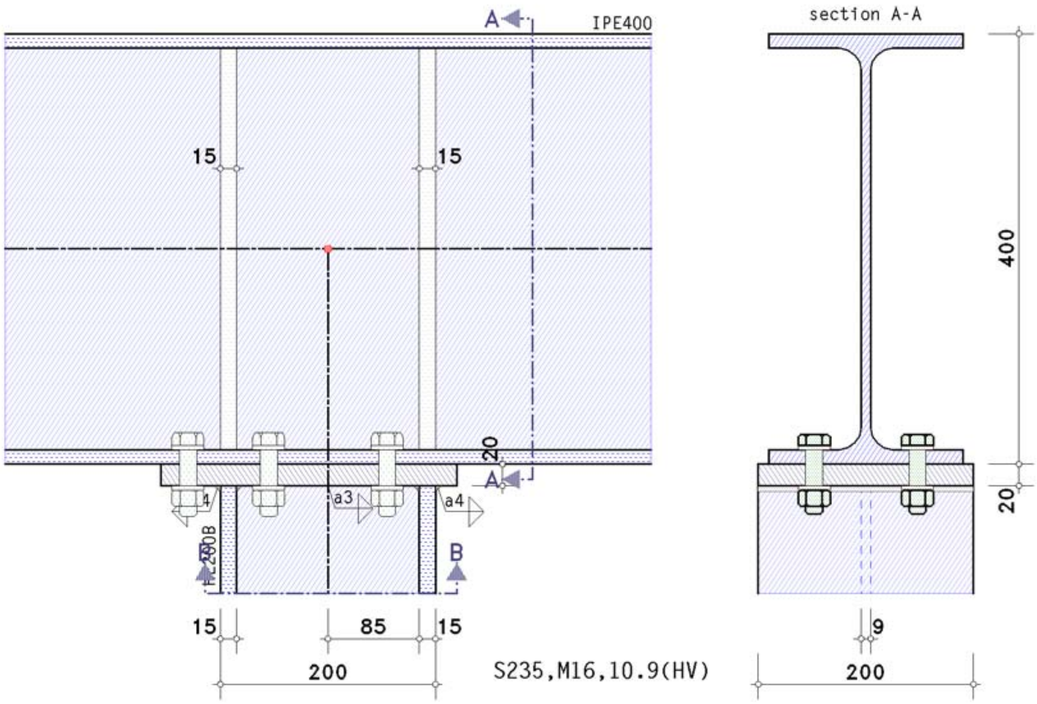
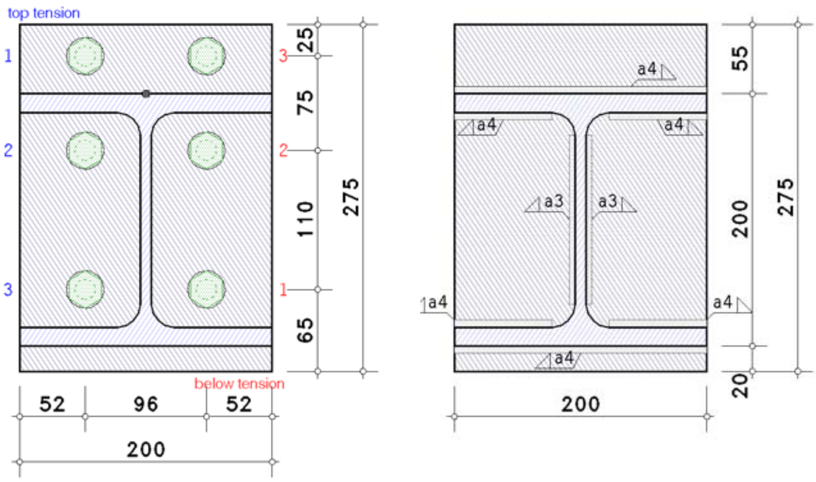
POS. 7: KAHLMEYER 7.3.1

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

1. input report



details (section B-B)



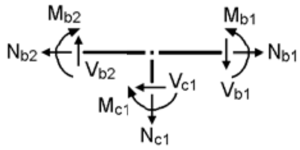
steel grade
steel grade S235
column parameters
section HE200B
slope angle of top of column towards the horizontal $\alpha_c = 0.0°$
bolts
bolt class 10.9, bolt size M16
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}{}^* = 0.7 \cdot f_{yb} \cdot A_s = 98.7$ kN)
shear plane passes through the unthreaded portion of the bolt
beam parameters
section IPE400
reinforcement of the section with transverse stiffeners (distance $d_{st} = 185.0$ mm):
thickness $t_{st} = 15.0$ mm, width $b_{st} = 80.0$ mm, length $l_{st} = 373.0$ mm
recess at stiffeners $c_{st} = 31.5$ mm
verification parameters
bolted end-plate connection (horizontal)
thickness $t_p = 20.0$ mm, width $b_p = 200.0$ mm, length $l_p = 275.0$ mm
projections $h_{p,o} = 55.0$ mm, $h_{p,u} = 20.0$ mm
bolts in connection:
3 bolt-rows with 2 bolts
of these 2 bolt-rows left (top) in tension (rows 1-2)

and 3 bolt-rows for shear transfer left (top) (rows 1-3)
of these 1 bolt-row right (below) in tension (row 3)
and 3 bolt-rows for shear transfer right (below) (rows 1-3)
centre distance of the bolts to the lateral edge of the end-plate e_2 = 52.0 mm
centre distance of the first bolt-row to the left (upper) edge of end plate (end row) e_o = 25.0 mm
centre distance of the last bolt-row to the right (lower) edge of end plate (end row) e_u = 65.0 mm
centre distance of the bolt-rows from each other p_{1-2} = 75.0 mm, p_{2-3} = 110.0 mm

welds at the connection point:
column flange left (top): fillet weld, weld thickness a = 4.0 mm
column web: fillet weld, weld thickness a = 3.0 mm
column flange right (below): fillet weld, weld thickness a = 4.0 mm

internal forces and moments in the intersection point of system axes

Lc 1: $N_{j,b1,Ed}$ = -3.43 kN $M_{j,b1,Ed}$ = -47.56 kNm $V_{j,b1,Ed}$ = 87.57 kN (right)



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

calculation of a joint with horizontal connection (variant 2) is made for the rotated model,
hereafter, beam and column and their internal forces and moments are changed.
all notes and messages refer to this calculation model.
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.
calculation of T-stub-resistance with the standard method.

check of data

ok

distances between bolts at end-plate

horizontal:	e_2 = 52.0 mm > 1.2·d_0 = 21.6 mm,	e_2 = 52.0 mm < 4·t + 40 mm = 94.0 mm
horizontal:	p_2 = 96.0 mm > 2.4·d_0 = 43.2 mm,	p_2 = 96.0 mm < min(14·t, 200 mm) = 189.0 mm
top-below:	e_1 = 25.0 mm > 1.2·d_0 = 21.6 mm,	e_1 = 25.0 mm < 4·t + 40 mm = 94.0 mm
top-below:	p_1 = 75.0 mm > 2.2·d_0 = 39.6 mm,	p_1 = 75.0 mm < min(14·t, 200 mm) = 189.0 mm
top-below:	p_1 = 110.0 mm > 2.2·d_0 = 39.6 mm,	p_1 = 110.0 mm < min(14·t, 200 mm) = 189.0 mm
top-below:	e_1 = 65.0 mm > 1.2·d_0 = 21.6 mm,	e_1 = 65.0 mm < 4·t + 40 mm = 94.0 mm

bolt distance from column edge

horizontal:	e_2 = 42.0 mm > 1.2·d_0 = 21.6 mm,	e_2 = 42.0 mm < 4·t + 40 mm = 94.0 mm

2. table of results

utilization

Lc	U_m	U_v	U_{wp}	U_{bw}	U_{ep}	U_{sb}	U_{ss}	U_{sw}	U
--	---	---	---	---	---	---	---	---	---
1	0.562	0.008	0.400	0.021	0.009	1.090	0.646	0.485	1.090*

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_{ss}: utilization due to stiffeners/ribs; U_{sw}: utilization due to shearfeld; U: utilization of the connection
*) maximum utilization

3. final result

maximum utilization: max U = 1.090 > 1 not ok !!

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

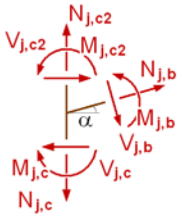
EN 1993-1-8, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-8: Bemessung von Anschlüssen; Deutsche Fassung EN 1993-1-8:2024, Ausgabe April 2025

5. Lc 1 (decisive)

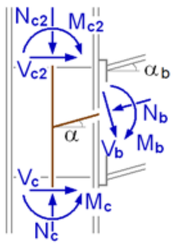
calculation model: verticalr beam-column-connection

5.1. design values

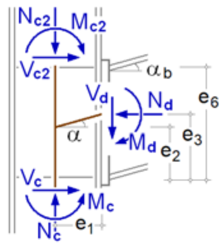
internal forces at node



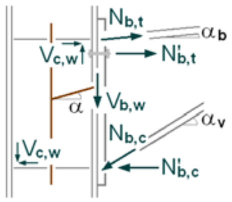
periphery connection



⊥ zur connection plane



partial internal forces and moments



slope angle: $\alpha_b = \alpha = \alpha_v = 0°$
distances: $e_1 = 200.0$ mm, $e_3 = 92.5$ mm, $e_2 = 92.5$ mm, $e_6 = 185.0$ mm

internal forces and moments perpendicular to the connection planes
periphery beam
$N_d = 87.57$ kN, $M_d = 46.87$ kNm, $V_d = 3.43$ kN
periphery column (below)
$N_c = 3.43$ kN, $M_c = 39.46$ kNm, $V_c = 87.57$ kN

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_p = 46.81$ kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b = 209.22$ kN, $z_b = 185.0$ mm, $z_{bu} = 92.5$ mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b = 296.79$ kN, $z_b = 185.0$ mm, $z_{bo} = 92.5$ mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = 0.00$ kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_b) = 0.00$ kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 3.43$ 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: $h_1 = 222.5$ mm, $h_2 = 147.5$ mm

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 186.5$ kN
row 2: $F_{tr,Rd} = 186.5$ kN
$\Sigma F_{tr,Rd}^* = 373.0$ kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 186.5$ kN
row 2: $F_{tr,Rd} = 186.5$ kN
$\Sigma F_{tr,Rd} = 373.0$ kN
potential failure by basic component 4

resistance of flanges (compression)
$\Sigma F_{c,Rd}^* = 1049.2$ kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 69.0$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 373.0$ kN
compression resistance
$N_{j,c,Rd} = \Sigma F_{c,Rd}^* = 1049.2$ kN

5.2.2. shear/bearing resistance

resistance per bolt-row (shear/bearing resistance)
row 1: $F_{vr,Rd} = 79.1$ kN
row 2: $F_{vr,Rd} = 79.1$ kN
row 3: $F_{vr,Rd} = 193.0$ kN
$\Sigma F_{vr,Rd} = 351.1$ kN

shear/bearing resistance

$V_{j,Rd} = \Sigma F_{vr,Rd} = 351.1$ kN

5.2.3. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd} = 363.62$ kN

shear resistance of the beam web
beam web: $V_{bw,Rd} = 163.63$ kN

shear resistance of column web
$V_{wp,Rd} = 524.59$ kN

5.2.4. total

$M_{j,Rd} = 69.0$ kNm $N_{j,t,Rd} = 373.0$ kN $N_{j,c,Rd} = 1049.2$ kN $V_{j,Rd} = 351.1$ kN $V_{wp,Rd} = 524.6$ kN $V_{ep,Rd} = 363.6$ kN $V_{bw,Rd} = 163.6$ kN

5.3. verifications

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

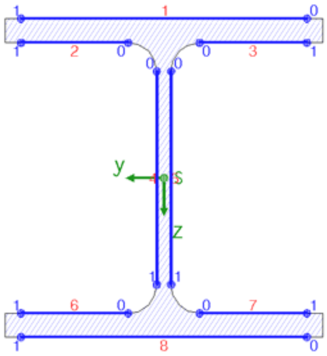
axial force:	$N_{b,Ed} = \lvert N_d \rvert = 87.57$ kN $<$ $5\% \cdot N_{pl,Rd} = 91.75$ kN $\Rightarrow$ moment resistance
internal moment:	$M_{Ed} = M_d - N_d \cdot z_{bu} = 38.77$ kNm, $z_{bu} = 92.5$ mm
shear force:	$V_{Ed} = \lvert V_d \rvert = 3.43$ kN
shear force:	$V_{c,w,Ed} = M_d/z - (V_{c1}-V_{c2})/2 = 209.59$ kN, $z = 185.0$ mm
shear force:	$V_{b,w,Ed} = 3.43$ kN

$M_{Ed}/M_{j,Rd} = 0.562 < 1$ ok
$V_{Ed}/V_{j,Rd} = 0.010 < 1$ ok
$V_{c,w,Ed}/V_{wp,Rd} = 0.400 < 1$ ok
$V_{b,w,Ed}/V_{bw,Rd} = 0.021 < 1$ ok
$V_{b,w,Ed}/V_{ep,Rd} = 0.009 < 1$ 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:	$a_w = 4.0$ mm	$l_w = 180.0$ mm
weld 2:	$a_w = 4.0$ mm	$l_w = 67.5$ mm
weld 3:	see weld 2	
weld 4:	$a_w = 3.0$ mm	$l_w = 134.0$ mm
weld 5:	see weld 4	
weld 6:	$a_w = 4.0$ mm	$l_w = 67.5$ mm
weld 7:	see weld 6	
weld 8:	$a_w = 4.0$ mm	$l_w = 180.0$ mm

design values refering to centroid of the section:
$N_{Ed} = -87.57$ kN, $M_{y,Ed} = -46.87$ kNm, $V_{z,Ed} = 3.43$ kN
cross-sectional properties refering to centroid of the line cross-section:
$\Sigma A_w = 33.24$ cm², $A_{w,z} = 8.04$ cm², $\Sigma l_w = 89.8$ cm
$I_{w,y} = 2340.61$ cm⁴, $I_{w,z} = 773.15$ cm⁴, $\Delta z_w = 0.0$ mm
verifications in weld edges:

weld 1,	pt. 0:	$\sigma_{w,x} = 173.92$ N/mm²		$\Rightarrow U_w = 0.837 < 1$ ok
weld 2,	pt. 0:	$\sigma_{w,x} = 143.88$ N/mm²		$\Rightarrow U_w = 0.692 < 1$ ok
weld 4,	pt. 0:	$\sigma_{w,x} = 107.83$ N/mm²	$\tau_{w,z} = 4.27$ N/mm²	$\Rightarrow U_w = 0.519 < 1$ ok
	pt. 1:	$\sigma_{w,x} = -160.52$ N/mm²	$\tau_{w,z} = 4.27$ N/mm²	$\Rightarrow U_w = 0.773 < 1$ ok
weld 6,	pt. 0:	$\sigma_{w,x} = -196.57$ N/mm²		$\Rightarrow U_w = 0.946 < 1$ ok
weld 8,	pt. 0:	$\sigma_{w,x} = -226.61$ N/mm²		$\Rightarrow U_w = 1.090 > 1$ not ok !!

Result:
weld 8, pt. 0: $\sigma_{w,x} = -226.61$ N/mm²
Max: $F_{w,Ed} = 906.44$ kN/m $>$ $F_{w,Rd} = 831.38$ kN/m $\Rightarrow U_w = 1.090 > 1$ not ok !!

5.3.3. verification of web stiffeners

compression stiffener (below)

$F_{c,Ed}$ = 297.28 kN

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r \cdot t_w)/b_f$ = 106.86 kN, $H = F \cdot e_F/e_H$ = 15.97 kN

assumption: stiffeners do not buckle: section class 1, section class 1 ≤ 3 ok

cross-section at flange

compression resistance $N_{c,Rd}$ = 170.96 kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 110.38 kN

F_{Ed} = 110.38 kN < F_{Rd} = 170.96 kN ⇒ U = 0.646 < 1 ok

cross-section at web

shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0})$ = 630.90 kN

design value: $F_{Ed} = F$ = 106.86 kN

F_{Ed} = 106.86 kN < F_{Rd} = 630.90 kN ⇒ U = 0.169 < 1 ok

stiffener in tension (top)

$F_{t,Ed}$ = 209.71 kN

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r \cdot t_w)/b_f$ = 75.38 kN, $H = F \cdot e_F/e_H$ = 11.27 kN

cross-section at flange

tension resistance $N_{t,Rd}$ = 170.96 kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 77.87 kN

F_{Ed} = 77.87 kN < F_{Rd} = 170.96 kN ⇒ U = 0.455 < 1 ok

cross-section at web

shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0})$ = 630.90 kN

design value: $F_{Ed} = F$ = 75.38 kN

F_{Ed} = 75.38 kN < F_{Rd} = 630.90 kN ⇒ U = 0.119 < 1 ok

5.3.4. elastic verification of the shear area

column web

requirements concerning stiffeners: s. verification of web stiffeners

requirements concerning shear area: shear buckling: h_p/t_p = 43.37 ≤ $72 \cdot \varepsilon/\eta$ = 60.00 ok

internal forces and moments at web (sign definition of statics):

N_3 = -3.43 kN, M_3 = -39.46 kNm, V_3 = -87.57 kN

N_4 = -87.57 kN, M_4 = -46.90 kNm, V_4 = 3.43 kN

dimensions of the shear area: h_b = 373.0 mm, h_t = 373.0 mm, h_l = 177.5 mm, h_r = 177.5 mm

stresses within the shear area:

τ_b = 65.3 N/mm², τ_t = 65.3 N/mm², τ_l = 65.8 N/mm², τ_r = 65.8 N/mm²

verification of the shear area:

max τ_{Ed} = 65.8 N/mm² < τ_{Rd} = 135.7 N/mm² ⇒ U = 0.485 < 1 ok

5.3.5. verification result

maximum utilization: max U = 1.090 > 1 not ok !!

failure at verification of welds: U = 1.090