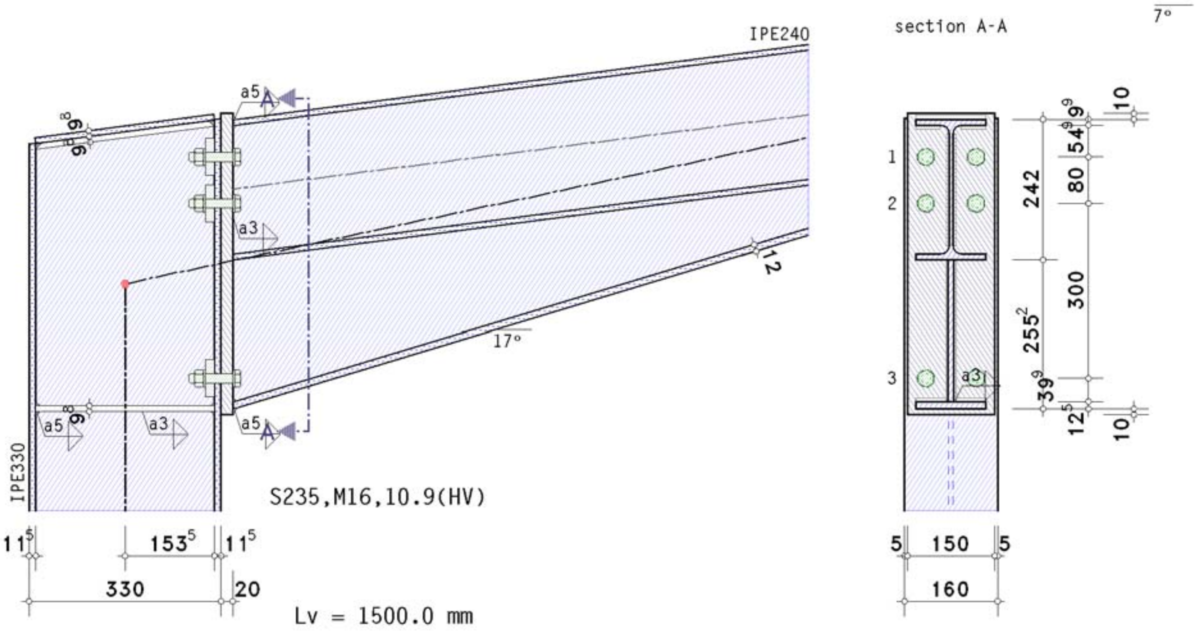
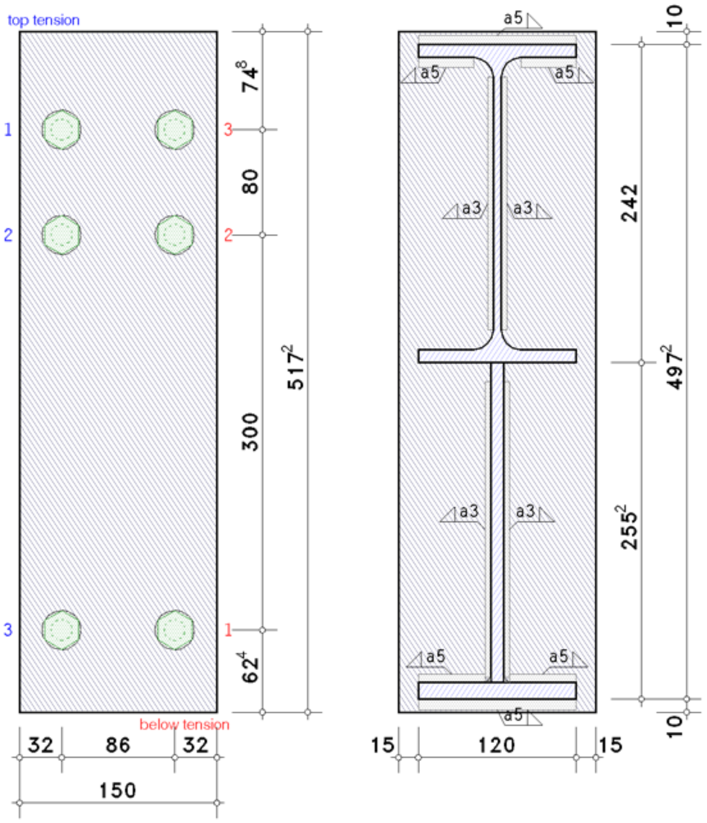
POS. 6: KINDMANN/STRACKE 3.9.6

frame corner EC 3-1-8 (04.25), NA: Deutschland 4H-EC3RE version: 6/2026-1b

1. input report



details (section A - A)



steel grade

steel grade S235

column parameters

section IPE330

reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 487.9 mm):

thickness t_{st} = 9.8 mm, width b_{st} = 76.3 mm, length l_{st} = 307.0 mm

recess at stiffeners c_{st} = 27.0 mm

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

cap plate t_k = 9.8 mm (has no effect)

bolts

bolt class 10.9, bolt size M16

large wrench size (high strength bolt), preloaded (for info: preloading $F_{p,C}^*$ = $0.7 \cdot f_{yb} \cdot A_s$ = 98.7 kN)

shear plane passes through the unthreaded portion of the bolt packing (flange reinforcement): thickness t_{bp} = 15.0 mm, width b_{bp} = 64.0 mm, length l_{bp} = 64.0 mm

beam parameters

section IPE240

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

slope angle of haunch 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, weld thickness a_v = 3.0 mm

total beam depth at the connection point $h_{ges} = h_b + h_v$ = 497.2 mm

verification parameters

bolted end-plate connection

thickness t_p = 20.0 mm, width b_p = 150.0 mm, length l_p = 517.2 mm

projections $h_{p,o}$ = 10.0 mm, $h_{p,u}$ = 10.0 mm

bolts in connection:

3 bolt-rows with 2 bolts

of these 2 bolt-rows top in tension (rows 1-2)

and 1 bolt-row for shear transfer top (row 3)

of these 1 bolt-row below in tension (row 3)

and 1 bolt-row for shear transfer below (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' = 74.8 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°

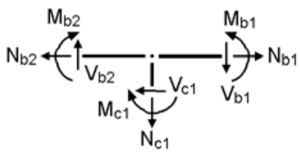
beam web: fillet weld, weld thickness a = 3.0 mm

beam flange below: fillet weld, weld thickness a = 5.0 mm, angle φ = 107°

internal forces and moments in the intersection point of system axes

Lc 1: $N_{j,b,Ed}$ = -31.01 kN $M_{j,b,Ed}$ = -133.00 kNm $V_{j,b,Ed}$ = 44.49 kN

$N_{j,c1,Ed}$ = -49.98 kN $M_{j,c1,Ed}$ = -133.00 kNm $V_{j,c1,Ed}$ = -21.04 kN (calculated)



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

notes

certain basic components have been deselected,

that may not ensure the total load-bearing capacity of the joint !

connection is verified due to EC 3-1-8 regardless of preloading.

however, connections may be constructed with prestressed high strength bolts.

for haunched beams, the bottom flange of the beam profile is not considered. a fictious profile is formed from the upper beam flange, the beam web and the haunch flange.

only at calcualation of end plate the lower beam flange is respected as a stiffener.

no verification for cross-sections.

no verification of haunch connection to beam.

welds of the haunch profile are not verified.

welds are not regarded by calculation the T-stub resistance.

local resistance of weld is not respected.

no verification of web stiffeners.

plated structures- and shear buckling is not investigated.

shear area is not investigated.

calculation of T-stub-resistance with the standard method.

the cap plate has no effect.

local shear resistance of end plate is not respected.

check of data

ok

distances between bolts at end-plate

horizontal:	e_2 = 32.0 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_2 = 32.0 mm < $4 \cdot t$ + 40 mm = 86.0 mm
horizontal:	p_2 = 86.0 mm > $2.4 \cdot d_0$ = 43.2 mm,	p_2 = 86.0 mm < min(14·t, 200 mm) = 161.0 mm
top-below:	e_1 = 74.8 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_1 = 74.8 mm < $4 \cdot t$ + 40 mm = 86.0 mm
top-below:	e_1 = 74.8 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_1 = 74.8 mm < $4 \cdot t$ + 40 mm = 86.0 mm
top-below:	p_1 = 80.0 mm > $2.2 \cdot d_0$ = 39.6 mm,	p_1 = 80.0 mm < min(14·t, 200 mm) = 161.0 mm
top-below:	p_1 = 300.0 mm > $2.2 \cdot d_0$ = 39.6 mm,	p_1 = 300.0 mm > min(14·t, 200 mm) = 161.0 mm !!
top-below:	e_1 = 62.4 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_1 = 62.4 mm < $4 \cdot t$ + 40 mm = 86.0 mm

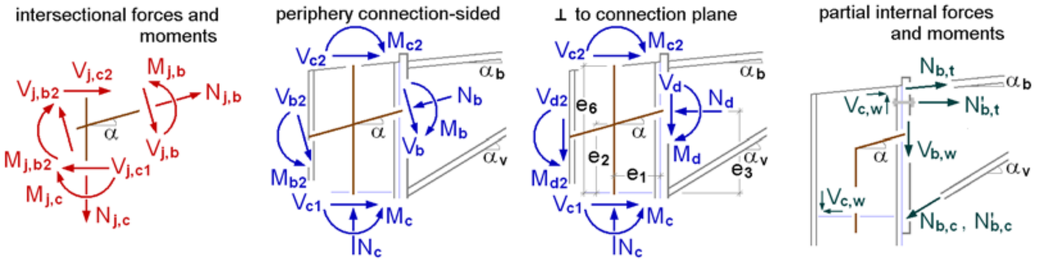
bolt distance from column edge

horizontal:	e_2 = 37.0 mm > $1.2 \cdot d_0$ = 21.6 mm,	e_2 = 37.0 mm < $4 \cdot t$ + 40 mm = 86.0 mm

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

2. Lc 1

2.1. design values



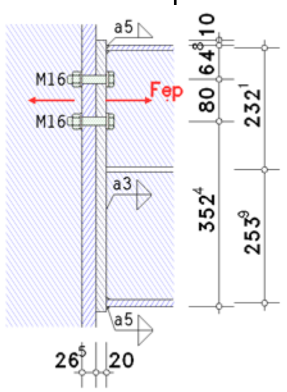
slope angle: $\alpha_b = 7.40°$, $\alpha_v = 16.70°$ $\Rightarrow$ $\alpha = (\alpha_b+\alpha_v)/2 = 12.05°$
distances: $e_1 = 165.0$ mm, $e_3 = 248.4$ mm, $e_2 = 213.2$ mm, $e_6 = 487.9$ mm

internal forces and moments perpendicular to the connection planes
periphery beam
$N_d = 21.04$ kN, $M_d = 125.49$ kNm, $V_d = 49.98$ kN
periphery column (below, calculated)
$N_c = 49.98$ kN, $M_c = 128.52$ kNm, $V_c = 21.04$ kN

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: $M'_d = M_d + N_d \cdot t_p \cdot \tan(\alpha) - V_d \cdot t_p = 124.58$ kNm
$N_{b,t} = (-N_d \cdot z_{bu}/z_b + M'_d/z_b) / \cos(\alpha_b) = 248.30$ kN, $z_b = 486.0$ mm, $z_{bu} = 233.5$ mm
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M'_d/z_b) / \cos(\alpha_v) = 279.04$ kN, $z_b = 486.0$ mm, $z_{bo} = 252.5$ mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b) = -31.98$ kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v) = 80.18$ kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c} = 1.78$ kN

2.2. basic components

2.2.1. bc 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 individual bolt-row):
here: number of bolt-rows $n_b = 1$
row 1
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 189.8$ mm, $l_{eff,cp} = 229.4$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 189.8$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 4.46$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 112.82$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 225.65$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 488.69$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 235.61$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 225.65$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 225.65$ kN
row 2
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = l_{eff,1} = \min(l_{eff,nc}, l_{eff,cp}) = 186.0$ mm, $l_{eff,cp} = 229.4$ mm
in mode 2: $\Sigma l_{eff,2} = l_{eff,2} = l_{eff,nc} = 186.0$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 4.37$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 112.82$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 225.65$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 479.00$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange

$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 233.03$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 225.65$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 225.65$ kN

resistances and effective lengths of end-plate in bending (per bolt-row):
$F_{ep,Rd,1} = 225.65$ kN, $l_{eff,1} = 189.8$ mm
$F_{ep,Rd,2} = 225.65$ kN, $l_{eff,2} = 186.0$ mm

equivalent T-stub flange (group of bolt-rows):
here: number of bolt-rows $n_b = 2$ (R1+R2)
effective length of the T-stub flange (end-plate):
in mode 1: $\Sigma l_{eff,1} = \min(\Sigma l_{eff,nc}, \Sigma l_{eff,cp}) = 216.8$ mm, $\Sigma l_{eff,cp} = 354.7$ mm
in mode 2: $\Sigma l_{eff,2} = \Sigma l_{eff,nc} = 216.8$ mm
tension resistance of the T-stub flange:
in mode 1+2: $M_{pl,Rd} = (0.25 \cdot \Sigma l_{eff} \cdot t_f^2 \cdot f_y) / \gamma_{M0} = 5.09$ kNm
$F_{t,Rd} = (k_2 \cdot f_{ub} \cdot A_s) / \gamma_{M2} = 112.82$ kN, $k_2 = 0.90$
in mode 3: $\Sigma F_{t,Rd} = 2 \cdot n_b \cdot F_{t,Rd} = 451.30$ kN
mode 1: complete yielding of the T-stub flange
$F_{T,1,Rd} = (4 \cdot M_{pl,1,Rd}) / m = 558.18$ kN
mode 2: bolt failure simultaneously with yielding of the T-stub flange
$F_{T,2,Rd} = (2 \cdot M_{pl,2,Rd} + n \cdot \Sigma F_{t,Rd}) / (m+n) = 359.53$ kN
mode 3: bolt failure
$F_{T,3,Rd} = \Sigma F_{t,Rd} = 451.30$ kN
tension resistance of the T-stub flange: $F_{T,Rd} = \min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 359.53$ kN
effective length: $\Sigma l_{eff} = 216.8$ mm, 2 rows

2.3. connection capacity

2.3.1. moment resistance

distance of tension-bolt-rows from centre of compression: $h_1 = 426.2$ mm, $h_2 = 346.2$ mm

resistances acc. to EC 3-1-8, B.3.2.2(6) for bolt-rows considered individually
row 1: $F_{tr,Rd} = 225.6$ kN
row 2: $F_{tr,Rd} = 225.6$ kN

deductions acc. to EC 3-1-8, B.3.2.2(8) for bolt-rows as part of a group (end-plate)
row 1: $F_{tr,Rd} = 225.6$ kN
row 2: $F_{tr,Rd} = 133.9$ kN

resistance per bolt-row (tension)
row 1: $F_{tr,Rd} = 225.6$ kN
row 2: $F_{tr,Rd} = 133.9$ kN
$\Sigma F_{tr,Rd}^* = 359.5$ kN

resistance per bolt-row (bending)
row 1: $F_{tr,Rd} = 225.6$ kN
row 2: $F_{tr,Rd} = 133.9$ kN
$\Sigma F_{tr,Rd} = 359.5$ kN
potential failure by basic component 5

resistance of flanges (compression)
$\Sigma F_{c,Rd}^* = 0.0$ kN

moment resistance
$M_{j,Rd} = \Sigma(F_{tr,Rd} \cdot h_r) = 142.5$ kNm
tension resistance
$N_{j,t,Rd} = \Sigma F_{tr,Rd}^* = 359.5$ kN

2.4. verifications

2.4.1. verification of the connection 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%$\cdot N_{pl,Rd} = 67.41$ kN $\Rightarrow$ moment resistance
internal moment: $M_{Ed} = M_d - N_d \cdot z_{bu} = 120.67$ kNm, $z_{bu} = 229.3$ mm

resistance not ensured (compression) !!

2.4.2. verification result

fault at calcualation !!
fault at calculation of moment resistance !!

3. final result

resistance not ensured !!
total loading capacity of the connection may not be guaranteed (s. notes) !

verification could not be executed, see Lc 1 !!

4. Regulations

EN 1990, Eurocode 0: Grundlagen der Tragwerksplanung;
Deutsche Fassung EN 1990:2002 + A1:2005 + A1:2005/AC:2010, Ausgabe Dezember 2010
EN 1990/NA, Nationaler Anhang zur EN 1990, Ausgabe Dezember 2010

EN 1993-1-1, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau;
Deutsche Fassung EN 1993-1-1:2022, Ausgabe April 2025
EN 1993-1-1/NA, Nationaler Anhang zur EN 1993-1-1, Ausgabe April 2026

EN 1993-1-8, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
Teil 1-8: Bemessung von Anschlüssen; Deutsche Fassung EN 1993-1-8:2024, Ausgabe April 2025
EN 1993-1-8/NA, Nationaler Anhang zur EN 1993-1-8, Ausgabe April 2026

EN 1993-1-5, Eurocode 3: Bemessung und Konstruktion von Stahlbauten - Teil 1-5: Plattenförmige Bauteile;
Deutsche Fassung EN 1993-1-5:2006 + AC:2009 + A1:2017 + A2:2019, Ausgabe Oktober 2019
EN 1993-1-5/NA, Nationaler Anhang zur EN 1993-1-5, Ausgabe Dezember 2010