row 2: $F_{vr,Rd}$ = 388.6 kN
row 3: $F_{vr,Rd}$ = 434.3 kN

resistance per bolt-row (shear/bearing resistance)
row 1: $F_{vr,Rd}$ = 202.7 kN
row 2: $F_{vr,Rd}$ = 388.6 kN
row 3: $F_{vr,Rd}$ = 434.3 kN
$\Sigma F_{vr,Rd}$ = 1025.6 kN

shear/bearing resistance
$V_{j,Rd} = \Sigma F_{vr,Rd}$ = 1025.6 kN

5.3.3. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd}$ = 1010.80 kN
welds: $F_{w,Rd}$ = 371.63 kN
shear resistance: $V_{ep,Rd} = F_{w,Rd}$ = 371.63 kN

shear resistance of column web
decisive basic component: 1
$V_{wp,Rd}$ = 484.93 kN

5.3.4. total

$M_{j,Rd}$ = 186.4 kNm $N_{j,t,Rd}$ = 570.5 kN $N_{j,c,Rd}$ = 907.6 kN $V_{j,Rd}$ = 1025.6 kN $V_{wp,Rd}$ = 484.9 kN $V_{ep,Rd}$ = 371.6 kN

5.4. verifications

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

internal moment: $M_{Ed} = M_d$ = 200.00 kNm
shear force: $V_{Ed} = |V_d|$ = 270.00 kN
shear force: $V_{c,w,Ed} = M_d/z - (V_{c1}-V_{c2})/2$ = 539.81 kN, z = 370.5 mm
shear force: $V_{b,w,Ed}$ = 270.00 kN

$M_{Ed}/M_{j,Rd}$ = 1.073 > 1 not ok !!
$V_{Ed}/V_{j,Rd}$ = 0.263 < 1 ok
$V_{c,w,Ed}/V_{wp,Rd}$ = 1.113 > 1 not ok !!
$V_{b,w,Ed}/V_{ep,Rd}$ = 0.727 < 1 ok

5.4.2. verification result

maximum utilization: max U = 1.113 > 1 not ok !!
failure at verification of bending: U = 1.073
failure at verification shear in column web: U = 1.113

5.5. rotational stiffness

stiffness coefficients
equivalent stiffness coefficient for 2 tension-bolt-rows:
1: k_3 = 10.56 mm, k_4 = 16.03 mm, k_5 = 30.72 mm ⇒ $k_{eff,1} = 1 / \Sigma(1/k_{i,1})$ = 5.274 mm
2: k_3 = 10.56 mm, k_4 = 16.03 mm, k_5 = 20.22 mm ⇒ $k_{eff,2} = 1 / \Sigma(1/k_{i,2})$ = 4.842 mm
equivalent internal lever arm $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r)$ = 382.69 mm
äquivalenter stiffness coefficient $k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq}$ = 9.862 mm
$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z)$ = 4.86 mm
$k_2 = 0.7 \cdot b_{eff,c,wc} \cdot t_{wc} / d_c$ = 11.30 mm
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i)$ = 77772.0 kNm/rad, $z = z_{eq}$ = 382.7 mm, $\Sigma(1/k_i) = 0.395\ mm^{-1}$
$|M_{j,Ed}|$ = 200.00 kNm > 2/3 $M_{j,Rd}$ = 124.25 kNm ⇒ $\mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi}$ = 3.616, Ψ = 2.7
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu$ = 21510.5 kNm/rad
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd}$ = 0.533°