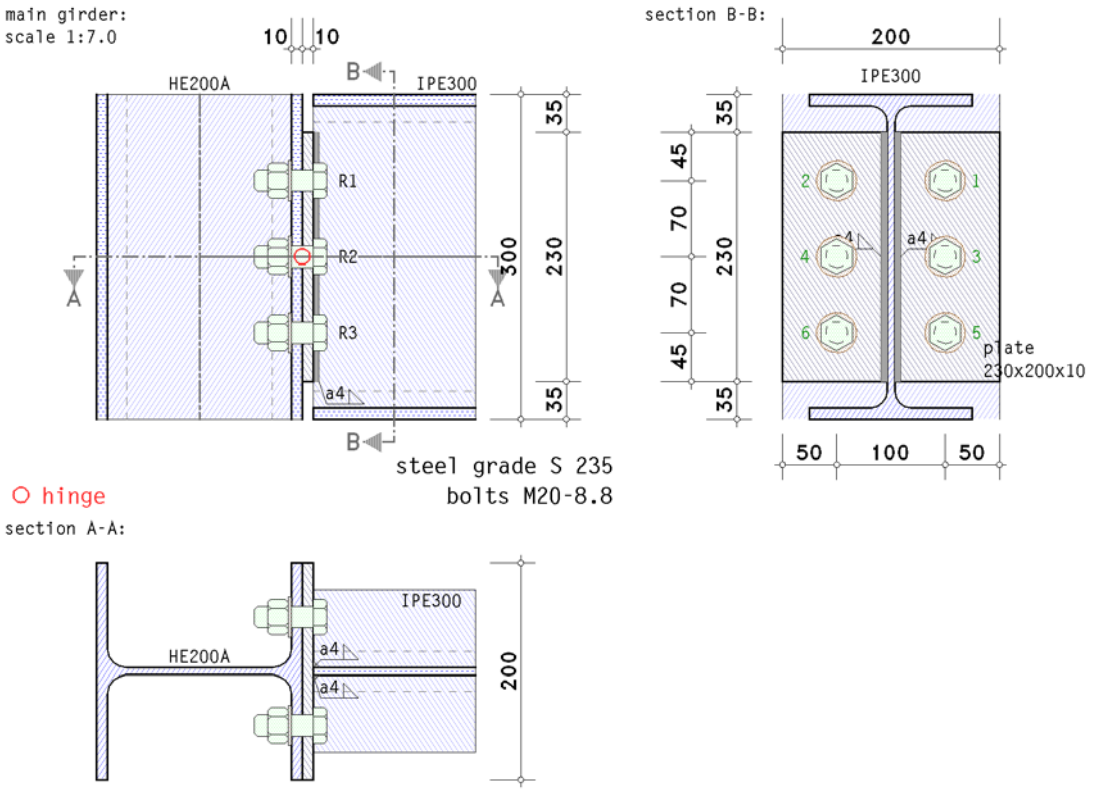
POS. 1: ECCS END-PLATE

4H-EC3GT version: 6/2015-1c

Simple Joint of Beams

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



beam connection with end-plate, connected to the flange of the main girder

steel grade

steel grade S 235

bolts

bolt: bolt class 8.8, bolt size M20

thread in shear plane

geometry

main girder

section HE200A

supported beam

section IPE300

joint configuration

end-plate: thickness t_p = 10.0 mm, length l_p = 230.0 mm, width b_p = 200.0 mm

plate lengths: h_o = 0.0 mm, $ü_o$ = 35.0 mm (h_u = 0.0 mm, $ü_u$ = 35.0 mm)

distances between bolts at main girder: $e_{z,0}$ = 45.0 mm, $p_{z,1-2}$ = 70.0 mm, $p_{z,2-3}$ = 70.0 mm, $e_{x,0}$ = 50.0 mm

thickness weld: a_w = 4.0 mm

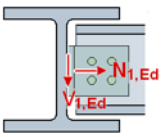
design resistance

design according to ECCS, 2009

verification of welds with the simplified method

internal forces and moments

Lk 1: $N_{1,Ed}$ = 200.00 kN $V_{1,Ed}$ = 200.00 kN



partial safety factors for material

resistance of cross sections γ_{M0} = 1.00

resistance of bolts, welds, plates in bearing γ_{M2} = 1.25

resistance with tension loads γ_{Mu} = 1.10

distance of bolt rows at main girder (right)

edge dist.:	$e_2 = 50.0$ mm $> 1.2 \cdot d_0 = 26.4$ mm,	$e_2 = 50.0$ mm $< 4 \cdot t_{min} + 40$ mm $= 80.0$ mm
pitch:	$p_2 = 100.0$ mm $> 2.4 \cdot d_0 = 52.8$ mm,	$p_2 = 100.0$ mm $< \min(14 \cdot t_{min}, 200$ mm$) = 140.0$ mm
edge dist.:	$e_1 = 45.0$ mm $> 1.2 \cdot d_0 = 26.4$ mm,	$e_1 = 45.0$ mm $< 4 \cdot t_1 + 40$ mm $= 80.0$ mm
pitch:	$p_1 = 70.0$ mm $> 2.2 \cdot d_0 = 48.4$ mm,	$p_1 = 70.0$ mm $< \min(14 \cdot t_{min}, 200$ mm$) = 140.0$ mm
pitch:	$p_1 = 70.0$ mm $> 2.2 \cdot d_0 = 48.4$ mm,	$p_1 = 70.0$ mm $< \min(14 \cdot t_{min}, 200$ mm$) = 140.0$ mm
edge dist.:	$e_1 = 45.0$ mm $> 1.2 \cdot d_0 = 26.4$ mm,	$e_1 = 45.0$ mm $< 4 \cdot t_1 + 40$ mm $= 80.0$ mm

assumption: hinge is in the periphery to the flange of the main girder

design resistance of the connection

requirements to ensure the safety of the approach
$l_p = 230.0$ mm $< d_w = 248.6$ mm **ok.**
sufficient rotation capacity has to ensure $\Rightarrow \Phi_{vorh} = t_p/(\ddot{u}_u - e_u) = 0.286 \geq \Phi_{erf}$!!
$d/t_p = 2.000 > 2.8 \cdot (f_{yp}/f_{ub})^{1/2} = 1.518$ **ok.**
$d/t_f = 2.000 > 2.8 \cdot (f_y/f_{ub})^{1/2} = 1.518$ **ok.**
$a_w = 4.0$ mm $> \beta_w/2^{1/2} \cdot f_y/f_u \cdot \gamma_{M2}/\gamma_{M0} \cdot t_w = 3.28$ mm **ok.**

shear force resistance
bolts in shear:
design shear resistance per shear plane: $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2} = 94.08$ kN, $\alpha_v = 0.60$
design shear resistance total: $V_{Rd,1} = 0.8 \cdot 6 \cdot F_{v,Rd} = 451.6$ kN

end-plate with bearing resistance:
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 98.18$ kN, $k_1 = 2.50$, $\alpha_b = 0.68$
design bearing resistance total: $V_{Rd,2} = 6 \cdot F_{b,Rd} = 589.1$ kN

beam flange with bearing resistance:
bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 116.73$ kN, $k_1 = 2.50$, $\alpha_b = 0.81$
design bearing resistance total: $V_{Rd,3} = 6 \cdot F_{b,Rd} = 700.4$ kN

end-plate with shear (gross section):
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0}) = 312.06$ kN
shear resistance total: $V_{Rd,4} = 2 \cdot V_{Rd}/1.27 = 491.4$ kN

end-plate with shear (net section):
shear resistance $V_{Rd} = (A_{v,net} \cdot f_u) / (3^{1/2} \cdot \gamma_{M2}) = 272.69$ kN
shear resistance total: $V_{Rd,5} = 2 \cdot V_{Rd} = 545.4$ kN

end-plate in tension and shear (shear block):
$l_p = 230.0$ mm $> 1.36 \cdot p_{22} = 136.0$ mm and $n_z = 3 > 1$:
shear resistance $V_{eff,Rd} = (0.5 \cdot A_{nt} \cdot f_u) / \gamma_{M2} + (A_{nv} \cdot f_y / 3^{1/3}) / \gamma_{M0} = 288.70$ kN
shear resistance total: $V_{Rd,6} = 2 \cdot V_{eff,Rd} = 577.4$ kN

end-plate in bending and shear:
$l_p = 230.0$ mm $> 1.36 \cdot p_{22} = 136.0$ mm:
shear resistance total: $V_{Rd,7} = \infty$

beam web with shear (next to the weld):
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0}) = 221.56$ kN
shear resistance total: $V_{Rd,8} = 221.6$ kN

shear resistance: min $V_{Rd,p} = V_{Rd,8} = 221.6$ kN

tension force resistance
bolts in tension:
tension resistance $F_{t,Rd} = 178.18$ kN
tension resistance total: $N_{Rd,1} = 6 \cdot F_{t,Rd} = 1069.1$ kN

end-plate in bending:
design resistance of end-plate in bending (projection)
$F_{t,ep,Rd,1} = 74.7$ kN, $l_{eff,1} = 80.0$ mm
tension resistance total: $N_{Rd,2} = 3 \cdot F_{t,ep,Rd,1} = 224.1$ kN

beam flange in bending:
design resistance of column flange in bending (per bolt-row)
$F_{fc,Rd,1} = 241.1$ kN, $l_{eff,1} = 191.9$ mm

$F_{fc,Rd,2}$ = 241.1 kN, $l_{eff,1}$ = 191.9 mm
$F_{fc,Rd,3}$ = 241.1 kN, $l_{eff,1}$ = 191.9 mm
design resistance of column flange in bending (group of bolts, 3 rows)
$F_{fc,Rd}$ = 426.3 kN, $\Sigma l_{eff,1}$ = 331.9 mm
tension resistance total: $N_{Rd,3}$ = 426.3 kN

beam web in tension:
design resistance of a beam web in tension
$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{u,wb} / \gamma_{M2}$ = 185.9 kN
tension resistance total: $N_{Rd,4} = 3 \cdot F_{t,wb,Rd}$ = 557.7 kN

tension resistance: min $N_{Rd,p} = N_{Rd,2}$ = 224.1 kN

Lk 1:

design values

transformation of member forces to the reference point (intersection point of beam axis')

$N_{1,Ed} = N_{j1,Ed}$ = 200.00 kN
$M_{1,Ed} = V_{j1,Ed} \cdot e_1$ = -19.00 kNm, e_1 = -95.0 mm
$V_{1,Ed} = V_{j1,Ed}$ = 200.00 kN

verification of the connection

V_{Ed} = 200.0 kN: V_{Ed} / min V_{Rd} = 0.903 < 1 **ok.**
N_{Ed} = 200.0 kN: N_{Ed} / min N_{Rd} = 0.892 < 1 **ok.**

weld verification

$F_{w,Ed}$ = 4.49 kN/cm < $F_{w,Rd}$ = 8.31 kN/cm $\Rightarrow$ U_w = 0.540 < 1 **ok.**

verification result

maximum utilization: max U = 0.903 < 1 **ok.**

Final Result

maximum utilization: design resistance max U = 0.903 < 1 **ok.**

verification succeeded