General

Connection no.: 1
Connection name: Column-Beam

Geometry

Column

Section: HEB 220
$\alpha = -90,0$ [Deg] Inclination angle
$h_c = 220$ [mm] Height of column section
$b_c = 220$ [mm] Width of column section
$t_{wc} = 10$ [mm] Thickness of the web of column section
$t_{fc} = 16$ [mm] Thickness of the flange of column section
$r_c = 18$ [mm] Radius of column section fillet
$A_c = 9104$ [mm$^2$] Cross-sectional area of a column
$I_{xc} = 80909700$ [mm$^4$] Moment of inertia of the column section
Material: S 275 M/ML
$f_{yc} = 275,00$ [MPa] Resistance
BEAM

Section: HEB 220

- \( \alpha = 0.0 \) [Deg] Inclination angle
- \( h_b = 220 \) [mm] Height of beam section
- \( b_i = 220 \) [mm] Width of beam section
- \( t_{wb} = 10 \) [mm] Thickness of the web of beam section
- \( t_b = 16 \) [mm] Thickness of the flange of beam section
- \( r_b = 18 \) [mm] Radius of beam section fillet
- \( r_{fb} = 18 \) [mm] Radius of beam section fillet
- \( A_b = 9104 \) [mm\(^2\)] Cross-sectional area of a beam
- \( I_{xb} = 80909700 \) [mm\(^4\)] Moment of inertia of the beam section

Material: S 275 M/ML

- \( f_{yb} = 275.00 \) [MPa] Resistance

COLUMN STIFFENER

Upper
- \( h_{su} = 188 \) [mm] Stiffener height
- \( b_{su} = 105 \) [mm] Stiffener width
- \( t_{su} = 10 \) [mm] Stiffener thickness

Material: S 275

- \( f_{ysu} = 275.00 \) [MPa] Resistance

Lower
- \( h_{sd} = 188 \) [mm] Stiffener height
- \( b_{sd} = 105 \) [mm] Stiffener width
- \( t_{sd} = 10 \) [mm] Stiffener thickness

Material: S 275

- \( f_{ysu} = 275.00 \) [MPa] Resistance

FILLET WELDS

- \( a_w = 7 \) [mm] Web weld
- \( a_f = 7 \) [mm] Flange weld
- \( a_s = 7 \) [mm] Stiffener weld

MATERIAL FACTORS

- \( \gamma_M0 = 1.00 \) Partial safety factor
- \( \gamma_M1 = 1.00 \) Partial safety factor
- \( \gamma_M2 = 1.25 \) Partial safety factor
- \( \gamma_M3 = 1.25 \) Partial safety factor

[2.2]

LOADS

Ultimate limit state

Case: Manual calculations.

- \( M_{b1,Ed} = 100,00 \) [kN\(\cdot\)m] Bending moment in the right beam
- \( V_{b1,Ed} = 200,00 \) [kN] Shear force in the right beam
- \( N_{b1,Ed} = -800,00 \) [kN] Axial force in the right beam

RESULTS
BEAM RESISTANCES

COMPRESSION

\[ A_b = \frac{9104}{10^4} [\text{mm}^2] \quad \text{Area} \]

\[ N_{cb,\text{Ed}} = A_b f_{\text{cb}} / \gamma_{M0} \]

\[ N_{cb,\text{Ed}} = 2503,63 [\text{kN}] \quad \text{Design compressive resistance of the section} \]

\[ N_{cb,\text{Ed}} / N_{cb,\text{Ed}} \leq 1,0 \quad 0,32 < 1,00 \quad \text{verified} \]

\( V_{cb,\text{Ed}} / V_{cb,\text{Ed}} \leq 1,0 \)

\( 0,45 < 1,00 \quad \text{verified} \)

\( M_{b,\text{pl,Rd}} = W_{pl} f_{y_b} / \gamma_{M0} \)

\[ M_{b,\text{pl,Rd}} = 227,45 [\text{kN} \cdot \text{m}] \quad \text{Plastic resistance of the section for bending (without stiffeners)} \]

\[ M_{b,\text{pl,Rd}} = 890,20 [\text{kN} \cdot \text{m}] \quad \text{Plastic resistance of the section for bending} \]

\[ M_{b,\text{pl,Rd}} / M_{b,\text{pl,Rd}} \leq 1,0 \quad 0,44 < 1,00 \quad \text{verified} \]

BENDING WITH AXIAL FORCE ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

\[ W_{pl} = 827088 [\text{mm}^3] \quad \text{Plastic section modulus} \]

\[ M_{cb,\text{Ed}} = W_{pl} f_{y_b} / \gamma_{M0} \]

\[ M_{cb,\text{Ed}} = 227,45 [\text{kN} \cdot \text{m}] \quad \text{Design resistance of the section for bending} \]

\[ M_{cb,\text{Ed}} / M_{cb,\text{Ed}} \leq 1,0 \quad 0,57 < 1,00 \quad \text{verified} \]

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

\[ W_{pl} = 827088 [\text{mm}^3] \quad \text{Plastic section modulus} \]

\[ M_{cb,\text{Ed}} = W_{pl} f_{y_b} / \gamma_{M0} \]

\[ M_{cb,\text{Ed}} = 227,45 [\text{kN} \cdot \text{m}] \quad \text{Design resistance of the section for bending} \]

FLANGE AND WEB - COMPRESSION

\[ M_{cw,\text{Ed}} = 227,45 [\text{kN} \cdot \text{m}] \quad \text{Design resistance of the section for bending} \]

\[ h_t = 104 [\text{mm}] \quad \text{Distance between the centroids of flanges} \]

\[ F_{cw,\text{Ed}} = M_{cw,\text{Ed}} / h_t \]

\[ F_{cw,\text{Ed}} = 1114,95 [\text{kN}] \quad \text{Resistance of the compressed flange and web} \]

AXIAL FORCES IN BEAM FLANGES

\[ h_t = 204 [\text{mm}] \quad \text{Distance between the centroids of flanges} \]

\[ e_N = 0 [\text{mm}] \quad \text{Axial force eccentricity} \]

\[ N_{\text{upp}} = N_{b,\text{Ed}} / 2 + (-N_{b,\text{Ed}} e_N + M_{b,\text{Ed}}) / h_t \]

\[ N_{\text{upp}} = 90,20 [\text{kN}] \quad \text{Axial force in the beam top flange} \]

\[ N_{\text{low}} = N_{b,\text{Ed}} / 2 - (-N_{b,\text{Ed}} e_N + M_{b,\text{Ed}}) / h_t \]

\[ N_{\text{low}} = -890,20 [\text{kN}] \quad \text{Axial force in the beam bottom flange} \]

COLUMN RESISTANCES

WEB PANEL - SHEAR

\[ M_{b1,\text{Ed}} = 100,00 [\text{kN} \cdot \text{m}] \quad \text{Bending moment (right beam)} \]

\[ M_{b2,\text{Ed}} = 0,00 [\text{kN} \cdot \text{m}] \quad \text{Bending moment (left beam)} \]

\[ V_{c1,\text{Ed}} = 0,00 [\text{kN}] \quad \text{Shear force (lower column)} \]

\[ V_{c2,\text{Ed}} = 0,00 [\text{kN}] \quad \text{Shear force (upper column)} \]

\[ z = 204 [\text{mm}] \quad \text{Lever arm} \]

\[ V_{wp,\text{Ed}} = (M_{b1,\text{Ed}} - M_{b2,\text{Ed}}) / z - (V_{c1,\text{Ed}} - V_{c2,\text{Ed}}) / 2 \]

\[ V_{wp,\text{Ed}} = 490,20 [\text{kN}] \quad \text{Shear force acting on the web panel} \]

\[ A_v = 2792 [\text{mm}^2] \quad \text{Shear area of the column web} \]

\[ A_v = 2792 [\text{mm}^2] \quad \text{Shear area} \]

WEB PANEL - SHEAR

\[ M_{b1,\text{Ed}} = 100,00 [\text{kN} \cdot \text{m}] \quad \text{Bending moment (right beam)} \]

\[ M_{b2,\text{Ed}} = 0,00 [\text{kN} \cdot \text{m}] \quad \text{Bending moment (left beam)} \]

\[ V_{c1,\text{Ed}} = 0,00 [\text{kN}] \quad \text{Shear force (lower column)} \]

\[ V_{c2,\text{Ed}} = 0,00 [\text{kN}] \quad \text{Shear force (upper column)} \]

\[ z = 204 [\text{mm}] \quad \text{Lever arm} \]

\[ V_{wp,\text{Ed}} = (M_{b1,\text{Ed}} - M_{b2,\text{Ed}}) / z - (V_{c1,\text{Ed}} - V_{c2,\text{Ed}}) / 2 \]

\[ V_{wp,\text{Ed}} = 490,20 [\text{kN}] \quad \text{Shear force acting on the web panel} \]

\[ A_v = 2792 [\text{mm}^2] \quad \text{Shear area of the column web} \]

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WEB PANEL - SHEAR

\[ M_{b1,\text{Ed}} = 100,00 [\text{kN} \cdot \text{m}] \quad \text{Bending moment (right beam)} \]

\[ M_{b2,\text{Ed}} = 0,00 [\text{kN} \cdot \text{m}] \quad \text{Bending moment (left beam)} \]

\[ V_{c1,\text{Ed}} = 0,00 [\text{kN}] \quad \text{Shear force (lower column)} \]

\[ V_{c2,\text{Ed}} = 0,00 [\text{kN}] \quad \text{Shear force (upper column)} \]

\[ z = 204 [\text{mm}] \quad \text{Lever arm} \]

\[ V_{wp,\text{Ed}} = (M_{b1,\text{Ed}} - M_{b2,\text{Ed}}) / z - (V_{c1,\text{Ed}} - V_{c2,\text{Ed}}) / 2 \]

\[ V_{wp,\text{Ed}} = 490,20 [\text{kN}] \quad \text{Shear force acting on the web panel} \]

\[ A_v = 2792 [\text{mm}^2] \quad \text{Shear area of the column web} \]

\[ A_v = 2792 [\text{mm}^2] \quad \text{Shear area} \]
\( A_{\text{vs}} = 2792 \text{ [mm}^2\text{]} \) Shear area of the column web
\( d_s = 210 \text{ [mm]} \) Distance between the centroids of stiffeners
\( M_{\text{pl,fc,Rd}} = 3,87 \text{ [kN} \cdot \text{m]} \) Plastic resistance of the column flange for bending
\( M_{\text{pl,stu,Rd}} = 1,51 \text{ [kN} \cdot \text{m]} \) Plastic resistance of the upper transverse stiffener for bending
\( M_{\text{pl,sl,Rd}} = 1,51 \text{ [kN} \cdot \text{m]} \) Plastic resistance of the lower transverse stiffener for bending
\( V_{\text{wp,Rd}} = 0.9 \left( \frac{A_{\text{vs}}}{t_{\text{wc}}} \right) \left( \frac{1}{\sqrt{3}} \right) \gamma_{\text{M0}} + \text{Min}(4 \frac{M_{\text{pl,fc,Rd}}}{d_s}, \frac{2 M_{\text{pl,sl,Rd}} + M_{\text{pl,stu,Rd}} + M_{\text{pl,sl,Rd}}}{d_s}) \)
\( V_{\text{wp,Rd}} = 450,26 \text{ [kN]} \) Resistance of the column web panel for shear
\( V_{\text{wp,Ed}} / V_{\text{wp,Rd}} \leq 1,0 \quad 1,09 > 1,00 \quad \text{not verified} \)

**WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE**

**Bearing:**
\( t_{\text{wc}} = 10 \text{ [mm]} \) Effective thickness of the column web
\( b_{\text{eff,wc}} = 206 \text{ [mm]} \) Effective width of the web for compression
\( A_{\text{wc}} = 2792 \text{ [mm}^2\text{]} \) Shear area
\( \omega = 0,78 \) Reduction factor for interaction with shear
\( \sigma_{\text{com,Ed}} = 0,00 \text{ [MPa]} \) Maximum compressive stress in web
\( k_{\text{wc}} = 1,00 \) Reduction factor conditioned by compressive stresses
\( A_s = 2105 \text{ [mm}^2\text{]} \) Area of the web stiffener
\( F_{\text{vc,wc,Rd1}} = \omega k_{\text{wc}} b_{\text{eff,wc}} t_{\text{wc}} f_{\text{yc}} / \gamma_{\text{M0}} + A_s f_{\text{ys}} / \gamma_{\text{M0}} \)
\( F_{\text{vc,wc,Rd1}} = 999,04 \text{ [kN]} \) Column web resistance

**Buckling:**
\( d_{\text{wc}} = 152 \text{ [mm]} \) Height of compressed web
\( \lambda_p = 0,63 \) Plate slenderness of an element
\( \rho = 1,00 \) Reduction factor for element buckling
\( \lambda_s = 2,34 \) Stiffener slenderness
\( \chi_s = 1,00 \) Buckling coefficient of the stiffener
\( F_{\text{vc,wc,Rd2}} = \omega k_{\text{wc}} \rho b_{\text{eff,wc}} t_{\text{wc}} f_{\text{yc}} / \gamma_{\text{M1}} + A_s f_{\text{ys}} / \gamma_{\text{M1}} \)
\( F_{\text{vc,wc,Rd2}} = 999,04 \text{ [kN]} \) Column web resistance

**Final resistance:**
\( F_{\text{vc,wc,Rd,low}} = \text{Min} (F_{\text{vc,wc,Rd1}}, F_{\text{vc,wc,Rd2}}) \)
\( F_{\text{vc,wc,Rd}} = 999,04 \text{ [kN]} \) Column web resistance
\( N_{\text{low}} / F_{\text{vc,wc,Rd,low}} \leq 1,0 \quad 0,89 < 1,00 \quad \text{verified} \)

**WEB - TRANSVERSE TENSION - LEVEL OF THE BEAM TOP FLANGE**

\( t_{\text{wc}} = 10 \text{ [mm]} \) Effective thickness of the column web
\( b_{\text{eff,wc}} = 206 \text{ [mm]} \) Effective width of the web for compression
\( A_{\text{wc}} = 2792 \text{ [mm}^2\text{]} \) Shear area
\( \omega = 0,78 \) Reduction factor for interaction with shear
\( A_s = 2105 \text{ [mm}^2\text{]} \) Area of the web stiffener
\( F_{\text{lw,wc,Rd,upp}} = \omega b_{\text{eff,wc}} t_{\text{wc}} f_{\text{yc}} / \gamma_{\text{M0}} + A_s f_{\text{ys}} / \gamma_{\text{M0}} \)
\( F_{\text{lw,wc,Rd}} = 999,04 \text{ [kN]} \) Column web resistance
\( N_{\text{upp}} / F_{\text{lw,wc,Rd,upp}} \leq 1,0 \quad 0,09 < 1,00 \quad \text{verified} \)

**WELD RESISTANCE**

\( A_{\text{aw}} = 7651 \text{ [mm}^2\text{]} \) Area of all welds
\( A_{\text{aw}} = 5523 \text{ [mm}^2\text{]} \) Area of horizontal welds
\( A_{\text{aw}} = 2128 \text{ [mm}^2\text{]} \) Area of vertical welds
\( I_{\text{wy}} = 63805772 \text{ [mm}^4\text{]} \) Moment of inertia of the weld arrangement with respect to the hor. axis
\( \sigma_{\text{m,max}}=\tau_{\text{m,max}} = -199,72 \text{ [MPa]} \) Normal stress in a weld
\( \sigma_{\text{m}}=\tau_{\text{m}} = -158,16 \text{ [MPa]} \) Stress in a vertical weld
\( t_{\text{m}} = 93,98 \text{ [MPa]} \) Tangent stress
\( \beta_w = 0,85 \) Correlation coefficient
\( \sqrt{\sigma_{\text{m,max}}^2 + 3\tau_{\text{m,max}}^2} \leq f_{\text{w}} (\beta_w \gamma_{\text{M2}}) \quad 399,44 > 348,24 \quad \text{not verified} \)
\[\sqrt{[\sigma_{\text{max}}^z + 3(\tau_{\text{max}}^z)]} \leq \frac{f_u}{(\beta_\omega \gamma M_2)}\]

\[399.44 \leq 348.24\]

not verified \((1, 15)\)

\[\sqrt{[\sigma_{\text{max}}^\sigma + 3(\tau_{\text{max}}^\sigma + \tau_{\text{II}}^\sigma)]} \leq \frac{f_u}{(\beta_\omega \gamma M_2)}\]

\[355.75 \leq 348.24\]

not verified \((1, 02)\)

\[\sigma_\perp \leq 0.9 \frac{f_u}{\gamma M_2}\]

\[199.72 < 266.40\]

verified \((0, 75)\)

**CONNECTION STIFFNESS**

\[A_{vc} = 2792 \text{ [mm}^2\text{]}\] Shear area

\[\beta = 1.00\] Transformation parameter

\[z = 204 \text{ [mm]}\] Lever arm

\[k_1 = 5 \text{ [mm]}\] Stiffness coefficient of the column web panel subjected to shear

\[k_2 = \infty\] Stiffness coefficient of the compressed column web

\[k_3 = \infty\] Stiffness coefficient of the column web subjected to tension

\[S_{j,\text{ini}} = \frac{E z^2}{\sum (1/k_1 + 1/k_2 + 1/k_3)}\]

\[S_{j,\text{ini}} = 45453.48 \text{ [kN*m]}\] Initial rotational stiffness

\[\eta = 2.00\] Stiffness coefficient of a connection

\[S_i = S_{j,\text{ini}} / \eta\]

\[S_i = 22726.74 \text{ [kN*m]}\] Final rotational stiffness

**Connection classification due to stiffness.**

\[S_{j,\text{rig}} = 27185.66 \text{ [kN*m]}\] Stiffness of a rigid connection

\[S_{j,\text{pin}} = 1699.10 \text{ [kN*m]}\] Stiffness of a pinned connection

\[S_{j,\text{ini}} \geq S_{j,\text{pin}}\] RIGID

**WEAKEST COMPONENT:**

WELDS

**Connection does not conform to the code**

Ratio \(1, 15\)