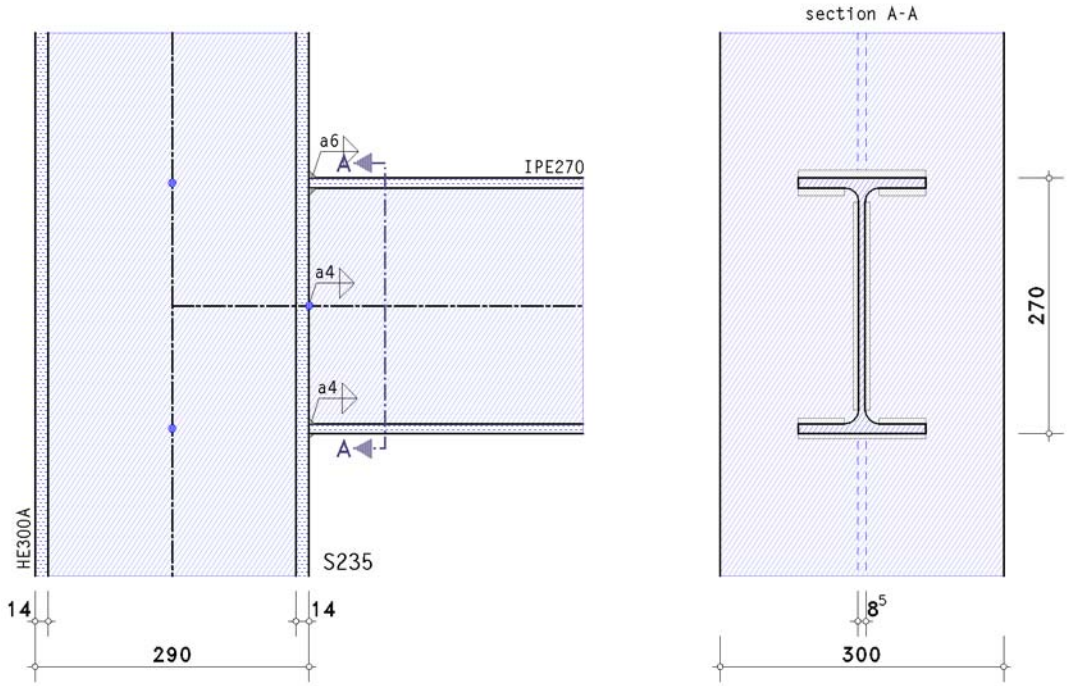
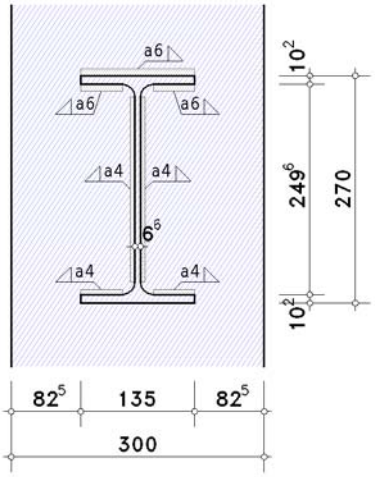
4H-ec3BT version: 4/2013-5l

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

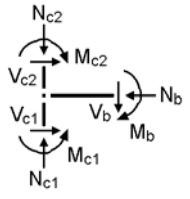
1. input report



details (section A - A)



steel grade
steel grade S235
column parameters
section HE300A
beam parameters
section IPE270
verification parameters
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 6.0 mm
beam web: fillet weld, weld thickness a = 4.0 mm
beam flange bottom: fillet weld, weld thickness a = 4.0 mm
internal forces and moments at the joint periphery refering to the system axes
Lk 1: $M_{b,Ed}$ = 75.00 kNm $V_{b,Ed}$ = 80.00 kN
$N_{c1,Ed}$ = 300.00 kN $M_{c1,Ed}$ = 55.00 kNm (bottom)
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



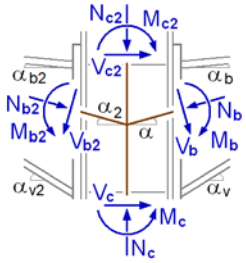
check of data
ok

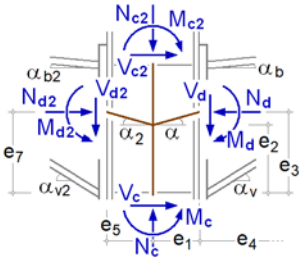
notes
there are several basic components selected which perhaps do not ensure the total loading capacity of the joint.
no verification for cross-sections.
no verification for column web area.
no verfication for welds within the connection.

2. Lk 1

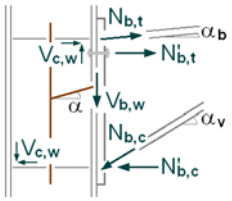
2.1. design values

periphery connection ⊥ zur connection plane





partial internal forces and moments



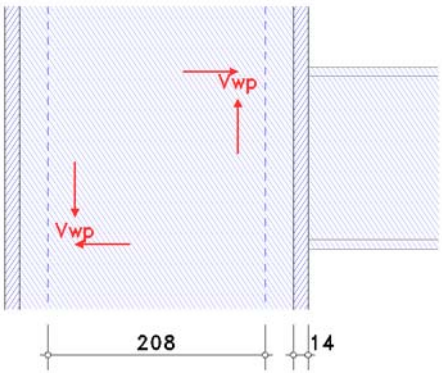
slope angle: $\alpha_b = \alpha_v = \alpha = 0°$
distance: $e_1 = 145.0$ mm, $e_3 = 129.9$ mm, $e_2 = 129.9$ mm

internal forces and moments perpendicular to the connection planes
periphery beam
$M_d = 75.00$ kNm, $V_d = 80.00$ kN
periphery column (bottom)
$N_c = 300.00$ kN, $M_c = 55.00$ kNm
calculation of internal forces and moments at periphery column (top)
$N_{c2} = N_c - V_d = 220.00$ kN
$M_{c2} = M_c + V_c \cdot e_6 - M_d - V_d \cdot e_1 - N_d \cdot (e_6 - e_3) = -31.60$ kNm
partial internal forces and moments
$N_{b,t} = -N_d \cdot z_{bu} / z_b + M_d / z_b = 288.68$ kN, $z_b = 259.8$ mm, $z_{bu} = 129.9$ mm
$N_{b,c} = N_d \cdot z_{bo} / z_b + M_d / z_b = 288.68$ kN, $z_b = 259.8$ mm, $z_{bo} = 129.9$ mm
$V_{b,w} = V_d = 80.00$ kN

2.2. basic components

2.2.1. Gk 1: Column web panel in shear

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.00$ for $M_{j1} = 75.00$ kNm ($M_{j2} = 0$)

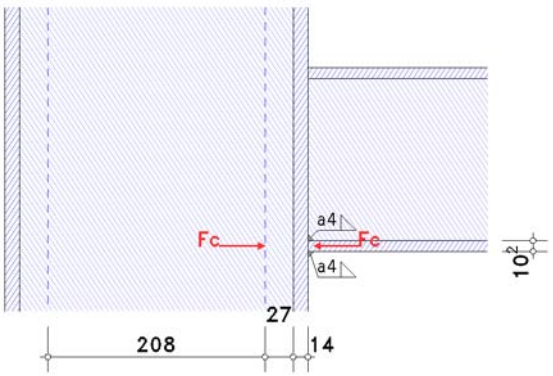


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

slenderness of column web $d_c / t_{wc} = 24.47 < 69 \cdot \varepsilon = 69.00 \Rightarrow$ method applicable
plastic shear resistance $V_{wp,Rd} = (0.9 \cdot f_{y,w} \cdot A_v) / (3^{1/2} \cdot \gamma_{M0}) = 455.2$ kN

2.2.2. Gk 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.00$ for $M_{j1} = 75.00$ kNm ($M_{j2} = 0$)
longitudinal compressive stress in column web $\sigma_{com,Ed} = 57.98$ N/mm²

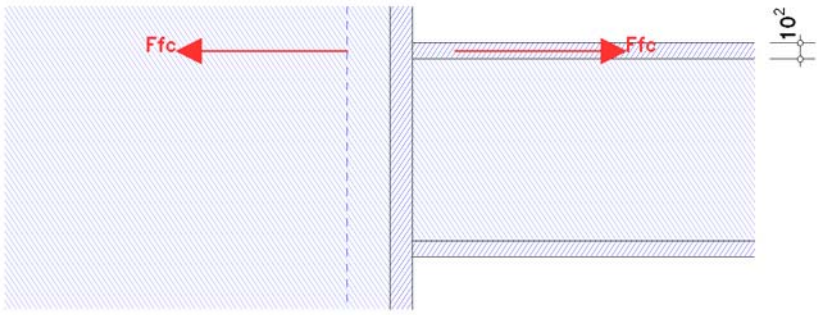


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

effective width of column web in transverse compression $b_{eff,c} = t_{f,b} + 2 \cdot 2^{1/2} \cdot a_b + 5 \cdot (t_{f,c} + s_c) = 226.5$ mm
reduction factor $k_w = 1.0$ for $\sigma_{com,Ed} = 58.0 \text{ N/mm}^2 \le 0.7 \cdot f_{y,w} = 164.5 \text{ N/mm}^2$
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.796$
reduction factor for web buckling $\rho = (\lambda_p - 0.2) / \lambda_p^2 = 0.941$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.862$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 389.88$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 333.35$ kN (decisive)

resistance of upper beam flange:
effective width of column web in transverse compression $b_{eff,c} = t_{f,b} + 2 \cdot 2^{1/2} \cdot a_b + 5 \cdot (t_{f,c} + s_c) = 232.2$ mm
reduction factor $k_w = 1.0$ for $\sigma_{com,Ed} = 58.0 \text{ N/mm}^2 \le 0.7 \cdot f_{y,w} = 164.5 \text{ N/mm}^2$
plate slenderness $\lambda_p = 0.932 \cdot [(b_{eff,c} \cdot d_w \cdot f_y) / (E \cdot t_w^2)]^{1/2} = 0.806$
reduction factor for web buckling $\rho = (\lambda_p - 0.2) / \lambda_p^2 = 0.933$
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.856$
resistance of an unstiffened web in transverse compression:
$F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M0} = 397.04$ kN
$F_{c,w,Rd} = \omega \cdot (k_w \cdot \rho \cdot b_{eff,c} \cdot t_w \cdot f_{y,w}) / \gamma_{M1} = 336.69$ kN (decisive)

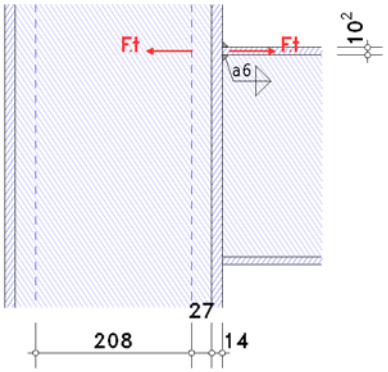
2.2.3. Gk 4: column flange in bending



effective width of unstiffened connections to flanges $b_{eff} = t_w + 2 \cdot s + 7 \cdot k \cdot t_f = 160.5$ mm, $s = 27.0$ mm, $k = 1.00$
$b_{eff} > b_p \Rightarrow b_{eff} = b_p = 135.0$ mm
resistance of column flange in bending
$F_{t,fc,Rd} = (b_{eff,b,fc} \cdot t_{fb} \cdot f_{y,fb}) / \gamma_{M0} = 323.6$ kN

2.2.4. Gk 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_j = 1.00$ for $M_{j1} = 75.00$ kNm ($M_{j2} = 0$)

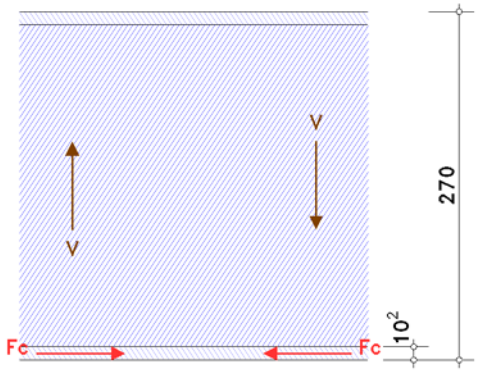


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

effective widtheffective width of column web in transverse tension $b_{eff,t} = t_{fb} + 2 \cdot 2^{1/2} \cdot a_b + 5 \cdot (t_{fc} + s_c) = 232.2$ mm
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.856$
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 397.0$ kN

2.2.5. Gk 7: beam flange and web in compression

flange bottom: section class for $c/(\varepsilon \cdot t) = 4.82$: 1
web: section class for $\alpha = 0.50$ and $c/(\varepsilon \cdot t) = 33.27$: 1
section class of beam: 1
taking into account the moment-shear force-interaction $V_{Ed} = 80.0$ kN



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

stress due to bending with shear force: $V_{Ed} = 80.0$ kN ≤ 150.2 kN $= V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 113.74$ kNm, $W_{pl} = 484.00$ cm³
resistance of a flange (and web) with compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 437.80$ kN

resistance of upper beam flange:
stress due to bending with shear force: $V_{Ed} = 80.0$ kN ≤ 150.2 kN $= V_{pl,Rd}/2 \Rightarrow$ no effect
resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 113.74$ kNm, $W_{pl} = 484.00$ cm³
resistance of a flange (and web) with compression
$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 437.80$ kN

2.3. shear resistance

shear resistance of column web $V_{wp,Rd}/\beta_j = 455.2$ kN

2.4. verifications

2.4.1. verification of the connection capacity with partial internal forces and moments

shear force in column web:
$V_{c,w,Ed} = M_{d,w}/z - (V_{c1}-V_{c2})/2 = 292.99$ kN, $M_{d,w} = 76.1$ kNm, $z = 259.8$ mm

Gk 1: $F_{Rd} = V_{wp,Rd}/\beta_j = 455.2$ kN, $F_{Ed} = |V_{c,w,Ed}| = 292.99$ kN
$F_{Ed} = 293.0$ kN $< F_{Rd} = 455.2$ kN $\Rightarrow U = 0.644 < 1$ ok
Gk 2: $F_{Rd} = F_{c,w,Rd} = 333.4$ kN, $F_{Ed} = N_{b,c} = 288.68$ kN
$F_{Ed} = 288.7$ kN $< F_{Rd} = 333.4$ kN $\Rightarrow U = 0.866 < 1$ ok
Gk 4: $F_{Rd} = F_{fc,Rd} = 323.6$ kN, $F_{Ed} = N_{b,t} = 288.68$ kN
$F_{Ed} = 288.7$ kN $< F_{Rd} = 323.6$ kN $\Rightarrow U = 0.892 < 1$ ok
Gk 3: $F_{Rd} = F_{t,wc,Rd} = 397.0$ kN, $F_{Ed} = N_{b,t} = 288.68$ kN
$F_{Ed} = 288.7$ kN $< F_{Rd} = 397.0$ kN $\Rightarrow U = 0.727 < 1$ ok
Gk 7: flange: $F_{Rd} = F_{c,f,Rd} = 437.8$ kN, $F_{Ed} = N_{b,c} = 288.68$ kN
$F_{Ed} = 288.7$ kN $< F_{Rd} = 437.8$ kN $\Rightarrow U = 0.659 < 1$ ok

utilization partial internal forces and moments $U_{Gk} = 0.892 < 1$ ok

2.4.2. verification result

maximum utilization: max $U = 0.892 < 1$ ok

3. final result

maximum utilization: max $U = 0.892 < 1$ ok

verification succeeded