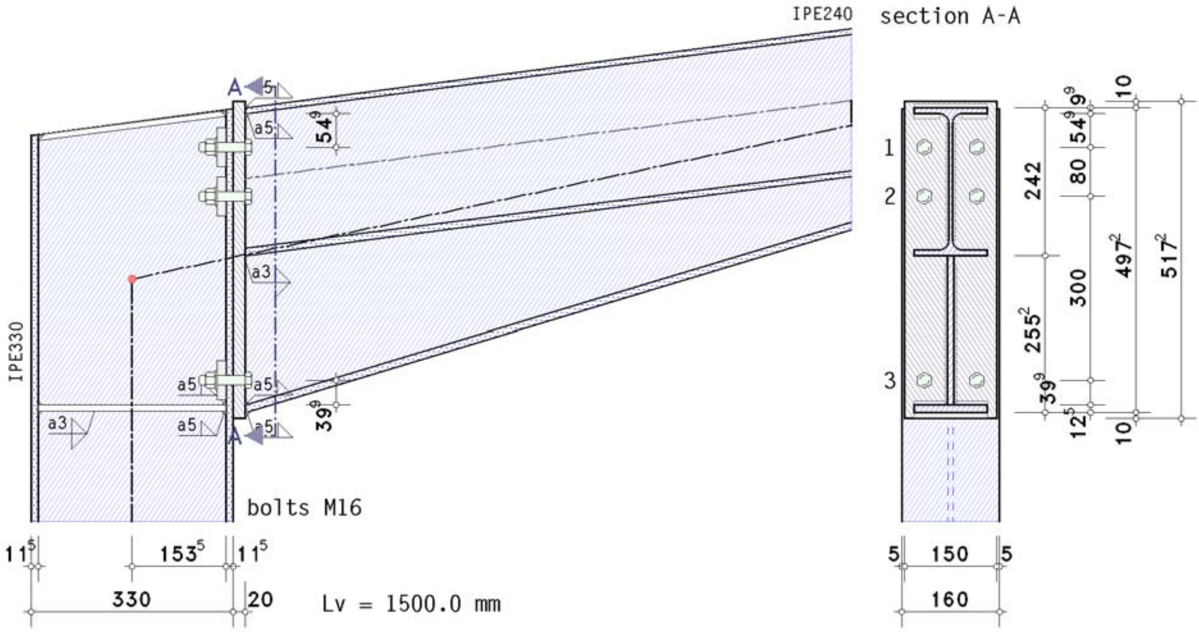
POS. 4: KINDMANN/STRACKE 3.9.6

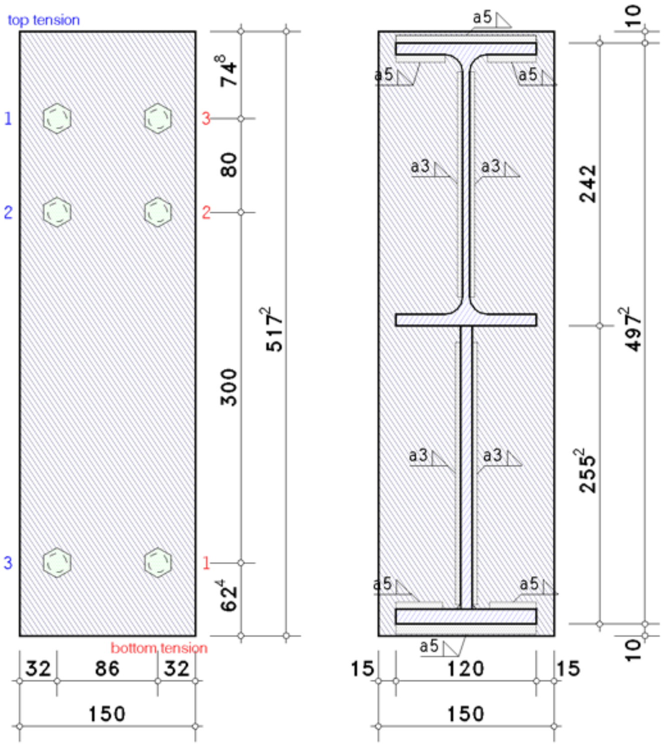
4H-EC3RE version: 5/2014-1x

frame corner

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



details



steel grade

steel grade S 235

column parameters

section IPE330

reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 481.9 mm):

thickness t_{st} = 9.8 mm

welds $a_{st,f}$ = 5.0 mm, $a_{st,w}$ = 3.0 mm

beam parameters

section IPE240

section angle of inclination about the horizontal axis α_b = 7.40° ⇒ section depth at the joint loc. $h_b = h/\cos(\alpha_b)$ = 242.0 mm

haunch angle of inclination about the horizontal axis α_v = 16.70° ⇒ haunch angle about the beam axis $\Delta\alpha_v$ = 9.30°

haunch length L_v = 1500.0 mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b))$ = 255.2 mm
web thickness $t_{w,v}$ = 10.0 mm, flange width, thickness $b_{f,v}$ = 120.0 mm, $t_{f,v}$ = 12.0 mm
total beam depth at the connection point $h_{ges} = h_b + h_v$ = 497.2 mm

bolts

bolt: bolt class 10.9, bolt size M16
shear plane passes through the unthreaded portion of the bolt
flange reinforcement: thickness t_{bp} = 15.0 mm, length l_{bp} = 64.0 mm, width b_{bp} = 64.0 mm

verification parameters

bolted end-plate joint:
thickness t_p = 20.0 mm, length l_p = 517.2 mm, width b_p = 150.0 mm
projections $h_{p,o}$ = 10.0 mm, $h_{p,u}$ = 10.0 mm
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 1 bolt-row for shear transfer at tension top (row 3)
of these 1 bolt-row bottom (M^-) in tension (row 3)
and 1 bolt-row for shear transfer at tension bottom (row 3)
centre distance of the bolts to the lateral edge of the end-plate e_2 = 32.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) e_o = 74.8 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) e_u = 62.4 mm
centre distance of the first bolt-row to the free edge of the column (end row) e_1' = 63.3 mm
centre distance of the bolt-rows from each other p_{1-2} = 80.0 mm, p_{2-3} = 300.0 mm
welds at the connection point:
beam flange top: fillet weld, weld thickness a = 5.0 mm, angle φ = 83°
beam web: fillet weld, weld thickness a = 3.0 mm
beam flange bottom: fillet weld, weld thickness a = 5.0 mm, angle φ = 107°

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

Lk 1: $N_{j,b1,Ed}$ = -31.01 kN $M_{j,b1,Ed}$ = -133.00 kNm $V_{j,b1,Ed}$ = 44.49 kN
$N_{j,c1,Ed}$ = -50.00 kN $M_{j,c1,Ed}$ = -133.00 kNm $V_{j,c1,Ed}$ = -21.00 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

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

distances between bolt-rows at end-plate

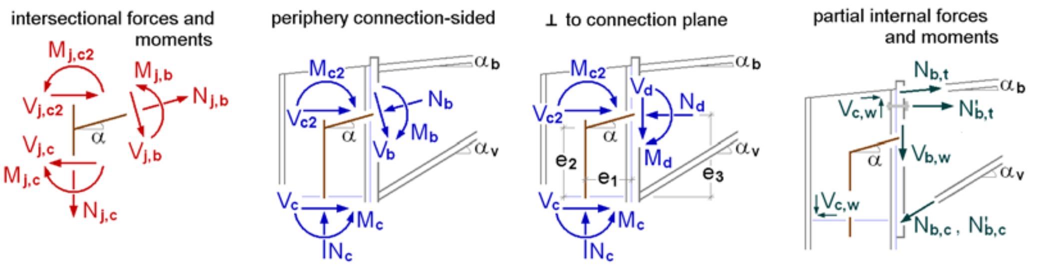
edge dist.:	e_2 = 32.0 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_2 = 32.0 mm < $4 \cdot t_{min}$ + 40 mm = 86.0 mm
pitch:	p_2 = 86.0 mm > $2.4 \cdot d_0$ = 43.2 mm,	p_2 = 86.0 mm < min(14·t_{min}, 200 mm) = 161.0 mm
edge dist.:	e_1 = 74.8 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_1 = 74.8 mm < $4 \cdot t_1$ + 40 mm = 120.0 mm
edge dist.:	e_1 = 63.3 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_1 = 63.3 mm < $4 \cdot t_2$ + 40 mm = 86.0 mm
pitch:	p_1 = 80.0 mm > $2.2 \cdot d_0$ = 39.6 mm,	p_1 = 80.0 mm < min(14·t_{min}, 200 mm) = 161.0 mm
pitch:	p_1 = 300.0 mm > $2.2 \cdot d_0$ = 39.6 mm,	p_1 = 300.0 mm > min(14·t_{min}, 200 mm) = 161.0 mm !!
edge dist.:	e_1 = 62.4 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_1 = 62.4 mm < $4 \cdot t_1$ + 40 mm = 120.0 mm

horizontal distance of bolts from column edge

edge dist.:	e_2 = 37.0 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_2 = 37.0 mm < $4 \cdot t_{min}$ + 40 mm = 86.0 mm

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

design values



angle of inclination: α_b = 7.4°, α_v = 16.7° ⇒ $\alpha = (\alpha_b+\alpha_v)/2$ = 12.1°

internal forces and moments in the periphery

$N_{b,Ed} = -N_{j,b,Ed}$ = 31.01 kN

$M_{b,Ed} = -M_{j,b,Ed} - V_{j,b,Ed}\cdot e_1/\cos(\alpha)$ = 125.49 kNm, e_1 = 165.0 mm

$V_{b,Ed} = V_{j,b,Ed}$ = 44.49 kN

periphery column (bottom):

$N_{c,Ed} = N_{b,Ed}\cdot\sin(\alpha) + V_{b,Ed}\cdot\cos(\alpha)$ = 49.98 kN

$M_{c,Ed} = M_{b,Ed} - V_{c,Ed}\cdot e_3 + N_{c,Ed}\cdot e_1$ = 128.86 kNm, e_1 = 165.0 mm, e_3 = 231.9 mm

$V_{c,Ed} = N_{b,Ed}\cdot\cos(\alpha) - V_{b,Ed}\cdot\sin(\alpha)$ = 21.04 kN

internal forces and moments perpendicular to the connection plane

$N_d = N_{b,Ed}\cdot\cos(\alpha) - V_{b,Ed}\cdot\sin(\alpha)$ = 21.04 kN

$M_d = M_{b,Ed}$ = 125.49 kNm

$V_d = N_{b,Ed}\cdot\sin(\alpha) + V_{b,Ed}\cdot\cos(\alpha)$ = 49.98 kN

partial internal forces and moments

internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d\cdot t_{ep}\cdot\tan(\alpha) - V_d\cdot t_{ep}$ = 124.58 kN

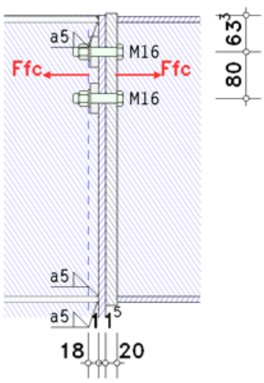
$N_{b,t} = (-N_d\cdot z_{bu}/z_b + M'_d/z_b) / \cos(\alpha_b)$ = 248.44 kN, z_b = 486.0 mm, z_{bu} = 230.1 mm

$N_{b,c} = (N_d\cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_v)$ = 279.19 kN, z_b = 486.0 mm, z_{bo} = 255.9 mm

basic components

end-plate joint: selected basic component(s): 4

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: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp})$ = 149.1 mm, $l_{eff,cp}$ = 156.1 mm

in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc}$ = 149.1 mm

design tension resistance of the T-stub flange:

in mode 1+2: $M_{pl,Rd} = (0.25\cdot\Sigma l_{eff}\cdot t_f^2\cdot f_y) / \gamma_{M0}$ = 1.16 kNm

flange reinforcement: $M_{bp,Rd} = (0.25\cdot\Sigma l_{eff,1}\cdot t_{bp}^2\cdot f_{y,bp}) / \gamma_{M0}$ = 1.97 kNm

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

L_b = 62.5 mm ≤ 93.5 mm = L_b^* ⇒ prying forces may develop !

mode 1: complete yielding of the T-stub flange

$F_{T,1,Rd} = (4\cdot M_{pl,1,Rd}+2\cdot M_{bp,Rd}) / m$ = 345.10 kN

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

$F_{T,2,Rd} = (2\cdot M_{pl,2,Rd} + n\cdot\Sigma F_{t,Rd}) / (m+n)$ = 167.04 kN

mode 3: bolt failure

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

tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd})$ = 167.04 kN

design resistance of the weld: $F_{w,Rd}$ = 154.95 kN per side

row 2
effective length of the T-stub flange (column flange):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 145.7$ mm, $l_{eff,cp} = 156.1$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 145.7$ mm
design tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 1.13$ kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_{y,bp}) / \gamma_{M0} = 1.93$ kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 226.08$ kN
$L_b = 62.5$ mm ≤ 95.7 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd} + 2 \cdot M_{bp,Rd}) / m = 337.11$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 166.08$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 226.08$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 166.08$ kN

design resistance of column flange in bending (per bolt-row)
$F_{fc,Rd,1} = 167.0$ kN, $l_{eff,1} = 149.1$ mm
$F_{fc,Rd,2} = 166.1$ kN, $l_{eff,1} = 145.7$ 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: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 229.1$ mm, $\Sigma l_{eff,cp} = 316.1$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 229.1$ mm
design tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 1.78$ kNm
flange reinforcement: $M_{bp,Rd} = (0.25 \cdot \Sigma l_{eff,1} \cdot t_{bp}^2 \cdot f_{y,bp}) / \gamma_{M0} = 3.03$ kNm
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 452.16$ kN
$L_b = 62.5$ mm ≤ 121.7 mm $= L_b^*$ $\Rightarrow$ prying forces may develop !
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd} + 2 \cdot M_{bp,Rd}) / m = 530.26$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 314.87$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 452.16$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 314.87$ kN

design resistance of column flange in bending (group of bolts, 2 rows)
$F_{fc,Rd} = 314.9$ kN, $\Sigma l_{eff,1} = 229.1$ mm

connection design capacity

moment resistance

distance between bolt-row(s) in tension and centre of compression:
$h_1 = 426.2$ mm, $h_2 = 346.2$ mm

design resistances acc. to 6.2.7.2(6) for bolt-rows considered individually
row 1: $F_{tr,Rd} = 167.0$ kN
row 2: $F_{tr,Rd} = 166.1$ kN

deductions acc. to 6.2.7.2(8) for bolt-rows as part of a group (column)
row 1: $\Sigma F_{tr,Rd} = 0.0$ kN
Gk 4: $\Delta F_{tr,Rd} = F_{t,fc,Rd} - \Sigma F_{tr,Rd} = 314.9$ kN $F_{tr,Rd} = 167.0$ kN $<$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 167.0$ kN
row 2: $\Sigma F_{tr,Rd} = 167.0$ kN (row 1)
Gk 4: $\Delta F_{tr,Rd} = F_{t,fc,Rd} - \Sigma F_{tr,Rd} = 147.8$ kN $F_{tr,Rd} = 166.1$ kN $>$ $\Delta F_{tr,Rd}$ $\Rightarrow$ $F_{tr,Rd} = 147.8$ kN

design resistance per bolt-row (finally)
row 1: $F_{tr,Rd} = 167.0$ kN
row 2: $F_{tr,Rd} = 147.8$ kN
potential failure by basic component 4

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 122.4$ kNm
tension resistance
$N_{j,Rd} = \Sigma F_{tr,Rd} = 314.9$ kNm

verifications

equivalent lever arm for 2 bolt-rows: $z_{eq} = \Sigma(k_{eff,r} \cdot h_r^2) / \Sigma(k_{eff,r} \cdot h_r) = 390.9$ mm

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

axial force: $N_{b,Ed} = |N_d \cdot \cos(\alpha) + V_d \cdot \sin(\alpha)| = 31.01$ kN $<$ $5\% \cdot N_{pl,Rd} = 68.46$ kN $\Rightarrow$ moment resistance

regarding beam axis

internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu} = 120.74$ kNm, $z_{bu} = 225.9$ mm

$M_{Ed}/M_{j,Rd} = 0.987 < 1$ ok.

verification result

maximum utilization: max U = 0.987 $<$ 1 ok.

Final result

maximum utilization: max U = 0.987 $<$ 1 ok.

verification succeeded