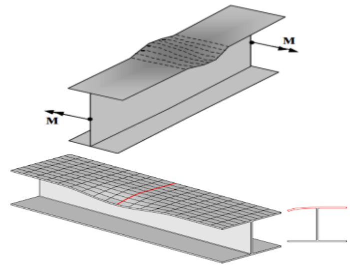
CLASSIFICATION OF SHAPES



AISC classifies cross-sectional shapes as compact, non-compact, or slender, depending on the values of the width-to-thickness ratios. For I shapes, the ratio for the projecting flange (an unstiffened element) is $\frac{bf}{2tf}$, and the ratio for the web (a stiffened element) is $\frac{h}{tw}$. The classification of shapes is found in Section B4 of the Specification, Member Properties," in Table B4.1b (Table B4.1a is for compression members). It can be summarized as follows

λ = width-to-thickness ratio ($\frac{b}{t}$)
λ p = upper limit for compact category
λr= upper limit for non-compact category
Then:-
if λ≥ λp the shape is compact
if λp< λ ≥ λr the shape is non-compact
if λ> λr the shape is slender

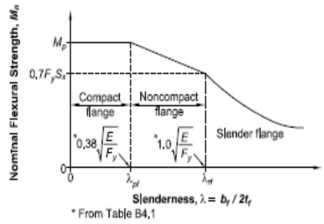


Fig. C-F1.1. Nominal flexural strength as a function of the flange width-thickness ratio of rolled I-shapes.

The category is based on the worst width-to-thickness ratio of the cross section. For example, if the web is compact and the flange is non-compact, the shape is classified as non-compact. Table 1 has been extracted from AISC Table B4.1b foe member subjected to flexural. The web criterion is met by all standard I and C shapes listed in the Manual for Fy ≤ 65 ksi; therefore, in most cases only the flange ratio needs to be checked (note that built-up welded I shapes can have non-compact or slender webs). Most shapes will also satisfy the flange requirement and will therefore be classified as compact. The non-compact shapes are identified in the dimensions and properties table with a footnote (^{f}). Note that compression members have different criteria than flexural members, so a shape could be compact for flexure but slender for compression, shapes with slender compression elements are identified with a footnote such as (W21x48^f).

Table 1 Summary of Table B 4.1 for flexural members

Shape	Element	λ	λ p	λr
I-shaped sections	Flange	$\frac{bf}{2tf}$	$0.38\sqrt{E/fy}$	$1\sqrt{E/fy}$
	Web	$\frac{h}{tw}$	$3.76\sqrt{E/fy}$	$5.7\sqrt{E/fy}$
C-shapes	Flange	$\frac{bf}{tf}$	$0.56\sqrt{E/fy}$	$0.56\sqrt{E/fy}$
	Web	$\frac{h}{tw}$	$3.76\sqrt{E/fy}$	$5.7\sqrt{E/fy}$
WT-shapes	Flange	$\frac{bf}{2tf}$	$0.38\sqrt{E/fy}$	$1\sqrt{E/fy}$
Outstanding legs of single angle	Unstiffened element	$\frac{b}{t}$	$0.54\sqrt{E/fy}$	$0.91\sqrt{E/fy}$
Square or rectangular HSS	Stiffened element	$\frac{h}{t}$	$2.42\sqrt{E/fy}$	$5.7\sqrt{E/fy}$
Round HSS or pipes	Stiffened element	$\frac{D}{t}$	$0.07\frac{E}{Fy}$	$0.31\frac{E}{Fy}$

Example

Use A992 steel material to classify the steel section according to AISC limitation (W18x35, and W21x48[f])

Solution

Steel	fy	fu
A992	50	65

Section	bf/2tf	h/tw
W18x35	7.06	53.5

-Flange

λ=$\frac{bf}{2tf}$=7.06 < 0.38 $\sqrt{\frac{E}{fy}}$

6.28<0.38 $\sqrt{\frac{29000}{50}}$

7.06<9.15 (λ≥ λp) *the flange is compact*

-Web

λ=$\frac{h}{tw}$=53.5 < 3.76 $\sqrt{\frac{E}{fy}}$

53.5<3.76 $\sqrt{\frac{29000}{50}}$

53.5<90.55 (λ≥ λp) *the web is compact*

The section is compact

W18x35: This shape can also be identified as compact because there is no footnote in the dimensions and properties tables.

Section	bf/2tf	h/tw
W21x48	9.47	53.6

- Flange

λ=$\frac{bf}{2tf}$=9.47

λp= 0.38 $\sqrt{\frac{E}{fy}}$ =9.15

λr= 1 $\sqrt{\frac{E}{fy}}$ = 24

λp< λ ≥ λr the shape is non-compact

9.15<9.24 ≥ 24 *the flange is noncompact*

-Web

λ=$\frac{h}{tw}$=51.6 < 3.76 $\sqrt{\frac{E}{fy}}$

5.3.6<3.76 $\sqrt{\frac{29000}{50}}$

53.6< 90.55 (λ≥ λp) *the web is compact*

The section is non-compact

W21x48[f]: This shape can also be identified as non-compact because there is a footnote in the dimensions and properties tables.

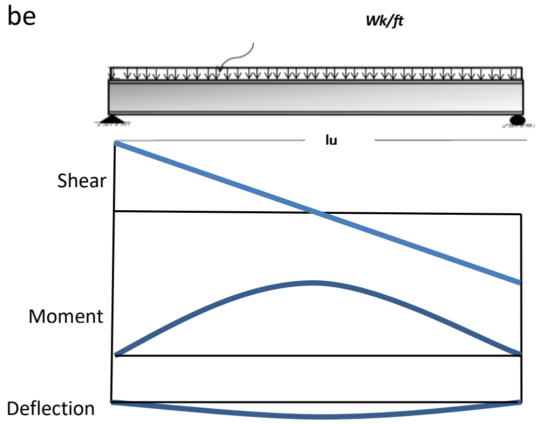
DESIGN AND ANALYSIS OF BEAMS FOR FLEXURAL REQUIREMENTS (CHAPTER F PP.44)

The Basic design and analysis consideration for beams must be checking the AISC limitation in:

1-Bending (flexural) requirements

2-shear requirements

3-deflection requirements



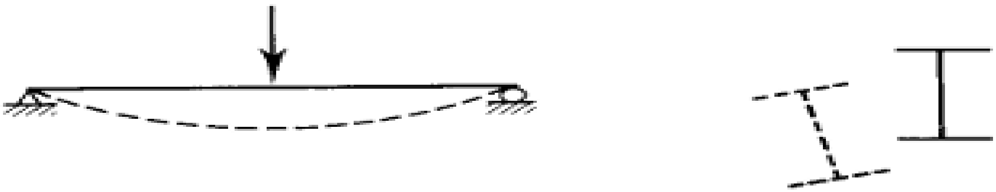
For the flexural requirements, the required and available strength are moment

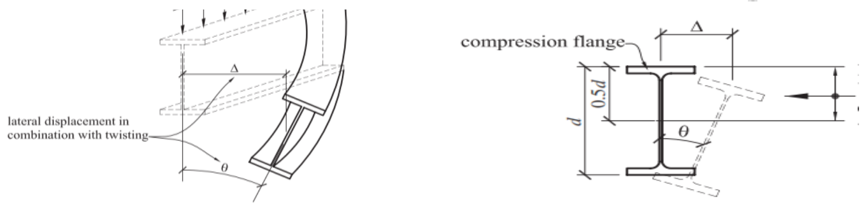
load and resistance factor design (LRFD)	For allowable strength design (ASD)
$Mu=1.2M_{d.l}+1.6M_{l.l}$ $Mu \le \phi_b Mn$ Where:- Mu =required moment strength = maximum moment applied ϕ_b= resistance factor for bending (flexure) =0.9 M_n = nominal moment strength $Mu \le 0.9Mn$	$Ma=M_{d.l}+M_{l.l}$ $Ma \le \frac{Mn}{\Omega b}$ Where:- Ma = required moment strength = maximum moment applied Ω_b = safety factor for bending = 1.67 M_n = nominal moment strength $M_a \le \frac{Mn}{1.67}$

Lateral torsional buckling (LTB), laterally unsupported length (Lb)

The compression flange of a beam behaves like an axially loaded column. Thus, in beams covering long spans the compression flange may tend to buckle. Unlike a column, however, the compression portion of the cross section is restrained by the tension portion, and the outward deflection (flexural buckling) is accompanied by twisting (torsion). This form of instability is called lateral-torsional buckling (LTB).

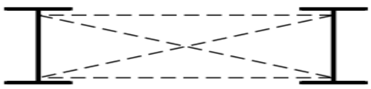
Lateral–torsional buckling occurs when the distance between lateral brace points is large enough that the beam fails by lateral, outward movement in combination with a twisting action (Δ and θ, respectively, in Figure below).

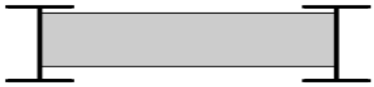




Lateral torsional buckling may be prevented through the following provisions:

1. Increases the Lateral supports at intermediate points in addition to lateral supports at the vertical supports
2. Using torsionally strong sections (for example, box sections) prevents twist directly; it can be either nodal or continuous, and it can take the form of either cross frames or diaphragms


Cross frame


Diaphragm

3. I-sections with relatively wide flanges: Beams with wider flanges are less susceptible to lateral–torsional buckling because the wider flanges provide more resistance to lateral displacement

Lateral-torsional buckling is fundamentally similar to the flexural buckling or flexural-torsional buckling of a column subjected to axial loading. The differences are that lateral torsional buckling is caused by flexural loading (M), and the buckling deformations are coupled in the lateral and torsional directions. Also there is one very important difference. For a column, the axial load causing buckling remains constant along the length. But, for a beam, usually the lateral-torsional buckling causing bending moment M(x) varies along the unbraced length.
If the maximum bending stress is less than the proportional limit when buckling occurs, the failure is said to be elastic. Otherwise, it is inelastic.
For convenience, we first categorize beams as compact, non-compact, or slender, and then determine the moment resistance based on the degree of lateral support.

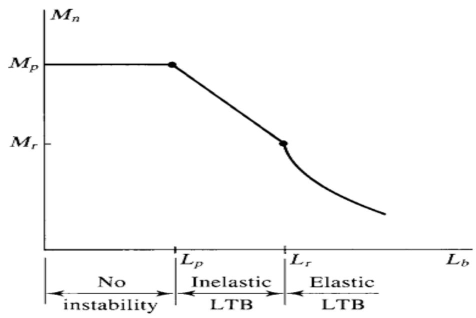
L_b= laterally unsupported length
L_p = Plastic length: maximum unbraced length at which the nominal bending strength equals the plastic moment capacity and at which inelastic lateral–torsional buckling occurs.
L_r = unbraced length at which **elastic** lateral– torsional buckling occurs.

$$L_p = 1.76\, r_y \sqrt{\frac{E}{F_y}}$$

$$L_r = 1.95\, r_{ts} \frac{E}{0.7 F_y} \sqrt{\frac{J\, c}{S_x\, h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7 F_y}{E} \frac{S_x\, h_o}{J\, c}\right)^2}}$$

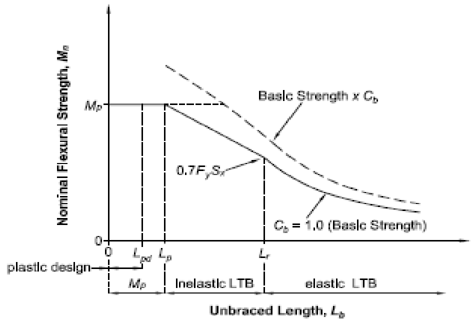
$$r_{ts} = \left(\frac{\sqrt{I_y C_w}}{S_x}\right)^{1/2}$$

$$c = \begin{cases} 1.0 \rightarrow \text{For doubly symmertic I} - \text{Shapes} \\ \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \rightarrow \text{For Channel Shapes} \end{cases}$$



Where:-
Iy = moment of inertia about the weak axis of the cross section (in.^{4})
G = shear modulus of structural steel = 11,200 ksi
h_o=distance between the flange centroids=d-tf
J = torsional constant (in.^{4})
C_w = warping constant (in.^{6})

The nominal bending strength, M_n, is a function of the following:
1. Lateral–torsional buckling (LTB),
2. Flange local buckling (FLB), and
3. Web local buckling (WLB).



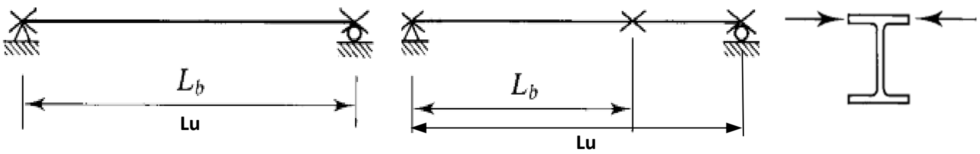
If the beam is compact and has continuous lateral support, or **if the unbraced length is very short, the nominal moment strength, Mn, is the full plastic moment capacity of the shape, Mp. For members with inadequate lateral support, the moment resistance is limited by the lateral-torsional buckling strength, either in elastic or elastic**

Laterally unsupported length (Lb)

1-when Lb≤Lp (Zone 1)

M_n = The moment strength of compact shapes is a function of the unbraced length, L_b, defined as the distance between points of lateral support, or bracing indicated as " × ". The relationship between the nominal strength, M_n, and the unbraced length is shown in Figure. If the unbraced length is no greater than L_p, the beam is considered to have full lateral support, and
$M_p = FyZ$. The first category, laterally supported compact beams, is quite common and is the simplest case. For a doubly-symmetric, compact I- or C-shaped section bent about its major axis, AISC F2.1 gives the nominal strength as

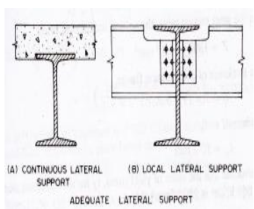
Mn = Mp (AISC Equation F2-1)
Where
$$Mp = FyZ$$

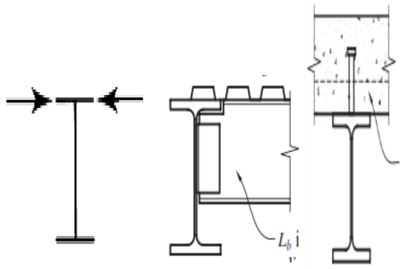


Lb = the full length (Lu) if there is no intermediate laterally supported between the supports

Lb = 0 if the beams has full laterally supported beams

Lb=0





Lb=0

2-when Lp<Lb≤Lr (Zone 2)

If L_b is greater than L_p but less than or equal to L_r, the strength is based on inelastic LTB.

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le \boldsymbol{M_p}$$ (AISC Equation F2-2)

Cb =laterally torsional buckling modification factor for non-uniform moments diagrams when both ends of the un- supported segment are brace

$$Cb = \frac{12.5Mmax}{2.5\,Mmax + 3MA + 4MB + 3MC}\,Rm \le 3$$

Where

M_{max} = absolute value of maximum moment in the unbraced segment, kip-in.

M_A = absolute value of moment at quarter point of the unbraced segment, kip-in.

M_B = absolute value of moment at centerline of the unbraced segment, kip-in.

M_C = absolute value of moment at three-quarter point of the unbraced segment, kip-in.

Rm = cross-section monosymmetry parameter

= 1.0, doubly symmetric members

= 1.0, singly symmetric member's subjected to single curvature bending

= $0.5 + 2(\frac{Iyc}{Iy})^2$

Iy = moment of inertia about the principal y-axis, $in.^4$

Iyc = moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending, referred to the smaller flange, $in.^4$

Cb : is permitted to be conservatively taken as 1.0 for all cases.

For cantilevers or overhangs where the free end is unbraced, Cb = 1.0.

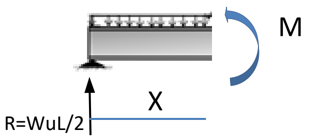
Table 3–1

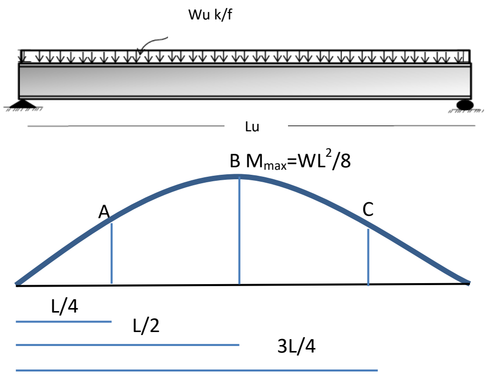
Values for C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
P	None (Load at midpoint)	1.32
	At load point	1.67, 1.67
P P	None (Loads at third points)	1.14
	At load points (Loads symmetrically placed)	1.67, 1.00, 1.67
P P P	None (Loads at quarter points)	1.14
	At load points (Loads at quarter points)	1.67, 1.11, 1.11, 1.67
w	None	1.14
	At midpoint	1.30, 1.30
	At third points	1.45, 1.01, 1.45
	At quarter points	1.52, 1.06, 1.06, 1.52
	At fifth points	1.56, 1.12, 1.00, 1.12, 1.56

Example

Determine the lateral torsional factor for the beam shown below





$$Cb = \frac{12.5Mmax}{2.5\ Mmax + 3MA + 4MB + 3MC}\ Rm \le 3$$

$$Mmax = \frac{Wul^2}{8}$$

$$At\ X = \frac{l}{4}$$

$$MA = \frac{Wul}{2} x \frac{L}{4} - \frac{Wu(\frac{L}{4})^2}{2}$$

$$= \frac{Wul^2}{8} - \frac{Wul^2}{32}$$

$$= \frac{3Wul^2}{32}$$

$$At\ X = \frac{l}{2}$$

$$MB = \frac{Wul}{2} x \frac{L}{2} - \frac{Wu(\frac{L}{2})^2}{2}$$

$$= \frac{Wul^2}{4} - \frac{Wul^2}{8}$$

$$= \frac{Wul^2}{8}$$

$$At\ X = \frac{3l}{4}$$

$$MC = \frac{Wul}{2} x \frac{3L}{4} - \frac{Wu(\frac{3L}{4})^2}{2}$$

$$= \frac{3Wul^2}{8} - \frac{9Wul^2}{32}$$

$$= \frac{3Wul^2}{32}$$

$$Cb = \frac{12.5Mmax}{2.5\ Mmax + 3MA + 4MB + 3MC}\ Rm \le 3$$

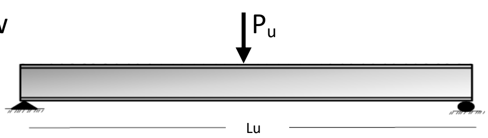
$$Cb = \frac{12.5\frac{Wul^2}{8}}{2.5\ \frac{Wul^2}{8} + 3\frac{3Wul^2}{32} + 4\frac{Wul^2}{8} + 3\frac{3Wul^2}{32}}\ 1 \le 3$$

$$Cb = \frac{\frac{12.5}{8}}{\frac{2.5}{8} + \frac{3x3}{32} + \frac{4}{8} + \frac{3x3}{32}}\ 1 \le 3\ = 1.136 = 1.14$$

$$or\ use\ table\ (3-1)Cb = 1.14$$

Homework

Determine the lateral torsional factor for the beam shown below



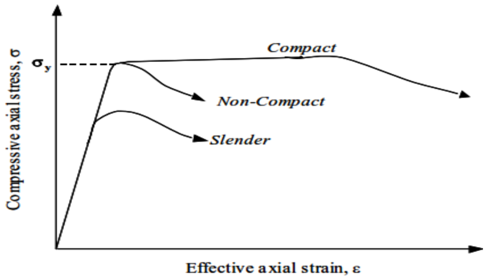
3- **When Lb>Lr(zone 3)**

The AISC Specification gives a different, but equivalent, form for the elastic buckling stress F_{cr}. AISC gives the nominal moment strength as

$$Mn = FcrSx \leq Mp \qquad \text{(AISC Equation F2-3)}$$

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2}\sqrt{1+0.078\frac{Jc}{S_x h_0}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(AISC Equation F2-4)}$$

Where the yield stress has been reduced by 30% to account for the effect of residual stress



Summary of Nominal Flexural Strength

The nominal bending strength for compact I and C-shaped sections can be summarized as follows:

For $L_b \leq L_p$,

$$M_n = M_p \qquad \text{(AISC Equation F2-1)}$$

For $L_p < L_b \leq L_r$,

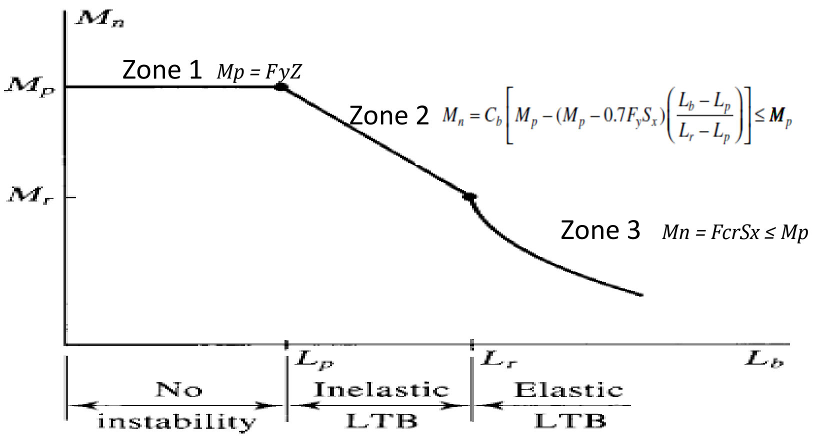
$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \qquad \text{(AISC Equation F2-2)}$$

For $L_b > L_r$,

$$M_n = F_{cr}S_x \leq M_p \qquad \text{(AISC Equation F2-3)}$$

where

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2}\sqrt{1+0.078\frac{Jc}{S_x h_0}\left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(AISC Equation F2-4)}$$



Zone 1: Plastic behavior, full plastic moment

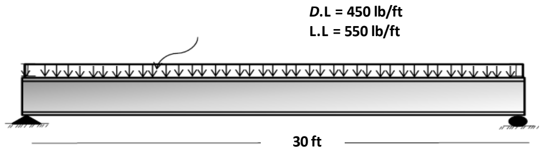
Zone 2: Inelastic lateral torsion buckling

Zone 3: Elastic lateral torsion buckling

Example

The beam shown in Figure below is a W16 × 31 of A992 steel. It supports a reinforced concrete floor slab that provides continuous lateral support of the compression flange. The service dead load is 450 lb/ft. This load is superimposed on the beam; it does not include the weight of the beam itself. The service live load is 550 lb/ft. Does this beam have adequate moment strength?

Solution



Steel	fy	fu	
A992	50	65	
Section	**bf/2tf**	**h/tw**	**Zx**
W16x31	6.28	51.6	54

First, determine the nominal flexural strength.

-Check the local buckling for

-flange

$\lambda = \frac{bf}{2tf} = 6.28 < 0.38\sqrt{\frac{E}{fy}}$

$6.28 < 0.38\sqrt{\frac{29000}{50}}$

$6.28 < 9.15$ ($\lambda \geq \lambda p$) *the flange is compact*

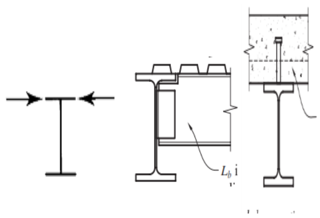
-web

$\lambda = \frac{h}{tw} = 51.6 < 3.76\sqrt{\frac{E}{fy}}$

$5.16 < 3.76\sqrt{\frac{29000}{50}}$

$51.6 < 90.55$ ($\lambda \geq \lambda p$) *the web is compact*

The section is compact



Lb = 0 the beams has full laterally supported

Since the beam is compact and laterally supported, the nominal flexural strength is

$Mn = Mp = FyZx$ = 50(54.0) = 2700 in.-kips = 225.0 ft kips.

Compute the maximum bending moment.

The total service dead load, including the weight of the beam, is

W_D = 450 + 31 = 481 lb/ft=0.481 k/ft

W_l=550 lb/ft =0.55 k/ft

$W_u = 1.2W_d + 1.6W_l = 1.2x0.481 + 1.6x0.55 = 1.456$ k/ft

For a simply supported, uniformly loaded beam, the maximum bending moment occurs at mid span and is equal to where w is the load in units of force per unit length, and L is the span length.

$Mu = \frac{wul^2}{8} = \frac{1.456\,x30^2}{8}$ = 164 ft.kip

Alternatively, the loads can be factored at the outset:

The design strength is

ϕMn = 0.90(225.0) = 203 ft-kips > 164 ft-kips (OK)

The design moment is greater than the factored-load moment,

So the W16 × 31 is satisfactory (adequate)

Homework

Re solve the above example by using ASD method

Example

Use LRFD method to determine the flexural strength of a W14 × 68 of A992 steel material if
a. Continuous lateral support.
b. An unbraced length of 20 ft with Cb = 1.0.
c. An unbraced length of 30 ft with Cb = 1.0.

Solution

Steel	fy	fu
A992	50	65

Section	bf/2tf	h/tw	Sx	Zx	rts	ho	J	cw	ry	Iy
W14x68	6.97	27.5	103	115	2.8	13.3	3.01	5380	2.46	121

Check the section

-Flange

$\lambda=\frac{bf}{2tf}=6.97 < 0.38\sqrt{\frac{E}{fy}}$

$6.97<0.38\sqrt{\frac{29000}{50}}$

6.97<9.15 (λ≥ λp) *the flange is compact*

-Web

$\lambda=\frac{h}{tw}=27.5 < 3.76\sqrt{\frac{E}{fy}}$

$27.5<3.76\sqrt{\frac{29000}{50}}$

27.5<90.55 (λ≥ λp) *the web is compact*

The section is compact Use equ. F_2

a. Continuous lateral support

L_b = 0 the beams has full laterally supported

$L_b < L_p$ ⟶ Zone 1

Since the beam is compact and laterally supported, the nominal flexural strength is
$Mn = Mp = FyZx$ = 50(115) = 5750in.-kips = 479.16 ft kips.
$\phi Mn = \phi\, Mp = \phi\, FyZx = 0.9x\, 479.16 = 431.25\, ft.kip$

b. An unbraced length of 20 ft with Cb = 1.0.
L_b=20ft

$Lp = 1.76ry\sqrt{\frac{E}{fy}}$ = $Lp = 1.762.46\sqrt{\frac{2900}{50}}$ = 104.27 in=$\frac{104.27}{12}$= 8.689 ft

or use table (3.2) Lp=8.69 ft

$$L_r = 1.95\, r_{ts}\, \frac{E}{0.7F_y}\sqrt{\frac{J\,c}{S_x\,h_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\,\frac{S_x\,h_o}{J\,c}\right)^2}}$$

$$r_{ts} = \left(\frac{\sqrt{I_y C_w}}{S_x}\right)^{1/2}$$

rts=2.798 in
rt=2.8
h_o=d-tf=13.28
c=1 I shape

Lr= 351.3 in $=\frac{351.3}{12}$= 29.28 ft
Or use table (3.2) Lr= 29.3 ft

Lp < Lb ≤Lr ⟶ Zone 2
8.69< 20 ≤29.3

$$Mn = Cb[Mp - (Mp - 0.7FySx)[\frac{lb - lp}{lr - lp}] \leq Mp$$

Cb=1,$Mp = FyZX$=50x115=5750 in.kip

$$Mn = 1[570 - (5750 - 0.7x50x103)[\frac{20 - 8.69}{29.3 - 8.69}] \leq 5750$$

=4572< 5750
Use Mn=4572 in.kip
ϕMn=0.9x4572=4114.8 in.kip
=342.9 ft.kip

c. An unbraced length of 30 ft with Cb = 1.0.

lb>Lr ⟶ Zone 3
30> 29.3

$$Mn = FcrxSx < MP$$

$$Fcr = \frac{cb\pi^2 E}{(\frac{lb}{rts})^2}\sqrt{(1 + 0.078\frac{Jc}{Sxho}(\frac{lb}{rts})^2)}$$

$$Fcr = \frac{1x\pi^2\,29000}{(\frac{30x12}{2.8})^2}\sqrt{(1 + 0.078\frac{3.01x1}{103x13.3}(\frac{30x12}{2.8})^2)}$$

= 33.9 ksi

$$Mn = 33.9x103 < 5750$$

= 3492 <5750

$=\frac{3492}{12}$ = 291 ft.kip

ϕMn=0.9x291=261.9 ft. kip

Homework

Re solve the above example by use ASD method

Example

Use ASD method to determine the flexural strength of a W14 × 74 of A992 steel material if
a. Continuous lateral support.
b. An unbraced length of 15 ft.
c. An unbraced length of 35 ft.

Solution

Steel	fy	fu
A992	50	65

Section	bf/2tf	h/tw	S_x	Z_x	r_{ts}	h_o	J	c_w	r_y	I_y
W14x74	6.41	25.4	112	126	2.82	13.4	3.87	5990	2.48	134

Check the section

-Flange

$\lambda=\frac{bf}{2tf}$=6.97 < $0.38\sqrt{\frac{E}{fy}}$

6.41<$0.38\sqrt{\frac{29000}{50}}$

6.41<9.15 (λ≥ λp) *the flange is compact*

-Web

$\lambda=\frac{h}{tw}$=25.4 < $3.76\sqrt{\frac{E}{fy}}$

25.4<$3.76\sqrt{\frac{29000}{50}}$

25.4<90.55 (λ≥ λp) *the web is compact*

The section is compact Use equ. F_2

a. Continuous lateral support

L_b = 0 the beams has full laterally supported

$L_b < L_p$ ⟶ Zone 1

Since the beam is compact and laterally supported, the nominal flexural strength is

$Mn = Mp = FyZx$ = 50(126) = 6300 in.-kips = 525 ft kips.

$$\frac{Mn}{\Omega b}=\frac{Mp}{\Omega b}=\frac{FyZx}{\Omega b}=\frac{525}{\Omega b}=314.4\ ft.kip$$

b. An unbraced length of 15 ft.

L_b=15 ft

Cb=1.3 table 3.1

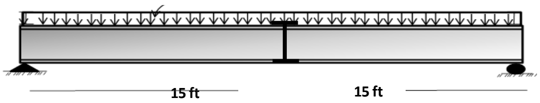
use table (3.2)

Lp=8.76 ft

Lr= 31 ft

Lp < Lb ≤Lr ⟶ Zone 2

8.76< 15 ≤31



$$Mn = Cb[Mp-(Mp-0.7FySx)[\frac{lb-lp}{lr-lp}] \le Mp$$

Cb=1.3,Mp = 6300 in.kip

$$Mn = 1.3x[6300-(6300-0.7x50x112)[\frac{15-8.76}{31-8.76}] \le 6300$$

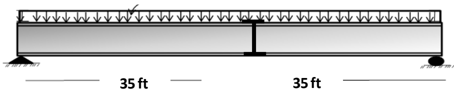
=7321.8> 6300

Use Mn=6300 in.kip

$$\frac{Mn}{\Omega b} = \frac{Mp}{\Omega b} = \frac{6300}{1.67} = \frac{3772.5}{12} = 314.44\ ft.kip$$

c. An unbraced length of 35 ft.

lb>Lr ⟶ Zone 3



$$Mn = FcrxSx < MP$$

$$Fcr = \frac{cb\pi^2 E}{(\frac{lb}{rts})^2}\sqrt{(1 + 0.078\frac{Jc}{Sxho}(\frac{lb}{rts})^2)}$$

$$Fcr = \frac{1.3x\pi^2\ 29000}{(\frac{35x12}{2.82})^2}\ \sqrt{(1 + 0.078\frac{3.87x1}{112x13.4}(\frac{35x12}{2.82})^2)}$$

= 39.2 ksi

$$Mn = 39.2x112 < 6300$$

= 4390.52 <6300

$=\frac{3949.2}{12}$ = 365.87 ft.kip

$$\frac{Mn}{\Omega b} = \frac{Mp}{\Omega b} = \frac{365.87}{1.67} = \ 219\ ft.kip$$

Homework

Re solve the above example by use LRFD method