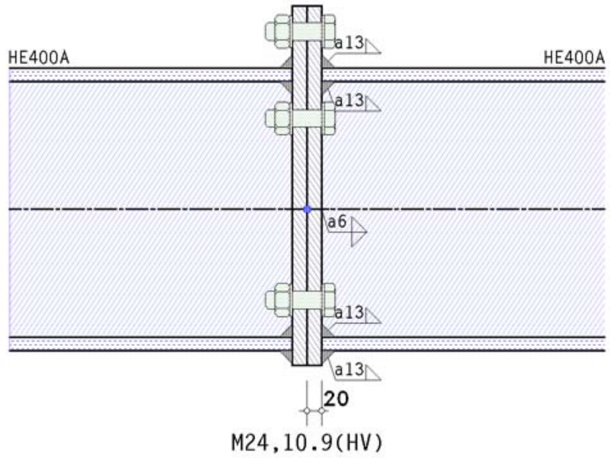
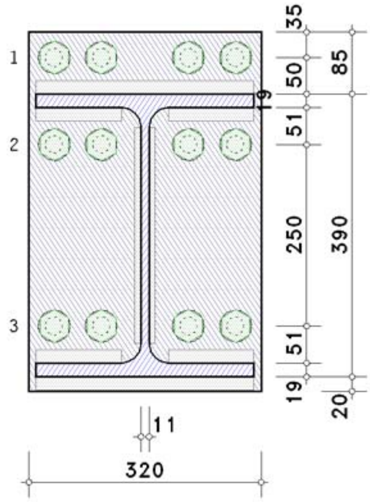
POS. 164: SCHMIDT IH4

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

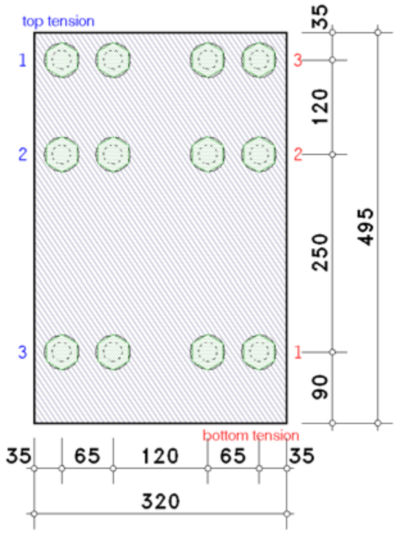
Rigid beam splice EC 3-1-8 (12.10), NA: Deutschland

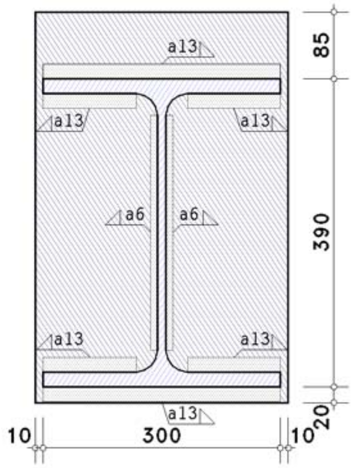
1. input report





details (section A - A)

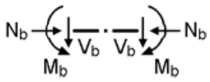




bolts
bolt class 10.9, bolt size M24
large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}* = 0.7 \cdot f_{yb} \cdot A_s = 222.4$ kN)
shear plane passes through the unthreaded portion of the bolt
beam parameters
section HE400A, steel grade S355
verification parameters
bolted end-plate connection:
thickness $t_p = 20.0$ mm, width $b_p = 320.0$ mm, length $l_p = 495.0$ mm, steel grade S235
projections $h_{p,o} = 85.0$ mm, $h_{p,u} = 20.0$ mm
bolts in connection:
3 bolt-rows with 4 bolts
row 1: 4 bolts, row 2: 4 bolts, row 3: 4 bolts
of these 2 bolt-rows top in tension (rows 1-2)
and 2 bolt-rows for shear transfer top (rows 2-3)
of these 1 bolt-row bottom in tension (row 3)
and 1 bolt-row for shear transfer bottom (row 3)
calculation method (4 bolts per row) acc. to B. Schmidt, Dissertation, TU Dortmund
centre distance between outer and inner bolt $w_2 = 65.0$ mm
centre distance of the bolts to the lateral edge of the end-plate $e_2 = 35.0$ 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 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 = 13.0 mm
beam web: fillet weld, weld thickness a = 6.0 mm
beam flange bottom: fillet weld, weld thickness a = 13.0 mm

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



Lk 1: $M_{b,Ed}$ = 460.00 kNm

partial safety factors for material

resistance of cross-sections γ_{M0} = 1.00
resistance of members in stability failure γ_{M1} = 1.10
resistance of bolts, welds, plates in bearing γ_{M2} = 1.25
prestressing of high strength bolts γ_{M7} = 1.10

check of data

ok

distances between bolt-rows at end-plate

horizontal:	e_2 = 35.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_2 = 35.0 mm < $4 \cdot t$ + 40 mm = 120.0 mm
horizontal:	p_2 = 65.0 mm > $2.4 \cdot d_0$ = 62.4 mm,	p_2 = 65.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
horizontal:	p_2 = 120.0 mm > $2.4 \cdot d_0$ = 62.4 mm,	p_2 = 120.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
vertical:	e_1 = 35.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 35.0 mm < $4 \cdot t$ + 40 mm = 120.0 mm
vertical:	p_1 = 120.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 120.0 mm < min($14 \cdot t$, 200 mm) = 200.0 mm
vertical:	p_1 = 250.0 mm > $2.2 \cdot d_0$ = 57.2 mm,	p_1 = 250.0 mm > min($14 \cdot t$, 200 mm) = 200.0 mm !!
vertical:	e_1 = 90.0 mm > $1.2 \cdot d_0$ = 31.2 mm,	e_1 = 90.0 mm < $4 \cdot t$ + 40 mm = 120.0 mm

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

notes

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

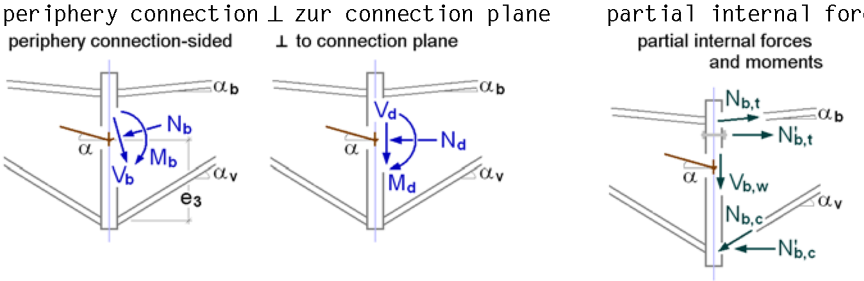
2. Lk 1

notes

connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.
no consideration of bolt groups in joints with 4 bolts per row.

2.1. design values

periphery connection ⊥ zur connection plane   partial internal forces and moments



slope angle: $\alpha_b = \alpha_v = \alpha = 0°$

internal forces and moments perpendicular to the connection planes

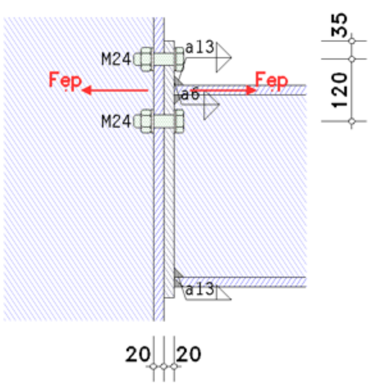
periphery beam
M_d = 460.00 kNm

partial internal forces and moments

internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_{ep}$ = 460.00 kNm
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 1239.89 kN, z_b = 371.0 mm, z_{bu} = 185.5 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b$ = 1239.89 kN, z_b = 371.0 mm, z_{bo} = 185.5 mm

2.2. basic components

2.2.1. Gk 5: end-plate in bending



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

IH4-connection: connections with 4 bolts per bolt-row are not treated in EC 3-1-8.
verification according to B. Schmidt, Dissertation, TU Dortmund.

extended part of end-plate

in projecting part of end plate only one bolt-row (n_b = 1) is considered (4 bolts per row).

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 160.0 mm, $l_{eff,cp}$ = 190.4 mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 160.0 mm

tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 3.76 kNm

in mode 3: $\Sigma F_{t,Rd} = 4 \cdot n_b \cdot F_{t,Rd}$ = 1016.64 kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (M_{pl,1,Rd} \cdot (4-e'/n)) / (m - ((m+n) \cdot e') / (2 \cdot n))$ = 734.70 kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 613.19 kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 1016.64 kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 613.19 kN

tension resistance of beam flange: max $F_{T,Rd} = (A_F \cdot f_{y,st}) / \gamma_{M0}$ = 2023.50 kN > $F_{T,Rd}$ ok

resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2})$ = 320.0 N/mm²

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff}$ = 941.30 kN (≥ 613.19 kN, not decisive)

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

$F_{t,ep,Rd,1}$ = 613.19 kN, $l_{eff,1}$ = 160.0 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 (4 bolts per row)

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 349.9 mm, $l_{eff,cp}$ = 349.9 mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 362.5 mm

tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 8.22 kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 8.52 kNm

in mode 3: $\Sigma F_{t,Rd} = 4 \cdot n_b \cdot F_{t,Rd}$ = 1016.64 kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,F,Rd} = (M_{pl,1,Rd} \cdot \delta_F \cdot (4-e'/n_s)) / (m_s - ((m_s+n_s) \cdot e') / (2 \cdot n_s))$ = 1159.91 kN

$F_{T,1,S,Rd} = (M_{pl,1,Rd} \cdot \delta_S \cdot (4-e'/n)) / (m - ((m+n) \cdot e') / 2 \cdot n)$ = 181.57 kN

$F_{T,1,Rd} = F_{T,1,F,Rd} + F_{T,1,S,Rd}$ = 1341.48 kN

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

$F_{T,2,F,Rd} = (2 \cdot M_{pl,2,Rd} \cdot \rho_{Fa} \cdot \delta_F + 2 \cdot n_s \cdot F_{t,Rd}) / (m_s+n_s)$ = 347.07 kN

$F_{T,2,S,Rd} = (2 \cdot M_{pl,2,Rd} \cdot (\rho_{Fi} \cdot \delta_F + \delta_S) + 2 \cdot n \cdot F_{t,Rd}) / (m+n)$ = 391.91 kN

$F_{T,2,Rd} = F_{T,2,F,Rd} + F_{T,2,S,Rd}$ = 738.97 kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 1016.64 kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 738.97 kN

A_F = 0: check of tension resistance of beam flange and web is not possible!

resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2})$ = 320.0 N/mm²

tension resistance of welds: $F_{T,w,Rd} = 2^{1/2} \cdot f_{1w,d} \cdot a \cdot l_{eff}$ = 950.06 kN (≥ 738.97 kN, not decisive)

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

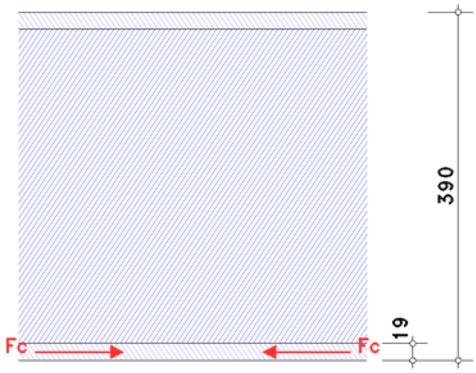
$F_{ep,Rd,2}$ = 738.97 kN, $l_{eff,2}$ = 349.9 mm

2.2.2. Gk 7: beam flange and web in compression

flange bottom: section class for $c/(\varepsilon \cdot t)$ = 7.60: 1

web: section class for α = 0.50 and $c/(\varepsilon \cdot t)$ = 33.30: 1

section class of beam: 1



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

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0}$ = 909.51 kNm, W_{pl} = 2562.00 cm³

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f)$ = 2451.51 kN

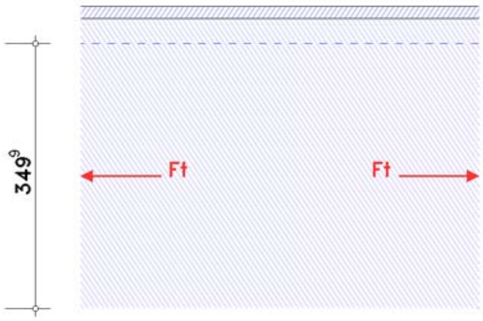
resistance of upper beam flange:

resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0}$ = 909.51 kNm, W_{pl} = 2562.00 cm³

resistance of a flange (and web) with compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 2451.51$ kN

2.2.3. Gk 8: beam web in tension



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

each individual bolt-row:

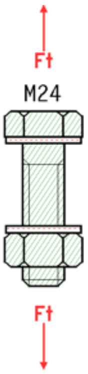
row 2

effective width $b_{eff,t,wb} = 349.9$ mm (l_{eff} from bc 5)

resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 1366.3$ kN

2.2.4. Gk 10: bolts in tension



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

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

punching shear load capicity $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 467.95$ kN, $t_p = 20.0$ mm

tension-/punching shear load capicity for 4 bolts: $\Sigma F_{tp,Rd} = 4 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 1016.64$ kN

2.3. connection capacity

2.3.1. moment resistance

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

resistance per bolt-row

row 1: $F_{tr,Rd} = 613.2$ kN

row 2: $F_{tr,Rd} = 739.0$ kN

$\Sigma F_{tr,Rd} = 1352.2$ kN

potential failure by basic component 5

resistance of flanges

$\Sigma F_{c,Rd}^{*} = 4903.0$ kN

moment resistance

$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 493.4$ kNm

tension resistance

$N_{j,t,Rd} = \Sigma F_{tr,Rd}^{*} = 1352.2$ kN

compression resistance

$N_{j,c,Rd} = \Sigma F_{c,Rd}^{*} = 4903.0$ kN

2.4. verifications

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

internal moment: $M_{Ed} = M_d = 460.00$ kNm

$M_{Ed}/M_{j,Rd} = 0.932 < 1$ ok

2.4.2. verification result

maximum utilization: max U = 0.932 < 1 ok

2.5. rotational stiffness

stiffness coefficients

equivalent stiffness coefficient for 2 tension-bolt-rows:

1: k_5 = 26.21 mm, k_{10} = 8.62 mm ⇒ $k_{eff,1} = 1 / \Sigma(1/k_{i,1})$ = 7.446 mm

2: k_5 = 4.44 mm, k_{10} = 8.62 mm ⇒ $k_{eff,2} = 1 / \Sigma(1/k_{i,2})$ = 7.904 mm

$k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq}$ = 14.954 mm, $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r)$ = 378.5 mm

rotational stiffness

initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i)$ = 449820.1 kNm/rad, $z = z_{eq}$ = 378.5 mm, $\Sigma(1/k_i)$ = 0.067 mm^{-1}

$|M_{j,Ed}|$ = 460.00 kNm > 2/3 $M_{j,Rd}$ = 329.0 kNm ⇒ $\mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi}$ = 2.473, Ψ = 2.7

rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu$ = 181908.5 kNm/rad

rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd}$ = 0.145°

3. final result

utilization/rotation of the connection

Lk	$S_{j,ini}$	S_j	φ_j	U_j	Gleichgewicht ΣH	ΣV	ΣM	
--	MNm/rad	MNm/rad	°		kN	kN	kNm	
1	449.8	181.9	0.145	0.932*	0.00	0.00	460.00	!!

$S_{j,ini}$: 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: max U = 0.932 < 1 ok

minimum rotational stiffness: min S_j = 181.9 MNm/rad, $S_{j,ini}$ = 449.8 MNm/rad, φ_j = 0.145°

verification succeeded