REFERENCES	CALCULATIONS	RESULTS

MEMBER #1 DESIGN REPORT

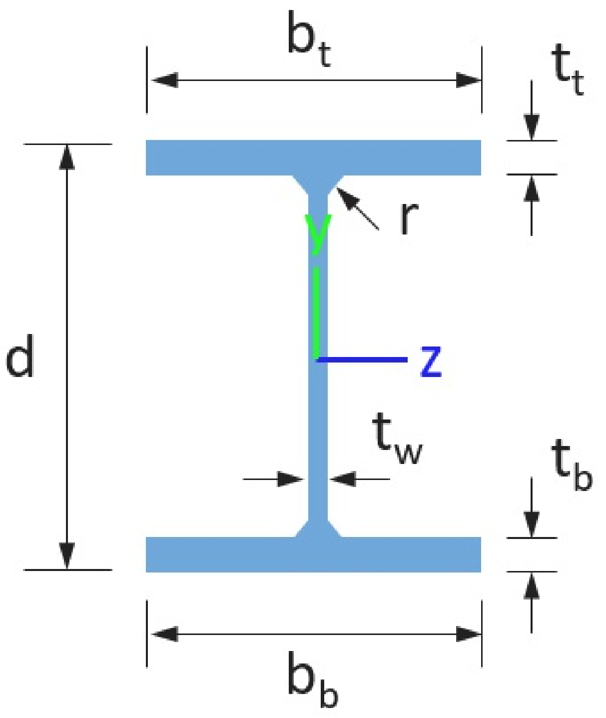
Code: AISC 360-16 LRFD

Project details

Project Name:
Project ID:
Company:
Designer:
Client:
Project Notes:
Project Units: imperial

General member design information

Section Name: W8x58
Shape: I-Beam (Rolled)



Dimensions:
Height $d = 8.750 \text{ in}$
Web Thick $t_w = 0.510 \text{ in}$
Top Flange Width $b_t = 8.220 \text{ in}$
Top Flange Thick $t_t = 0.810 \text{ in}$
Bottom Flange Width $b_b = 8.220 \text{ in}$
Bottom Flange Thick $t_b = 0.810 \text{ in}$
Fillet $r = 0.390 \text{ in}$

Properties:
Area $A = 17.100 \text{ in}^2$
Moment of Inertia about the z-axis $I_z = 228.000 \text{ in}^4$
Moment of Inertia about the y-axis $I_y = 75.100 \text{ in}^4$
Plastic Section Modulus about the z-axis $Z_z = 59.3478 \text{ in}^3$
Plastic Section Modulus about the y-axis $Z_y = 27.829 \text{ in}^3$
Torsion Contant $J = 3.330 \text{ in}^4$
Warping Contant $I_w = 1168.250 \text{ in}^6$

Material properties:
Material Name: Structural Steel
Modulus of Elasticity $E = 29000 \text{ ksi}$
Yield Strength $F_y = 38 \text{ ksi}$
Ultimate Tensile Strength $F_u = 60 \text{ ksi}$

Design parameters:
Member length $L = 10.000 \text{ ft}$
Length between braced points $L_b = 10.000 \text{ ft}$
Effective Length factor for flexural buckling about y-axis $K_y = 2.100$
Effective Length factor for flexural buckling about z-axis $K_z = 2.100$

Load case

Axial Force $P = 5.000 \text{ kip}$
Major Banding Moment $M_z = 86.243 \text{ kip-ft}$
Minor Banding Moment $M_y = 0.000 \text{ kip-ft}$
Shear Force $V_z = 0.000 \text{ kip}$
Shear Force $V_y = 15.582 \text{ kip}$

CHECK AXIAL STRENGTH (axial compression member)

Check slenderness ratio of axial compression member (AISC E2)
Slenderness ratio z-axis

$$\lambda_z = \frac{K_z L}{r_z} = \frac{21.000}{0.304} = 69.013$$

Slenderness ratio y-axis

$$\lambda_y = \frac{K_y L}{r_y} = \frac{21.000}{0.175} = 120.249$$

Maximum slenderness ratio

$$\lambda = \max(\lambda_y, \lambda_z) = \max(69.013, 120.249) = 120.249$$

$$\lambda = 120.249 < 200$$

STATUS OK!

Check width-thickness ratio of flange (B4. Table B4.1a)

$$\lambda_f = \frac{0.5 \cdot b_t}{t_t} = \frac{0.5 \cdot 8.220}{0.810} = 5.074$$

$$\lambda_{rf} = 0.56\sqrt{\frac{E_s}{F_y}} = 0.56\sqrt{\frac{29000}{38}} = 15.470$$

$$\lambda_f = 5.074 < \lambda_{rf} = 15.470 \rightarrow \text{ non-slender section}$$

$$\lambda_w = \frac{d - tt - tb - 2r}{t_w} = \frac{8.750 - 0.810 - 0.810 - 2 \cdot 0.390}{0.510} = 12.451$$

$$\lambda_{rw} = 1.49\sqrt{\frac{E_s}{F_y}} = 1.49\sqrt{\frac{29000}{38}} = 41.162$$

$$\lambda_w = 12.451 < \lambda_{rw} = 41.162 \rightarrow \text{ non-slender section}$$

Calculate Flexural Buckling Stress

Calculate the elastic critical buckling stress F_e .

$$F_e = \frac{\pi^2 E}{\lambda^2} = \frac{\pi^2 \cdot 29000}{120.248^2} = 19.794 \text{ ksi}$$

Calculate the flexural buckling stress F_{cr} .

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{38}} = 130.115$$

Because:

$$\lambda = 120.248 < 130.115$$

$$F_{cr} = [0.658^{\frac{F_y}{F_e}}]F_y = [0.658^{\frac{38}{19.794}}] \cdot 38 = 17.015 \text{ ksi}$$

Nominal Compressive Strength P_n .

$$P_n = F_{cr} A_g = 17.015 \cdot 17.100 = 290.951 \text{ kip}$$

Calculate axial compressive strength.

$$\text{Resistance factor for compression: } \phi_c = 0.900$$

$$\phi_c P_n = 0.900 \cdot 290.951 = 261.856 \text{ kip}$$

Calculate the flexural buckling stress F_{cr} .

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{38}} = 130.115$$

Because:

$$\lambda = 120.248 < 130.115$$

$$F_{cr} = [0.658^{\frac{F_y}{F_e}}]F_y = [0.658^{\frac{38}{19.794}}] \cdot 38 = 17.015 \text{ ksi}$$

Check ratio of axial strength $\frac{P}{\phi_c P_n}$

$$\frac{P}{\phi_c P_n} = \frac{5.000}{261.856} = 0.019 < 1.0$$

STATUS OK!

CHECK FLEXURAL STRENGTH ABOUT MAJOR AXIS

Calculate limiting width-thickness ratio of flange for flexure (AISC B4.Table B4.1b)

$$\lambda_f = \frac{0.5 \cdot b_t}{t_t} = \frac{0.5 \cdot 8.220}{0.810} = 5.074$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29000}{38}} = 10.498$$

$$\lambda_{rf} = 1.00\sqrt{\frac{E}{F_y}} = 1.00\sqrt{\frac{29000}{38}} = 27.625$$

$$\lambda_f = 5.074 < \lambda_{pf} = 10.498 \rightarrow \text{ COMPACT}$$

Calculate limiting width-thickness ratio of web for flexure

$$\lambda_w = \frac{d - tt - tb - 2r}{t_w} = \frac{8.750 - 0.810 - 0.810 - 2 \cdot 0.390}{0.510} = 12.451$$

$$\lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29000}{38}} = 103.871$$

$$\lambda_{rw} = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29000}{38}} = 157.464$$

$$\lambda_w = 12.451 < \lambda_{pw} = 103.871 \rightarrow \text{ COMPACT}$$

Calculate lateral-torsional buckling modification factor

$$C_b = 2.230$$

Yielding

Calculate nominal flexural strength for Yielding (AISC F2.1 (F2-1))

$$M_{n1} = M_p = F_y Z_x = 38.000 \cdot 59.348 = 187.18300141350002 \text{ k-ft}$$

Lateral-Torsional Buckling

Compute limiting laterally unbraced length for the limit state of yielding F2.2 (F2-5)

$$L_p = 1.76 r_y\sqrt{\frac{E}{F_y}} = 1.76 \cdot 2.096 \cdot \sqrt{\frac{29000}{38}} = 101.892 \text{ in}$$

Laterally unbraced length for the limit state of inelastic lateral-torsional buckling F2.2 (F2-6)
For doubly symmetric I-shapes F2-8a

$$c = 1.00$$

Distance between the flange centroid

$$h_0 = d - 0.5(tt + tb) = 8.750 - 0.5(0.810 + 0.810) = 7.940 \text{ in}$$

$$r_{ts} = \sqrt[4]{\frac{I_y C_w}{S_z^2}} = \sqrt[4]{\frac{75.100 \cdot 1168.250}{52.114^2}} = 2.384 \text{ in}$$

$$L_r = 1.95 r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_z h_0} + \sqrt{\left(\frac{Jc}{S_z h_0}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}}$$

$$L_r = 1.95 \cdot 2.384 \cdot \frac{29000.000}{0.7 \cdot 38.000}\sqrt{\frac{3.330 \cdot 1.000}{52.114 \cdot 7.940} + \sqrt{\left(\frac{3.330 \cdot 1.000}{52.114 \cdot 7.940}\right)^2 + 6.76\left(\frac{0.7 \cdot 38.000}{29000.000}\right)^2}}$$

$$= 649.879 \text{ in}$$

Calculate nominal flexural strength for Lateral-torsional buckling F2.2 (F2-2)
Because:

$$L_b = 120.000 > L_p = 101.892 \text{ and } L_b = 120.000 < L_r = 649.879$$

then

$$M_{n2} = C_b\left[M_p - (M_p - 0.7F_yS_z)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]$$

$$M_{n2} = 2.230\left[2255.217 - (2255.217 - 0.7 \cdot 38.000 \cdot 52.114)\left(\frac{120.000 - 101.892}{649.879 - 101.892}\right)\right] = 413.757 \text{ k-ft}$$

Nominal flexural strength about major axis M_{nz}.

$$M_{nz} = \min(M_{n1}, M_{n2}) = \min(187.934, 413.757) = 187.934 \text{ k-ft}$$

Calculate flexural strength about major axis

$$\text{Resistance factor for flexure: } \phi_b = 0.900$$

$$\phi_b M_{nz} = 0.900 \cdot 187.934 = 169.141 \text{ k-ft}$$

Check ratio of shear strength $\frac{M_z}{\phi_b \mathrm{M}_{nz}}$

$$\frac{\mathrm{M}_z}{\phi_b \mathrm{M}_{nz}} = \frac{86.243}{169.141} = 0.510 < 1.0$$

STATUS OK!

CHECK FLEXURAL STRENGTH ABOUT MINOR AXIS

Calculate limiting width-thickness ratio of flange for flexure (AISC B4.Table B4.1b)

$$\lambda_f = \frac{0.5 \cdot b_t}{t_t} = \frac{0.5 \cdot 8.220}{0.810} = 5.074$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29000}{38}} = 10.498$$

$$\lambda_{rf} = 1.00\sqrt{\frac{E}{F_y}} = 1.00\sqrt{\frac{29000}{38}} = 27.625$$

$$\lambda_f = 5.074 < \lambda_{pf} = 10.498 \rightarrow \text{ COMPACT}$$

Calculate limiting width-thickness ratio of web for flexure

$$\lambda_w = \frac{d - tt - tb - 2r}{t_w} = \frac{8.750 - 0.810 - 0.810 - 2 \cdot 0.390}{0.510} = 12.451$$

$$\lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29000}{38}} = 103.871$$

$$\lambda_{rw} = 5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29000}{38}} = 157.464$$

$$\lambda_w = 12.451 < \lambda_{pw} = 103.871 \rightarrow \text{ COMPACT}$$

Yielding
Calculate nominal flexural strength for Yielding (AISC F6.1 (F6-1))

$$M_{n1} = M_p = F_y Z_y \leq 1.6 F_y S_y = \min(38.000 \cdot 27.829 = 88.124; 1.6 \cdot 38.000 \cdot 18.273 = 92.580) = 88.124 \text{ k-ft}$$

Nominal flexural strength about minor axis M_{ny}.

$$M_{ny} = M_{n1} = 88.124 \text{ k-ft}$$

Calculate flexural strength about minor axis

$$\text{Resistance factor for flexure: } \phi_b = 0.900$$

$$\phi_b M_{ny} = 0.900 \cdot 88.124 = 79.312 \text{ k-ft}$$

Check ratio of shear strength $\frac{M_y}{\phi_b \mathrm{M}_{ny}}$

$$\frac{\mathrm{M}_y}{\phi_b \mathrm{M}_{ny}} = \frac{0.000}{79.312} = 0.000 < 1.0$$

STATUS OK!

CHECK SHEAR STRENGTH Y-AXIS

Nominal shear strength y-axis V_{ny}.

$$V_{ny} = 0.6 F_y A_w C_v = 0.6 \cdot 38.000 \cdot 4.463 \cdot 1.000 = 101.745 \text{ kip}$$

Calculate shear strength y-axis

$$\text{Resistance factor for shear: } \phi_v = 1.000$$

$$\phi_b V_{ny} = 1.000 \cdot 101.745 = 101.745 \text{ k-ft}$$

Check ratio of shear strength $\frac{V_y}{\phi_v V_{ny}}$

$$\frac{V_y}{\phi_v V_{ny}} = \frac{15.582}{101.745} = 0.153 < 1.0$$

STATUS OK!

CHECK SHEAR STRENGTH Z-AXIS

Calculate the web plate buckling coefficient (AISC G2.1).

$$\text{for singly and doubly symmetric shapes loaded in the weak axis: } k_v = 1.2$$

Calculate the web shear coefficient (AISC G2.1)

$$\text{Because: } \frac{h}{t_w} = \frac{8.220}{1.620} = 5.074 \leq 1.10\sqrt{\frac{k_v E}{F_y}} = 1.10 \cdot \sqrt{\frac{1.200 \cdot 29000}{38}} = 33.288$$

$$\text{web shear coefficient } C_v = 1.000$$

Nominal shear strength z-axis V_{nz}.

$$V_{ny} = 0.6 F_y A_w C_v = 0.6 \cdot 38.000 \cdot 13.316 \cdot 1.000 = 303.614 \text{ kip}$$

Calculate shear strength z-axis

$$\text{Resistance factor for shear: } \phi_v = 0.900$$

$$\phi_v V_{nz} = 0.900 \cdot 303.614 = 273.253 \text{ k-ft}$$

Check ratio of shear strength $\frac{V_z}{\phi_v V_{nz}}$

$$\frac{V_z}{\phi_v V_{nz}} = \frac{0.000}{273.253} = 0.000 < 1.0$$

STATUS OK!

	CHECK INTERACTION OF COMBINED STRENGTH Because Pr/Pc<0.2: $$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) = \frac{5.000}{2 \cdot 261.856} + \left(\frac{86.243}{169.141} + \frac{0.000}{79.312}\right) = 0.519 < 1.0$$	**STATUS OK!**