

Chapter F

Design of Members for Flexure

INTRODUCTION

This *Specification* chapter contains provisions for calculating the flexural strength of members subject to simple bending about one principal axis. Included are specific provisions for I-shaped members, channels, HSS, tees, double angles, single angles, rectangular bars, rounds and unsymmetrical shapes. Also included is a section with proportioning requirements for beams and girders.

There are selection tables in the *AISC Manual* for standard beams in the commonly available yield strengths. The section property tables for most cross sections provide information that can be used to conveniently identify noncompact and slender element sections. LRFD and ASD information is presented side-by-side.

Most of the formulas from this chapter are illustrated by the following examples. The design and selection techniques illustrated in the examples for both LRFD and ASD will result in similar designs.

F1. GENERAL PROVISIONS

Selection and evaluation of all members is based on deflection requirements and strength, which is determined as the design flexural strength, $\phi_b M_n$, or the allowable flexural strength, M_n/Ω_b , where

M_n = the lowest nominal flexural strength based on the limit states of yielding, lateral torsional-buckling, and local buckling, where applicable

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

This design approach is followed in all examples.

The term L_b is used throughout this chapter to describe the length between points which are either braced against lateral displacement of the compression flange or braced against twist of the cross section. Requirements for bracing systems and the required strength and stiffness at brace points are given in *AISC Specification Appendix 6*.

The use of C_b is illustrated in several of the following examples. *AISC Manual* Table 3-1 provides tabulated C_b values for some common situations.

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

AISC Specification Section F2 applies to the design of compact beams and channels. As indicated in the User Note in Section F2 of the *AISC Specification*, the vast majority of rolled I-shaped beams and channels fall into this category. The curve presented as a solid line in Figure F-1 is a generic plot of the nominal flexural strength, M_n , as a function of the unbraced length, L_b . The horizontal segment of the curve at the far left, between $L_b = 0$ ft and L_p , is the range where the strength is limited by flexural yielding. In this region, the nominal strength is taken as the full plastic moment strength of the section as given by *AISC Specification* Equation F2-1. In the range of the curve at the far right, starting at L_r , the strength is limited by elastic buckling. The strength in this region is given by *AISC Specification* Equation F2-3. Between these regions, within the linear region of the curve between $M_n = M_p$ at L_p on the left, and $M_n = 0.7M_y = 0.7F_y S_x$ at L_r on the right, the strength is limited by inelastic buckling. The strength in this region is provided in *AISC Specification* Equation F2-2.

The curve plotted as a heavy solid line represents the case where $C_b = 1.0$, while the heavy dashed line represents the case where C_b exceeds 1.0. The nominal strengths calculated in both AISC *Specification* Equations F2-2 and F2-3 are linearly proportional to C_b , but are limited to M_p as shown in the figure.

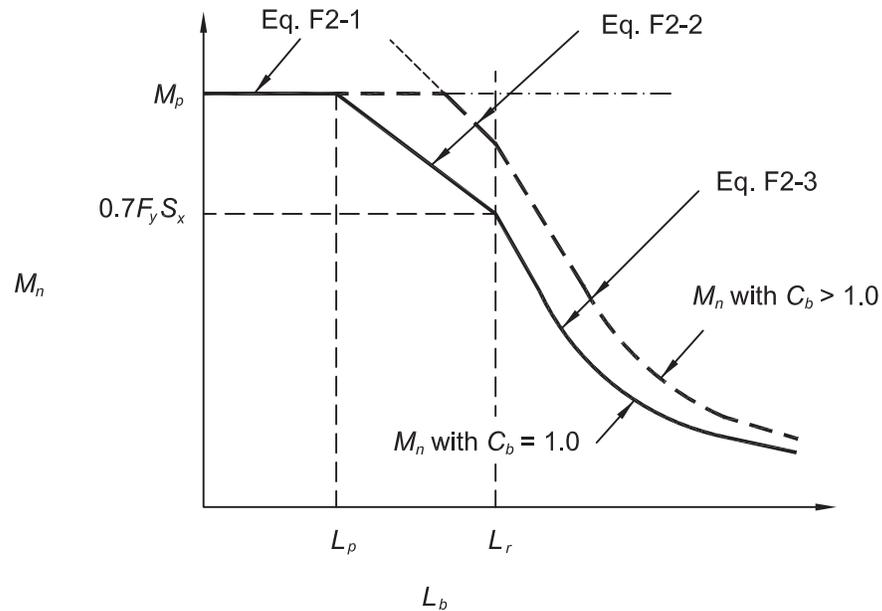


Fig. F-1. Beam strength versus unbraced length.

$$M_n = M_p = F_y Z_x \quad (\text{Spec. Eq. F2-1})$$

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad (\text{Spec. Eq. F2-2})$$

$$M_n = F_{cr} S_x \leq M_p \quad (\text{Spec. Eq. F2-3})$$

where

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}} \right)^2} \quad (\text{Spec. Eq. F2-4})$$

The provisions of this section are illustrated in Example F.1(W-shape beam) and Example F.2 (channel).

Plastic design provisions are given in AISC *Specification* Appendix 1. L_{pd} , the maximum unbraced length for prismatic member segments containing plastic hinges is less than L_p .

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

The strength of shapes designed according to this section is limited by local buckling of the compression flange. Only a few standard wide flange shapes have noncompact flanges. For these sections, the strength reduction for $F_y = 50$ ksi steel varies. The approximate percentages of M_p about the strong axis that can be developed by noncompact members when braced such that $L_b \leq L_p$ are shown as follows:

W21×48 = 99%	W14×99 = 99%	W14×90 = 97%	W12×65 = 98%
W10×12 = 99%	W8×31 = 99%	W8×10 = 99%	W6×15 = 94%
W6×8.5 = 97%			

The strength curve for the flange local buckling limit state, shown in Figure F-2, is similar in nature to that of the lateral-torsional buckling curve. The horizontal axis parameter is $\lambda = b_f/2t_f$. The flat portion of the curve to the left of λ_{pf} is the plastic yielding strength, M_p . The curved portion to the right of λ_{rf} is the strength limited by elastic buckling of the flange. The linear transition between these two regions is the strength limited by inelastic flange buckling.

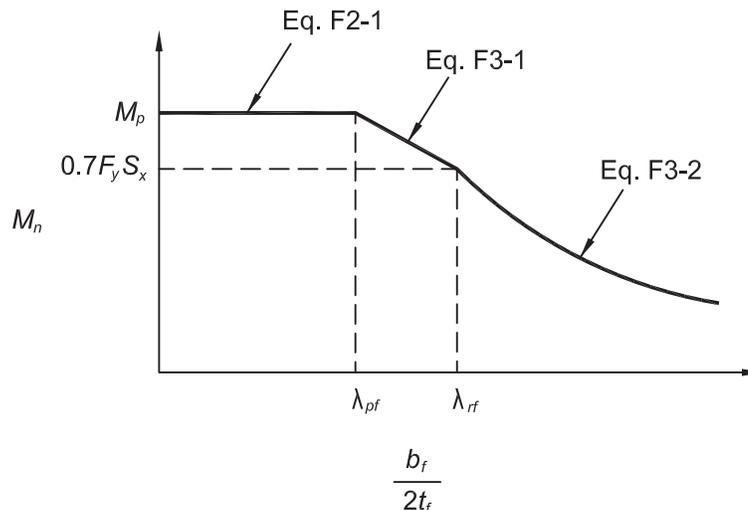


Fig. F-2. Flange local buckling strength.

$$M_n = M_p = F_y Z_x \quad (\text{Spec. Eq. F2-1})$$

$$M_n = \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \quad (\text{Spec. Eq. F3-1})$$

$$M_n = \frac{0.9Ek_c S_x}{\lambda^2} \quad (\text{Spec. Eq. F3-2})$$

where

$$k_c = \frac{4}{\sqrt{h/t_w}} \text{ from AISC Specification Table B4.1b footnote [a], where } 0.35 \leq k_c \leq 0.76$$

The strength reductions due to flange local buckling of the few standard rolled shapes with noncompact flanges are incorporated into the design tables in Chapter 3 of the AISC Manual.

There are no standard I-shaped members with slender flanges. The noncompact flange provisions of this section are illustrated in Example F.3.

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

This section of the AISC *Specification* applies to doubly symmetric I-shaped members with noncompact webs and singly symmetric I-shaped members (those having different flanges) with compact or noncompact webs.

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to I-shaped members with slender webs, formerly designated as “plate girders”.

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

I-shaped members and channels bent about their minor axis are not subject to lateral-torsional buckling. Rolled or built-up shapes with noncompact or slender flanges, as determined by AISC *Specification* Tables B4.1a and B4.1b, must be checked for strength based on the limit state of flange local buckling using Equations F6-2 or F6-3 as applicable.

The vast majority of W, M, C and MC shapes have compact flanges, and can therefore develop the full plastic moment, M_p , about the minor axis. The provisions of this section are illustrated in Example F.5.

F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

Square and rectangular HSS need only be checked for the limit states of yielding and local buckling. Although lateral-torsional buckling is theoretically possible for very long rectangular HSS bent about the strong axis, deflection will control the design as a practical matter.

The design and section property tables in the AISC *Manual* were calculated using a design wall thickness of 93% of the nominal wall thickness. Strength reductions due to local buckling have been accounted for in the AISC *Manual* design tables. The selection of rectangular or square HSS with compact flanges is illustrated in Example F.6. The provisions for rectangular or square HSS with noncompact flanges are illustrated in Example F.7. The provisions for HSS with slender flanges are illustrated in Example F.8. Available flexural strengths of rectangular and square HSS are listed in Tables 3-12 and 3-13, respectively.

F8. ROUND HSS

The definition of HSS encompasses both tube and pipe products. The lateral-torsional buckling limit state does not apply, but round HSS are subject to strength reductions from local buckling. Available strengths of round HSS and Pipes are listed in AISC *Manual* Tables 3-14 and 3-15, respectively. The tabulated properties and available flexural strengths of these shapes in the AISC *Manual* are calculated using a design wall thickness of 93% of the nominal wall thickness. The design of a Pipe is illustrated in Example F.9.

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

The AISC *Specification* provides a check for flange local buckling, which applies only when the flange is in compression due to flexure. This limit state will seldom govern. A check for local buckling of the web was added in the 2010 edition of the *Specification*. Attention should be given to end conditions of tees to avoid inadvertent fixed end moments which induce compression in the web unless this limit state is checked. The design of a WT-shape in bending is illustrated in Example F.10.

F10. SINGLE ANGLES

Section F10 permits the flexural design of single angles using either the principal axes or geometric axes (x - x and y - y axes). When designing single angles without continuous bracing using the geometric axis design provisions, M_y must be multiplied by 0.80 for use in Equations F10-1, F10-2 and F10-3. The design of a single angle in bending is illustrated in Example F.11.

F11. RECTANGULAR BARS AND ROUNDS

There are no design tables in the *AISC Manual* for these shapes. The local buckling limit state does not apply to any bars. With the exception of rectangular bars bent about the strong axis, solid square, rectangular and round bars are not subject to lateral-torsional buckling and are governed by the yielding limit state only. Rectangular bars bent about the strong axis are subject to lateral-torsional buckling and are checked for this limit state with Equations F11-2 and F11-3, as applicable.

These provisions can be used to check plates and webs of tees in connections. A design example of a rectangular bar in bending is illustrated in Example F.12. A design example of a round bar in bending is illustrated in Example F.13.

F12. UNSYMMETRICAL SHAPES

Due to the wide range of possible unsymmetrical cross sections, specific lateral-torsional and local buckling provisions are not provided in this *Specification* section. A general template is provided, but appropriate literature investigation and engineering judgment are required for the application of this section. A Z-shaped section is designed in Example F.14.

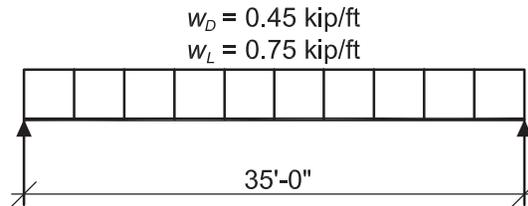
F13. PROPORTIONS OF BEAMS AND GIRDERS

This section of the *Specification* includes a limit state check for tensile rupture due to holes in the tension flange of beams, proportioning limits for I-shaped members, detail requirements for cover plates and connection requirements for built-up beams connected side-to-side. Also included are unbraced length requirements for beams designed using the moment redistribution provisions of AISC *Specification* Section B3.7.

EXAMPLE F.1-1A W-SHAPE FLEXURAL MEMBER DESIGN IN STRONG-AXIS BENDING, CONTINUOUSLY BRACED

Given:

Select an ASTM A992 W-shape beam with a simple span of 35 ft. Limit the member to a maximum nominal depth of 18 in. Limit the live load deflection to $L/360$. The nominal loads are a uniform dead load of 0.45 kip/ft and a uniform live load of 0.75 kip/ft. Assume the beam is continuously braced.



*Beam Loading & Bracing Diagram
(full lateral support)*

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992
 $F_y = 50 \text{ ksi}$
 $F_u = 65 \text{ ksi}$

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.45 \text{ kip/ft}) + 1.6(0.75 \text{ kip/ft})$ $= 1.74 \text{ kip/ft}$	$w_a = 0.45 \text{ kip/ft} + 0.75 \text{ kip/ft}$ $= 1.20 \text{ kip/ft}$
$M_u = \frac{1.74 \text{ kip/ft}(35.0 \text{ ft})^2}{8}$ $= 266 \text{ kip-ft}$	$M_a = \frac{1.20 \text{ kip/ft}(35.0 \text{ ft})^2}{8}$ $= 184 \text{ kip-ft}$

Required Moment of Inertia for Live-Load Deflection Criterion of $L/360$

$$\begin{aligned} \Delta_{max} &= \frac{L}{360} \\ &= \frac{35.0 \text{ ft}(12 \text{ in./ft})}{360} \\ &= 1.17 \text{ in.} \end{aligned}$$

$$\begin{aligned} I_{x(\text{reqd})} &= \frac{5w_L l^4}{384E\Delta_{max}} \text{ from AISC Manual Table 3-23 Case 1} \\ &= \frac{5(0.750 \text{ kip/ft})(35.0 \text{ ft})^4 (12 \text{ in./ft})^3}{384(29,000 \text{ ksi})(1.17 \text{ in.})} \\ &= 746 \text{ in.}^4 \end{aligned}$$

Beam Selection

Select a W18×50 from AISC *Manual* Table 3-2.

Per the User Note in AISC *Specification* Section F2, the section is compact. Because the beam is continuously braced and compact, only the yielding limit state applies.

From AISC *Manual* Table 3-2, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = \phi_b M_{px}$ $= 379 \text{ kip-ft} > 266 \text{ kip-ft}$	$\frac{M_n}{\Omega_b} = \frac{M_{px}}{\Omega_b}$ $= 252 \text{ kip-ft} > 184 \text{ kip-ft}$
o.k.	o.k.

From AISC *Manual* Table 3-2, $I_x = 800 \text{ in.}^4 > 746 \text{ in.}^4$ **o.k.**

EXAMPLE F.1-1B W-SHAPE FLEXURAL MEMBER DESIGN IN STRONG-AXIS BENDING, CONTINUOUSLY BRACED

Given:

Verify the available flexural strength of the W18×50, ASTM A992 beam selected in Example F.1-1A by applying the requirements of the AISC *Specification* directly.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

$$\begin{aligned} & \text{W18} \times 50 \\ & \text{ASTM A992} \\ & F_y = 50 \text{ ksi} \\ & F_u = 65 \text{ ksi} \end{aligned}$$

From AISC *Manual* Table 1-1, the geometric properties are as follows:

$$\begin{aligned} & \text{W18} \times 50 \\ & Z_x = 101 \text{ in.}^3 \end{aligned}$$

The required flexural strength from Example F.1-1A is:

LRFD	ASD
$M_u = 266 \text{ kip-ft}$	$M_a = 184 \text{ kip-ft}$

Nominal Flexural Strength, M_n

Per the User Note in AISC *Specification* Section F2, the section is compact. Because the beam is continuously braced and compact, only the yielding limit state applies.

$$\begin{aligned} M_n &= M_p \\ &= F_y Z_x \\ &= 50 \text{ ksi}(101 \text{ in.}^3) \\ &= 5,050 \text{ kip-in. or } 421 \text{ kip-ft} \end{aligned} \quad (\text{Spec. Eq. F2-1})$$

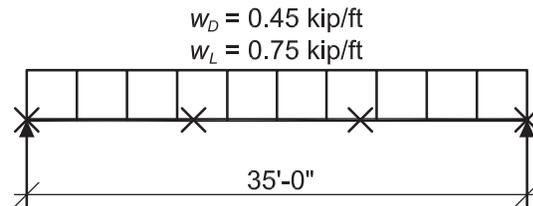
From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(421 \text{ kip-ft})$	$\frac{M_n}{\Omega_b} = \frac{421 \text{ kip-ft}}{1.67}$
$= 379 \text{ kip-ft} > 266 \text{ kip-ft}$	$= 252 \text{ kip-ft} > 184 \text{ kip-ft}$
o.k.	o.k.

EXAMPLE F.1-2A W-SHAPE FLEXURAL MEMBER DESIGN IN STRONG-AXIS BENDING, BRACED AT THIRD POINTS

Given:

Verify the available flexural strength of the W18×50, ASTM A992 beam selected in Example F.1-1A with the beam braced at the ends and third points. Use the AISC *Manual* tables.



*Beam Loading & Bracing Diagram
(bracing at ends and third points)*

Solution:

The required flexural strength at midspan from Example F.1-1A is:

LRFD	ASD
$M_u = 266$ kip-ft	$M_a = 184$ kip-ft

Unbraced Length

$$L_b = \frac{35.0 \text{ ft}}{3}$$

$$= 11.7 \text{ ft}$$

By inspection, the middle segment will govern. From AISC *Manual* Table 3-1, for a uniformly loaded beam braced at the ends and third points, $C_b = 1.01$ in the middle segment. Conservatively neglect this small adjustment in this case.

Available Strength

Enter AISC *Manual* Table 3-10 and find the intersection of the curve for the W18×50 with an unbraced length of 11.7 ft. Obtain the available strength from the appropriate vertical scale to the left.

From AISC *Manual* Table 3-10, the available flexural strength is:

LRFD	ASD
$\phi_b M_n \approx 302$ kip-ft > 266 kip-ft	$\frac{M_n}{\Omega_b} \approx 201$ kip-ft > 184 kip-ft
o.k.	o.k.

EXAMPLE F.1-2B W-SHAPE FLEXURAL MEMBER DESIGN IN STRONG-AXIS BENDING, BRACED AT THIRD POINTS

Given:

Verify the available flexural strength of the W18×50, ASTM A992 beam selected in Example F.1-1A with the beam braced at the ends and third points. Apply the requirements of the AISC *Specification* directly.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

$$\begin{aligned} &\text{ASTM A992} \\ &F_y = 50 \text{ ksi} \\ &F_u = 65 \text{ ksi} \end{aligned}$$

From AISC *Manual* Table 1-1, the geometric properties are as follows:

$$\begin{aligned} &\text{W18}\times\text{50} \\ &S_x = 88.9 \text{ in.}^3 \end{aligned}$$

The required flexural strength from Example F.1-1A is:

LRFD	ASD
$M_u = 266 \text{ kip-ft}$	$M_a = 184 \text{ kip-ft}$

Nominal Flexural Strength, M_n

Calculate C_b .

For the lateral-torsional buckling limit state, the nonuniform moment modification factor can be calculated using AISC *Specification* Equation F1-1.

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \quad (\text{Spec. Eq. F1-1})$$

For the center segment of the beam, the required moments for AISC *Specification* Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: $M_{max} = 1.00$, $M_A = 0.972$, $M_B = 1.00$, and $M_C = 0.972$.

$$\begin{aligned} C_b &= \frac{12.5(1.00)}{2.5(1.00) + 3(0.972) + 4(1.00) + 3(0.972)} \\ &= 1.01 \end{aligned}$$

For the end-span beam segments, the required moments for AISC *Specification* Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: $M_{max} = 0.889$, $M_A = 0.306$, $M_B = 0.556$, and $M_C = 0.750$.

$$\begin{aligned} C_b &= \frac{12.5(0.889)}{2.5(0.889) + 3(0.306) + 4(0.556) + 3(0.750)} \\ &= 1.46 \end{aligned}$$

Thus, the center span, with the higher required strength and lower C_b , will govern.

From AISC *Manual* Table 3-2:

$$L_p = 5.83 \text{ ft}$$

$$L_r = 16.9 \text{ ft}$$

For a compact beam with an unbraced length of $L_p < L_b \leq L_r$, the lesser of either the flexural yielding limit state or the inelastic lateral-torsional buckling limit state controls the nominal strength.

$$M_p = 5,050 \text{ kip-in. (from Example F.1-1B)}$$

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad (\text{Spec. Eq. F2-2})$$

$$= 1.01 \left\{ 5,050 \text{ kip-in.} - \left[5,050 \text{ kip-in.} - 0.7(50 \text{ ksi})(88.9 \text{ in.}^3) \right] \left(\frac{11.7 \text{ ft} - 5.83 \text{ ft}}{16.9 \text{ ft} - 5.83 \text{ ft}} \right) \right\} \leq 5,050 \text{ kip-in.}$$

$$= 4,060 \text{ kip-in. or } 339 \text{ kip-ft}$$

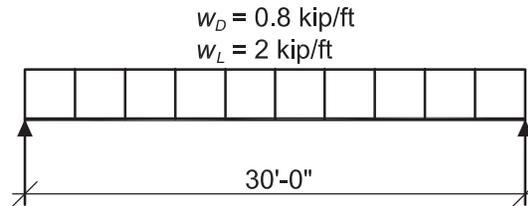
From AISC *Specification* Section F1, the available flexural strength is:

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(339 \text{ kip-ft})$ $= 305 \text{ kip-ft} > 266 \text{ kip-ft}$	$\Omega_b = 1.67$ $\frac{M_n}{\Omega_b} = \frac{339 \text{ kip-ft}}{1.67}$ $= 203 \text{ kip-ft} > 184 \text{ kip-ft}$
o.k.	o.k.

EXAMPLE F.4 W-SHAPE FLEXURAL MEMBER, SELECTION BY MOMENT OF INERTIA FOR STRONG-AXIS BENDING

Given:

Select an ASTM A992 W-shape flexural member by the moment of inertia, to limit the live load deflection to 1 in. The span length is 30 ft. The loads are a uniform dead load of 0.80 kip/ft and a uniform live load of 2 kip/ft. The beam is continuously braced.



Beam Loading & Bracing Diagram
(full lateral support)

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

$$\begin{aligned} &\text{ASTM A992} \\ &F_y = 50 \text{ ksi} \\ &F_u = 65 \text{ ksi} \end{aligned}$$

From Chapter 2 of ASCE/SEI 7, the required flexural strength is:

LRFD	ASD
$w_u = 1.2(0.800 \text{ kip/ft}) + 1.6(2 \text{ kip/ft})$ $= 4.16 \text{ kip/ft}$	$w_a = 0.80 \text{ kip/ft} + 2 \text{ kip/ft}$ $= 2.80 \text{ kip/ft}$
$M_u = \frac{4.16 \text{ kip/ft}(30.0 \text{ ft})^2}{8}$ $= 468 \text{ kip-ft}$	$M_a = \frac{2.80 \text{ kip/ft}(30.0 \text{ ft})^2}{8}$ $= 315 \text{ kip-ft}$

Minimum Required Moment of Inertia

The maximum live load deflection, Δ_{max} , occurs at midspan and is calculated as:

$$\Delta_{max} = \frac{5w_L l^4}{384EI} \text{ from AISC Manual Table 3-23 case 1}$$

Rearranging and substituting $\Delta_{max} = 1.00 \text{ in.}$,

$$\begin{aligned} I_{min} &= \frac{5(2 \text{ kips/ft})(30.0 \text{ ft})^4 (12 \text{ in./ft})^3}{384(29,000 \text{ ksi})(1.00 \text{ in.})} \\ &= 1,260 \text{ in.}^4 \end{aligned}$$

Beam Selection

Select the lightest section with the required moment of inertia from the bold entries in AISC *Manual* Table 3-3.

Try a W24×55.

$$I_x = 1,350 \text{ in.}^4 > 1,260 \text{ in.}^4 \quad \mathbf{o.k.}$$

Because the W24×55 is continuously braced and compact, its strength is governed by the yielding limit state and AISC *Specification* Section F2.1

From AISC *Manual* Table 3-2, the available flexural strength is:

LRFD	ASD
$\phi_b M_n = \phi_b M_{px}$ $= 503 \text{ kip-ft}$	$\frac{M_n}{\Omega_b} = \frac{M_{px}}{\Omega_b}$ $= 334 \text{ kip-ft}$
$503 \text{ kip-ft} > 468 \text{ kip-ft}$	$334 \text{ kip-ft} > 315 \text{ kip-ft}$
o.k.	o.k.

Chapter G

Design of Members for Shear

INTRODUCTION

This chapter covers webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles, HSS sections, and shear in the weak direction of singly or doubly symmetric shapes.

Most of the equations from this chapter are illustrated by example. Tables for all standard ASTM A992 W-shapes and ASTM A36 channels are included in the *AISC Manual*. In the tables, where applicable, LRFD and ASD shear information is presented side-by-side for quick selection, design and verification.

LRFD and ASD will produce identical designs for the case where the live load effect is approximately three times the dead load effect.

G1. GENERAL PROVISIONS

The design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n/Ω_v , are determined as follows:

$$V_n = \text{nominal shear strength based on shear yielding or shear buckling}$$

$$V_n = 0.6F_y A_w C_v \quad (\text{Spec. Eq. G2-1})$$

$$\phi_v = 0.90 \text{ (LRFD)} \quad \Omega_v = 1.67 \text{ (ASD)}$$

Exception: For all current ASTM A6, W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 for $F_y = 50$ ksi:

$$\phi_v = 1.00 \text{ (LRFD)} \quad \Omega_v = 1.50 \text{ (ASD)}$$

AISC Specification Section G2 does not utilize tension field action. *AISC Specification* Section G3 specifically addresses the use of tension field action.

Strong axis shear values are tabulated for W-shapes in *AISC Manual* Tables 3-2 and 3-6, for S-shapes in *AISC Manual* Table 3-7, for C-shapes in *AISC Manual* Table 3-8, and for MC-shapes in *AISC Manual* Table 3-9. Weak axis shear values for W-shapes, S-shapes, C-shapes and MC-shapes, and shear values for angles, rectangular HSS and box members, and round HSS are not tabulated.

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

As indicated in the User Note of this section, virtually all W, S and HP shapes are not subject to shear buckling and are also eligible for the more liberal safety and resistance factors, $\phi_v = 1.00$ (LRFD) and $\Omega_v = 1.50$ (ASD). This is presented in Example G.1 for a W-shape. A channel shear strength design is presented in Example G.2.

G3. TENSION FIELD ACTION

A built-up girder with a thin web and transverse stiffeners is presented in Example G.8.

G4. SINGLE ANGLES

Rolled angles are typically made from ASTM A36 steel. A single angle example is illustrated in Example G.3.

G5. RECTANGULAR HSS AND BOX-SHAPED MEMBERS

The shear height, h , is taken as the clear distance between the flanges less the inside corner radius on each side. If the corner radii are unknown, h shall be taken as the corresponding outside dimension minus 3 times the thickness. A rectangular HSS example is provided in Example G.4.

G6. ROUND HSS

For all round HSS and pipes of ordinary length listed in the *AISC Manual*, F_{cr} can be taken as $0.6F_y$ in *AISC Specification* Equation G6-1. A round HSS example is illustrated in Example G.5.

G7. WEAK AXIS SHEAR IN DOUBLY SYMMETRIC AND SINGLY SYMMETRIC SHAPES

For examples of weak axis shear, see Example G.6 and Example G.7.

G8. BEAMS AND GIRDERS WITH WEB OPENINGS

For a beam and girder with web openings example, see *AISC Design Guide 2, Steel and Composite Beams with Web Openings* (Darwin, 1990).

EXAMPLE G.1A W-SHAPE IN STRONG AXIS SHEAR**Given:**

Determine the available shear strength and adequacy of a W24×62 ASTM A992 beam using the AISC *Manual* with end shears of 48 kips from dead load and 145 kips from live load.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992

$F_y = 50$ ksi

$F_u = 65$ ksi

From Chapter 2 of ASCE/SEI 7, the required shear strength is:

LRFD	ASD
$V_u = 1.2(48.0 \text{ kips}) + 1.6(145 \text{ kips})$ $= 290 \text{ kips}$	$V_a = 48.0 \text{ kips} + 145 \text{ kips}$ $= 193 \text{ kips}$

From AISC *Manual* Table 3-2, the available shear strength is:

LRFD	ASD
$\phi_v V_n = 306 \text{ kips}$ $306 \text{ kips} > 290 \text{ kips}$	$\frac{V_n}{\Omega_v} = 204 \text{ kips}$ $204 \text{ kips} > 193 \text{ kips}$
o.k.	o.k.