

Steel Beam Design

Steel beam design for the Casablanca Hotel steel-joist replacement. Demands determined assuming the new beam takes all load, non-composite action with concrete slab.

Factored Demands

As determined in "Load Determination section"

$$M_u = 8.12 \text{ kip}\cdot\text{ft}$$

Factored moment from load determination

$$V_u = 3.61 \text{ kip}$$

Factored reaction on connector (factored shear)

$$L = 9 \text{ ft}$$

Beam span

Material And Section Properties

$$E = 29000 \text{ ksi}$$

Steel Young's modulus

$$F_y = 50 \text{ ksi}$$

Steel yield strength

$$\text{Section} = \text{W10X15}$$

AISC W-shape

$$d = 10 \text{ in}, b = 4 \text{ in}$$

Depth and flange width

$$t_f = 0.25 \text{ in}, t_w = 0.25 \text{ in}$$

Flange and web thickness

$$A = 4.41 \text{ in}^2$$

Section area

$$r_y = 0.81 \text{ in}$$

Weak-axis radius of gyration

$$S_x = 13.8 \text{ in}^3, Z_x = 16 \text{ in}^3$$

Elastic and plastic major-axis modulus

$$S_y = 1.45 \text{ in}^3, Z_y = 2.3 \text{ in}^3$$

Elastic and plastic minor-axis modulus

$$h_o = 9.72 \text{ in}, J = 0.104 \text{ in}^4$$

Flange centroid distance and torsional constant

$$I_x = 68.9 \text{ in}^4, I_y = 2.89 \text{ in}^4$$

Major and minor-axis inertia

$$\lambda = 38.5$$

Web slenderness ratio

Bending About Major Axis

$$L_b = 9 \text{ ft}$$

Unbraced length of compression flange

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 2.86 \text{ ft}$$

Limit for yielding

$$C_b = 1, c = 1$$

Moment gradient and I-shape coefficient

$$r_{ts} = \sqrt{I_y \frac{h_o}{2S_x}} = 1.01 \text{ in}$$

Effective radius of gyration

$$L_r = 1.95 r_{ts} \frac{E}{0.7 F_y} \sqrt{\frac{J_c}{S_x h_o} + \sqrt{\left(\frac{J_c}{S_x h_o}\right)^2 + 6.76 \left(0.7 \frac{F_y}{E}\right)^2}} = 8.61 \text{ ft}$$

Limit for inelastic torsional buckling

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{J_c}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} = 32.56 \text{ ksi}$$

Elastic lateral-torsional-buckling stress

$$M_p = F_y Z_x = 66.67 \text{ kip}\cdot\text{ft}$$

Plastic moment

$$M_{n(LTB)} = M_p, \text{ for } L_b \leq L_p$$

LTB moment for yielding range

$$M_{n(LTB)} = C_b \left[M_p - (M_p - 0.7 F_y S_x) \frac{L_b - L_p}{L_r - L_p} \right], \text{ for } L_p < L_b \leq L_r$$

LTB moment for inelastic range

Casablanca Hotel - 16 Steel Joists Replacement

$$M_{n(LTB)} = F_{cr} S_x, \text{ for } L_b > L_r$$

LTB moment for elastic range

$$M_{n(LTB)} = 37.44 \text{ kip}\cdot\text{ft}$$

Lateral-torsional-buckling strength

$$M_n = \min(M_p, M_{n(LTB)}) = 37.44 \text{ kip}\cdot\text{ft}$$

Nominal bending capacity

$$\phi M_n = 0.9 M_n = 33.7 \text{ kip}\cdot\text{ft}$$

Design bending capacity

Flexure

OK

$$M_u: 8.12 \text{ kip}\cdot\text{ft} \quad \phi M_n: 33.7 \text{ kip}\cdot\text{ft}$$

D/C: 0.24

Shear Of Web

$$A_w = dt_w = 2.5 \text{ in}^2$$

Web area

$$\lambda_{lim} = 2.24 \sqrt{\frac{E}{F_y}} = 53.946$$

Limiting web slenderness

$$k_v = 5.34$$

Web shear buckling coefficient

$$C_{v1} = 1$$

Web shear strength coefficient

$$\phi_v = 1$$

Shear reduction factor

$$\phi V_n = \phi_v 0.6 F_y A_w C_{v1} = 75 \text{ kip}$$

Design shear capacity

Shear

OK

$$V_u: 3.61 \text{ kip} \quad \phi V_n: 75 \text{ kip}$$

D/C: 0.05

Deflection

$$\delta_{max} = \frac{L}{240} = 0.45 \text{ in}$$

Maximum allowed deflection

$$\delta = \frac{5}{384} \frac{w L^4}{E I_x} = 0.041 \text{ in}$$

Expected center deflection under service uniform load

Deflection

OK

$$\delta: 0.04 \text{ in} \quad \delta_{max}: 0.45 \text{ in}$$

D/C: 0.09