IPE300-H=488 Properties :

$b = 0.15\ m$ $t_f = 0.0107\ m$

$h = 0.4875\ m$ $t_w = 0.0071\ m$

$A = 0.00661437\ m^2$

$I_x = 0.00024744\ m^4$

$I_y = 6.0364E\text{-}06\ m^4$

$S_x = 8.04854E\text{-}05\ m^3$

$S_y = 8.04854E\text{-}05\ m^3$

$Z_x = 0.00117278\ m^3$

$Z_y = 0.00012681\ m^3$

$i_x = 0.193415\ m$

$i_y = 0.0302096\ m$

$C_w = 3.43076E\text{-}07\ m^6$

$J_c = 2.23554E\text{-}07\ m^4$

Material Properties: S275

F_y :Yield stress = 275000 kN/m²

F_u :Ultimate stres = 430000 kN/m²

E :Elasticity modulus = 2.000E+8 kN/m²

G : Shear modulus = 7.692E+7 kN/m²

The psi value of the moment diagram, as per Table 18

psi: 0.838901135592712

The c1, c2 and c3 constants as per Table 42 (Annex E), used in Clause E-1.2

Bending Moment Diagram Type: Type1

c1: 1.000

c2: 0.000

c3: 1.000

Calculation of the Elastic Critical Moment as per Annex E

$$M_{cr}=c_1\frac{\pi^2 EI_y}{L_{LT}}\left\{\left[\left(\frac{K}{K_w}\right)^2\frac{I_w}{I_y}+\frac{GI_t(L_{LT})^2}{\pi^2 EI_y}+(c_2y_g-c_3y_j)\right]^{0.5}-(c_2y_g-c_3y_j)\right\}$$

$$c_1\frac{\pi^2 EI_y}{L_{LT}}=1\frac{\pi^2\cdot 2.000E+8\cdot 6.04E\text{-}06}{1.72}=6935.77$$

$$\left(\frac{K}{K_w}\right)^2\frac{I_w}{I_y}=\left(\frac{1}{1}\right)^2\frac{3E\text{-}07}{6.04E\text{-}06}=0.06$$

$$\frac{GI_t(L_{LT})^2}{\pi^2 EI_y}=\frac{7.692E+7\cdot 2.24E\text{-}07\cdot(1.72)^2}{\pi^2\cdot 2.000E+8\cdot 6.04E\text{-}06}=0.00426$$

$$(c_2y_g-c_3y_j)=0.0\cdot 0.0-1\cdot 0.0=0.0$$

$$M_{cr}=6935.77\{\,[0.06+0.00426+0.0]^{0.5}-0.0\}$$

$$M_{cr}=1714.33\ kN.m$$

Check against flexure and axial force

Load Combinations

$ComboType=ULS$

$Combo\ Name=$ 1.5xD+1.5xSELF+1.5xPUR+1.5xSHE+1.5xSNOW1

Station

Loc :	0.859	m
N :	-107.74	kN
M33 :	-209.18	m.kN
V22 :	0.0	kN
M22 :	0.0	m.kN
V33 :	80.8	kN
T :	0.0	kN

Classifies a H Section bending about ZZ axis

Type of extreme section fiber = Top

$$\sigma=\frac{P}{A}+\frac{M_3}{W_{ely}}=\frac{-107.74}{0.00661437249999999}+\frac{-209.18}{0.00102}=-222353.25\ kN/m^2$$

$-222353.25<0\ \Rightarrow$ Compression

Rolled Section flange

$ratio\le 9.4\cdot\varepsilon=7.01\le 9.4\cdot 0.953\Rightarrow 1$

Type of extreme section fiber = Top

$$\sigma=\frac{P}{A}+\frac{M_3}{W_{ely}}=\frac{-107.74}{0.00661437249999999}+\frac{-209.18}{0.00102}=-222353.25\ kN/m^2$$

$-222353.25<0\ \Rightarrow$ Compression

$$d = h - t_f = 0.4875 - 0.0107 = 0.466$$

$$r_1 = \frac{\frac{N}{ratio_{Area}}}{d \cdot t_w \cdot f_y \cdot \gamma_0} = \frac{\frac{107.74}{1}}{0.466 \cdot 0.0071 \cdot 275000 \cdot 1.1} = 0.13$$

$$r_2 = \frac{\frac{N}{ratio_{Area}}}{Area \cdot f_y \cdot \gamma_0} = \frac{\frac{107.74}{1}}{0.00661 \cdot 275000 \cdot 1.1} = 0.0652$$

Classifies a H Section bending about YY axis

$Type\ of\ extreme\ section\ fiber = Left$

$$\sigma = \frac{P}{A} - \frac{M_2}{W_{elz}} = \frac{-107.74}{0.00661437249999999} - \frac{0.0}{8.05E\text{-}05} = -16289.06\ kN/m^2$$

$-16289.06 < 0 \Rightarrow Compression$

$Type\ of\ extreme\ section\ fiber = Right$

$$\sigma = \frac{P}{A} + \frac{M_2}{W_{elz}} = \frac{-107.74}{0.00661437249999999} + \frac{0.0}{8.05E\text{-}05} = -16289.06\ kN/m^2$$

$-16289.06 < 0 \Rightarrow Compression$

$Rolled\ Section\ \ flange$

$ratio \leq 9.4 \cdot \varepsilon = 7.01 \leq 9.4 \cdot 0.953 \Rightarrow 1$

$$d = h - t_f = 0.4875 - 0.0107 = 0.466$$

$$r_1 = \frac{\frac{N}{ratio_{Area}}}{d \cdot t_w \cdot f_y \cdot \gamma_0} = \frac{\frac{107.74}{1}}{0.466 \cdot 0.0071 \cdot 275000 \cdot 1.1} = 0.13$$

$$r_2 = \frac{\frac{N}{ratio_{Area}}}{Area \cdot f_y \cdot \gamma_0} = \frac{\frac{107.74}{1}}{0.00661 \cdot 275000 \cdot 1.1} = 0.0652$$

Checks whether the calculated effective slenderness ratio Is within the allowable limits ot Table 3

MemberType: No1 max(KL/r)=180

(A member carrying compressive loads resulting from dead loads and imposed loads)

$$\left(\frac{KL}{r}\right)_{z\text{-}z} = \frac{k_z L_z}{i_z} = \frac{k_z L_z}{\sqrt{\frac{I_z}{A_g}}} = \frac{1 \cdot 1.72}{\sqrt{\frac{6.04E\text{-}06}{0.00661}}} = 8.88 \leq 180\ \ Satisfactory\ \checkmark o.k.$$

$$\left(\frac{KL}{r}\right)_{y\text{-}y} = \frac{k_y L_y}{i_y} = \frac{k_y L_y}{\sqrt{\frac{I_y}{A_g}}} = \frac{1 \cdot 1.72}{\sqrt{\frac{0.000247}{0.00661}}} = 56.87 \leq 180\ \ Satisfactory\ \checkmark o.k.$$

COMPRESSION CHECK AS PER SECTION 7

- About Z-Z

Buckling Class (*IPE300-H=488*) $-$ *Table10* : 1

α *from Table 7* (1) = 0.21

Euler buckling stress (f_{cc}) *from Section* 7.1.2.1

$$f_{cc}=\frac{\pi^2 E}{(KL/r)^2_{z\text{-}z}}=\frac{\pi^2 2.000E+8}{(8.88)^2}=2.502E+7$$

$$\lambda=\sqrt{\frac{f_y}{f_{cc}}}=\sqrt{\frac{275000}{2.502E+7}}=0.105$$

$$\Phi=0.5[1+\alpha(\lambda-0.2)+\lambda^2]=0.5[1+0.21(0.105-0.2)+0.105^2]=0.496$$

$$\chi=\frac{1}{[\Phi+(\Phi^2-\lambda^2)^{0.5}]}=\frac{1}{[0.496+(0.496^2-0.105^2)^{0.5}]}=1.02$$

$$f_{cd}=\min\{\chi f_y/\gamma_{m0}, f_y/\gamma_{m0}\}=\min\{1.02\cdot 275000/1.1, sFys/sGama\}=250000\ kN/m^2$$

$$P_d=A_e\cdot f_{cd}=0.00661\cdot 250000=1653.59\ kN$$

$$Ratio_{z\text{-}z}=\frac{P}{Pd}=\frac{107.74}{1653.59}=0.0652\leq 1.0 \Rightarrow Satisfactory$$ ✔ o.k.

- About Y-Y

Buckling Class (*IPE300-H=488*) $-$ *Table10* : 2

α *from Table 7* (2) = 0.34

Euler buckling stress (f_{cc}) *from Section* 7.1.2.1

$$f_{cc}=\frac{\pi^2 E}{(KL/r)^2_{y\text{-}y}}=\frac{\pi^2 2.000E+8}{(56.9)^2}=610369.61$$

$$\lambda=\sqrt{\frac{f_y}{f_{cc}}}=\sqrt{\frac{275000}{610369.61}}=0.671$$

$$\Phi=0.5[1+\alpha(\lambda-0.2)+\lambda^2]=0.5[1+0.34(0.671-0.2)+0.671^2]=0.805$$

$$\chi=\frac{1}{[\Phi+(\Phi^2-\lambda^2)^{0.5}]}=\frac{1}{[0.805+(0.805^2-0.671^2)^{0.5}]}=0.8$$

$$f_{cd}=\min\{\chi f_y/\gamma_{m0}, f_y/\gamma_{m0}\}=\min\{0.8\cdot 275000/1.1, sFys/sGama\}=199926.57\ kN/m^2$$

$$P_d=A_e\cdot f_{cd}=0.00661\cdot 199926.57=1653.59\ kN$$

$$Ratio_{y\text{-}y}=\frac{P}{Pd}=\frac{107.74}{1653.59}=0.0652\leq 1.0 \Rightarrow Satisfactory$$ ✔ o.k.

Vn: SHEAR RESISTANCE IN Y DIRECTION, AS PER 8.4.1

Section Name= IPE300-H=488

web Depth (d)= 0.4661 m

web Thickness= 0.0071 m

$$V_{p,y}=\frac{A_v\cdot F_{yw}}{\sqrt{3}}=\frac{0.0038\cdot 275000}{\sqrt{3}}=603.54\ kN$$

Transverse Stiffeners Provided Only At Supports

$K_{w,y} = 5.35$

Resistance to shear buckling shall be verified, as per 8.4.2.1

Frame Element Web Has No Stiffeners

$\varepsilon = \sqrt{250000/f_y} = \sqrt{250000/275000} = 0.953$

$\frac{d}{t_w} = \frac{0.466}{0.0071} = 65.6 > 67\,\varepsilon \Rightarrow Consider$

Non-dimensional web slendernes ratio for buckling stress

$\tau_{cr,y} = \frac{K_{w,y} \cdot \pi^2 \cdot E}{12(1-\mu^2)(d/t_w)^2} = \frac{5.35 \cdot \pi^2 \cdot 2.000E+8}{12(1-smu^2)(0.466/0.0071)^2} = 224398.2$

$\lambda_{w,y} = \sqrt{f_{yw}(\sqrt{3}\tau_{cr,y})/} = \sqrt{sfyw(\sqrt{3} \cdot 224398.2)/} = 5.35$

Shear stress correspoing to web buckling

$\lambda_{w,y} > 1.2 \Rightarrow \tau_{b,y} = f_{yw}(\sqrt{3} \cdot \lambda_{w,y}^2)/ = 275000\sqrt{3} \cdot 0.841 = 153543.86/$

$V_{cr,y} = A_w \cdot \tau_{b,y} = sAw \cdot 153543.86 = 583.67$

Design shear strength in Y direction as per 8.4.1 and 8.4.2

$V_n = \min\{V_p, V_{cr}\} = \min\{sVp, sVcr\} = 583.67\ kN$

Design shear ratio in Y direction

$Shear_{ratio,y} = \frac{V}{V_n} = \frac{80.8}{530.61} = 0.152 \leq 1.0 \Rightarrow Satisfactory$ ✔ o.k.

Design bending strength of the section (Mdz) 8.2.1.2

Section class for flexure about ZZ axis: 4

$\beta_b = \frac{W_{el\ y}}{W_{pl,y}}\ 0.866\ \textit{for semi compact sections}$

d / tw > 67 ==>ε

$M_{d,y} = \beta_b \cdot W_{pl,y} \cdot f_y / \gamma_M$

$M_{d,y} = 0.866 \cdot 0.000765 \cdot 275000/1.1 = 165.6\ kN.m$

Bending ration about ZZ axis

$Bending_{ratio,z} = M_{3\text{-}3}/M_{d,z} = M_{dv,z} = -209.18/165.6 = 1.26 > 1.0$ Not satisfactory ✘

Calculation of the resistance against Lateral Torsional Buckling (MdLTB) 8.2.2

$\alpha_{LT} = 0.21$ *for rolled steel sections*

$\lambda_{LT} = \min\{\sqrt{\beta_b \cdot W_{pl,y} \cdot f_y / M_{cr}}, \sqrt{1.2 \cdot W_{el,y} \cdot f_y / M_{cr}}\}$

$\lambda_{LT} = \min\{\sqrt{0.866 \cdot 0.00117 \cdot 275000 / sMcr}, \sqrt{1.2 \cdot 0.00102 \cdot 275000 / 1714.33}\} = 0.404$

$\Phi_{LT} = 0.5[1 + \alpha_{LT} \cdot (\lambda_{LT} - 0.2) + \lambda_{LT}^2] = 0.5[1 + 0.21 \cdot (0.404 - 0.2) + 0.404^2] = 0.603$

$$\chi_{LT} = \min\left\{\frac{1}{\{\Phi_{LT}[\Phi_{LT}^2 - \lambda_{LT}^2]\}^{0.5}}, 1\right\} = \min\left\{\frac{1}{\{0.603[0.603^2 - 0.404^2]\}^{0.5}}, 1\right\} = 0.952$$

$f_{bd} = \chi_{LT} \cdot f_y / \gamma_{M0} = 0.952 \cdot 275000 / 1.1 = 237963.13\ kN/m^2$

$M_{d,LTB} = \beta_b \cdot W_{pl,y} \cdot f_{bd} = 0.866 \cdot 0.00117 \cdot 237963.13 = 241.57\ kN.m$

$LTB_{ratio,z} = M_{3\text{-}3} / M_{LTB,z} = -209.18 / 241.57 = 0.866 \leq 1.0$ *Satisfactory* ✔ *o.k.*

Vn: SHEAR RESISTANCE IN Z DIRECTION, AS PER 8.4.1

Section Name= IPE300-H=488

web Depth (d)= 0.15 m

web Thickness= 0.0107 m

$$V_{p,z} = \frac{A_v \cdot F_{yw}}{\sqrt{3}} = \frac{0.00331 \cdot 275000}{\sqrt{3}} = 0.0\ kN$$

Resistance to shear buckling shall be verified, as per 8.4.2.1

Frame Element Web Has No Stiffeners

$\varepsilon = \sqrt{250000 / f_y} = \sqrt{250000 / 275000} = 0.953$

$$\frac{d}{t_w} = \frac{0.15}{0.0107} = 14 \leq 67\ \varepsilon \Rightarrow \textit{Not Consider}$$

Design shear ratio in Z direction

$$Shear_{ratio,z} = \frac{V}{V_n} = \frac{80.8}{477.04} = 0.169 \leq 1.0 \Rightarrow \textit{Satisfactory}\ ✔\ \textit{o.k.}$$

Combined Axial force and Bending moment check regarding overall member strength, as per 9.3.2

$n_y = P / P_{dy} = -107.74 / 1322.39 = 0.0815$

$n_z = P / P_{dz} = -107.74 / 1653.59 = 0.0652$

$K_y = \min\{1 + (\lambda_y - 0.2) \cdot n_y, 1 + 0.8 \cdot n_y\} = \min\{1 + (0.671 - 0.2) \cdot 0.0815, 1 + 0.8 \cdot 0.0815\} = 1.04$

$K_z = \min\{1 + (\lambda_z - 0.2) \cdot n_z, 1 + 0.8 \cdot n_z\} = \min\{1 + (0.105 - 0.2) \cdot 0.0652, 1 + 0.8 \cdot 0.0652\} = 0.994$

$$K_{LT} = \max\left\{1 - \frac{0.1 \cdot \lambda_{LT} \cdot n_y}{C_{mLT} - 0.25}, 1 - \frac{0.1 \cdot n_y}{C_{mLT} - 0.25}\right\} = \max\left\{1 - \frac{0.1 \cdot 0.404 \cdot 0.0815}{0.6 - 0.25}, 1 - \frac{0.1 \cdot 0.0815}{0.6 - 0.25}\right\} = 0.994$$

$$\frac{P}{P_{dy}} + K_y \frac{C_{my} \cdot M_y}{M_{dy}} + K_{LT} \cdot \frac{M_z}{M_{dz}} \leq 1.0$$

$$\frac{-107.74}{1322.39} + 1.04 \frac{0.6 \cdot 0.0}{20.1} + 0.991 \cdot \frac{-209.18}{165.6} = 1.33 > \textit{Not satisfactory}$$ ✘

$$\frac{P}{P_{dz}} + 0.6 \cdot K_y \frac{C_{my} \cdot M_y}{M_{dy}} + K_z \cdot \frac{C_{mz} \cdot M_z}{M_{dz}} \leq 1.0$$

$$\frac{-107.74}{1653.59} + 0.6 \cdot 1.04 \frac{0.6 \cdot 0.0}{20.1} + 0.994 \cdot \frac{0.6 \cdot -209.18}{165.6} = 0.818 > \textit{Satisfactory}$$ ✔ o.k.

Combined Axial force and Bending moment check regarding section strength, as per 9.3.1

In the design of members subjected to combined axial and bending moment-Conservatively

$$\textit{Section Strength}_{ratio} = \frac{N}{N_d} + \frac{M_y}{M_{ndy}} + \frac{M_z}{M_{ndz}} \leq 1.0$$

$$\textit{Section Strength}_{ratio} = \frac{107.74}{1653.59} + \frac{0.0}{20.1} + \frac{209.18}{165.6} > 1.33 \ \textit{Not satisfactory}$$ ✘