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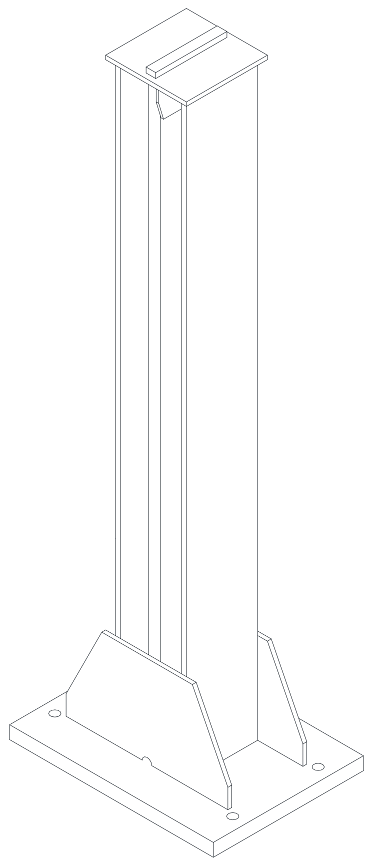
Metal Structures II

design of column



Conditions:

1. Finishing layers of the roof:	Insulated
2. Steel grade:	S235
3. Span of the truss:	$L = 29.4m$
4. Length: n x l, where :	$n = 10$ $l = 6.0m$
5. Height (max) at the ridge:	$H = 7.2m$
6. Roof slope:	$\alpha = 3deg$
7. Snow load:	I zone
8. Wind load:	I zone

9. COLUMN

9.1. Characteristic geometrical cross-section:

HEB 160

$G = 0.043 \frac{kg}{mm}$ $= 160 \cdot mm$ $b = 160 \cdot mm$ $t_f = 13 \cdot mm$

$h_w = 104 \cdot mm$ $r = 15 \cdot mm$ $t_w = 8 \cdot mm$

$h_i = 134 \cdot mm$ $A = 5430 \cdot mm^2$

$W_{el.y} = 311500 \cdot mm^3$ $W_{el.z} = 111200 \cdot mm^3$ $i_y = 67.8 \cdot mm$ $J_y = 24920000 \cdot mm^4$

$W_{pl.y} = 354000 \cdot mm^3$ $W_{pl.z} = 170000 \cdot mm^3$ $i_z = 40.5 \cdot mm$ $J_z = 8892000 \cdot mm^4$

$I_T = 312400 \cdot mm^4$ $I_\omega = 47940000000 \cdot mm^6$

9.2. Class of cross-section:

$$\varepsilon = \sqrt{\frac{235MPa}{f_y}} = \sqrt{\frac{235 \cdot MPa}{275 \cdot MPa}} \rightarrow \varepsilon = 0.924$$

- Web:

$$c = h - 2 \cdot (t_f + r) = 160 \cdot mm - 2 \cdot (13 \cdot mm + 15 \cdot mm) \rightarrow c = 104.0 \cdot mm$$

$t_w = 8 \cdot mm$

$\frac{c}{t_w} = 13$ < $33 \cdot \varepsilon = 30.506$ → web has class 1

- flange:

$$c = 0.5 \cdot (b - t_w - 2 \cdot r) = 0.5 \cdot (160 \cdot mm - 8 \cdot mm - 2 \cdot 15 \cdot mm) \rightarrow c = 61 \cdot mm$$

$t_f = 13.0 \cdot mm$

$\frac{c}{t_f} = 4.692$ < $9 \cdot \varepsilon = 8.32$ → flanges have class 1

- **whole cross-section:**

<u>whole cross-section is first class</u>

9.3. Determination of buckling resistance due to force

$\frac{N_{Ed}}{N_{b.Rd}} \le 1.0$ - required condition

$E = 210GPa$

$N_{Ed} = 260.36kN$

$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$

$L_y = 5560mm$ $\mu_y = 2.0$

$L_z = 5560mm$ $\mu_z = 1.0$

$L_{cr.y} = \mu_y \cdot L_y = 11120 \cdot mm$ $L_{cr.z} = \mu_z \cdot L_z = 5560 \cdot mm$

$\lambda_1 = \pi \cdot \sqrt{\frac{E}{f_y}} = 86.815$

$i_y = 67.8 \cdot mm$ $i_z = 40.5 \cdot mm$

$\lambda_y = \frac{L_{cr.y}}{i_y} \cdot \frac{1}{\lambda_1} = 1.889$ $\lambda_z = \frac{L_{cr.z}}{i_z} \cdot \frac{1}{\lambda_1} = 1.581$

$\frac{h}{b} = 1.0$ $\rightarrow$ $\frac{h}{b} \le 1.2$ $\rightarrow$ from tab. 6.2: buckling due to y-y axis $\rightarrow$ curve b $\rightarrow$ $\alpha_y = 0.34$

z-z axis $\rightarrow$ curve c $\rightarrow$ $\alpha_z = 0.49$

$$\phi_y = 0.5 \cdot \left[1 + \alpha_y \cdot \left(\lambda_y - 0.2\right) + \lambda_y^2\right] = 0.5 \cdot \left[1 + 0.34 \cdot (1.889 - 0.2) + 1.889^2\right] \rightarrow \phi_y = 2.571$$

$$\phi_z = 0.5 \cdot \left[1 + \alpha_z \cdot \left(\lambda_z - 0.2\right) + \lambda_z^2\right] = 0.5 \cdot \left[1 + 0.49 \cdot (1.581 - 0.2) + 1.581^2\right] \rightarrow \phi_z = 2.088$$

$$\chi_y = \frac{1}{\phi_y + \sqrt{\phi_y^2 - \lambda_y^2}} = \frac{1}{2.571 + \sqrt{2.571^2 - 1.889^2}} \rightarrow \chi_y = 0.232$$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_z^2}} = \frac{1}{2.088 + \sqrt{2.088^2 - 1.581^2}} \rightarrow \chi_z = 0.290$$

$\chi = \min\left(\chi_y, \chi_z\right) = 0.232$

$$N_{b.Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0.232 \cdot 54.3 \cdot 10^2 \cdot mm^2 \cdot 275 \cdot MPa}{1.0} \rightarrow N_{b.Rd} = 346.434 \cdot kN$$

$\frac{N_{Ed}}{N_{b.Rd}} = 0.752$ < 1.0 - condition satisfied

$0.8 < \frac{N_{Ed}}{N_{b.Rd}} < 1$ perfect situation

9.4. Determination of buckling resistance due to moment

$$\frac{M_{Ed}}{M_{b.Rd}} \le 1.0$$ - required condition

Critical spring moment in buckling:

$k = \mu_z = 1.0$ $E = 210 \cdot GPa$ $k_w = 1.0$ $I_\omega = 47940000000 \cdot mm^6$

$G = 81000 \frac{N}{mm^2}$ $L = 5560\,mm$ $I_T = 312400 \cdot mm^4$ $I_z = J_z = 8892000 \cdot mm^4$

$$\psi = \frac{0}{171.961} = 0 \quad \rightarrow \quad C_1 = 1.88 \qquad M_{cr} = C_1 \cdot \frac{\pi^2 \cdot E \cdot I_z}{(k \cdot L)^2} \cdot \sqrt{\left(\frac{k}{k_w}\right)^2 \cdot \frac{I_\omega}{I_z} + \frac{(k \cdot L)^2 \cdot G \cdot I_T}{\pi^2 \cdot E \cdot I_z}}$$

$$M_{cr} = 1.88 \cdot \frac{\pi^2 \cdot 210 \cdot GPa \cdot 8892000 \cdot mm^4}{(5560 \cdot mm)^2} \cdot \sqrt{\left(\frac{1}{1.0}\right)^2 \cdot \frac{47.94 \cdot 10^9 \cdot mm^6}{8892000 \cdot mm^4} + \frac{(5560 \cdot mm)^2 \cdot 81000 \cdot \frac{N}{mm^2} \cdot 31.24 \cdot 10^4 \cdot mm^4}{\pi^2 \cdot 210 \cdot GPa \cdot 8892000 \cdot mm^4}}$$

$$M_{cr} = 245.135 \cdot kN \cdot m$$

Relative slenderness:

$W_y = W_{pl.y} = 354000 \cdot mm^3$ $f_y = 275 \cdot MPa$ $\lambda_{LT} = \sqrt{\frac{W_y \cdot f_y}{M_{cr}}} = 0.63$

Buckling coefficient:

$\frac{h}{b} = 1.0 \rightarrow \frac{h}{b} \le 2 \rightarrow$ from tab. 6.5: buckling curve b $\rightarrow \alpha_{LT} = 0.34$

$\lambda_{LT.0} = 0.4$

$\beta = 0.85$

$$\phi_{LT} = 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{LT} - \lambda_{LT.0}) + \beta \cdot \lambda_{LT}^2\right] = 0.5 \cdot \left[1 + 0.34 \cdot (0.63 - 0.4) + 0.85 \cdot 0.63^2\right]$$

$\phi_{LT} = 0.708$

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta \cdot \lambda_{LT}^2}} = \frac{1}{0.708 + \sqrt{0.708^2 - 0.85 \cdot 0.63^2}} \quad \rightarrow \quad \chi_{LT} = 0.899$$

$\chi_{LT} = 0.899 \le 1.0$ oraz $\chi_{LT} = 0.899 \le \frac{1}{\lambda_{LT}^2} = 2.52$ (conditions satisfied)

Element resistance on buckling:

$$M_{b.Rd} = \frac{\chi_{LT} \cdot W_y \cdot f_y}{\gamma_{M1}} = \frac{0.899 \cdot 354000 \cdot mm^3 \cdot 275 \cdot MPa}{1.0} \quad \rightarrow \quad M_{b.Rd} = 87.518 \cdot kN \cdot m$$

$M_{Ed} = 52.23 kN \cdot m$

$$\frac{M_{Ed}}{M_{b.Rd}} = 0.597 < 1.0$$ - condition satisfied

9.5. Checking the cross-section resistance due to compressing and bending considering buckling and lateral torsional buckling

Conditions:

$$\frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT} \cdot \frac{M_{y.Rk}}{\gamma_{M1}}} + k_{yz} \cdot \frac{M_{z.Ed} + \Delta M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} \leq 1.0$$

$$\frac{N_{Ed}}{\frac{\chi_z \cdot N_{Rk}}{\gamma_{M1}}} + k_{zy} \cdot \frac{M_{y.Ed} + \Delta M_{y.Ed}}{\chi_{LT} \cdot \frac{M_{y.Rk}}{\gamma_{M1}}} + k_{zz} \cdot \frac{M_{z.Ed} + \Delta M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} \leq 1.0$$

In cases of compressing and unidirectional bending $M_{y.Ed}$ we can adopt k_{zy}=0.

After simplification the conditions are:

$$\frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y.Ed}}{\chi_{LT} \cdot \frac{M_{y.Rk}}{\gamma_{M1}}} \leq 1.0 \qquad \frac{N_{Ed}}{\frac{\chi_z \cdot N_{Rk}}{\gamma_{M1}}} \leq 1.0$$

$$N_{Rk} = A \cdot f_y = 54.3 \cdot 10^2 \cdot mm^2 \cdot 275 \cdot MPa = 1493.25\ kN \qquad \rightarrow N_{Rk} = 1493.25 \cdot kN$$

$$M_{y.Rk} = W_{pl.y} \cdot f_y = 354 \cdot 10^3 \cdot mm^3 \cdot 275 \cdot MPa \qquad \rightarrow \qquad M_{y.Rk} = 97.35 \cdot kN \cdot m$$

$$\lambda_y = 1.889 \qquad \chi_y = 0.232 \qquad \chi_z = 0.29 \qquad \chi_{LT} = 0.899$$

For max values of bending (combination G+S1+W4) :

$N_{Ed} = 62.68kN$ $M_{y.Ed} = 52.23kN \cdot m$ $M_h = 52.23kN \cdot m$ $M_s = 0$ $\psi = 0$

$$\alpha_s = \frac{M_s}{M_h} = 0$$

for requirements $-1 \le \alpha_s < 0$ and $0 \le \psi \le 1$ and for concentrated load:

$C_{my} = -0.8 \cdot \alpha_s = 0$ and $C_{my} \ge 0.4$

$$C_{my} = 0.4$$

$$k_{yy} = C_{my} \cdot \left[1 + \left(\lambda_y - 0.2\right) \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}}\right] \quad \text{and} \quad k_{yy} \le C_{my} \cdot \left(1 + 0.8 \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}}\right)$$

$$k_{yy} = 0.4 \cdot \left[1 + (1.889 - 0.2) \cdot \frac{62.68 \cdot kN}{\frac{0.232 \cdot 1493.25 \cdot kN}{1.0}}\right] \rightarrow k_{yy} = 0.522$$

$$C_{my} \cdot \left(1 + 0.8 \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}}\right) = 0.4 \cdot \left(1 + 0.8 \cdot \frac{62.68 \cdot kN}{\frac{0.232 \cdot 1493.25 \cdot kN}{1.0}}\right) = 0.458$$

$k_{yy} \le 0.458$ $k_{yy} = 0.458$

$$\frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y.Ed}}{\chi_{LT} \cdot \frac{M_{y.Rk}}{\gamma_{M1}}} = \frac{62.68 \cdot kN}{\frac{0.232 \cdot 1493.25 \cdot kN}{1.0}} + 0.458 \cdot \frac{52.23 \cdot kN \cdot m}{0.899 \cdot \frac{354 \cdot 10^3 \cdot mm^3 \cdot 275 \cdot MPa}{1.0}} = 0.454 < 1.0$$

- condition satisfied

$$\frac{N_{Ed}}{\frac{\chi_z \cdot N_{Rk}}{\gamma_{M1}}} = \frac{62.68 \cdot kN}{\frac{0.290 \cdot 1493.25 \cdot kN}{1.0}} = 0.145 \quad < 1.0$$

- condition satisfied

For max values of compressive force:

$N_{Ed} = 260.36kN$ $M_{y.Ed} = 4.28kN \cdot m$ $\psi = 0$

$C_{my} = 0.6 + 0.4 \cdot \psi = 0.6$ and $C_{my} \geq 0.4$

$C_{my} = 0.6$

$$k_{yy} = C_{my} \cdot \left[1 + (\lambda_y - 0.2) \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} \right]$$ and $$k_{yy} \leq C_{my} \cdot \left(1 + 0.8 \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} \right)$$

$$k_{yy} = 0.6 \cdot \left[1 + (1.889 - 0.2) \cdot \frac{260.36 \cdot kN}{\frac{0.232 \cdot 1493.25 \cdot kN}{1.0}} \right] \rightarrow \quad k_{yy} = 1.362$$

$$C_{my} \cdot \left(1 + 0.8 \cdot \frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} \right) = 0.6 \cdot \left(1 + 0.8 \cdot \frac{260.36 \cdot kN}{\frac{0.232 \cdot 1493.25 \cdot kN}{1.0}} \right) = 0.961$$

$k_{yy} \leq 0.961$ $k_{yy} = 0.961$

$$\frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y.Ed}}{\chi_{LT} \cdot \frac{M_{y.Rk}}{\gamma_{M1}}} = \frac{260.36 \cdot kN}{\frac{0.232 \cdot 1493.25 \cdot kN}{1.0}} + 0.961 \cdot \frac{4.28 \cdot kN \cdot m}{0.899 \cdot \frac{354 \cdot 10^3 \cdot mm^3 \cdot 275 \cdot MPa}{1.0}} = 0.799 \quad < 1.0$$

- condition satisfied

$$\frac{N_{Ed}}{\frac{\chi_z \cdot N_{Rk}}{\gamma_{M1}}} = \frac{260.36 \cdot kN}{\frac{0.290 \cdot 1493.25 \cdot kN}{1.0}} = 0.601 \quad < 1.0$$ - condition satisfied

9.6. Checking the column at the rigid end

$M_{Ed} \le M_{N.Rd}$ - required condition

In case of bi-symmetric I beams we can omit the influence of longitudinal force on plastics capacity in bending due to y-y axis, when below condition is satisfied:

$N_{Ed} \le 0.25 \cdot N_{pl.Rd}$ and $N_{Ed} \le \dfrac{0.5 \cdot h_w \cdot t_w \cdot f_y}{\gamma_{M0}}$

$$N_{pl.Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{54.3 \cdot 10^2 \cdot mm^2 \cdot 275 \cdot MPa}{1.0} \rightarrow N_{pl.Rd} = 1493.25 \cdot kN$$

$0.25 \cdot N_{pl.Rd} = 373.313 \cdot kN$

$$\frac{0.5h_w \cdot t_w \cdot f_y}{\gamma_{M0}} = \frac{0.5 \cdot 104 \cdot mm \cdot 8 \cdot mm \cdot 275 \cdot MPa}{1.0} \rightarrow \frac{0.5h_w \cdot t_w \cdot f_y}{\gamma_{M0}} = 114.4 \cdot kN$$

$N_{Ed} = 260.360\ kN$

→ conditions not satisfied influence of longitudinal force cannot be omitted

$$n = \frac{N_{Ed}}{N_{pl.Rd}} = \frac{260.36 \cdot kN}{1493.25 \cdot kN} \rightarrow n = 0.174$$

$$a = \frac{A - 2 \cdot b \cdot t_f}{A} = \frac{54.3 \cdot 10^2 \cdot mm^2 - 2 \cdot 160 \cdot mm \cdot 13 \cdot mm}{54.3 \cdot 10^2 \cdot mm^2} \quad \text{but} \quad a \le 0.5 \qquad a = 0.234 < 0.5$$

$$M_{pl.y.Rd} = \frac{W_{pl.y} \cdot f_y}{\gamma_{M0}} = \frac{354 \cdot 10^3 \cdot mm^3 \cdot 275 \cdot MPa}{1.0} \rightarrow M_{pl.y.Rd} = 97.35 \cdot kN \cdot m$$

$$M_{N.y.Rd} = M_{pl.y.Rd} \cdot \frac{(1-n)}{1 - 0.5 \cdot a} \quad \text{but} \quad M_{N.y.Rd} \le M_{pl.y.Rd}$$

$$M_{N.y.Rd} = 97.35 \cdot kN \cdot m \cdot \frac{1 - \dfrac{260.36 \cdot kN}{1493.25 \cdot kN}}{1 - 0.5 \cdot 0.234}$$

$M_{N.y.Rd} = 91.026 \cdot kN \cdot m < M_{pl.y.Rd} = 97.35 \cdot kN \cdot m$

$M_{y.Ed} = 52.53 kN \cdot m$

$$\frac{M_{y.Ed}}{M_{N.y.Rd}} = \frac{52.53 \cdot kN \cdot m}{91.026 \cdot kN \cdot m} = 0.577$$ - condition satisfied

9.7. Checking of horizontal displacements

$u_{max} \le \frac{H}{150}$ - in one storage schemas (without gantries)

H - height of chosen dead bolt to upper part of foundation

(adopted distance from floor to chosen part of

$H = 5.56m$

Max horizontal displacement :

$u_{max} = 14mm$ < $\frac{H}{150} = 37.067 \cdot mm$ -condition satisfied

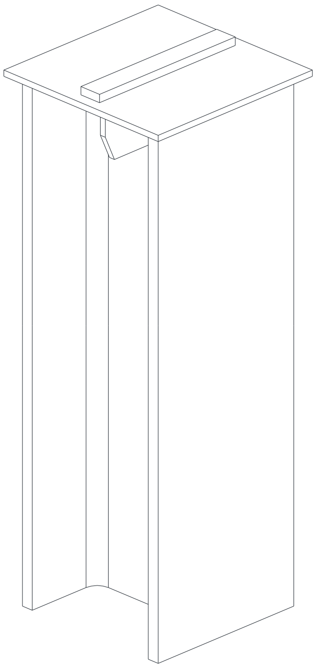
10. COLUMNS CAP

Dimension of closing plate:

$b_g = b + 2 \cdot 15mm = 160 \cdot mm + 2 \cdot 15 \cdot mm \rightarrow \quad b_g = 190 \cdot mm$

$h_g = h + 2 \cdot 15mm = 160 \cdot mm + 2 \cdot 15 \cdot mm \rightarrow \quad h_g = 190 \cdot mm$

$t_g = 12mm$



Dimension of concentric plate:

$b_{pc} = 30mm$

$t_{pc} = 20mm$

$l_{pc} = h_g - 2 \cdot 15mm = 160 \cdot mm + 2 \cdot 15 \cdot mm - 2 \cdot 15 \cdot mm \rightarrow l_{pc} = 160 \cdot mm$

10.1. Compressive strength of concentric plate

$$\sigma_d = \frac{N_{Ed}}{A_d} \le f_d$$ **- condition**

Area of compressive stresses considering concentric plate:

$l'_{pc} = l_{pc} - 2 \cdot 5mm = 150 \cdot mm$

$b'_{pc} = b_{pc} - 2 \cdot 5mm = 20 \cdot mm$

$A_d = l'_{pc} \cdot b'_{pc} = 3000 \cdot mm^2$

$N_{Ed} = 260.36\,kN$

$$\sigma_d = \frac{N_{Ed}}{A_d} = \frac{260.36 \cdot kN}{3000 \cdot mm^2} \rightarrow \sigma_d = 86.787\,MPa$$

$$f_d = 1.25 \cdot \frac{f_y}{\gamma_{M0}} = 1.25 \cdot \frac{275 \cdot MPa}{1.0} \rightarrow f_d = 343.750\,MPa$$

$\sigma_d = 86.787\,MPa \quad < \quad f_d = 343.750\,MPa$ **-condition satisfied**

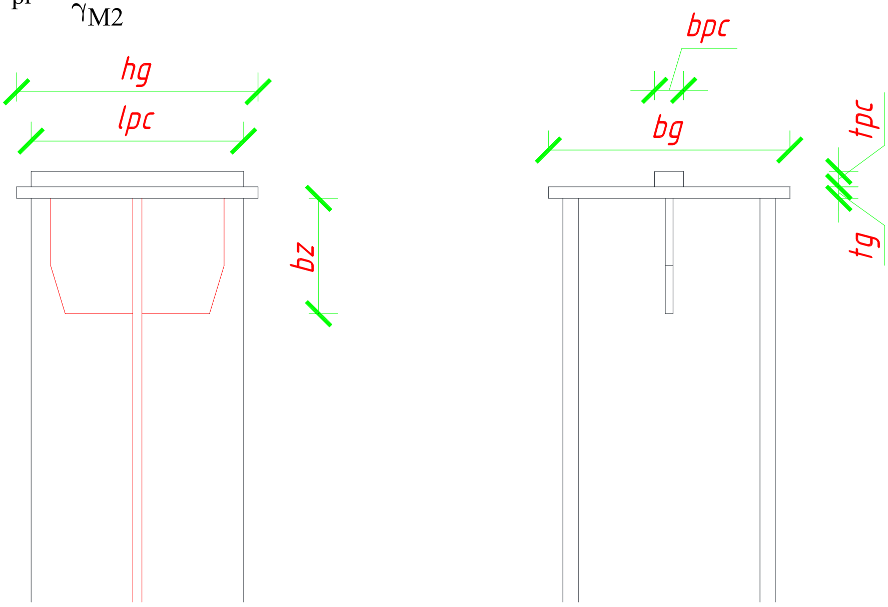
10.2. Weld connecting the centric plate with closing plate:

$$\sqrt{\sigma_{pr}^2 + 3 \cdot \left(\tau_{pr}^2 + \tau_{||}^2\right)} \le \frac{f_u}{\beta_w \cdot \gamma_{M2}} \qquad \sigma_{pr} \le \frac{0.9 \cdot f_u}{\gamma_{M2}}$$

Where:

$\sigma_{\perp}$ - σ_{pr}

$\tau_{\perp}$ - τ_{pr}



For steal S275

$f_u = 430\,MPa$

$\beta_w = 0.9$

$\gamma_{M2} = 1.25$

According to construction conditions:

$0.2 \cdot t_{max} \le a \le 0.7 \cdot t_{min}$ and $a \ge 3mm$

$a(t_{pc};t_g) = a(20;12) \rightarrow \quad 4 \le a \le 8.4$

adopted welds **a = 4mm**.

Weld area: $A_w = a \cdot (2 \cdot l_{pc} + 2 \cdot b_{pc}) = 4 \cdot mm \cdot (2 \cdot 160 \cdot mm + 2 \cdot 30 \cdot mm) \rightarrow A_w = 1520 \cdot mm^2$

Stresses in welds: $\sigma = \frac{N_{Ed}}{A_w} = \frac{260.36 \cdot kN}{1520 \cdot mm^2} \rightarrow \sigma = 171.289\,MPa$

$\tau_{||} = 0$

$\sigma_{pr} = \frac{\sigma}{\sqrt{2}} = 121.12\,MPa \qquad \tau_{pr} = \frac{\sigma}{\sqrt{2}} = 121.12\,MPa$

$\sqrt{\sigma_{pr}^2 + 3 \cdot \tau_{pr}^2} = 2 \cdot \sigma_{pr} = 242.24\,MPa \quad < \quad \frac{f_u}{\beta_w \cdot \gamma_{M2}} = 382.222\,MPa$ **- condition satisfied**

$\sigma_{pr} = 121.12\,MPa \quad < \quad 0.9 \cdot \frac{f_u}{\gamma_{M2}} = 309.600\,MPa$ **-condition satisfied**

10.3. Weld connecting the closing plate with ribs/stiffeners:

thickness of plate $t_g = 12 \cdot mm$

thickness of rib $t_z = 1.6 \cdot a = 6.4 \cdot mm$ → assumed $t_z = 8.0mm$

width of rib $b_z = 0.5(b - t_w) = 0.5 \cdot (160 \cdot mm - 8 \cdot mm) \rightarrow b_z = 76 \cdot mm$

→ adopted $b_z = 80mm$

Adopted Weld connecting the closing plate with ribs/stiffeners: a = 5mm.

$$A_w = a \cdot (4b_z + 2t_z) = 5 \cdot mm \cdot (4 \cdot 80 \cdot mm + 2 \cdot 8.0 \cdot mm) \rightarrow A_{w.z} = A_w = 1680 \cdot mm^2$$

$$A_{w.pc} = 1520 \cdot mm^2$$

$A_{w.z} = 1680\,mm^2$ > $A_{w.pc} = 1520 \cdot mm^2$ condition $A_{w.z} \ge A_{w.pc}$ satisfied, so condition of weld will be also satisfied

10.4. Weld connecting ribs/stiffeners with web of the column

$$\sqrt{3 \cdot \tau_{||}^2} \le \frac{f_u}{\beta_w \cdot \gamma_{M2}}$$ - condition of weld (cause $\sigma_{\perp} = 0$ and $\tau_{\perp} = 0$)

thickness of web $t_w = 8 \cdot mm$

Dimensions of rib/stiffener:

thickness $t_z = 8 \cdot mm$

with $b_z = 80 \cdot mm$

height $h_z \ge \dfrac{N_{Ed}}{4 \cdot a_1 \cdot \dfrac{0.58 \cdot f_u}{\beta_w \cdot \gamma_{M2}}}$

$0.2 \cdot t_{max} \le a_1 \le 0.7 \cdot t_{min}$ and $a_1 \ge 3mm$

$a_1(t_z;t_w) = a_1(8;8)$ → $1.6 \le a_1 \le 5.6$ Adopted weld **a_1 = 3mm**.

$$h_z \ge \frac{N_{Ed}}{4 \cdot a_1 \cdot \frac{0.58 \cdot f_u}{\beta_w \cdot \gamma_{M2}}} = \frac{260.36 \cdot kN}{4 \cdot 3 \cdot mm \cdot \frac{0.58 \cdot 430 \cdot MPa}{0.9 \cdot 1.25}} \rightarrow h_{z.min} = 97.87 \cdot mm$$ Adopted $h_z = 100mm$

$$\tau_{||} = \frac{N_{Ed}}{4 \cdot a_1 \cdot h_z} = \frac{260.36 \cdot kN}{4 \cdot 3 \cdot mm \cdot 100 \cdot mm} \rightarrow \tau_{||} = 216.967\,MPa$$

$$\sqrt{3} \cdot \tau_{||} = 375.797\,MPa < \frac{f_u}{\beta_w \cdot \gamma_{M2}} = 382.222\,MPa$$ -condition satisfied

10.5. Weld connecting the closing plate with the profile

According to construction conditions:

$0.2 \cdot t_{max} \le a \le 0.7 \cdot t_{min}$ oraz $a \ge 3mm$

$a(t_g;t_w) = a(12;8)$ → $2.4 \le a \le 5.6$

Adopted weld **a = 3mm**.

11. COLUMNS BASE

$M_{Ed} = 53.65kN \cdot m$

$N_{Ed} = 260.36kN$

$V_{Ed} = 25.47kN$

11.1. Choosing and pacing of anchors

Adopted plate anchors Ø16 mm from steel S275

$A_s = 201mm^2$ $d = 16mm$

$f_{ub} = 420MPa$

Tensional forces resistance of bolts

$$F_{T.Rd} = \frac{0.9 \cdot A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0.9 \cdot 201 \cdot mm^2 \cdot 420 \cdot MPa}{1.25} \rightarrow F_{T.Rd} = 60.782\,kN$$

Placing of the

$$d_p = \frac{M_{Ed}}{0.5 \cdot N_{Ed} + n \cdot F_{T.Rd}} = \frac{53.65 \cdot kN \cdot m}{0.5 \cdot 260.36 \cdot kN + 2 \cdot 60.782 \cdot kN} \rightarrow d_p = 213.113 \cdot mm$$

Przyjęto $d_p = 250mm$

$n = 2$ - nr of anchors in tension zone

Diameter of hole in base plate and anchor

$D_o = (5/3 \div 2) \cdot d = (5/3 \div 2) \cdot 24 = 40 \div 48$ mm

Adopted $D_o = 45mm$

Dimensions of plate under the anchor

thickness $t_1 = 10mm$

with $b_1 = D_o + 30mm = 75 \cdot mm$

Distance e_1 - from anchor to edge of base plate

for bolt with diameter $14 \le d \le 24 \rightarrow$ $\Delta = 2mm$

$d_o = d + \Delta = 18 \cdot mm$

$1.2d_o \le e_1 \le 4t + 40mm = 21.6 \cdot mm \le e_1 \le 80 \cdot mm$

Adopted distance $e_1 = 70mm$

Spacing p_2 of anchors along y-y axis

$2.4d_o \le p_2 \le \min(14t, 200mm) = 43.2 \cdot mm \le p_2 \le \min(140 \cdot mm, 200 \cdot mm)$

Adopted spacing $p_2 = 100mm$

(dimensions e_1 and p_2 where calculated assuming that t = 10mm)

11.2. Finding base plate

For calculations we adopted thickness of trapezoidal metal sheet $t_g = 10mm$

Dimensions of base plate

length $D_p = d_p + 2 \cdot e_1 = 250 \cdot mm + 2 \cdot 70 \cdot mm \rightarrow D_p = 390 \cdot mm$

with $B_p = b + 2 \cdot t_g + 2 \cdot c_1 = 160 \cdot mm + 2 \cdot 10 \cdot mm + 2 \cdot 80 \cdot mm \rightarrow B_p = 340 \cdot mm \rightarrow B_p = 340mm$

Thickness of base

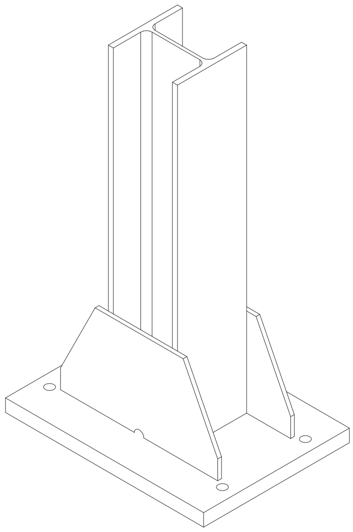
$$t_p \geq c \cdot \sqrt{\frac{3 \cdot f_{jd} \cdot \gamma_{M0}}{f_y}}$$

Overhang of substitutional support

$c = 0.5 \cdot b = 80 \cdot mm$

Class of foundation

C20/25 $\rightarrow f_{ck} = 20MPa$



$$f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_c} = \frac{1.0 \cdot 20 \cdot MPa}{1.5} \rightarrow f_{cd} = 13.33\ MPa$$

Designed strength on compression

$$f_{jd} = \alpha \cdot \beta \cdot f_{cd} = 1.5 \cdot \frac{2}{3} \cdot \frac{1.0 \cdot 20 \cdot MPa}{1.5} \rightarrow f_{jd} = 13.33\ MPa$$

Base plate is from steel S275

$f_{y.p} = 275MPa$

$$t_{p.min} = c \cdot \sqrt{\frac{3 \cdot f_{jd} \cdot \gamma_{M0}}{f_{y.p}}} = 80 \cdot mm \cdot \sqrt{\frac{3 \cdot 13.33 \cdot MPa \cdot 1.0}{275 \cdot MPa}} \rightarrow t_{p.min} = 30.507 \cdot mm$$

Adopted thickness of base plate $t_p = 35mm$

11.3. Length of compressive zone

Step 1.

$$x_p = \frac{\frac{M_{Ed}}{d_p} + \frac{N_{Ed}}{2}}{B_p \cdot f_{jd}} = \frac{\frac{53.65 \cdot kN \cdot m}{250 \cdot mm} + \frac{260.36 \cdot kN}{2}}{340 \cdot mm \cdot 13.33 \cdot MPa} \rightarrow x_p = 76.073 \cdot mm$$

$$l_a = D_p - e_1 - 0.5 \cdot x_p = 390 \cdot mm - 70 \cdot mm - 0.5 \cdot 76.073 \cdot mm \rightarrow l_a = 281.964 \cdot mm$$

Step 2.

$$x_p = \frac{\frac{M_{Ed}}{l_a} + \frac{N_{Ed}}{2}}{B_p \cdot f_{jd}} = \frac{\frac{53.65 \cdot kN \cdot m}{281.964 \cdot mm} + \frac{260.36 \cdot kN}{2}}{340 \cdot mm \cdot 13.33 \cdot MPa} \rightarrow x_p = 70.706 \cdot mm$$

$$l_a = D_p - e_1 - 0.5 \cdot x_p = 390 \cdot mm - 70 \cdot mm - 0.5 \cdot 70.706 \cdot mm \rightarrow l_a = 284.647 \cdot mm$$

Because new l_a differs from last one less then 1%, we can calculate new values of x_p and l_a.

$$x_p = \frac{\frac{M_{Ed}}{l_a} + \frac{N_{Ed}}{2}}{B_p \cdot f_{jd}} = \frac{\frac{53.65 \cdot kN \cdot m}{284.647 \cdot mm} + \frac{260.36 \cdot kN}{2}}{340 \cdot mm \cdot 13.33 \cdot MPa} \rightarrow x_p = 70.31 \cdot mm$$

$$l_a = D_p - e_1 - 0.5 \cdot x_p = 390 \cdot mm - 70 \cdot mm - 0.5 \cdot 70.31 \cdot mm \rightarrow l_a = 284.845 \cdot mm$$

11.4. Checking the anchors

Finding the tensional force applied on fundamental anchor

$$F_{bt} = \frac{M_{Ed} - N_{Ed} \cdot \left(0.5D_p - 0.5x_p\right)}{n \cdot l_a} = \frac{53.65 \cdot kN \cdot m - 260.36 \cdot kN \cdot (0.5 \cdot 390 \cdot mm - 0.5 \cdot 70.31 \cdot mm)}{2 \cdot 284.845 \cdot mm}$$

$F_{bt} = 21.122\,kN$

$F_{bt} = 21.122\,kN$ < $F_{T.Rd} = 60.782\,kN$ - condition satisfied

11.5. Finding trapezoidal metal sheet

Distance between net force of ground pressure and 1 node steel plate

$$F_g = f_{jd} \cdot \frac{B_p}{2} \cdot x_p = 13.33 \cdot MPa \cdot \frac{340 \cdot mm}{2} \cdot 70.31 \cdot mm \rightarrow F_g = 159.329\,kN$$

Length of node steel plate

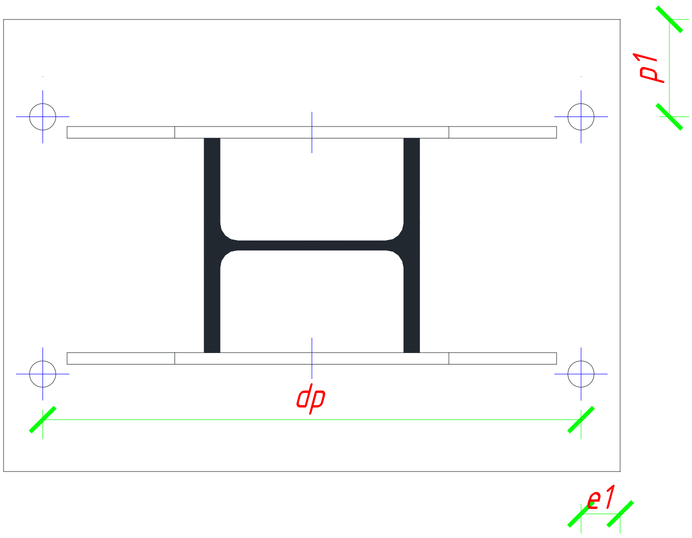
$$L_g = \frac{D_p - 2 \cdot x - h}{2} = \frac{390 \cdot mm - 2 \cdot 10 \cdot mm - 160 \cdot mm}{2} \rightarrow L_g = 105 \cdot mm$$

Finding weld between trapezoidal metal sheet and I beam (according to construction measures)

$a(t_g;t_f) = a(10;13)$

$0.2 \cdot t_f \le a \le 0.7 \cdot t_g = 2.6 \cdot mm \le a \le 7.0 \cdot mm$

Adopted $a = 7mm$



Height of trapezoidal metal sheet

weld stresses

$$f_{vw.d} = \frac{f_u}{\sqrt{3}\cdot\beta_w\cdot\gamma_{M2}} = \frac{430\cdot MPa}{\sqrt{3}\cdot 0.9\cdot 1.25} \quad \rightarrow \quad f_{vw.d} = 220.676\,MPa$$

Max compressive force on the edge of I-beam

$$F_{c.Ed} = \frac{M_{Ed}}{h} + \frac{N_{Ed}}{2} = \frac{53.65\cdot kN\cdot m}{160\cdot mm} + \frac{260.36\cdot kN}{2}$$

$$F_{c.Ed} = 465.493\,kN$$

$$H_g \geq \frac{F_{c.Ed}}{f_{vw.d}\cdot n\cdot a} = \frac{465.493\cdot kN}{220.676\cdot MPa\cdot 2\cdot 7\cdot mm} \quad \rightarrow \quad H_{g.min} = 150.671\cdot mm$$

Adopted height of trapezoidal metal sheet $H_g = 200mm$

Width of trapezoidal metal sheet

$$B_g \geq \frac{L_g}{\sqrt{1+\left(\frac{L_g}{H_g}\right)^2}} = \frac{105\cdot mm}{\sqrt{1+\left(\frac{105\cdot mm}{200\cdot mm}\right)^2}} \quad \rightarrow \quad B_{g.min} = 92.967\cdot mm$$

Adopted width of trapezoidal metal sheet $B_g = 200mm$

Thickness of trapezoidal metal sheet

Distance between net force of ground pressure and I beam

$$S_g = \frac{D_p - h}{2} - \frac{x_p}{2} = \frac{390\cdot mm - 160\cdot mm}{2} - \frac{70.31\cdot mm}{2} \quad \rightarrow \quad S_g = 79.845\cdot mm$$

$$t_g = \frac{2\cdot F_g\cdot S_g}{\frac{f_y}{\gamma_{M1}}\cdot B_g^2} + \frac{B_g}{80} = \frac{2\cdot 159.329\cdot kN\cdot 79.845\cdot mm}{\frac{275\cdot MPa}{1.0}\cdot(200\cdot mm)^2} + \frac{200\cdot mm}{80} \quad \rightarrow \quad t_g = 4.813\cdot mm$$

Adopted thickness of trapezoidal metal sheet $t_g = 10mm$

Slenderness of trapezoidal metal sheet

$$\frac{L_g}{i_g} = \frac{2\cdot\sqrt{3}\cdot B_g}{t_g} = 69.282 \quad < 185$$ - condition satisfied

11.6. Welds connecting node steel plate with base plate

Finding the thickness of weld:
$a_1(t_g;t_p) = a_1(10;35)$

$0.2 \cdot t_p \le a_1 \le 0.7 \cdot t_g = 7.0 \cdot mm \le a_1 \le 7.0 \cdot mm$ oraz $a_1 \ge 3mm$

assumption: $a_{1w} = 7mm$

Cross-section area of welds
Assumption: In calculation we do not consider welds connecting node steel plate with base plate placed between profile shelves due to hardships of welding.

$$A_w = 2 \cdot a_1 \cdot (D_p - 2 \cdot x) + 4 \cdot a_1 \cdot L_g = 2 \cdot 7 \cdot mm \cdot (390 \cdot mm - 2 \cdot 10 \cdot mm) + 4 \cdot 7 \cdot mm \cdot 105 \cdot mm \rightarrow A_w = 8120 \cdot mm^2$$

Moment of inertia of

$$J_{wy} = 2 \cdot \frac{a_1 \cdot (D_p - 2 \cdot x)^3}{12} + 4 \cdot \frac{a_1 \cdot L_g^3}{12} + a_1 \cdot L_g \cdot \left(\frac{L_g}{2} + \frac{h}{2}\right)^2$$

$$J_{wy} = 2 \cdot \frac{7 \cdot mm \cdot (390 \cdot mm - 2 \cdot 10 \cdot mm)^3}{12} + 4 \cdot \frac{7 \cdot mm \cdot (105 \cdot mm)^3}{12} + 7 \cdot mm \cdot 105 \cdot mm \cdot \left(\frac{105 \cdot mm}{2} + \frac{160 \cdot mm}{2}\right)^2$$

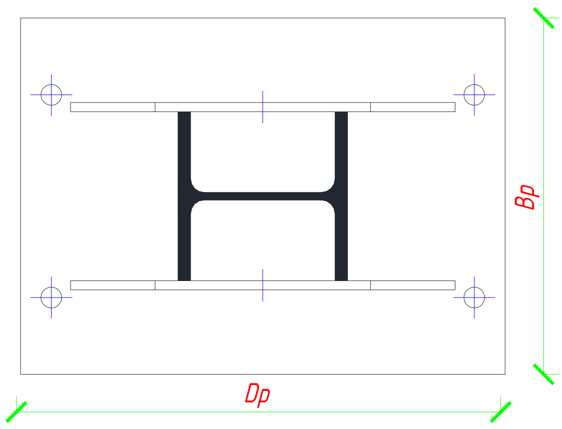
$$J_{wy} = 74700135.417\ mm^4$$

Stresses:

$$\sigma = \frac{N_{Ed}}{A_w} + \frac{M_{Ed} \cdot 0.5 \cdot (D_p - x)}{J_{wy}} = \frac{260.36 \cdot kN}{8120 \cdot mm^2} + \frac{53.65 \cdot kN \cdot m \cdot 0.5 \cdot (390 \cdot mm - 10 \cdot mm)}{74700135.417 \cdot mm^4} \rightarrow \sigma = 168.523\ MPa$$

$$\sigma_{pr} = \tau_{pr} = \frac{\sigma}{\sqrt{2}} = 119.164\ MPa$$

$$\tau_{par} = \frac{V_{Ed}}{A_w} = \frac{25.47 \cdot kN}{8120 \cdot mm^2} \rightarrow \tau_{par} = 3.137\ MPa$$



Conditions:

$$\sqrt{\sigma_{pr}^2 + 3\cdot\left(\tau_{pr}^2 + \tau_{równ}^2\right)} \le \frac{f_u}{\beta_w\cdot\gamma_{M2}} \qquad \text{and} \qquad \sigma_{pr} \le \frac{0.9\cdot f_u}{\gamma_{M2}}$$

f_u - nominal tensional resistance of weaker part of connected

$f_u = 430\,MPa \qquad \beta_w = 0.9 \qquad \gamma_{M2} = 1.25$

$$\sqrt{\sigma_{pr}^2 + 3\cdot\left(\tau_{pr}^2 + \tau_{par}^2\right)} = 238.389\,MPa \quad < \quad \frac{f_u}{\beta_w\cdot\gamma_{M2}} = 382.222\,MPa$$

$$\sigma_{pr} = 119.164\,MPa \quad < \quad \frac{0.9\cdot f_u}{\gamma_{M2}} = 309.600\,MPa$$

→ conditions satisfied

11.7. Additional checking of welds connecting node steel plate with column

Thickness of welds a = 7mm adopted earlier.
In calculations we consider 4 welds connecting the column with node steel plate placed 'outside' the column (welds 'inside' where omitted due to eventual difficulties in welding). Center of gravity is placed in the middle of height of node steel plate and in the middle of height of column.

Geometrical characteristics:

Area of cross-section of weld

$$A_w = 4\cdot a\cdot H_g = 4\cdot 7\cdot mm\cdot 200\cdot mm \quad \rightarrow \quad A_w = 5600\,mm^2$$

Moment of inertia due to x-x

$$J_{wx} = 4\cdot\frac{a\cdot H_g^3}{12} = 4\cdot\frac{7\cdot mm\cdot(200\cdot mm)^3}{12} \quad \rightarrow \quad J_{wx} = 18.667\times 10^6\,mm^4$$

Moment of inertia due to y-y

$$J_{wy} = 4\cdot\left[\frac{a\cdot H_g^3}{12} + H_g\cdot a\cdot(0.5\cdot h)^2\right] = 4\cdot\left[\frac{7\cdot mm\cdot(200\cdot mm)^3}{12} + 200\cdot mm\cdot 7\cdot mm\cdot(0.5\cdot 160\cdot mm)^2\right] \rightarrow \quad J_{wy} = 5.451\times 10^7\,mm^4$$

Moment of

$$J_{wo} = J_{wx} + J_{wy} = 73173333.33\,mm^4$$

Finding distance from the max stresses poin'

$$r = \sqrt{(0.5\cdot h)^2 + \left(0.5\cdot H_g\right)^2} = \sqrt{(0.5\cdot 160\cdot mm)^2 + (0.5\cdot 200\cdot mm)^2} \quad \rightarrow \quad r = 128.062\,mm$$

Stresses in welds:

Normal stresses

$$\tau_N = \frac{N_{Ed}}{A_w} = \frac{260.36\cdot kN}{5600\cdot mm^2} \rightarrow \tau_N = 46.493\ MPa$$

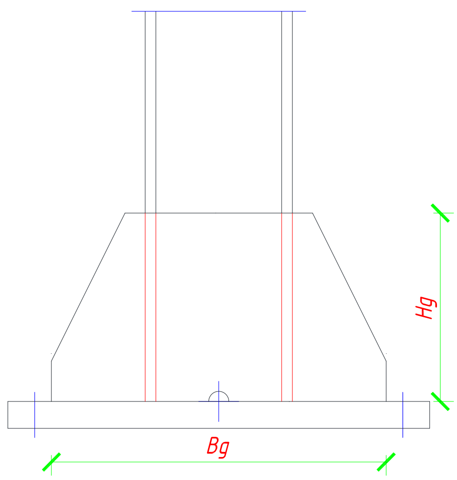
Vertical stresses

$$\tau_V = \frac{V_{Ed}}{A_w} = \frac{25.47\cdot kN}{5600\cdot mm^2} \rightarrow \tau_V = 4.55\ MPa$$

Bending moment stresses

$$\tau_M = \frac{\left[M_{Ed} - V_{Ed}\cdot(0.5\cdot H_g)\right]\cdot r}{J_{wo}} = \frac{(53.65\cdot kN\cdot m - 25.47\cdot kN\cdot 0.5\cdot 200\cdot mm)\cdot\sqrt{(0.5\cdot 160\cdot mm)^2 + (0.5\cdot 200\cdot mm)^2}}{73173333.33\cdot mm^4}$$

$$\tau_M = 89.437\ MPa$$



$$\alpha = atan\left(\frac{h}{2r}\right) = atan\left(\frac{160\cdot mm}{2\cdot 15\cdot mm}\right) \qquad \alpha = 79.38\cdot deg$$

$$\tau_{Mx} = \tau_M\cdot\cos(\alpha) = 16.482\ MPa$$

$$\tau_{My} = \tau_M\cdot\sin(\alpha) = 87.905\ MPa$$

Net stresses

$$\tau_w = \sqrt{(\tau_N + \tau_{My})^2 + (\tau_V + \tau_{Mx})^2} = 136.033\ MPa$$

Condition

$$\tau_w \le f_{v.wd}$$

$$f_{v.wd} = \frac{f_u}{\sqrt{3}\cdot\beta_w\cdot\gamma_{M2}} = \frac{430\cdot MPa}{\sqrt{3}\cdot 0.9\cdot 1.25} \rightarrow f_{v.wd} = 220.676\ MPa$$

$\tau_w = 136.033\ MPa$ < $f_{v.wd} = 220.676\ MPa$ - condition satisfied