Member:	Test1	**Firm:**	ABC Company
Project:	123 Maple Street, San Francisco CA	**Engineer:**	Jesse
Level:	2	**Checker:**	Joey
Date:	2021-03-28	**Page:**	1

W12x14 Design Report

1 Applied Loading

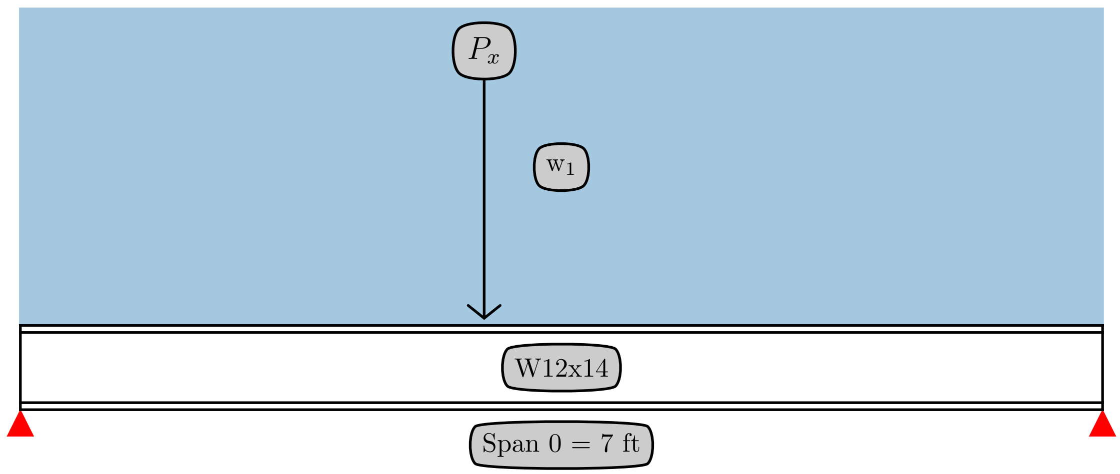

Figure 1: Applied Loads

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_1	0 ft	14.2 plf	7 ft	14.2 plf	D	Self weight

Table 2: Applied Point Loads

Load	Loc.	Shear	Type	Description
P_1	3 ft	1.6 kip	D	No description
P_2	3 ft	2.4 kip	L	No description

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 Ω_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).

Table 3: Strength (LRFD) Load Combinations

Load Combo	Loads and Factors	Reference
LC 1	1.2D + 1.6L0	ASCE7-16 §2.3.1 (LC 2)
LC 2	1.2D	ASCE7-16 §2.3.1 (LC 3)
LC 3	1.2D + 1.0L0	ASCE7-16 §2.3.1 (LC 3)
LC 4	1.4D	ASCE7-16 §2.3.1 (LC 1)
LC 5	1.4D + 0.5L0	ASCE7-16 §2.3.6 (LC 6)
LC 6	0.7D	ASCE7-16 §2.3.6 (LC 7)
LC 7	0.9D	ASCE7-16 §2.3.1 (LC 5)

Table 4: Deflection (ASD) Load Combinations

Load Combo	Loads and Factors	Reference
LC 1	0.4D	ASCE7-16 §2.4.5 (LC 9)
LC 2	1.0D	ASCE7-16 §2.4.1 (LC 1)
LC 3	1.0D + 0.75L0	ASCE7-16 §2.4.1 (LC 4)
LC 4	1.0L0	L only deflection check
LC 5	1.1D + 0.75L0	ASCE7-16 §2.4.5 (LC 8)
LC 6	1.0D + 1.0L0	ASCE7-16 §2.4.1 (LC 2)
LC 7	1.14D	ASCE7-16 §2.4.5 (LC 8)
LC 8	0.6D	ASCE7-16 §2.4.1 (LC 7)

3 Sectional and Material Properties

The following are sectional and material properties used for analysis **(W12x14, Grade A992)**:

Table 5: Sectional and Material Properties

Property	Value
A_w	2.4 in^2
C_w	80.4 in^6
F_u	65 ksi
F_y	50 ksi
I_x	88.6 in^4
I_y	2.4 in^4

Property	Value
S_x	14.9 in^3
S_y	1.2 in^3
Z_x	17.4 in^3
Z_y	1.9 in^3
r_x	4.6 in
r_y	0.8 in

Property	Value
b_f	4.0 in
t_f	0.2 in
t_w	0.2 in
h_0	11.7 in
U.W.	490 pcf

4 Bending Check

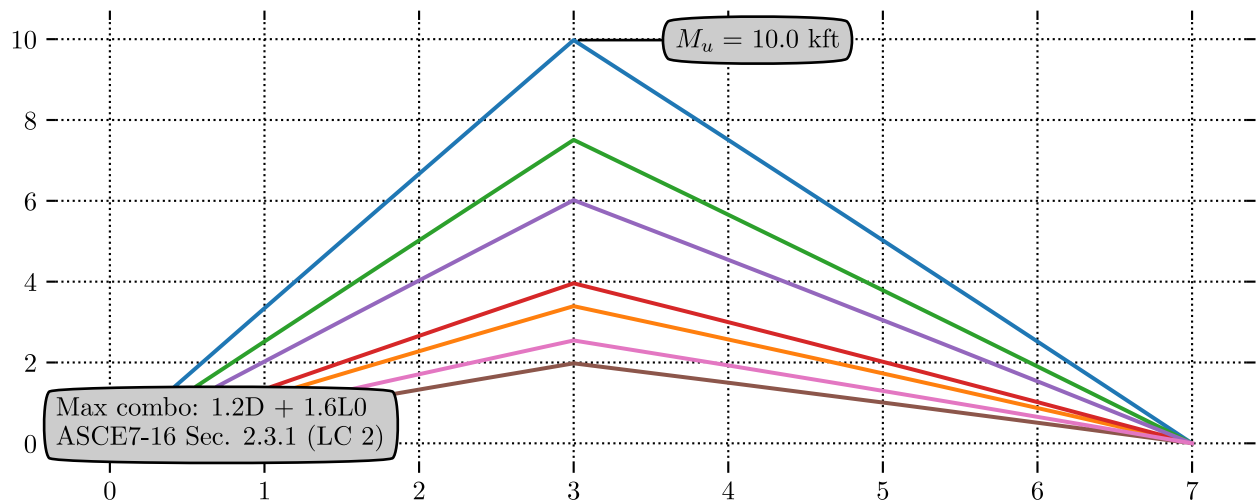

Figure 2: Moment Demand Envelope

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:

$$L_p = 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 1.76 \cdot 0.753 \text{ in} \cdot \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = \mathbf{2.7} \text{ ft}$$

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:

$$r_{ts} = \sqrt{\frac{\sqrt{I_y \cdot C_w}}{S_x}} = \sqrt{\frac{\sqrt{2.36 \text{ in}^4 \cdot 80.4 \text{ in}^6}}{14.9 \text{ in}^3}} = \mathbf{1.0} \text{ in}$$

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:

$$L_r = 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y}\sqrt{\frac{J \cdot c}{S_x \cdot h_0} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_0}\right)^2 + 6.76\left(\frac{0.7 \cdot F_y}{E}\right)^2}}$$

$$= 1.95 \cdot 1.0 \text{ in} \cdot \frac{29000 \text{ ksi}}{0.7 \cdot 50 \text{ ksi}}\sqrt{\frac{0.07 \text{ in}^4 \cdot 1}{14.9 \text{ in}^3 \cdot 11.7 \text{ in}} + \sqrt{\left(\frac{0.07 \text{ in}^4 \cdot 1}{14.9 \text{ in}^3 \cdot 11.7 \text{ in}}\right)^2 + 6.76\left(\frac{0.7 \cdot 50 \text{ ksi}}{29000 \text{ ksi}}\right)^2}}$$

$$= \mathbf{7.7} \text{ ft}$$

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

$$\lambda_{web} = \frac{h}{t_w} = \frac{10.85\ \text{in}}{0.2\ \text{in}} = \mathbf{54.2}$$

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

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

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

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

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

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

$$\lambda_{flange} = \frac{b}{t} = \frac{1.99\ \text{in}}{0.225\ \text{in}} = \mathbf{8.8}$$

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

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

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

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

$\lambda_{\text{flange}} < \lambda_{\text{P-flange}} \rightarrow$ **Compact Flange**

Since $\mathbf{L_p} < \mathbf{L_b} <= \mathbf{L_r}$ and the beam's flanges are **compact**, controlling limit state for flexure is **LTB (not to exceed capacity based on yielding)**.

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

$$M_p = F_y \cdot Z_x = 50\ \text{ksi} \cdot 17.4\ \text{in}^3 = \mathbf{72.5}\ \text{kft}$$

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:

$$\begin{aligned} C_b &= \frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C} \\ &= \frac{12.5 \cdot 10.0\ \mathbf{kft}}{2.5 \cdot 10.0\ \mathbf{kft} + 3 \cdot 5.7\ \mathbf{kft} + 4 \cdot 8.6\ \mathbf{kft} + 3 \cdot 4.3\ \mathbf{kft}} \\ &= \mathbf{1.4} \end{aligned}$$

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

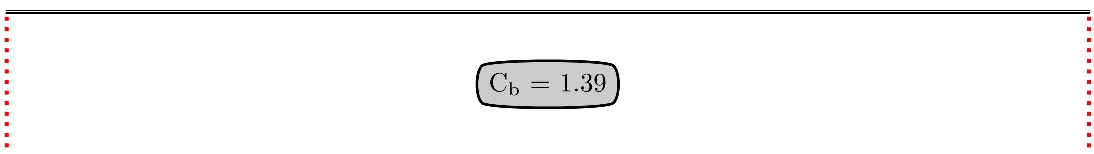

Figure 3: C_b Along Member

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:

$$\begin{aligned} F_{cr} &= \frac{C_b \cdot \pi^2 \cdot E}{\left(\frac{L_b}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \cdot \left(\frac{L_b}{r_{ts}}\right)^2} \\ &= \frac{1.39 \cdot \pi^2 \cdot 29000\ \mathbf{ksi}}{\left(\frac{7\ \mathbf{ft}}{1.0\ \mathbf{in}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{0.0704\ \mathbf{in^4} \cdot 1}{14.9\ \mathbf{in^3} \cdot 11.7\ \mathbf{in}} \cdot \left(\frac{7\ \mathbf{ft}}{1.0\ \mathbf{in}}\right)^2} = \mathbf{58.2\ ksi} \end{aligned}$$

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

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

$$\begin{aligned} \phi_b M_n &= \phi_b \cdot C_b \cdot M_p - 0.7 \cdot F_y \cdot S_x \cdot \frac{L_b - L_p}{L_r - L_p} < \phi_b \cdot M_p \\ &= 0.9 \cdot 1.39 \cdot 72.5\ \mathbf{kft} - 0.7 \cdot 50\ \mathbf{ksi} \cdot 14.9\ \mathbf{in^3} \cdot \frac{7\ \mathbf{ft} - 2.7\ \mathbf{ft}}{7.7\ \mathbf{ft} - 2.7\ \mathbf{ft}} < 0.9 \cdot 72.5\ \mathbf{kft} = \mathbf{59.8\ kft} \end{aligned}$$

$|\mathbf{M_u}| = \mathbf{10.0}$ kft $< \phi_\mathbf{b} \cdot \mathbf{M_n} = \mathbf{59.8}$ kft (**DCR = 0.17 – OK**)

(LTB controls)

5 Shear Check

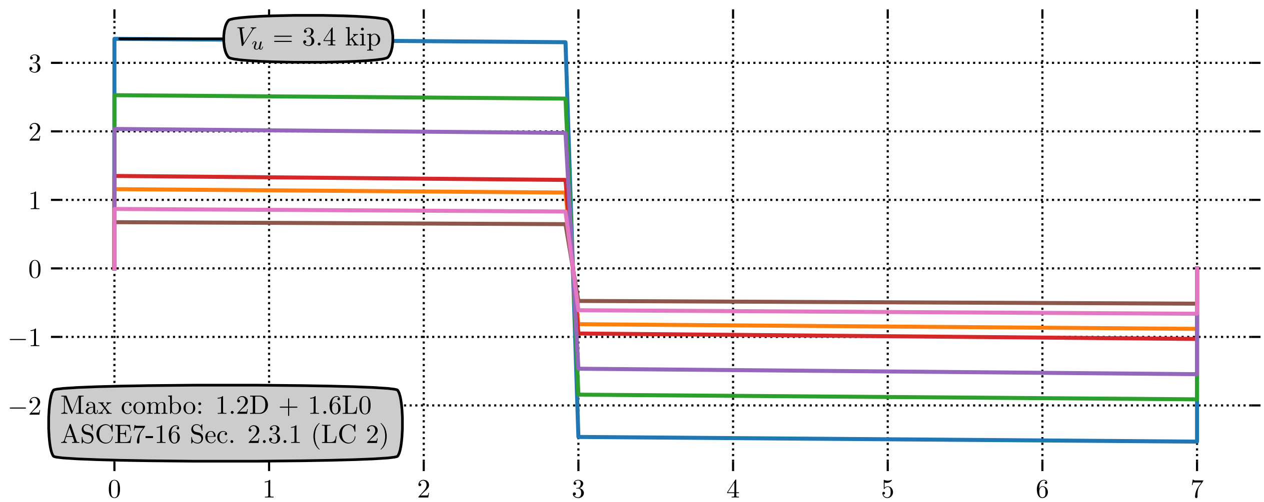

Figure 4: Shear Demand Envelope

C_{v1}, the web shear strength coefficient, is calculated per AISC/ANSI 360-16 Eq. G2-3 as follows, based on the ratio of the clear distance between flanges to web thickness:

$$\frac{h}{t_w} = \frac{10.8 \text{ in}}{0.2 \text{ in}} = \mathbf{54.24999999999999} <= 1.1\sqrt{\frac{k_v E}{F_y}} = 1.1\sqrt{\frac{5.3429000 \text{ ksi}}{50 \text{ ksi}}} = \mathbf{61.2} \rightarrow C_{v1} = \mathbf{1.0}$$

φ_v, the resistance factor for shear, is calculated per AISC/ANSI 360-16 §G1.a as follows:

$$\frac{h}{t_w} = \frac{10.8 \text{ in}}{0.2 \text{ in}} = \mathbf{54.24999999999999} > 2.24 \cdot \sqrt{\frac{E}{F_y}} = 2.24 \cdot \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = \mathbf{53.9} \rightarrow \phi_v = \mathbf{0.9}$$

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

$$\phi_v V_n = 0.6 \cdot F_y \cdot A_w \cdot C_{v1} = 0.6 \cdot 50 \text{ ksi} \cdot 2.3800000000000003 \text{ in}^2 \cdot C_{v1} = \mathbf{71.4} \text{ kip}$$

$|\mathbf{V_u}| = \mathbf{3.4}$ kip $< \phi_\mathbf{v} \cdot \mathbf{V_n} = \mathbf{71.4}$ kip (**DCR = 0.05 – OK**)

6 Deflection Check

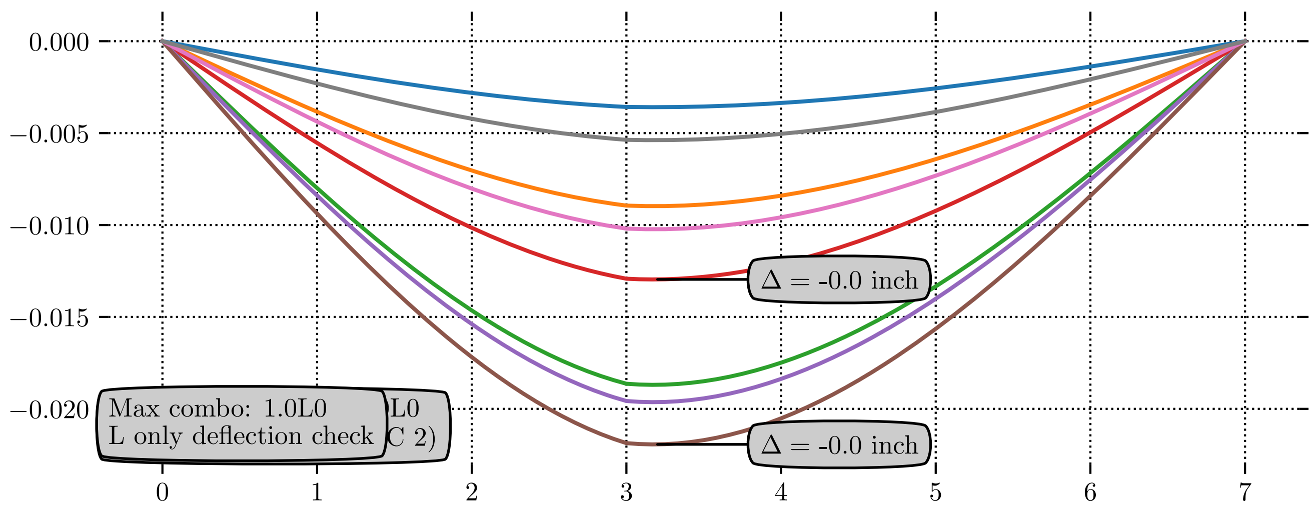

Figure 5: Deflection Envelope

Tl Deflection Check: $\Delta_{max} = -0.02$ **in** $= \dfrac{L}{3830} < \dfrac{L}{1.0}$ (**OK**)

Ll Deflection Check: $\Delta_{max} = -0.01$ **in** $= \dfrac{L}{6482} < \dfrac{L}{1.0}$ (**OK**)

7 Reactions

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

Table 6: Reactions at Supports

Loc.	Type	D	L0
0 **ft**	Shear	1.0 **kip**	1.4 **kip**
7 **ft**	Shear	0.7 **kip**	1.0 **kip**