POS. 9: 2 BOLTS (BSP. KOMP. 7 LK)

standardized IM-joint

moment resistant joints IM acc. to EC 3-1-8 (12.10), NA: Deutschland

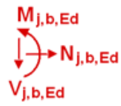
dimensions of beam, bolts, end-plate and welds, material
and arrangement of bolts are taken of the following literature:
'Typisierte Anschlüsse im Stahlhochbau nach DIN EN 1993-1-8, Ergänzungsband 2018,
Stahlbau Verlags- und Service GmbH, Ausgabe 2018'
the current number and associated parameters are recorded.
the column has no reference to the literature, web stiffeners are continuously fixed.
MN-interaction follows Cerfontaine (in Jaspart/Weynand: Design of Joints in Steel Structures).

beam-column connection, steel grade S235, bolt class of bolts 10.9

10106: beam section IPE240, bolt size M16, connection with 2 bolts per row
end-plate: t_p = 15 mm, b_p = 130 mm, h_p = 325 mm, e_1 = 40 mm, $p_{1,1}$ = 80 mm, $p_{1,2}$ = 150 mm
u_1 = 75 mm, w = 80 mm
fillet welds: a_f = 5 mm, a_w = 3 mm

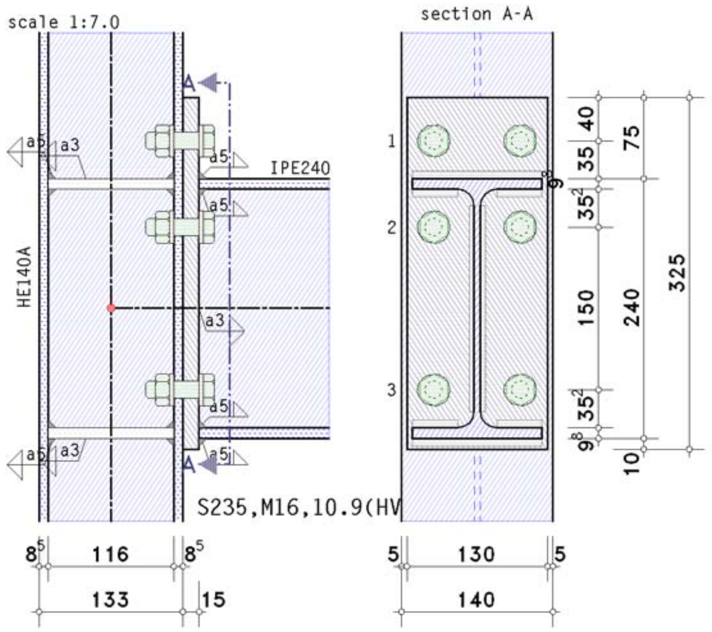
column: section HE140A
horizontal web stiffeners

internal forces and moments in the intersection point of system axes:
$M_{j,b1,Ed}$,$V_{j,b1,Ed}$: internal forces and moments by sign definition of statics

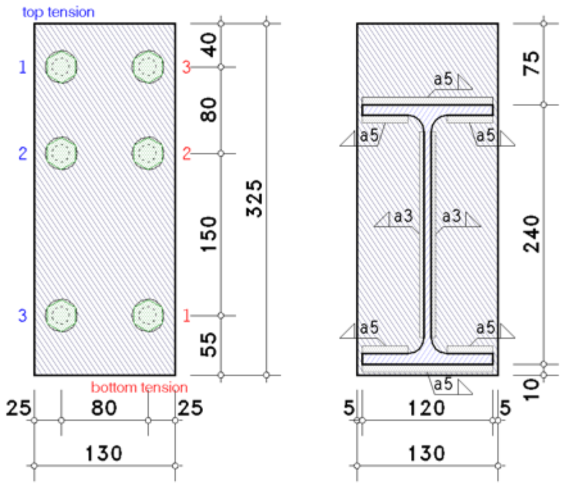


Lk	$M_{j,b1,Ed}$	$V_{j,b1,Ed}$	Lk	$M_{j,b1,Ed}$	$V_{j,b1,Ed}$
--	kNm	kN	--	kNm	kN
1	-2.81	-15.82	5	-4.88	-23.13
2	25.58	-51.86	6	7.26	21.11
3	5.10	14.07	7	-5.15	-22.54
4	-3.17	-15.04			

Rigid beam connection



details



Component method

notes
connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.
no verification for cross-sections.
the welds are not regarded by calculation the T-stub resistance.
simplified calculation of shear force resistance takes all bolt-rows into account.

Final Result

maximum utilization [Lk 2]:	max U = 1.028 > 1 fault !!
minimum rotational stiffness [Lk 2]:	min S_j = 2.3 MNm/rad, $S_{j,ini}$ = 6.4 MNm/rad
maximum rotation [Lk 2]:	max $\varphi_{j,Ed}$ = 0.616°

resistance not ensured !!

Decisive load case combination

resistance of cross-section

plastic cross-sectional check for N = 21.30 kN, M_y = -21.35 kNm, V_z = 51.86 kN
valid normal/shear stress: zul σ_{Rd} = 23.50 kN/cm², zul τ_{Rd} = 13.57 kN/cm²
top flange: resistance forces $N_{max,O}$ = 276.36 kN, $N_{min,O}$ = -276.36 kN
bottom flange: resistance forces $N_{max,U}$ = 276.36 kN, $N_{min,U}$ = -276.36 kN
web: shear force V_S = 51.86 kN, shear stress τ_S = 3.63 kN/cm² $\Rightarrow$ $U_{\tau,S}$ = 0.268
resistance forces $N_{max,S}$ = 323.15 kN, $N_{min,S}$ = -323.15 kN
main bending: axial force N = 21.30 kN, resistance forces N_{max} = 875.87 kN, N_{min} = -875.87 kN $\Rightarrow$ U_N = 0.024
moment M_y = -21.35 kNm, resistance moments $M_{y,max}$ = 82.13 kNm, $M_{y,min}$ = -82.13 kNm $\Rightarrow$ U_{My} = 0.260
total (possibly due to load increase): max U = 0.302 < 1 ok
utilizations: resistance U_σ = 0.302 < 1 ok, c/t-ratio $U_{c/t}$ = 0.154 < 1 ok

connection capacity

moment resistance

distance of tension-bolt-rows from centre of compression: h_1 = 190.1 mm, h_2 = 40.1 mm

resistance per bolt-row (MNV-interaction)
row 1: $F_{tr,Rd}$ = 126.0 kN
row 2: $F_{tr,Rd}$ = 29.7 kN
$\Sigma F_{tr,Rd}$ = 155.7 kN

resistance of flanges (MNV-interaction)
$F_{c,Rd}$ = 133.9 kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r)$ = 25.1 kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}$ = 155.7 kN

compression resistance
$N_{j,c,Rd} = F_{c,Rd} = 133.9$ kN

shear/bearing resistance

$V_{j,Rd} = 53.1$ kN (MNV-interaction)

shear resistance

shear resistance of end plate
plate: $V_{ep,Rd} = 387.49$ kN
resistance of a weld (req.1): $f_{1w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0$ N/mm²
welds: $F_{w,Rd} = 237.44$ kN
shear resistance of end plate: $V_{ep,Rd} = F_{w,Rd} = 237.44$ kN

shear resistance of column web
$V_{wp,Rd}/\beta = 133.9$ kN

plastic shear resistance
$V_{pl,Rd} = 0.5 \cdot A_v \cdot (f_y/3^{1/2}) / \gamma_{M0} = 129.9$ kN (requirement, s. 'Typisierte Anschlüsse')

total

$M_{j,Rd} = 25.1$ kNm $N_{j,t,Rd} = 155.7$ kN $N_{j,c,Rd} = 133.9$ kN $V_{j,Rd} = 53.1$ kN $V_{wp,Rd}/\beta = 133.9$ kN $V_{pl,Rd} = 129.9$ kN
$V_{ep,Rd} = 237.4$ kN

verifications

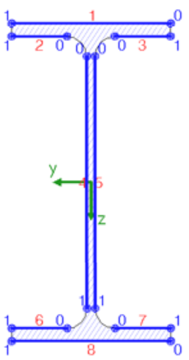
verification of the connection capacity by means of the component method

$U_{MNV} = 0.977 < 1$ ok
$V_{wp,Ed}/(V_{wp,Rd}/\beta) = 1.028 > 1$ fault !!
$V_{Ed}/V_{pl,Rd} = 0.399 < 1$ ok
$V_{Ed}/V_{ep,Rd} = 0.218 < 1$ ok

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

weld 4: NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 3.0$ mm $< a_{min} = t_{max}^{1/2} - 0.5 = 3.37$ mm !!
weld 5: NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 3.0$ mm $< a_{min} = t_{max}^{1/2} - 0.5 = 3.37$ mm !!
calculation section:



weld 1: $a_w = 5.0$ mm $l_w = 120.0$ mm
weld 2: $a_w = 5.0$ mm $l_w = 41.9$ mm
weld 3: siehe weld 2
weld 4: $a_w = 3.0$ mm $l_w = 190.4$ mm
weld 5: siehe weld 4
weld 6: $a_w = 5.0$ mm $l_w = 41.9$ mm
weld 7: siehe weld 6
weld 8: $a_w = 5.0$ mm $l_w = 120.0$ mm

design values refering to centroid of the section:
$N_{Ed} = 21.30$ kN, $M_{y,Ed} = -22.13$ kNm, $V_{z,Ed} = 51.86$ kN
cross-sectional properties refering to centroid of the line cross-section:
$\Sigma A_w = 31.80$ cm², $A_{w,z} = 11.42$ cm², $\Sigma l_w = 78.8$ cm
$I_{w,y} = 3090.79$ cm⁴, $I_{w,z} = 285.14$ cm⁴, $W_{w,t} = 23.47$ cm³, $\Delta z_w = 0.0$ mm
verifications in the edge points of the individual welds:
weld 1, pt. 0: $\sigma_{w,x} = 92.61$ N/mm² $\Rightarrow U_w = 0.364 < 1$ ok
weld 2, pt. 0: $\sigma_{w,x} = 85.59$ N/mm² $\Rightarrow U_w = 0.336 < 1$ ok
weld 4, pt. 0: $\sigma_{w,x} = 74.85$ N/mm² $\tau_{w,z} = 45.40$ N/mm² $\Rightarrow U_w = 0.366 < 1$ ok
pt. 1: $\sigma_{w,x} = -61.46$ N/mm² $\tau_{w,z} = 45.40$ N/mm² $\Rightarrow U_w = 0.326 < 1$ ok
weld 6, pt. 0: $\sigma_{w,x} = -72.20$ N/mm² $\Rightarrow U_w = 0.284 < 1$ ok
weld 8, pt. 0: $\sigma_{w,x} = -79.22$ N/mm² $\Rightarrow U_w = 0.311 < 1$ ok
Result:
weld 4, pt. 0: $\sigma_{w,x} = 74.85$ N/mm² $\tau_{w,z} = 45.40$ N/mm²
Max: $\sigma_{1,w,Ed} = 13.19$ kN/cm² $< f_{1w,d} = 36.00$ kN/cm²,
$\sigma_{2,w,Ed} = 5.29$ kN/cm² $< f_{2w,d} = 25.92$ kN/cm² $\Rightarrow U_w = 0.366 < 1$ ok

verification of web stiffeners

compression stiffener

$F_{c,Ed}$ = 128.61 kN

forces per rib

$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 50.8$ kN, $H = F \cdot e_F/e_H = 16.5$ kN

assumption: stiffeners do not buckle: $c/t = 5.8 \cdot \varepsilon \leq 33 \cdot \varepsilon \Rightarrow$ section class $1 \leq 2$ ok

cross-section at flange

compression resistance $N_{c,Rd} = (A \cdot f_y) / \gamma_{M0} = 90.39$ kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 58.2$ kN

$F_{Ed} = 58.2$ kN $<$ $F_{Rd} = 90.4$ kN $\Rightarrow$ U = 0.644 $<$ 1 ok

cross-section at web

shear resistance V_{Rd} = 154.24 kN

design value: $F_{Ed} = F = 50.8$ kN

$F_{Ed} = 50.8$ kN $<$ $F_{Rd} = 154.2$ kN $\Rightarrow$ U = 0.329 $<$ 1 ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 6.47$ kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 2.10$ kN/cm, $b_1 = 39.2$ mm

$\sigma_{1,w,Ed} = 14.83$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm² $\Rightarrow$ U = 0.412 $<$ 1 ok

$\sigma_{2,w,Ed} = 12.93$ kN/cm² $<$ $f_{2w,d} = 25.92$ kN/cm² $\Rightarrow$ U = 0.499 $<$ 1 ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 3.17$ kN/cm, $l_1 = 80.0$ mm

$\sigma_{1,w,Ed} = 18.31$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm² $\Rightarrow$ U = 0.509 $<$ 1 ok

stiffener in tension

$F_{t,Ed}$ = 149.90 kN

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 59.2$ kN, $H = F \cdot e_F/e_H = 19.2$ kN

cross-section at flange

tension resistance $N_{t,Rd}$ = 90.39 kN

design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 67.9$ kN

$F_{Ed} = 67.9$ kN $<$ $F_{Rd} = 90.4$ kN $\Rightarrow$ U = 0.751 $<$ 1 ok

cross-section at web

shear resistance V_{Rd} = 154.24 kN

design value: $F_{Ed} = F = 59.2$ kN

$F_{Ed} = 59.2$ kN $<$ $F_{Rd} = 154.2$ kN $\Rightarrow$ U = 0.384 $<$ 1 ok

flange welds

design values: $F_{Ed}(\sigma_s) = F / (2 \cdot b_1) = 7.54$ kN/cm, $F_{Ed}(\tau_p) = H / (2 \cdot b_1) = 2.44$ kN/cm, $b_1 = 39.2$ mm

$\sigma_{1,w,Ed} = 17.29$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm² $\Rightarrow$ U = 0.480 $<$ 1 ok

$\sigma_{2,w,Ed} = 15.07$ kN/cm² $<$ $f_{2w,d} = 25.92$ kN/cm² $\Rightarrow$ U = 0.581 $<$ 1 ok

web welds

design value: $F_{Ed}(\tau_p) = F / (2 \cdot l_1) = 3.70$ kN/cm, $l_1 = 80.0$ mm

$\sigma_{1,w,Ed} = 21.35$ kN/cm² $<$ $f_{1w,d} = 36.00$ kN/cm² $\Rightarrow$ U = 0.593 $<$ 1 ok

verification result

maximum utilization: max U = 1.028 $>$ 1 fault !!

failure at verification shear in column web panel: U = 1.028

rotational stiffness

stiffness coefficients

$k_1 = 0.38 \cdot A_{vc} / (\beta \cdot z) = 2.35$ mm

$k_2 = \infty$ (stiffened)

equivalent stiffness coefficient for 2 tension-bolt-rows:

1: $k_3 = 6.63$ mm, $k_4 = 4.14$ mm, $k_5 = 13.70$ mm, $k_{10} = 5.84$ mm $\Rightarrow$ $k_{eff,1} = 1 / \Sigma(1/k_{i,1}) = 1.572$ mm

2: $k_3 = 6.63$ mm, $k_4 = 4.14$ mm, $k_5 = 13.70$ mm, $k_{10} = 5.84$ mm $\Rightarrow$ $k_{eff,2} = 1 / \Sigma(1/k_{i,2}) = 1.572$ mm

$k_{eq} = \Sigma(k_{eff,r} \cdot h_r) / z_{eq} = 2.207$ mm, $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r) = 164.0$ mm

rotational stiffness

initial rotational stiffness: $S_{j,ini} = (E \cdot z^2) / \Sigma(1/k_i) = 6420.5$ kNm/rad, $z = z_{eq} = 164.0$ mm, $\Sigma(1/k_i) = 0.879$ mm⁻¹

$N_{b,Ed} = N_d = 21.30$ kN

$|N_{b,Ed}| = 21.30$ kN $<$ $5\% \cdot N_{pl,Rd} = 45.96$ kN ok

$|M_{j,Ed}| = 24.58$ kNm $>$ $2/3\ M_{j,Rd} = 16.8$ kNm $\Rightarrow$ $\mu = ((1.5 \cdot M_{j,Ed}) / M_{j,Rd})^{\Psi} = 2.809$, $\Psi = 2.7$

rotational stiffness: $S_{j,Rd} = S_{j,ini} / \mu = 2285.7$ kNm/rad

rotation: $\varphi_{j,Ed} = M_{j,Ed} / S_{j,Rd} = 0.616°$