1. input report

S235, M24, 10.9 (HV)

Steel grade

Steel grade S235, calculation parameters:
- Char. yield strength $f_y = 235.0 \text{ N/mm}^2$ for $t \leq 40 \text{ mm}$,
- $f_y = 215.0 \text{ N/mm}^2$ for $t > 40 \text{ mm}$
- Char. tensile strength $f_u = 360.0 \text{ N/mm}^2$ for $t \leq 40 \text{ mm}$,
- $f_u = 360.0 \text{ N/mm}^2$ for $t > 40 \text{ mm}$
- Correlation value of fillet weld $f_{w} = 0.80$
- Elastic modulus $E = 210000.0 \text{ N/mm}^2$

Column parameters

Section HE300B

Design values:
- Overall depth $h = 300.0 \text{ mm}$, web thickness $t_w = 11.0 \text{ mm}$
flange width br = 300.0 mm, flange thickness tr = 19.0 mm
rolled section, root radius r = 27.0 mm
clear depth of web without root/weld dw = 208.0 mm
width of one flange side without root/weld α = 117.5 mm
centroid distance from top zα = 150.0 mm
cross-sectional area A = 149.08 cm²
second moment of area Iy = 25164.07 cm⁴
plastic section modulus Wpl,y = 1869.00 cm³
elastic section modulus We,ly = ly/zα = 1677.60 cm³
effective shear area Aeq = 47.43 cm²
torsional moment of inertia Iτ = 186.00 cm⁴

bolts

bolt class 10.9
calculation parameters:
  char. yield strength fy = 900.0 N/mm²
  char. tensile strength fub = 1000.0 N/mm²
bolt size M24
large wrench size (high strength bolt), preloaded (for info: preloading Fp,c = 0.7·fy·A = 222.4 kN)
calculation parameters:
  shaft diameter d = 24.0 mm, clearance ∆d = 2.0 mm ⇒ hole diameter d0 = 26.0 mm
gross cross-section area A = 4.524 cm²
tensile stress area Aa = 3.530 cm²
diameter of the bolt head (across flats dimension) da = 41.0 mm
diameter of the bolt head (across points dimension) da = 45.20 mm
thickness of the bolt head tk = 15.0 mm
thickness of nut tm = 20.0 mm
diameter of the plate under the bolt or the nut dp = 44.0 mm
thickness of the plate under the bolt or the nut tp = 4.0 mm
washer double-sided
shear plane passes through the unthreaded portion of the bolt

beam parameters

section HE400A
design values:
  overall depth h = 390.0 mm, web thickness tw = 11.0 mm
flange width br = 300.0 mm, flange thickness tr = 19.0 mm
rolled section, root radius r = 27.0 mm
clear depth of web without root/weld dw = 298.0 mm
width of one flange side without root/weld α = 117.5 mm
centroid distance from top zα = 195.0 mm
cross-sectional area A = 158.98 cm²
second moment of area Iy = 45067.79 cm⁴
plastic section modulus Wpl,y = 2562.00 cm³
elastic section modulus We,ly = ly/zα = 2311.17 cm³
effective shear area Aeq = 57.33 cm²
torsional moment of inertia Iτ = 190.00 cm⁴

verification parameters

bolted end-plate connection:
thickness tp = 25.0 mm, width bp = 300.0 mm, length lp = 495.0 mm
projections hρ,o = 85.0 mm, hρ,u = 20.0 mm
bolts in connection:
  3 bolt-rows with 2 bolts
  all bolt-rows considered individually
tensile edge top: group of bolts with 2 rows (1-2)
tensile edge bottom: no group of bolts
  all bolt-rows for shear transfer (rows 1-3)
centre distance of the bolts to the lateral edge of the end-plate e2 = 75.0 mm
centre distance of the first bolt-row to the upper edge of the end-plate (end row) eα = 35.0 mm
centre distance of the last bolt-row to the bottom edge of the end-plate (end row) eu = 90.0 mm
centre distance of the first bolt-row to the free edge of the column (end row) e1 = 200.0 mm
centre distance of the bolt-rows from each other p1-2 = 120.0 mm, p2-3 = 250.0 mm
welds at the connection point:
  beam flange top: fillet weld, weld thickness a = 8.0 mm
  beam web: fillet weld, weld thickness a = 3.0 mm
  beam flange bottom: fillet weld, weld thickness a = 8.0 mm

internal forces and moments at the joint periphery referring to the system axes

Lk 1:  \( M_{N0,Ed} = 200.0 \text{ kNm} \quad V_{N0,Ed} = 270.0 \text{ kN} \)

partial safety factors for material
resistance of cross-sections γM0 = 1.00
resistance of members in stability failure γM1 = 1.10
resistance of bolts, welds, plates in bearing γM2 = 1.25
prestressng of high strength bolts γM7 = 1.10
check of data
ok
distances between bolt-rows at end-plate
horizontal: \( e_2 = 75.0 \text{ mm} < 1.2 \times d_0 = 31.2 \text{ mm} \), \( e_2 = 75.0 \text{ mm} < 4 t + 40 \text{ mm} = 116.0 \text{ mm} \)
horizontal: \( p_2 = 150.0 \text{ mm} > 2.4 \times d_0 = 62.4 \text{ mm} \), \( p_2 = 150.0 \text{ mm} < \min(14t, 200 \text{ mm}) = 200.0 \text{ mm} \)
vertical: \( e_1 = 35.0 \text{ mm} > 1.2 \times d_0 = 31.2 \text{ mm} \), \( e_1 = 35.0 \text{ mm} < 4 t + 40 \text{ mm} = 116.0 \text{ mm} \)
vertical: \( e_1 = 200.0 \text{ mm} > 1.2 \times d_0 = 31.2 \text{ mm} \), \( e_1 = 200.0 \text{ mm} > 4 t + 40 \text{ mm} = 116.0 \text{ mm} \)
vertical: \( p_1 = 120.0 \text{ mm} < 2.2 \times d_0 = 57.2 \text{ mm} \), \( p_1 = 120.0 \text{ mm} < \min(14t, 200 \text{ mm}) = 200.0 \text{ mm} \)
vertical: \( p_1 = 250.0 \text{ mm} > 2.2 \times d_0 = 57.2 \text{ mm} \), \( p_1 = 250.0 \text{ mm} > \min(14t, 200 \text{ mm}) = 200.0 \text{ mm} \)
vertical: \( e_1 = 90.0 \text{ mm} > 1.2 \times d_0 = 31.2 \text{ mm} \), \( e_1 = 90.0 \text{ mm} < 4 t + 40 \text{ mm} = 116.0 \text{ mm} \)

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

notes
no verification for cross-sections.
no verification for welds within the connection.

2. Lk 1
notes
the following verification is applied to the connection of a girder to a continuous column.
to dimension a frame corner completely, further verifications are required.
connection is verified due to EC 3-1-8 regardless of preloading.
however, connections may be constructed with prestressed high strength bolts.
calculation of T-stub-resistance with the standard method.

2.1. design values

periphery connection parallel to connection plane partial internal forces and moments

sign definition of EC3: a positive axial force means compression, a positive bending moment produces tension at the top

slope angle: \( \alpha = \alpha_v = 0^\circ \)
distance: \( e_1 = 150.0 \text{ mm} \), \( e_3 = 185.5 \text{ mm} \), \( e_2 = 185.5 \text{ mm} \)

transformation forces joint values -> design values
\( M_d = 200.00 \text{ kNm} \), \( V_d = 270.00 \text{ kN} \)

internal forces and moments perpendicular to the connection planes
periphery beam
\( M_d = 200.00 \text{ kNm} \), \( V_d = 270.00 \text{ kN} \)
calculation of internal forces and moments at periphery column (top)
\( N_d = N_c + V_d e_3 \cdot M_d - V_d e_1 - N_d (e_6 - e_3) = -240.50 \text{ kNm} \)

partial internal forces and moments
internal forces and moments in the periphery end-plate-beam: \( M_d = M_d - V_d e_p = 193.25 \text{ kNm} \)
\( N_{b,t} = -N_a z_b / z_b + M_d z_b = 520.89 \text{ kN} \), \( z_b = 371.0 \text{ mm} \), \( z_{bb} = 185.5 \text{ mm} \)
\( N_{b,c} = N_d z_b / z_b + M_d z_b = 520.89 \text{ kN} \), \( z_b = 371.0 \text{ mm} \), \( z_{bc} = 185.5 \text{ mm} \)

2.2. basic components
end-plate joint: basic components: 1, 2, 3, 4, 5, 7, 8, 10, 11, 12

2.2.1. Gk 1: Column web panel in shear

transformation parameter (EC 3-1-8, 5.3(9)) \( \beta_1 = 1.00 \) for \( M_d = 200.00 \text{ kNm} \) (\( M_d = 0 \))
assumption
slenderness of column web \( d_c/t_{wc} = 18.91 < 69.5 = 69.00, \ c = 1.00 \Rightarrow \) method applicable

shear area
shear area \( A_v = 47.43 \text{ cm}^2 \)

plastic shear resistance
plastic shear resistance \( V_{wp,Rd} = (0.9 f_{y,w} A_v) / (3^{1/2} \gamma M_0) = 579.1 \text{ kN} \)

2.2.2. Gk 2: column web in transverse compression

transformation parameter (EC 3-1-8, 5.3(9)) \( \beta_l = 1.00 \) for \( M_{11} = 200.00 \text{ kNm} \) \( M_{22} = 0 \)

effective width
effective width of column web in transverse compression \( b_{eff,c} = t_l b + 2.21^{1/2} a_p + 5(t_l,c + s_c) + s_p = 305.3 \text{ mm}, \ s_p = 33.7 \text{ mm} \)

longitudinal compressive stress in web
reduction factor \( k_w = 1.0 \) \((\sigma_{nom,Ed} = 0)\)

plate buckling
plate slenderness \( \lambda_p = 0.932 \left( \frac{(b_{eff,c} d_w f_y)}{E t_w^2} \right)^{1/2} = 0.714, \ E = 210000 \text{ N/mm}^2 \)
reduction factor for web buckling \( p = 1 \)

shear area
shear area \( A_v = 47.43 \text{ cm}^2 \)

interaction with shear stress
reduction factor for interaction with shear stress \( \beta_l = 1 \Rightarrow \omega = \omega_1 = 0.778 \)
with \( \omega_1 = 1 / [1 + 1.3 (b_{eff,tw} A_v / \lambda_{M0})^{1/2} = 0.778 \)

resistance of an unstiffened web in transverse compression
\[ F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_l \cdot f_y) / \gamma_{M0} = 614.07 \text{ kN}, \ f_y = 235.0 \text{ N/mm}^2 \]
\[ F_{c,w,Rd} = \omega \cdot (k_w p \cdot b_{eff,c} \cdot t_l \cdot f_y) / \gamma_{M1} = 558.25 \text{ kN} \text{ (decisive)} \]

resistance of upper beam flange:
effective width
effective width of column web in transverse compression \( b_{eff,c} = t_l b + 2.21^{1/2} a_p + 5(t_l,c + s_c) + s_p = 321.6 \text{ mm}, \ s_p = 50.0 \text{ mm} \)

longitudinal compressive stress in web
reduction factor \( k_w = 1.0 \) \((\sigma_{nom,Ed} = 0)\)

plate buckling
plate slenderness \( \lambda_p = 0.932 \left( \frac{(b_{eff,c} d_w f_y)}{E t_w^2} \right)^{1/2} = 0.733, \ E = 210000 \text{ N/mm}^2 \)
reduction factor for web buckling \( p = (\lambda_p - 0.2) / \lambda_p^2 = 0.992 \)

shear area
shear area \( A_v = 47.43 \text{ cm}^2 \)

interaction with shear stress
reduction factor for interaction with shear stress \( \beta_l = 1 \Rightarrow \omega = \omega_1 = 0.762 \)
with \( \omega_1 = 1 / [1 + 1.3 (b_{eff,tw} A_v / \lambda_{M0})^{1/2} = 0.762 \)

resistance of an unstiffened web in transverse compression
\[ F_{c,w,Rd} = \omega \cdot (k_w \cdot b_{eff,c} \cdot t_l \cdot f_y) / \gamma_{M0} = 633.92 \text{ kN}, \ f_y = 235.0 \text{ N/mm}^2 \]
\[ F_{c,w,Rd} = \omega \cdot (k_w p \cdot b_{eff,c} \cdot t_l \cdot f_y) / \gamma_{M1} = 571.11 \text{ kN} \text{ (decisive)} \]
2.2.3. Gk 4: column flange in bending

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

equivalent T-stub flange (each individual bolt-row):
- distance centre-line of the bolt to the edge of flange e = 75.0 mm
- distance centre-line of the bolt to the stub web m = 47.9 mm

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

**row 1**

- (other) end bolt-row
  - \( \text{left,cp,a} = \min(2 \times m, \pi \times m + 2 \times e_1) = 301.0 \text{ mm} \)
  - \( \text{left,nc,a} = \min(4 \times m + 1.25 \times e, 2 \times m + 0.625 \times e + e_1) = 285.4 \text{ mm} \)
- in mode 1: \( \Sigma \text{left,1} = \text{left,1} = \min(\text{left,nc, left,cp}) = 285.4 \text{ mm} \)
- in mode 2: \( \Sigma \text{left,2} = \text{left,2} = \text{left,nc} = 285.4 \text{ mm} \)

**tension resistance of the T-stub flange**

- \( n = \min(\text{emin,1,25-m}) = 59.9 \text{ mm, } \epsilon_{\text{emin}} = 75.0 \text{ mm, } m = 47.9 \text{ mm} \)
- resisting plastic moments:
  - in mode 1-2: \( M_{\text{pl,Rd}} = (0.25 \times \Sigma \text{left} \times t^2 \times f_y) / \gamma_{M0} = 6.05 \text{ kNm, } t_r = 19.0 \text{ mm, } f_y = 235.0 \text{ N/mm}^2, \gamma_{M0} = 1.00 \)
  - force value of tension resistance:
    - tension resistance of one bolt:
      - \( F_{\text{T,Rd}} = (k_2 \times \text{ub} \times A_s) / \gamma_{M2} = 254.16 \text{ kN, } k_2 = 0.90 \)
  - in mode 3:
    - \( \Sigma F_{\text{T,Rd}} = 2 \times n \times F_{\text{T,Rd}} = 508.32 \text{ kN, } n = 1 \)
  - prying forces always appear at preloaded bolts
  - calculation with the standard method
    - mode 1: complete yielding of the T-stub flange
      - \( F_{\text{T,1,Rd}} = (4 \times M_{\text{pl,1,Rd}}) / m = 505.38 \text{ kN} \)
    - mode 2: bolt failure simultaneously with yielding of the T-stub flange
      - \( F_{\text{T,2,Rd}} = (2 \times M_{\text{pl,2,Rd}} + n \times \Sigma F_{\text{T,Rd}}) / (m+n) = 394.71 \text{ kN} \)
    - mode 3: bolt failure
      - \( F_{\text{T,3,Rd}} = \Sigma F_{\text{T,Rd}} = 508.32 \text{ kN} \)
  - tension resistance of the T-stub flange:
    - \( F_{\text{T,Rd}} = \min(F_{\text{T,1,Rd}}, F_{\text{T,2,Rd}}, F_{\text{T,3,Rd}}) = 394.71 \text{ kN} \)

**row 2**

- distance centre-line of the bolt to the edge of flange e = 75.0 mm
- distance centre-line of the bolt to the stub web m = 47.9 mm

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

**row 2**

- (other) end bolt-row
  - \( \text{left,cp,a} = 2 \times \pi \times m = 301.0 \text{ mm (e_1 = } \infty) \)
  - \( \text{left,nc,a} = 4 \times m + 1.25 \times e = 285.4 \text{ mm (e_1 = } \infty) \)
- in mode 1: \( \Sigma \text{left,1} = \text{left,1} = \min(\text{left,nc, left,cp}) = 285.4 \text{ mm} \)
- in mode 2: \( \Sigma \text{left,2} = \text{left,2} = \text{left,nc} = 285.4 \text{ mm} \)

**tension resistance of the T-stub flange**

- \( n = \min(\text{emin,1,25-m}) = 59.9 \text{ mm, } \epsilon_{\text{emin}} = 75.0 \text{ mm, } m = 47.9 \text{ mm} \)
- resisting plastic moments:
  - in mode 1-2: \( M_{\text{pl,Rd}} = (0.25 \times \Sigma \text{left} \times t^2 \times f_y) / \gamma_{M0} = 6.05 \text{ kNm, } t_r = 19.0 \text{ mm, } f_y = 235.0 \text{ N/mm}^2, \gamma_{M0} = 1.00 \)
  - design value of tension resistance:
    - tension resistance of one bolt:
      - \( F_{\text{T,Rd}} = (k_2 \times \text{ub} \times A_s) / \gamma_{M2} = 254.16 \text{ kN, } k_2 = 0.90 \)
  - in mode 3:
    - \( \Sigma F_{\text{T,Rd}} = 2 \times n \times F_{\text{T,Rd}} = 508.32 \text{ kN, } n = 1 \)
  - prying forces always appear at preloaded bolts
  - calculation with the standard method
    - mode 1: complete yielding of the T-stub flange
      - \( F_{\text{T,1,Rd}} = (4 \times M_{\text{pl,1,Rd}}) / m = 505.38 \text{ kN} \)
    - mode 2: bolt failure simultaneously with yielding of the T-stub flange
      - \( F_{\text{T,2,Rd}} = (2 \times M_{\text{pl,2,Rd}} + n \times \Sigma F_{\text{T,Rd}}) / (m+n) = 394.71 \text{ kN} \)
    - mode 3: bolt failure
      - \( F_{\text{T,3,Rd}} = \Sigma F_{\text{T,Rd}} = 508.32 \text{ kN} \)
  - tension resistance of the T-stub flange:
    - \( F_{\text{T,Rd}} = \min(F_{\text{T,1,Rd}}, F_{\text{T,2,Rd}}, F_{\text{T,3,Rd}}) = 394.71 \text{ kN} \)
left, nc, l = 4 m + 1.25 e = 285.4 mm
in mode 1: $\Sigma_{left,1} = left,1 = min(left,nc, left,cp) = 285.4$ mm
in mode 2: $\Sigma_{left,2} = left,2 = left,nc = 285.4$ mm

tension resistance of the T-stub flange
\[ n = min(\varepsilon_{\text{min}}, 1.25 m) = 59.9 \text{ mm}, \quad \varepsilon_{\text{min}} = 75.0 \text{ mm}, \quad m = 47.9 \text{ mm} \]
resisting plastic moments:
\[ \text{in mode 1+2: } M_{pl,Rd} = \left( 0.25 \Sigma_{left} t^2 f_y \right) / \gamma_{M0} = 6.05 \text{ kNm}, \quad t_s = 19.0 \text{ mm}, \quad f_y = 235.0 \text{ N/mm}^2, \quad \gamma_{M0} = 1.00 \]
design value of tension resistance:
tension resistance of one bolt: $F_{T,Rd} = (k_2 f_{ub} A_s) / \gamma_{M2} = 254.16 \text{ kN}, \quad k_2 = 0.90$
in mode 3: $\Sigma F_{T,Rd} = 2 \eta_b F_{T,Rd} = 508.32 \text{ kN}, \quad \eta_b = 1$
prying forces always appear at preloaded bolts!
calculation with the standard method
mode 1: complete yielding of the T-stub flange
\[ F_{T,1,Rd} = (4 M_{pl,1,Rd}) / m = 505.38 \text{ kN} \]
mode 2: bolt failure simultaneously with yielding of the T-stub flange
\[ F_{T,2,Rd} = \left( 2 M_{pl,2,Rd} + n \Sigma F_{T,Rd} \right) / (m+n) = 394.71 \text{ kN} \]
mode 3: bolt failure
\[ F_{T,3,Rd} = \Sigma F_{T,Rd} = 508.32 \text{ kN} \]
tension resistance of the T-stub flange: $F_{T,Rd} = min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 394.71 \text{ kN}$

resistances and effective lengths of column flange in bending (per bolt-row)
\[ F_{T,\text{fc,Rd,1}} = 394.71 \text{ kN}, \quad left,1 = 285.4 \text{ mm} \]
\[ F_{T,\text{fc,Rd,2}} = 394.71 \text{ kN}, \quad left,2 = 285.4 \text{ mm} \]
\[ F_{T,\text{fc,Rd,3}} = 394.71 \text{ kN}, \quad left,3 = 285.4 \text{ mm} \]
equivalent T-stub flange (group of bolt-rows):
here: number of bolt-rows $n_b = 2$

row 1
distance centre-line of the bolt to the edge of flange $e = 75.0$ mm
distance centre-line of the bolt to the stub web $m = 47.9$ mm
distance between bolt-rows $p = 120.0$ mm

(Other) end bolt-row
\[ left,cp,a = min(\pi \cdot m+p, 2 \cdot e_1+p) = 270.5 \text{ mm} \]
\[ left,nc,a = min(2 \cdot m+0.625 \cdot e+0.5 \cdot p, e_1+0.5 \cdot p) = 202.7 \text{ mm} \]

row 2
distance centre-line of the bolt to the edge of flange $e = 75.0$ mm
distance centre-line of the bolt to the stub web $m = 47.9$ mm
distance between bolt-rows $p = 120.0$ mm

(Other) end bolt-row
\[ left,cp,a = \pi \cdot m+p = 270.5 \text{ mm} \quad (e_1 = \infty) \]
\[ left,nc,a = 2 \cdot m+0.625 \cdot e+0.5 \cdot p = 202.7 \text{ mm} \quad (e_1 = \infty) \]
effective length of the T-stub flange (column flange)
in mode 1: $\Sigma left,1 = min(\Sigma left,nc, left,cp) = 405.4 \text{ mm}, \quad left,cp = 541.0 \text{ mm}$
in mode 2: $\Sigma left,2 = left,nc = 405.4 \text{ mm}$
tension resistance of the T-stub flange
\[ n = min(\varepsilon_{\text{min}}, 1.25 m) = 59.9 \text{ mm}, \quad \varepsilon_{\text{min}} = 75.0 \text{ mm}, \quad m = 47.9 \text{ mm} \]
resisting plastic moments:
\[ \text{in mode 1+2: } M_{pl,Rd} = \left( 0.25 \Sigma_{left} t^2 f_y \right) / \gamma_{M0} = 8.60 \text{ kNm}, \quad t_s = 19.0 \text{ mm}, \quad f_y = 235.0 \text{ N/mm}^2, \quad \gamma_{M0} = 1.00 \]
design value of tension resistance:
tension resistance of one bolt: $F_{T,Rd} = (k_2 f_{ub} A_s) / \gamma_{M2} = 254.16 \text{ kN}, \quad k_2 = 0.90$
in mode 3: $\Sigma F_{T,Rd} = 2 \eta_b F_{T,Rd} = 1016.64 \text{ kN}, \quad \eta_b = 2$
prying forces always appear at preloaded bolts!
calculation with the standard method
mode 1: complete yielding of the T-stub flange
\[ F_{T,1,Rd} = (4 M_{pl,1,Rd}) / m = 717.91 \text{ kN} \]
mode 2: bolt failure simultaneously with yielding of the T-stub flange
\[ F_{T,2,Rd} = \left( 2 M_{pl,2,Rd} + n \Sigma F_{T,Rd} \right) / (m+n) = 724.34 \text{ kN} \]
mode 3: bolt failure
\[ F_{T,3,Rd} = \Sigma F_{T,Rd} = 1016.64 \text{ kN} \]
tension resistance of the T-stub flange: $F_{T,Rd} = min(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 717.91 \text{ kN}$
effective length: $left = 405.4 \text{ mm}, \quad 2 \text{ rows}$

2.2.4. Gk 3: column web in transverse tension

transformation parameter (EC 3-1-8, 5.3(9)) $\beta_1 = 1.00$ for $M_{pl} = 200.00 \text{ kNm}$ ($M_2 = 0$)
each individual bolt-row:
row 1
effective width
effective width of column web in transverse tension $b_{eff,t} = 285.4$ mm (left from bc 4)
shear area
shear area $A_v = 47.43$ cm$^2$
interaction with shear stress
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \phi = \phi_1 = 0.798$
with $\phi_1 = 1 / [1 + 1.3 \cdot (b_{eff,t}/A_v)^2]^{1/2} = 0.798$
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \phi \cdot (b_{eff,t} \cdot f_{yw,wc}) / \gamma_M = 588.8$ kN, $f_{yw,wc} = 235.0$ N/mm$^2$
row 2
effective width
effective width of column web in transverse tension $b_{eff,t} = 285.4$ mm (left from bc 4)
shear area
shear area $A_v = 47.43$ cm$^2$
interaction with shear stress
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \phi = \phi_1 = 0.798$
with $\phi_1 = 1 / [1 + 1.3 \cdot (b_{eff,t}/A_v)^2]^{1/2} = 0.798$
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \phi \cdot (b_{eff,t} \cdot f_{yw,wc}) / \gamma_M = 588.8$ kN, $f_{yw,wc} = 235.0$ N/mm$^2$
row 3
effective width
effective width of column web in transverse tension $b_{eff,t} = 285.4$ mm (left from bc 4)
shear area
shear area $A_v = 47.43$ cm$^2$
interaction with shear stress
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \phi = \phi_1 = 0.798$
with $\phi_1 = 1 / [1 + 1.3 \cdot (b_{eff,t}/A_v)^2]^{1/2} = 0.798$
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \phi \cdot (b_{eff,t} \cdot f_{yw,wc}) / \gamma_M = 588.8$ kN, $f_{yw,wc} = 235.0$ N/mm$^2$
each group of bolt-rows:
effective width
effective width of column web in transverse tension $b_{eff,t} = 405.4$ mm (left from bc 4)
shear area
shear area $A_v = 47.43$ cm$^2$
interaction with shear stress
reduction factor for interaction with shear stress $\beta = 1 \Rightarrow \phi = \phi_1 = 0.682$
with $\phi_1 = 1 / [1 + 1.3 \cdot (b_{eff,t}/A_v)^2]^{1/2} = 0.682$
resistance of a column web with transverse tension
$F_{t,wc,Rd} = \phi \cdot (b_{eff,t} \cdot f_{yw,wc}) / \gamma_M = 714.8$ kN, $f_{yw,wc} = 235.0$ N/mm$^2$

2.2.5. Gk 5: end-plate in bending

extended part of end-plate
in the extended part of the end-plate only one bolt-row is considered ($n_0 = 1$).
distance centre-line of the bolt to beam flange $m_1 = 40.9$ mm
effective length of the T-stub flange (end-plate)

\[ e_x = e - 35.0 \text{ mm}, \quad m = m - 40.9 \text{ mm}, \quad w = b = 2e - 150.0 \text{ mm} \text{ with } b_0 = 300.0 \text{ mm}, \quad e = 75.0 \text{ mm} \]

end-bolt-row outside tension flange of beam

\[ \text{left,cp,sa} = \min(2\pi m, \pi mx + w, \pi mx + 2 e) = 257.3 \text{ mm} \]

\[ \text{left,nc,sa} = \min(4m + 1.25e_x, e + 2m + 0.625e_x, 0.5b, 0.5w + 2m + 0.625e_x) = 150.0 \text{ mm} \]

in mode 1: \( 1 = \text{left,1} = \min(\text{left,nc, left,cp}) = 150.0 \text{ mm} \)

in mode 2: \( 2 = \text{left,2} = \text{left,nc} = 150.0 \text{ mm} \)

**tension resistance of the T-stub flange**

\[ n = \min(e_{\min}, 1.25m) = 35.0 \text{ mm}, \quad e_{\min} = 35.0 \text{ mm}, \quad m = 40.9 \text{ mm} \]

resisting plastic moments:

- in mode 1: \( M_{pl,Rd} = (0.25\Sigma \text{left} t^2 I_y) / \gamma_m = 5.51 \text{ kNm} \)
- in mode 2: \( f_t = 250.0 \text{ mm}, \quad I_y = 235.0 \text{ N/mm}^2, \quad \gamma_m = 1.00 \)

design value of tension resistance:

- tension resistance of one bolt: \( f_t, Rd = (k_2 f_u, A_s) / \gamma_m = 254.16 \text{ kN} \)
- in mode 3: \( 2 = \Sigma f_t, Rd = 508.32 \text{ kN} \)

prying forces always appear at preloaded bolts!

calculation with the standard method

- mode 1: complete yielding of the T-stub flange

\[ F_{t,1,Rd} = (4M_{pl,1,Rd}) / m = 538.02 \text{ kN} \]

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

\[ F_{t,2,Rd} = (2 M_{pl,2,Rd} + n \Sigma f_t, Rd) / (m + n) = 379.29 \text{ kN} \]

- mode 3: bolt failure

\[ F_{t,3,Rd} = \Sigma f_t, Rd = 508.32 \text{ kN} \]

**tension resistance of the T-stub flange**

\[ F_{t,Rd} = \min(F_{t,1,Rd}, F_{t,2,Rd}, F_{t,3,Rd}) = 379.29 \text{ kN} \]

resistance of a weld (req. 1):

\[ f_{w,d} = f_w / (\beta_w \gamma_m) = 360.0 \text{ N/mm}^2, \quad f_w = 360.0 \text{ N/mm}^2, \quad \beta_w = 0.80 \]

tension resistance of welds:

\[ F_{t,w,Rd} = 2^{1/2} f_{w,d} a_{left} = 610.94 \text{ kN} \]

**resistance and effective length of end-plate in bending (projection)**

\[ F_{t,sp,Rd,1} = 379.29 \text{ kN}, \quad \text{left,1} = 150.0 \text{ mm} \]

**part of end-plate between beam flanges**

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

- number of bolt-rows \( n_0 = 1 \)

**row 2**

distance centre-line of the bolt to the stiffener \( m_2 = 41.9 \text{ mm} \)

distance centre-line of the bolt to the edge of flange \( e = 75.0 \text{ mm} \)

distance centre-line of the bolt to the stub web \( m = 66.1 \text{ mm} \)

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

inner bolt-row outside tension flange of beam

- coefficient for stiffened column flanges and end-plates:

\[ \begin{align*}
  \lambda_1 &= m / (m + e) = 0.468, \\
  \lambda_2 &= e / (m + e) = 0.113 \\
  \alpha &= (\gamma_m = 1.00) \\
  \left(cp, sa\right) &= 2 \pi m = 415.4 \text{ mm} \\
  \left(nc, sa\right) &= \alpha m = 438.6 \text{ mm} \\
\end{align*} \]

in mode 1: \( 1 = \text{left,1} = \min(\text{left,nc, left,cp}) = 415.4 \text{ mm} \)

in mode 2: \( 2 = \text{left,2} = \text{left,nc} = 438.6 \text{ mm} \)

**tension resistance of the T-stub flange**

\[ n = \min(e_{\min}, 1.25m) = 75.0 \text{ mm}, \quad e_{\min} = 75.0 \text{ mm}, \quad m = 66.1 \text{ mm} \]

resisting plastic moments:

- in mode 1: \( M_{pl,1,Rd} = (0.25\Sigma \text{left} t^2 I_y) / \gamma_m = 15.25 \text{ kNm} \)
- in mode 2: \( M_{pl,2,Rd} = (0.25\Sigma \text{left} t^2 I_y) / \gamma_m = 16.10 \text{ kNm} \)

- in mode 3: \( \Sigma f_t, Rd = 2 n_0 f_t, Rd = 508.32 \text{ kN} \)

prying forces always appear at preloaded bolts!

calculation with the standard method

- mode 1: complete yielding of the T-stub flange

\[ F_{t,1,Rd} = (4M_{pl,1,Rd}) / m = 922.84 \text{ kN} \]

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

\[ F_{t,2,Rd} = (2 M_{pl,2,Rd} + n \Sigma f_t, Rd) / (m + n) = 498.42 \text{ kN} \]

- mode 3: bolt failure

\[ F_{t,3,Rd} = \Sigma f_t, Rd = 508.32 \text{ kN} \]

**tension resistance of the T-stub flange**

\[ F_{t,Rd} = \min(F_{t,1,Rd}, F_{t,2,Rd}, F_{t,3,Rd}) = 498.42 \text{ kN} \]

resistance of a weld (req. 1):

\[ f_{w,d} = f_w / (\beta_w \gamma_m) = 360.0 \text{ N/mm}^2, \quad f_w = 360.0 \text{ N/mm}^2, \quad \beta_w = 0.80 \]

tension resistance of welds:

\[ F_{t,w,Rd} = 2^{1/2} f_{w,d} a_{left} = 634.39 \text{ kN} \]

**row 3**

distance centre-line of the bolt to the stiffener \( m_2 = 41.9 \text{ mm} \)

distance centre-line of the bolt to the edge of flange \( e = 75.0 \text{ mm} \)

distance centre-line of the bolt to the stub web \( m = 66.1 \text{ mm} \)

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

inner bolt-row outside tension flange of beam

- coefficient for stiffened column flanges and end-plates:

\[ \begin{align*}
  \lambda_1 &= m / (m + e) = 0.468, \\
  \lambda_2 &= e / (m + e) = 0.297 \\
  \alpha &= (\gamma_m = 1.00) \\
  \left(cp, sa\right) &= 2 \pi m = 415.4 \text{ mm} \\
  \left(nc, sa\right) &= \alpha m = 438.6 \text{ mm} \\
\end{align*} \]

in mode 1: \( 1 = \text{left,1} = \min(\text{left,nc, left,cp}) = 415.4 \text{ mm} \)

in mode 2: \( 2 = \text{left,2} = \text{left,nc} = 438.6 \text{ mm} \)
tension resistance of the T-stub flange
\[ n = \text{min}(\alpha_{\text{min}}, 1.25 \cdot m) = 75.0 \text{ mm}, \quad \alpha_{\text{min}} = 75.0 \text{ mm}, \quad m = 66.1 \text{ mm} \]
resisting plastic forces:
in mode 1: \[ M_{\text{pl},1,Rd} = (0.25 \cdot S_{\text{eff},1} \cdot t^2 \cdot f_y) / \gamma_{M0} = 15.25 \text{ kNm}, \quad t = 25.0 \text{ mm}, \quad f_y = 235.0 \text{ N/mm}^2, \quad \gamma_{M0} = 1.00 \]
in mode 2: \[ M_{\text{pl},2,Rd} = (0.25 \cdot S_{\text{eff},2} \cdot t^2 \cdot f_y) / \gamma_{M0} = 16.10 \text{ kNm} \]
design value of tension resistance:
tension resistance of one bolt: \[ F_{T,\text{Rd}} = (k_2 \cdot f_u \cdot A_s) / \gamma_{M2} = 254.16 \text{ kN}, \quad k_2 = 0.90 \]
in mode 3: \[ S_{\text{eff},3,Rd} = 2 \cdot n_{\text{B}} \cdot F_{T,\text{Rd}} = 508.32 \text{ kN}, \quad n_{\text{B}} = 1 \]
prying forces always appear at preloaded bolts!
calculation with the standard method
mode 1: complete yielding of the T-stub flange
\[ F_{T,1,Rd} = (4 \cdot M_{\text{pl},1,Rd}) / m = 922.84 \text{ kN} \]
mode 2: bolt failure simultaneously with yielding of the T-stub flange
\[ F_{T,2,Rd} = (2 \cdot M_{\text{pl},2,Rd} + n \cdot S_{\text{eff},3,Rd}) / (m + n) = 498.42 \text{ kN} \]
mode 3: bolt failure
\[ F_{T,3,Rd} = S_{\text{eff},3,Rd} = 508.32 \text{ kN} \]
tension resistance of the T-stub flange: \[ F_{T,Rd} = \text{min}(F_{T,1,Rd}, F_{T,2,Rd}, F_{T,3,Rd}) = 498.42 \text{ kN} \]
resistance of a weld (req.1): \[ f_{w,d} = f_u / (\beta_w \cdot \gamma_{M2}) = 360.0 \text{ N/mm}^2, \quad f_u = 360.0 \text{ N/mm}^2, \quad \beta_w = 0.80 \]
tension resistance of welds: \[ F_{T,w,Rd} = 2^{1/2} \cdot f_{w,d} \cdot A_{\text{eff}} = 634.39 \text{ kN} (\geq 498.42 \text{ kN}, \text{not decisive}) \]
resistances and effective lengths of end-plate in bending (per bolt-row):
\[ F_{E_{p,Rd},2} = 498.42 \text{ kN}, \quad F_{E_{p,Rd},2} = 415.4 \text{ mm} \]
\[ F_{E_{p,Rd},3} = 498.42 \text{ kN}, \quad F_{E_{p,Rd},3} = 415.4 \text{ mm} \]

2.2.6. Gk 7: beam flange and web in compression

section class of beam (\(c = 1.00\)):
flange bottom: section class for \(c/\gamma_{c} = 6.18\) (outstand flange): 1
web: section class for \(c = 0.50\) and \(c/\gamma_{c} = 27.09\) (internal compression parts, bending): 1
total: section class: 1
taking into account the moment-shear force-interaction \(V_{Ed} = 270.0 \text{ kN}\)
plastic section modulus \(W_{pl} = 2582.000 \text{ cm}^3\)

2.2.7. Gk 8: beam web in tension
each individual bolt-row:
row 2
  effective width
  effective width of the beam web in tension $b_{eff,t,wb} = 415.4$ mm (left from bc 5)
  resistance of a beam web in tension
  $F_{t,wb,Rd} = b_{eff,t,wb} \cdot f_{y,wb} / \gamma_M = 1073.7$ kN, $f_{y,wb} = 235.0$ N/mm²
row 3
  effective width
  effective width of the beam web in tension $b_{eff,t,wb} = 415.4$ mm (left from bc 5)
  resistance of a beam web in tension
  $F_{t,wb,Rd} = b_{eff,t,wb} \cdot f_{y,wb} / \gamma_M = 1073.7$ kN, $f_{y,wb} = 235.0$ N/mm²

2.2.8. Gk 10: bolts in tension

Bolt category D:
tension resistance of one bolt: $F_{t,Rd} = (k_2 f_{ub} A_d) / \gamma_M = 254.16$ kN, $k_2 = 0.90$, $f_{ub} = 1000.0$ N/mm²
p. sh. load capacity: $B_{p,Rd} = (0.6 \cdot d \cdot d f_{ub}) / \gamma_M = 444.55$ kN, $d = 43.1$ mm, $f_{ub} = 19.0$ mm, $f_{ub} = 360.0$ N/mm²
tension-punching shear load capacity for 2 bolts: $\Sigma F_{p,Rd} = 2 \cdot \min (F_{t,Rd}, B_{p,Rd}) = 508.32$ kN

2.2.9. Gk 11: bolts in shear

Bolt category A:
shear plane passes through the unthreaded portion of the bolt: $\alpha_v = 0.6$, $A = 4.52$ cm²
shear resistance per shear plane: $F_{v,Rd} = \alpha_v f_{ub} A / \gamma_M = 217.15$ kN, $f_{ub} = 1000.0$ N/mm²
shear resistance of 2 bolts (1-shear): $\Sigma F_{v,Rd} = 2 \cdot F_{v,Rd} = 434.29$ kN

2.2.10. Gk 12: plate with bearing resistance
row 1
end-plate (for $V_b \geq 0$):

bolt 1:
in direction of load transfer: $a_d, a = e_1/(3d_0) = 0.45$ (end bolt)

\[ a_b = 0.45 \] (smallest value of $a_d$ or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,1} = 1.4-p_2/d_o-1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,1} = \min(2.8-e_2/d_o-1.7, 1.4-p_2/d_o-1.7) = 6.38$ (end bolt)

\[ k_1 = 2.5 \] (smallest value of $k_1$ or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot x_{ib} \cdot f_{ub} \cdot d_1) / \gamma_M = 193.85$ kN, $f_u = 360.0$ N/mm$^2$, $t = 25.0$ mm, $d = 24.0$ mm

bolt 2:
in direction of load transfer: $a_d, a = e_1/(3d_0) = 0.45$ (end bolt)

\[ a_b = 0.45 \] (smallest value of $a_d$ or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,1} = 1.4-p_2/d_o-1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,1} = \min(2.8-e_2/d_o-1.7, 1.4-p_2/d_o-1.7) = 6.38$ (end bolt)

\[ k_1 = 2.5 \] (smallest value of $k_1$ or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot x_{ib} \cdot f_{ub} \cdot d_1) / \gamma_M = 328.32$ kN, $f_u = 360.0$ N/mm$^2$, $t = 19.0$ mm, $d = 24.0$ mm

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 387.69$ kN

column flange (for $V_b \geq 0$):

bolt 1:
in direction of load transfer: $a_d, a = 1.00$

across to the direction of load transfer: $k_{1,1} = 1.4-p_2/d_o-1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,1} = \min(2.8-e_2/d_o-1.7, 1.4-p_2/d_o-1.7) = 6.38$ (end bolt)

\[ k_1 = 2.5 \] (smallest value of $k_1$ or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot x_{ib} \cdot f_{ub} \cdot d_1) / \gamma_M = 328.32$ kN, $f_u = 360.0$ N/mm$^2$, $t = 19.0$ mm, $d = 24.0$ mm

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 656.64$ kN

row 2
end-plate (for $V_b = 0$):

bolt 1:
in direction of load transfer: $a_d, a = 1.9/(3d_0)-1/4 = 1.29$ (inner bolt)

\[ a_b = 1.00 \] (smallest value of $a_d$ or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,1} = 1.4-p_2/d_o-1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,1} = \min(2.8-e_2/d_o-1.7, 1.4-p_2/d_o-1.7) = 6.38$ (end bolt)

\[ k_1 = 2.5 \] (smallest value of $k_1$ or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot x_{ib} \cdot f_{ub} \cdot d_1) / \gamma_M = 432.00$ kN, $f_u = 360.0$ N/mm$^2$, $t = 25.0$ mm, $d = 24.0$ mm

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 864.00$ kN

column flange (for $V_b = 0$):

bolt 1:
in direction of load transfer: $a_d, a = 1.9/(3d_0)-1/4 = 2.96$ (inner bolt)

\[ a_b = 1.00 \] (smallest value of $a_d$ or $f_{ub}/f_u = 2.78$ or 1.0)

across to the direction of load transfer: $k_{1,1} = 1.4-p_2/d_o-1.7 = 6.38$ (inner bolt)

across to the direction of load transfer: $k_{1,1} = \min(2.8-e_2/d_o-1.7, 1.4-p_2/d_o-1.7) = 6.38$ (end bolt)

\[ k_1 = 2.5 \] (smallest value of $k_1$ or 2.5)

bearing resistance: $F_{b,Rd} = (k_1 \cdot x_{ib} \cdot f_{ub} \cdot d_1) / \gamma_M = 328.32$ kN, $f_u = 360.0$ N/mm$^2$, $t = 19.0$ mm, $d = 24.0$ mm

bearing resistance of 1x2 bolts: $\Sigma F_{b,Rd} = 656.64$ kN
across to the direction of load transfer: \( k_{1,i} = 1.4 - p_2/d_2 - 1.7 = 6.38 \) (inner bolt)
across to the direction of load transfer: \( k_{1,a} = \min(2.8 - e_2/d_2 - 1.7, 1.4 - p_2/d_2 - 1.7) = 6.38 \) (end bolt)
\( k_1 = 2.50 \) (smallest value of \( k_1 \) or 2.5)
bearing resistance: \( F_{b,rd} = (k_1 \alpha_b f_u d_1) / \gamma_{M2} = 328.32 \text{ kN}, f_u = 360.0 \text{ N/mm}^2, t = 19.0 \text{ mm}, d = 24.0 \text{ mm} 
\) bearing resistance of 1x2 bolts: \( \Sigma F_{b,rd} = 656.64 \text{ kN} 
row 3 
end-plate (for \( v_0 > 0 \)):
bolt 1:
in direction of load transfer: \( \alpha_{d,1} = p_1/(3d_0 - 1/4) = 2.96 \) (inner bolt)
\( \alpha_b = 1.00 \) (smallest value of \( \alpha_d \) or \( f_{ab}/f_u = 2.78 \) or 1.0)
across to the direction of load transfer: \( k_{1,i} = 1.4 - p_2/d_2 - 1.7 = 6.38 \) (inner bolt)
across to the direction of load transfer: \( k_{1,a} = \min(2.8 - e_2/d_2 - 1.7, 1.4 - p_2/d_2 - 1.7) = 6.38 \) (end bolt)
\( k_1 = 2.50 \) (smallest value of \( k_1 \) or 2.5)
bearing resistance: \( F_{b,rd} = (k_1 \alpha_b f_u d_1) / \gamma_{M2} = 432.00 \text{ kN}, f_u = 360.0 \text{ N/mm}^2, t = 25.0 \text{ mm}, d = 24.0 \text{ mm} 
\) bolt 2:
in direction of load transfer: \( \alpha_{d,1} = p_1/(3d_0 - 1/4) = 2.96 \) (inner bolt)
\( \alpha_b = 1.00 \) (smallest value of \( \alpha_d \) or \( f_{ab}/f_u = 2.78 \) or 1.0)
across to the direction of load transfer: \( k_{1,i} = 1.4 - p_2/d_2 - 1.7 = 6.38 \) (inner bolt)
across to the direction of load transfer: \( k_{1,a} = \min(2.8 - e_2/d_2 - 1.7, 1.4 - p_2/d_2 - 1.7) = 6.38 \) (end bolt)
\( k_1 = 2.50 \) (smallest value of \( k_1 \) or 2.5)
bearing resistance: \( F_{b,rd} = (k_1 \alpha_b f_u d_1) / \gamma_{M2} = 432.00 \text{ kN}, f_u = 360.0 \text{ N/mm}^2, t = 25.0 \text{ mm}, d = 24.0 \text{ mm} 
\) bearing resistance of 1x2 bolts: \( \Sigma F_{b,rd} = 664.00 \text{ kN} 
column flange (for \( v_0 > 0 \)):
bolt 1:
in direction of load transfer: \( \alpha_b = 1.00 \)
across to the direction of load transfer: \( k_{1,i} = 1.4 - p_2/d_2 - 1.7 = 6.38 \) (inner bolt)
across to the direction of load transfer: \( k_{1,a} = \min(2.8 - e_2/d_2 - 1.7, 1.4 - p_2/d_2 - 1.7) = 6.38 \) (end bolt)
\( k_1 = 2.50 \) (smallest value of \( k_1 \) or 2.5)
bearing resistance: \( F_{b,rd} = (k_1 \alpha_b f_u d_1) / \gamma_{M2} = 328.32 \text{ kN}, f_u = 360.0 \text{ N/mm}^2, t = 19.0 \text{ mm}, d = 24.0 \text{ mm} 
\) bolt 2:
in direction of load transfer: \( \alpha_b = 1.00 \)
across to the direction of load transfer: \( k_{1,i} = 1.4 - p_2/d_2 - 1.7 = 6.38 \) (inner bolt)
across to the direction of load transfer: \( k_{1,a} = \min(2.8 - e_2/d_2 - 1.7, 1.4 - p_2/d_2 - 1.7) = 6.38 \) (end bolt)
\( k_1 = 2.50 \) (smallest value of \( k_1 \) or 2.5)
bearing resistance: \( F_{b,rd} = (k_1 \alpha_b f_u d_1) / \gamma_{M2} = 328.32 \text{ kN}, f_u = 360.0 \text{ N/mm}^2, t = 19.0 \text{ mm}, d = 24.0 \text{ mm} 
\) bearing resistance of 1x2 bolts: \( \Sigma F_{b,rd} = 656.64 \text{ kN} 

2.3. connection capacity

2.3.1. moment resistance

distance of tension-bolt-rows from centre of compression: \( h_1 = 430.5 \text{ mm}, h_2 = 310.5 \text{ mm}, h_3 = 60.5 \text{ mm} 

resistances acc. to EC 3-1-8, 6.2.7.2(6) for bolt-rows considered individually

decisive basic components: 3, 4, 6, 8
row 1: \( F_{tr,rd} = 379.3 \text{ kN} 
row 2: \( F_{tr,rd} = 394.7 \text{ kN} 
row 3: \( F_{tr,rd} = 394.7 \text{ kN} 

deductions acc. to EC 3-1-8, 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 \text{ kN} 
Gk 1: \( \Delta F_{tr,rd} = F_{tr,nc,rd} - \Sigma F_{tr,rd} = 714.8 \text{ kN} \quad F_{tr,rd} = 379.3 \text{ kN} < \Delta F_{tr,rd} = 379.3 \text{ kN} 
Gk 4: \( \Delta F_{tr,rd} = F_{tr,nc,rd} - \Sigma F_{tr,rd} = 338.6 \text{ kN} \quad F_{tr,rd} = 394.7 \text{ kN} < \Delta F_{tr,rd} = 394.7 \text{ kN} 
row 2: \( \Sigma F_{tr,rd} = 379.3 \text{ kN} \) (row 1)
Gk 2: \( \Delta F_{tr,rd} = F_{tr,nc,rd} - \Sigma F_{tr,rd} = 335.5 \text{ kN} \quad F_{tr,rd} = 394.7 \text{ kN} > \Delta F_{tr,rd} = 335.5 \text{ kN} 
Gk 4: \( \Delta F_{tr,rd} = F_{tr,nc,rd} - \Sigma F_{tr,rd} = 338.6 \text{ kN} \quad F_{tr,rd} = 394.7 \text{ kN} < \Delta F_{tr,rd} = 394.7 \text{ kN} 

resistance per bolt-row (tension)

row 1: \( F_{tr,rd} = 379.3 \text{ kN} 
row 2: \( F_{tr,rd} = 394.5 \text{ kN} 
row 3: \( F_{tr,rd} = 394.7 \text{ kN} 
\Sigma F_{tr,rd} = 1109.5 \text{ kN} 

deductions acc. to EC 3-1-8, 6.2.7.2(7)

decisive basic components: 3, 4
Gk 1: \( \Delta F_{tr,rd} = W_{np,rd}/t_1 - \Sigma F_{tr,rd} = 579.1 \text{ kN}, F_{tr,rd} = 379.3 \text{ kN} < \Delta F_{tr,rd} = 379.3 \text{ kN} 
Gk 2: \( \Delta F_{tr,rd} = F_{c,nc,rd} - \Sigma F_{tr,rd} = 558.2 \text{ kN}, F_{tr,rd} = 379.3 \text{ kN} < \Delta F_{tr,rd} = 379.3 \text{ kN} 
Gk 7: \( \Delta F_{tr,rd} = F_{c,nc,rd} - \Sigma F_{tr,rd} = 1622.8 \text{ kN}, F_{tr,rd} = 379.3 \text{ kN} < \Delta F_{tr,rd} = 379.3 \text{ kN} 

row 2: $\Sigma F_{tr,rd} = 379.3 \text{ kN}$ (row 1)
Gk 1: $\Delta F_{tr,rd} = V_{wp,rd}/\beta_1 \cdot \Sigma F_{tr,rd} = 199.8 \text{ kN} \cdot \Delta F_{tr,rd} = 335.5 \text{ kN} > \Delta F_{tr,rd} \Rightarrow F_{tr,rd} = 199.8 \text{ kN}
Gk 2: $\Delta F_{tr,rd} = F_{c,rd} \cdot \Delta F_{tr,rd} = 179.0 \text{ kN} \cdot \Delta F_{tr,rd} = 179.0 \text{ kN} > \Delta F_{tr,rd} \Rightarrow F_{tr,rd} = 179.0 \text{ kN}
Gk 7: $\Delta F_{tr,rd} = F_{c,rd} \cdot \Sigma F_{tr,rd} = 1243.5 \text{ kN} \cdot F_{c,rd} = 179.0 \text{ kN} < \Delta F_{tr,rd} \Rightarrow F_{tr,rd} = 179.0 \text{ kN}
row 3: $\Sigma F_{tr,rd} = 558.2 \text{ kN}$ (rows 1 bis 2)
Gk 1: $\Delta F_{tr,rd} = V_{wp,rd}/\beta_1 \cdot \Sigma F_{tr,rd} = 20.9 \text{ kN} \cdot \Delta F_{tr,rd} = 349.7 \text{ kN} > \Delta F_{tr,rd} \Rightarrow F_{tr,rd} = 20.9 \text{ kN}
Gk 2: $\Delta F_{tr,rd} = F_{c,rd} \cdot \Delta F_{tr,rd} = 0.0 \text{ kN} \cdot \Delta F_{tr,rd} = 20.9 \text{ kN} > \Delta F_{tr,rd} \Rightarrow F_{tr,rd} = 0.0 \text{ kN}
Gk 7: $\Delta F_{tr,rd} = F_{c,rd} \cdot \Sigma F_{tr,rd} = 1046.0 \text{ kN} \cdot F_{c,rd} = 0.0 \text{ kN} < \Delta F_{tr,rd} \Rightarrow F_{tr,rd} = 0.0 \text{ kN}

check acc. to EC 3-1-8, 6.2.7.2(9)
declarative basic component: 10
row 1: $F_{tr,rd} = 379.3 \text{ kN}$ \hspace{1cm} $h_x = 430.5 \text{ mm} \Rightarrow F_{tr,rd} \leq \lim F_{tr,rd} = 482.9 \text{ kN}$, no deduction
row 2: $F_{tr,rd} = 179.0 \text{ kN}$ \hspace{1cm} $h_x = 310.5 \text{ mm} \Rightarrow F_{tr,rd} \leq \lim F_{tr,rd} = 482.9 \text{ kN}$, no deduction

resistance per bolt-row (bending)
row 1: $F_{tr,rd} = 379.3 \text{ kN}$
row 2: $F_{tr,rd} = 179.0 \text{ kN}$
row 3: $F_{tr,rd} = 0.0 \text{ kN}$
$\Sigma F_{tr,rd} = 558.2 \text{ kN}$
potential failure by basic component 2, 3, 4, 5

resistance of flanges (compression)
$\Sigma F_{c,rd} = 579.1 \text{ kN}$

moment resistance regarding the centre of compression
$M_{b,rd} = \Sigma(F_{tr,rd} \cdot h_x) = 218.9 \text{ kNm}$
tension resistance
$N_{j,1,rd} = \Sigma(F_{tr,rd} \cdot h_x) = 1109.5 \text{ kN}$
compression resistance
$N_{j,c,rd} = \Sigma F_{c,rd} = 579.1 \text{ kN}$

2.3.2. shear/bearing resistance
resistance per bolt-row
declarative basic components: 11, 12
row 1: $F_{tr,rd} = 387.7 \text{ kN}$
row 2: $F_{tr,rd} = 434.3 \text{ kN}$
row 3: $F_{tr,rd} = 434.3 \text{ kN}$
deductions depending on tension force (at 100% utilization of moment resistance)
declarative basic component: 10
row 1: $F_{tr,rd} = F_{b} \cdot 387.7 \text{ kN} = 181.1 \text{ kN}$ with $f_{bt} = 1 \cdot F_{tr,rd} / (1.4 \Sigma F_{tr,rd}) = 0.467$
row 2: $F_{tr,rd} = F_{b} \cdot 434.3 \text{ kN} = 325.1 \text{ kN}$ with $f_{bt} = 1 \cdot F_{tr,rd} / (1.4 \Sigma F_{tr,rd}) = 0.749$
row 3: $F_{tr,rd} = F_{b} \cdot 434.3 \text{ kN} = 434.3 \text{ kN}$ with $f_{bt} = 1 \cdot F_{tr,rd} / (1.4 \Sigma F_{tr,rd}) = 1.000$
resistance per bolt-row
row 1: $F_{tr,rd} = 181.1 \text{ kN}$
row 2: $F_{tr,rd} = 325.1 \text{ kN}$
row 3: $F_{tr,rd} = 434.3 \text{ kN}$
$\Sigma F_{tr,rd} = 940.4 \text{ kN}$

shear/bearing resistance
$V_{i,rd} = \Sigma F_{tr,rd} = 940.4 \text{ kN}$

2.3.3. shear resistance
shear resistance of column web
declarative basic component: 1
$V_{wp,rd}/\beta_1 = 579.1 \text{ kN}$

2.3.4. total
$M_{i,rd} = 218.9 \text{ kNm}$ $N_{j,1,rd} = 1109.5 \text{ kN}$ $N_{j,c,rd} = 579.1 \text{ kN}$ $V_{i,rd} = 940.4 \text{ kN}$ $V_{wp,rd}/\beta_1 = 579.1 \text{ kN}$

2.4. verifications
calculation of internal lever arm $z_{eq}$ s. rotational stiffness
2.4.1. Verification of the connection capacity by means of the component method

Internal moment: \( M_{Ed} = M_d = 200.00 \text{ kNm} \)

Shear force: \( V_{Ed} = |V_d| = 270.00 \text{ kN} \)

Parallel to connection plane

Shear force: \( V_{c,w,Ed} = M_d \cdot w / z - (V_{c1} - V_{c2}) / 2 = 553.68 \text{ kN}, \quad M_d \cdot w = 205.1 \text{ kNm}, \quad z = 370.5 \text{ mm} \)

Moment resistance

\( M_{Ed} / M_{J,Rd} = 0.914 \times 1 \text{ ok} \)

Shear/bearing resistance at 100\% utilization of moment resistance

\( V_{Ed} / V_{J,Rd} = 0.287 \times 1 \text{ ok} \)

Shear resistance of column web

\( V_{c,w,Ed} / (V_{wc,Rd} / \beta) = 0.966 \times 1 \text{ ok} \)

2.4.2. Verification result

Maximum utilization: max \( U = 0.956 < 1 \text{ ok} \)

2.5. Rotational stiffness

Stiffness coefficients

equivalent stiffness coefficient for 2 tension-bolt-rows:

effective stiffness coefficient for bolt-row 1:

\( k_5 = 0.9 \beta_{eff,fp} \cdot L / m = 30.72 \text{ mm}, \quad \beta_{eff,fp} = 5.00 \text{ mm}, \quad m = 40.9 \text{ mm} \)

\( k_{10} = 1.6 A_o / L_o = 8.13 \text{ mm}, \quad L_o = t_{geo} + 2 \cdot t_p + (t_{eff} + t_{mn}) / 2 = 69.5 \text{ mm}, \quad t_{geo} = 44.0 \text{ mm} \)

\( k_3 = 0.7 \beta_{eff,wc,twc} / d_c = 10.56 \text{ mm}, \quad \beta_{eff,wc,twc} = 285.4 \text{ mm} \)

\( k_4 = 0.9 \beta_{eff,fc} / m^2 = 16.03 \text{ mm}, \quad \beta_{eff,fc} = 285.4 \text{ mm}, \quad m = 47.9 \text{ mm} \)

\( Z(1/k_5) = 1/k_5 + 1/k_4 + 1/k_3 + 1/k_{10} = 0.313 \Rightarrow \beta_{eff,1} = 1 / \Sigma(1/k_i) = 3.198 \text{ mm} \)

effective stiffness coefficient for bolt-row 2:

\( k_5 = 0.9 \beta_{eff,fp} \cdot L / m = 20.22 \text{ mm}, \quad \beta_{eff,fp} = 41.5 \text{ mm}, \quad m = 66.1 \text{ mm} \)

\( k_{10} = 1.6 A_o / L_o = 8.13 \text{ mm}, \quad L_o = t_{geo} + 2 \cdot t_p + (t_{eff} + t_{mn}) / 2 = 69.5 \text{ mm}, \quad t_{geo} = 44.0 \text{ mm} \)

\( k_3 = 0.7 \beta_{eff,wc,twc} / d_c = 10.56 \text{ mm}, \quad \beta_{eff,wc,twc} = 285.4 \text{ mm} \)

\( k_4 = 0.9 \beta_{eff,fc} / m^2 = 16.03 \text{ mm}, \quad \beta_{eff,fc} = 285.4 \text{ mm}, \quad m = 47.9 \text{ mm} \)

\( Z(1/k_5) = 1/k_5 + 1/k_4 + 1/k_3 + 1/k_{10} = 0.330 \Rightarrow \beta_{eff,2} = 1 / \Sigma(1/k_i) = 3.034 \text{ mm} \)

Equivalent internal lever arm: \( Z_{eq} = Z(k_{eff,1} h_1^2) / Z(k_{eff,2} h_2) = 381.7 \text{ mm} \)

\( k_{eff} = Z(k_{eff,1} h_1) / Z_{eq} = 6.075 \text{ mm} \)

Stiffness coefficient of basic component 1:

\( k_1 = 0.38 A_{vc} / (\beta - z) = 4.86 \text{ mm}, \quad \beta = 1.0, \quad z = 370.5 \text{ mm} \)

Stiffness coefficient of basic component 2:

\( k_2 = 0.7 \beta_{eff,wc,twc} / d_c = 11.30 \text{ mm}, \quad \beta_{eff,wc,twc} = 305.3 \text{ mm} \)

Sum of stiffness coefficients \( Z(k_i) = 1/k_1 + 1/k_2 + 1/k_{eq} = 0.459 \)

Rotational stiffness

Initial rotational stiffness: \( S_{J,i} = (E \cdot z^2) / Z(1/k_i) = 66722.9 \text{ kNm/rad}, \quad z = 381.7 \text{ mm} \)

Internal moment at the connection point: \( M_{J,E} = M_{Ed} = 200.00 \text{ kNm} \)

\( M_{J,E} = 200.00 \text{ kNm} > 2/3 M_{J,Rd} = 145.9 \text{ kNm} \Rightarrow \mu = (1.5 M_{J,E}) / M_{J,Rd} = 2.343, \quad \Psi = 2.7 \)

Rotational stiffness: \( S_{J,Rd} = S_{J,i} / \mu = 28474.0 \text{ kNm/rad} \)

Rotation: \( \Phi_{J,E} = M_{J,E} / S_{J,Rd} = 0.402^\circ \)

3. Final result

Maximum utilization: max \( U = 0.956 < 1 \text{ ok} \)

Minimum rotational stiffness: \( \min S_j = 28.5 \text{ MNm/rad}, \quad S_{J,i} = 66.7 \text{ MNm/rad}, \quad \Psi = 0.402^\circ \)

Verification succeeded

4. Selected Design Parameters of the National Annex

DIN EN 1993-1-1 (EC 3)

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<th>Chapter</th>
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5. Regulations

DIN EN 1990, Eurocode 0: Grundlagen der Tragwerksplanung;
DIN EN 1990/NA, Nationaler Anhang zur DIN EN 1990, Ausgabe Dezember 2010

DIN EN 1993-1-1, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
  Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau;
DIN EN 1993-1-1/NA, Nationaler Anhang zur DIN EN 1993-1-1, Ausgabe September 2017

DIN EN 1993-1-8, Eurocode 3: Bemessung und Konstruktion von Stahlbauten -
  Teil 1-8: Bemessung von Anschlüssen;