New Equations and Table for Design of Eccentrically Loaded WT Compression Members

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ABSTRACT:

Equations and a design table are developed to determine the available axial compressive strength of eccentrically loaded WT shapes with F_y = 50 ksi using both the LRFD and ASD methods. WTs considered are made from W-shapes ordinarily used as columns. Tabulated values account for the bending moment created in the member due to the load eccentricity, including second-order effects, and follow the provisions of Section H1.1 of the AISC *Specification for Structural Steel Buildings* (2022) for design of members subject to combined forces. Applicable limit states and cross-section classifications are considered in the development of the equations and the design table. Numerical example problems are presented.

Keywords: WT shapes, compression, combined forces, eccentric loading, design aid.

INTRODUCTION

WT shapes are commonly used as compression members, particularly in bracing systems. Typically, the member is connected at the ends using gusset plates attached to the outer surface of the flange (Figure 1) using bolted or welded joints. This arrangement causes the applied load to be eccentric, resulting in both flexural and compressive stresses. The member should therefore be designed for combined forces in accordance with Section H1.1 of the AISC *Specification for Structural Steel Buildings* (2022), hereafter referred to as the AISC *Specification*, including AISC *Specification* Equations H1-1a and H1-1b.

$$\frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)\leq 1.0$$

(AISC *Spec*. Eq. H1-1a)

$$\frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}+\frac{M_{ry}}{M_{cy}}\right)\leq 1.0$$

(AISC *Spec*. Eq. H1-1b)

The LRFD and ASD versions of these equations are presented and used later. A list of symbols, their definitions, and units are provided later in this article. The term *force* in this article refers to both axial load as well as bending moment.

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A table of reduction factors (P_r/P_c) for WT sections subjected to compressive loads with connection eccentricity was previously developed by Mark E. Gordon (2010), based on the provisions of the 2005 AISC *Specification* (AISC, 2005a) and the 13th Edition of the AISC *Steel Construction Manual* (AISC, 2005b), hereafter referred to as the AISC *Manual*. The table was developed based on ½ in. gusset plates in all cases.

This article introduces newly developed equations and a design table that directly provide the compressive strength of eccentrically loaded WT members, in accordance with the latest editions of the AISC *Specification* (2022) and *Manual* (2023a). Gusset plate thicknesses based on member flange thickness are used. Other member properties useful in the design are listed at the bottom of the table.

RATIONALE FOR THE DEVELOPMENT OF THE COMPRESSIVE STRENGTH EQUATIONS AND DESIGN TABLE

It is common to connect WT compression members using a gusset plate connected to the flange of the WT at each end as discussed. This configuration results in load eccentricity in the member, considerably reducing its available compressive strength.

AISC *Manual* Table 4-7, titled "Available Strength in Axial Compression, Concentrically Loaded WT-Shapes," assumes that the applied load does not have any eccentricity and gives very unconservative results for members subject to eccentric loading. For example, a 14-ft-long WT7×66 connected with a ¾ in. gusset plate at each end will have an available eccentric compressive strength of 184 kips using the LRFD method. The same member has an available concentric compressive strength of 438 kips given in

Table 4-7. Use of Table 4-7 in this case results in significant under-design.

Shortly, two example problems will be presented that will include manual solutions based on AISC *Specification* provisions and equations. As will be observed, the proposed design table reduces the work necessary for calculating the available strength of eccentrically loaded WT compression members from a few pages of calculations to simply looking up the value from the design table resulting in significant savings of time and effort.

CLARIFICATION OF LATERALLY UNBRACED LENGTHS FOR WT COMPRESSION MEMBERS

Laterally bracing a WT compression member against flexural buckling about the *y*-axis braces the member at the same location against lateral torsional buckling as well, namely, $L_b = L_y$.

Ordinarily, there is no restriction on the relative magnitude of unbraced lengths L_x, L_y, L_z, and L_b for a member subject to combined forces. One can use manual calculations to determine the strength of an eccentrically loaded WT compression member for different unbraced lengths using AISC *Specification* provisions and equations. As explained later, this study requires all the unbraced lengths to be equal, as shown in Figure 2, namely, $L_x = L_y = L_z = L_b = L$.

Plate connections at the ends of a member as described earlier create pin connections for both column and beam actions. Further, unbraced lengths for flexural buckling and flexural torsional buckling are also pin-ended. For this study, the values of K_x, K_y, and K_z are all taken as 1.0. Consequently, $L_{cx} = K_x L_x = (1)L_x = L_x$. Similarly, L_{cy} equals L_y, and L_{cz} equals L_z.

Given that the members in question are subject to combined compression and bending, the available eccentric compressive strength for any L_{cx} depends on the nominal

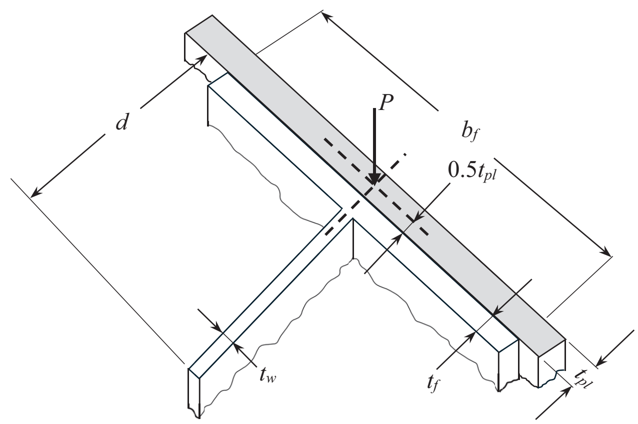


Fig. 1. WT member subject to eccentric axial compression (joint details not shown).

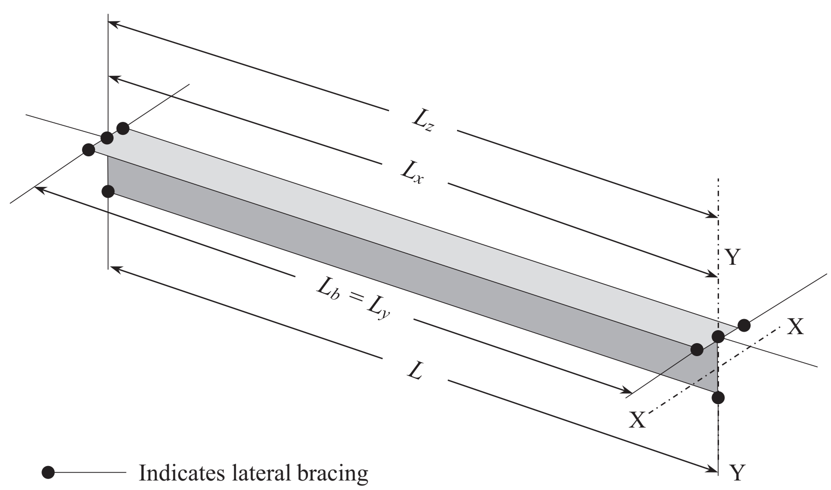


Fig. 2. Identification of lateral bracing and unbraced lengths.

flexural strength, M_n, which, in turn, depends on L_b and thus L_{cy}. Therefore, the member's flexural and compressive strengths are interlinked as are their unbraced lengths. This concept is important to recognize in the development and use of the equations and design table that follow.

ANALYSIS OF THE MEMBER, INCLUDING SECOND-ORDER EFFECT

In the following discussion, it is assumed that the compression load is applied at the mid-width of the WT flange and at mid-thickness of the plate as shown in Figure 3. Further, it is assumed that end connections are identical, no transverse loads are applied along the member length, and there is no bending about the *y*-axis. Member weight is not included as is the case in AISC *Manual* tables.

In the proposed design table, it is assumed that $L_{cx} = L_{cy} = L_{cz} = L_b = L$. Also assumed are connection plate widths and thicknesses. These same assumptions were made in the two design example manual solutions so that the computed results could be compared with the tabulated results. The manual solutions can be modified for other unbraced lengths and connection plate dimensions.

As shown in Figure 4, the eccentricity of the load, measured from the centroid of the WT, is as follows.

$$e = 0.5t_{pl} + \bar{y} \tag{1}$$

This eccentricity results in a constant bending moment of force times the eccentricity, *Pe*, in the member.

The approximate second-order analysis presented in AISC *Specification* Appendix 8, Section 8.1, is used to calculate the required flexural strength of the member as follows.

$$M_r = B_1 M_{nt} + B_2 M_{lt}$$

(AISC *Spec*. Eq. A-8-1)

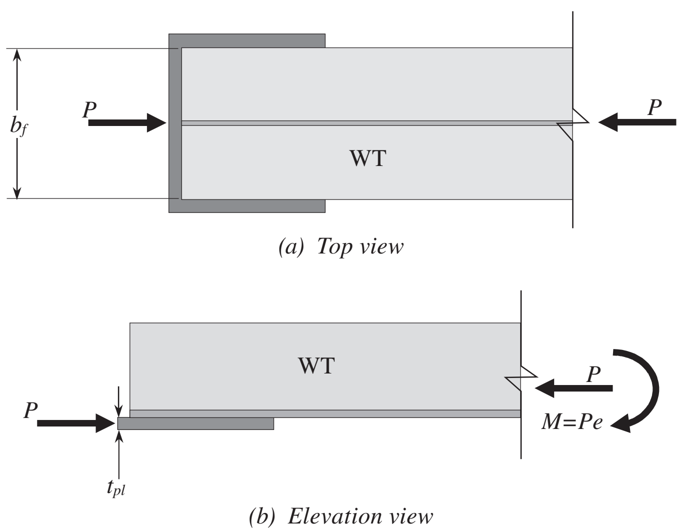


Fig. 3. Typical end connection of WT compression members (joint details not shown).

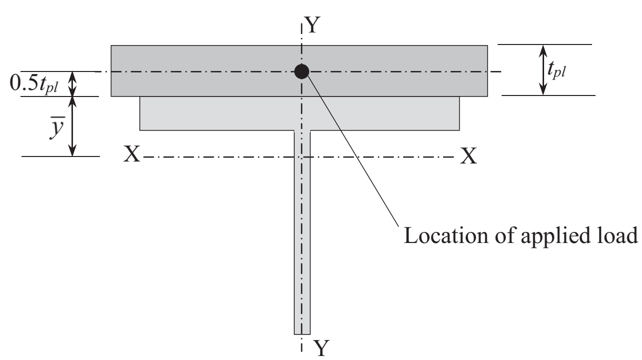


Fig. 4. Anatomy of the WT section and connected plate.

$$P_r = P_{nt} + B_2 P_{lt}$$

(AISC *Spec*. Eq. A-8-2)

Because there is no relative lateral translation of the member ends, there is no P-Δ effect, thus B_2, M_{lt}, and P_{lt} do not apply. Further, recall that the member is not subject to transverse loads.

LRFD Method

There is no bending about the y-axis ($M_{uy} = 0$). Therefore, member forces including P-δ effect are as follows.

$$M_{ux} = B_{1x} M_{ntx} \tag{2}$$

$$P_u = P_{nt} \tag{3}$$

with

$$M_{ntx} = P_u e \tag{4}$$

Therefore,

$$M_{ux} = B_{1x} P_u e \tag{5}$$

where

$$B_{1x} = \frac{C_{mx}}{1 - \dfrac{\alpha P_r}{P_{e1x}}} \geq 1.0$$

(AISC *Spec*. Eq. A-8-3)

and

$$C_{mx} = 0.6 - 0.4\left(M_{1x}/M_{2x}\right)$$

(AISC *Spec*. Eq. A-8-4)

M_{1x} and M_{2x} are the member end moments. In this case, $M_{1x} = M_{2x}$ and the member bends in single curvature. Therefore, $\left(M_{1x}/M_{2x}\right) = -1.0$ with the negative sign for single curvature bending.

$$\begin{aligned} C_{mx} &= 0.6 - 0.4\left(M_{1x}/M_{2x}\right) \\ &= 0.6 - 0.4(-1) \\ &= 1.0 \end{aligned}$$

AISC *Specification* Equation A-8-3, modified with C_{mx} = 1.0 and utilizing the LRFD design method (α = 1.0 and $P_r = P_u$) results in the following.

$$B_{1x} = \frac{C_{mx}}{1 - \dfrac{\alpha P_r}{P_{e1x}}} = \frac{1}{1 - \dfrac{(1.0) P_u}{P_{e1x}}} \geq 1.0 \tag{6}$$

or

$$B_{1x} = \frac{P_{e1x}}{P_{e1x} - P_u} \geq 1.0 \tag{7}$$

where

$$P_{e1x} = \frac{\pi^2 E I_x}{\left(L_{c1x}\right)^2}$$

(AISC *Spec*. Eq. A-8-5)

ASD Method

The following are pertinent equations, without discussion, for the ASD method developed similar to those presented for the LRFD method. Note that C_{mx} = 1.0 and for ASD, α = 1.6 and $P_r = P_a$. These equations will be used and referenced in Design Example 2.

$$M_{ax} = B_{1x} M_{ntx} \tag{8}$$

$$P_a = P_{nt} \tag{9}$$

$$M_{ntx} = P_a e \tag{10}$$

$$M_{ax} = B_{1x} P_a e \tag{11}$$

$$B_{1x} = \frac{1}{1 - \dfrac{1.6 P_a}{P_{e1x}}} \geq 1.0$$

(AISC *Spec*. Eq. A-8-3)

or

$$B_{1x} = \frac{P_{e1x}}{P_{e1x} - 1.6 P_a} \geq 1.0 \tag{12}$$

EXAMPLE PROBLEMS WITH MANUAL SOLUTIONS

The following example problems are provided to offer detailed manual solutions for calculation of the available strengths of eccentrically loaded WT compression members using ASD and LRFD methods.

The use of the proposed design table is based on a number of assumptions such as certain plate width thickness and that $L_{cx} = L_{cy} = L_{cz} = L_b$. However, the solutions to the following problems are not limited to any of those restrictions. In the examples that follow, the condition $L_{cx} = L_{cy} = L_{cz} = L_b$ was used so that the results may be compared with the values in the proposed design table. However, the manual solutions can be modified appropriately for any unbraced lengths.

Design Example 1

Given:

The WT7×45 compression member shown in Figure 5 with ASTM A992/A992M (2020) steel is braced at the ends and connected at each end with a 14½ in. × ½ in. gusset plate of ASTM A572/A572M (2021) Grade 50 steel. $L_{cx} = L_{cy} = L_{cz} = L_b = L =$ 12 ft. Assume an adequate bolted or welded joint exists between each gusset plate and the member. Determine the available eccentric axial compressive strength of the member using AISC *Specification* provisions for the LRFD method.

Solution:

From AISC *Manual* Table 1-8, the geometric properties for a WT7×45 are as follows:

$A = 13.2 \text{ in.}^2$ $b_f = 14.5$ in. $t_f = 0.710$ in. $b_f/2t_f = 10.2$ $d/t_w = 15.9$ $I_x = 36.5 \text{ in.}^4$ $S_x = 6.16 \text{ in.}^3$
$r_x = 1.66$ in. $\bar{y} = 1.09$ in. $Z_x = 11.5 \text{ in.}^3$ $r_y = 3.70$ in. $J = 2.03 \text{ in.}^4$ $C_w = 8.31 \text{ in.}^6$

Beam Action

Check the section element width-to-thickness ratios using the limits given in AISC *Specification* Table B4.1b.

For Case 10(3), the flange element compact limit is:

$$\begin{aligned}\lambda_{pf} &= 0.38\sqrt{\frac{E}{F_y}}\\ &= 0.38\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}}\\ &= 9.15\end{aligned}$$

For Case 10(3), the flange element noncompact limit is:

$$\begin{aligned}\lambda_{rf} &= 1.0\sqrt{\frac{E}{F_y}}\\ &= 1.0\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}}\\ &= 24.1\end{aligned}$$

Fig. 5. Design Example 1 detail.

Because $\lambda_{pf} = 9.15 < \lambda_f = b_f/2t_f = 10.2 < \lambda_{rf} = 24.1$, the flange element is noncompact.

The stem is in tension for flexure, and therefore, there is no need to check the width-to-thickness ratio of the stem for flexure.

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \qquad \text{(AISC } \textit{Spec}\text{. Eq. F9-8)}$$

$$= (1.76)(3.70 \text{ in.})\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}}$$

$$= 157 \text{ in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)$$

$$= 13.1 \text{ ft}$$

Because $L_b = 12$ ft $< L_p = 13.1$ ft, lateral torsional buckling does not apply per AISC *Specification* Section F9.2(a)(1).

Because the section is noncompact, AISC *Specification* Equation F9-14 applies for calculation of M_n.

$$M_{nx} = M_{px} - \left(M_{px} - 0.7F_y S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \le 1.6M_y \qquad \text{(AISC } \textit{Spec}\text{. Eq. F9-14)}$$

where

$$M_{px} = F_y Z_x \le 1.6M_y \qquad \text{(AISC } \textit{Spec}\text{. Eq. F9-2)}$$

$$M_y = F_y S_x \qquad \text{(AISC } \textit{Spec}\text{. Eq. F9-3)}$$

$$= (50 \text{ ksi})(6.16 \text{ in.}^3)$$

$$= 308 \text{ kip-in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)$$

$$= 25.7 \text{ kip-ft}$$

$$M_{px} = F_y Z_x \le 1.6M_y$$

$$= (50 \text{ ksi})(11.5 \text{ in.}^3)$$

$$= 575 \text{ kip-in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)$$

$$= 47.9 \text{ kip-ft, but must be} \le 1.6M_y = (1.6)(25.7 \text{ kip-ft}) = 41.1 \text{ kip-ft}$$

Therefore, use $M_{px} = 41.1$ kip-ft.

$$S_{xc} = \frac{I_x}{\bar{y}}$$

$$= \frac{36.5 \text{ in.}^4}{1.09 \text{ in.}}$$

$$= 33.5 \text{ in.}^3$$

$$0.7F_y S_{xc} = (0.70)(50 \text{ ksi})(33.5 \text{ in.}^3)$$

$$= 1{,}172 \text{ kip-in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)$$

$$= 97.7 \text{ kip-ft}$$

Apply AISC *Specification* Equation F9-14.

$$M_{nx} = 41.1 \text{ kip-ft} - \left\{ \left[(41.1 \text{ kip-ft}) - (97.5 \text{ kip-ft}) \right] \left(\frac{10.2 - 9.15}{24.1 - 9.15} \right) \right\}$$
$$= 45.1 \text{ kip-ft, but must be} \leq 1.6M_y = (1.60)(25.7 \text{ kip-ft}) = 41.1 \text{ kip-ft}$$

Therefore, use $M_{nx} = 41.1$ kip-ft.

$$\phi_b M_{nx} = (0.90)(41.1 \text{ kip-ft})$$
$$= 37.0 \text{ kip-ft}$$

Column Action

Check the section element width-to-thickness ratios using the limits given in AISC *Specification* Table B4.1a.

Case 1(5), flange element nonslender limit

$$\lambda_{rf} = 0.56\sqrt{\frac{E}{F_y}}$$
$$= 0.56\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}}$$
$$= 13.5$$

Because $\lambda_f = \dfrac{b_f}{2t_f} = 10.2 < \lambda_{rf} = 13.5$, the flange element is nonslender.

Case 4, web element nonslender limit

$$\lambda_{rw} = 0.75\sqrt{\frac{E}{F_y}}$$
$$= 0.75\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}}$$
$$= 18.1$$

Because $\lambda_w = \dfrac{d}{t_w} = 15.9 < \lambda_{rw} = 18.1$, the web element is nonslender.

Flexural Buckling Strength

AISC *Specification* Section E3 applies.

$$\frac{L_{cx}}{r_x} = \frac{(12 \text{ ft})(12 \text{ in./ft})}{1.66 \text{ in.}}$$
$$= 86.7 < 200 \qquad \textbf{o.k.}$$

$$\frac{L_{cy}}{r_y} = \frac{(12 \text{ ft})(12 \text{ in./ft})}{3.70 \text{ in.}}$$
$$= 38.9 < 200 \qquad \textbf{o.k.}$$

Use $\dfrac{L_c}{r} = 86.7$.

In the following equations, subscripts *FB* and *FTB* are used to distinguish between values calculated for flexural buckling and flexural-torsional buckling, respectively.

$$(F_e)_{FB} = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \qquad \text{(AISC } \textit{Spec}\text{. Eq. E3-4)}$$

$$= \frac{\pi^2 (29{,}000 \text{ ksi})}{(86.7)^2}$$

$$= 38.0 \text{ ksi}$$

Because $\dfrac{F_y}{(F_e)_{FB}} = \dfrac{50.0 \text{ ksi}}{38.0 \text{ ksi}} = 1.32 < 2.25$, use AISC *Specification* Equation E3-2.

$$(F_n)_{FB} = \left[0.658^{\frac{F_y}{F_e}}\right](F_y) \qquad \text{(AISC } \textit{Spec}\text{. Eq. E3-2)}$$

$$= \left[0.658^{(1.32)}\right](50 \text{ ksi})$$

$$= 28.8 \text{ ksi}$$

Flexural-Torsional Buckling Strength (AISC *Specification* Section E4 applies)

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{L_{cy}}{r_y}\right)^2} \qquad \text{(AISC } \textit{Spec}\text{. Eq. E4-6)}$$

$$= \frac{\pi^2 (29{,}000 \text{ ksi})}{(38.9)^2}$$

$$= 189 \text{ ksi}$$

From the AISC *Shapes Database* (2023b), $H = 0.968$ and $\bar{r}_o = 4.12$ in. Alternatively, AISC *Specification* Equations E4-8 and E4-9 can be used to calculate H and $\bar{r}_o^2$.

Calculate F_{ez} as follows.

$$F_{ez} = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \qquad \text{(AISC } \textit{Spec}\text{. Eq. E4-7)}$$

$$= \left\{\frac{\pi^2 (29{,}000 \text{ ksi})(8.31 \text{ in.}^6)}{\left[(12 \text{ ft})(12 \text{ in./ft})\right]^2} + (11{,}200 \text{ ksi})(2.03 \text{ in.}^4)\right\}\left[\frac{1}{(13.2 \text{ in.}^2)(4.12 \text{ in.})^2}\right]$$

$$= 102 \text{ ksi}$$

The first component of the preceding equation that includes C_w is about 0.5% of the calculated F_{ez}. This is consistent with a User Note in AISC *Specification* Section E4 that states, "For tees and double angles, the term with C_w may be omitted when computing F_{ez}."

Calculate $(F_e)_{FTB}$ as follows.

$$(F_e)_{FTB} = \left(\frac{F_{ey}+F_{ez}}{2H}\right)\left[1-\sqrt{1-\frac{4F_{ey}F_{ez}H}{(F_{ey}+F_{ez})^2}}\right] \quad \text{(AISC } \textit{Spec.} \text{ Eq. E4-3)}$$

$$= \left[\frac{189 \text{ ksi}+102 \text{ ksi}}{2(0.968)}\right]\left[1-\sqrt{1-\frac{4(189 \text{ ksi})(102 \text{ ksi})(0.968)}{(189 \text{ ksi}+102 \text{ ksi})^2}}\right]$$

$$= 98.6 \text{ ksi}$$

Because $\dfrac{F_y}{(F_e)_{FTB}} = \dfrac{50.0 \text{ ksi}}{98.6 \text{ ksi}} = 0.507 < 2.25$, use AISC *Specification* Equation E3-2.

$$(F_n)_{FTB} = \left[0.658^{\frac{F_y}{F_e}}\right](F_y) \quad \text{(AISC } \textit{Spec.} \text{ Eq. E3-2)}$$

$$= \left[0.658^{(0.507)}\right](50 \text{ ksi})$$

$$= 40.4 \text{ ksi}$$

$(F_n)_{FTB} = 40.4 \text{ ksi} > (F_n)_{FB} = 28.8 \text{ ksi}$; therefore, use $F_n = (F_n)_{FB} = 28.8$ ksi.

$$P_n = F_n A_g$$

$$= (28.8 \text{ ksi})(13.2 \text{ in.}^2)$$

$$= 380 \text{ kips}$$

$$\phi_c P_n = (0.90)(380 \text{ kips})$$

$$= 342 \text{ kips}$$

Required Flexural Strength, Including Second-Order Effect

In the following calculations, the available strength in compression of the eccentrically loaded member is referred to as $\phi_c P_{n\,ecc}$. Note that this replaces P_u used in Equation 7 as well as in the interaction equations later.

The required moment in the member, including a second-order effect, is calculated as follows.

$$P_{e1x} = \frac{\pi^2 E I_x}{(L_{c1x})^2} \quad \text{(AISC } \textit{Spec.} \text{ Eq. A-8-5)}$$

$$= \frac{\pi^2 (29{,}000 \text{ ksi})(36.5 \text{ in.}^4)}{[(12 \text{ ft})(12 \text{ in./ft})]^2}$$

$$= 503 \text{ kips}$$

Use Equation 7 to calculate B_{1x} as follows.

$$B_{1x} = \frac{P_{e1x}}{P_{e1x} - \phi_c P_{n\,ecc}} \geq 1.0$$

$$= \frac{503 \text{ kips}}{503 \text{ kips} - \phi_c P_{n\,ecc}} \geq 1.0$$

Use Equation 1 to calculate the total load eccentricity.

$$e = 0.5t_{pl} + \overline{y} \tag{1}$$
$$= (0.50)(0.50 \text{ in.}) + 1.09 \text{ in.}$$
$$= 1.34 \text{ in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)$$
$$= 0.112 \text{ ft}$$

Recall that, $P_u = \phi_c P_{n\,ecc}$ as described before. Therefore,

$$M_{ux} = B_{1x} P_u e \tag{5}$$
$$= (B_{1x})(\phi_c P_{n\,ecc})(e)$$
$$= \left(\frac{503 \text{ kips}}{503 \text{ kips} - \phi\ P_{n\,ecc}}\right)(\phi_c P_{n\,ecc})(0.112 \text{ ft})$$
$$= \frac{56.3(\phi_c P_{n\,ecc})}{503 - \phi_c P_{n\,ecc}} \text{ kip-ft}$$

Check the Appropriate Interaction Equation for Combined Forces

Assume $\dfrac{P_u}{\phi_c P_n} = \dfrac{\phi_c P_{n\,ecc}}{\phi_c P_n} \ge 0.20$, and apply the LRFD version of AISC *Specification* Equation H1-1a as follows. Set the interaction equation equal to unity (1.0) to find the member available eccentric axial compressive strength. Note that $\phi_c = \phi_b = 0.90$ and $M_{uy} = 0$. Also, recall that, $\phi_c P_n = 342$ kips, $\phi_b M_{nx} = 37.0$ kip-ft, and P_u is replaced by $\phi_c P_{n\,ecc}$.

$$\frac{\phi_c P_{n\,ecc}}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) = \frac{\phi_c P_{n\,ecc}}{\phi_c P_n} + \frac{8M_{ux}}{9\phi_b M_{nx}} = 1.0$$

Insert values of available strengths and replace M_{ux} with its equivalent found earlier.

$$\frac{\phi_c P_{n\,ecc}}{342 \text{ kips}} + \frac{8\left[\dfrac{56.3(\phi_c P_{n\,ecc})}{503 - \phi_c P_{n\,ecc}}\right]}{9(37.0 \text{ kip-ft})} = \frac{\phi_c P_{n\,ecc}}{342 \text{ kips}} + \frac{1.35(\phi_c P_{n\,ecc})}{503 - \phi_c P_{n\,ecc}}$$
$$= 1.0$$

This is a quadratic equation with a root $\phi_c P_{n\,ecc} = 149$ kips (solution details not shown).

Check the value of B_{1x} using Equation 7.

$$B_{1x} = \frac{503 \text{ kips}}{503 \text{ kips} - \phi_c P_{n\,ecc}}$$
$$= \frac{503 \text{ kips}}{503 \text{ kips} - 149 \text{ kips}}$$
$$= 1.42 > 1.0 \quad \textbf{o.k.}$$

Check the assumption that $\dfrac{P_u}{\phi_c P_n} = \dfrac{\phi_c P_{n\,ecc}}{\phi_c P_n} \geq 0.20$.

$$\begin{aligned}\frac{P_u}{\phi_c P_n} &= \frac{P_{n\,ecc}}{\phi_c P_n}\\ &= \frac{149 \text{ kips}}{342 \text{ kips}}\\ &= 0.436 > 0.20 \quad \textbf{o.k.}\end{aligned}$$

Had this assumption turned out to be incorrect, the solution would be repeated using AISC *Specification* Eq. H1-1b instead of AISC *Specification* Eq. H1-1a. All assumptions made earlier were proven correct. Therefore, the available eccentric compressive strength of the WT7×45 compression member is $\phi_c P_{n\,ecc} = 149$ kips.

As a check, calculate the compressive strength of the gusset plate assuming that flexural buckling does not apply. See AISC *Specification* Section J4.4(a) and Equation J4–6.

The gross area of the plate is

$$\begin{aligned}(A_g)_{pl} &= (b)(t)\\ &= (14.5 \text{ in.})(0.50 \text{ in.})\\ &= 7.25 \text{ in.}^2\end{aligned}$$

$$\begin{aligned}(P_n)_{pl} &= (F_y A_g)_{pl} \qquad \text{(AISC Spec. Eq. J4-6)}\\ &= (50 \text{ ksi})(7.25 \text{ in.}^2)\\ &= 362 \text{ kips}\end{aligned}$$

$$\begin{aligned}(\phi_c P_n)_{pl} &= (0.90)(P_n)_{pl}\\ &= (0.90)(362 \text{ kips})\\ &= 326 \text{ kips} > 149 \text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Design Example 2

Given:

The WT7×30.5 compression member shown in Figure 6 is ASTM A992/A992M steel, braced at the ends and connected at each end with a 10 in. × ½ in. gusset plate of ASTM A572/A572M Grade 50 steel. $L_{cx} = L_{cy} = L_{cz} = L_b = L = 10$ ft. Assume an adequate bolted or welded joint exists between each gusset plate and the member. Determine the available eccentric axial compressive strength of the member using AISC *Specification* provisions for the ASD method.

Fig. 6. Design Example 2 detail.

Solution:

From AISC *Manual* Table 1-8, the geometric properties for a WT7×30.5 are:

$A = 8.96 \text{ in.}^2$ $d = 6.95 \text{ in.}$ $b_f = 10.0 \text{ in.}$ $t_f = 0.645 \text{ in.}$ $b_f/2t_f = 7.75$ $d/t_w = 18.5$ $I_x = 28.9 \text{ in.}^4$ $S_x = 5.07 \text{ in.}^3$
$r_x = 1.80 \text{ in.}$ $\bar{y} = 1.25 \text{ in.}$ $Z_x = 9.15 \text{ in.}^3$ $I_y = 53.7 \text{ in.}^4$ $r_y = 2.45 \text{ in.}$ $J = 1.09 \text{ in.}^4$ $C_w = 2.29 \text{ in.}^6$

Beam Action

Check the section element width-to-thickness ratios using the limits given in AISC *Specification* Table B4.1b.

For Case 10(3), the flange element compact limit is:

$$\begin{aligned}\lambda_{pf} &= 0.38\sqrt{\frac{E}{F_y}} \\ &= 0.38\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 9.15\end{aligned}$$

Because $\lambda_f = b_f/2t_f = 7.75 < \lambda_{pf} = 9.15$, the flange element is compact.

Because the web is in tension for flexure, there is no need to check width-to-thickness ratio for flexure.

$$L_p = 1.76 r_y\sqrt{\frac{E}{F_y}} \qquad \text{(AISC } \textit{Spec}\text{. Eq. F9-8)}$$

$$\begin{aligned} &= (1.76)(2.45 \text{ in.})\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 104 \text{ in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right) \\ &= 8.67 \text{ ft}\end{aligned}$$

$$L_r = 1.95\left(\frac{E}{F_y}\right)\frac{\sqrt{I_y J}}{S_x}\sqrt{2.36\left(\frac{F_y}{E}\right)\frac{dS_x}{J}+1} \qquad \text{(AISC } \textit{Spec}\text{. Eq. F9-9)}$$

$$\begin{aligned} &= 1.95\left(\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}\right)\frac{\sqrt{(53.7 \text{ in.}^4)(1.09 \text{ in.}^4)}}{(5.07 \text{ in.}^3)}\sqrt{2.36\left(\frac{50 \text{ ksi}}{29{,}000 \text{ ksi}}\right)\frac{(6.95 \text{ in.})(5.07 \text{ in.}^3)}{(1.09 \text{ in.}^4)}+1} \\ &= 1{,}816 \text{ in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right) \\ &= 151 \text{ ft}\end{aligned}$$

Note that AISC *Specification Commentary* Section F9, Tees and Double Angles Loaded in the Plane of Symmetry, states, "For most shapes, the length, L_r, is impractically long."

$L_p = 8.67 \text{ ft} < L_b = 10 \text{ ft} < L_r = 151 \text{ ft}$

Inelastic lateral torsional buckling governs. Provisions of AISC *Specification* Section F9.2(a)(2) and Equation F9-6 apply to calculating M_n. But first calculate M_{px} as follows.

$$M_{px} = F_y Z_x < 1.6M_y \qquad \text{(AISC } \textit{Spec}\text{. Eq. F9-2)}$$

$$M_y = F_y S_x \qquad \text{(AISC } \textit{Spec}\text{. Eq. F9-3)}$$

$$\begin{aligned} &= (50 \text{ ksi})(5.07 \text{ in.}^3) \\ &= 254 \text{ kip-in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right) \\ &= 21.1 \text{ kip-ft} \end{aligned}$$

$$\begin{aligned} M_{px} &= F_y Z_x \\ &= (50 \text{ ksi})(9.15 \text{ in.}^3) \\ &= 458 \text{ kip-in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right) \\ &= 38.1 \text{ kip-ft, but must be} \le 1.6M_y = (1.6)(21.1 \text{ kip-ft}) = 33.8 \text{ kip-ft} \end{aligned}$$

Therefore, use M_{px} = 33.8 kip-ft.

$$M_{nx} = M_{px} - (M_{px} - M_y)\left(\frac{L_b - L_p}{L_r - L_p}\right) \le 1.6M_y \qquad \text{(AISC } \textit{Spec}\text{. Eq. F9-6)}$$

$$\begin{aligned} &= (33.8 \text{ kip-ft}) - \left[(33.8 \text{ kip-ft}) - (21.1 \text{ kip-ft})\right]\left(\frac{10.0 \text{ ft} - 8.67 \text{ ft}}{151 \text{ ft} - 8.67 \text{ ft}}\right) \\ &= 33.7 \text{ kip-ft} < 1.6M_y = 33.8 \text{ kip-ft} \end{aligned}$$

Therefore, M_{nx} = 33.7 kip-ft and M_n/Ω_b= (33.7 kip-ft)/(1.67) = 20.2 kip-ft.

Column Action

Check the section element width-to-thickness ratios using the limits given in AISC *Specification* Table B4.1a.

For Case 1(5), the flange element nonslender limit is:

$$\begin{aligned} \lambda_{rf} &= 0.56\sqrt{\frac{E}{F_y}} \\ &= 0.56\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 13.5 \end{aligned}$$

Because $\lambda_f = \dfrac{b_f}{2t_f} = 7.75 < \lambda_{rf} = 13.5$, the flange element is nonslender.

For Case 4, the web element nonslender limit is:

$$\begin{aligned} \lambda_{rw} &= 0.75\sqrt{\frac{E}{F_y}} \\ &= 0.75\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} \\ &= 18.1 \end{aligned}$$

Because $\lambda_w = \dfrac{d}{t_w} = 18.5 > \lambda_{rw} = 18.1$, the web element is slender.

Flexural Buckling Strength

AISC *Specification* Section E3 applies.

$$\frac{L_{cx}}{r_x} = \frac{(10\text{ ft})(12\text{ in./ft})}{1.80\text{ in.}}$$
$$= 66.7 < 200 \quad \textbf{o.k.}$$

$$\frac{L_{cy}}{r_y} = \frac{(10\text{ ft})(12\text{ in./ft})}{2.45\text{ in.}}$$
$$= 49.0 < 200 \quad \textbf{o.k.}$$

Use $\frac{L_c}{r} = 66.7$.

In the following equations, subscripts *FB* and *FTB* are used to distinguish between values calculated for flexural buckling and flexural-torsional buckling, respectively.

$$(F_e)_{FB} = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \quad \text{(AISC } \textit{Spec}\text{. Eq. E3-4)}$$
$$= \frac{\pi^2(29{,}000\text{ ksi})}{(66.7)^2}$$
$$= 64.3\text{ ksi}$$

Because $\frac{F_y}{(F_e)_{FB}} = \frac{50.0\text{ ksi}}{64.3\text{ ksi}} = 0.778 < 2.25$, use AISC *Specification* Equation E3-2.

$$(F_n)_{FB} = \left[0.658^{\frac{F_y}{F_e}}\right](F_y) \quad \text{(AISC } \textit{Spec}\text{. Eq. E3-2)}$$
$$= \left[0.658^{(0.778)}\right](50\text{ ksi})$$
$$= 36.1\text{ ksi}$$

Flexural-Torsional Buckling Strength (AISC *Specification* Section E4 applies)

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{L_{cy}}{r_y}\right)^2} \quad \text{(AISC } \textit{Spec}\text{. Eq. E4-6)}$$
$$= \frac{\pi^2(29{,}000\text{ ksi})}{(49.0)^2}$$
$$= 119\text{ ksi}$$

From the AISC *Shapes Database* V16.0 (2023b), $H = 0.915$ and $\bar{r}_o = 3.17$ in. Alternatively, AISC *Specification* Equations E4-8 and E4-9 can be used to calculate H and $\bar{r}_o^2$.

Calculate F_{ez} as follows.

$$F_{ez} = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \qquad \text{(AISC } \textit{Spec}\text{. Eq. E4-7)}$$

$$= \left\{\frac{\pi^2 (29{,}000 \text{ ksi})(2.29 \text{ in.}^6)}{\left[(10 \text{ ft})(12 \text{ in./ft})\right]^2} + (11{,}200 \text{ ksi})(1.09 \text{ in.}^4)\right\}\left[\frac{1}{(8.96 \text{ in.}^2)(3.17 \text{ in.})^2}\right]$$

$$= 136 \text{ ksi}$$

Calculate $(F_e)_{FTB}$ as follows.

$$(F_e)_{FTB} = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right] \qquad \text{(AISC } \textit{Spec}\text{. Eq. E4-3)}$$

$$= \left[\frac{119 \text{ ksi} + 136 \text{ ksi}}{2(0.915)}\right]\left[1 - \sqrt{1 - \frac{4(119 \text{ ksi})(136 \text{ ksi})(0.915)}{(119 \text{ ksi} + 136 \text{ ksi})^2}}\right]$$

$$= 97.8 \text{ ksi}$$

$$\frac{F_y}{(F_e)_{FTB}} = \frac{50.0 \text{ ksi}}{97.8 \text{ ksi}}$$

$$= 0.511 < 2.25$$

AISC *Specification* Equation E3-2 applies.

$$(F_n)_{FTB} = \left[0.658^{\frac{F_y}{F_e}}\right](F_y) \qquad \text{(AISC } \textit{Spec}\text{. Eq. E3-2)}$$

$$= \left[0.658^{(0.511)}\right](50 \text{ ksi})$$

$$= 40.4 \text{ ksi}$$

$$(F_n)_{FTB} = 40.4 \text{ ksi} > (F_n)_{FB} = 36.1 \text{ ksi}$$

Therefore, $F_n = (F_n)_{FB} = 36.1$ ksi.

Effective Area

Recall that $\lambda_w = \dfrac{d}{t_w} = 18.5 > (\lambda_r)_w = 18.1$. Therefore, the web element is slender.

$$\lambda_r\sqrt{\frac{F_y}{F_n}} = (18.1)\sqrt{\frac{50 \text{ ksi}}{36.1 \text{ ksi}}} \qquad \text{(AISC } \textit{Spec}\text{. Eq. E7-2)}$$

$$= 21.3$$

$$\lambda = 18.5 < \lambda_r\sqrt{\frac{F_y}{F_n}} = 21.3$$

AISC *Specification* Equation E7-2 applies. The stem is fully effective; $d_e = d = 6.95$ in. and $A_e = A = 8.96$ in.^{2}

$$P_n = F_n A_g$$

$$= (36.1 \text{ ksi})(8.96 \text{ in.}^2)$$

$$= 323 \text{ kips}$$

$$P_n/\Omega = 323 \text{ kips}/1.67$$

$$= 193 \text{ kips}$$

Required Flexural Strength, Including Second-Order Effects

In the following calculations, the available strength in compression of the eccentrically loaded member is referred to as $(P_{n\,ecc})/\Omega_c$. Note that this replaces P_a used in Equation 12 as well as in the interaction equations later.

The required moment in the member, including second-order effects, is calculated as follows.

First, calculate B_{1x}.

$$P_{e1x} = \frac{\pi^2 EI_x}{(L_{c1x})^2} \quad \text{(AISC } \textit{Spec}\text{. Eq. A-8-5)}$$

$$= \frac{\pi^2 (29{,}000 \text{ ksi})(28.9 \text{ in.}^4)}{\left[(10 \text{ ft})(12 \text{ in./ft})\right]^2}$$

$$= 574 \text{ kips}$$

and

$$B_{1x} = \frac{P_{e1x}}{P_{e1x} - 1.6P_a} \quad (12)$$

$$= \frac{574 \text{ kips}}{574 \text{ kips} - 1.6\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)}$$

$$= \frac{359 \text{ kips}}{359 \text{ kips} - \left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)}$$

Use Equation 1 to calculate the total load eccentricity as follows.

$$e = 0.5t_{pl} + \bar{y} \quad (1)$$

$$= (0.50)(0.50 \text{ in.}) + 1.25 \text{ in.}$$

$$= 1.50 \text{ in.}\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)$$

$$= 0.125 \text{ ft}$$

Recall from Equation 11 that

$$M_{ax} = B_{1x}P_a e$$

$$= \left[\frac{359 \text{ kips}}{359 \text{ kips} - \left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)}\right]\left(\frac{P_{n\,ecc}}{\Omega_b}\right)(0.125 \text{ ft})$$

$$= \frac{44.9\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)}{359 - \left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)} \text{ kip-ft}$$

Check the Appropriate Interaction Equation for Combined Forces

Assume $\dfrac{\Omega_c P_a}{P_n} = \dfrac{\Omega_c}{P_n}\left(\dfrac{P_{n\,ecc}}{\Omega_c}\right) \geq 0.20$, and apply the ASD version of AISC *Specification* Equation H1-1a as follows. Set the interaction equation equal to unity (1.0) to find the member available eccentric axial compressive strength. Note that $\Omega_c = \Omega_b = 1.67$ and $M_{ay} = 0$. Also, recall that, $P_n/\Omega_c = 193$ kips, $M_{nx}/\Omega_b = 20.2$ kip-ft, and P_a is replaced by $\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)$.

$$\frac{P_a}{\left(\dfrac{P_n}{\Omega_c}\right)} + \frac{8}{9}\left[\frac{M_{ax}}{\left(\dfrac{M_{nx}}{\Omega_b}\right)} + \frac{M_{ay}}{\left(\dfrac{M_{ny}}{\Omega_b}\right)}\right] = \frac{\Omega_c P_a}{P_n} + \frac{8}{9}\left(\frac{\Omega_b M_{ax}}{M_{nx}} + 0\right) = 1.0$$

or

$$\frac{\Omega_c}{P_n}\left(\frac{P_{n\,ecc}}{\Omega_b}\right) + \frac{8}{9}\left(\frac{\Omega_b M_{ax}}{M_{nx}}\right) = 1.0$$

Insert values of available strengths and replace M_{ax} with its equivalent found earlier.

$$\frac{\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)}{193 \text{ kips}} + \frac{8\left[\dfrac{44.9\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)}{359-\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)} \text{ kip-ft}\right]}{9(20.2 \text{ kip-ft})} = \frac{\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)}{193} + \left[\frac{1.98\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)}{359-\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)} \text{ kip-ft}\right]$$
$$= 1.0$$

This is a quadratic equation with a root $\dfrac{P_{n\,ecc}}{\Omega_b} = 81.5$ kips (solution details not shown).

Check the value of B_{1x} using the equation developed previously.

$$\begin{aligned} B_{1x} &= \frac{359 \text{ kips}}{359 \text{ kips} - \left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)} \\ &= \frac{359 \text{ kips}}{359 \text{ kips} - 81.5 \text{ kips}} \\ &= 1.29 > 1.0 \quad \textbf{o.k.} \end{aligned}$$

Check the assumption that $\dfrac{\Omega_c P_a}{P_n} = \dfrac{\Omega_c}{P_n}\left(\dfrac{P_{n\,ecc}}{\Omega_c}\right) \geq 0.20$.

$$\begin{aligned} \frac{\Omega_c P_a}{P_n} &= \frac{\left(\dfrac{P_{n\,ecc}}{\Omega_b}\right)}{193 \text{ kips}} \\ &= \frac{81.5 \text{ kips}}{193 \text{ kips}} \\ &= 0.422 > 0.20 \quad \textbf{o.k.} \end{aligned}$$

Had this assumption turned out to be incorrect, the solution would be repeated using AISC *Specification* Eq. H1-1b instead of AISC *Specification* Eq. H1-1a.

All assumptions made earlier were proven correct. Therefore, the available eccentric compressive strength of the WT7×30.5 compression member is $P_a = 81.5$ kips.

As a check, calculate the compressive strength of the gusset plate assuming that flexural buckling does not apply. See AISC *Specification* Section J4.4(a) and Equation J4-6.

$$\begin{aligned}(A_g)_{pl} &= (b)(t)\\ &= (10.0\text{ in.})(0.50\text{ in.})\\ &= 5.00\text{ in.}^2\end{aligned}$$

$$\begin{aligned}(P_n)_{pl} &= (F_y A_g)_{pl}\\ &= (50\text{ ksi})(5.00\text{ in.}^2)\\ &= 250\text{ kips}\end{aligned}$$

$$\begin{aligned}\left(\frac{P_n}{\Omega_c}\right)_{pl} &= \left(\frac{P_n}{1.67}\right)_{pl}\\ &= \frac{250\text{ kips}}{1.67}\\ &= 150\text{ kips} > 81.5\text{ kips}\end{aligned}$$

DEVELOPMENT OF EQUATIONS FOR THE DESIGN TABLE VALUES

Development of the equations for the available compressive strength of eccentrically loaded WT compression members, $\phi c P_{n\ ecc}$ (LRFD) or $(P_{n\ ecc})/\Omega c$ (ASD), including a second-order effect, based on the AISC *Specification* Section H1.1 provisions follows.

LRFD Method

Given the dominance of the axial force and that the bending moment present in the member is due to the load eccentricity, it is expected that in a majority of cases, the ratio $P_u/\phi_c P_n \geq 0.20$.

When $\dfrac{P_u}{\phi_c P_n} \geq 0.20$, the LRFD version of AISC *Specification* Equation H1-1a applies as follows.

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0$$

(AISC *Spec*. Eq. H1-1a)

To obtain the available compressive strength of the eccentrically loaded member, set the preceding interaction equation equal to 1.0. At the same time, note that there is no bending about the y-axis of the member; therefore, $M_{uy} = 0$. AISC *Specification* Equation H1-1a then simplifies to the following.

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}}\right) = 1.0 \tag{13}$$

Recall from Equation 5 that,

$$M_{ux} = B_{1x} P_u e \tag{5}$$

Replace P_u with $\phi_c P_{n\ ecc}$ in Equation 5.

$$M_{ux} = B_{1x}\left(\phi_c P_{n\,ecc}\right)e \tag{14}$$

Substituting M_{ux} from Equation 14 and replacing P_u with $\phi_c P_{n\ ecc}$ into Equation 13, we obtain the following.

$$\begin{aligned}\frac{\phi_c P_{n\ ecc}}{\phi_c P_n} + \frac{8}{9}\left[\frac{B_{1x}(\phi_c P_{n\ ecc})e}{\phi_b M_{nx}}\right] &= (\phi_c P_{n\ ecc})\left(\frac{1}{\phi_c P_n} + \frac{8B_{1x}e}{9\phi_b M_{nx}}\right)\\ &= 1.0\end{aligned} \tag{15}$$

Equation 15 is simplified to the following.

$$(\phi_c P_{n\ ecc})\left(\frac{9\phi_b M_{nx} + 8B_{1x}e\phi_c P_n}{9\phi_c P_n \phi_b M_{nx}}\right) = 1.0 \tag{16}$$

or

$$\phi_c P_{n\ ecc} = \left(\frac{9\phi_c P_n \phi_b M_{nx}}{9\phi_b M_{nx} + 8B_{1x}e\phi_c P_n}\right) \tag{17}$$

Simplify Equation 17 using $\phi_b = \phi_c = 0.90$ to obtain the following equation, which is in terms of nominal strengths only. Note that this is still an LRFD equation.

$$\phi_c P_{n\ ecc} = \frac{8.1 P_n M_{nx}}{9M_{nx} + 8B_{1x}P_n e} \tag{18}$$

When $\frac{P_u}{\phi_c P_n} = \frac{\phi_c P_{n\,ecc}}{\phi_c P_n} < 0.20$, the LRFD version of AISC *Specification* Equation H1-1b applies, and the following equation is developed.

$$\phi_c P_{n\,ecc} = \frac{1.8 P_n M_{nx}}{M_{nx} + 2B_{1x} P_n e} \tag{19}$$

ASD Method

A process similar to the one used for the LRFD method leads to the following equations for the eccentric available compressive strength of the member (details not shown).

When $\frac{\Omega_c P_a}{P_n} = \frac{\Omega_c \left(\frac{P_{n\,ecc}}{\Omega_c}\right)}{P_n} \geq 0.20$,

$$\left(\frac{P_{n\,ecc}}{\Omega_c}\right) = \frac{P_n M_{nx}}{1.67 M_{nx} + 1.48 B_{1x} P_n e} \tag{20}$$

When $\frac{\Omega_c \left(\frac{P_{n\,ecc}}{\Omega_c}\right)}{P_n} < 0.20$,

$$\left(\frac{P_{n\,ecc}}{\Omega_c}\right) = \frac{P_n M_{nx}}{0.835 M_{nx} + 1.67 B_{1x} P_n e} \tag{21}$$

Values of P_n and M_{nx} are calculated based on the appropriate provisions of the AISC *Specification* as listed in Table 1.

DEVELOPMENT OF THE DESIGN TABLE

Equations developed earlier are summarized in Table 2. These equations were used in the development of the proposed design table (Table 4).

Note that e, P_n, P_{e1x}, and M_{nx} are all known values. The only unknown in the equations of Table 2 is the desired available compressive strength, $\phi_c P_{n\,ecc}$ (LRFD) or $(P_{n\,ecc})/\Omega_c$ (ASD).

Equations for $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$ are all a function of B_{1x}, which itself is a function of the available eccentric axial load $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$. Therefore, equations of Table 2 are all quadratic equations in terms of available strengths $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$. For example, the quadratic equation for case 1 of Table 2 is as follows.

$$9M_{nx}(\phi_c P_{n\,ecc})^2 - (\phi_c P_{n\,ecc})(9M_{nx}P_{e1x} + 8eP_nP_{e1x} + 8.1P_nM_{nx}) + 8.1P_nM_{nx}P_{e1x} = 0 \tag{22}$$

with

$$\phi_c P_{n\,ecc} = \frac{(9M_{nx}P_{e1x} + 8eP_nP_{e1x} + 8.1P_nM_{nx}) \pm \sqrt{(9M_{nx}P_{e1x} + 8eP_nP_{e1x} + 8.1P_nM_{nx})^2 - 4(9M_{nx})(8.1P_nM_{nx}P_{e1x})}}{18M_{nx}} \tag{23}$$

Given the complexity of the quadratic equations for $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$ such as Equation 22 and its solution, Equation 23, it was decided to use an iterative process to find the available strengths $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$ using a spreadsheet. The procedure used is as follows.

1. Calculate $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$ using the equation for case 1 (LRFD) or 3 (ASD) from Table 2; initially, use $B_{1x} = 1.0$.
2. Use the value of $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$ found in step 1 to calculate a new B_{1x} using Equation 7 (LRFD) or 12 (ASD).
3. Use this new value of B_{1x} in the equation for case 1 (LRFD) or 3 (ASD) from Table 2 to calculate a new value for $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$.
4. Use the value of $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$ found in step 3 to calculate a new B_{1x} using Equation 7 (LRFD) or 12 (ASD).
5. Repeat this process until the values of $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$ and B_{1x} stabilize.

If calculations resulted in $\frac{\phi_c P_{n\,ecc}}{\phi_c P_n} < 0.20$ (LRFD) or $\frac{\Omega_c \left(\frac{P_{n\,ecc}}{\Omega_c}\right)}{P_c} < 0.20$ (ASD), equations in case 2 (LRFD) or case 4 (ASD) of Table 2 were used as appropriate. In the worst-case scenario, seven iterations were used in the spreadsheet to calculate the tabulated $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$ values with the largest difference between $\phi_c P_{n\,ecc}$ or $(P_{n\,ecc})/\Omega_c$ of the last two iterations being 0.68%.

Tabulated member strengths were calculated in the spreadsheet for effective lengths of 0 to 40 ft at 1 ft increments. However, the design table skips some unbraced lengths due to limited space per page as is the case in AISC *Manual* tables.

Table 4 values follow the practice of using three significant figures. For more information on this topic as well as on the use of interpolation between tabulated values see the section titled Using the Manual Tables in Part 2 of the AISC *Manual*.

Table 1. Applicable Limit States and Appropriate Provisions/Equations of the AISC *Specification*[1]		
Limit State[2]	**AISC *Specification* Provisions**	**AISC *Specification* Equation**
	Flexure[3]	
Yielding	F9.1	F9-1 to F9-3
Flange local buckling[4]	F9.3(a)(2)	F9-14
Lateral torsional buckling	F9.2(a)(1) to F9.2(a)(3)	F9-6 to F9-11
	Compression	
Flexural buckling	E3	E3-1 to E3-4
Stem local buckling[5]	E7.1	E7-1 to E7-5
Flexural-torsional buckling	E4(b)	E4-1, E4-3, and E4-5 to E4-9[6]

[1] For WT shapes made from column section listed in AISC *Manual* Table 4-1a.
[2] For $F_y = 50$ ksi.
[3] Stem in tension for flexure.
[4] Only WT7×49.5, WT7×45, WT6×32.5, and WT4×15.5 have noncompact flange elements under flexural compression. There are no WT shapes among those under consideration with slender flange elements under flexural compression.
[5] Only WT7×30.5, WT7×26.5, WT7×24, WT7×21.5, and WT6×20 have slender stems under axial compression. There are no WT shapes among those under consideration with slender flange element under axial compression.
[6] Values of r_o and H are listed in the AISC *Shapes Database*, but not in AISC *Manual* Table 1-8.

Table 2. Equations for Calculating the Available Eccentric Axial Compressive Strength of WTs		
Case	**Axial Load Ratio,** $\frac{P_r}{P_c}$	**Equation**
	LRFD Method[a]	
1	$\frac{\phi_c P_{n\ ecc}}{\phi_c P_n} \geq 0.20$	$\phi_c P_{n\ ecc} = \frac{8.1 P_n M_{nx}}{9M_{nx} + 8B_{1x} P_n e}$ (Eq. 18)
2	$\frac{\phi_c P_{n\ ecc}}{\phi_c P_n} < 0.20$	$\phi_c P_{n\ ecc} = \frac{1.8 P_n M_{nx}}{M_{nx} + 2B_{1x} P_n e}$ (Eq. 19)
	ASD Method[b]	
3	$\frac{\Omega_c \left(\frac{P_{n\ ecc}}{\Omega_c}\right)}{P_n} \geq 0.20$	$\left(\frac{P_{n\ ecc}}{\Omega_c}\right) = \frac{P_n M_{nx}}{1.67 M_{nx} + 1.48 B_{1x} P_n e}$ (Eq. 20)
4	$\frac{\Omega_c \left(\frac{P_{n\ ecc}}{\Omega_c}\right)}{P_n} < 0.20$	$\left(\frac{P_{n\ ecc}}{\Omega_c}\right) = \frac{P_n M_{nx}}{0.835 M_{nx} + 1.67 B_{1x} P_n e}$ (Eq. 21)

[a] $B_{1x} = \frac{P_{e1x}}{P_{e1x} - \phi_c P_{n\ ecc}}$ LRFD (Eq. 7: P_u replaced with $\phi_c P_{n\ ecc}$)

[b] $B_{1x} = \frac{P_{e1x}}{P_{e1x} - 1.6\left(\frac{P_{n\ ecc}}{\Omega_c}\right)}$ ASD (Eq. 12: P_a replaced with $P_{n\ ecc}/\Omega_c$)

ESTABLISHING THE GUSSET PLATE DIMENSIONS FOR DEVELOPMENT OF THE DESIGN TABLE

For the purposes of connection design, it was decided that the gusset plate width will be the same as b_f, the flange width of the connected WT. Larger plate widths may be selected with no adverse effects.

It is convenient to use a standard such as a multiple of the WT's flange thickness, t_f, for the plate thickness in developing Table 4.

In the first round of calculating the available strengths, a gusset plate thickness of $t_{pl} = 1.5t_f$ was used, where t_f is the thickness of the WT flange. This is a similar approach to the development of AISC *Manual* Table 4–12, Available Strength in Axial Compression, Eccentrically Loaded Single Angles, where $t_{pl} = 1.5t$ was used with t being the thickness of the angle.

Using $t_{pl} = 1.5t_f$ resulted in plate thicknesses that were unreasonably large. The fact is that load eccentricity significantly reduces the available strength of WT compression members. Therefore, the $t_{pl} = 1.5t_f$ assumption resulted in considerably overdesigned plates. Further, a larger plate thickness results in a larger eccentricity (see Equation 1) and leads to a lower available compressive strength of the member.

Several plate thicknesses were tried in the spreadsheet to calculate the available compressive strength while ensuring that the plate had sufficient compressive strength to resist the applied load. This process involved taking the largest available strength of each member and calculating the required plate thickness to resist this load using F_y = 50 ksi and assuming that the gusset plate is as wide as b_f of the WT. In the end, it was determined that a standard plate thickness $t_{pl} = 0.6t_f$ works for all shapes. The calculated values of plate thicknesses were then rounded up to practical plate thicknesses considering availability.

The thickest plate needed based on the preceding approach is 3¾ in. (for WT7×436.5), which still meets the thickness limitations of plates available in the preferred material specification (A572/A572M Grade 50 steel) per AISC *Manual* Table 2–5.

LIMITATIONS OF UNBRACED LENGTHS

As discussed earlier under the section titled "Clarification of Laterally Unbraced Lengths for WT Compression Members," the laterally unbraced length for flexural buckling about the y-axis, L_y, and the laterally unbraced length for lateral torsional buckling, L_b, are equal ($L_y = L_b$).

Further, all equations of Table 2 include P_n, the nominal concentric compressive strength of the member. P_n itself depends on both L_{cx} and L_{cy}. Therefore, it was assumed in the development of the table that $L_{cx} = L_{cy}$. Otherwise, the table would be three-dimensional. In summary, the values of Table 4 assume that $L_{cx} = L_{cy} = L_{cz} = L_b$. Note that based on the member end connections, $K_x = K_y = K_z = 1.0$. Therefore, laterally unbraced lengths and the corresponding effective lengths are equal, namely, $L_{cx} = L_x$, $L_{cy} = L_y$, and $L_{cz} = L_z$.

There is no interaction between the member strength for flexural torsional buckling about x-axis versus the y-axis and z-axis buckling. The values of Table 4 are all based on L_{cx} which governs member strength in all cases.

ASSUMPTIONS MADE IN THE DEVELOPMENT OF THE DESIGN TABLE VALUES

The following is a list of assumptions made in the development of the design table.

1. WTs considered are those made from W-shapes ordinarily used as columns per AISC *Manual* Table 4–1a (some very large shapes excluded).
2. ASTM A992/A992M (F_y = 50 ksi) steel is used for WT shapes.
3. The width of the gusset plate was taken as the flange width, b_f, of the connected WT shape. Wider plates may be used without adverse effect.
4. Plates are of A572/A572M Grade 50 steel.
5. The gusset plates have a thickness of about $t_{pl} = 0.6t_f$, where t_f is the thickness of the flange of the WT member. Plate thicknesses used and identified at the top of each column in Table 4 were rounded up to the next practical available plate thickness.
6. The compression load is applied at the mid-width of the flange of the WT and at mid-thickness of the gusset plate.
7. No transverse loads are applied between the member ends.
8. The member is braced against lateral displacement at the ends only.
9. There is no relative lateral translation of the member ends ($P_{lt} = M_{lt} = 0$) and thus no P-D effect.
10. There is no bending about the y-axis of the member.
11. Based on the end connections described earlier, the member will act as pinned at the ends for flexural and flexural-torsional buckling limit states ($K_x = K_y = K_z$ = 1.0).
12. Unbraced lengths are identified as follows.
    a. For flexural buckling, L_{cx} and L_{cy}, ft (mm).
    b. For flexural-torsional buckling, L_{cz}, ft (mm).

c. For lateral-torsional buckling, L_b, ft (mm).

d. $L_{cx} = L_{cy} = L_{cz} = L_b$, ft (mm).

13. Design of the member end connections, including bolted or welded joints between the gusset plates and the flange of the WT compression member, is not addressed in this article. It is assumed that the designer will establish appropriate designs for those components.

14. Member weight is not accounted for in the development of the design table as is the case with AISC *Manual* tables.

OBSERVATIONS FROM THE DATA

Variation of Nominal Flexural Strength (M_n) and Nominal Compressive Strengths (P_n)

The nominal flexural strengths, M_n, of the WT members considered over the unbraced lengths 0 ft $\le L_b \le$ 40 ft do not change significantly (Figure 7). But their nominal concentric compressive strengths, P_n, follow the typical pattern of singly symmetric compression members with considerable change of strength over the range of the unbraced lengths with flexural-torsional buckling governing over very short unbraced lengths.

Generally, the values of L_r for WT flexural members are unusually large. Among the WT members considered in this study, the smallest value of L_r is 89.5 ft, which is far larger than any practical unbraced length. This is consistent with AISC *Specification* Commentary Section F9, Tees and Double Angles Loaded in the Plane of Symmetry, which states, "For most shapes, the length, L_r, is impractically long." Therefore, it can be concluded that elastic lateral torsional buckling limit state does not govern the design of any of the WT shapes and for the range of unbraced lengths (0 ft $\le L_b \le$ 40 ft) considered in this study.

Per AISC *Specification* Equation F9-6, the slope of the chart for the nominal flexural strength, M_n, over the range $L_p < L_b \le L_r$ of the WTs considered is very shallow. In all but a few cases (WT7×213 to WT7×436.5), the slope of this portion of the chart is less than 0.10 kip-ft/ft. In other words, while the nominal flexural strength, M_n, remains

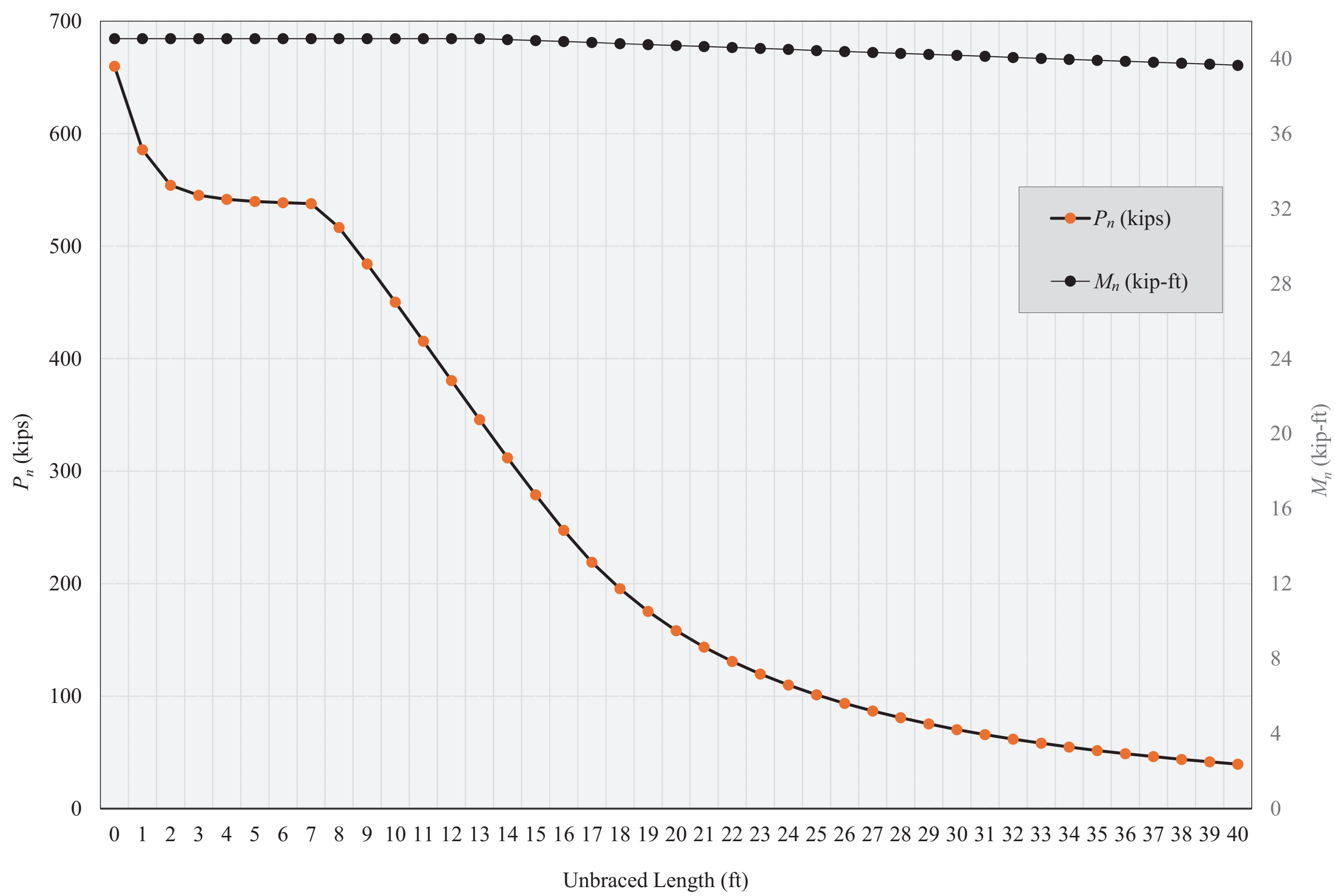


Fig. 7. Variations of P_n *and* M_n *versus unbraced length for a* WT7×45.

Table 3. Sample Variation of M_n Over the Range $0 \le L_b \le 40$ ft			
	(kip-ft)		
Shape	$L_b = 0$ ft	$L_b = 40$ ft	Change (%)
WT7×436.5	873	868	0.576
WT7×199	251	248	1.21
WT7×96.5	96.0	94.4	1.69
WT5×24.5	15.9	15.1	5.29
WT4×15.5	8.53	7.89	8.11

constant over the range $L_b \le L_p$, it does not change considerably over the range $L_b > L_P$ either. Table 3 provides five examples over the range of heavy to light WT sections.

The largest change in M_n among all members over 0 ft ≤ $L_b \le 40$ ft is WT7×21.5 with $M_n = 26.5$ kip-ft at $L_b = 0$ ft and $M_n = 22.5$ kip-ft at $L_b = 40$ ft with a change in M_n of 17.8% (Figure 8). Based on the preceding discussion, it can be stated that for the WT shapes considered and for unbraced lengths used ($0 \le L_b \le 40$ ft), the values of the nominal flexural strength, M_n, are significantly less dependent on the unbraced length L_b than values of P_n are dependent on L_c.

The available strengths of WTs listed in Table 4 are functions of nominal flexural strength, M_n, and nominal (concentric) axial compressive strength, P_n, with the assumptions that $L_{cx} = L_{cy} = L_{cz} = L_b$. Under these circumstances, the available strength in compression based on L_{cx} governs for all shapes and values of unbraced length considered.

Variation of Available Eccentric Axial Compressive Strength versus Effective Length

The general shape of the charts for the available eccentric axial compressive strengths of WTs is similar to those

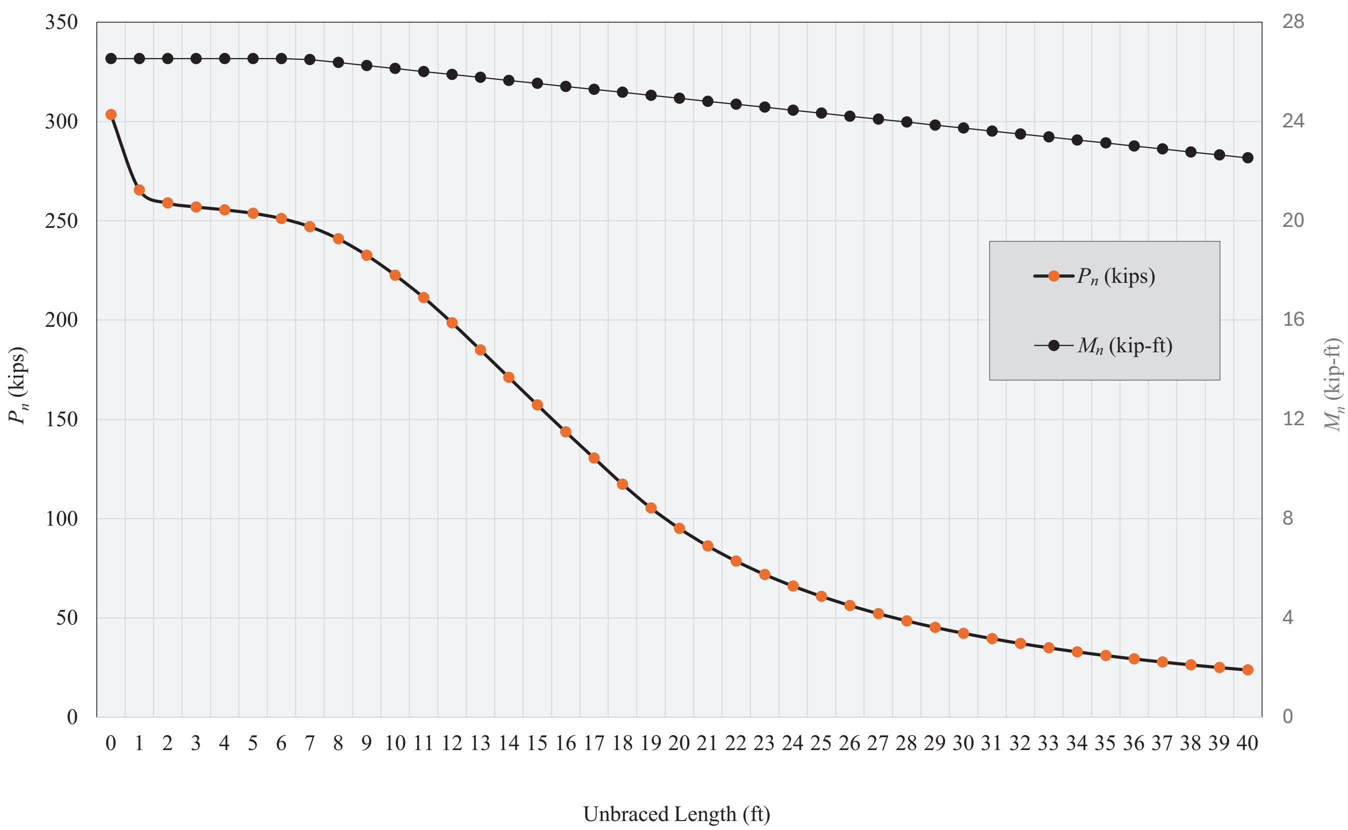


Fig. 8. Variations of P_n *and* M_n *versus unbraced length for a* WT7×21.5 *(worst-case scenario.)*

of members subject to concentric axial load. The available strength is reduced as the effective unbraced length increases. Further, the strength charts have similar curvature to concentrically loaded members—concave down over shorter unbraced lengths and concave up for larger lengths indicating inelastic and elastic flexural buckling. Figure 9 shows charts of the available eccentric axial compressive strengths of a number of WT members with relatively light to relatively heavy weights over the slenderness ratio range $L_c/r \leq 200$.

Cost of Eccentricity

As expected, there is a cost for applying the axial compressive load eccentrically to WT members. The load eccentricity causes bending moment in the member, reducing its remaining available strength in axial compression. For example, an 8-ft-long WT7×185 has an available concentric compressive strength $\phi_c P_n$ = 2,080 kips. However, for the same conditions, the available eccentric axial compressive strength is 589 kips, resulting in a ratio of available eccentric to concentric axial compressive strength of 0.283, reflecting a severe penalty due to the eccentricity.

Figure 10 shows the charts for available axial compressive strengths for concentric and eccentric loads applied to a WT7×185 compression member. Figure 11 shows the variation of the ratio of the available eccentric to concentric axial compressive strength of a WT7×185 over unbraced lengths between zero and 40 ft.

The cost of eccentricity is related to the member size and its unbraced length with it being considerably higher for heavy shapes with short unbraced lengths. In such cases, the high axial compressive strength causes large bending moments due to load eccentricity, leaving less available compressive strength for an eccentric load.

As observed from Figure 9, the ratios of eccentric to concentric available compressive strengths ranges from a minimum of 26.9% to 60.1% for a WT7×185, with the lowest values belonging to smaller unbraced lengths. For all members considered, the ratios of the available eccentric to concentric axial compressive strength ranged from a minimum of 0.184 to a maximum of 0.876. As the slenderness ratio increases, the effect of eccentricity is reduced, and flexural buckling strength becomes more dominant.

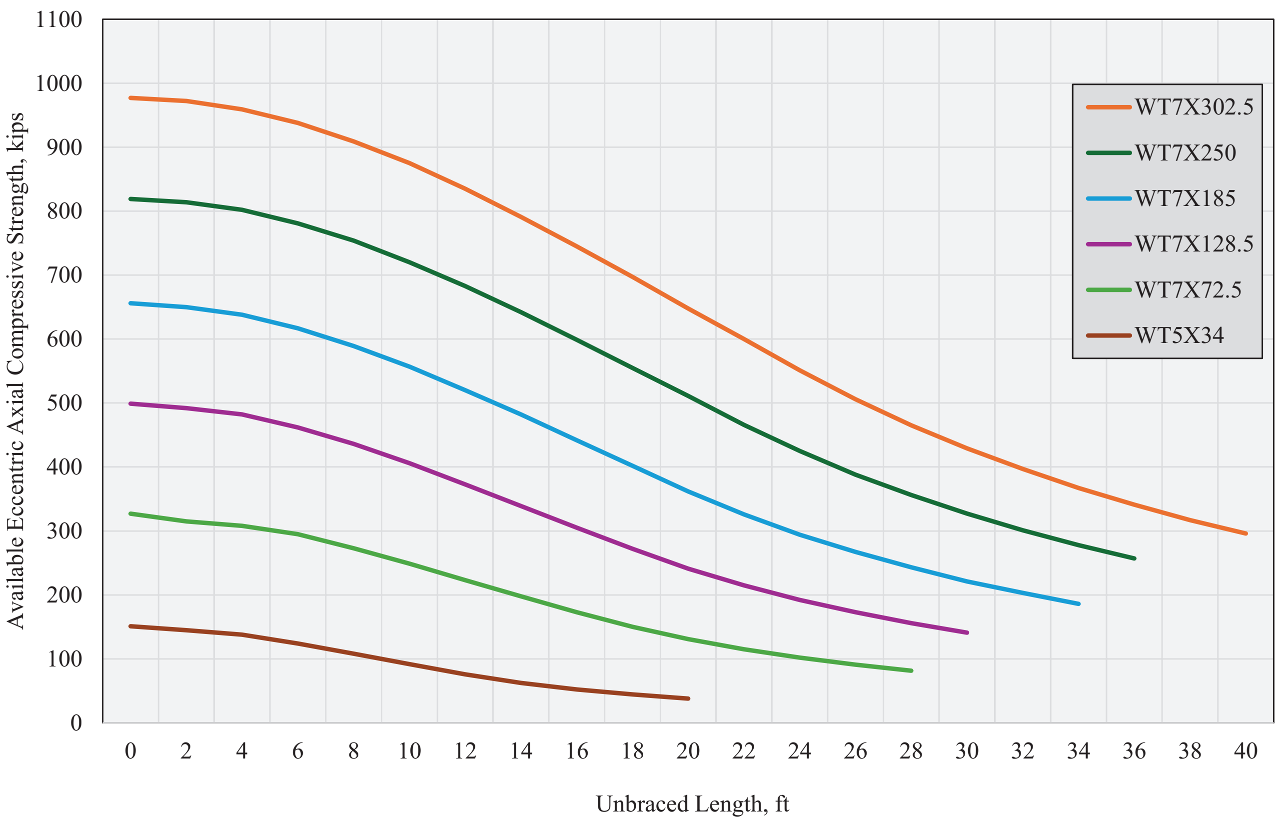


Fig. 9. Sample available strengths of eccentrically loaded WT compression members for $L_c/r \leq 200$.

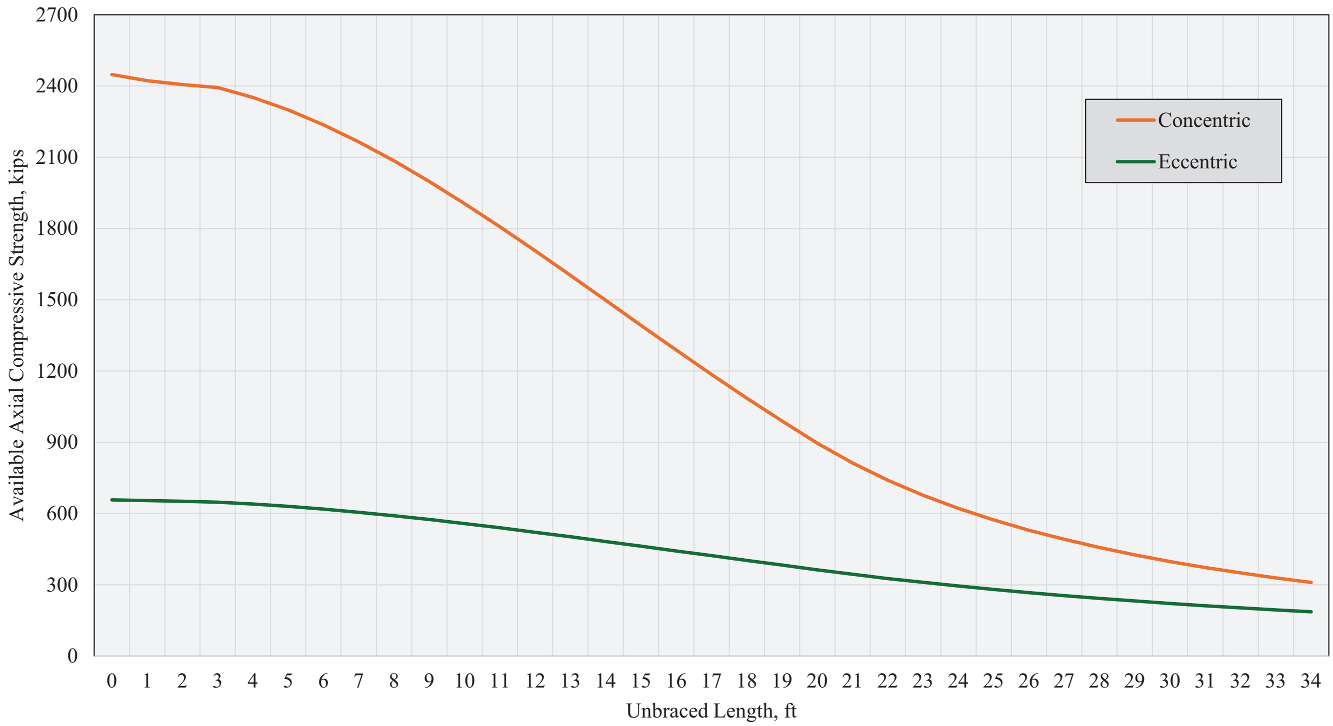


Fig. 10. Available concentric and eccentric axial compressive strengths of WT7×185.

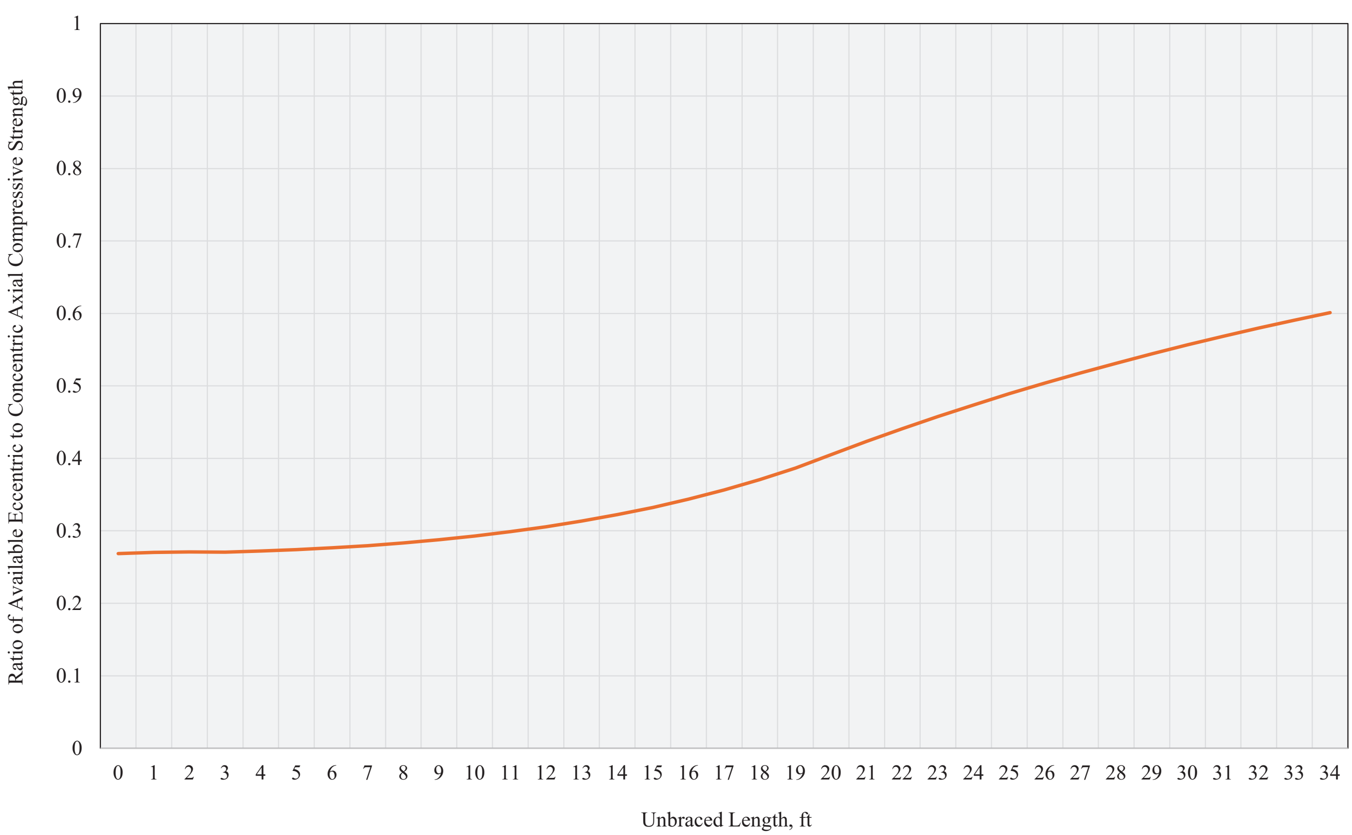
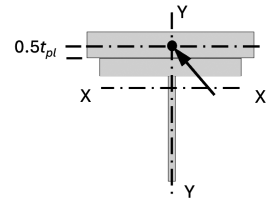


Fig. 11. Ratios of available eccentric to concentric axial compressive strengths of WT7×185.

F_y = 50 ksi

Table 4
Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT7×											
lb/ft		436.5[h]		404[h]		365[h]		332.5[h]		302.5[h]		275[h]	
t_{pl} (in.)		3¾		3½		3¼		3		2¾		2½	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	933	1400	871	1310	769	1150	708	1060	652	977	603	904
	1	932	1400	870	1300	768	1150	707	1060	650	975	602	902
	2	930	1390	868	1300	766	1150	705	1060	648	972	600	899
	3	926	1390	864	1300	762	1140	701	1050	645	967	596	894
	4	920	1380	858	1290	757	1140	696	1040	639	959	591	886
	5	913	1370	851	1280	750	1130	689	1030	633	950	584	877
	6	905	1360	842	1270	742	1110	681	1020	625	938	576	865
	8	884	1330	821	1230	722	1090	661	994	605	909	556	836
	10	858	1290	796	1200	698	1050	637	959	581	875	532	801
	12	828	1250	766	1150	670	1010	609	919	553	835	505	762
	14	794	1200	733	1110	639	964	579	874	524	791	476	719
	16	758	1150	697	1050	606	915	547	827	492	745	445	674
	18	721	1090	660	999	571	864	513	777	460	697	413	627
	20	681	1030	622	943	535	812	479	726	427	648	382	580
	22	641	973	583	885	500	759	444	675	394	600	350	533
	24	601	913	544	827	464	706	410	624	362	551	320	487
	26	561	854	506	770	429	652	377	575	332	506	292	445
	28	522	794	468	713	396	603	348	530	305	465	268	409
	30	485	738	434	662	367	559	321	490	281	429	246	376
	32	451	687	403	615	340	518	297	454	260	397	227	347
	34	420	641	375	572	316	482	276	421	241	367	210	321
	36	392	599	350	534	294	449	256	391	223	341	194	297
	38	367	560	327	499	274	419	239	365	208	317	181	276
	40	344	525	306	467	256	392	223	341	193	296		

Properties

A_g, in.^{2}	129	119	107	97.8	89.0	80.9
r_x, in.	2.84	2.75	2.62	2.52	2.43	2.34
r_y, in.	4.89	4.82	4.69	4.62	4.55	4.49
r_x/r_y	0.581	0.571	0.559	0.545	0.534	0.521

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 1.67	ϕ_c = 0.90

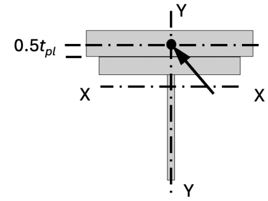
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1d.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

$F_y = 50$ ksi

Table 4 (continued)

Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT7×											
lb/ft		250[h]		227.5[h]		213[h]		199[h]		185[h]		171[h]	
t_{pl} (in.)		2½		2¼		2		2		1¾		1¾	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	546	819	509	763	487	731	455	683	437	656	405	608
	1	545	817	508	761	486	729	454	681	436	653	403	605
	2	543	814	506	758	484	725	452	678	433	650	401	602
	3	540	809	502	754	480	721	449	673	430	646	398	598
	4	534	802	497	746	475	713	443	666	425	638	393	590
	5	528	793	490	737	468	703	437	656	418	629	387	581
	6	520	781	483	725	460	692	429	645	410	617	379	570
	8	501	754	463	698	441	664	411	618	391	589	361	544
	10	478	720	441	664	418	630	388	586	369	557	339	512
	12	452	683	415	627	392	593	364	550	344	520	316	477
	14	424	642	388	587	365	553	338	511	318	482	291	441
	16	395	599	359	545	337	511	311	471	291	442	265	403
	18	366	555	331	502	309	469	284	431	264	402	240	365
	20	336	511	302	459	281	427	257	391	238	362	215	328
	22	306	466	274	417	254	386	232	353	214	326	193	295
	24	279	425	249	379	230	350	209	319	193	294	174	266
	26	255	388	226	345	209	318	190	290	175	267	158	241
	28	233	356	207	316	190	290	173	264	159	243	143	219
	30	214	327	189	289	174	266	158	242	145	221	130	199
	32	197	301	174	266	160	244	145	222	132	203	119	182
	34	182	278	160	245	147	225	133	204	122	186		
	36	168	257	148	227								
	38												
	40												
Properties													
A_g, in.^{2}		73.5		66.9		62.7		58.4		54.4		50.3	
r_x, in.		2.26		2.19		2.14		2.10		2.05		2.01	
r_y, in.		4.43		4.38		4.34		4.31		4.27		4.24	
r_x/r_y		0.510		0.500		0.493		0.487		0.480		0.474	

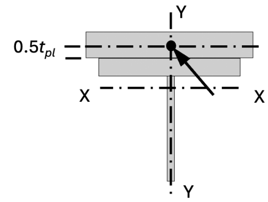
ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1d.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

$F_y = 50$ ksi

Table 4 (continued)
Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT7×											
lb/ft		155.5[h]		141.5[h]		128.5		116.5		105.5		96.5	
t_{pl} (in.)		1½		1½		1¼		1¼		1¼		1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	382	573	350	525	333	499	304	456	279	418	268	402
	1	380	570	348	522	330	495	301	452	276	414	265	397
	2	378	567	346	518	328	492	299	449	274	411	262	393
	3	375	563	343	515	325	488	296	445	271	407	259	389
	4	370	556	338	508	321	482	293	440	268	403	256	385
	5	364	547	332	499	314	473	287	431	262	394	251	377
	6	356	535	325	488	307	462	279	420	256	385	244	367
	8	338	509	307	463	289	436	262	396	239	361	227	343
	10	316	477	287	433	268	406	243	367	221	334	209	316
	12	292	442	264	400	246	373	222	337	201	305	189	287
	14	268	406	241	366	223	339	201	305	181	276	169	258
	16	243	369	218	332	201	305	180	273	162	246	150	228
	18	219	333	195	297	178	272	159	242	142	217	131	200
	20	195	297	174	265	158	241	140	214	126	192	115	176
	22	174	266	155	237	141	215	125	190	111	170	102	156
	24	157	239	139	212	126	192	111	170	99	152	90.7	139
	26	141	216	125	191	113	173	100	153	89.0	136	81.1	124
	28	128	196	113	173	102	156	90.0	138	80.2	123	72.9	112
	30	116	178	103	158	92.4	141	81.5	125	72.5	111		
	32	106	162	93.9	144								
	34												
	36												
	38												
	40												
Properties													
A_g, in.^{2}		45.7		41.6		37.8		34.2		31.0		28.4	
r_x, in.		1.96		1.92		1.88		1.84		1.81		1.78	
r_y, in.		4.20		4.17		4.13		4.10		4.07		4.05	
r_x/r_y		0.467		0.460		0.455		0.449		0.445		0.440	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

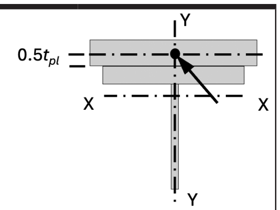
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1d.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4 (continued)

Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT7×											
lb/ft		88		79.5		72.5		66		60		54.5	
t_{pl} (in.)		1		1		¾		¾		⅝		⅝	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	248	372	226	339	218	327	202	304	191	287	175	262
	1	244	366	222	333	213	320	198	297	186	279	169	254
	2	241	362	219	329	210	315	195	292	183	274	166	249
	3	239	359	217	325	208	312	192	289	181	271	164	246
	4	236	355	214	321	205	308	190	285	178	268	161	242
	5	232	348	210	316	202	303	187	281	175	263	158	238
	6	225	339	204	307	196	295	182	274	171	258	155	234
	8	209	316	190	286	181	273	168	254	158	238	143	216
	10	192	290	173	262	164	249	153	231	143	216	129	196
	12	173	263	156	236	147	223	137	208	127	194	115	174
	14	155	235	139	211	130	198	121	184	112	171	101	153
	16	137	208	122	186	114	173	106	161	97.6	149	87.1	133
	18	119	182	106	162	98.5	150	91.8	140	84.3	129	75.0	115
	20	105	160	92.9	142	85.9	131	80.0	122	73.3	112	65.1	100
	22	92.4	141	81.9	125	75.5	115	70.3	108	64.3	98.3	57.0	87.2
	24	82.1	126	72.7	111	66.8	102	62.2	95.2	56.7	86.8	50.3	76.9
	26	73.3	112	64.8	99.2	59.5	91.0	55.4	84.8	50.4	77.2	44.6	68.3
	28	65.9	101	58.2	89.1	53.3	81.5	49.6	75.9	45.1	69.1	39.9	61.1
	30												
	32												
	34												
	36												
	38												
	40												

Properties

	88	79.5	72.5	66	60	54.5
A_g, in.^{2}	25.9	23.4	21.3	19.4	17.7	16.0
r_x, in.	1.76	1.73	1.71	1.73	1.71	1.68
r_y, in.	4.02	4.00	3.98	3.76	3.74	3.73
r_x/r_y	0.438	0.433	0.430	0.460	0.457	0.450

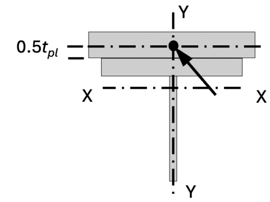
ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1d.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

$F_y = 50$ ksi

Table 4 (continued)
Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT7×											
lb/ft		49.5[f]		45[f]		41		37		34		30.5[c]	
t_{pl} (in.)		5⁄8		1⁄2		5⁄8		5⁄8		1⁄2		1⁄2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	161	242	153	229	139	209	127	190	120	180	109	163
	1	155	233	145	218	135	203	123	184	115	173	104	155
	2	151	227	141	212	134	200	121	181	113	170	102	153
	3	149	224	139	209	132	199	120	180	112	168	101	151
	4	147	221	137	206	131	196	118	178	111	166	99.3	149
	5	145	217	134	202	129	193	116	175	109	164	97.7	147
	6	142	213	132	198	126	189	114	172	107	161	95.8	144
	8	132	199	123	186	117	176	106	160	100	150	90.0	136
	10	119	180	111	168	107	161	96.3	146	90.3	137	81.5	123
	12	105	160	97.7	149	95.9	146	86.3	131	80.7	123	72.7	110
	14	92.0	140	85.1	130	85.3	130	76.5	116	71.3	108	64.1	97.5
	16	79.4	121	73.2	112	75.1	114	67.0	102	62.3	95.0	55.9	85.3
	18	68.3	104	62.7	95.9	65.4	100	58.2	88.9	53.9	82.4	48.3	73.8
	20	59.3	90.6	54.3	83.0	57.0	87.1	50.6	77.4	46.8	71.6	41.9	64.0
	22	51.9	79.3	47.4	72.5	50.0	76.5	44.4	67.9	41.0	62.7	36.6	56.0
	24	45.7	69.9	41.7	63.8	44.2	67.6	39.2	60.0	36.1	55.3	32.2	49.4
	26	40.5	62.1	36.9	56.6	39.3	60.2	34.8	53.3	32.1	49.1	28.6	43.8
	28					35.2	53.9	31.1	47.7	28.6	43.9	25.5	39.1
	30					31.6	48.5	28.0	42.9	25.7	39.4	22.9	35.1
	32												
	34												
	36												
	38												
	40												

Properties

	49.5	45	41	37	34	30.5
A_g, in.^{2}	14.6	13.2	12.0	10.9	10.0	8.96
r_x, in.	1.67	1.66	1.85	1.82	1.81	1.80
r_y, in.	3.71	3.70	2.48	2.48	2.46	2.45
r_x/r_y	0.450	0.449	0.746	0.734	0.736	0.735

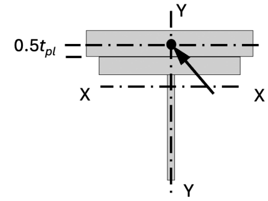
ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4 (continued)

Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT7×						WT6×					
lb/ft		26.5[c]		24[c]		21.5[c]		168[h]		152.5[h]		139.5[h]	
t_{pl} (in.)		½		½		⅜		2		2		1¾	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	96.0	144	87.6	131	80.2	120	379	569	343	515	327	490
	1	92.2	138	83.4	125	75.4	113	378	567	342	513	325	488
	2	90.9	137	82.3	124	74.1	111	376	564	341	511	324	486
	3	90.0	135	81.4	122	73.3	110	373	559	337	506	320	481
	4	88.8	133	80.4	121	72.4	109	368	552	333	499	316	474
	5	87.3	131	79.1	119	71.3	107	362	544	327	491	310	465
	6	85.4	129	77.4	117	69.8	105	355	533	320	481	303	455
	8	79.9	120	72.7	110	65.7	99.2	337	508	303	457	286	431
	10	73.0	110	66.5	101	60.3	91.2	317	478	284	429	266	402
	12	65.7	100	59.8	90.8	54.3	82.4	295	446	263	398	245	371
	14	58.4	88.9	53.2	80.9	48.1	73.2	271	411	241	365	223	339
	16	51.4	78.3	46.8	71.4	42.2	64.3	247	375	218	331	201	306
	18	44.7	68.3	40.7	62.2	36.6	55.9	223	339	196	298	180	273
	20	38.8	59.4	35.3	54.0	31.7	48.4	200	304	175	266	160	243
	22	34.0	52.0	30.9	47.3	27.7	42.3	180	274	157	239	143	218
	24	30.0	45.9	27.2	41.7	24.3	37.3	162	247	141	215	128	195
	26	26.6	40.8	24.2	37.0	21.6	33.0	146	224	127	195	115	176
	28	23.8	36.4	21.6	33.1	19.2	29.5	133	203	116	177	104	159
	30	21.4	32.7	19.4	29.7	17.2	26.4	121	185	105	161	94.8	145
	32							111	170				
	34												
	36												
	38												
	40												

Properties

	26.5	24	21.5	168	152.5	139.5
A_g, in.^{2}	7.80	7.07	6.31	49.5	44.7	41.0
r_x, in.	1.88	1.88	1.86	1.96	1.90	1.86
r_y, in.	1.92	1.91	1.89	3.47	3.42	3.38
r_x/r_y	0.979	0.984	0.984	0.565	0.556	0.550

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_c = 1.67	ϕ_c = 0.90

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1d.
Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

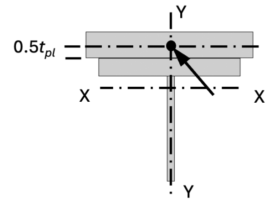


Table 4 (continued)
Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes

Shape	WT6×											
lb/ft	126[h]		115[h]		105		95		85		76	
t_{pl} (in.)	1½		1½		1¼		1¼		1¼		1	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft												
0	306	460	281	421	268	403	243	364	219	329	208	313
1	305	457	280	419	267	400	241	362	217	326	206	309
2	303	455	278	417	265	397	239	359	216	323	204	307
3	300	450	275	413	262	393	237	355	213	320	202	304
4	295	443	270	406	257	386	232	349	209	314	198	298
5	289	435	264	398	251	378	227	341	204	307	193	290
6	282	424	258	388	244	368	220	331	198	298	186	281
8	265	400	242	364	228	344	204	309	183	276	172	259
10	246	371	223	337	209	317	187	283	167	253	155	235
12	225	341	204	308	190	288	169	256	150	227	138	210
14	203	309	183	279	170	258	150	228	133	202	122	185
16	182	277	163	249	150	228	132	201	116	177	106	161
18	161	246	144	220	131	200	115	176	101	154	91.5	140
20	143	218	127	194	116	177	101	154	88.7	135	79.9	122
22	127	194	113	173	103	157	89.4	137	78.3	120	70.3	107
24	114	174	101	154	91.3	140	79.5	122	69.5	106	62.2	95.2
26	102	156	90.8	139	81.8	125	71.1	109	62.1	95.0	55.4	84.9
28	92.3	141	81.9	125	73.6	113	63.9	97.9				
30	83.7	128										
32												
34												
36												
38												
40												
Properties												
A_g, in.^{2}	37.1		33.8		30.9		28.0		25.0		22.4	
r_x, in.	1.81		1.77		1.73		1.68		1.65		1.62	
r_y, in.	3.34		3.31		3.28		3.25		3.22		3.19	
r_x/r_y	0.542		0.535		0.527		0.517		0.512		0.508	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

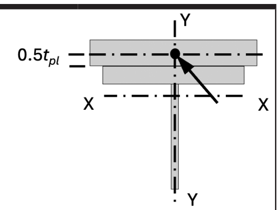
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1d.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

$F_y = 50$ ksi

Table 4 (continued)

Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT6×											
lb/ft		68		60		53		48		43.5		39.5	
t_{pl} (in.)		1		¾		¾		⅝		⅝		½	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	189	284	178	267	158	237	149	223	137	206	130	195
	1	187	280	175	262	155	232	145	218	133	200	125	188
	2	185	277	173	260	153	230	143	214	131	197	123	184
	3	183	274	171	257	151	227	141	212	129	194	121	182
	4	179	270	168	253	149	223	139	208	127	191	119	179
	5	174	262	163	245	144	217	135	203	124	187	116	175
	6	174	262	158	238	138	209	129	195	119	180	112	169
	7	167	252	151	228	132	200	123	186	114	171	106	161
	8	159	241	144	217	126	190	117	177	108	163	101	152
	9	151	229	136	206	119	180	110	167	101	154	94.7	143
	10	143	217	129	195	112	170	104	157	95.1	144	88.6	134
	11	135	205	121	184	105	159	96.8	147	88.8	135	82.5	125
	12	127	193	113	172	98.0	149	90.1	137	82.5	126	76.5	116
	13	119	180	106	161	91.2	139	83.6	127	76.4	116	70.7	108
	14	111	168	98.4	150	84.5	129	77.2	118	70.5	108	65.0	99.2
	15	103	157	91.1	139	78.0	119	71.1	108	64.8	98.9	59.7	91.1
	16	95.2	145	84.2	129	72.1	110	65.5	100	59.7	91.2	54.9	83.8
	18	82.1	125	72.5	111	61.9	94.6	56.1	85.7	51.0	78.0	46.8	71.5
	20	71.5	109	62.9	96.2	53.6	82.0	48.5	74.2	44.1	67.4	40.3	61.6
	22	62.7	95.9	55.1	84.2	46.9	71.7	42.3	64.7	38.4	58.7	35.0	53.6
	24	55.4	84.7	48.5	74.3	41.3	63.2	37.1	56.9	33.7	51.6	30.7	47.0
	26	49.2	75.3	43.1	66.0								
	28												
	30												

Properties

	68	60	53	48	43.5	39.5
A_g, in.^{2}	20.0	17.6	15.6	14.1	12.8	11.6
r_x, in.	1.59	1.57	1.53	1.51	1.50	1.49
r_y, in.	3.16	3.13	3.11	3.09	3.07	3.05
r_x/r_y	0.503	0.502	0.492	0.489	0.489	0.489

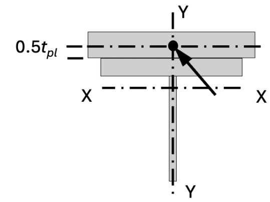
ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4 (continued)

Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes

$F_y = 50$ ksi



Shape	WT6×											
lb/ft	36		32.5[f]		29		26.5		25		22.5	
t_{pl} (in.)	½		½		½		⅜		½		⅜	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft: 0	120	180	110	164	97.9	147	94.3	141	86.9	130	81.4	122
1	115	172	104	156	93.8	141	89.3	134	83.6	125	77.5	116
2	112	169	101	152	92.2	138	87.6	131	82.4	124	76.3	115
3	111	166	100	150	90.9	137	86.3	130	81.4	122	75.3	113
4	109	164	97.9	147	89.4	134	84.8	128	80.1	120	74.1	111
5	106	160	95.8	144	87.4	132	83.0	125	78.4	118	72.5	109
6	103	156	93.3	141	84.3	127	80.9	122	76.0	114	70.5	106
7	98.0	148	89.0	134	80.1	121	76.9	116	72.5	109	67.5	102
8	92.6	140	84.0	127	75.7	115	72.6	110	68.8	104	63.9	96.6
9	86.9	132	78.8	119	71.2	108	68.1	103	64.9	98.3	60.2	91.1
10	81.2	123	73.5	112	66.5	101	63.6	96.5	61.0	92.5	56.4	85.5
11	75.6	115	68.3	104	61.9	94.1	59.1	89.9	57.1	86.7	52.7	80.0
12	70.0	107	63.2	96.2	57.4	87.3	54.7	83.3	53.2	80.9	49.0	74.5
13	64.6	98.4	58.2	88.7	53.0	80.7	50.5	76.9	49.4	75.3	45.4	69.2
14	59.3	90.5	53.4	81.5	48.7	74.4	46.4	70.8	45.8	69.8	42.0	64.0
15	54.4	83.0	48.9	74.6	44.7	68.2	42.4	64.8	42.3	64.5	38.7	59.0
16	49.9	76.3	44.9	68.6	41.1	62.7	38.9	59.5	38.9	59.4	35.5	54.2
18	42.5	65.1	38.2	58.4	35.0	53.5	33.1	50.6	33.2	50.8	30.2	46.2
20	36.6	56.0	32.8	50.2	30.1	46.0	28.4	43.5	28.6	43.8	26.0	39.8
22	31.8	48.7	28.5	43.6	26.1	40.0	24.6	37.7	24.9	38.1	22.6	34.5
24	27.8	42.7	24.9	38.2	22.9	35.1	21.5	33.0	21.8	33.5	19.8	30.3
26									19.3	29.6	17.4	26.7
28												
30												

Properties

	36	32.5	29	26.5	25	22.5
A_g, in.^{2}	10.6	9.54	8.52	7.78	7.30	6.56
r_x, in.	1.48	1.47	1.50	1.51	1.60	1.59
r_y, in.	3.04	3.02	2.51	2.48	1.96	1.95
r_x/r_y	0.487	0.487	0.598	0.609	0.816	0.815

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

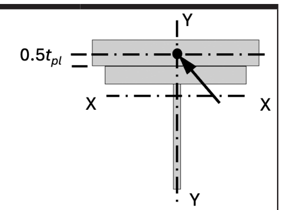
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

$F_y = 50$ ksi

Table 4 (continued)

Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT6×		WT5×									
lb/ft		20[c]		56		50		44		38.5		34	
t_{pl} (in.)		⅜		1		¾		¾		⅝		⅝	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	72.3	108	144	215	137	206	123	184	112	168	101	151
	1	68.4	103	142	213	136	203	121	181	110	165	98.1	147
	2	67.3	101	140	211	134	201	119	179	108	163	96.5	145
	3	66.4	100	138	207	131	197	117	176	106	160	94.7	142
	4	65.3	98.2	134	201	127	191	113	170	103	155	91.7	138
	5	63.9	96.2	128	193	122	183	108	163	97.9	148	87.2	131
	6	62.3	93.9	123	185	116	174	103	155	92.4	140	82.2	124
	7	60.1	90.7	116	175	109	165	96.6	146	86.6	131	76.9	116
	8	56.8	85.8	109	166	102	155	90.3	137	80.6	122	71.3	108
	9	53.4	80.8	102	155	95.2	144	83.9	127	74.6	113	65.8	100
	10	49.9	75.8	95.5	145	88.2	134	77.5	118	68.5	104	60.3	91.7
	11	46.6	70.7	88.5	135	81.2	124	71.2	108	62.6	95.4	54.9	83.7
	12	43.2	65.8	81.6	124	74.5	113	65.1	99.2	56.9	86.7	49.7	75.8
	13	40.0	61.0	74.9	114	67.9	104	59.2	90.3	51.6	78.7	45.0	68.7
	14	36.9	56.3	68.7	105	62.1	94.8	54.1	82.5	46.9	71.7	40.9	62.5
	15	33.9	51.8	63.2	96.4	56.9	86.9	49.5	75.6	42.9	65.5	37.4	57.1
	16	31.1	47.6	58.3	89.0	52.5	80.2	46.0	70.4	39.6	60.5	34.7	53.1
	18	26.5	40.5	49.9	76.3	44.7	68.4	39.1	59.9	33.5	51.3	29.3	44.9
	20	22.7	34.8	43.2	66.1	38.5	58.9	33.6	51.5	28.7	44.0	25.1	38.4
	22	19.7	30.2	37.7	57.7								
	24	17.3	26.4										
	26	15.2	23.3										
	28												
	30												
Properties													
A_g, in.^{2}		5.84		16.5		14.7		13.0		11.3		10.0	
r_x, in.		1.57		1.32		1.29		1.27		1.24		1.22	
r_y, in.		1.94		2.67		2.65		2.63		2.60		2.58	
r_x/r_y		0.809		0.494		0.487		0.483		0.477		0.473	

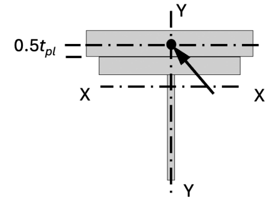
ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

$F_y = 50$ ksi

Table 4 (continued)
Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT5×											
lb/ft		30		27		24.5		22.5		19.5		16.5	
t_{pl} (in.)		½		½		⅜		½		⅜		⅜	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	93.6	140	84.5	127	81.1	122	73.0	109	66.9	100	58.8	88.2
	1	90.5	136	81.1	122	77.2	116	70.6	106	63.9	96.0	55.0	82.6
	2	88.9	134	79.5	119	75.5	113	69.5	104	62.8	94.4	54.0	81.0
	3	87.2	131	77.9	117	73.9	111	68.2	103	61.7	92.7	53.0	79.6
	4	84.9	128	75.9	114	72.0	108	66.3	100	60.1	90.5	51.7	77.8
	5	80.5	121	72.2	109	68.8	104	63.0	95.0	57.5	86.7	50.1	75.6
	6	75.7	114	67.7	102	64.3	97.2	59.3	89.5	54.0	81.5	47.5	71.7
	7	70.5	107	62.9	95.3	59.6	90.3	55.3	83.7	50.2	76.1	44.2	66.9
	8	65.2	99.0	58.0	88.1	54.8	83.2	51.2	77.7	46.4	70.4	40.8	61.9
	9	60.0	91.1	53.2	80.8	50.0	76.1	47.1	71.5	42.5	64.6	37.4	56.9
	10	54.7	83.3	48.4	73.7	45.3	69.1	43.0	65.5	38.8	59.0	34.1	52.0
	11	49.7	75.7	43.8	66.8	40.9	62.4	39.1	59.7	35.2	53.6	31.0	47.2
	12	44.8	68.4	39.4	60.1	36.6	56.0	35.4	54.0	31.7	48.4	27.9	42.7
	13	40.5	61.8	35.5	54.3	33.0	50.4	32.0	48.8	28.6	43.6	25.1	38.4
	14	36.7	56.1	32.2	49.2	29.8	45.6	29.0	44.3	25.8	39.5	22.7	34.8
	15	33.4	51.1	29.3	44.8	27.1	41.4	26.4	40.3	23.5	35.9	20.6	31.6
	16	30.5	46.7	26.7	40.9	24.7	37.8	24.1	36.9	21.4	32.8	18.8	28.8
	17	28.0	42.9	24.5	37.5	22.6	34.6	22.1	33.8	19.6	30.0	17.2	26.4
	18	25.8	39.5	22.5	34.5	20.7	31.8	20.3	31.1	18.0	27.6	15.8	24.2
	19	23.8	36.4	20.8	31.8	19.1	29.3	18.7	28.7	16.6	25.4	14.6	22.3
	20	22.0	33.7					17.3	26.6	15.3	23.5	13.4	20.6
	21											12.5	19.1
	22												
	23												
Properties													
A_g, in.^{2}		8.84		7.90		7.21		6.63		5.73		4.85	
r_x, in.		1.21		1.19		1.18		1.24		1.24		1.26	
r_y, in.		2.57		2.56		2.54		2.01		1.98		1.94	
r_x/r_y		0.471		0.465		0.465		0.617		0.626		0.649	

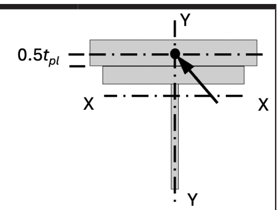
ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

$F_y = 50$ ksi

Table 4 (continued)

Available Strength
Axial Compression, kips
Eccentrically Loaded WT-Shapes



Shape		WT4×											
lb/ft		33.5		29		24		20		17.5		15.5[f]	
t_{pl} (in.)		⅝		⅝		½		⅜		⅜		⅜	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$L_{cx} = L_{cy} = L_{cz} = L_b$, ft	0	91.2	137	81.2	122	69.5	104	63.3	95.0	56.1	84.2	50.8	76.2
	1	89.8	135	79.6	119	67.8	102	61.0	91.6	53.6	80.4	47.9	71.9
	2	88.4	133	78.3	118	66.5	100	59.7	89.7	52.3	78.6	46.7	70.2
	3	85.3	128	75.6	114	64.2	96.6	58.1	87.4	50.8	76.5	45.4	68.3
	4	81.1	122	71.7	108	60.5	91.2	54.7	82.5	48.1	72.6	43.5	65.6
	5	76.2	115	67.2	101	56.3	85.0	50.7	76.6	44.4	67.1	40.1	60.6
	6	70.8	107	62.2	94.1	51.7	78.3	46.3	70.2	40.5	61.3	36.5	55.3
	7	65.2	98.8	57.0	86.5	47.0	71.3	41.9	63.6	36.5	55.4	32.8	49.9
	8	59.5	90.4	51.8	78.7	42.3	64.3	37.5	57.1	32.5	49.5	29.3	44.5
	9	53.8	81.9	46.7	71.1	37.7	57.5	33.3	50.8	28.7	43.8	25.8	39.4
	10	48.3	73.7	41.7	63.6	33.4	50.9	29.3	44.8	25.2	38.5	22.6	34.6
	11	43.2	65.9	37.2	56.7	29.6	45.2	25.9	39.6	22.2	34.0	19.9	30.5
	12	38.8	59.3	33.3	50.9	26.4	40.4	23.1	35.3	19.7	30.2	17.7	27.1
	13	35.0	53.5	30.0	45.8	23.7	36.2	20.6	31.5	17.6	27.0	15.8	24.2
	14	31.7	48.5	27.1	41.5	21.4	32.7	18.5	28.4	15.8	24.2	14.2	21.7
	15	28.8	44.1	24.6	37.7	19.3	29.6	16.7	25.6	14.3	21.9	12.8	19.6
	16	26.3	40.2	22.4	34.3	17.6	26.9	15.2	23.3	12.9	19.8	11.6	17.7
	17	24.1	36.9	20.5	31.4								
	18												
	19												
	20												
	21												
	22												
	23												
Properties													
A_g, in.^{2}		9.84		8.54		7.05		5.87		5.14		4.56	
r_x, in.		1.05		1.03		0.986		0.988		0.968		0.969	
r_y, in.		2.12		2.10		2.08		2.04		2.03		2.02	
r_x/r_y		0.495		0.490		0.474		0.484		0.477		0.480	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_c = 1.67$	$\phi_c = 0.90$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Design Example 3 Using Table 4

Given:

Member of Design Example 1; WT7×45, pin-connected and braced at the ends using 14½ in. × ½ in. gusset plates, LRFD method, $L_{cx} = L_{cy} = L_{cz} = L_b = L = 12$ ft. Determine the available eccentric axial compressive strength using Table 4.

Solution:

From Table 4; for a WT7×45, at $L_{cx} = 12$ ft, $\phi_c P_{n\,ecc} = 149$ kips. This is the same value calculated in Example 1 earlier.

Design Example 4 Using Table 4

Given:

Member of Design Example 2; WT7×30.5, pin-connected and braced at the ends using 10 in. × ½ in. gusset plates, ASD method, $L_{cx} = L_{cy} = L_{cz} = L_b = L = 10$ ft. Determine the available eccentric axial compressive strength using Table 4.

Solution:

From Table 4; for a WT7×30.5, at $L_{cx} = 10$ ft, $(P_{n\,ecc})/\Omega_c = 78.5$ kips. This is the same value calculated in Example 2 earlier.

USING THICKER GUSSET PLATES THAN RECOMMENDED

The gusset plate thickness recommended in this paper for eccentrically loaded WT compression members are about $t_{pl} = 0.6t_f$, rounded up to the next practical and readily available plate thickness. Recommended plate thicknesses are listed at the top of each column in Table 4.

Plate thicknesses recommended in Table 4 may seem low. For instance, a recommended plate thickness of 2.50 in. for connecting to a WT7×250 with a flange thickness of $t_f =$ 3.50 in. may appear inadequate. The designer may feel it is conservative to use a thicker gusset plate for the connection than recommended. While this may be true for the gusset plate strength, a thicker plate increases load eccentricity (see Eq. 1), resulting in a decrease in the WT compression member's available strength.

It is noted that plate thicknesses recommended are based on plate compressive strength assuming that plate flexural buckling does not apply and $F_n = F_y$. See AISC *Specification* Section J4.4(a) and Equation J4-6.

IMPACT OF α ON THE AVAILABLE STRENGTH BASED ON THE LRFD AND ASD METHODS

Ordinarily, the ratio of the available strength of a member in LRFD and ASD is 1.50, namely,

$$\phi R_n = (1.50)\left(\frac{R_n}{\Omega}\right) \tag{24}$$

This ratio was checked for all values of Table 4. The 1.50 ratio holds for most, but not all cases in Table 4. The values of this ratio for shapes and unbraced length considered in this study ranged from 1.49 to 1.54. This variation is due to the role of α in the B_{1x} equation for LRFD and ASD. Further discussion of this subject is beyond the scope of this paper.

SYMBOLS

B_1 Multiplier to account for P-δ effects, determined for each member, and each direction of bending of the member (B_{1x} and B_{1y}) in accordance with AISC *Specification* Appendix 8, Section 8.1.2

B_2 Multiplier to account for P-Δ effects, determined for each story of the structure and each direction of lateral translation of the story (B_{2x} and B_{2y}) in accordance with AISC *Specification* Appendix 8, Section 8.1.3

C_m Equivalent uniform moment factor, assuming no relative translation of the member ends, for bending in each direction (C_{mx} and C_{my})

L_{c1} Effective length in the plane of bending (L_{c1x} and L_{c1y}), calculated based on the assumption of no lateral translation at the member ends, set equal to the laterally unbraced length of the member unless analysis justifies a smaller value, in. (mm)

M_{ax} Required moment including P-δ effect using ASD load combinations, kip-in. (N-mm)

M_{lt} First-order moment using LRFD or ASD load combinations, due to lateral translation of the structure only, kip-in. (N-mm)

M_{ntx} First-order moment about x-axis using LRFD or ASD load combinations, with the structure restrained against lateral translation, kip-in. (N-mm)

M_{nx} Nominal strength in flexure based on the limit states of yielding, flange local buckling, and lateral torsional buckling, kip-in. (N-mm)

M_r Required second-order flexural strength, kip-in. (N-mm)
$= M_u$ (LRFD)
$= M_a$ (ASD)

M_{ux} Required moment including P-δ effect using LRFD load combinations, kip-in. (N-mm)

P Axial compressive load, kips (N)
$= P_u$ (LRFD)
$= P_a$ (ASD)

P_a Axial compressive force using ASD load combinations, kips (N)

P_{e1} Elastic critical buckling strength of the member in the plane of bending (P_{e1x} and P_{e1y}), calculated based on the assumption of no lateral translation at the member ends, kips (N)

$$= \frac{\pi^2 EI}{(L_{c1})^2} \qquad \text{(AISC } \textit{Spec}\text{. Eq. A-85)}$$

P_{lt} First-order axial force due to lateral translation of the structure only using LRFD or ASD load combinations, kips (N)

P_n Nominal concentric compressive strength based on the limit states of flexural and flexural-torsional buckling in interaction with local buckling, kips (N)

$P_{n\,ecc}$ Nominal eccentric compressive strength based on the limit states of flexural and flexural-torsional buckling in interaction with local buckling, kips (N)

P_{nt} First-order axial force with the structure restrained against lateral translation using LRFD or ASD load combinations, kips (N)

P_r Required second-order axial strength using LRFD or ASD load combinations, kips (N)
$= P_u$ (LRFD)
$= P_a$ (ASD)

P_u Axial compressive force using LRFD load combinations, kips (N)

e Load eccentricity along the y-axis of the applied load measured from the centroid of the WT section, in. (mm)
$= 0.5t_f + \bar{y}$

t_f Thickness of the flange of the WT-section, in. (mm)

t_{pl} Thickness of the gusset plate, in. (mm)

$\bar{y}$ Distance along the y-axis from the centroid of the WT-section to the outside of the flange, in. (mm)

Ω_b Safety factor for flexure (ASD)
$= 1.67$

Ω_c Safety factor for compression (ASD)
$= 1.67$

α $= 1.0$ (LRFD)
$= 1.6$ (ASD)

$\phi_c P_{n\,ecc}$ Available strength in axial compression for an eccentrically loaded WT member using LRFD load combinations, kips (N)

$\dfrac{P_{n\,ecc}}{\Omega_c}$ Available strength in axial compression for an eccentrically loaded WT member using ASD load combinations, kips (N)

ϕ_b Resistance factor for flexure (LRFD)
$= 0.90$

ϕ_c Resistance factor for compression (LRFD)
$= 0.90$

REFERENCES

AISC (2005a), *Specification for Structural Steel Buildings*, ANSI/AISC 360-05, American Institute of Steel Construction, Chicago, Ill.

AISC (2005b), *Steel Construction Manual*, 13th Ed., American Institute of Steel Construction, Chicago, Ill.

AISC (2022), *Specification for Structural Steel Buildings*, ANSI/AISC 360-22, American Institute of Steel Construction, Chicago, Ill.

AISC (2023a), *Steel Construction Manual*, 16th Ed., American Institute of Steel Construction, Chicago, Ill.

AISC (2023b), *AISC Shapes Database*, Version 16.0, American Institute of Steel Construction, Chicago, Ill., https://bit.ly/3AS6RqW.

ASTM (2020), *Standard Specification for High-Strength Low-Alloy Columbuam-Vanadium Structural Steel*, ASTM A992/992M, ASTM International, West Conshohocken, Pa.

ASTM (2021), *Standard Specification for Structural Steel Shapes*, ASTM A572/572M, ASTM International, West Conshohocken, Pa.

Gordon, M.E. (2010), “Tables for Eccentrically Loaded WT Shapes in Compression,” *Engineering Journal*, AISC, Vol. 47, No. 2, pp. 91–100, https://doi.org/10.62913/engj.v47i2.979.