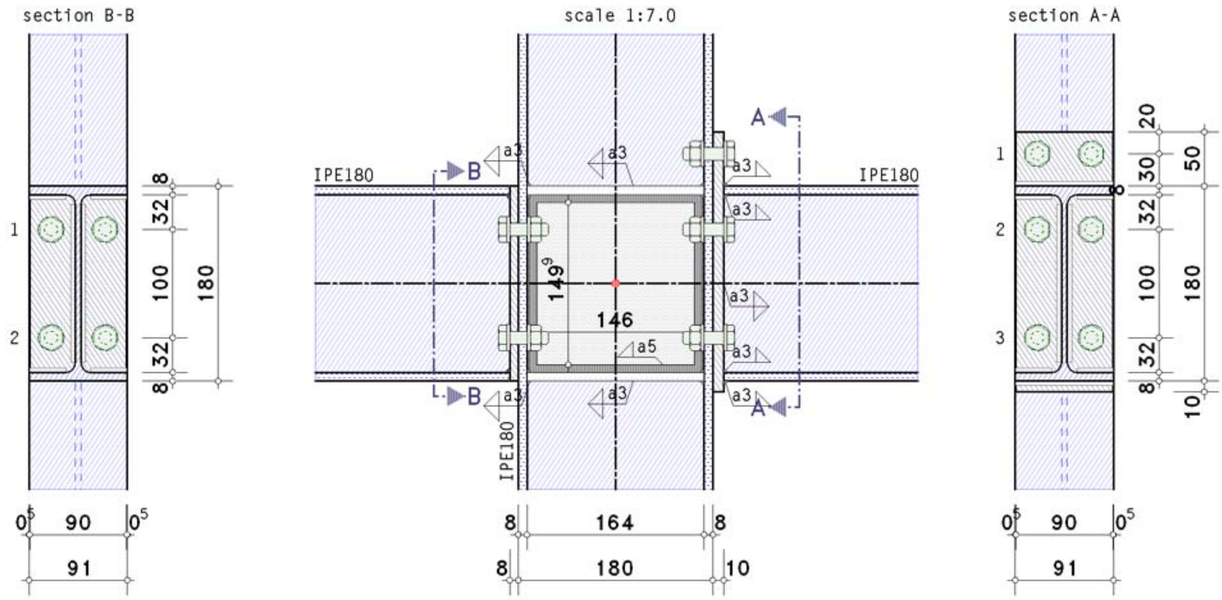
POS. 7: BEIDS. ANSCHL. BSP 2

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

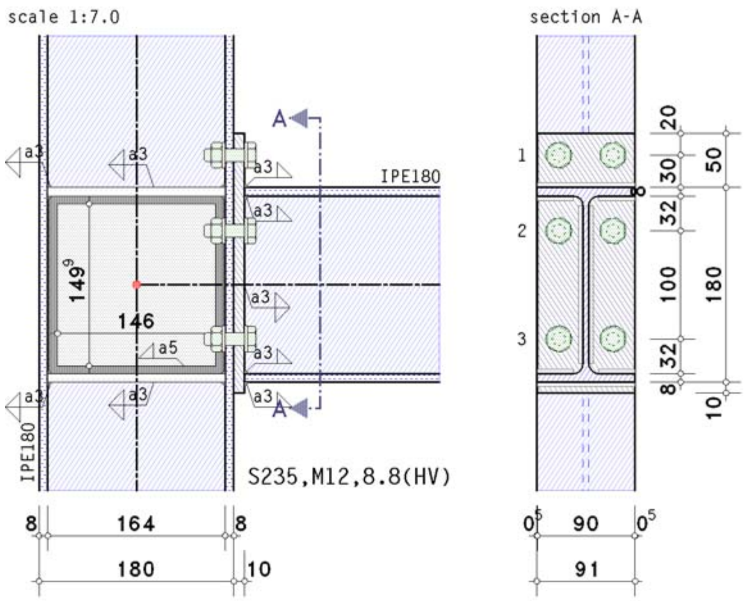
Rigid beam connection EC 3-1-8 (12.10), NA: Deutschland

1. input report

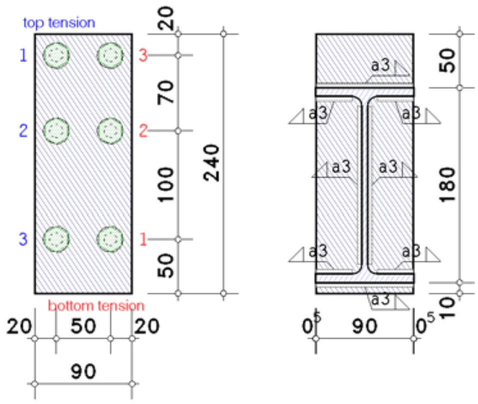


web stiffener (single-sided), t = 7.0 mm

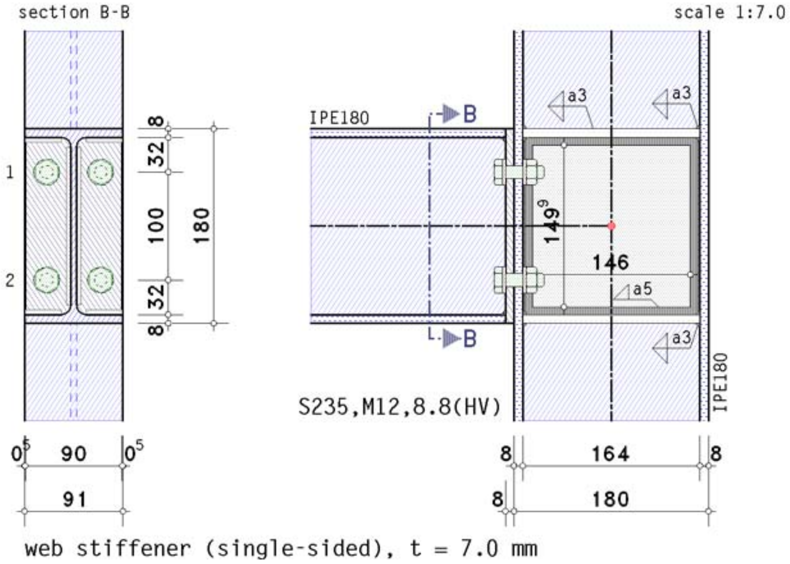
connection right



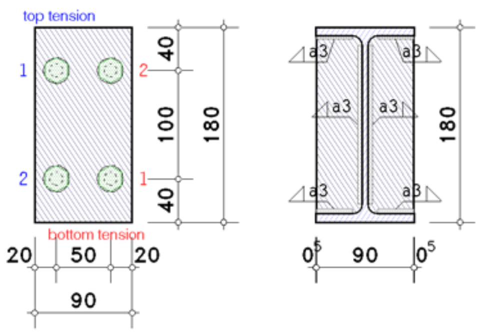
web stiffener (single-sided), t = 7.0 mm details (section A - A)



connection left



details (section B - B)



According to EC 3-1-8, 5.3 in a double-sided beam-columnn-joint each joint is modelled separately.

steel grade
steel grade S235
column parameters
section IPE180
reinforcement of the section by 1 supplementary web plate(s):
  thickness t_s = 7.0 mm, width b_s = 146.0 mm
  weld thickness a_s = 5.0 mm
reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 172.0 mm):
  thickness t_{st} = 8.0 mm, width b_{st} = 42.9 mm, length l_{st} = 164.0 mm
  recess at stiffeners c_{st} = 13.5 mm
  welds $a_{st,f}$ = 3.0 mm, $a_{st,w}$ = 3.0 mm
double-sided beam-column joint, right
bolts
bolt class 8.8, bolt size M12
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}^*$ = $0.7 \cdot f_{yb} \cdot A_s$ = 37.8 kN)
shear plane passes through the unthreaded portion of the bolt
beam parameters
section IPE180
verification parameters

bolted end-plate connection:
thickness t_p = 10.0 mm, width b_p = 90.0 mm, length l_p = 240.0 mm
projections $h_{p,o}$ = 50.0 mm, $h_{p,u}$ = 10.0 mm
bolts in connection:
3 bolt-rows with 2 bolts
all bolt-rows considered individually
all bolt-rows for shear transfer (rows 1-3)
bolt groups generated automatically, considering all groups bzgl. row 1
verification with der Component method: MNV-interaction acc. to Cerfontaine (in Jaspart/Weynand)
centre distance of the bolts to the lateral edge of the end-plate e_2 = 20.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 20.0 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 50.0 mm
centre distance of the bolt-rows from each other p_{1-2} = 70.0 mm, p_{2-3} = 100.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 3.0 mm
beam web: fillet weld, weld thickness a = 3.0 mm
beam flange bottom: fillet weld, weld thickness a = 3.0 mm

double-sided beam-column joint, left

bolts

bolt class 8.8, bolt size M12
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}^* = 0.7 \cdot f_{yb} \cdot A_s$ = 37.8 kN)
shear plane passes through the unthreaded portion of the bolt

beam parameters

section IPE180

verification parameters

bolted end-plate connection:
thickness t_p = 8.0 mm, width b_p = 90.0 mm, length l_p = 180.0 mm
projections $h_{p,o}$ = 0.0 mm, $h_{p,u}$ = 0.0 mm
bolts in connection:
2 bolt-rows with 2 bolts
all bolt-rows considered individually
all bolt-rows for shear transfer (rows 1-2)
bolt groups generated automatically, considering all groups bzgl. row 1
verification with der Component method: MNV-interaction acc. to Cerfontaine (in Jaspart/Weynand)
centre distance of the bolts to the lateral edge of the end-plate e_2 = 20.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 40.0 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 40.0 mm
centre distance of the bolt-rows from each other p_{1-2} = 100.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 3.0 mm
beam web: fillet weld, weld thickness a = 3.0 mm
beam flange bottom: fillet weld, weld thickness a = 3.0 mm

internal forces and moments in the intersection point of system axes refering to the non-haunched axis

$N_{j,b1,Ed}$,$M_{j,b1,Ed}$,$V_{j,b1,Ed}$,$N_{j,b2,Ed}$,$M_{j,b2,Ed}$,$V_{j,b2,Ed}$,$N_{j,c1,Ed}$,$M_{j,c1,Ed}$,$V_{j,c1,Ed}$,$N_{j,c2,Ed}$,$M_{j,c2,Ed}$,$V_{j,c2,Ed}$: internal forces and moments by sign definition of statics

Lk	$N_{j,b1,Ed}$	$M_{j,b1,Ed}$	$V_{j,b1,Ed}$	$N_{j,b2,Ed}$	$M_{j,b2,Ed}$	$V_{j,b2,Ed}$	$N_{j,c1,Ed}$	$M_{j,c1,Ed}$	$V_{j,c1,Ed}$	$N_{j,c2,Ed}$	$M_{j,c2,Ed}$	$V_{j,c2,Ed}$	
--	kN	kNm	kN	kN	kNm	kN	kN	kNm	kN	kN	kNm	kN	
1	-1.38	-13.58	14.22	0.10	-2.68	-1.22	-16.22	-5.98	-4.70	-0.77	4.92	-3.22	Import LK 1
2	0.95	5.21	-1.38	-7.30	-9.93	-6.18	-7.84	11.00	13.62	-3.05	-4.14	5.37	Import LK 2
3	0.87	5.40	-1.57	-7.32	-9.86	-6.06	-7.11	11.04	13.65	-2.62	-4.22	5.46	Import LK 3
4	-1.30	-13.76	14.41	0.13	-2.75	-1.33	-16.95	-6.01	-4.73	-1.20	5.00	-3.30	Import LK 4
5	-0.20	-3.73	8.02	-7.26	-11.60	-6.68	-16.99	6.95	10.44	-2.29	-0.92	3.37	Import LK 5
6	-0.12	-3.91	8.20	-7.24	-11.67	-6.80	-17.72	6.92	10.41	-2.72	-0.85	3.29	Import LK 6
7	-1.24	-13.77	14.41	0.16	-2.76	-1.34	-16.95	-5.99	-4.69	-1.20	5.02	-3.31	Import LK 7
8	-1.32	-13.58	14.23	0.13	-2.68	-1.22	-16.22	-5.96	-4.67	-0.77	4.94	-3.23	Import LK 8
9	-0.83	-10.20	13.15	-4.26	-8.56	-4.78	-19.96	0.71	3.56	-2.03	2.35	0.11	Import LK 9
10	-0.92	-10.01	12.96	-4.28	-8.48	-4.67	-19.23	0.74	3.59	-1.60	2.27	0.20	Import LK 10
11	-0.92	-10.20	13.15	-4.31	-8.54	-4.78	-19.97	0.67	3.51	-2.04	2.33	0.12	Import LK 11
12	0.23	-0.53	0.53	0.07	-0.21	-0.32	-2.08	-0.10	-0.08	-1.23	0.22	-0.24	Import LK 12
13	0.88	5.40	-1.57	-7.31	-9.86	-6.05	-7.09	11.04	13.65	-2.61	-4.22	5.46	Import LK 13
14	0.96	5.22	-1.39	-7.29	-9.93	-6.16	-7.82	11.01	13.62	-3.04	-4.14	5.37	Import LK 14
15	-0.18	-3.92	8.20	-7.27	-11.66	-6.81	-17.74	6.89	10.38	-2.73	-0.85	3.29	Import LK 15
16	-0.26	-3.74	8.02	-7.30	-11.58	-6.70	-17.01	6.92	10.41	-2.30	-0.93	3.37	Import LK 16
17	0.31	-0.71	0.72	0.09	-0.29	-0.43	-2.81	-0.13	-0.11	-1.67	0.30	-0.33	Import LK 17

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

connection right:
ok
connection left:
ok
bolts right:
distances between bolt-rows at end-plate

horizontal: $e_2 = 20.0$ mm > $1.2 \cdot d_0 = 15.6$ mm, $e_2 = 20.0$ mm < $4 \cdot t + 40$ mm = 72.0 mm
horizontal: $p_2 = 50.0$ mm > $2.4 \cdot d_0 = 31.2$ mm, $p_2 = 50.0$ mm < min(14·t, 200 mm) = 112.0 mn
vertical: $e_1 = 20.0$ mm > $1.2 \cdot d_0 = 15.6$ mm, $e_1 = 20.0$ mm < $4 \cdot t + 40$ mm = 72.0 mm
vertical: $p_1 = 70.0$ mm > $2.2 \cdot d_0 = 28.6$ mm, $p_1 = 70.0$ mm < min(14·t, 200 mm) = 112.0 mn
vertical: $p_1 = 100.0$ mm > $2.2 \cdot d_0 = 28.6$ mm, $p_1 = 100.0$ mm < min(14·t, 200 mm) = 112.0 m
vertical: $e_1 = 50.0$ mm > $1.2 \cdot d_0 = 15.6$ mm, $e_1 = 50.0$ mm < $4 \cdot t + 40$ mm = 72.0 mm

horizontal distance of bolts from column edge

vertical: $e_1 = 20.5$ mm > $1.2 \cdot d_0 = 15.6$ mm, $e_1 = 20.5$ mm < $4 \cdot t + 40$ mm = 72.0 mm

bolts left:

distances between bolt-rows at end-plate

horizontal: $e_2 = 20.0$ mm > $1.2 \cdot d_0 = 15.6$ mm, $e_2 = 20.0$ mm < $4 \cdot t + 40$ mm = 72.0 mm
horizontal: $p_2 = 50.0$ mm > $2.4 \cdot d_0 = 31.2$ mm, $p_2 = 50.0$ mm < min(14·t, 200 mm) = 112.0 mn
vertical: $e_1 = 40.0$ mm > $1.2 \cdot d_0 = 15.6$ mm, $e_1 = 40.0$ mm < $4 \cdot t + 40$ mm = 72.0 mm
vertical: $p_1 = 100.0$ mm > $2.2 \cdot d_0 = 28.6$ mm, $p_1 = 100.0$ mm < min(14·t, 200 mm) = 112.0 m
vertical: $e_1 = 40.0$ mm > $1.2 \cdot d_0 = 15.6$ mm, $e_1 = 40.0$ mm < $4 \cdot t + 40$ mm = 72.0 mm

horizontal distance of bolts from column edge

vertical: $e_1 = 20.5$ mm > $1.2 \cdot d_0 = 15.6$ mm, $e_1 = 20.5$ mm < $4 \cdot t + 40$ mm = 72.0 mm

utilizations of each joint (right)

Lk	$U_{\sigma,b}$	$U_{\sigma,c}$	U_{MNV}	U_{wp}	U_{ep}	U_{sb}	U_{ss}	U
--	---	---	---	---	---	---	---	---
1	0.320	0.155	0.572	0.199	0.078	0.507	0.652	0.652
2	0.132	0.262	0.442	0.974	0.008	0.211	0.366	0.974
3	0.136	0.263	0.456	0.981	0.009	0.217	0.378	0.981
4	0.325	0.157	0.580	0.201	0.079	0.514	0.660	0.660
5	0.083	0.172	0.152	---	0.044	0.124	0.159	0.172
6	0.087	0.172	0.160	---	0.045	0.130	0.167	0.172
7	0.325	0.156	0.580	0.201	0.079	0.514	0.660	0.660
8	0.321	0.155	0.572	0.199	0.078	0.507	0.652	0.652
9	0.237	0.062	0.419	0.005	0.072	0.371	0.477	0.477
10	0.232	0.060	0.411	0.005	0.071	0.365	0.469	0.469
11	0.237	0.061	0.420	0.006	0.072	0.372	0.478	0.478
12	0.013	0.006	0.023	0.004	0.003	0.020	0.026	0.026
13	0.136	0.263	0.456	0.982	0.009	0.218	0.378	0.982
14	0.132	0.262	0.442	0.975	0.008	0.211	0.367	0.975
15	0.088	0.172	0.160	---	0.045	0.131	0.168	0.172
16	0.083	0.172	0.153	---	0.044	0.124	0.159	0.172
17	0.017	0.009	0.031	0.006	0.004	0.027	0.036	0.036

$U_{\sigma,b}$: stress utilization at the beam; $U_{\sigma,c}$: stress utilization at the column; U_{MNV}: utilization due to MNV-interaction
U_{wp}: utilization due to shear in column web; U_{pl}: utilization due to shear in end-plate; U_{sb}: utilization due to weld
U_{ss}: utilization due to stiffeners/ribs; U: utilization of each joint; U: utilization of each joint

utilizations of each joint (left)

Lk	$U_{\sigma,b}$	$U_{\sigma,c}$	U_{MNV}	U_{wp}	U_{ep}	U_{sb}	U_{ss}	U
--	---	---	---	---	---	---	---	---
1	0.067	0.155	0.228	---	0.008	0.106	0.183	0.228
2	0.245	0.262	0.730	0.743	0.039	0.400	0.679	0.743
3	0.243	0.263	0.724	0.759	0.038	0.398	0.675	0.759
4	0.068	0.157	0.234	---	0.008	0.108	0.187	0.234
5	0.286	0.172	0.865	0.177	0.042	0.467	0.794	0.865
6	0.288	0.172	0.871	0.171	0.043	0.469	0.798	0.871
7	0.068	0.156	0.234	---	0.008	0.108	0.188	0.234
8	0.067	0.155	0.229	---	0.008	0.106	0.183	0.229
9	0.211	0.062	0.648	---	0.030	0.342	0.584	0.648
10	0.210	0.060	0.642	---	0.029	0.340	0.580	0.642
11	0.211	0.061	0.646	---	0.030	0.342	0.583	0.646
12	0.005	0.006	0.016	---	0.002	0.008	0.013	0.016
13	0.243	0.263	0.725	0.760	0.038	0.398	0.675	0.760
14	0.245	0.262	0.730	0.744	0.039	0.400	0.679	0.744
15	0.288	0.172	0.869	0.170	0.043	0.469	0.797	0.869
16	0.286	0.172	0.864	0.176	0.042	0.466	0.793	0.864
17	0.007	0.009	0.021	---	0.003	0.010	0.018	0.021

$U_{\sigma,b}$: stress utilization at the beam; $U_{\sigma,c}$: stress utilization at the column; U_{MNV}: utilization due to MNV-interaction
U_{wp}: utilization due to shear in column web; U_{pl}: utilization due to shear in end-plate; U_{sb}: utilization due to weld
U_{ss}: utilization due to stiffeners/ribs; U: utilization of each joint; U: utilization of each joint

2. final result

utilization/rotation of the connection

	right			left				Gleichgewicht			
Lk	$S_{j,ini}$	S_j	φ_j	$S_{j,ini}$	S_j	φ_j	U_j	ΣH	ΣV	ΣM	
--	MNm/rad	MNm/rad	°	MNm/rad	MNm/rad	°		kN	kN	kNm	
1	9.3	9.3	0.075	3.3	3.3	0.045	0.652	0.00	0.00	0.00	ok
2	3.5	3.5	0.086	3.6	3.6	0.138	0.974	0.00	0.00	0.00	ok
3	3.5	3.5	0.088	3.6	3.6	0.138	0.981	0.00	0.00	0.00	ok
4	9.3	9.3	0.076	3.3	3.3	0.046	0.660	0.00	0.00	0.00	ok
5	6.3	6.3	0.027	4.4	4.4	0.135	0.865	0.01	0.00	0.00	ok
6	6.3	6.3	0.029	4.4	4.4	0.135	0.871	0.01	0.00	0.00	ok
7	9.3	9.3	0.076	3.3	3.3	0.046	0.660	0.02	0.00	0.00	ok
8	9.3	9.3	0.075	3.3	3.3	0.045	0.652	0.02	0.00	0.00	ok
9	12.5	12.5	0.041	5.0	5.0	0.089	0.648	0.03	0.00	0.00	ok
10	12.6	12.6	0.040	5.0	5.0	0.087	0.642	0.03	0.00	0.00	ok
11	12.5	12.5	0.041	5.0	5.0	0.088	0.646	0.00	0.00	0.00	ok
12	10.2	10.2	0.003	3.7	3.7	0.003	0.026	0.00	0.00	0.00	ok
13	3.5	3.5	0.089	3.6	3.6	0.138	0.982*	0.00	0.00	0.00	ok
14	3.5	3.5	0.086	3.6	3.6	0.138	0.975	0.00	0.00	0.00	ok
15	6.4	6.4	0.028	4.4	4.4	0.135	0.869	0.00	0.00	0.00	ok
16	6.3	6.3	0.027	4.4	4.4	0.135	0.864	0.00	0.00	0.00	ok
17	10.2	10.2	0.004	3.7	3.7	0.004	0.036	0.00	0.00	0.00	ok

$S_{j,ini}$: initial rotational stiffness; S_j: rotational stiffness; φ_j: rotation; U_j: utilization of the connection; tolerances of equilibrium 1 kN / 1 kNm
*) maximum utilization

maximum utilization [Lk 13]: max U = 0.982 < 1 ok
minimum rotational stiffness (right): min S_j = 3.5 MNm/rad, $S_{j,ini}$ = 3.5 MNm/rad, φ_j = 0.086°
minimum rotational stiffness (left): min S_j = 3.3 MNm/rad, $S_{j,ini}$ = 3.3 MNm/rad, φ_j = 0.045°

verification succeeded

3. Detailed edition of Lk 13 (decisive)

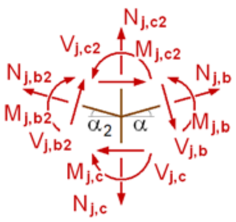
notes
no verfication for welds of supplementary web plates.

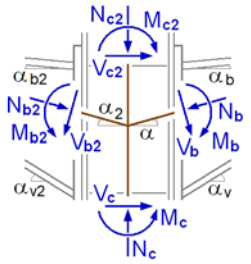
3.1. connection right

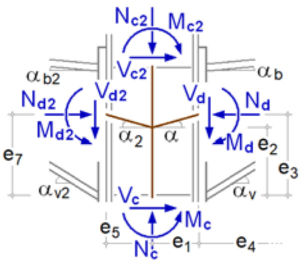
notes
connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.

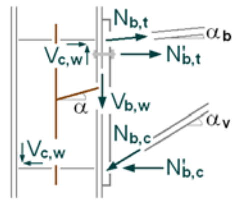
3.1.1. design values

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









slope angle: $\alpha_b = \alpha_v = \alpha = 0°$
distance: e_1 = 90.0 mm, e_3 = 86.0 mm, e_2 = 86.0 mm, e_5 = 90.0 mm, e_7 = 86.0 mm

internal forces and moments perpendicular to the connection planes
periphery beam (right)
N_d = -0.88 kN, M_d = -5.26 kNm, V_d = -1.57 kN
periphery beam (left)
N_{d2} = 7.31 kN, M_{d2} = 9.31 kNm, V_{d2} = -6.05 kN
periphery column (bottom)
N_c = 7.09 kN, M_c = -9.87 kNm, V_c = 13.65 kN
periphery column (top)
N_{c2} = 2.61 kN, M_{c2} = 3.75 kNm, V_{c2} = 5.46 kN
negative internal moment M_d ⇒ mirrored model
N_d = -0.88 kN, M_d = 5.26 kNm, V_d = 1.57 kN
N_{d2} = 7.31 kN, M_{d2} = -9.31 kNm, V_{d2} = -6.05 kN
N_c = 2.61 kN, M_c = 3.75 kNm, V_c = 5.46 kN
N_{c2} = 7.09 kN, M_{c2} = -9.87 kNm, V_{c2} = -13.65 kN
partial internal forces and moments refering to the mirrored model
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_{ep}$ = 5.25 kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 30.93 kN, z_b = 172.0 mm, z_{bu} = 86.0 mm

$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b = 30.06$ kN, $z_b = 172.0$ mm, $z_{bo} = 86.0$ mm

3.1.2. resistance of cross-section

column bottom

plastic cross-sectional check for $N = -2.61$ kN, $M_y = -3.75$ kNm, $V_z = -5.46$ kN
valid normal/shear stress: zul $\sigma_{Rd} = 23.50$ kN/cm², zul $\tau_{Rd} = 13.57$ kN/cm²
top flange: resistance forces $N_{max,O} = 171.08$ kN, $N_{min,O} = -171.08$ kN
bottom flange: resistance forces $N_{max,U} = 171.08$ kN, $N_{min,U} = -171.08$ kN
web: shear force $V_S = -5.46$ kN, shear stress $\tau_S = 0.60$ kN/cm² $\Rightarrow$ $U_{\tau,S} = 0.044$
resistance forces $N_{max,S} = 214.02$ kN, $N_{min,S} = -214.02$ kN
main bending: axial force $N = -2.61$ kN, resistance forces $N_{max} = 556.18$ kN, $N_{min} = -556.18$ kN $\Rightarrow$ $U_N = 0.005$
moment $M_y = -3.75$ kNm, resistance moments $M_{y,max} = 38.63$ kNm, $M_{y,min} = -38.63$ kNm $\Rightarrow$ $U_{My} = 0.097$
total (possibly due to load increase): max $U = 0.100 < 1$ ok
utilizations: resistance $U_\sigma = 0.100 < 1$ ok, c/t-ratio $U_{c/t} = 0.101 < 1$ ok

column top

plastic cross-sectional check for $N = -7.09$ kN, $M_y = 9.87$ kNm, $V_z = -13.65$ kN
valid normal/shear stress: zul $\sigma_{Rd} = 23.50$ kN/cm², zul $\tau_{Rd} = 13.57$ kN/cm²
top flange: resistance forces $N_{max,O} = 171.08$ kN, $N_{min,O} = -171.08$ kN
bottom flange: resistance forces $N_{max,U} = 171.08$ kN, $N_{min,U} = -171.08$ kN
web: shear force $V_S = -13.65$ kN, shear stress $\tau_S = 1.50$ kN/cm² $\Rightarrow$ $U_{\tau,S} = 0.110$
resistance forces $N_{max,S} = 212.92$ kN, $N_{min,S} = -212.92$ kN
main bending: axial force $N = -7.09$ kN, resistance forces $N_{max} = 555.08$ kN, $N_{min} = -555.08$ kN $\Rightarrow$ $U_N = 0.013$
moment $M_y = 9.87$ kNm, resistance moments $M_{y,max} = 38.57$ kNm, $M_{y,min} = -38.57$ kNm $\Rightarrow$ $U_{My} = 0.256$
total (possibly due to load increase): max $U = 0.263 < 1$ ok
utilizations: resistance $U_\sigma = 0.263 < 1$ ok, c/t-ratio $U_{c/t} = 0.164 < 1$ ok

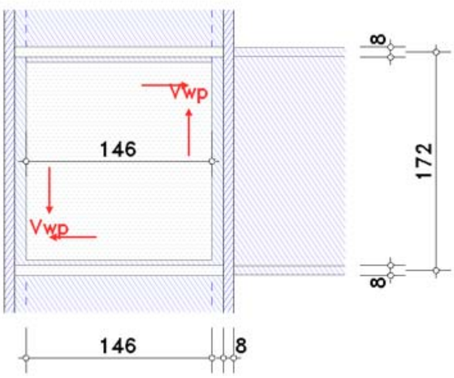
beam

plastic cross-sectional check for $N = 0.88$ kN, $M_y = -5.25$ kNm, $V_z = 1.57$ kN
valid normal/shear stress: zul $\sigma_{Rd} = 23.50$ kN/cm², zul $\tau_{Rd} = 13.57$ kN/cm²
top flange: resistance forces $N_{max,O} = 171.08$ kN, $N_{min,O} = -171.08$ kN
bottom flange: resistance forces $N_{max,U} = 171.08$ kN, $N_{min,U} = -171.08$ kN
web: shear force $V_S = 1.57$ kN, shear stress $\tau_S = 0.17$ kN/cm² $\Rightarrow$ $U_{\tau,S} = 0.013$
resistance forces $N_{max,S} = 214.21$ kN, $N_{min,S} = -214.21$ kN
main bending: axial force $N = 0.88$ kN, resistance forces $N_{max} = 556.37$ kN, $N_{min} = -556.37$ kN $\Rightarrow$ $U_N = 0.002$
moment $M_y = -5.25$ kNm, resistance moments $M_{y,max} = 38.64$ kNm, $M_{y,min} = -38.64$ kNm $\Rightarrow$ $U_{My} = 0.136$
total (possibly due to load increase): max $U = 0.136 < 1$ ok
utilizations: resistance $U_\sigma = 0.136 < 1$ ok, c/t-ratio $U_{c/t} = 0.117 < 1$ ok

3.1.3. basic components

3.1.3.1. Gk 1: Column web panel in shear

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = |1 - M_{j2}/M_{j1}| = 2.00$ for $M_{j1} = -5.40$ kNm, $M_{j2} = 9.86$ kNm

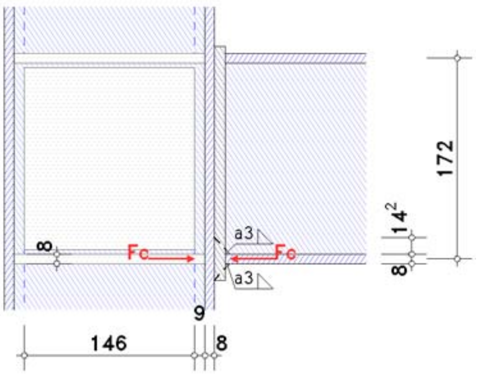


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

web thickness incl. reinforcement $t_{wc} = 10.6$ mm
slenderness of column web $d_c/t_{wc} = 13.77 < 69 \cdot \varepsilon = 69.00$ $\Rightarrow$ method applicable
shear area with reinforcement $A_v = 18.99$ cm²
plastic shear resistance without stiffeners $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_v) / (3^{1/2} \cdot \gamma_{M0}) = 231.9$ kN
placing of intermediate web stiffeners:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/d_{st} = 8.0$ kN
$V_{wp,add,Rd} > 2 \cdot (M_{pl,fc,Rd} + M_{pl,st,Rd})/d_{st} = 7.7$ kN $\Rightarrow$ $V_{wp,add,Rd} = 7.7$ kN
plastic shear resistance with transverse stiffeners $V_{wp,Rd} = 239.6$ kN

3.1.3.2. Gk 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = |1 - M_{j2}/M_{j1}| = 2.00$ for $M_{j1} = -5.40$ kNm, $M_{j2} = 9.86$ kNm
longitudinal compressive stress in column web $\sigma_{com,Ed} = 21.88$ N/mm²

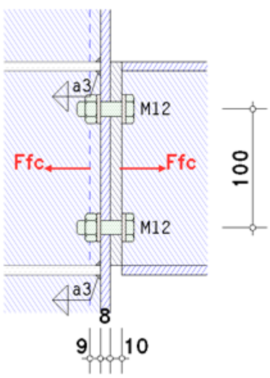


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: $c/t = 5.4{\cdot}\varepsilon \le 33{\cdot}\varepsilon \Rightarrow$ section class $1 \le 2$ ok
minimum demands of the moment of inertia of stiffeners:
  length of buckling field (distance of stiffeners) a = 172.0 mm
  web height between the flanges $h_{wc} = 164.0$ mm
  moment of inertia of stiffeners $I_{st} = 50.24$ cm⁴
  minimum moment of inertia for $a/h_{wc} = 1.05 < 2^{1/2}$: $I_{st,min} = 3.33$ cm⁴ $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
  torsional moment of inertia of stiffeners $I_T = 0.73$ cm⁴
  polar moment of inertia of stiffeners $I_p = 5.43$ cm⁴
  $I_T / I_p \approx 0.135 > 0.006 = 5.3{\cdot}f_{y,st}/E_{st}$ ok
resistance of stiffened webs with transverse compression:
  area of stiffeners incl. web $A_{st} = 7.28$ cm²
  slenderness $\lambda = 0.066$
  $\lambda \le 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
  design value of resistance of flexural buckling $F_{c,w,Rd} = 155.5$ kN

resistance of upper beam flange:
reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: $c/t = 5.4{\cdot}\varepsilon \le 33{\cdot}\varepsilon \Rightarrow$ section class $1 \le 2$ ok
minimum demands of the moment of inertia of stiffeners:
  length of buckling field (distance of stiffeners) a = 172.0 mm
  web height between the flanges $h_{wc} = 164.0$ mm
  moment of inertia of stiffeners $I_{st} = 50.24$ cm⁴
  minimum moment of inertia for $a/h_{wc} = 1.05 < 2^{1/2}$: $I_{st,min} = 3.33$ cm⁴ $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
  torsional moment of inertia of stiffeners $I_T = 0.73$ cm⁴
  polar moment of inertia of stiffeners $I_p = 5.43$ cm⁴
  $I_T / I_p \approx 0.135 > 0.006 = 5.3{\cdot}f_{y,st}/E_{st}$ ok
resistance of stiffened webs with transverse compression:
  area of stiffeners incl. web $A_{st} = 7.28$ cm²
  slenderness $\lambda = 0.066$
  $\lambda \le 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
  design value of resistance of flexural buckling $F_{c,w,Rd} = 155.5$ kN

3.1.3.3. 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
  effective length of the T-stub flange (column flange):
    in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 87.0$ mm, $l_{eff,cp} = 95.2$ mm
    in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 87.0$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.33$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 97.11$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 123.51$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 73.14$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 97.11$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 73.14$ kN

row 2

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 87.0$ mm, $l_{eff,cp} = 95.2$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 87.0$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.33$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 97.11$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 123.51$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 73.14$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 97.11$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 73.14$ kN

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

$F_{t,fc,Rd,1} = 73.14$ kN, $l_{eff,1} = 87.0$ mm

$F_{t,fc,Rd,2} = 73.14$ kN, $l_{eff,2} = 87.0$ mm

equivalent T-stub flange (group of bolts 1):

here: number of bolt-rows $n_b = 2$ (between stiffeners)

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

in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 187.7$ mm, $\Sigma l_{eff,cp} = 295.2$ mm

in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 187.7$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.71$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 194.23$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 266.59$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 149.31$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 194.23$ kN

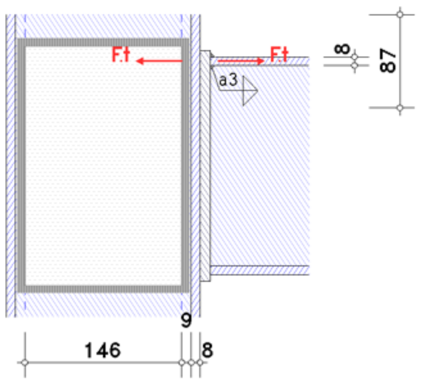
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 149.31$ kN

resistances and effective lengths of column flange in bending (per bolt group):

$F_{ep,Rd,1-2} = 149.31$ kN, $\Sigma l_{eff} = 187.7$ mm, 2 rows

3.1.3.4. Gk 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = |1 - M_{j2}/M_{j1}| = 2.00$ for $M_{j1} = -5.40$ kNm, $M_{j2} = 9.86$ kNm



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

each individual bolt-row:

row 1

effective width $b_{eff,t} = 87.0$ mm (l_{eff} from bc 4)

reinforcement of column web with 1 supplementary web plate:

fillet weld with $a_s \geq t_s/2^{1/2} = 4.9$ mm: effective web thickness $t_{w,eff} = t_{wc} + 0.4 \cdot t_s = 7.4$ mm for S235

reduction factor for interaction with shear stress $\beta = 2 \Rightarrow \omega = 0.790$

resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 119.9$ kN

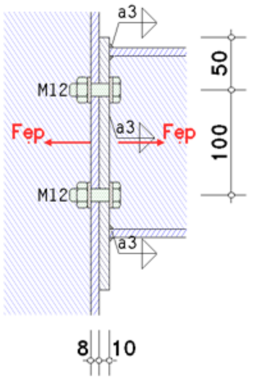
row 2

effective width $b_{eff,t} = 87.0$ mm (l_{eff} from bc 4)

reinforcement of column web with 1 supplementary web plate:
fillet weld with $a_s \geq t_s/2^{1/2} = 4.9$ mm: effective web thickness $t_{w,eff} = t_{wc}+0.4 \cdot t_s = 7.4$ mm for S235
reduction factor for interaction with shear stress $\beta = 2 \Rightarrow \omega = 0.790$
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 119.9$ kN

group of bolt-rows, group 1:
effective width $b_{eff,t} = 187.7$ mm (l_{eff} from bc 4)
reinforcement of column web with 1 supplementary web plate:
fillet weld with $a_s \geq t_s/2^{1/2} = 4.9$ mm: effective web thickness $t_{w,eff} = t_{wc}+0.4 \cdot t_s = 7.4$ mm for S235
reduction factor for interaction with shear stress $\beta = 2 \Rightarrow \omega = 0.513$
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 168.0$ kN

3.1.3.5. Gk 5: end-plate in bending



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

part of end-plate between beam flanges
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 (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 102.8$ mm, $l_{eff,cp} = 119.1$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 102.8$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.60$ kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 97.11$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n-2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n-e_w \cdot (m+n)) = 170.45$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 80.87$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 97.11$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 80.87$ kN
resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm²
tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 157.04$ kN (≥ 80.87 kN, not decisive)
row 2
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 102.8$ mm, $l_{eff,cp} = 119.1$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 102.8$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.60$ kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 97.11$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n-2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n-e_w \cdot (m+n)) = 170.45$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 80.87$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 97.11$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 80.87$ kN
resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm²
tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 157.04$ kN (≥ 80.87 kN, not decisive)
resistances and effective lengths of end-plate in bending (per bolt-row):
$F_{ep,Rd,1} = 80.87$ kN, $l_{eff,1} = 102.8$ mm
$F_{ep,Rd,2} = 80.87$ kN, $l_{eff,2} = 102.8$ mm

equivalent T-stub flange (group of bolts 1):
here: number of bolt-rows $n_b = 2$
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 204.8$ mm, $\Sigma l_{eff,cp} = 319.1$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 204.8$ mm
tension resistance of the T-stub flange:

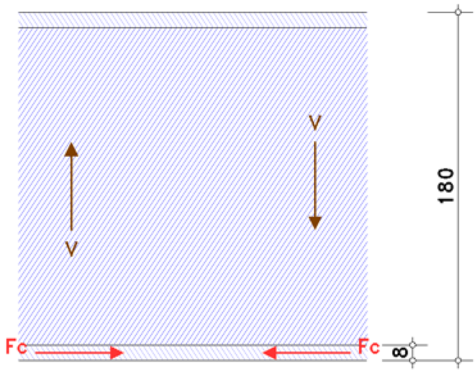
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 1.20$ kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 194.23$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 339.54$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 161.49$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 194.23$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 161.49$ kN
resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm²
tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 312.83$ kN (≥ 161.49 kN, not decisive)

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

$F_{ep,Rd,1-2} = 161.49$ kN, $\Sigma l_{eff} = 204.8$ mm, 2 rows

3.1.3.6. Gk 7: beam flange and web in compression

flange bottom: section class for $c/(\varepsilon \cdot t) = 4.23$: 1
web: section class for $\alpha = 0.49$ and $c/(\varepsilon \cdot t) = 27.55$: 1
section class of beam: 1
taking into account the moment-shear force-interaction $V_{Ed} = 1.6$ kN

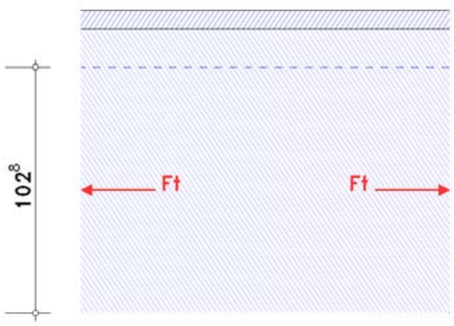


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

stress due to bending with shear force: $V_{Ed} = 1.6 \text{ kN} \leq 76.3 \text{ kN} = V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 39.01$ kNm, $W_{pl} = 166.00$ cm³
resistance of a flange (and web) with compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 226.80$ kN

resistance of upper beam flange:
stress due to bending with shear force: $V_{Ed} = 1.6 \text{ kN} \leq 76.3 \text{ kN} = V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 39.01$ kNm, $W_{pl} = 166.00$ cm³
resistance of a flange (and web) with compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 226.80$ kN

3.1.3.7. Gk 8: beam web in tension



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

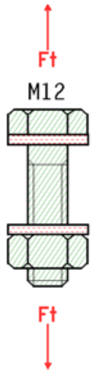
each individual bolt-row:
row 1
effective width $b_{eff,t,wb} = 102.8$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 128.1$ kN
row 2
effective width $b_{eff,t,wb} = 102.8$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 128.1$ kN

group of bolt-rows, group 1:
effective width $b_{eff,t,wb} = 204.8$ mm (l_{eff} from bc 5)
resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 255.1$ kN

3.1.3.8. Gk 10: bolts in tension

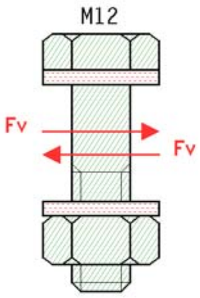
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



tension resistance of one bolt $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 48.56$ kN, $k_2 = 0.90$
punching shear load capicity $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 99.69$ kN, $t_p = 8.0$ mm
tension-/punching shear load capicity for 2 bolts: $\Sigma F_{tp,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 97.11$ kN

3.1.3.9. Gk 11: bolts in shear

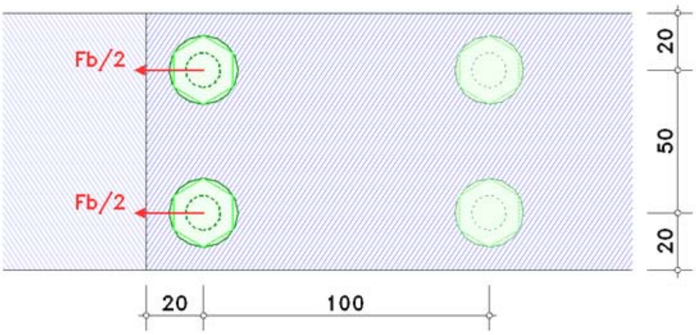
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



shear resistance per shear plane $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2} = 43.43$ kN, $\alpha_v = 0.60$
shear resistance of 2 bolts (1-shear): $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd} = 86.86$ kN

3.1.3.10. Gk 12: plate with bearing resistance

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



row 1
  end-plate:
  bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 86.40$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
  bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 86.40$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
  bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 172.80$ kN
  column flange:
  bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
  bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
  bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 138.24$ kN
row 2
  end-plate:
  bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 86.40$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
  bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 86.40$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
  bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 172.80$ kN
  column flange:
  bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
  bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
  bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 138.24$ kN
row 3
  end-plate:

bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 86.40$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 86.40$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 172.80$ kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 138.24$ kN

bearing resistance (3 rows)
$\Sigma F_{b,Rd,1} = 138.24$ kN
$\Sigma F_{b,Rd,2} = 138.24$ kN
$\Sigma F_{b,Rd,3} = 138.24$ kN

3.1.4. connection capacity

3.1.4.1. moment resistance

distance of tension-bolt-rows from centre of compression: $h_1 = 136.0$ mm, $h_2 = 36.0$ mm

resistance per bolt-row (MNV-interaction)
row 1: $F_{tr,Rd} = 73.1$ kN
row 2: $F_{tr,Rd} = 48.6$ kN

resistance of flanges (MNV-interaction)
bottom: $F_{c,Rd} = 119.8$ kN

moment resistance (MNV-interaction)
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 11.7$ kNm

shear force resistance (MNV-interaction)
$V_{j,Rd} = 3.4$ kN

3.1.4.2. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd} = 198.09$ kN
welds: $F_{w,Rd} = 182.07$ kN
shear resistance of end plate: $V_{ep,Rd} = F_{w,Rd} = 182.07$ kN

shear resistance of column web
$V_{wp,Rd}/\beta_j = 119.8$ kN

3.1.4.3. total

$V_{wp,Rd}/\beta_j = 119.8$ kN $V_{ep,Rd} = 182.1$ kN

3.1.5. verifications

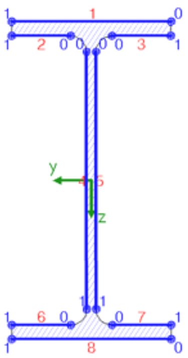
3.1.5.1. verification of the connection capacity by means of the component method

$U_{MNV} = 0.456 < 1$ ok
$V_{c,w,Ed}/(V_{wp,Rd}/\beta_j) = 0.982 < 1$ ok
$V_{Ed}/V_{ep,Rd} = 0.009 < 1$ ok

3.1.5.2. 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:	$a_w = 3.0$ mm	$l_w = 90.0$ mm
weld 2:	$a_w = 3.0$ mm	$l_w = 33.4$ mm
weld 3:	siehe weld 2	
weld 4:	$a_w = 3.0$ mm	$l_w = 146.0$ mm
weld 5:	siehe weld 4	
weld 6:	$a_w = 3.0$ mm	$l_w = 33.4$ mm
weld 7:	siehe weld 6	
weld 8:	$a_w = 3.0$ mm	$l_w = 90.0$ mm

design values refering to centroid of the section:
$N_{Ed} = 0.88$ kN, $M_{y,Ed} = -5.26$ kNm, $V_{z,Ed} = 1.57$ kN

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

$\Sigma A_w = 18.16\ cm^2$, $A_{w,z} = 8.76\ cm^2$, $\Sigma l_w = 60.5\ cm$

$I_{w,y} = 862.10\ cm^4$, $I_{w,z} = 72.88\ cm^4$, $W_{w,t} = 14.27\ cm^3$, $\Delta z_w = 0.0\ mm$

distribution of internal forces and moments:

weld 1: $N_w = 14.96\ kN$

weld 2: $N_w = 5.05\ kN$

weld 3: siehe weld 2

weld 4: $N_w = 0.21\ kN$ $M_{y,w} = -0.47\ kNm$

weld 5: siehe weld 4

weld 6: $N_w = -4.96\ kN$

weld 7: siehe weld 6

weld 8: $N_w = -14.70\ kN$

from conventionel distribution of shear force: $V_{z,w} = 1.57\ kN$

verifications in weld edges:

weld 1, pt. 0: $\sigma_{w,x} = 55.40\ N/mm^2$ $\Rightarrow U_w = 0.218 < 1$ ok

weld 2, pt. 0: $\sigma_{w,x} = 50.52\ N/mm^2$ $\Rightarrow U_w = 0.198 < 1$ ok

weld 4, pt. 0: $\sigma_{w,x} = 45.03\ N/mm^2$ $\tau_{w,z} = 1.79\ N/mm^2$ $\Rightarrow U_w = 0.177 < 1$ ok

pt. 1: $\sigma_{w,x} = -44.06\ N/mm^2$ $\tau_{w,z} = 1.79\ N/mm^2$ $\Rightarrow U_w = 0.173 < 1$ ok

weld 6, pt. 0: $\sigma_{w,x} = -49.56\ N/mm^2$ $\Rightarrow U_w = 0.195 < 1$ ok

weld 8, pt. 0: $\sigma_{w,x} = -54.44\ N/mm^2$ $\Rightarrow U_w = 0.214 < 1$ ok

Result:

weld 1, pt. 0: $\sigma_{w,x} = 55.40\ N/mm^2$

Max: $\sigma_{1,w,Ed} = 7.84\ kN/cm^2 < f_{1w,d} = 36.00\ kN/cm^2$,

$\sigma_{2,w,Ed} = 3.92\ kN/cm^2 < f_{2w,d} = 25.92\ kN/cm^2 \Rightarrow U_w = 0.218 < 1$ ok

3.1.5.3. verification of web stiffeners

compression stiffener

$F_{c,Ed} = 45.50\ kN$

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 16.9\ kN$, $H = F \cdot e_F/e_H = 2.9\ kN$

assumption: stiffeners do not buckle: $c/t = 5.4 \cdot \varepsilon \le 33 \cdot \varepsilon \Rightarrow$ section class $1 \le 2$ ok

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 55.18\ kN$

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 17.7\ kN$

$F_{Ed} = 17.7\ kN < F_{Rd} = 55.2\ kN \Rightarrow U = 0.320 < 1$ ok

cross-section at web

shear resistance $V_{Rd} = 178.01\ kN$

design value: $F_{Ed} = F = 16.9\ kN$

$F_{Ed} = 16.9\ kN < F_{Rd} = 178.0\ kN \Rightarrow U = 0.095 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 2.88\ kN/cm$, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 0.50\ kN/cm$, $b_1 = 29.4\ mm$

$\sigma_{1,w,Ed} = 10.03\ kN/cm^2 < f_{1w,d} = 36.00\ kN/cm^2 \Rightarrow U = 0.279 < 1$ ok

$\sigma_{2,w,Ed} = 9.61\ kN/cm^2 < f_{2w,d} = 25.92\ kN/cm^2 \Rightarrow U = 0.371 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 0.62\ kN/cm$, $l_1 = 137.0\ mm$

$\sigma_{1,w,Ed} = 3.57\ kN/cm^2 < f_{1w,d} = 36.00\ kN/cm^2 \Rightarrow U = 0.099 < 1$ ok

stiffener in tension

$F_{t,Ed} = 46.37\ kN$

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 17.2\ kN$, $H = F \cdot e_F/e_H = 3.0\ kN$

cross-section at flange

tension resistance $N_{t,Rd} = 55.18\ kN$

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 18.0\ kN$

$F_{Ed} = 18.0\ kN < F_{Rd} = 55.2\ kN \Rightarrow U = 0.326 < 1$ ok

cross-section at web

shear resistance $V_{Rd} = 178.01\ kN$

design value: $F_{Ed} = F = 17.2\ kN$

$F_{Ed} = 17.2\ kN < F_{Rd} = 178.0\ kN \Rightarrow U = 0.097 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 2.94\ kN/cm$, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 0.50\ kN/cm$, $b_1 = 29.4\ mm$

$\sigma_{1,w,Ed} = 10.22\ kN/cm^2 < f_{1w,d} = 36.00\ kN/cm^2 \Rightarrow U = 0.284 < 1$ ok

$\sigma_{2,w,Ed} = 9.80\ kN/cm^2 < f_{2w,d} = 25.92\ kN/cm^2 \Rightarrow U = 0.378 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 0.63\ kN/cm$, $l_1 = 137.0\ mm$

$\sigma_{1,w,Ed} = 3.63\ kN/cm^2 < f_{1w,d} = 36.00\ kN/cm^2 \Rightarrow U = 0.101 < 1$ ok

3.1.5.4. verification result

maximum utilization: max U = 0.982 $<$ 1 ok

3.1.6. rotational stiffness

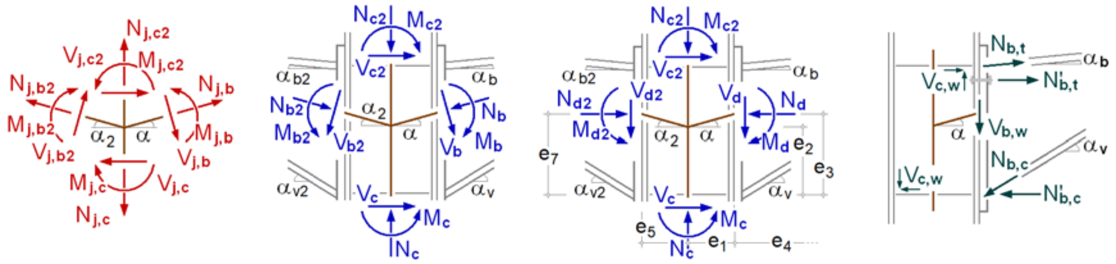
stiffness coefficients
equivalent stiffness coefficient for 2 tension-bolt-rows:
1: k_3 = 3.09 mm, k_4 = 11.52 mm, k_5 = 13.59 mm, k_{10} = 4.12 mm $\Rightarrow$ $k_{eff,1} = 1 / \Sigma(1/k_{i,1})$ = 1.377 mm
2: k_3 = 3.09 mm, k_4 = 11.52 mm, k_5 = 13.59 mm, k_{10} = 4.12 mm $\Rightarrow$ $k_{eff,2} = 1 / \Sigma(1/k_{i,2})$ = 1.377 mm
$k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq}$ = 2.058 mm, $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r)$ = 115.1 mm
$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z)$ = 3.14 mm
$k_2 = \infty$ (stiffened)
rotational stiffness
initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i)$ = 3454.4 kNm/rad, $z = z_{eq}$ = 115.1 mm, $\Sigma(1/k_i)$ = 0.805 mm^{-1}
$|N_{b,Ed}|$ = 0.88 kN $<$ 5%$\cdot N_{pl,Rd}$ = 28.14 kN ok
rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu$ = 3454.4 kNm/rad, μ = 1
rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd}$ = 0.089°

3.2. connection left

notes
connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.

3.2.1. design values

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



slope angle: $\alpha_b = \alpha_v = \alpha$ = 0°
distance: e_1 = 90.0 mm, e_3 = 86.0 mm, e_2 = 86.0 mm, e_5 = 90.0 mm, e_7 = 86.0 mm

internal forces and moments perpendicular to the connection planes
periphery beam (right)
N_d = 7.31 kN, M_d = 9.31 kNm, V_d = 6.05 kN
periphery beam (left)
N_{d2} = -0.88 kN, M_{d2} = -5.26 kNm, V_{d2} = 1.57 kN
periphery column (bottom)
N_c = 7.09 kN, M_c = 9.87 kNm, V_c = -13.65 kN
periphery column (top)
N_{c2} = 2.61 kN, M_{c2} = -3.75 kNm, V_{c2} = -5.46 kN
partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_{ep}$ = 9.27 kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 50.22 kN, z_b = 172.0 mm, z_{bu} = 86.0 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b$ = 57.53 kN, z_b = 172.0 mm, z_{bo} = 86.0 mm

3.2.2. resistance of cross-section

column bottom
plastic cross-sectional check for N = -7.09 kN, M_y = -9.87 kNm, V_z = -13.65 kN
valid normal/shear stress: zul σ_{Rd} = 23.50 kN/cm², zul τ_{Rd} = 13.57 kN/cm²
top flange: resistance forces $N_{max,O}$ = 171.08 kN, $N_{min,O}$ = -171.08 kN
bottom flange: resistance forces $N_{max,U}$ = 171.08 kN, $N_{min,U}$ = -171.08 kN
web: shear force V_S = -13.65 kN, shear stress τ_S = 1.50 kN/cm² $\Rightarrow$ $U_{\tau,S}$ = 0.110
resistance forces $N_{max,S}$ = 212.92 kN, $N_{min,S}$ = -212.92 kN
main bending: axial force N = -7.09 kN, resistance forces N_{max} = 555.08 kN, N_{min} = -555.08 kN $\Rightarrow$ U_N = 0.013
moment M_y = -9.87 kNm, resistance moments $M_{y,max}$ = 38.57 kNm, $M_{y,min}$ = -38.57 kNm $\Rightarrow$ U_{My} = 0.256
total (possibly due to load increase): max U = 0.263 $<$ 1 ok
utilizations: resistance U_σ = 0.263 $<$ 1 ok, c/t-ratio $U_{c/t}$ = 0.164 $<$ 1 ok
column top
plastic cross-sectional check for N = -2.61 kN, M_y = 3.75 kNm, V_z = -5.46 kN
valid normal/shear stress: zul σ_{Rd} = 23.50 kN/cm², zul τ_{Rd} = 13.57 kN/cm²
top flange: resistance forces $N_{max,O}$ = 171.08 kN, $N_{min,O}$ = -171.08 kN
bottom flange: resistance forces $N_{max,U}$ = 171.08 kN, $N_{min,U}$ = -171.08 kN

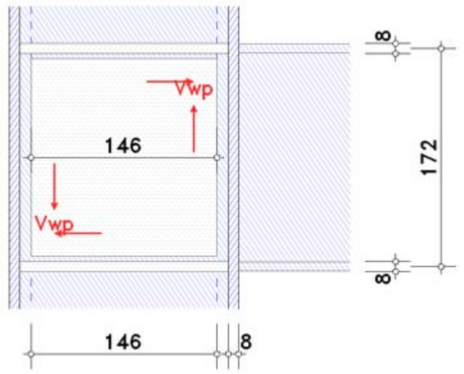
web: shear force $V_S = -5.46$ kN, shear stress $\tau_S = 0.60$ kN/cm² $\Rightarrow$ $U_{\tau,S} = 0.044$
resistance forces $N_{max,S} = 214.02$ kN, $N_{min,S} = -214.02$ kN
main bending: axial force $N = -2.61$ kN, resistance forces $N_{max} = 556.18$ kN, $N_{min} = -556.18$ kN $\Rightarrow$ $U_N = 0.005$
moment $M_y = 3.75$ kNm, resistance moments $M_{y,max} = 38.63$ kNm, $M_{y,min} = -38.63$ kNm $\Rightarrow$ $U_{My} = 0.097$
total (possibly due to load increase): max $U = 0.100 < 1$ ok
utilizations: resistance $U_\sigma = 0.100 < 1$ ok, c/t-ratio $U_{c/t} = 0.101 < 1$ ok

beam
plastic cross-sectional check for $N = -7.31$ kN, $M_y = -9.27$ kNm, $V_z = 6.05$ kN
valid normal/shear stress: zul $\sigma_{Rd} = 23.50$ kN/cm², zul $\tau_{Rd} = 13.57$ kN/cm²
top flange: resistance forces $N_{max,O} = 171.08$ kN, $N_{min,O} = -171.08$ kN
bottom flange: resistance forces $N_{max,U} = 171.08$ kN, $N_{min,U} = -171.08$ kN
web: shear force $V_S = 6.05$ kN, shear stress $\tau_S = 0.66$ kN/cm² $\Rightarrow$ $U_{\tau,S} = 0.049$
resistance forces $N_{max,S} = 213.97$ kN, $N_{min,S} = -213.97$ kN
main bending: axial force $N = -7.31$ kN, resistance forces $N_{max} = 556.13$ kN, $N_{min} = -556.13$ kN $\Rightarrow$ $U_N = 0.013$
moment $M_y = -9.27$ kNm, resistance moments $M_{y,max} = 38.62$ kNm, $M_{y,min} = -38.62$ kNm $\Rightarrow$ $U_{My} = 0.240$
total (possibly due to load increase): max $U = 0.243 < 1$ ok
utilizations: resistance $U_\sigma = 0.243 < 1$ ok, c/t-ratio $U_{c/t} = 0.160 < 1$ ok

3.2.3. basic components

3.2.3.1. Gk 1: Column web panel in shear

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = |1 - M_{j2}/M_{j1}| = 1.55$ for $M_{j1} = 9.86$ kNm, $M_{j2} = -5.40$ kNm

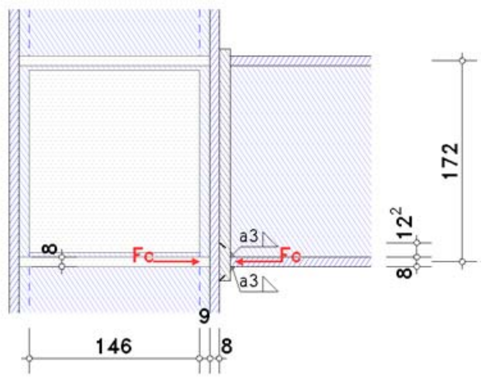


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

web thickness incl. reinforcement $t_{wc} = 10.6$ mm
slenderness of column web $d_c/t_{wc} = 13.77 < 69 \cdot \varepsilon = 69.00$ $\Rightarrow$ method applicable
shear area with reinforcement $A_v = 18.99$ cm²
plastic shear resistance without stiffeners $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_v) / (3^{1/2} \cdot \gamma_{M0}) = 231.9$ kN
placing of intermediate web stiffeners:
additional resistance $V_{wp,add,Rd} = 4 \cdot M_{pl,fc,Rd}/d_{st} = 8.0$ kN
$V_{wp,add,Rd} > 2 \cdot (M_{pl,fc,Rd} + M_{pl,st,Rd})/d_{st} = 7.7$ kN $\Rightarrow$ $V_{wp,add,Rd} = 7.7$ kN
plastic shear resistance with transverse stiffeners $V_{wp,Rd} = 239.6$ kN

3.2.3.2. Gk 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = |1 - M_{j2}/M_{j1}| = 1.55$ for $M_{j1} = 9.86$ kNm, $M_{j2} = -5.40$ kNm
longitudinal compressive stress in column web $\sigma_{com,Ed} = 57.65$ N/mm²



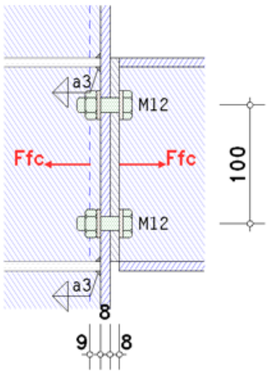
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: $c/t = 5.4 \cdot \varepsilon \leq 33 \cdot \varepsilon$ $\Rightarrow$ section class $1 \leq 2$ ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) $a = 172.0$ mm
web height between the flanges $h_{wc} = 164.0$ mm
moment of inertia of stiffeners $I_{st} = 50.24$ cm⁴
minimum moment of inertia for $a/h_{wc} = 1.05 < 2^{1/2}$: $I_{st,min} = 3.33$ cm⁴ $< I_{st}$ ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners $I_T = 0.73$ cm⁴
polar moment of inertia of stiffeners $I_p = 5.43$ cm⁴

I_T / $I_p \approx 0.135 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of stiffened webs with transverse compression:
area of stiffeners incl. web A_{st} = 7.28 cm²
slenderness λ = 0.066
$\lambda \leq 0.2 \Rightarrow$ no deduction (χ = 1.0)
design value of resistance of flexural buckling $F_{c,w,Rd}$ = 155.5 kN

resistance of upper beam flange:
reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: $c/t = 5.4 \cdot \varepsilon \leq 33 \cdot \varepsilon \Rightarrow$ section class 1 ≤ 2 ok
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 172.0 mm
web height between the flanges h_{wc} = 164.0 mm
moment of inertia of stiffeners I_{st} = 50.24 cm⁴
minimum moment of inertia for $a/h_{wc} = 1.05 < 2^{1/2}$: $I_{st,min}$ = 3.33 cm⁴ < I_{st} ok
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners I_T = 0.73 cm⁴
polar moment of inertia of stiffeners I_p = 5.43 cm⁴
I_T / $I_p \approx 0.135 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ ok
resistance of stiffened webs with transverse compression:
area of stiffeners incl. web A_{st} = 7.28 cm²
slenderness λ = 0.066
$\lambda \leq 0.2 \Rightarrow$ no deduction (χ = 1.0)
design value of resistance of flexural buckling $F_{c,w,Rd}$ = 155.5 kN

3.2.3.3. Gk 4: column flange in bending



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

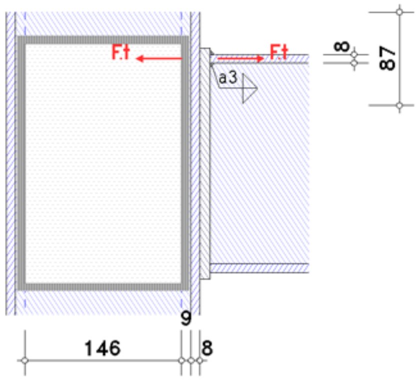
equivalent T-stub flange (each individual bolt-row):
here: number of bolt-rows n_b = 1
row 1
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 87.0 mm, $l_{eff,cp}$ = 95.2 mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 87.0 mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 0.33 kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd}$ = 97.11 kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n))$ = 123.51 kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 73.14 kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 97.11 kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 73.14 kN
row 2
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 87.0 mm, $l_{eff,cp}$ = 95.2 mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 87.0 mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 0.33 kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd}$ = 97.11 kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n))$ = 123.51 kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 73.14 kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 97.11 kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 73.14 kN
resistances and effective lengths of column flange in bending (per bolt-row)
$F_{t,fc,Rd,1}$ = 73.14 kN, $l_{eff,1}$ = 87.0 mm

$F_{t,fc,Rd,2}$ = 73.14 kN, $l_{eff,2}$ = 87.0 mm

equivalent T-stub flange (group of bolts 1):
here: number of bolt-rows n_b = 2 (between stiffeners)
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp})$ = 187.7 mm, $\Sigma l_{eff,cp}$ = 295.2 mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc}$ = 187.7 mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 0.71 kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd}$ = 194.23 kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n))$ = 266.59 kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 149.31 kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 194.23 kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 149.31 kN
resistances and effective lengths of column flange in bending (per bolt group):
$F_{ep,Rd,1-2}$ = 149.31 kN, Σl_{eff} = 187.7 mm, 2 rows

3.2.3.4. Gk 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = |1 - M_{j2}/M_{j1}|$ = 1.55 for M_{j1} = 9.86 kNm, M_{j2} = -5.40 kNm

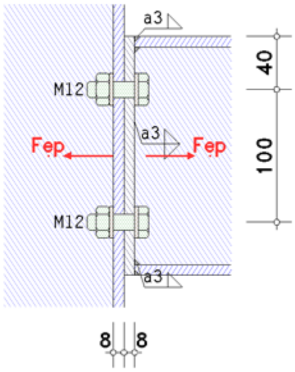


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

each individual bolt-row:
row 1
effective width $b_{eff,t}$ = 87.0 mm (l_{eff} from bc 4)
reinforcement of column web with 1 supplementary web plate:
fillet weld with $a_s \geq t_s/2^{1/2}$ = 4.9 mm: effective web thickness $t_{w,eff} = t_{wc} + 0.4 \cdot t_s$ = 7.4 mm for S235
reduction factor for interaction with shear stress $1 < \beta < 2 \Rightarrow \omega$ = 0.855
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0}$ = 129.6 kN
row 2
effective width $b_{eff,t}$ = 87.0 mm (l_{eff} from bc 4)
reinforcement of column web with 1 supplementary web plate:
fillet weld with $a_s \geq t_s/2^{1/2}$ = 4.9 mm: effective web thickness $t_{w,eff} = t_{wc} + 0.4 \cdot t_s$ = 7.4 mm for S235
reduction factor for interaction with shear stress $1 < \beta < 2 \Rightarrow \omega$ = 0.855
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0}$ = 129.6 kN

group of bolt-rows, group 1:
effective width $b_{eff,t}$ = 187.7 mm (l_{eff} from bc 4)
reinforcement of column web with 1 supplementary web plate:
fillet weld with $a_s \geq t_s/2^{1/2}$ = 4.9 mm: effective web thickness $t_{w,eff} = t_{wc} + 0.4 \cdot t_s$ = 7.4 mm for S235
reduction factor for interaction with shear stress $1 < \beta < 2 \Rightarrow \omega$ = 0.628
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0}$ = 205.5 kN

3.2.3.5. Gk 5: end-plate in bending



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

part of end-plate between beam flanges

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 (end-plate):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 102.8$ mm, $l_{eff,cp} = 119.1$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 102.8$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.39$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 97.11$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 109.09$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 69.71$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 97.11$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 69.71$ kN

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

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 157.04$ kN (≥ 69.71 kN, not decisive)

row 2

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 102.8$ mm, $l_{eff,cp} = 119.1$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 102.8$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.39$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 97.11$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 109.09$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 69.71$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 97.11$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 69.71$ kN

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

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 157.04$ kN (≥ 69.71 kN, not decisive)

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

$F_{ep,Rd,1} = 69.71$ kN, $l_{eff,1} = 102.8$ mm

$F_{ep,Rd,2} = 69.71$ kN, $l_{eff,2} = 102.8$ mm

equivalent T-stub flange (group of bolts 1):

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

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

in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 204.8$ mm, $\Sigma l_{eff,cp} = 319.1$ mm

in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 204.8$ mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 0.77$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 194.23$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = ((8 \cdot n - 2 \cdot e_w) \cdot M_{pl,1,Rd}) / (2 \cdot m \cdot n - e_w \cdot (m+n)) = 217.30$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 139.25$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 194.23$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 139.25$ kN

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

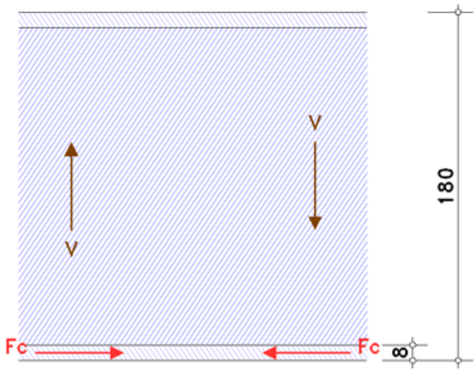
tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff} = 312.83$ kN (≥ 139.25 kN, not decisive)

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

$F_{ep,Rd,1-2} = 139.25$ kN, $\Sigma l_{eff} = 204.8$ mm, 2 rows

3.2.3.6. Gk 7: beam flange and web in compression

flange bottom: section class for $c/(\varepsilon \cdot t) = 4.23$: 1
web: section class for $\alpha = 0.53$ and $c/(\varepsilon \cdot t) = 27.55$: 1
section class of beam: 1
taking into account the moment-shear force-interaction $V_{Ed} = 6.1$ kN

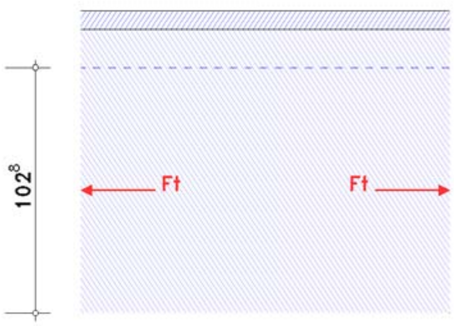


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

stress due to bending with shear force: $V_{Ed} = 6.1 \text{ kN} \le 76.3 \text{ kN} = V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 39.01$ kNm, $W_{pl} = 166.00 \text{ cm}^3$
resistance of a flange (and web) with compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 226.80$ kN

resistance of upper beam flange:
stress due to bending with shear force: $V_{Ed} = 6.1 \text{ kN} \le 76.3 \text{ kN} = V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 39.01$ kNm, $W_{pl} = 166.00 \text{ cm}^3$
resistance of a flange (and web) with compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 226.80$ kN

3.2.3.7. Gk 8: beam web in tension

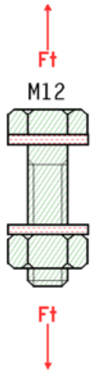


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

each individual bolt-row:
row 1
effective width $b_{eff,t,wb} = 102.8$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 128.1$ kN
row 2
effective width $b_{eff,t,wb} = 102.8$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 128.1$ kN

group of bolt-rows, group 1:
effective width $b_{eff,t,wb} = 204.8$ mm (l_{eff} from bc 5)
resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 255.1$ kN

3.2.3.8. Gk 10: bolts in tension

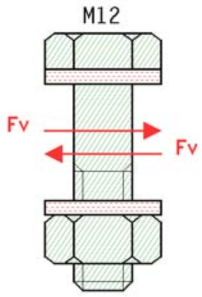


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

tension resistance of one bolt $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 48.56$ kN, $k_2 = 0.90$
punching shear load capicity $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 99.69$ kN, $t_p = 8.0$ mm
tension-/punching shear load capicity for 2 bolts: $\Sigma F_{tp,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 97.11$ kN

3.2.3.9. Gk 11: bolts in shear

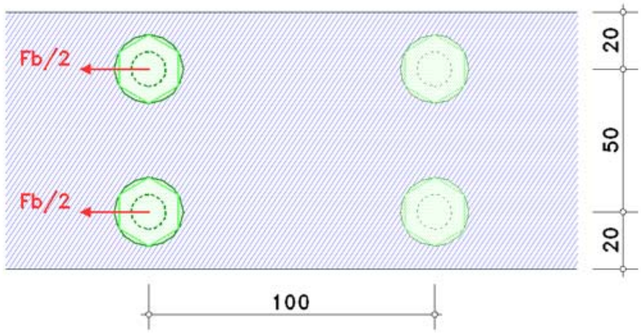
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.



shear resistance per shear plane $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2} = 43.43$ kN, $\alpha_v = 0.60$
shear resistance of 2 bolts (1-shear): $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd} = 86.86$ kN

3.2.3.10. Gk 12: plate with bearing resistance

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



row 1
end-plate:
bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 138.24$ kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 138.24$ kN
row 2
end-plate:
bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 138.24$ kN
column flange:
bolt 1: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bolt 2: bearing resistance $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 69.12$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 138.24$ kN

bearing resistance (2 rows)
$\Sigma F_{b,Rd,1} = 138.24$ kN
$\Sigma F_{b,Rd,2} = 138.24$ kN

3.2.4. connection capacity

3.2.4.1. moment resistance

distance of tension-bolt-rows from centre of compression: h_1 = 136.0 mm, h_2 = 36.0 mm

resistance per bolt-row (MNV-interaction)
row 1: $F_{tr,Rd}$ = 69.7 kN
row 2: $F_{tr,Rd}$ = 69.5 kN

resistance of flanges (MNV-interaction)
bottom: $F_{c,Rd}$ = 149.3 kN

moment resistance (MNV-interaction)
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r)$ = 12.0 kNm

shear force resistance (MNV-interaction)
$V_{j,Rd}$ = 8.3 kN

3.2.4.2. shear resistance

shear resistance of end plate
end-plate: $V_{ep,Rd}$ = 158.47 kN
welds: $F_{w,Rd}$ = 182.07 kN (≥ 158.47 kN, not decisive)

shear resistance of column web
$V_{wp,Rd}/\beta_j$ = 154.8 kN

3.2.4.3. total

$V_{wp,Rd}/\beta_j$ = 154.8 kN $V_{ep,Rd}$ = 158.5 kN

3.2.5. verifications

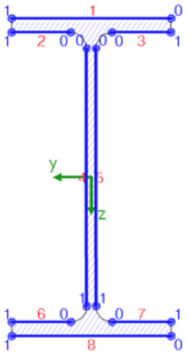
3.2.5.1. verification of the connection capacity by means of the component method

U_{MNV} = 0.725 < 1 ok
$V_{c,w,Ed}/(V_{wp,Rd}/\beta_j)$ = 0.760 < 1 ok
$V_{Ed}/V_{ep,Rd}$ = 0.038 < 1 ok

3.2.5.2. 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 = 3.0 mm	l_w = 90.0 mm
weld 2:	a_w = 3.0 mm	l_w = 33.4 mm
weld 3:	siehe weld 2	
weld 4:	a_w = 3.0 mm	l_w = 146.0 mm
weld 5:	siehe weld 4	
weld 6:	a_w = 3.0 mm	l_w = 33.4 mm
weld 7:	siehe weld 6	
weld 8:	a_w = 3.0 mm	l_w = 90.0 mm

design values refering to centroid of the section:
N_{Ed} = -7.31 kN, $M_{y,Ed}$ = -9.31 kNm, $V_{z,Ed}$ = 6.05 kN
cross-sectional properties refering to centroid of the line cross-section:
ΣA_w = 18.16 cm², $A_{w,z}$ = 8.76 cm², Σl_w = 60.5 cm
$I_{w,y}$ = 862.10 cm⁴, $I_{w,z}$ = 72.88 cm⁴, $W_{w,t}$ = 14.27 cm³, Δz_w = 0.0 mm
distribution of internal forces and moments:

weld 1:	N_w = 25.17 kN	
weld 2:	N_w = 8.46 kN	
weld 3:	siehe weld 2	
weld 4:	N_w = -1.76 kN	$M_{y,w}$ = -0.84 kNm
weld 5:	siehe weld 4	
weld 6:	N_w = -9.27 kN	
weld 7:	siehe weld 6	
weld 8:	N_w = -27.34 kN	

from conventionel distribution of shear force: $V_{z,w}$ = 6.05 kN

verifications in weld edges:

weld 1, pt. 0: $\sigma_{w,x} = 93.21$ N/mm² $\Rightarrow U_w = 0.366 < 1$ ok

weld 2, pt. 0: $\sigma_{w,x} = 84.57$ N/mm² $\Rightarrow U_w = 0.332 < 1$ ok

weld 4, pt. 0: $\sigma_{w,x} = 74.84$ N/mm² $\tau_{w,z} = 6.91$ N/mm² $\Rightarrow U_w = 0.296 < 1$ ok

pt. 1: $\sigma_{w,x} = -82.90$ N/mm² $\tau_{w,z} = 6.91$ N/mm² $\Rightarrow U_w = 0.327 < 1$ ok

weld 6, pt. 0: $\sigma_{w,x} = -92.62$ N/mm² $\Rightarrow U_w = 0.364 < 1$ ok

weld 8, pt. 0: $\sigma_{w,x} = -101.26$ N/mm² $\Rightarrow U_w = 0.398 < 1$ ok

Result:

weld 8, pt. 0: $\sigma_{w,x} = -101.26$ N/mm²

Max: $\sigma_{1,w,Ed} = 14.32$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm²,

$\sigma_{2,w,Ed} = 7.16$ kN/cm² $< f_{2w,d} = 25.92$ kN/cm² $\Rightarrow U_w = 0.398 < 1$ ok

3.2.5.3. verification of web stiffeners

compression stiffener

$F_{c,Ed} = 82.79$ kN

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 30.8$ kN, $H = F \cdot e_F/e_H = 5.3$ kN

assumption: stiffeners do not buckle: $c/t = 5.4 \cdot \varepsilon \leq 33 \cdot \varepsilon \Rightarrow$ section class $1 \leq 2$ ok

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 55.18$ kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 32.1$ kN

$F_{Ed} = 32.1$ kN $< F_{Rd} = 55.2$ kN $\Rightarrow U = 0.582 < 1$ ok

cross-section at web

shear resistance $V_{Rd} = 178.01$ kN

design value: $F_{Ed} = F = 30.8$ kN

$F_{Ed} = 30.8$ kN $< F_{Rd} = 178.0$ kN $\Rightarrow U = 0.173 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 5.25$ kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 0.90$ kN/cm, $b_1 = 29.4$ mm

$\sigma_{1,w,Ed} = 18.25$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm² $\Rightarrow U = 0.507 < 1$ ok

$\sigma_{2,w,Ed} = 17.49$ kN/cm² $< f_{2w,d} = 25.92$ kN/cm² $\Rightarrow U = 0.675 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 1.12$ kN/cm, $l_1 = 137.0$ mm

$\sigma_{1,w,Ed} = 6.49$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm² $\Rightarrow U = 0.180 < 1$ ok

stiffener in tension

$F_{t,Ed} = 75.48$ kN

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 28.1$ kN, $H = F \cdot e_F/e_H = 4.8$ kN

cross-section at flange

tension resistance $N_{t,Rd} = 55.18$ kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 29.3$ kN

$F_{Ed} = 29.3$ kN $< F_{Rd} = 55.2$ kN $\Rightarrow U = 0.531 < 1$ ok

cross-section at web

shear resistance $V_{Rd} = 178.01$ kN

design value: $F_{Ed} = F = 28.1$ kN

$F_{Ed} = 28.1$ kN $< F_{Rd} = 178.0$ kN $\Rightarrow U = 0.158 < 1$ ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 4.78$ kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 0.82$ kN/cm, $b_1 = 29.4$ mm

$\sigma_{1,w,Ed} = 16.63$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm² $\Rightarrow U = 0.462 < 1$ ok

$\sigma_{2,w,Ed} = 15.94$ kN/cm² $< f_{2w,d} = 25.92$ kN/cm² $\Rightarrow U = 0.615 < 1$ ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 1.02$ kN/cm, $l_1 = 137.0$ mm

$\sigma_{1,w,Ed} = 5.92$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm² $\Rightarrow U = 0.164 < 1$ ok

3.2.5.4. verification result

maximum utilization: max $U = 0.760 < 1$ ok

3.2.6. rotational stiffness

stiffness coefficients

equivalent stiffness coefficient for 2 tension-bolt-rows:

1: $k_3 = 3.09$ mm, $k_4 = 11.52$ mm, $k_5 = 6.96$ mm, $k_{10} = 4.39$ mm $\Rightarrow k_{eff,1} = 1 / \Sigma(1/k_{i,1}) = 1.279$ mm

2: $k_3 = 3.09$ mm, $k_4 = 11.52$ mm, $k_5 = 6.96$ mm, $k_{10} = 4.39$ mm $\Rightarrow k_{eff,2} = 1 / \Sigma(1/k_{i,2}) = 1.279$ mm

$k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq} = 1.912$ mm, $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r) = 115.1$ mm

$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z) = 4.05$ mm

$k_2 = \infty$ (stiffened)

rotational stiffness

initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i) = 3611.8$ kNm/rad, $z = z_{eq} = 115.1$ mm, $\Sigma(1/k_i) = 0.770$ mm^{-1}

$|N_{b,Ed}| = 7.31$ kN $< 5\% \cdot N_{pl,Rd} = 28.14$ kN ok

rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu = 3611.8$ kNm/rad, $\mu = 1$

rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd} = 0.138°$