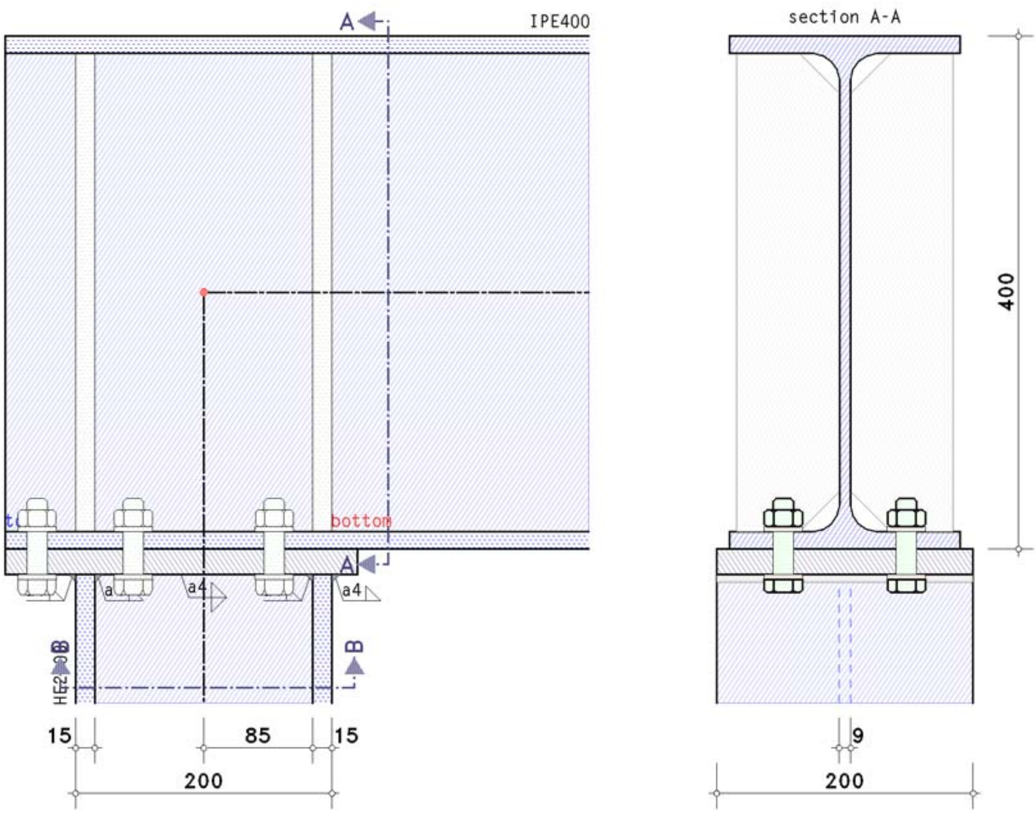
POS. 5: KAHLMEYER 7.3.1

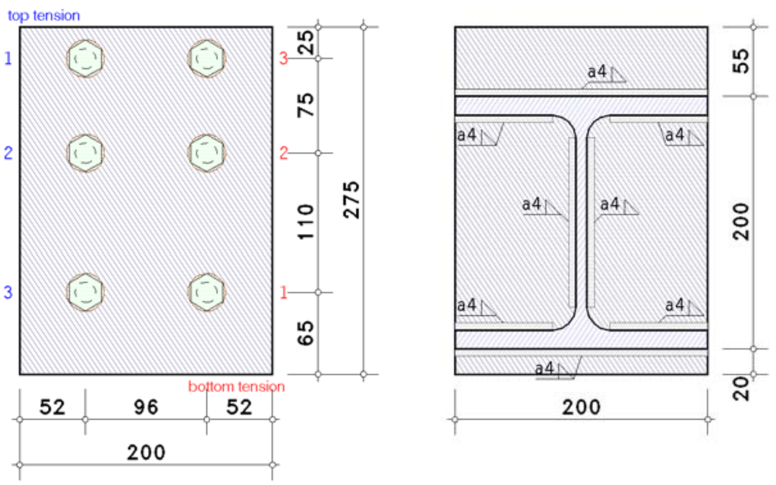
4H-EC3RE version: 5/2014-1x

frame corner

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



details (section B-B)



steel grade

steel grade S 235

column parameters

section HE200B

beam parameters

section IPE400

section reinforcement by web stiffeners:

thickness t_{st} = 15.0 mm, width b_{st} = 80.0 mm

welds $a_{st,f}$ = 0.0 mm, $a_{st,w}$ = 0.0 mm

bolts

bolt: bolt class 10.9,

large width across flats

dimensions:

shaft diameter d = 16.0 mm, clearance Δd = 1.0 mm $\Rightarrow$ hole diameter d_0 = 17.0 mm
gross cross-section area A = 2.011 cm^2
tensile stress area A_s = 1.570 cm^2
diameter of the bolt head (across flats dimension) d_s = 27.0 mm
diameter of the bolt head (across points dimension) d_e = 29.56 mm
thickness of the bolt head t_k = 10.0 mm
thickness of nut t_m = 13.0 mm
diameter of the plate under the bolt or the nut d_p = 30.0 mm
thickness of the plate under the bolt or the nut t_p = 4.0 mm
shear plane passes through the unthreaded portion of the bolt

verification parameters

bolted end-plate joint (horizontal):
thickness t_p = 20.0 mm, length l_p = 275.0 mm, width b_p = 200.0 mm
projections $h_{p,o}$ = 55.0 mm (left), $h_{p,u}$ = 20.0 mm (right)
bolts at the connection point:
3 bolt-row(s) with 2 bolts each
of these 2 bolt-rows top (M^+) in tension (rows 1-2)
and 3 bolt-rows for shear transfer at tension left (top) (rows 1-3)
of these 1 bolt-row bottom (M^-) in tension (row 3)
and 3 bolt-rows for shear transfer at tension right (bottom) (rows 1-3)
centre distance of the bolts to the lateral edge of the end-plate e_2 = 52.0 mm
centre distance of the first bolt row to the left (upper) edge of end plate (end row) e_o = 25.0 mm
centre distance of the last bolt row to the right (lower) edge of end plate (end row) e_u = 65.0 mm
centre distance of the first bolt-row to the free edge of the column (end row) e_1' = 25.0 mm
centre distance of the bolt-rows from each other p_{1-2} = 75.0 mm, p_{2-3} = 110.0 mm
welds at the connection point:
column flange left (top): fillet weld, weld thickness a = 4.0 mm
column web: fillet weld, weld thickness a = 4.0 mm
column flange right (bottom): fillet weld, weld thickness a = 4.0 mm

internal forces and moments in the intersection point of system axes (sign convention of statics)

Lk 1: $N_{j,b1,Ed}$ = -3.43 kN $M_{j,b1,Ed}$ = -47.56 kNm $V_{j,b1,Ed}$ = 87.57 kN

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

Component method

notes
calculation of a frame corner with horizontal connection (variant 2) is made for the rotated model, i.e. beam, column and the internal forces and moments are permuted. All notations and messages apply to this calculation model.
high strength bolts have to be controlled prestressed, bolt category D (tension), A (shear).
no verification for cross sections within the connection area.

calculation model: vertical column-beam-connection

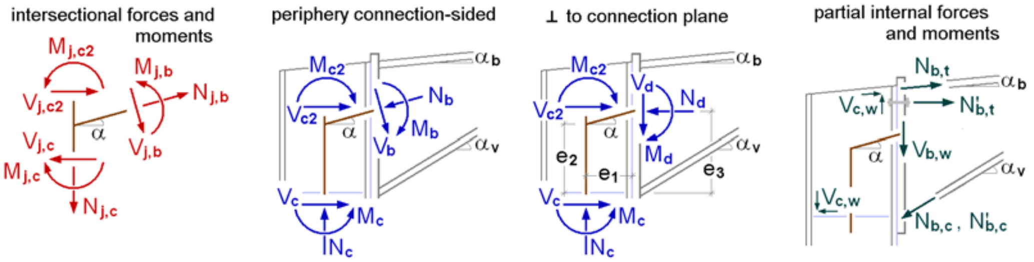
distances between bolt-rows at end-plate

edge dist.:	e_2 = 52.0 mm > $1.2 \cdot d_0$ = 20.4 mm,	e_2 = 52.0 mm < $4 \cdot t_{min}$ + 40 mm = 94.0 mm
pitch:	p_2 = 96.0 mm > $2.4 \cdot d_0$ = 40.8 mm,	p_2 = 96.0 mm < min($14 \cdot t_{min}$, 200 mm) = 189.0 mm
edge dist.:	e_1 = 25.0 mm > $1.2 \cdot d_0$ = 20.4 mm,	e_1 = 25.0 mm < $4 \cdot t_1$ + 40 mm = 120.0 mm
edge dist.:	e_1 = 25.0 mm > $1.2 \cdot d_0$ = 20.4 mm,	e_1 = 25.0 mm < $4 \cdot t_2$ + 40 mm = 94.0 mm
pitch:	p_1 = 75.0 mm > $2.2 \cdot d_0$ = 37.4 mm,	p_1 = 75.0 mm < min($14 \cdot t_{min}$, 200 mm) = 189.0 mm
pitch:	p_1 = 110.0 mm > $2.2 \cdot d_0$ = 37.4 mm,	p_1 = 110.0 mm < min($14 \cdot t_{min}$, 200 mm) = 189.0 mm
edge dist.:	e_1 = 65.0 mm > $1.2 \cdot d_0$ = 20.4 mm,	e_1 = 65.0 mm < $4 \cdot t_1$ + 40 mm = 120.0 mm

horizontal distance of bolts from column edge

edge dist.:	e_2 = 42.0 mm > $1.2 \cdot d_0$ = 20.4 mm,	e_2 = 42.0 mm < $4 \cdot t_{min}$ + 40 mm = 94.0 mm

design values



internal forces and moments at beam

$N_{j,b,Ed}$ = -87.57 kN
$M_{j,b,Ed}$ = -47.56 kNm
$V_{j,b,Ed}$ = 3.43 kN

internal forces and moments in the periphery

$N_{b,Ed} = -N_{j,b,Ed}$ = 87.57 kN
$M_{b,Ed} = -M_{j,b,Ed} - V_{j,b,Ed} \cdot e_1$ = 46.87 kNm, e_1 = 200.0 mm
$V_{b,Ed} = V_{j,b,Ed}$ = 3.43 kN

periphery column (bottom):

$N_{c,Ed} = N_{b,Ed} \cdot \sin(\alpha) + V_{b,Ed} \cdot \cos(\alpha)$ = 3.43 kN
$M_{c,Ed} = M_{b,Ed} - V_{c,Ed} \cdot e_3 + N_{c,Ed} \cdot e_1$ = 39.46 kNm, e_1 = 200.0 mm, e_3 = 92.5 mm
$V_{c,Ed} = N_{b,Ed} \cdot \cos(\alpha) - V_{b,Ed} \cdot \sin(\alpha)$ = 87.57 kN

internal forces and moments perpendicular to the connection plane

$N_d = N_{b,Ed}$ = 87.57 kN
$M_d = M_{b,Ed}$ = 46.87 kNm
$V_d = V_{b,Ed}$ = 3.43 kN

partial internal forces and moments

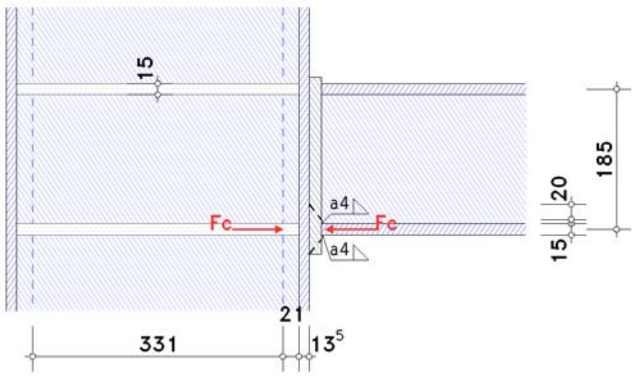
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d - V_d \cdot t_{ep}$ = 46.81 kN
$N_{b,t} = -N_d \cdot z_{bu}/z_b + M'_d/z_b$ = 209.22 kN, z_b = 185.0 mm, z_{bu} = 92.5 mm
$N_{b,c} = N_d \cdot z_{bo}/z_b + M'_d/z_b$ = 296.79 kN, z_b = 185.0 mm, z_{bo} = 92.5 mm

basic components

end-plate joint: decisive basic components: 2, 3, 4, 5, 7, 8, 10, 11, 12

basic component 2: column web in transverse compression

transformation parameter (table 5.4) β = 1.0
maximum longitudinal compressive stress in column web $\sigma_{com,Ed}$ = 28.64 N/mm²

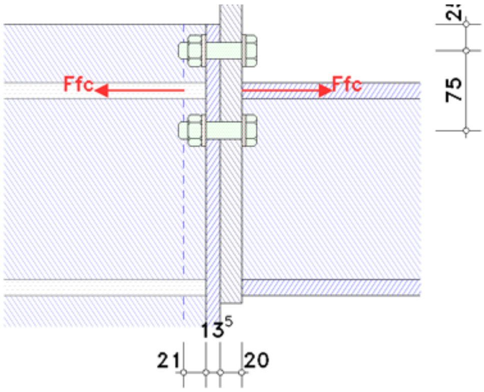


Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: $c/t = 5.3 \cdot \varepsilon \le 33 \cdot \varepsilon \Rightarrow$ section class $1 \le 2$ **ok.**
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 185.0 mm
web height between the flanges h_{wc} = 373.0 mm
moment of inertia of stiffeners I_{st} = 599.08 cm⁴
minimum moment of inertia for $a/h_{wc} = 0.50 < 2^{1/2}$: $I_{st,min}$ = 144.67 cm⁴ < I_{st} **ok.**
requirement concerning stiffeners to aviod lateral torsional buckling:
torsional moment of inertia of stiffeners I_T = 9.00 cm⁴
polar moment of inertia of stiffeners I_p = 66.25 cm⁴
$I_T / I_p \approx 0.136 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ **ok.**
design resistance of stiffened webs with transverse compression:
area of stiffeners incl. web A_{st} = 25.29 cm²
slenderness λ = 0.082

$\lambda \leq 0.2 \Rightarrow$ no deduction ($\chi = 1.0$)
design value of resistance of flexural buckling $F_{c,w,Rd} = 540.3$ kN

basic component 4: column flange in bending



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

equivalent T-stub flange (each bolt-row decisive):

here: number of bolt rows $n_b = 1$

row 1

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 120.8$ mm, $l_{eff,cp} = 134.5$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 120.8$ mm

design tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 1.29$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 226.08$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 192.33$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 168.34$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 226.08$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 168.34$ kN

row 2

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 169.0$ mm, $l_{eff,cp} = 169.0$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 175.8$ mm

design tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 1.81$ kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 1.88$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 226.08$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 269.10$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 187.82$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 226.08$ kN

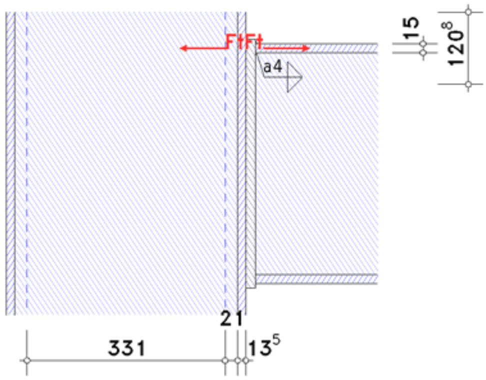
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 187.82$ kN

design resistance of column flange in bending (per bolt-row)

$F_{fc,Rd,1} = 168.3$ kN, $l_{eff,1} = 120.8$ mm

$F_{fc,Rd,2} = 187.8$ kN, $l_{eff,1} = 169.0$ mm basic component 3: column web in transverse tension

transformation parameter (table 5.4) $\beta = 1.0$



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

each bolt-row decisive:

row 1

effective width $b_{eff,t} = 120.8$ mm (l_{eff} from bc 4)

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.964$

design resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 235.2$ kN

row 2

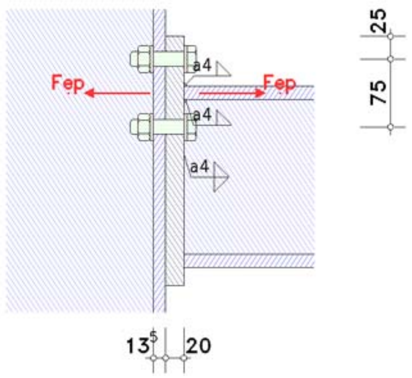
effective width $b_{eff,t} = 169.0$ mm (l_{eff} from bc 4)

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.932$

design resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 318.4$ kN

basic component 5: end-plate in bending



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

extended part of end-plate

in the extended part of the end-plate only one bolt-row is considered ($n_b = 1$).

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 100.0$ mm, $l_{eff,cp} = 160.1$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 100.0$ mm

design tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 2.35$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 226.08$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 369.00$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 205.09$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 226.08$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 205.09$ kN

design resistance of the weld: $F_{w,Rd} = 83.14$ kN per side

total loading capacity of the T-stub flange: $F_{T,Rd} = 2 \cdot F_{w,Rd} = 166.28$ kN

design resistance of end-plate in bending (projection)

$F_{t,ep,Rd,1} = 166.3$ kN, $l_{eff,1} = 100.0$ mm

part of end-plate between beam flanges

equivalent T-stub flange (each bolt-row decisive):

here: number of bolt rows $n_b = 1$

row 2

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 244.9$ mm, $l_{eff,cp} = 244.9$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 272.8$ mm design tension resistance of the T-stub flange:

in mode 1: $M_{pl,1,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.75$ kNm

in mode 2: $M_{pl,2,Rd} = (0.25 \cdot \Sigma l_{eff,2} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 6.41$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 226.08$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 590.62$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 275.62$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 226.08$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 226.08$ kN

design resistance of the weld: $F_{w,Rd} = 203.59$ kN per side

design resistances of end-plate in bending (per bolt-row):

$F_{ep,Rd,2} = 226.1$ kN, $l_{eff,1} = 244.9$ mm

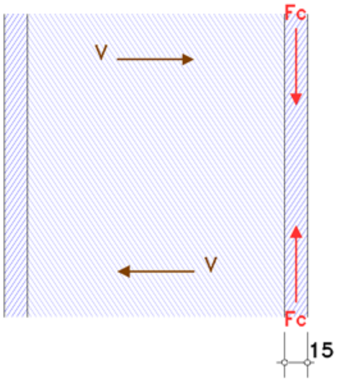
basic component 7: beam flange and web in compression

flange bottom: section class for $c/(\varepsilon \cdot t) = 5.17$: 1

web: section class for $\alpha = 0.60$ and $c/(\varepsilon \cdot t) = 14.89$: 1

section class of the beam in connection plane: 1

taking into account the moment-shear force-interaction $V_{Ed} = 3.4$ kN



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

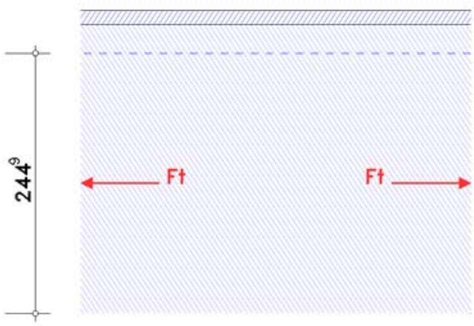
stress due to bending with shear force: $V_{Ed} = 3.4 \text{ kN} \leq 168.5 \text{ kN} = V_{pl,Rd}/2 \Rightarrow$ no effect

moment resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 151.10$ kNm

design resistance of a flange and web in compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 816.78$ kN

basic component 8: beam web in tension



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

each bolt-row decisive:

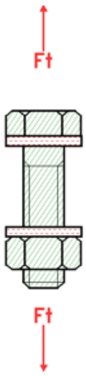
row 2

effective width $b_{eff,t,wb} = 244.9$ mm (l_{eff} from bc 5)

design resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 517.9$ kN

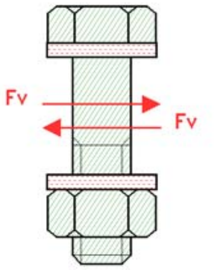
basic component 10: bolts in tension



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

bolt category D:
tension resistance of one bolt: $F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 113.04$ kN, $k_2 = 0.90$
p. sh. load capicity: $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 207.26$ kN, $t_p = 13.5$ mm
tension-/punching shear load capicity for 2 bolts: $\Sigma F_{t,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 226.08$ kN

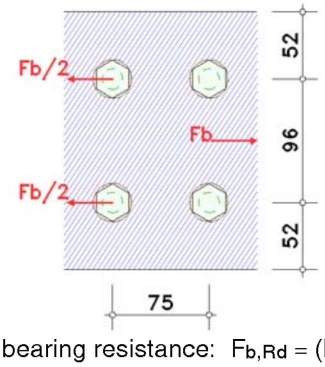
basic component 11: bolts in shear



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

bolt category A:
design shear resistance per shear plane: $F_{v,Rd} = \alpha_v \cdot f_{ub} \cdot A / \gamma_{M2} = 96.51$ kN, $\alpha_v = 0.60$
design shear resistance of 2 bolts: $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd} = 193.02$ kN

basic component 12: bolts in bearing



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

bearing resistance: $F_{b,Rd} = (k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t) / \gamma_{M2} = 155.52$ kN, $k_1 = 2.50$, $\alpha_b = 1.00$
design bearing resistance of 2 bolts per row: $\Sigma F_{b,Rd} = 2 \cdot F_{b,Rd} = 311.04$ kN

connection design capacity

moment resistance

distance between bolt-row(s) in tension and centre of compression:
$h_1 = 222.5$ mm, $h_2 = 147.5$ mm

design resistances acc. to 6.2.7.2(6) for bolt-rows considered individually
decisive basic components: 3, 4, 5, 8
row 1: $F_{tr,Rd} = 166.3$ kN
row 2: $F_{tr,Rd} = 187.8$ kN

deductions acc. to 6.2.7.2(7)
decisive basic components: 1, 2, 7

row 1: $\Sigma F_{tr,Rd}$ = 0.0 kN

Gk 2: $\Delta F_{tr,Rd} = F_{c,wc,Rd} - \Sigma F_{tr,Rd}$ = 540.3 kN $F_{tr,Rd}$ = 166.3 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 166.3 kN

Gk 7: $\Delta F_{tr,Rd} = F_{c,fb,Rd} - \Sigma F_{tr,Rd}$ = 816.8 kN $F_{tr,Rd}$ = 166.3 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 166.3 kN

row 2: $\Sigma F_{tr,Rd}$ = 166.3 kN (row 1)

Gk 2: $\Delta F_{tr,Rd} = F_{c,wc,Rd} - \Sigma F_{tr,Rd}$ = 374.0 kN $F_{tr,Rd}$ = 187.8 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 187.8 kN

Gk 7: $\Delta F_{tr,Rd} = F_{c,fb,Rd} - \Sigma F_{tr,Rd}$ = 650.5 kN $F_{tr,Rd}$ = 187.8 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 187.8 kN

check acc. to 6.2.7.2(9)

decisive basic component: 10

row 1: $F_{tx,Rd}$ = 166.3 kN, h_x = 222.5 mm $\Rightarrow$ $F_{tx,Rd} \leq 0.95 \cdot \Sigma F_{t,Rd}$ = 214.8 kN, no deduction

design resistance per bolt-row (finally)

row 1: $F_{tr,Rd}$ = 166.3 kN

row 2: $F_{tr,Rd}$ = 187.8 kN

potential failure by basic component 4, 5

moment resistance

$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r)$ = 64.7 kNm

tension resistance

$N_{j,Rd} = \Sigma F_{tr,Rd}$ = 354.1 kNm

compression resistance

$N_{j,c,Rd} = \min F_{c,Rd}$ = 540.3 kNm

shear/design bearing resistance

design resistance per bolt-row

decisive basic components: 11, 12

row 1: $F_{vr,Rd}$ = 193.0 kN

row 2: $F_{vr,Rd}$ = 193.0 kN

row 3: $F_{vr,Rd}$ = 193.0 kN

deductions depending on tension force (at 100% utilization of moment resistance)

decisive basic component: 10

$\Sigma F_{t,Rd}$ = 226.1 kN

row 1: $F_{vr,Rd} = f_v \cdot$ 193.0 kN = 91.6 kN with $f_v = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd})$ = 0.475

row 2: $F_{vr,Rd} = f_v \cdot$ 193.0 kN = 78.5 kN with $f_v = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd})$ = 0.407

row 3: $F_{vr,Rd} = f_v \cdot$ 193.0 kN = 193.0 kN with $f_v = 1 - F_{tr,Rd} / (1.4 \cdot \Sigma F_{t,Rd})$ = 1.000

design resistance per bolt-row (finally)

row 1: $F_{vr,Rd}$ = 91.6 kN

row 2: $F_{vr,Rd}$ = 78.5 kN

row 3: $F_{vr,Rd}$ = 193.0 kN

shear/design bearing resistance

$V_{j,Rd} = \Sigma F_{vr,Rd}$ = 363.1 kN

total

$N_{j,Rd}$ = 354.1 kN $N_{j,c,Rd}$ = 540.3 kN $M_{j,Rd}$ = 64.7 kNm $V_{j,Rd}$ = 363.1 kN

verifications

verification of the connection design capacity by means of the component method

axial force: $N_{b,Ed} = |N_d|$ = 87.57 kN $<$ $5\% \cdot N_{pl,Rd}$ = 91.75 kN $\Rightarrow$ moment resistance

regarding beam axis

internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu}$ = 38.77 kNm, z_{bu} = 92.5 mm

shear force: $V_{Ed} = |V_d|$ = 3.43 kN

$M_{Ed}/M_{j,Rd}$ = 0.599 $<$ 1 **ok.**

$V_{Ed}/V_{j,Rd}$ = 0.009 $<$ 1 **ok.**

shear/design bearing resistance at 59.9% utilization of moment resistance $V_{j,Rd}$ = 449.6 kN

$V_{Ed}/V_{j,Rd}$ = 0.008 $<$ 1 **ok.**

verification of welds at beam section

weld 1: beam flange in tension outer

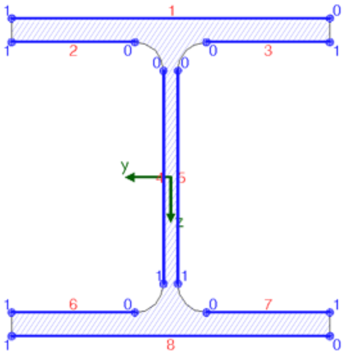
welds 2,3: beam flange in tension inner

welds 4,5: beam web double-sided

weld 8: beam flange in compression outer

welds 6,7: beam flange in compression inner

calculation section:



weld 1: a_w = 4.0 mm l_w = 200.0 mm
weld 2: a_w = 4.0 mm l_w = 77.5 mm
weld 3: siehe weld 2
weld 4: a_w = 4.0 mm l_w = 134.0 mm
weld 5: siehe weld 4
weld 6: a_w = 4.0 mm l_w = 77.5 mm
weld 7: siehe weld 6
weld 8: a_w = 4.0 mm l_w = 200.0 mm

design values:
N_{Ed} = -87.57 kN, $M_{y,Ed}$ = -46.87 kNm, $V_{z,Ed}$ = 3.43 kN
cross-sectional properties refering to centroid of the line cross section:
ΣA_w = 39.12 cm², $A_{w,z}$ = 10.72 cm², Σl_w = 97.8 cm
$I_{w,y}$ = 2656.69 cm⁴, $I_{w,z}$ = 1062.91 cm⁴, Δz_w = 0.0 mm
member forces distributed to the individual welds:
weld 1: N_w = 123.24 kN $M_{y,w}$ = -0.00 kNm
weld 2: N_w = 39.55 kN
weld 4: N_w = -12.00 kN $M_{y,w}$ = -1.42 kNm
weld 6: N_w = -53.43 kN
weld 8: N_w = -159.06 kN $M_{y,w}$ = -0.00 kNm
verifications in the edge points of the individual welds:
weld 1, pt. 0: $\sigma_{w,x}$ = 154.05 N/mm² ⇒ U_w = 0.741 < 1 ok.
weld 2, pt. 0: $\sigma_{w,x}$ = 127.59 N/mm² ⇒ U_w = 0.614 < 1 ok.
weld 4, pt. 0: $\sigma_{w,x}$ = 95.83 N/mm² $\tau_{w,z}$ = 3.20 N/mm² ⇒ U_w = 0.461 < 1 ok.
pt. 1: $\sigma_{w,x}$ = -140.60 N/mm² $\tau_{w,z}$ = 3.20 N/mm² ⇒ U_w = 0.677 < 1 ok.
weld 6, pt. 0: $\sigma_{w,x}$ = -172.36 N/mm² ⇒ U_w = 0.829 < 1 ok.
weld 8, pt. 0: $\sigma_{w,x}$ = -198.82 N/mm² ⇒ U_w = 0.957 < 1 ok.
Result:
weld 8, pt. 0: $\sigma_{w,x}$ = -198.82 N/mm²
$F_{w,Ed}$ = 7.95 kN/cm < $F_{w,Rd}$ = 8.31 kN/cm ⇒ U_w = 0.957 < 1 ok.

verification of the shear area

column web
requirements concerning stiffeners: verification of web stiffeners required !!
requirements concerning shear area: verification of buckling resistance required !!
internal forces and moments at shear area (sign definition of statics):
N_3 = -3.43 kN, M_3 = -39.46 kNm, V_3 = -87.57 kN, N_4 = -87.57 kN, M_4 = -46.90 kNm, V_4 = 3.43 kN
dimensions of the shear area: h_b = 373.0 mm, h_t = 373.0 mm, h_l = 177.5 mm, h_r = 177.5 mm
stresses within the shear area:
τ_b = 65.3 N/mm², τ_t = 65.3 N/mm², τ_l = 65.8 N/mm², τ_r = 65.8 N/mm²
verification of the shear area:
max τ_{Ed} = 65.8 N/mm² < τ_{Rd} = 135.7 N/mm² ⇒ U = 0.485 < 1 ok.
stresses in edge stiffeners:
σ_b = 117.5 N/mm², σ_t = 174.7 N/mm², σ_l = 41.3 N/mm², σ_r = 42.7 N/mm²
verification of the edge stiffeners:
max σ_{Ed} = 174.7 N/mm² < σ_{Rd} = 235.0 N/mm² ⇒ U = 0.743 < 1 ok.

verification result

maximum utilization: max U = 0.957 < 1 ok.

Final result

maximum utilization: max U = 0.957 < 1 ok.

verification succeeded