

Member: Test3  
Project: 123 Maple Street, San Francisco CA  
Level: 2  
Date: 2021-03-28

Firm: ABC Company  
Engineer: Jesse  
Checker: Joey  
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# W10x12 Design Report

## 1 Applied Loading

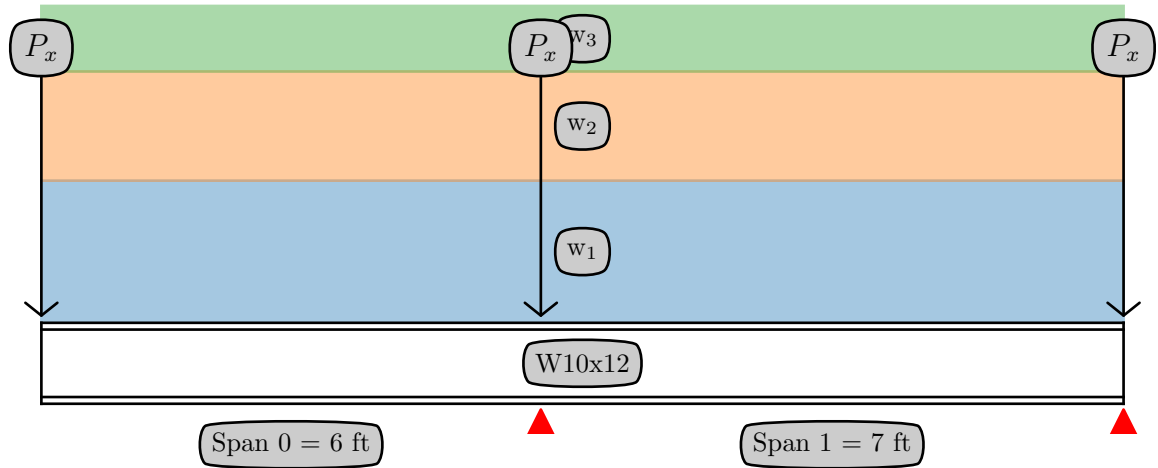


Figure 1: Applied Loads

The following distributed loads are applied to the beam. The program can handle all possible mass and force units in both metric and imperial systems simultaneously. Loads are plotted to scale according to their relative magnitudes. A "positive" load is defined as a load acting in the direction of gravity.

Table 1: Applied Distributed Loads

Load	Start Loc.	Start Mag.	End Loc.	End Mag.	Type	Description
w <sub>1</sub>	0 ft	0.026 klf	13 ft	0.026 klf	D	floor
w <sub>2</sub>	0 ft	0.02 klf	13 ft	0.02 klf	Lr	floor
w <sub>3</sub>	0 ft	12.0 plf	13 ft	12.0 plf	D	Self weight

## 2 Load Combinations

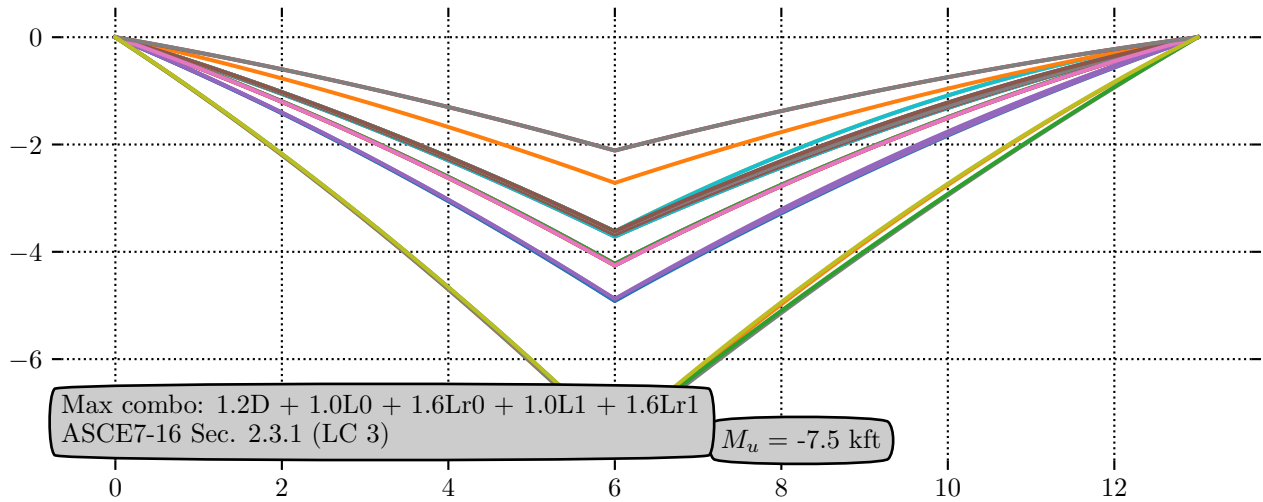
The following load combinations are used for the design. Duplicate load combinations are not listed and only loads that are used on the beam are included in the load combinations (i.e. If soil load is not included as a load type in any of the applied loads, then "H" loads will not be included in the listed load combinations).  $S_{DS}$  is input as 1.0 and  $\Omega_0$  is input as 2.5 for use in seismic load combinations. Any load designated as a pattern load is applied to spans in all possible permutations to create the most extreme loading condition. Numbers after a load indicate the span over which the pattern load is applied (i.e. L0 indicates that live load is applied only on the first span).

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**Table 2:** Applied Point Loads

Load	Loc.	Shear	Type	Description
P <sub>1</sub>	0 ft	0.39 kip	D	No description
P <sub>2</sub>	0 ft	0.34 kip	Lr	No description
P <sub>3</sub>	0 ft	0.01 kip	L	No description
P <sub>4</sub>	6 ft	1.3 kip	D	No description
P <sub>5</sub>	6 ft	0.7 kip	Lr	No description
P <sub>6</sub>	6 ft	0.3 kip	L	No description
P <sub>7</sub>	13 ft	3.57 kip	D	No description
P <sub>8</sub>	13 ft	0.64 kip	Lr	No description
P <sub>9</sub>	13 ft	2.67 kip	L	No description
P <sub>10</sub>	13 ft	0.11 kip	E	No description

### 3 Bending Check



**Figure 2:** Moment Demand Envelope

$L_p$ , the limiting laterally unbraced length for the limit state of yielding, is calculated per AISC/ANSI 360-16 Eq. F2-5 as follows:

$$L_p = 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 1.76 \cdot 0.785 \text{ in} \cdot \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 2.8 \text{ ft}$$

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$r_{ts}$ , a coefficient used in the calculation of  $L_r$  and  $C_b$ , is calculated per AISC/ANSI 360-16 Eq. F2-7 as follows:

$$r_{ts} = \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = \sqrt{\frac{\sqrt{2.18 \text{ in}^4 \cdot 50.9 \text{ in}^6}}{10.9 \text{ in}^3}} = 1.0 \text{ in}$$

$L_r$ , the limiting unbraced length for the limit state of inelastic lateral-torsional buckling, is calculated per AISC/ANSI 360-16 Eq. F2-6 as follows:

$$\begin{aligned} L_r &= 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76 \left(\frac{0.7 \cdot F_y}{E}\right)^2}} \\ &= 1.95 \cdot 1.0 \text{ in} \cdot \frac{29000 \text{ ksi}}{0.7 \cdot 50 \text{ ksi}} \sqrt{\frac{0.05 \text{ in}^4 \cdot 1}{10.9 \text{ in}^3 \cdot 9.7 \text{ in}} + \sqrt{\left(\frac{0.05 \text{ in}^4 \cdot 1}{10.9 \text{ in}^3 \cdot 9.7 \text{ in}}\right)^2 + 6.76 \left(\frac{0.7 \cdot 50 \text{ ksi}}{29000 \text{ ksi}}\right)^2}} \\ &= 8.1 \text{ ft} \end{aligned}$$

$\lambda_{web}$ , the web width-to-thickness ratio, is calculated per AISC/ANSI 360-16 Table B4.1b as follows:

$$\lambda_{web} = \frac{h}{t_w} = \frac{8.85 \text{ in}}{0.19 \text{ in}} = 46.6$$

$\lambda_{P-web}$ , the limiting width-to-thickness ratio for compact/noncompact web, is calculated per AISC/ANSI 360-16 Table B4.1b as follows:

$$\lambda_{P-web} = 3.76 \cdot \sqrt{\frac{E}{F_y}} = 3.76 \cdot \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 90.6$$

$\lambda_{R-web}$ , the limiting width-to-thickness ratio for noncompact/slender web, is calculated per AISC/ANSI 360-16 Table B4.1b as follows:

$$\lambda_{R-web} = 5.7 \cdot \sqrt{\frac{E}{F_y}} = 5.7 \cdot \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 137.3$$

$\lambda_{web} < \lambda_{P-web} \rightarrow$  **Compact Web**

$\lambda_{flange}$ , the flange width-to-thickness ratio, is calculated per AISC/ANSI 360-16 Table B4.1b as follows:

$$\lambda_{flange} = \frac{b}{t} = \frac{1.98 \text{ in}}{0.21 \text{ in}} = 9.4$$

$\lambda_{P-flange}$ , the limiting width-to-thickness ratio for compact/noncompact flange, is calculated per AISC/ANSI 360-16 Table B4.1b as follows:

$$\lambda_{P-flange} = 0.38 \cdot \sqrt{\frac{E}{F_y}} = 0.38 \cdot \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 9.2$$

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$\lambda_{R-flange}$ , the limiting width-to-thickness ratio for noncompact/slender flange, is calculated per AISC/ANSI 360-16 Table B4.1b as follows:

$$\lambda_{R-flange} = 1 \cdot \sqrt{\frac{E}{F_y}} = 1 \cdot \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = \mathbf{24.1}$$

$\lambda_{P-flange} < \lambda_{flange} < \lambda_{R-flange} \rightarrow \mathbf{Noncompact Flange}$

Since  $\mathbf{L_p} < \mathbf{L_b} \leq \mathbf{L_r}$  and the beam's flanges are **noncompact**, controlling limit state for flexure is **LTB (not to exceed capacity based on compression flange local buckling)**.

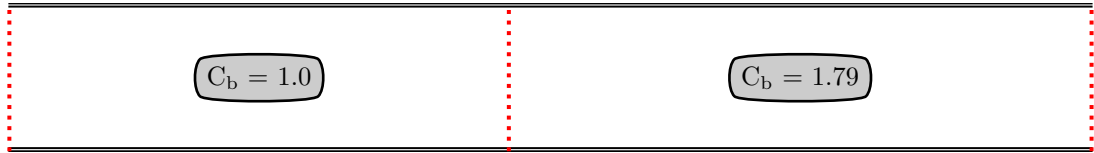
$M_p$ , the plastic bending moment, is calculated per AISC/ANSI 360-16 Eq. F2-1 as follows:

$$M_p = F_y \cdot Z_x = 50 \text{ ksi} \cdot 12.6 \text{ in}^3 = \mathbf{52.5 \text{ kft}}$$

$C_b$ , the lateral-torsional buckling modification factor in the critical unbraced span for the critical load combination, is calculated per AISC/ANSI 360-16 Sec. F1 as follows:

$$C_b =_b f_{1.0dimensionless}$$

For brevity, the  $C_b$  calculation is not shown for each span. The following figure illustrates the value of  $C_b$  for each span.



**Figure 3:**  $C_b$  Along Member

$F_{cr}$ , the buckling stress for the critical section in the critical unbraced span, is calculated per AISC/ANSI 360-16 Eq. F2-4 as follows:

$$\begin{aligned}
 F_{cr} &= \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \cdot \left(\frac{L_b}{r_{ts}}\right)^2} \\
 &= \frac{1.0 \cdot \pi^2 \cdot 29000 \text{ ksi}}{\left(\frac{6 \text{ ft}}{1.0 \text{ in}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{0.0547 \text{ in}^4 \cdot 1}{10.9 \text{ in}^3 \cdot 9.7 \text{ in}} \cdot \left(\frac{6 \text{ ft}}{1.0 \text{ in}}\right)^2} = \mathbf{58.9 \text{ ksi}}
 \end{aligned}$$

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$\phi_b$ , the resistance factor for bending, is determined per AISC/ANSI 360-16 §F1a as **0.9**.

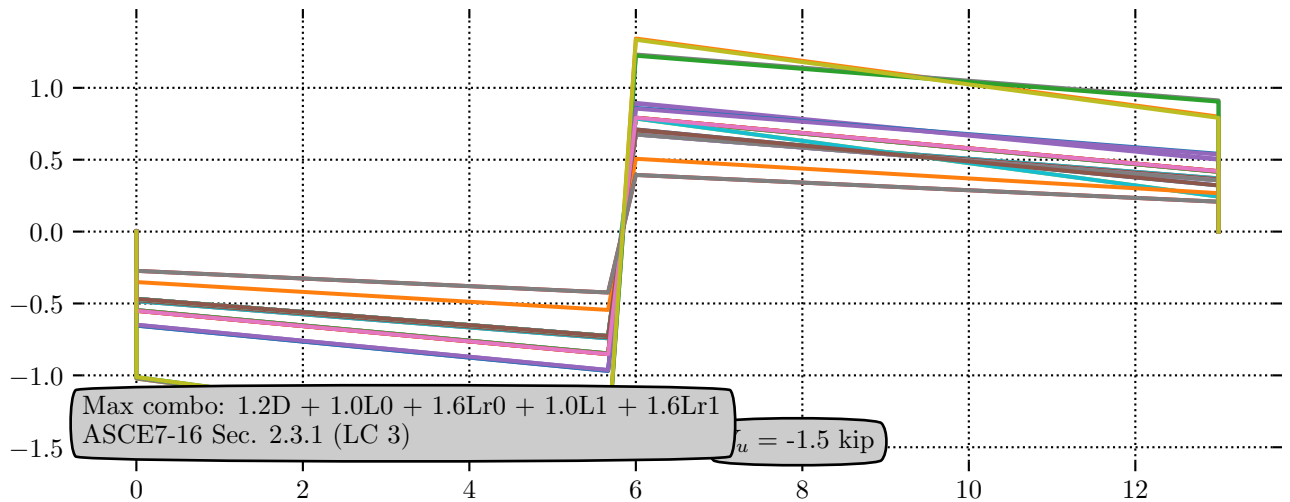
$\phi_b M_n$ , the design flexural strength, is calculated per AISC/ANSI 360-16 Eq. F2-2 as follows:

$$\begin{aligned}\phi_b M_n &= \phi_b \cdot C_b \cdot M_p - 0.7 \cdot F_y \cdot S_x \cdot \frac{L_b - L_p}{L_r - L_p} < \phi_b \cdot M_p - 0.7 \cdot F_y \cdot S_x \cdot \frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \\ &= 0.9 \cdot 1.0 \cdot 52.5 \text{ kft} - 0.7 \cdot 50 \text{ ksi} \cdot 10.9 \text{ in}^3 \cdot \frac{6 \text{ ft} - 2.8 \text{ ft}}{8.1 \text{ ft} - 2.8 \text{ ft}} < 0.9 \cdot 52.5 \text{ kft} - 0.7 \cdot 50 \text{ ksi} \cdot 10.9 \text{ in}^3 \cdot\end{aligned}$$

$$|M_u| = 7.5 \text{ kft} < \phi_b \cdot M_n = 35.9 \text{ kft} \text{ (DCR} = 0.21 \text{ - OK)}$$

(LTB controls)

## 4 Shear Check



**Figure 4:** Shear Demand Envelope

$C_{v1}$ , the web shear strength coefficient, is calculated per AISC/ANSI 360-16 Eq. G2-2 as follows, based on the ratio of the clear distance between flanges to web thickness:

$$\frac{h}{t_w} = \frac{8.8 \text{ in}}{0.19 \text{ in}} = 46.57894736842105 \leq 2.24 \sqrt{\frac{E}{F_y}} = 2.24 \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 53.9 \rightarrow C_{v1} = 1.0$$

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$\phi_v$ , the resistance factor for shear, is calculated per AISC/ANSI 360-16 §G2.1.a as follows:

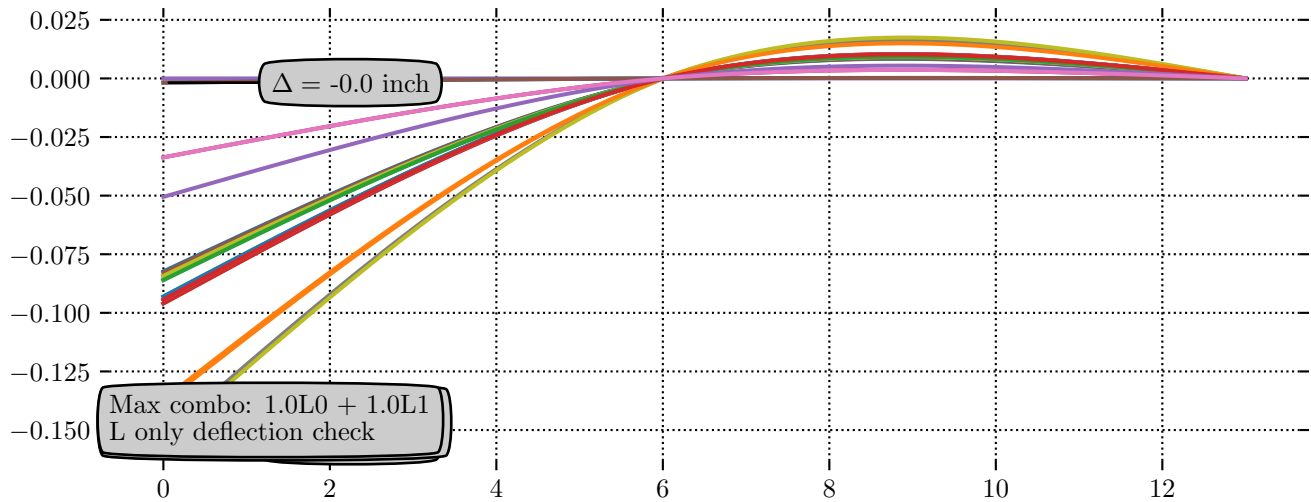
$$\frac{h}{t_w} = \frac{8.8 \text{ in}}{0.19 \text{ in}} = 46.57894736842105 \leq 2.24 \cdot \sqrt{\frac{E}{F_y}} = 2.24 \cdot \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 53.9 \rightarrow \phi_v = 1.0$$

$\phi_v V_n$ , the design shear strength, is calculated per AISC/ANSI 360-16 Eq. G2-1 as follows:

$$\phi_v V_n = 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 0.6 \cdot 50 \text{ ksi} \cdot 1.8753 \text{ in}^2 \cdot C_{v1} = 56.3 \text{ kip}$$

$$|V_u| = 1.5 \text{ kip} < \phi_v \cdot V_n = 56.3 \text{ kip} \text{ (DCR} = 0.03 \text{ - OK)}$$

## 5 Deflection Check



**Figure 5:** Deflection Envelope

$$\text{TL Deflection Check: } \Delta_{max} = -0.15 \text{ in} = \frac{L}{465} < \frac{L}{120.0} \text{ (OK)}$$

$$\text{LL Deflection Check: } \Delta_{max} = -0.0 \text{ in} = \frac{L}{40191} < \frac{L}{180.0} \text{ (OK)}$$

## 6 Reactions

The following is a summary of service-level reactions at each support:

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**Table 4:** Reactions at Supports

Loc.	Type	D	E	L0	L1	Lr0	Lr1
6 <b>ft</b>	Shear	2.5 <b>kip</b>	0.0 <b>kip</b>	0.3 <b>kip</b>	0.0 <b>kip</b>	1.5 <b>kip</b>	0.1 <b>kip</b>
13 <b>ft</b>	Shear	3.3 <b>kip</b>	0.1 <b>kip</b>	-0.0 <b>kip</b>	2.7 <b>kip</b>	-0.3 <b>kip</b>	0.7 <b>kip</b>