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Chapter 5 – Noncompact Shapes

➤ For flexural members with compact webs and noncompact or slender flanges, refer to **AISC F3**.

CHAPTER F
DESIGN OF MEMBERS FOR FLEXURE

TABLE USER NOTE F1.1
DESIGN OF MEMBERS FOR FLEXURE

This chapter applies to members subjected to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at load points and supports. The shapes are organized as follows:

F1. General Provisions
F2. Doubly Symmetric Compact I-Shaped Members and Channel Beams About Their Major Axis
F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis
F4. Doubly Symmetric and Slender Symmetric I-Shaped Members with Slender Webs Bent About Their Major Axis
F5. I-Shaped Members and Channel Beams About Their Minor Axis
F6. Square and Rectangular HSS and Box Sections
F7. Round HSS
F8. Tee and Double Angle L-sections in the Plane of Symmetry
F9. Single Angles
F10. Rectangular Beams and Boards
F11. Unsymmetrical Shapes
F12. Proportioning of Beams and Girders

Use Note: For cases not included in this chapter, the following sections apply:
• Chapter G – Design provisions for shear
• F10-F12 – Members subjected to biaxial flexure or to combined flexure and axial force
• B3 – Members subjected to tension and torsion
• Appendix 3 – Members subjected to fatigue

For guidance in determining the appropriate sections of this chapter to apply, Table User Note F1.1 may be used.

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	GFY, LTB, FLB, TFY

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Chapter 5 – Noncompact Shapes

➤ For flexural members with compact webs and noncompact or slender flanges, refer to **AISC F3**.

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS
This section applies to doubly symmetric I-shaped members bent about their major axis having compact webs and noncompact or slender flanges as defined in Section B4.1 for flexure.

User Note: The following shapes have noncompact flanges for $F_y = 50$ ksi (345 MPa): W14-90, W14-60, W14-40, W12-65, W12-52, W10-51, W10-30, W8-15, W8-8, and M4-6. All other ASTM A6/A588 W, S, and M shapes have compact flanges for $F_y \leq 50$ ksi (345 MPa).

The nominal flexural strength, M_n , shall be the lower value obtained according to the limit states of lateral-torsional buckling and compression flange local buckling.

1. **Lateral-Torsional Buckling**
For lateral-torsional buckling, the provisions of Section F2.2 shall apply.
2. **Compression Flange Local Buckling**
 - (a) For sections with noncompact flanges

$$M_n = M_p - (M_p - 0.7F_y S_x) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \quad (F3-1)$$
 - (b) For sections with slender flanges

$$M_n = 0.92 S_x F_y \quad (F3-2)$$

where:
 $\lambda = \frac{L_b}{r_{ts}}$ and shall not be taken as less than 0.35 nor greater than 0.76 for calculation purposes.
 k = distance as defined in Section B4.1b, in. (mm)
 t_w = thickness of the web, in. (mm)

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Chapter 5 – Noncompact Shapes

➤ For **flange local buckling**, if $\lambda_{pf} < \lambda < \lambda_{rf}$, the flange is **noncompact**, and buckling will be inelastic:

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

$$\lambda = \frac{b_f}{2t_f} \quad \lambda_{pf} = 0.38 \sqrt{\frac{E}{F_y}} \quad \lambda_{rf} = 1.0 \sqrt{\frac{E}{F_y}} \quad \text{From Table B4.1b}$$

➤ The webs of all hot-rolled shapes in the Manual are **compact**, so the **noncompact** shapes are subject only to the limit states of **lateral-torsional buckling** and **flange local buckling**.

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Chapter 5 – Noncompact Shapes

➤ For **flange local buckling**, if $\lambda_{pf} < \lambda < \lambda_{rf}$, the flange is **noncompact**, and buckling will be inelastic:

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

$$\lambda = \frac{b_f}{2t_f} \quad \lambda_{pf} = 0.38 \sqrt{\frac{E}{F_y}} \quad \lambda_{rf} = 1.0 \sqrt{\frac{E}{F_y}} \quad \text{From Table B4.1b}$$

➤ Built-up welded shapes, however, can have **noncompact** or **slender webs** as well as **noncompact or slender flanges**.

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Chapter 5 – Noncompact Shapes

➤ These cases are covered in **AISC Sections F4 and F5**.

TABLE USER NOTE F1.1
DESIGN OF MEMBERS FOR FLEXURE

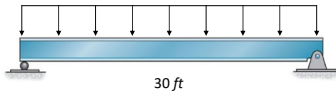
TABLE USER NOTE F1.1
Selection Table for the Application of Chapter F Sections

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	GFY, LTB, FLB, TFY
F5		C, NC, S	S	GFY, LTB, FLB, TFY

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Chapter 5 – Noncompact Shapes

- **Example 5.6:** Consider the simply supported beam shown.
- The beam is subjected to a dead load = 0.5 k/ft (including the weight of the beam) and live load = 1 k/ft.
- If $F_y = 50 \text{ ksi}$, is a **W12 x 65** adequate?



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Chapter 5 – Noncompact Shapes

- **Example 5.6:** Check for compactness.
- From Table 1-1 (1-26) for a **W12 x 65**

Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A in. ²	Depth, d in.	Web		Flange		Distance		Workable Gage	
			Thickness, t _w in.	t _w in.	Width, b _f in.	Thickness, t _f in.	k in.	A ₁ in.		T in.
W12	44.3	12.7	0.25	0.25	12.5	0.40	1/4	12.5	1/4	1/4
x16	39.9	13.4	0.25	0.25	12.5	0.40	1/4	12.5	1/4	1/4
x12	35.2	13.1	0.25	0.25	12.5	0.40	1/4	12.5	1/4	1/4
x10	31.2	12.7	0.25	0.25	12.5	0.40	1/4	12.5	1/4	1/4
x8	27.2	12.3	0.25	0.25	12.5	0.40	1/4	12.5	1/4	1/4
x6	23.2	11.9	0.25	0.25	12.5	0.40	1/4	12.5	1/4	1/4
x4	19.1	11.5	0.25	0.25	12.5	0.40	1/4	12.5	1/4	1/4
W12x65	65.0	12.2	0.375	0.375	12.0	0.60	1/4	12.0	1/4	1/4
x50	58.3	12.1	0.345	0.345	11.8	0.55	1/4	11.8	1/4	1/4
W12x50	50.0	12.2	0.370	0.370	11.8	0.55	1/4	11.8	1/4	1/4
x45	45.0	12.1	0.335	0.335	11.8	0.55	1/4	11.8	1/4	1/4
x40	40.0	11.9	0.325	0.325	11.8	0.55	1/4	11.8	1/4	1/4

Section has footnote f¹ Shape exceeds the compact limit for flexure with $F_y = 50 \text{ ksi}$.

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Chapter 5 – Noncompact Shapes

- **Example 5.6:** Here, we will *manually* check for compactness.
- From Table 1-1 (1-27) for a **W12 x 65**

Table 1-1 (continued)
W-Shapes
Properties

W14-W12

Nominal Wt.	Compact Section Criteria	Axis X-X			Axis Y-Y			Torsional Properties						
		b_f in.	t_f in.	r in.	b_f in.	t_f in.	r in.	J in. ⁴	C_w in. ⁶					
87	7.48	18.9	740	118	5.38	132	241	39.7	3.07	60.4	3.46	11.7	5.10	8270
79	8.22	20.7	662	107	5.34	119	216	35.8	3.05	54.3	3.43	11.7	3.84	7330
65	9.92	4.9	533	87.9	5.25	96.8	74	29.1	3.02	44.1	3.38	11.5	2.18	5780
58	7.92	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570
53	8.89	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
50	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440

$\frac{b_f}{2t_f} = 9.92$
 $S_x = 87.9 \text{ in}^3$
 $Z_x = 96.8 \text{ in}^3$

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Chapter 5 – Noncompact Shapes

- **Example 5.6:** Check for compactness.
- From Table B4.1b, the limiting width-to-thickness ratios are:

TABLE B4.1b
Width-to-Thickness Ratios: Compression Elements
Members Subjected to Flexure

Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Examples
			λ_p (compact)	λ_r (non-compact)	
10	(1) Flanges of rolled I-shaped sections (2) Flanges of channels (3) Flanges of tees	$\frac{b_f}{t_f}$	$0.38 \sqrt{\frac{E}{F_y}}$	$1.0 \sqrt{\frac{E}{F_y}}$	
11	Flanges of doubly and singly symmetric I-shaped built-up sections	$\frac{b_f}{t_f}$	$0.38 \sqrt{\frac{E}{F_y}}$	$1.0 \sqrt{\frac{E}{F_y}}$	
12	Legs of angle	$\frac{b_f}{t_f}$	$0.38 \sqrt{\frac{E}{F_y}}$	$1.0 \sqrt{\frac{E}{F_y}}$	
13	Flanges of all I-shaped sections and channels that flange about the minor axis	$\frac{b_f}{t_f}$	$0.38 \sqrt{\frac{E}{F_y}}$	$1.0 \sqrt{\frac{E}{F_y}}$	
14	Stems of tees	$\frac{b_f}{t_f}$	$0.38 \sqrt{\frac{E}{F_y}}$	$1.0 \sqrt{\frac{E}{F_y}}$	

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Chapter 5 – Noncompact Shapes

- **Example 5.6:** Check for compactness.
- From Table B4.1b, the limiting width-to-thickness ratios are:

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

$\lambda_{rf} = 1.0 \sqrt{\frac{E}{F_y}} = 1.0 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 24.08$

$\lambda = \frac{b_f}{2t_f} = 9.92$

Since $\lambda_{pf} < \lambda < \lambda_{rf}$, this shape is: **noncompact**

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Chapter 5 – Noncompact Shapes

- **Example 5.6:** Check the capacity based on the limit state of **flange local buckling**:

AISC Equation F3-1

$$M_n = \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right]$$

$M_p = F_y Z_x = 50 \text{ ksi} (96.8 \text{ in}^3) = 4,840 \text{ k in}$

$0.7F_y S_x = 0.7(50 \text{ ksi})(87.9 \text{ in}^3) = 3,076.5 \text{ k in}$

$M_n = \left[4,840 \text{ k in} - (4,840 \text{ k in} - 3,076.5 \text{ k in}) \left(\frac{9.92 - 9.15}{24.08 - 9.15} \right) \right]$

$= 4,749.25 \text{ k in} \leq M_p = 4,840 \text{ k in}$

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Chapter 5 – Noncompact Shapes

➤ Example 5.6: Check the capacity based on the *lateral-torsional buckling limit state*. From the Z_x table, Table 3-2 (3-25):

Table 3-2 (continued)
W-Shapes
Selection by Z_x

$F_y = 50$ ksi

Z_x

$L_p = 11.9$ ft
 $L_r = 35.1$ ft

Shape	Z_x in ³	M_{p_c}/Q_c		M_{p_o}/Q_o		M_{p_x}/Q_x		$\phi_b M_p$ kips	$\phi_b B F$ kips	L_p ft	L_r ft	I_x in ⁴	S_x		S_y	
		ASD	LRFD	ASD	LRFD	ASD	LRFD						in ³	in ³	in ³	in ³
W21x50	110	274	413	165	248	12.1	18.3	4.59	13.6	984	158	237				
W12x72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159				
W21x48 ⁽¹⁾	107	265	398	162	244	9.89	14.8	6.09	16.5	959	144	216				
W16x57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212				
W14x81	102	254	383	161	242	4.83	7.48	8.65	27.5	640	104	156				
W18x50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192				
W12x65 ⁽¹⁾	99.8	237	356	154	231	3.59	5.39	11.9	35.1	533	94.4	142				
W21x44	95.4	226	338	143	214	11.1	16.8	4.43	12.1	643	145	217				
W16x50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186				

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Chapter 5 – Noncompact Shapes

➤ Example 5.6: Check the capacity based on the *lateral-torsional buckling limit state*. From the Z_x table, Table 3-2 (3-25):

$L_b = 30$ ft $L_p < L_b < L_r$ ∴ Failure is by *inelastic LTB*

Table 3-2 (continued)
W-Shapes
Selection by Z_x

$F_y = 50$ ksi

Z_x

$L_p = 11.9$ ft
 $L_r = 35.1$ ft

Shape	Z_x in ³	M_{p_c}/Q_c		M_{p_o}/Q_o		M_{p_x}/Q_x		$\phi_b M_p$ kips	$\phi_b B F$ kips	L_p ft	L_r ft	I_x in ⁴	S_x		S_y	
		ASD	LRFD	ASD	LRFD	ASD	LRFD						in ³	in ³	in ³	in ³
W21x50	110	274	413	165	248	12.1	18.3	4.59	13.6	984	158	237				
W12x72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159				
W21x48 ⁽¹⁾	107	265	398	162	244	9.89	14.8	6.09	16.5	959	144	216				
W16x57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212				
W14x81	102	254	383	161	242	4.83	7.48	8.65	27.5	640	104	156				
W18x50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192				
W12x65 ⁽¹⁾	99.8	237	356	154	231	3.59	5.39	11.9	35.1	533	94.4	142				
W21x44	95.4	226	338	143	214	11.1	16.8	4.43	12.1	643	145	217				
W16x50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186				

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Chapter 5 – Noncompact Shapes

➤ Example 5.6: Check the capacity based on the *lateral-torsional buckling limit state*. From the Z_x table, Table 3-2 (3-25):

$L_b = 30$ ft $L_p < L_b < L_r$ ∴ Failure is by *inelastic LTB*

$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right]$ **AISC Equation F3-1**

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Chapter 5 – Noncompact Shapes

➤ Example 5.6: Values of C_b for other cases are available in Table 3-1 of Part 3 of the Manual.

Table 3-1
Values of C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
None	Load at midpoint	1.12
	Load at load point	1.10
None	Loads at third points	1.14
	At load points Loads symmetrically placed	1.10
None	Load at quarter points	1.14
	At load points Loads at quarter points	1.10

Table 3-1
Values of C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
None	Load at midpoint	1.12
	At midpoint	1.12
None	At third points	1.14
	At quarter points	1.14
None	At fifth points	1.14
	At fifth points	1.14

For a uniformly loaded, simply supported beam with lateral support at the ends, $C_b = 1.14$

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Chapter 5 – Noncompact Shapes

➤ Example 5.6: AISC Equation F2-2 gives:

$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right]$

$= 1.14 \left[4,840 \text{ k in} - (4,840 \text{ k in} - 3,076.5 \text{ k in}) \left(\frac{30 \text{ ft} - 11.9 \text{ ft}}{35.15 \text{ ft} - 11.9 \text{ ft}} \right) \right]$

$= 3,929.83 \text{ k in} < M_p = 4,840 \text{ k in}$

FLB: $M_n = 4,749.25 \text{ k in}$

Inelastic LTB: $M_n = 3,929.83 \text{ k in}$ **Controls**

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Chapter 5 – Noncompact Shapes

➤ Example 5.6: The smaller moment strength controls, in this case, lateral-torsional buckling, $M_n = 3,929.83 \text{ k in}$.

The LRFD design strength is:

$\phi_b M_n = 0.9(3,929.83 \text{ k in}) = 3,536.85 \text{ k in} = 294.74 \text{ k ft}$

The factored load and moment are:

$w_u = 1.2w_d + 1.6w_l = 1.2(0.5 \text{ k/ft}) + 1.6(1.0 \text{ k/ft})$

$= 2.20 \text{ k/ft}$

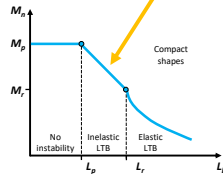
$M_u = \frac{w_u L^2}{8} = \frac{(2.20 \text{ k/ft})(30 \text{ ft})^2}{8} = 247.50 \text{ k ft} < 294.74 \text{ k ft}$

O.K.

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Chapter 5 – Noncompact Shapes

- Noncompact shapes are identified in the Z_x table by an “F”.
- Noncompact shapes are also treated differently in the Z_x table in the following way.
- The tabulated value of L_p is the unbraced length at which the nominal strength based on **inelastic lateral-torsional buckling** equals **flange local buckling** strength.



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Chapter 5 – Noncompact Shapes

- This is the **maximum unbraced length** for which the nominal strength can be taken as the strength based on **flange local buckling**.
- Recall that L_p for **compact** shapes is the maximum unbraced length for which the nominal strength can be taken as the **plastic moment**.

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Chapter 5 – Noncompact Shapes

- For the shape in **Example 5.6**, equate the nominal strength based on **FLB** to the strength based on **inelastic LTB** (AISC Equation F2-2), with $C_b = 1.0$.

$$M_n = M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \quad \text{AISC Equation F2-2}$$

The value of L_r given in **Example 5.6** is unchanged.

The value of L_p is computed using **AISC Equation F2-5**:

$$L_p = 1.76r_y \sqrt{\frac{E}{F_y}} = 1.76(3.02 \text{ in}) \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 128.01 \text{ in} = 10.67 \text{ ft}$$

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Chapter 5 – Noncompact Shapes

- Returning to **AISC Equation F2-2** gives:

$$M_n = M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \quad \text{FLB: } M_n = 4,749.25 \text{ k in}$$

$$M_p = 4,840 \text{ k in}$$

$$4,749.25 \text{ k in} = \left[4,840 \text{ k in} - (4,840 \text{ k in} - 3,076.5 \text{ k in}) \left(\frac{L_b - 10.67 \text{ ft}}{35.1 \text{ ft} - 10.67 \text{ ft}} \right) \right]$$

$$\therefore L_p = 11.93 \text{ ft}$$

- In the Z_x table, $L_p = 11.9 \text{ ft}$ for a **W12 x 65** with $F_y = 50 \text{ ksi}$.
- In addition, the available strength values, $\phi_b M_{px}$ are based on **flange local buckling** rather than the **plastic moment**.

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Chapter 5 – Moment Strength

- Let's present a summary of the procedure for the computation of **nominal moment strength** for **I** and **C**-shaped sections bent about the **x**-axis.
- All terms in the following equations have been previously defined.
- To save space, the **AISC** equation numbers will not be shown.
- This summary is for **compact** and **noncompact** shapes only.
- **Slender** shapes are not included.

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Chapter 5 – Moment Strength

1. Determine whether the shape is **compact**.
2. If the shape is **compact**, check for **lateral-torsional buckling** as follows.

For $L_b \leq L_p$ there is no **LTB**, and $M_n = M_p$

For $L_p < L_b \leq L_r$ compute **inelastic LTB**

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$$

For $L_b > L_r$ compute **elastic LTB** $M_n = F_{cr} S_x \leq M_p$

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_0 (r_{ts})^2}}$$

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Chapter 5 – Moment Strength

3. If the shape is **noncompact** because of the **flange**, the nominal strength will be:

$$\min \begin{cases} \text{flange local buckling (FLB)} \\ \text{lateral-torsional buckling (LTB)} \end{cases}$$

- a. Flange local buckling:

If $\lambda \leq \lambda_{pf}$ there is no **FLB**, and

If $\lambda_{pf} < \lambda \leq \lambda_{rf}$ the flange is **noncompact**, and

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right]$$

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Chapter 5 – Moment Strength

3. If the shape is **noncompact** because of the **flange**, the nominal strength will be:

$$\min \begin{cases} \text{flange local buckling (FLB)} \\ \text{lateral-torsional buckling (LTB)} \end{cases}$$

- b. Lateral-torsional buckling:

If $L_b \leq L_p$ there is no **LTB**, and

If $L_p < L_b \leq L_r$ there is inelastic **LTB**, and

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$$

If $L_b > L_r$ there is elastic **LTB**, and $M_n = F_{cr} S_x \leq M_p$

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b / r_{ts})^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}} \right)^2}$$

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Chapter 5 – Beams

Any questions?



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