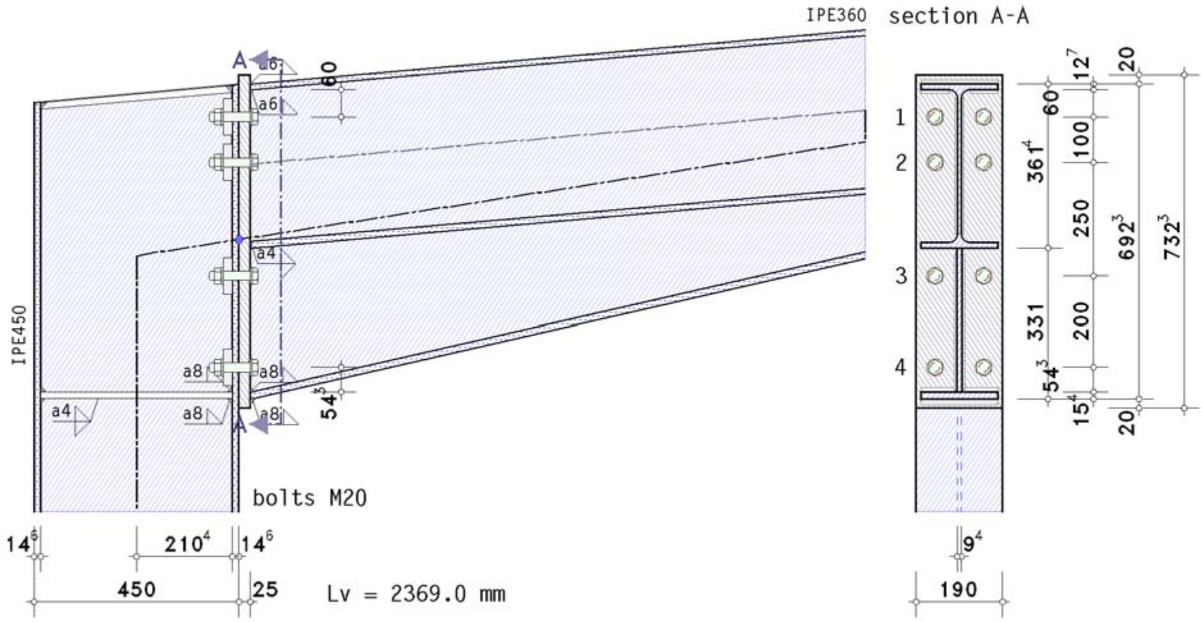
POS. 2: WAGENKNECHT 7.7.2

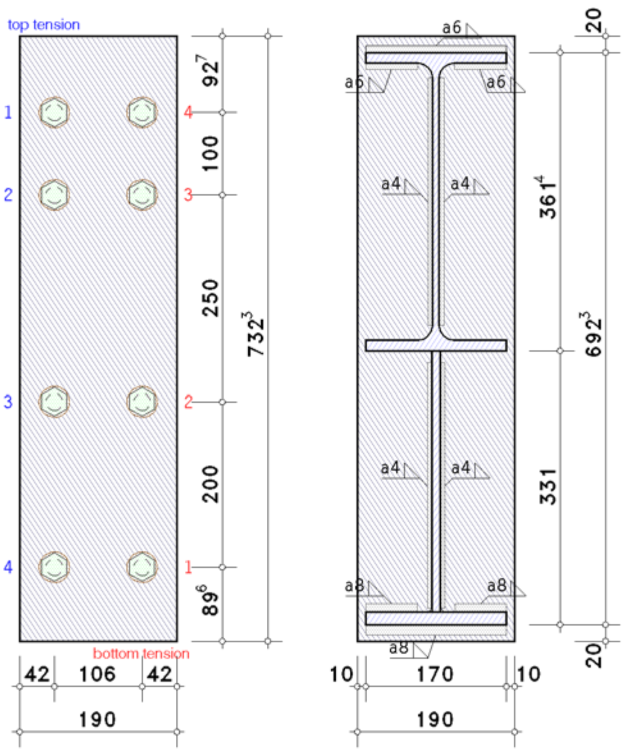
4H-EC3RE version: 5/2014-1x

frame corner

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



details



steel grade

steel grade S 235

column parameters

section IPE450

reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 674.8 mm):

thickness t_{st} = 15.0 mm

recess at stiffeners c_{st} = 25.0 mm

welds $a_{st,f}$ = 8.0 mm, $a_{st,w}$ = 4.0 mm

beam parameters

section IPE360

section angle of inclination about the horizontal axis α_b = 5.00° ⇒ section depth at the joint loc. $h_b = h/\cos(\alpha_b)$ = 361.4 mm

haunch angle of inclination about the horizontal axis α_v = 12.80° ⇒ haunch angle about the beam axis $\Delta\alpha_v$ = 7.80°
haunch length L_v = 2369.0 mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b))$ = 331.0 mm
web thickness $t_{w,v}$ = 10.0 mm, flange width, thickness $b_{f,v}$ = 170.0 mm, $t_{f,v}$ = 15.0 mm
total beam depth at the connection point $h_{ges} = h_b + h_v$ = 692.3 mm

bolts

bolt: bolt class 10.9, bolt size M20
large width across flats (high strength bolt), slip resistant connection (prestressed)
shear plane passes through the unthreaded portion of the bolt
class of friction surfaces A (slip factor μ = 0.50)
flange reinforcement: thickness t_{bp} = 20.0 mm, length l_{bp} = 80.0 mm, width b_{bp} = 80.0 mm

verification parameters

bolted end-plate joint:
thickness t_p = 25.0 mm, length l_p = 732.3 mm, width b_p = 190.0 mm
projections $h_{p,o}$ = 20.0 mm, $h_{p,u}$ = 20.0 mm
bolts at the connection point:
4 bolt-row(s) with 2 bolts each
of these 2 bolt-rows top (M^+) in tension (rows 1-2)
and 2 bolt-rows for shear transfer at tension top (rows 3-4)
of these 2 bolt-rows bottom (M^-) in tension (rows 3-4)
and 1 bolt-row for shear transfer at tension bottom (row 4)
centre distance of the bolts to the lateral edge of the end-plate e_2 = 42.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 92.7 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 89.6 mm
centre distance of the first bolt-row to the free edge of the column (end row) e_1' = 71.4 mm
centre distance of the bolt-rows from each other p_{1-2} = 100.0 mm, p_{2-3} = 250.0 mm, p_{3-4} = 200.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 6.0 mm, angle φ = 85°
beam web: fillet weld, weld thickness a = 4.0 mm
beam flange bottom: fillet weld, weld thickness a = 8.0 mm, angle φ = 103°

internal forces and moments at the joint periphery perpendicular to the connection plane

Lk 1: N_{d1} = 45.70 kN M_{d1} = 283.00 kNm V_{d1} = 87.00 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

high strength bolts have to be controlled prestressed, bolt category E (tension), C (shear).
In haunched beams the bottom flange of the rolled section is not considered. A fictive welded section is shaped from the top beam flange, the beam web and the haunch flange.
no verification of haunch connection to beam.
no verfication for welds within the connection.

distances between bolt-rows at end-plate

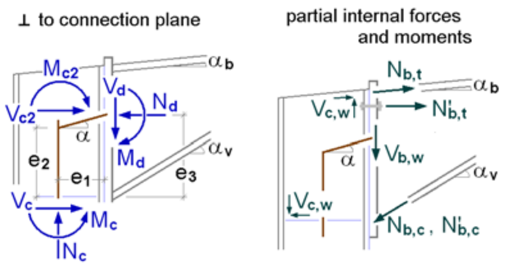
edge dist.:	e_2 = 42.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_2 = 42.0 mm < $4 \cdot t_{min}$ + 40 mm = 98.4 mm
pitch:	p_2 = 106.0 mm > $2.4 \cdot d_0$ = 52.8 mm,	p_2 = 106.0 mm < min($14 \cdot t_{min}$, 200 mm) = 200.0 mm
edge dist.:	e_1 = 92.7 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 92.7 mm < $4 \cdot t_1$ + 40 mm = 140.0 mm
edge dist.:	e_1 = 71.4 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 71.4 mm < $4 \cdot t_2$ + 40 mm = 98.4 mm
pitch:	p_1 = 100.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 100.0 mm < min($14 \cdot t_{min}$, 200 mm) = 200.0 mm
pitch:	p_1 = 250.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 250.0 mm > min($14 \cdot t_{min}$, 200 mm) = 200.0 mm !!
pitch:	p_1 = 200.0 mm > $2.2 \cdot d_0$ = 48.4 mm,	p_1 = 200.0 mm ≥ min($14 \cdot t_{min}$, 200 mm) = 200.0 mm
edge dist.:	e_1 = 89.6 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_1 = 89.6 mm < $4 \cdot t_1$ + 40 mm = 140.0 mm

horizontal distance of bolts from column edge

edge dist.:	e_2 = 42.0 mm > $1.2 \cdot d_0$ = 26.4 mm,	e_2 = 42.0 mm < $4 \cdot t_{min}$ + 40 mm = 98.4 mm

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

design values



angle of inclination: $\alpha_b = 5.0°$, $\alpha_v = 12.8°$ $\Rightarrow$ $\alpha = (\alpha_b+\alpha_v)/2 = 8.9°$

internal forces and moments perpendicular to the connection plane

N_d = 45.70 kN
M_d = 283.00 kNm
V_d = 87.00 kN
periphery column (bottom):
$N_{c,Ed}$ = V_d = 87.00 kN
$V_{c,Ed}$ = N_d = 45.70 kN
$M_{c,Ed}$ = $M_d - N_d \cdot e_3 + V_d \cdot e_1$ = 287.65 kNm, e_1 = 225.0 mm, e_3 = 326.6 mm

partial internal forces and moments

internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d \cdot t_{ep} \cdot \tan(\alpha) - V_d \cdot t_{ep}$ = 281.00 kN
$N_{b,t}$ = $(-N_d \cdot z_{bu}/z_b + M'_d/z_b) / \cos(\alpha_b)$ = 393.90 kN, z_b = 678.3 mm, z_{bu} = 324.9 mm
$N_{b,c}$ = $(N_d \cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_v)$ = 449.27 kN, z_b = 678.3 mm, z_{bo} = 353.4 mm

resistance of cross section

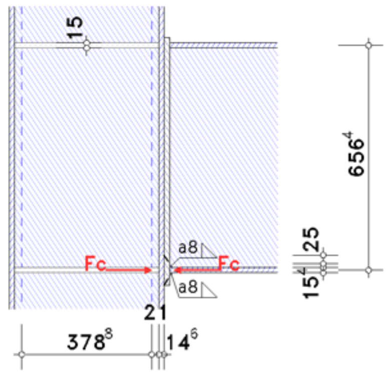
column
plastic cross-sectional check for M_{Ed} = -287.65 kNm, N_{Ed} = -87.00 kN, V_{Ed} = 45.70 kN
elastic stresses: max σ_x = 18.30 kN/cm², min σ_x = -20.06 kN/cm², max τ = 1.23 kN/cm², max σ_v = 20.07 kN/cm²
plastic design resistance moment: $M_{pl,N,Q}$ = 387.31 kNm
utilizations: design resistance U_σ = 0.747 < 1 ok., c/t-ratio $U_{c/t}$ = 0.323 < 1 ok.
beam
plastic cross-sectional check for M_{Ed} = -281.00 kNm, N_{Ed} = -45.70 kN, V_{Ed} = 87.00 kN
elastic stresses: max σ_x = 13.17 kN/cm², min σ_x = -13.01 kN/cm², max τ = 1.82 kN/cm², max σ_v = 13.18 kN/cm²
plastic design resistance moment: $M_{pl,N,Q}$ = 585.86 kNm
utilizations: design resistance U_σ = 0.486 < 1 ok., c/t-ratio $U_{c/t}$ = 0.483 < 1 ok.

basic components

end-plate joint: decisive basic components: 2, 3, 4, 5, 8, 10, 20

basic component 2: column web in transverse compression

transformation parameter (table 5.4) β = 1.0
maximum longitudinal compressive stress in column web $\sigma_{com,Ed}$ = 170.26 N/mm²



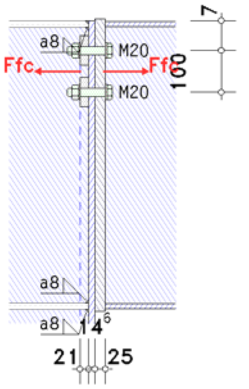
Only the essential sizes are sketched to scale.
The connection geometry is only hinted.

reinforcement of web with transverse stiffeners:
assumption: stiffeners do not buckle: $c/t = 6.0 \cdot \varepsilon \le 33 \cdot \varepsilon$ $\Rightarrow$ section class 1 ≤ 2 ok.
minimum demands of the moment of inertia of stiffeners:
length of buckling field (distance of stiffeners) a = 656.4 mm
web height between the flanges h_{wc} = 420.8 mm
moment of inertia of stiffeners I_{st} = 857.38 cm⁴
minimum moment of inertia for $a/h_{wc} = 1.56 \ge 2^{1/2}$: $I_{st,min}$ = 26.21 cm⁴ < I_{st} ok.
requirement concerning stiffeners to aviod lateral torsional buckling:

torsional moment of inertia of stiffeners I_T = 10.16 cm^4
polar moment of inertia of stiffeners I_p = 94.58 cm^4
$I_T / I_p \approx 0.107 > 0.006 = 5.3 \cdot f_{y,st}/E_{st}$ **ok.**

design resistance of stiffened webs with transverse compression:
area of stiffeners incl. web A_{st} = 28.50 cm^2
slenderness λ = 0.082
$\lambda \leq 0.2 \;\Rightarrow$ no deduction (χ = 1.0)
design value of resistance of flexural buckling $F_{c,w,Rd}$ = 608.9 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: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 189.0 mm, $l_{eff,cp}$ = 197.9 mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 189.0 mm
design tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 2.37 kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_{y,bp}) / \gamma_{M0}$ = 4.44 kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd}$ = 352.80 kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd} + 2 \cdot M_{bp,Rd}) / m$ = 582.56 kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 262.79 kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 352.80 kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 262.79 kN
design resistance of the weld: $F_{w,Rd}$ = 314.26 kN per side

row 2
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 178.5 mm, $l_{eff,cp}$ = 197.9 mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 178.5 mm
design tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 2.24 kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_{y,bp}) / \gamma_{M0}$ = 4.19 kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd}$ = 352.80 kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd} + 2 \cdot M_{bp,Rd}) / m$ = 550.19 kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n)$ = 259.08 kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd}$ = 352.80 kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 259.08 kN

design resistance of column flange in bending (per bolt-row)
$F_{fc,Rd,1}$ = 262.8 kN, $l_{eff,1}$ = 189.0 mm
$F_{fc,Rd,2}$ = 259.1 kN, $l_{eff,1}$ = 178.5 mm

equivalent T-stub flange (group of bolt-rows decisive):
here: number of bolt rows n_b = 2 (between stiffeners)
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp})$ = 289.0 mm, $\Sigma l_{eff,cp}$ = 397.9 mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc}$ = 289.0 mm
design tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0}$ = 3.62 kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_{y,bp}) / \gamma_{M0}$ = 6.79 kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd}$ = 705.60 kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd} + 2 \cdot M_{bp,Rd}) / m = 890.79$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 494.13$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 705.60$ kN

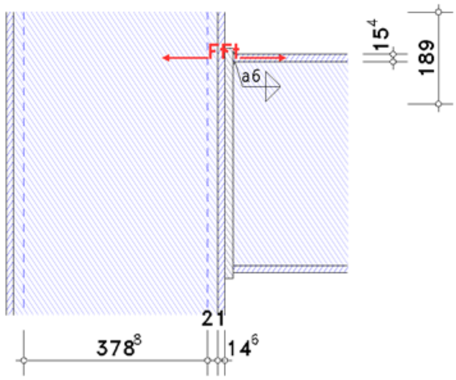
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 494.13$ kN

design resistance of column flange in bending (group of bolts, 2 rows)

$F_{fc,Rd} = 494.1$ kN, $\Sigma l_{eff,1} = 289.0$ 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 $b_{eff,t} = 189.0$ mm (l_{eff} from bc 4)

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.929$

design resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 387.9$ kN

row 2

effective width $b_{eff,t} = 178.5$ mm (l_{eff} from bc 4)

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.936$

design resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 369.0$ kN

group of bolt-rows decisive:

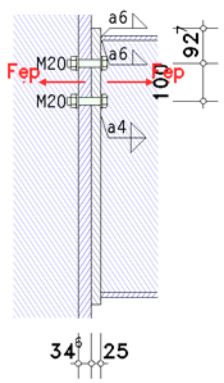
effective width $b_{eff,t} = 289.0$ mm (l_{eff} from bc 4)

reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \omega = 0.854$

design resistance of a column web with transverse tension

$F_{t,wc,Rd} = \omega \cdot (b_{eff,t} \cdot t_{wc} \cdot f_{y,wc}) / \gamma_{M0} = 545.2$ kN

basic component 5: end-plate in bending



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

part of end-plate between beam flanges

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 (end-plate):

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 244.6$ mm, $l_{eff,cp} = 279.4$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 244.6$ mm

design tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.98$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.80$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 807.81$ kN mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 379.08$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.80$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 352.80$ kN

NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0$ mm $< a_{min} = t_{max}^{1/2} - 0.5 = 4.50$ mm !!

design resistance of the weld: $F_{w,Rd} = 203.36$ kN per side

row 2

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

in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 230.4$ mm, $l_{eff,cp} = 279.4$ mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 230.4$ mm

design tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 8.46$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 352.80$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 760.88$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 367.02$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 352.80$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 352.80$ kN

NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0$ mm $< a_{min} = t_{max}^{1/2} - 0.5 = 4.50$ mm !!

design resistance of the weld: $F_{w,Rd} = 191.55$ kN per side

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

$F_{ep,Rd,1} = 352.8$ kN, $l_{eff,1} = 244.6$ mm

$F_{ep,Rd,2} = 352.8$ kN, $l_{eff,1} = 230.4$ mm

equivalent T-stub flange (group of bolt-rows decisive):

here: number of bolt rows $n_b = 2$

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

in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 344.6$ mm, $\Sigma l_{eff,cp} = 479.4$ mm

in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 344.6$ mm

design tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 12.65$ kNm

in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 705.60$ kN

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 1138.06$ kN

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

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 635.36$ kN

mode 3: bolt failure

$F_{T,3,Rd} = \Sigma F_{t,Rd} = 705.60$ kN

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 635.36$ kN

NA-DE: plate thickness $t_{max} \geq 3$ mm: weld thickness $a = 4.0$ mm $< a_{min} = t_{max}^{1/2} - 0.5 = 4.50$ mm !!

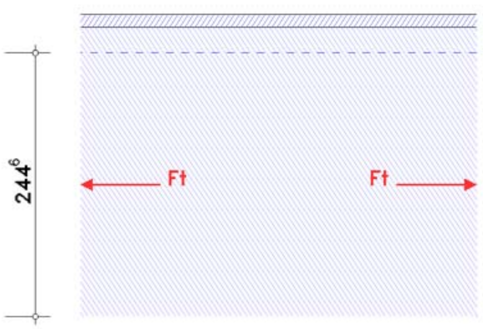
design resistance of the weld: $F_{w,Rd} = 286.50$ kN per side

total loading capacity of the T-stub flange: $F_{T,Rd} = 2 \cdot F_{w,Rd} = 573.01$ kN

design resistance of end-plate in bending (group of bolts, 2 row(s))

$F_{t,ep,Rd} = 573.0$ kN, $\Sigma l_{eff,1} = 344.6$ mm

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 1

effective width $b_{eff,t,wb} = 244.6$ mm (l_{eff} from bc 5)

design resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 459.9$ kN

row 2

effective width $b_{eff,t,wb} = 230.4$ mm (l_{eff} from bc 5) design resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 433.1$ kN

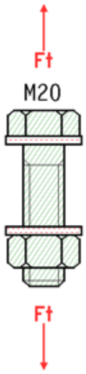
group of bolt-rows decisive:

effective width $b_{eff,t,wb} = 344.6$ mm (l_{eff} from bc 5)

design resistance of a beam web in tension

$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot f_{y,wb} / \gamma_{M0} = 647.9$ kN

basic component 10: bolts in tension



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

bolt category E:

tension resistance of one bolt: $F_{t,Rd} = F_{p,Cd} = 0.7 \cdot f_{ub} \cdot A_s / \gamma_{M7} = 155.9$ kN

p. sh. load capicity: $B_{p,Rd} = (0.6 \cdot \pi \cdot d_m \cdot t_p \cdot f_u)) / \gamma_{M2} = 265.64$ kN, $t_p = 14.6$ mm

tension-/punching shear load capicity for 2 bolts: $\Sigma F_{t,Rd} = 2 \cdot \min(F_{t,Rd}, B_{p,Rd}) = 311.82$ kN

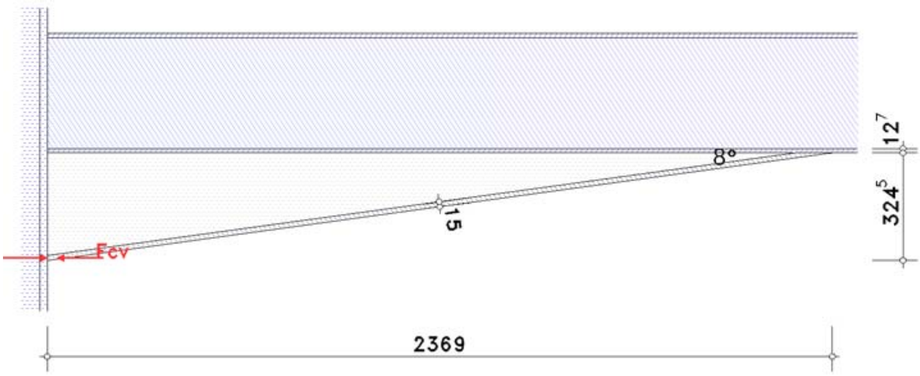
basic component 20: haunched beam in compression

flange bottom: section class for $c/(\varepsilon \cdot t) = 4.10$: 1

web: section class for $\alpha = 0.49$ and $c/(\varepsilon \cdot t) = 78.53$: 2

section class of the beam in connection plane: 2

taking into account the moment-shear force-interaction $V_{Ed} = 87.0$ kN



assumption: haunch flange no hazard of buckling: $c/t = 5.1 \cdot \varepsilon \leq 9 \cdot \varepsilon \Rightarrow$ section class 1 ok.

connection haunch-column: **(basic component 7: beam flange and web in compression)**

$h = h_b + h_v = 684.5$ mm > 600 mm $\Rightarrow$ assumption: $t'_{w,v} = 0.2 \cdot t_{w,v} = 1.6$ mm

stress due to bending with shear force: $V_{Ed} = 87.0$ kN ≤ 336.9 kN $= V_{pl,Rd}/2 \Rightarrow$ no effect

moment resistance $M_{c,Rd} = M_{pl,Rd} = (W_{pl} \cdot f_y) / \gamma_{M0} = 407.75$ kNm

design resistance of a flange and web in compression

$F_{c,f,Rd} = M_{c,Rd} / (h - t_f) = 609.02$ kN

connection design capacity

moment resistance

distance between bolt-row(s) in tension and centre of compression:

$h_1 = 611.9$ mm, $h_2 = 511.9$ mm

design resistances acc. to 6.2.7.2(6) for bolt-rows considered individually

decisive basic components: 3, 4, 5, 8

row 1: $F_{tr,Rd} = 262.8$ kN

row 2: $F_{tr,Rd} = 259.1$ kN

deductions acc. to 6.2.7.2(8) for bolt-rows as part of a group (column)

decisive basic components: 3, 4

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

Gk 3: $\Delta F_{tr,Rd} = F_{t,wc,Rd} - \Sigma F_{tr,Rd} = 545.2$ kN $F_{tr,Rd} = 262.8$ kN $< \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 262.8$ kN

Gk 4: $\Delta F_{tr,Rd} = F_{t,fc,Rd} - \Sigma F_{tr,Rd} = 494.1$ kN $F_{tr,Rd} = 262.8$ kN $< \Delta F_{tr,Rd} \Rightarrow F_{tr,Rd} = 262.8$ kN

row 2: $\Sigma F_{tr,Rd}$ = 262.8 kN (row 1)
Gk 3: $\Delta F_{tr,Rd} = F_{t,wc,Rd} - \Sigma F_{tr,Rd}$ = 282.4 kN $F_{tr,Rd}$ = 259.1 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 259.1 kN
Gk 4: $\Delta F_{tr,Rd} = F_{t,fc,Rd} - \Sigma F_{tr,Rd}$ = 231.3 kN $F_{tr,Rd}$ = 259.1 kN $>$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 231.3 kN

deductions acc. to 6.2.7.2(8) for bolt-rows as part of a group (end-plate)
decisive basic components: 5, 8
row 1: $\Sigma F_{tr,Rd}$ = 0.0 kN
Gk 5: $\Delta F_{tr,Rd} = F_{t,ep,Rd} - \Sigma F_{tr,Rd}$ = 573.0 kN $F_{tr,Rd}$ = 262.8 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 262.8 kN
Gk 8: $\Delta F_{tr,Rd} = F_{t,wb,Rd} - \Sigma F_{tr,Rd}$ = 647.9 kN $F_{tr,Rd}$ = 262.8 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 262.8 kN
row 2: $\Sigma F_{tr,Rd}$ = 262.8 kN (row 1)
Gk 5: $\Delta F_{tr,Rd} = F_{t,ep,Rd} - \Sigma F_{tr,Rd}$ = 310.2 kN $F_{tr,Rd}$ = 231.3 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 231.3 kN
Gk 8: $\Delta F_{tr,Rd} = F_{t,wb,Rd} - \Sigma F_{tr,Rd}$ = 385.1 kN $F_{tr,Rd}$ = 231.3 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 231.3 kN

deductions acc. to 6.2.7.2(7)
decisive basic components: 1, 2, 20
row 1: $\Sigma F_{tr,Rd}$ = 0.0 kN
Gk 2: $\Delta F_{tr,Rd} = F_{c,wc,Rd} - \Sigma F_{tr,Rd}$ = 608.9 kN $F_{tr,Rd}$ = 262.8 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 262.8 kN
Gk 20: $\Delta F_{tr,Rd} = F_{c,fv,Rd} - \Sigma F_{tr,Rd}$ = 609.0 kN $F_{tr,Rd}$ = 262.8 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 262.8 kN
row 2: $\Sigma F_{tr,Rd}$ = 262.8 kN (row 1)
Gk 2: $\Delta F_{tr,Rd} = F_{c,wc,Rd} - \Sigma F_{tr,Rd}$ = 346.1 kN $F_{tr,Rd}$ = 231.3 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 231.3 kN
Gk 20: $\Delta F_{tr,Rd} = F_{c,fv,Rd} - \Sigma F_{tr,Rd}$ = 346.2 kN $F_{tr,Rd}$ = 231.3 kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd}$ = 231.3 kN

check acc. to 6.2.7.2(9)
decisive basic component: 10
row 1: $F_{tx,Rd}$ = 262.8 kN, h_x = 611.9 mm $\Rightarrow$ $F_{tx,Rd} \leq 0.95 \cdot \Sigma F_{t,Rd}$ = 296.2 kN, no deduction

design resistance per bolt-row (finally)
row 1: $F_{tr,Rd}$ = 262.8 kN
row 2: $F_{tr,Rd}$ = 231.3 kN
potential failure by basic component 4

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r)$ = 279.2 kNm
tension resistance
$N_{j,Rd} = \Sigma F_{tr,Rd}$ = 494.1 kNm
compression resistance
$N_{j,c,Rd} = \min F_{c,Rd}$ = 608.9 kNm

verifications

equivalent lever arm for 2 bolt-rows: $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r)$ = 568.1 mm

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

axial force: $N_{b,Ed} = |N_d \cdot \cos(\alpha) + V_d \cdot \sin(\alpha)|$ = 58.61 kN $<$ 5%$\cdot N_{pl,Rd}$ = 121.89 kN $\Rightarrow$ moment resistance
regarding beam axis
internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu}$ = 268.33 kNm, z_{bu} = 320.9 mm

$M_{Ed}/M_{j,Rd}$ = 0.961 $<$ 1 ok.

verification of web stiffeners (ribs)

column
compression stiffener
$F_{c,Ed}$ = 518.04 kN
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 188.9 kN, $H = F \cdot e_F/e_H$ = 25.9 kN
assumption: stiffeners do not buckle: $c/t = 6.0 \cdot \varepsilon \leq 33 \cdot \varepsilon$ $\Rightarrow$ section class 1 $\leq$ 2 ok.
note: $b_R > 79.0$ mm $\Rightarrow$ end return is not possible, pay attention to corrosion protection !!
cross section at flange
compression resistance: $F_{c,Rd} = (A \cdot f_y) / \gamma_{M0}$ = 230.18 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 194.2 kN
F_{Ed} = 194.2 kN $<$ F_{Rd} = 230.2 kN $\Rightarrow$ U = 0.844 $<$ 1 ok.
cross section at web
shear resistance: $V_{p,Rd} = (f_y \cdot A_v) / (3^{1/2} \cdot \gamma_{M0})$ = 856.40 kN
design value: $F_{Ed} = F$ = 188.9 kN
F_{Ed} = 188.9 kN $<$ F_{Rd} = 856.4 kN $\Rightarrow$ U = 0.221 $<$ 1 ok.
flange welds
design resistance of the weld: $F_{w,Rd}$ = 16.63 kN/cm
design value: $F_{Ed} = (F^2 + H^2)^{1/2} / (2 \cdot b_1)$ = 14.60 kN/cm, b_1 = 65.3 mm
F_{Ed} = 14.60 kN/cm $<$ F_{Rd} = 16.63 kN/cm $\Rightarrow$ U = 0.878 $<$ 1 ok.
web welds

design resistance of the weld: $F_{w,Rd} = 8.31$ kN/cm
design value: $F_{Ed} = F / (2 \cdot l_1) = 2.55$ kN/cm, $l_1 = 370.8$ mm
$F_{Ed} = 2.55$ kN/cm $<$ $F_{Rd} = 8.31$ kN/cm $\Rightarrow$ $U = 0.306 < 1$ **ok.**

stiffener in tension

$F_{t,Ed} = 472.34$ kN

forces per rib

$F = 0.5 \cdot F_{t,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f = 172.3$ kN, $H = F \cdot e_F/e_H = 23.6$ kN
note: $b_R > 79.0$ mm $\Rightarrow$ end return is not possible, pay attention to corrosion protection !!

cross section at flange

tension resistance: $F_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = 230.18$ kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2} = 177.1$ kN
$F_{Ed} = 177.1$ kN $<$ $F_{Rd} = 230.2$ kN $\Rightarrow$ $U = 0.769 < 1$ **ok.**

cross section at web

shear resistance: $V_{p,Rd} = (f_y \cdot A_v) / (3^{1/2} \cdot \gamma_{M0}) = 856.40$ kN
design value: $F_{Ed} = F = 172.3$ kN
$F_{Ed} = 172.3$ kN $<$ $F_{Rd} = 856.4$ kN $\Rightarrow$ $U = 0.201 < 1$ **ok.**

flange welds

design resistance of the weld: $F_{w,Rd} = 16.63$ kN/cm
design value: $F_{Ed} = (F^2 + H^2)^{1/2} / (2 \cdot b_1) = 13.31$ kN/cm, $b_1 = 65.3$ mm
$F_{Ed} = 13.31$ kN/cm $<$ $F_{Rd} = 16.63$ kN/cm $\Rightarrow$ $U = 0.801 < 1$ **ok.**

web welds

design resistance of the weld: $F_{w,Rd} = 8.31$ kN/cm
design value: $F_{Ed} = F / (2 \cdot l_1) = 2.32$ kN/cm, $l_1 = 370.8$ mm
$F_{Ed} = 2.32$ kN/cm $<$ $F_{Rd} = 8.31$ kN/cm $\Rightarrow$ $U = 0.279 < 1$ **ok.**

verification of the shear area

column web

requirements concerning stiffeners: s. verification of web stiffeners
requirements concerning shear area: s. verification of buckling resistance
internal forces and moments at shear area (sign definition of statics):
$N_3 = -87.00$ kN, $M_3 = -287.65$ kNm, $V_3 = -45.70$ kN, $N_4 = -45.70$ kN, $M_4 = -283.64$ kNm, $V_4 = 87.00$ kN
dimensions of the shear area: $h_b = 420.8$ mm, $h_t = 420.8$ mm, $h_l = 560.6$ mm, $h_r = 560.6$ mm
stresses within the shear area:
$\tau_b = 119.4$ N/mm², $\tau_t = 119.4$ N/mm², $\tau_l = 117.1$ N/mm², $\tau_r = 117.1$ N/mm²
verification of the shear area:
max $\tau_{Ed} = 119.4$ N/mm² $<$ $\tau_{Rd} = 135.7$ N/mm² $\Rightarrow$ $U = 0.880 < 1$ **ok.**

verification of buckling resistance

column web

requirements concerning stiffeners: s. verification of web stiffeners
plate buckling: section class of the web plate $1 \leq 2$ **ok.**
shear buckling: $h_p/t_p = 44.77 \leq 72/(\eta \cdot \varepsilon) = 60.00$ **ok.**

verification result

maximum utilization: max $U = 0.961 < 1$ **ok.**

Final result

maximum utilization: max $U = 0.961 < 1$ **ok.**

verification succeeded