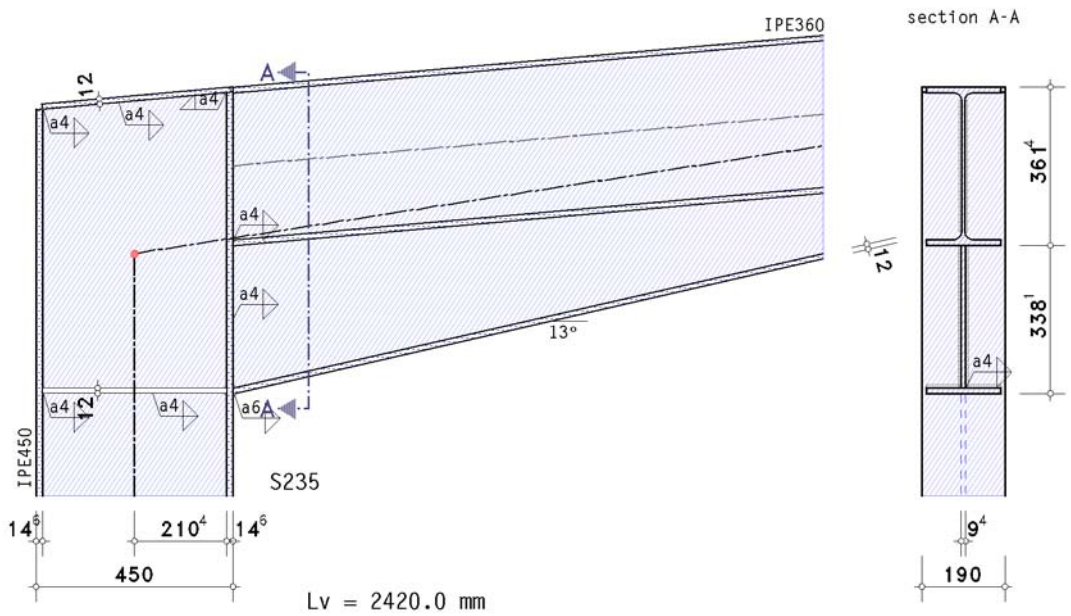
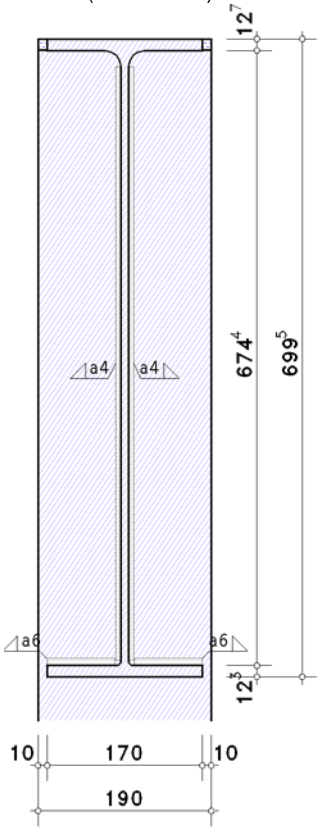
POS. 5: WAGENKNECHT BD.2 7.7

frame corner EC 3-1-8 (04.25), NA: Deutschland 4H-EC3RE version: 2/2020-2q

1. input report



details (section A - A)



steel grade

steel grade S235

column parameters

section IPE450

reinforcement of the section with transverse stiffeners (web stiffeners, d_{st} = 686.0 mm):

thickness t_{st} = 12.0 mm, width b_{st} = 90.3 mm, length l_{st} = 420.8 mm

recess at stiffeners c_{st} = 31.5 mm

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

beam parameters

section IPE360

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

slope angle of haunch about the horizontal axis α_v = 12.80° $\Rightarrow$ haunch angle about the beam axis $\Delta\alpha_v$ = 7.80°

haunch length L_v = 2420.0 mm, haunch depth at the connection point $h_v = L_v \cdot (\tan(\alpha_v) - \tan(\alpha_b))$ = 338.1 mm

web thickness $t_{w,v}$ = 10.0 mm, flange width, thickness $b_{f,v}$ = 170.0 mm, $t_{f,v}$ = 12.0 mm, weld thickness a_v = 4.0 mm
total beam depth at the connection point $h_{ges} = h_b + h_v$ = 699.5 mm

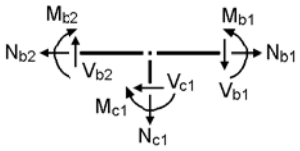
verification parameters

welded connection

tension plate: thickness t_z = 12.0 mm, width b_z = 190.0 mm
welds $a_{z,f}$ = 4.0 mm, $a_{z,w}$ = 4.0 mm
welds at the connection point:
fillet weld, weld thickness a = 0.0 mm
beam web: fillet weld, weld thickness a = 4.0 mm
beam flange below: fillet weld, weld thickness a = 6.0 mm, angle φ = 103°

internal forces and moments in the intersection point of system axes

Lc 1: $N_{j,b,Ed}$ = -58.20 kN $M_{j,b,Ed}$ = -301.00 kNm $V_{j,b,Ed}$ = 78.80 kN
$V_{j,b2,Ed}$ = -23.00 kN (cantilever)
$N_{j,c1,Ed}$ = -109.58 kN $M_{j,c1,Ed}$ = -301.00 kNm $V_{j,c1,Ed}$ = -41.75 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

notes

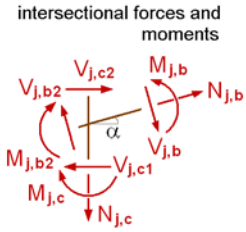
for haunched beams, the bottom flange of the beam profile is not considered. a fictious profile is formed from the upper beam flange, the beam web and the haunch flange.
only at calcualation of end plate the lower beam flange is respected as a stiffener.
no verification for cross-sections.
no verification of haunch connection to beam.
welds of the haunch profile are not verified.
local resistance of weld is not respected.
plated structures- and shear buckling is not investigated.

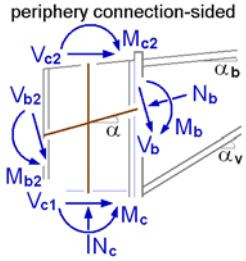
check of data

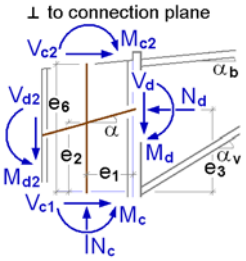
haunch flangedicke $t_{f,v}$ = 12.0 mm < beam flange thickness $t_{f,b}$ = 12.7 mm EC 3-requirement !!
ok

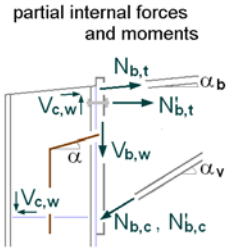
2. Lc 1

2.1. design values









slope angle: α_b = 5.00°, α_v = 12.80° ⇒ $\alpha = (\alpha_b+\alpha_v)/2$ = 8.90°
distances: e_1 = 225.0 mm, e_3 = 347.3 mm, e_2 = 312.1 mm, e_6 = 686.0 mm

internal forces and moments perpendicular to the connection planes

periphery beam
N_d = 45.31 kN, M_d = 283.05 kNm, V_d = 86.86 kN
periphery beam
N_{d2} = 3.56 kN, M_{d2} = -5.24 kNm, V_{d2} = 22.72 kN
periphery column (below, calculated)
N_c = 109.58 kN, M_c = 287.97 kNm, V_c = 41.75 kN

partial internal forces and moments

$N_{b,t} = (-N_d \cdot z_{bu}/z_b + M_d/z_b) / \cos(\alpha_b)$ = 390.43 kN, z_b = 686.9 mm, z_{bu} = 350.3 mm
$N_{b,c} = (N_d \cdot z_{bo}/z_b + M_d/z_b) / \cos(\alpha_v)$ = 445.32 kN, z_b = 686.9 mm, z_{bo} = 336.6 mm
$V_{b,t} = -N_{b,t} \cdot \sin(\alpha_b)$ = -34.03 kN, $V_{b,c} = N_{b,c} \cdot \sin(\alpha_v)$ = 98.66 kN, $V_{b,w} = V_d - V_{b,t} - V_{b,c}$ = 22.22 kN

2.2. verifications

2.2.1. verification of web stiffeners / of the tension plate

compression stiffener (below)

$F_{c,Ed}$ = 435.26 kN
forces per rib
$F = 0.5 \cdot F_{c,Ed} \cdot (b_f - 2 \cdot r - t_w)/b_f$ = 158.75 kN, $H = F \cdot e_F/e_H$ = 22.98 kN
assumption: stiffeners do not buckle: section class 1, section class 1 ≤ 3 ok
cross-section at flange
compression resistance $N_{c,Rd}$ = 165.82 kN
design value: $F_{Ed} = (F^2 + 3 \cdot H^2)^{1/2}$ = 163.67 kN

F_{Ed} = 163.67 kN < F_{Rd} = 165.82 kN ⇒ U = 0.987 < 1 ok
cross-section at web
shear resistance $V_{Rd} = (A_v \cdot f_y) / (3^{1/2} \cdot \gamma_{M0})$ = 582.54 kN
design value: F_{Ed} = F = 158.75 kN
F_{Ed} = 158.75 kN < F_{Rd} = 582.54 kN ⇒ U = 0.273 < 1 ok
flange welds
design values: $F_{Ed(\sigma s)} = F / (2 \cdot b_1)$ = 1349.95 kN/m, $F_{Ed(\tau p)} = H / (2 \cdot b_1)$ = 195.37 kN/m, b_1 = 58.8 mm
$\sigma_{1,w,Ed}$ = 347.9 N/mm² < $f_{1w,d}$ = 360.0 N/mm² ⇒ U = 0.966 < 1 ok
$\sigma_{2,w,Ed}$ = 337.5 N/mm² > $f_{2w,d}$ = 259.2 N/mm² ⇒ U = 1.302 > 1 not ok !!
web welds
design value: $F_{Ed(\tau p)} = F / (2 \cdot l_1)$ = 221.85 kN/m, l_1 = 357.8 mm
$\sigma_{1,w,Ed}$ = 96.1 N/mm² < $f_{1w,d}$ = 360.0 N/mm² ⇒ U = 0.267 < 1 ok
tension plate
$F_{t,Ed}$ = 391.44 kN
cross-section
tension resistance $N_{t,Rd}$ = 535.80 kN
F_{Ed} = 391.44 kN < F_{Rd} = 535.80 kN ⇒ U = 0.731 < 1 ok
welds flange
design value: $F_{Ed(\sigma s)} = 0.5 \cdot F / b_z$ = 751.43 kN/m, F = 285.5 kN
$\sigma_{1,w,Ed}$ = 187.9 N/mm² < $f_{1w,d}$ = 360.0 N/mm² ⇒ U = 0.522 < 1 ok
$\sigma_{2,w,Ed}$ = 187.9 N/mm² < $f_{2w,d}$ = 259.2 N/mm² ⇒ U = 0.725 < 1 ok
web welds
design value: $F_{Ed(\tau p)} = F / (2 \cdot d_w)$ = 376.90 kN/m, F = 285.5 kN
$\sigma_{1,w,Ed}$ = 163.2 N/mm² < $f_{1w,d}$ = 360.0 N/mm² ⇒ U = 0.453 < 1 ok

2.2.2. elastic verification of the shear area

column web
requirements concerning stiffeners: s. verification of web stiffeners
requirements concerning shear area: shear buckling: $h_p/t_p = 44.77 \le 72 \cdot \varepsilon/\eta = 60.00$ ok
internal forces and moments at web (sign definition of statics):
N_1 = -3.56 kN, M_1 = 5.07 kNm, V_1 = -22.72 kN
N_3 = -109.58 kN, M_3 = -287.97 kNm, V_3 = -41.75 kN
N_4 = -45.31 kN, M_4 = -283.64 kNm, V_4 = 86.86 kN
dimensions of the shear area: h_b = 420.8 mm, h_t = 422.4 mm, h_l = 635.3 mm, h_r = 673.3 mm
stresses within the shear area:
τ_b = 101.3 N/mm², τ_t = 101.3 N/mm², τ_l = 105.2 N/mm², τ_r = 104.8 N/mm²
verification of the shear area:
max τ_{Ed} = 105.2 N/mm² < τ_{Rd} = 135.7 N/mm² ⇒ U = 0.776 < 1 ok

2.2.3. verification result

maximum utilization: max U = 1.302 > 1 not ok !!
failure at verification of web stiffeners: U = 1.302

3. final result

maximum utilization: max U = 1.302 > 1 not ok !!

resistance not ensured !!
requirement acc. to EC 3-1-8 should be respected

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