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Course Description

July 10, 2017– Fundamentals of Stability for Steel Design: Design of Flexural Members

This lecture will focus on the design of flexural members for the pertinent stability limit states. Solutions for the effects of moment gradient and load position will be covered including moment gradient factors for a variety of common design situations. This lecture will include material pertinent to both rolled sections as well as built-up members. Efficient use of the design aids in the AISC 360-10/14th Ed. Manual will be addressed as well as methods for the preliminary sizing of built-up girders.





Learning Objectives

- Explain the compact section criteria of a flexural member to reach M_p .
- Explain the application of moment gradient factor in beam design.
- Determine moment gradient factors for beams braced on one flange.
- List the steps for preliminary design of a built-up shape.



Fundamentals of Stability for Steel Design Session 4: Design of Flexural Members

July 10, 2017



Presented by
Todd A. Helwig, Ph.D., P.E.
University of Texas at Austin



There's always a solution in steel.



There's always a solution in steel.

Fundamentals of Stability for Steel Design



Session 4 Design of Flexural Members

Todd A. Helwig, Ph.D., P.E.



Structural Stability Research Council

- SSRC values being a part of the AISC Night School series. Many thanks to Brent Leu, Christina Harber, and Nancy Gavlin for all the work they do with the speakers to develop a successful program.
- SSRC is an organization consisting of educators, practicing engineers, and industry professionals interested in stability as it relates to analysis, design, and behavior. If you have any interest in stability, consider becoming a member and/or attending our Annual Stability Conference (ASC).
- SSRC holds the Annual Stability Conference (ASC) as part of AISC's North American Steel Construction Conference.
- The next ASC/NASCC is in Baltimore April 10-13, 2018. The SSRC ASC begins on Tuesday April 10 at approximately 1:00pm.
- For more information on SSRC/membership/etc. visit www.ssrcweb.org

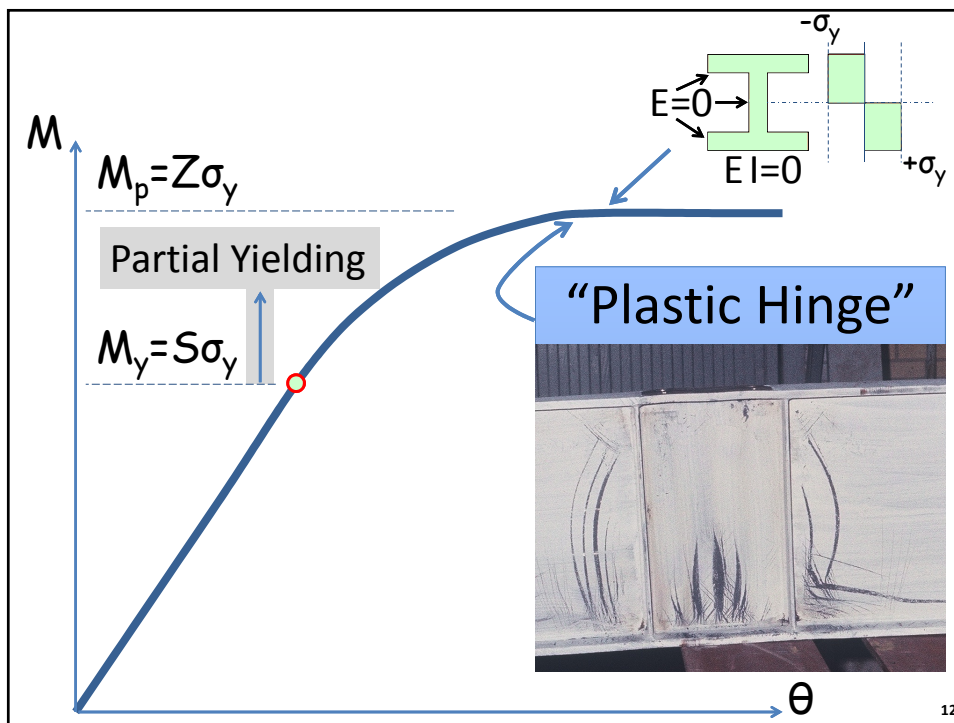
10



Design of Flexural Members

- Cross-Sectional Requirements to reach M_p (Compact Section Criteria)
- Lateral Torsional Buckling
 - Moment Gradient Factor (C_b) Gravity Loading
 - Using AISC LTB Charts along with Z_x Tables
- Moment Gradient Factors with Unconventional Bracing/Loading
 - Beams braced on one flange (reverse curvature)
 - Buckling of unbraced cantilevers
- Design of Built-up Shapes

11



12

Reaching/Designing Based Upon M_p

In order to make the most efficient use of the material in a cross section, we often want to be able to design based upon the plastic moment capacity, M_p . While there are a number of limit states that prevent us from being able to design based upon M_p , we will focus tonight on the following categories that involve local and global stability modes :

- Local Web Buckling (LWB)
- Local Flange Buckling (LFB)
- Lateral Torsional Buckling (LTB)

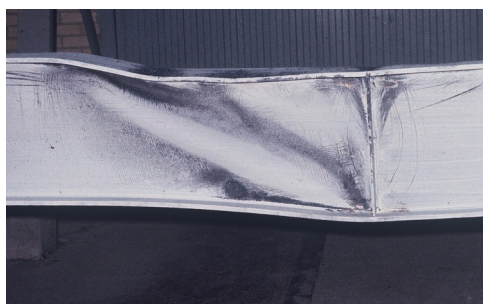
13

Local Buckling Modes

Local buckling is a failure of the cross section and is related to the width and thickness of the plate elements that make up the cross section.



Local Flange Buckling (LFB)

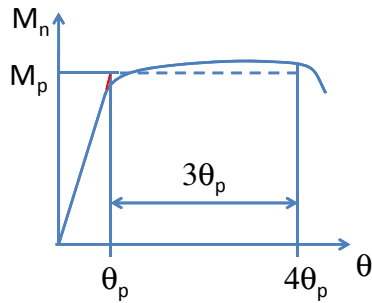


Local Web (Bend) Buckling (LWB)

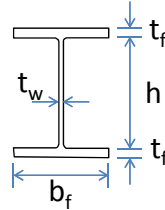
14

Local Buckling Modes

A flange or a web that can undergo an inelastic rotation capacity of “3” and maintain $M_n \geq M_p$ without local buckling is said to be compact.



Local buckling of the section is controlled by limiting the slenderness, λ , of the plate element.



Web: $\lambda_w = h/t_w$

Flange: $\lambda_f = b_f/2t_f$

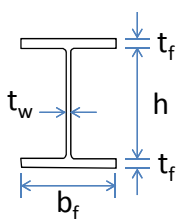
The compact section requirements only address local buckling modes. The compact section requirements have nothing to do with lateral torsional buckling (LTB).

15

Compact Section Requirements

The compact section requirements are defined in the AISC Section B4.1 (Table B4.1b, Page 16.1-17)

To be compact $\lambda \leq \lambda_p$.



Web: $\lambda_w = h/t_w \leq 3.76 \sqrt{\frac{E}{F_y}} = \lambda_{pw}$

Flange: $\lambda_f = b_f/2t_f \leq 0.38 \sqrt{\frac{E}{F_y}} = \lambda_{pf}$

In order for a cross section to be compact, both the flange and the web must be compact.

16

Compact Section Requirements

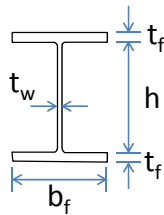
Consider the webs of all rolled wide flange shapes:

$$\text{Web: } \lambda_{pw} = 3.76 \sqrt{\frac{E}{F_y}} = \frac{640}{\sqrt{F_y}} \quad \text{US Units}$$

For Grade 50 Steel, $\lambda_{pw} = 90.5$

The rolled W-shape with the most slender web is a W30x90 with $h/t_w = 57.5 \ll 90.5 = \lambda_{pw}$.

Note: The W30x90 would need to have $F_y = 124$ ksi to have a non-compact web



Therefore, all rolled W-shapes have compact webs for the current grades of steel that are widely used in practice.

17

Compact Section Requirements

Considering flanges of all rolled wide flange shapes:

$$\text{Flange: } \lambda_{pf} = \frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}} = \frac{65}{\sqrt{F_y}} \quad \text{US Units}$$

For Grade 50 Steel, $\lambda_{pw} = 9.2$

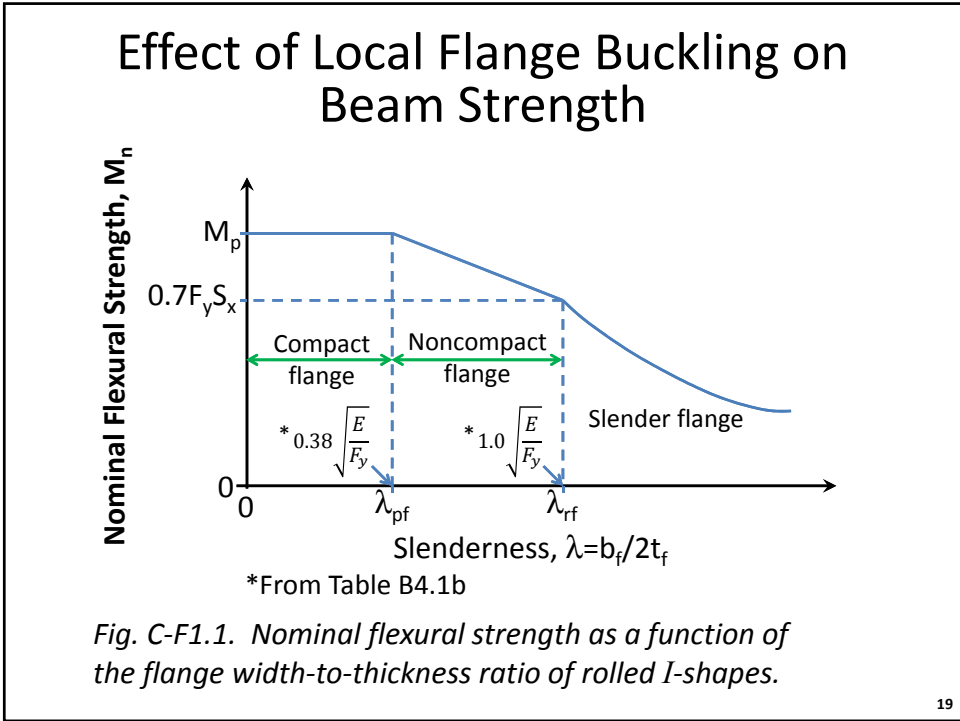
11 sections do not have compact flanges ($F_y = 50$ ksi).

Consider User Note: Section F2 - Page 16.1-47

User Note: All current ASTM A6 W, S, M, C and MC shapes except W21x48, W14x99, W14x90, W12x65, W10x12, W8x31, W8x10, W6x15, W6x9, W6x8.5 and M4x6 have compact flanges for $F_y = 50$ ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at $F_y \leq 65$ ksi (450 MPa).

For built-up shapes, we generally design the web for the applied shear and will often have a non-compact or slender web.

18



Z_x Tables in AISC Manual (Pg. 3-24)

Z_x

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

F_y = 50 ksi

Shape	Z _x in. ³	M _{px} /Ω _b		φ _b M _{px}		M _{rx} /Ω _b		φ _b M _{rx}		BF/Ω _b		φ _b BF		L _p ft	L _r ft	I _x in. ⁴	V _{nx} /Ω _v		φ _v V _{nx}	
		kip-ft	LRFD	kip-ft	LRFD	kip-ft	LRFD	kip-ft	LRFD	kip-ft	LRFD	kip-ft	LRFD				kip-ft	LRFD		
W24×84	224	559	840	342	515	16.2	24.2	6.89	20.3	2370	227	340								
W21×93	221	551	829	335	504	14.6	22.0	6.50	21.3	2070	251	376								
W12×136	214	534	803	325	488	4.02	6.06	11.2	63.2	1240	212	318								
W14×120	212	529	795	332	499	5.09	7.65	13.2	51.9	1380	171	257								
W18×97	211	526	791	328	494	9.41	14.1	9.36	30.4	1750	199	299								
W24×76	200	499	750	307	462	15.1	22.6	6.78	19.5	2100	210	315								
W16×100	198	494	743	306	459	7.86	11.9	8.87	32.8	1490	199	298								
W21×83	196	489	735	299	449	13.8	20.8	6.46	20.2	1830	220	331								
W14×109	192	479	720	302	454	5.01	7.54	13.2	48.5	1240	150	225								
W18×86	186	464	698	290	436	9.01	13.6	9.29	28.6	1530	177	265								
W12×120	186	464	698	285	428	3.94	5.95	11.1	56.5	1070	186	279								
W24×68	177	442	664	269	404	14.1	21.2	6.61	18.9	1830	197	295								
W16×89	175	437	656	271	407	7.76	11.6	8.80	30.2	1300	176	265								
W14×99 ^a	173	430	646	274	412	4.91	7.36	13.5	45.3	1110	138	207								
ASD	LRFD																			

^a Shape exceeds compact limit for flexure with *F_y = 50 ksi*.
^b Shape does not meet the *h/t_w* limit for shear in AISC Specification Section G2.1(a) with *F_y = 50 ksi*; therefore, φ_v = 0.90 and Ω_v = 1.67.

Ω_b = 1.67 φ_b = 0.90
 Ω_v = 1.50 φ_v = 1.00

The Design of Rolled Beams Typically starts in the Z_x Tables. There is a great deal of information in these tables and we will be spending some time looking at the efficient use of these tables.

Consider Design Z_x Tables in AISC Manual

Pg. 3-24

Table 3-2 (continued)
W-Shapes
 Selection by Z_x $F_y = 50$ ksi

Shape	Z_x in. ³	M_p/Ω_b		$\phi_p M_p$		M_p/Ω_b		$\phi_p M_p$		BF/Ω_b		$\phi_p BF$		L_p ft	L_r ft	I_x in. ⁴	M_p/Ω_b		$\phi_p M_p$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD				kips	LRFD	kips	LRFD
W24x84	224	559	840	342	515	16.2	24.2	6.89	20.3	2370	227	340								
W21x93	221	551	829	335	504	14.8	22.0	6.50	21.3	2070	251	376								
W12x136	214	534	809	325	488	4.02	6.06	11.2	33.2	1240	212	318								
W14x120	212	529	795	332	499	5.09	7.65	13.2	51.9	1380	171	257								
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W24x76	200	499	750	307	462	15.1	22.6	6.78	19.5	2100	210	315								
W16x100	198	494	743	306	459	7.86	11.9	8.87	32.8	1490	199	298								
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W14x99	173	430	646	274	412	4.91	7.36	13.5	45.3	1110	138	207								

* Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
 † Shape does not meet the h/t_w limit for shear in AISC Specification Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

W14x99: the footnote 'f' means the section is non-compact

$$Z_x = 173 \text{ in}^3$$

$$\phi M_p = \phi Z_x F_y$$

$$\phi M_p = \frac{0.9 \times 173 \times 50 \text{ ksi}}{12 \text{ in/ft}}$$

$$\phi M_p = 649 \text{ k} - \text{ft}$$

From Table: $\phi M_p = 646 \text{ k} - \text{ft} < 649 \text{ k} - \text{ft}$
 The Table reflects that the section is non-compact.

Survey Question: Choose the Best Answer to the Question

Choose the best answer that shows what the term "Compact Section" indicates:

- A) The section is not likely to fail due to lateral torsional buckling (LTB)
- B) The section is not likely to fail due to local web buckling (LWB)
- C) The section is not likely to fail due to local flange buckling (LFB)
- D) (b) and (c), only
- E) (a), (b), and (c)

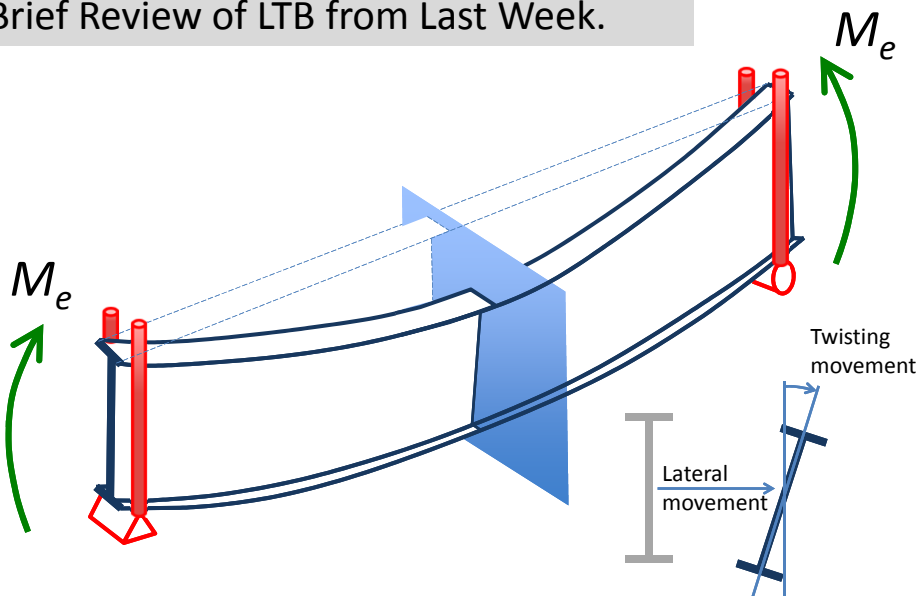
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- C) The section is not likely to fail due to local flange buckling (LFB)
- D) (b) and (c), only
- E) (a), (b), and (c)

23

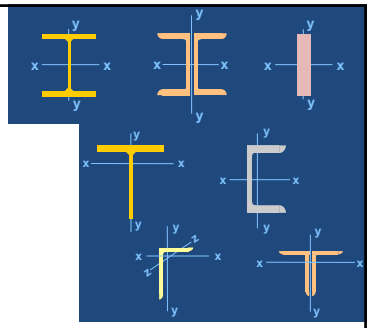
Brief Review of LTB from Last Week.



“Lateral torsional buckling (LTB)”

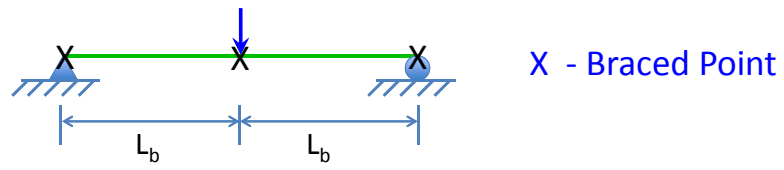
24

Member instability: Lateral Torsional Buckling



25

Definition of L_b – Unbraced Length for Beams



Page 16.1-47:

L_b is the spacing between points that are either braced against lateral displacement of the compression flange OR braced against twist of the cross section.

Therefore, effective bracing for beams can be achieved by preventing EITHER lateral movement of the compression flange OR twist of the section.

26



Elastic LTB: Chapter F2 Equations (Pg. 16.1-47,48)

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

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \quad C_w = \frac{I_y h_o^2}{4}$$

$$M_{cr} = F_{cr} S_x$$

27

Elastic LTB: Chapter F Equations (Pg. 16.1-47,48)

Page 16.1-48

User Note: Equations F2-3 and F2-4 provide identical solutions to the following expression for lateral-torsional buckling of doubly symmetric sections that has been presented in past editions of the AISC LRFD Specification:

$$M_{cr} = C_b \frac{\pi}{L_b} \sqrt{EI_y GJ + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w}$$

The advantage of Equations F2-3 and F2-4 is that the form is very similar to the expression for lateral-torsional buckling of singly symmetric sections given in Equations F4-4 and F4-5.

There are benefits to both solutions. Equation F2-4 (F_{cr}), provides a similar solution for both doubly- and singly-symmetric sections. One of the nice features about the above (Timoshenko) solution is that the St. Venant and Warping terms are clearly evident, which is explained further later in the lecture.

28



Effect of Moment Gradient

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$

All moments are the absolute value of the moment at the specified location.

M_{max} = Maximum moment anywhere along L_b

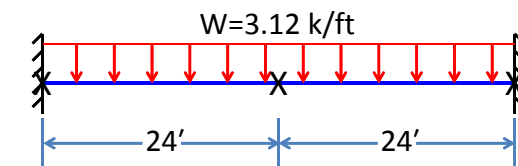
M_A = Moment at $L_b/4$

M_B = Moment at $L_b/2$

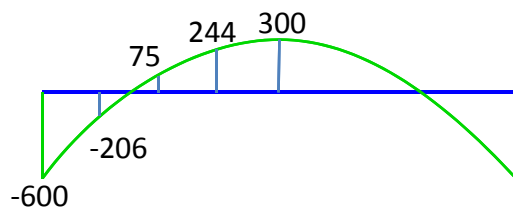
M_C = Moment at $3L_b/4$

29

Effect of Moment Gradient



X – brace point



$$C_b = \frac{12.5(600)}{2.5(600) + 3(206) + 4(75) + 3(244)} = 2.38$$

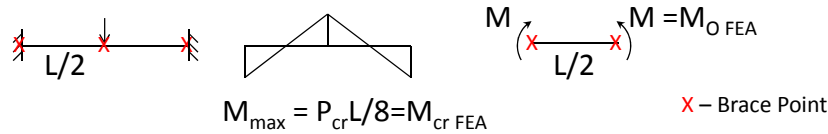
30

Calculating C_b factors using FEA programs

$$C_b = \frac{M_{cr \text{ FEA}}}{M_{O \text{ FEA}}}$$

$M_{cr \text{ FEA}}$ = Maximum moment along unbraced length for beam from eigenvalue buckling analysis with given support conditions and brace conditions (Eigenvalue buckling analysis).

$M_{O \text{ FEA}}$ = Buckling moment from eigenvalue buckling analysis with uniform moment loading and same unbraced length from M_{cr} analysis.



31

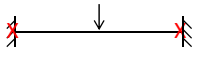
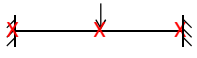

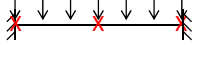
C_b – FEA versus Equation Comparison

	BASP - C_b W16x26 - L/d = 30	AISC - C_b
	1.40	1.32
	1.89	1.67
	1.16	1.14
	1.37	1.30

X – Brace Point

32

C_b – FEA versus Equation Comparison

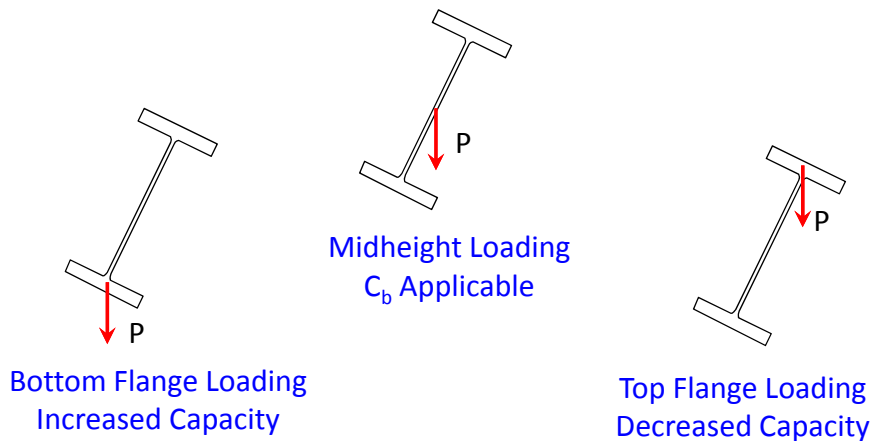
	BASP - C_b W16x26 – L/d = 30	AISC - C_b
	1.76	<u>1.92</u>
	2.70	2.27
	2.66	2.38
	3.39	2.38

Note: Unconservative, however the load point is usually a braced point

X – Brace Point

33

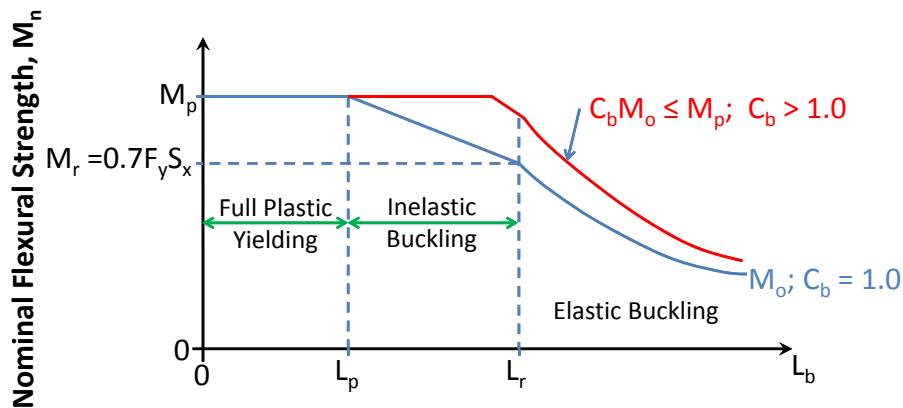
Load Position on the Cross Section



There are a number of mitigating factors that offset the effects of top flange loading is not an issue and C_b factors are directly applicable. We are therefore not going to focus on load position; however consult the literature if you think you have an issue.

34

Lateral Torsional Buckling Strength (Elastic and Inelastic Region)

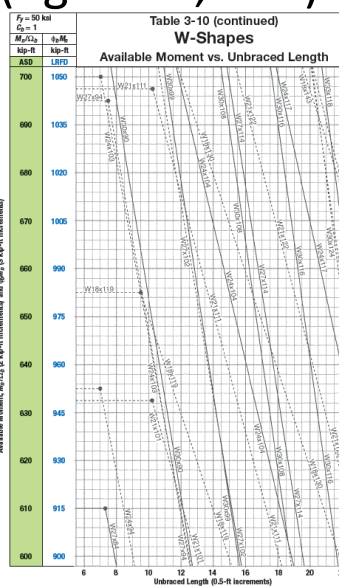
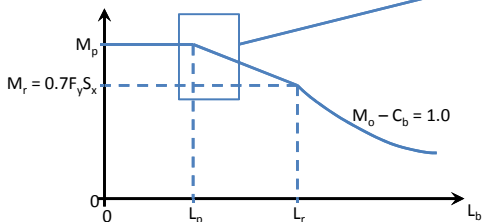


To design for LTB, it is imperative that we have a good understanding of the above curve.

35

Lateral Torsional Buckling Strength Curves in AISC Manual (Pg. 3-99, 134)

The AISC Manual provides the graphs of ϕM_n versus L_b for $C_b = 1.0$ for all of the rolled wide flange shapes. For a given shape, the curve stretches over 2 to 4 pages. While finding the appropriate curve for a given shape can be difficult at first, with experience it can be found very quickly.



36

Design Process (LFB, LWB, and LTB)

- When designing rolled beams based upon the elastic distribution of moments, LFB and LWB, have no impact on the design approach. (LWB not a problem and LFB is already accounted for in the Z_x Tables).
- In some cases with redundant structures, engineers may make use of inelastic re-distribution of moments, which requires a compact section (no LWB or LFB). In this situation, the LTB requirements also require more stringent brace spacing (L_{pd} requirements – Appendix 1); however we will not be focusing on plastic design concepts in this course.
- In designing for LTB, there are a number of different methods that can be used to evaluate the buckling capacity. All of the methods start the same way, by determining the maximum factored moment (M_u) and finding the lightest section in the Z_x tables with $\phi M_p > M_u$. (in ASD $M_p/\Omega > M_a$)

37

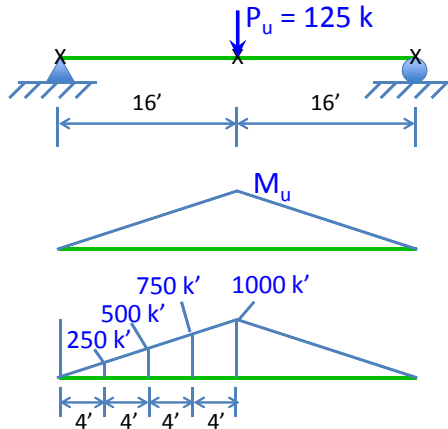
Design Process (LTB)

- Finding the appropriate beam chart can often be the most difficult aspect of evaluating the LTB capacity using the AISC Manual Design Aides.
- The beam charts extend from (LRFD) design moments ranging from 0 k-ft to 12000 k-ft. on 36 pages and include the design strength curves for over 260 W-shapes.
- To efficiently use the tables, the designer needs to be able to quickly locate one or two of the charts that might have the curve with the appropriate L_b so that the buckling strength (with $C_b = 1.0$) can be determined.
- With experience, the designer can not only locate the page with the curve but also often relatively closely locate the point on the page that has the specific curve to be found.

38



Design Example 1



Find the lightest W-shape that can safely support the factored 125 k load. Start by neglecting self weight.

$$M_u = \frac{P_u L}{4} = \frac{125 \times 32}{4} = 1000 \text{ k'}$$

Find the lightest W-shape that supports $M_u=1000 \text{ k'}$, with $L_b=16'$ and $C_b=1.67$.

$$C_b = \frac{12.5(1000)}{2.5(1000) + 3(750) + 4(500) + 3(250)} = \underline{\underline{1.67}}$$

39

Design Ex. 1 (continued) Pg. 3-23 (Cropped and Spliced Table)

$F_y = 50 \text{ ksi}$
Table 3-2 (continued)
W-Shapes
 Selection by Z_x

Shape	Z_x in. ³	M_{px}/Ω_b		M_{rx}/Ω_b		BF/Ω_b		L_p ft	L_r ft	I_x in. ⁴	V_{hx}/Ω_v		V_{vx}/Ω_v	
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips		
		ASD	LRFD	ASD	LRFD	ASD	LRFD				ASD	LRFD		
W30×90*	283	706	1060	428	643	20.6	30.8	7.38	20.9	3610	249	374		
W24×103	280	699	1050	428	643	18.2	27.4	7.03	21.9	3000	270	404		
W21×111	279	696	1050	435	654	12.4	18.9	10.2	31.2	2670	237	355		
W27×94	278	694	1040	424	638	19.1	28.5	7.49	21.6	3270	264	395		
W12×170	275	686	1030	410	617	4.11	6.15	11.4	78.5	1650	269	403		
W18×119	262	654	983	403	606	10.1	15.2	9.50	34.3	2190	249	373		
W14×145	260	649	975	405	609	5.13	7.69	14.1	61.7	1710	201	302		
W24×94	254	634	953	388	583	17.3	26.0	6.99	21.2	2700	250	375		
W21×101	253	631	949	396	596	11.8	17.7	10.2	30.1	2420	214	321		
W27×84	244	609	915	372	559	17.6	26.4	7.31	20.8	2850	246	368		
W12×152	243	606	911	365	549	4.06	6.10	11.3	70.6	1430	238	358		

40



Design Example 1 (continued)

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

$F_y = 50$ ksi

Shape	Z_x in. ³	M_n/Ω_b		$\phi_b M_n$		M_p/Ω_b		$\phi_b M_p$		BF/Ω_b		$\phi_b BF$		L_p ft.	L_r ft.	I_x in. ⁴	M_u/Ω_b		$\phi_b M_u$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft.	ft.	ASD	LRFD							
W30x116	378	943	1420	575	864	24.8	37.4	7.74	22.6	4930	339	509								
W21x147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3630	318	477								
W24x131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445								
W18x158	356	888	1340	541	814	10.5	15.9	9.68	42.8	3060	319	479								
W14x193	355	886	1330	541	814	5.30	7.93	14.3	79.4	2400	276	414								
W12x210	348	868	1310	510	767	4.25	6.45	11.6	95.8	2140	347	520								
W30x108	346	863	1300	522	785	23.5	35.5	7.59	22.1	4470	325	487								
W27x114	343	856	1290	522	785	21.7	32.8	7.70	23.1	4080	311	467								
W21x132	333	831	1250	515	774	13.2	19.9	10.3	34.2	3220	283	425								
W24x117	327	816	1230	508	764	15.4	23.3	10.4	30.4	3540	287	401								
W18x143	322	803	1210	493	740	10.3	15.7	9.61	39.6	2750	285	427								
W14x176	320	798	1200	491	738	5.20	7.83	14.2	73.2	2140	252	378								
W30x99	312	778	1170	470	706	22.2	33.4	7.42	21.3	3990	309	463								
W12x190	311	776	1170	459	690	4.18	6.33	11.5	87.3	1890	305	458								
W21x122	307	766	1150	477	717	12.9	19.3	10.3	32.7	2960	260	391								
W27x102	305	761	1140	466	701	20.1	29.8	7.59	22.3	3620	279	419								
W18x130	290	724	1090	447	672	10.2	15.4	9.54	36.6	2460	259	388								
W24x104	289	721	1080	451	677	14.3	21.3	10.3	29.2	3100	241	362								
W14x159	287	716	1080	444	667	5.17	7.85	14.1	66.7	1900	224	335								
W30x90	283	706	1060	428	643	20.6	30.8	7.38	20.9	3610	249	374								
W24x103	280	699	1050	428	643	18.2	27.4	7.03	21.9	3000	270	404								
W21x111	279	696	1050	435	654	12.4	18.9	10.2	31.2	2670	237	355								
W27x94	278	694	1040	424	638	19.1	28.5	7.49	21.6	3270	264	395								
W12x170	275	686	1030	410	617	4.11	6.15	11.4	78.5	1650	269	403								
W18x119	262	654	983	403	606	10.1	15.2	9.50	34.3	2190	249	373								
W14x145	260	649	975	405	609	5.13	7.69	14.1	61.7	1710	201	302								
W24x94	254	634	953	388	583	17.3	26.0	6.99	21.2	2700	250	375								
W21x101	253	631	949	396	596	11.8	17.7	10.2	30.1	2420	214	321								
W27x84	244	609	915	372	559	17.6	26.4	7.31	20.8	2850	246	368								
W12x152	243	606	911	365	549	4.06	6.10	11.3	70.6	1430	238	358								

Fig. 3-23 of AISC Manual

$M_u = 1000$ k'

A W30x90 is the lightest section with $\phi M_p = 1060$ k' > 1000 k' = M_u .

Although it doesn't matter in this problem, the section is also compact (no footnote 'f'). The footnote 'V' just means that the web slenderness is such that $\phi_v = 0.9$.

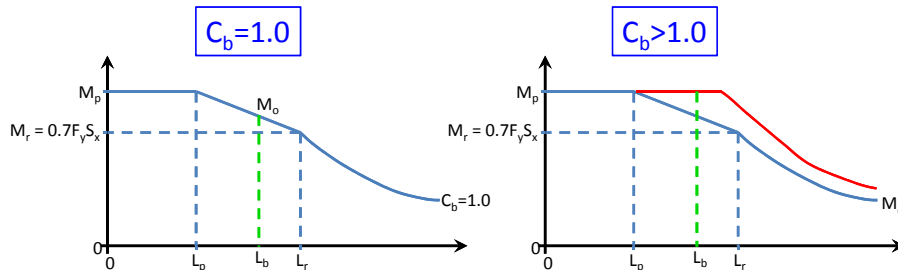
Hypothetical: What if, due to architectural limitations we had a depth limit for the beam equal to 18 inches?

41

Design Example 1 (continued)

We now need to check if the W30x90 can support the design moment, $M_u = 1000$ k', with $L_b = 16$ ft. and $C_b = 1.67$.

One way to do this is to find the beam chart that has the curve for the W30x90 for $L_b = 16'$. That will give us the buckling capacity of the beam with $C_b = 1.0$ - I'm going to call that ϕM_o . We can then check that:
 $\phi M_n = C_b \phi M_o = 1.67 \times \phi M_o > 1000$ k' = M_u



42

Design Example 1 (continued)

$F_y = 50$ ksi

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

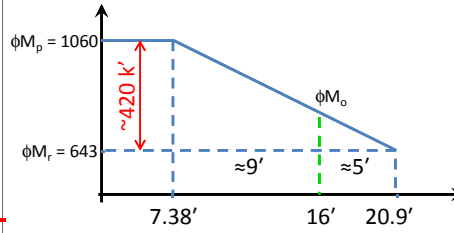
Z_x

Shape	Z_x in ²	M_p/Ω_b		M_r/Ω_b		$\phi_b M_p$		L_p ft	L_r ft	I_x in ⁴	K_{tr}/Ω_b		$\phi_b W_{plz}$ kips
		ASD	LRFD	ASD	LRFD	ASD	LRFD				ASD	LRFD	
W30x116	378	943	1420	575	864	24.8	37.4	7.74	22.6	4830	339	505	
W21x147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3830	318	477	
W24x131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445	
W18x158	356	888	1340	541	814	10.5	15.9	9.68	42.8	3060	319	479	
W14x193	355	886	1330	541	814	5.30	7.93	14.3	79.4	2400	276	414	
W12x210	348	868	1310	510	767	4.25	6.45	11.6	95.8	2140	347	520	
W30x108	346	863	1300	522	785	23.5	35.5	7.59	22.1	4470	325	487	
W27x114	343	856	1290	522	785	21.7	32.8	7.70	23.1	4080	311	467	
W21x132	333	831	1250	515	774	13.2	19.9	10.3	34.2	3220	283	425	
W24x117	327	816	1230	508	764	15.4	23.3	10.4	30.4	3540	267	401	
W18x143	322	803	1210	483	740	10.3	15.7	9.61	39.6	2750	285	427	
W14x176	320	798	1200	491	738	5.20	7.83	14.2	73.2	2140	252	378	
W30x99	312	778	1170	470	706	22.2	33.4	7.42	21.3	3990	309	463	
W12x190	311	776	1170	459	690	4.18	6.33	11.5	87.3	1890	305	458	
W21x122	307	766	1150	477	717	12.9	19.3	10.3	32.7	2960	260	391	
W27x102	305	761	1140	466	701	20.1	29.8	7.59	22.3	3620	279	419	
W18x130	290	724	1090	447	672	10.2	15.4	9.54	36.6	2460	259	388	
W24x104	289	721	1080	451	677	14.3	21.3	10.3	29.2	3100	241	362	
W14x159	287	716	1080	444	667	5.17	7.85	14.1	66.7	1900	224	335	
W30x90	283	706	1060	428	643	20.6	30.8	7.38	20.9	3610	249	374	
W24x103	280	699	1050	428	643	18.2	27.4	7.03	21.9	3000	270	404	
W21x111	279	696	1050	435	654	12.4	18.9	10.2	31.2	2670	237	355	
W27x94	278	694	1040	424	638	19.1	28.5	7.49	21.6	3270	264	395	
W12x170	275	686	1030	410	617	4.11	6.15	11.4	78.5	1650	269	403	
W18x119	262	654	983	403	606	10.1	15.2	9.50	34.3	2190	249	373	
W14x145	260	649	975	405	609	5.13	7.69	14.1	61.7	1710	201	302	
W24x94	254	634	953	388	583	17.3	26.0	6.99	21.2	2700	250	375	
W21x101	253	631	949	396	596	11.8	17.7	10.2	30.1	2420	214	321	
W27x84	244	609	915	372	559	17.6	26.4	7.31	20.8	2850	246	368	
W12x152	243	606	911	365	549	4.06	6.10	11.3	70.6	1430	238	358	

Fig. 3-23 of AISC Manual

Find ϕM_o with $L_b = 16'$

Before randomly flipping pages in the charts, consider information in Z_x tables. $\phi M_p = 1060$ k', $\phi M_r = 643$ k', $L_p = 7.38'$, and $L_r = 20.9'$

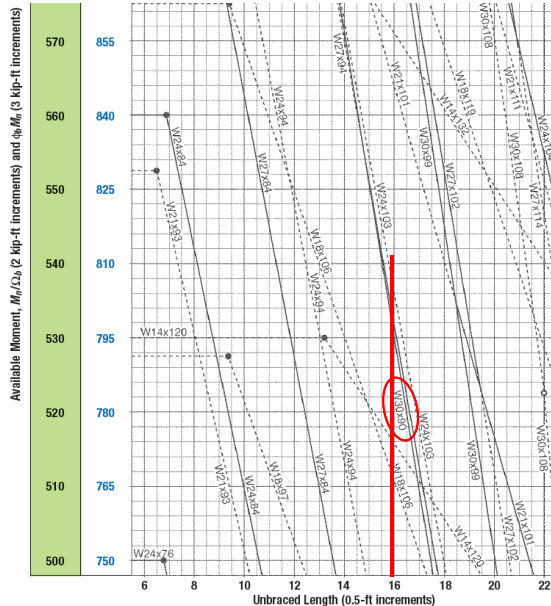


$$\phi M_o = 643 + \frac{5}{14}(420) \approx 780$$

Find chart with $\phi M_o \approx 780$ and $L_b = 16'$

43

Design Example 1 (continued)



Page 3-118:

$L_b = 16'$

$\phi M_o = 795$ k'

Considering Moment Gradient:

$\phi M_n = 1.67 \times 795$ k'

$\phi M_n = 1328$ k' > 1060 k' = ϕM_p

$\phi M_n = 1060$ k' = $\phi M_p > 1000$ k'

OK

How much does self-weight add?

$$\frac{1.2(90 \text{ lbs/ft})(32)^2}{8000} = 14 \text{ k'}$$

No Problem – use W30x90
(check shear and LL deflections)

44

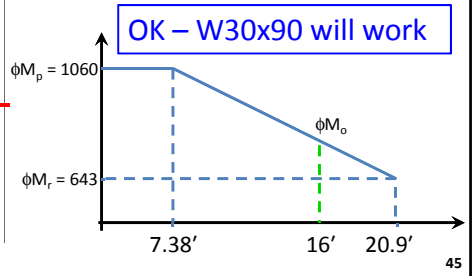


Other Solutions to Design Example 1

$F_y = 50 \text{ ksi}$
Table 3-2 (continued)
W-Shapes
Selection by Z_x

Shape	Z_x in. ²	M_p/F_y		$\phi_b M_p$		M_u/F_y		$\phi_b M_u$		$B/F A_b$	$\phi_b B F$	L_p ft	L_r ft	I_x in. ⁴	M_u/F_y		$\phi_b M_u$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD						ASD	LRFD		
W30x116	378	943	1420	575	864	24.6	37.4	7.74	22.6	4930	339	599						
W21x147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3630	318	477						
W24x131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445						
W18x158	356	888	1340	541	814	10.5	15.9	9.68	42.8	3060	319	479						
W14x193	355	886	1330	541	814	5.30	7.93	14.3	79.4	2400	276	414						
W12x210	348	868	1310	510	767	4.25	6.45	11.6	95.8	2140	347	520						
W30x108	346	863	1300	522	785	23.5	35.5	7.59	22.1	4470	325	487						
W27x114	343	856	1290	522	785	21.7	32.8	7.70	23.1	4080	311	467						
W21x132	333	831	1250	515	774	13.2	19.9	10.3	34.2	3220	283	425						
W24x117	327	816	1230	508	764	15.4	23.3	10.4	30.4	3540	297	401						
W18x143	322	803	1210	493	740	10.3	15.7	9.61	39.6	2750	285	427						
W14x176	320	798	1200	491	738	5.20	7.83	14.2	73.2	2140	252	378						
W30x99	312	778	1170	470	706	22.2	33.4	7.42	21.3	3990	309	463						
W12x190	311	776	1170	459	690	4.18	6.33	11.5	67.3	1890	305	458						
W21x122	307	766	1150	477	717	12.9	19.3	10.3	32.7	2960	260	391						
W27x102	305	761	1140	466	701	20.1	29.8	7.59	22.3	3620	279	419						
W18x130	290	724	1090	447	672	10.2	15.4	9.54	36.6	2460	259	388						
W24x104	289	721	1080	451	677	14.3	21.3	10.3	29.2	3100	241	362						
W14x159	287	716	1080	444	667	5.17	7.85	14.1	66.7	1900	224	335						
W30x90	283	706	1060	428	643	20.6	30.8	7.38	20.9	3610	249	374						
W24x103	280	699	1050	428	643	18.2	27.4	7.03	21.9	3000	270	404						
W21x111	279	696	1050	435	654	12.4	18.9	10.2	31.2	2670	237	355						
W27x94	278	694	1040	424	638	19.1	28.5	7.49	21.6	3270	264	395						
W12x170	275	686	1030	410	617	4.11	6.15	11.4	78.5	1650	269	403						
W18x119	262	654	983	403	606	10.1	15.2	9.50	34.3	2190	249	373						
W14x145	260	649	975	405	609	5.13	7.69	14.1	61.7	1710	201	302						
W24x94	254	634	953	388	583	17.3	26.0	6.99	21.2	2700	250	375						
W21x101	253	631	949	396	596	11.8	17.7	10.2	30.1	2420	214	321						
W27x84	244	609	915	372	559	17.6	26.4	7.31	20.8	2850	246	368						
W12x152	243	606	911	365	549	4.06	6.10	11.3	70.6	1430	238	358						

Pg. 3-23 of AISC Manual
There are other approaches that can also be used to evaluate LTB – some of which may be much quicker.
Since $L_b = 16' < 20.9' = L_r$, we can conservatively check if $C_b \phi M_r > M_u$. If it is, clearly $C_b \phi M_o > M_u$.
For our problem:
 $C_b \phi M_r = 1.67 \times 643$
 $C_b \phi M_r = 1074 \text{ k}' > 1060 \text{ k}' = \phi M_p$.
 $\phi M_n = \phi M_p = 1060 \text{ k}' > 1000 \text{ k}' = M_u$.



Other Solutions to Design Example 1

Of course we can always use the equations from Chapter F of the specification!!!

We have a doubly symmetric beam that is compact so Section F2 Provisions Apply (Page 16.1-47).

2. Lateral-Torsional Buckling

- (a) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.
- (b) When $L_p < L_b \leq L_r$

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

Since $L_b = 16' < 20.9' = L_r$ (in the interest of time, we are “cheating” and using the values directly from the Z_x tables).

From Z_x Tables: $\phi M_p = 1060 \text{ k}'$, $\phi M_r = 643 \text{ k}'$, $L_p = 7.38'$, and $L_r = 20.9'$



Other Solutions to Design Example 1

Specification Equations (applying ϕ -factor throughout):

Try the W30x90: $\phi M_p = 1060 \text{ k}'$, $\phi M_r = 643 \text{ k}'$,
 $L_p = 7.38'$, and $L_r = 20.9'$

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

$$\phi M_n = 1.67 \left[1060 - (1060 - 643) \frac{(16 - 7.38)}{(20.9 - 7.38)} \right] = 1326 \text{ k}' > 1060 \text{ k}' = \phi M_p$$

Therefore: $\phi M_n = 1060 \text{ k}' = \phi M_p > 1000 \text{ k}' = M_u$

As before, the W30x90 works.

47

Survey Question

For a beam bent about the strong axis (X-axis), the unbraced length of a beam (L_b) cannot exceed L_p in order to reach the plastic moment capacity ($M_p = Z_x F_y$):

- A) True
- B) False

48



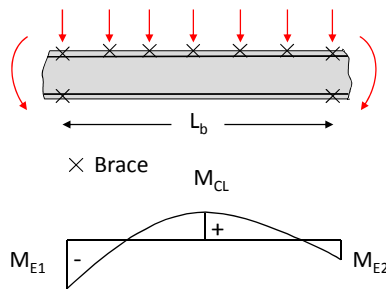
Beams Braced on One Flange With Reverse Curvature Bending

The C_b expression that is included in Section F1 of the AISC Specification is intended for beams buckling between the braced points (points with twist prevented OR lateral movement of the compression flange).

In many common situations, continuous or restrained beams may have lateral bracing on one flange; however because the beam is subjected to compression in both flanges, evaluation of the buckling resistance is not clear.

49

Moment Gradient Factors with Top Flange Continuously Braced (Rolled Sections)



- 1) If neither end moment causes compression in the bottom flange, there is not a buckling problem.
- 2) When one or both end moments cause compression in the bottom flange, use C_b with L_b .

M_{E1} = End moment that causes largest compressive stress in bottom flange

M_{E2} = Other End Moment

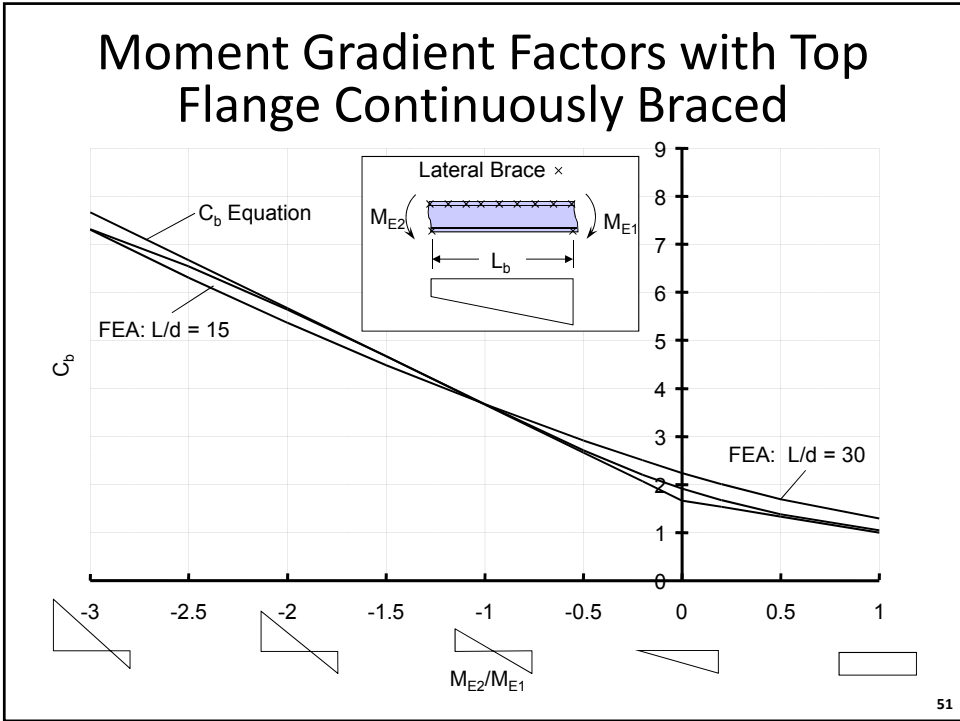
M_{CL} = Midspan Moment

$$C_b = 3.0 - \frac{2}{3} \left(\frac{M_{E2}}{M_{E1}} \right) - \frac{8}{3} \frac{M_{CL}}{(M_{E1} + M_{E2})^*}$$

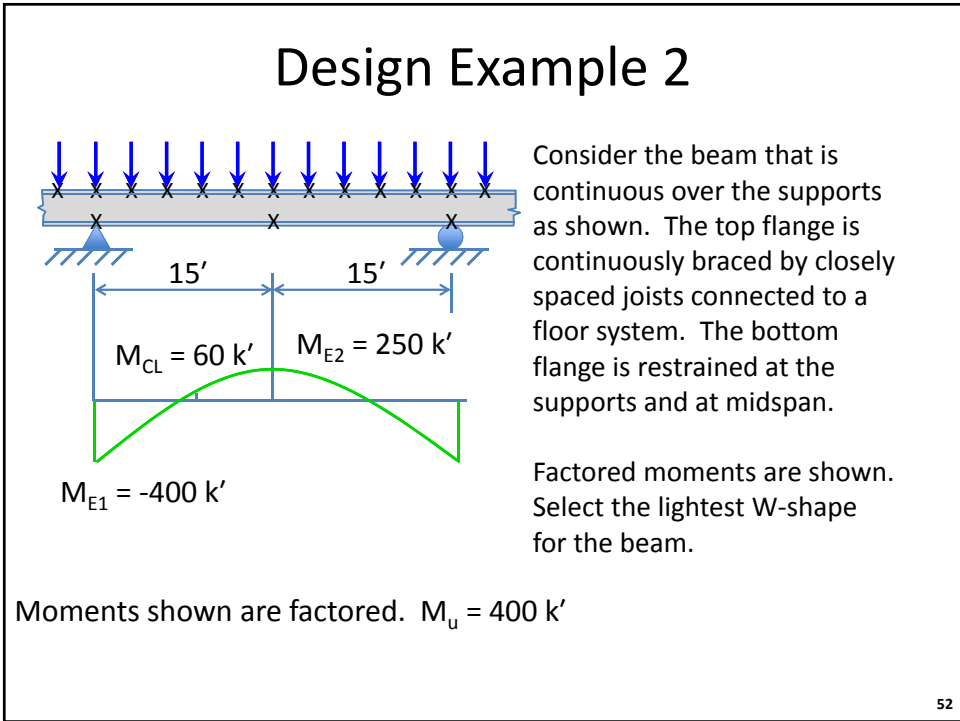
*Take $M_{E2}=0$ in this term if M_{E2} is positive

This equation is in Chapter F Commentary
 Eqn. C-F1-5 on Page 16.1-305.

50



51



52

Design Example 2 (Continued)

Look in Z_x tables to find lightest shape with $\phi M_p > 400 \text{ k}' = M_u$

Pg. 3-25

Shape	Z_x in. ³	M_{px}/Ω_b		$\phi_b M_{px}$		M_{rx}/Ω_b		$\phi_b M_{rx}$		BF/Ω_b		$\phi_b BF$		L_p ft	L_r ft	I_x in. ⁴	V_{nx}/Ω_v		$\phi_v V_{nx}$	
		kip-ft	LRFD	kip-ft	LRFD	kip-ft	LRFD	ASD	LRFD	ASD	LRFD	kip	LRFD				kip	LRFD		
W21x55	126	314	473	192	289	10.8	16.3	6.11	17.4	1140	156	234								
W14x74	126	314	473	196	294	5.31	8.05	8.76	31.0	795	128	192								
W18x60	123	307	461	189	284	9.62	14.4	5.93	18.2	984	151	227								
W12x79	119	297	446	187	281	3.78	5.67	10.8	39.9	662	117	175								
W14x68	115	287	431	180	270	5.19	7.81	8.69	29.3	722	116	174								
W10x88	113	282	424	172	259	2.62	3.94	9.29	51.2	534	131	196								
W18x55	112	279	420	172	258	9.15	13.8	5.90	17.6	890	141	212								
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 ^f	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								
W14x61	102	254	383	161	242	4.93	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								
W10x77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169								
W12x65 ^f	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142								

Try the W21x50 with $\phi M_p = 413 \text{ k}' > 400 \text{ k}' = M_u$
 $L_r = 13.6' < 15' = L_b$ – Elastic LTB

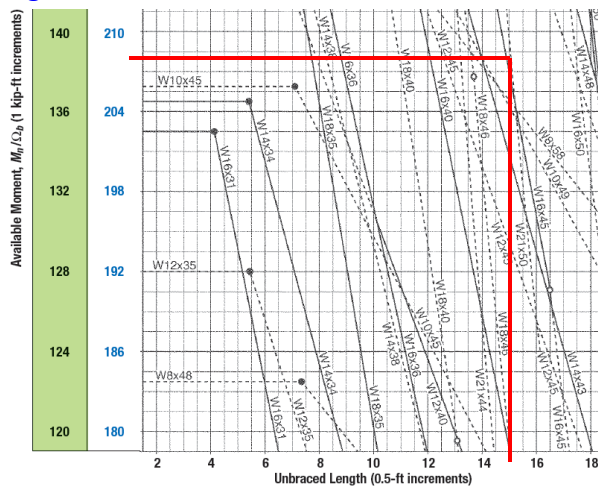
$\phi M_o < 248 \text{ k}' = \phi M_r$ Look for page with $\phi M_o \sim 200 \text{ k}'$ at $L_b = 15'$

53

Design Example 2 (Continued)

$\phi M_o < 248 \text{ k}' = \phi M_r$ Look for page with $\phi M_o \sim 200 \text{ k}'$ at $L_b = 15'$

Page 3-128



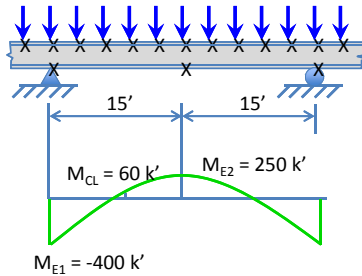
From Chart:
 $\phi M_o = 208 \text{ k}'$

Still Need to evaluate C_b considering top flange continuously braced and $L_b = 15'$.

54

Design Example 2 (Continued)

From Chart: $\phi M_o = 208 \text{ k}'$



$$C_b = 3.0 - \frac{2}{3} \left(\frac{M_{E2}}{M_{E1}} \right) - \frac{8}{3} \frac{M_{CL}}{(M_{E1} + M_{E2})^*}$$

*Take $M_{E2} = 0$ in this term if M_{E2} is positive

$$C_b = 3.0 - \frac{2}{3} \left(\frac{+250}{-400} \right) - \frac{8}{3} \frac{+60}{(-400 + 0)^*}$$

$$C_b = 3.82$$

$$C_b \phi M_o = 3.82(208) = 795 \text{ k}' > \phi M_p = 413 \text{ k}'$$

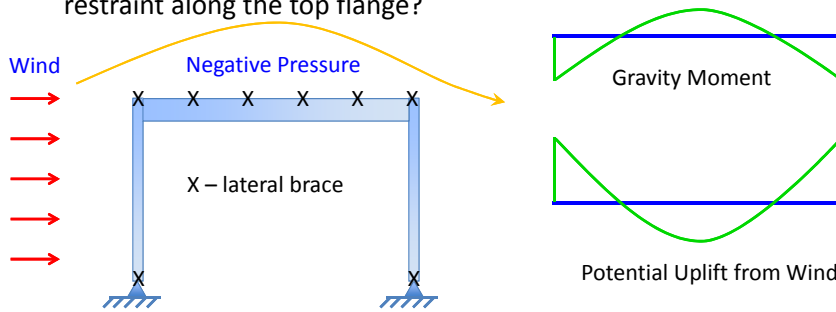
$$\phi M_n = \phi M_p = 413 \text{ k}' > 400 \text{ k}'$$

OK - Use W21x50
 (Check Shear and LL Deflections)

55

Potential for Uplift on Beams in Roofing Systems

The efficiency of steel structures often lead to relatively light structures. Negative pressure from wind loads can cause uplift on roofing beams that can overcome the gravity load moments and lead to compression in the bottom flange. Therefore buckling of the bottom flange needs to be considered, but how to account for the restraint along the top flange?



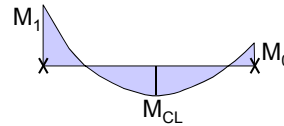
If the negative pressure doesn't cause compression in the bottom flange there is no beam buckling issue since top flange is continuously braced.

56

Moment Gradient Factors with Top Flange Continuously Braced (Uplift)

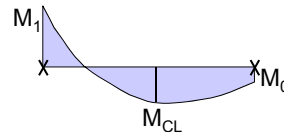
CASE A: Both end moments are positive or zero

$$C_b = 2.0 - \frac{(M_0 + 0.6M_1)}{M_{CL}}$$



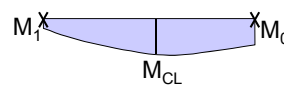
CASE B: One end moment is negative

$$C_b = \frac{(2M_1 - 2M_{CL} + 0.165M_0)}{0.5M_1 - M_{CL}}$$



CASE C: Both end moments are negative

$$C_b = 2.0 - \frac{(M_0 + M_1)}{M_{CL}} \left[0.165 + \frac{1}{3} \frac{M_1}{M_0} \right]$$

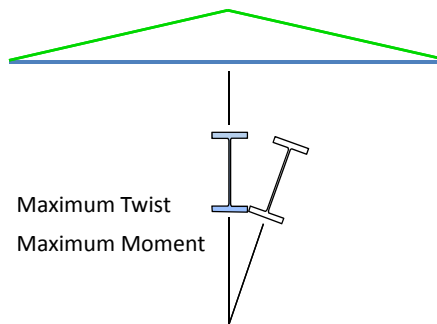
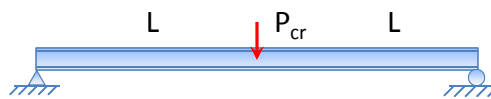
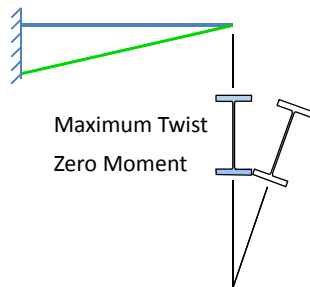


These equations are in Chapter F Commentary (Page 16.1-307).

57

Buckling of Unbraced Cantilevers

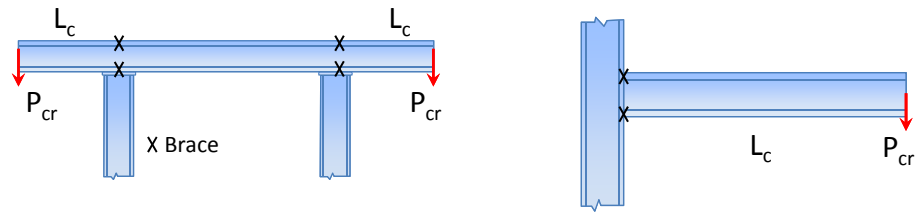
Do not use " $L_b = 2L$ " for unbraced cantilevers.



58

Buckling of Unbraced Cantilevers

Use $L_b = L_c$ for unbraced cantilevers (overhanging beams).

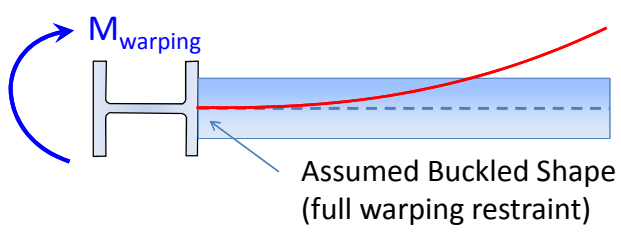
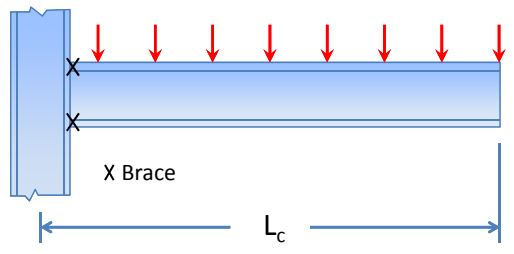


The AISC Specification (Section F1) recommends to use $C_b = 1.0$ for unbraced cantilevers. Lower Bound C_b values (from Timoshenko):

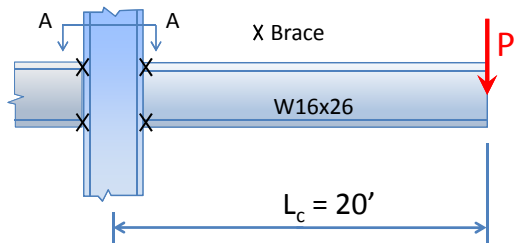


Effect of Fixed End Condition

Assumption when using $L_b = L_c$ for unbraced cantilevers:

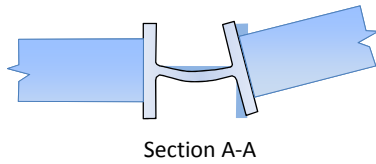


Effect of Fixed End Condition



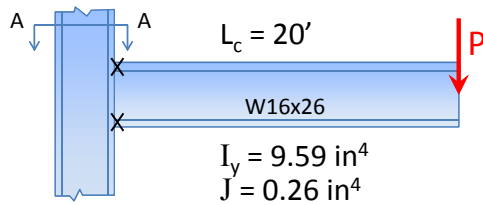
The results tabulated for P_{cr} below were determined from BASP – Buckling Analysis of Stiffened Plates

Column Size	P_{cr}
"0"	2.11 min
W8x24	2.18
W14x211	3.08
Fixed	3.61 max



61

Approximation for Unbraced Cantilevers



Neglect the warping term in the calculation of the buckling capacity

$$M_{cr} = C_b \frac{\pi}{L_b} \sqrt{EI_y GJ}$$

$$M_{cr} = C_b \frac{\pi}{L_b} \sqrt{EI_y GJ} = 475 \text{ k"}$$

\swarrow 1.28 \swarrow 240 in.

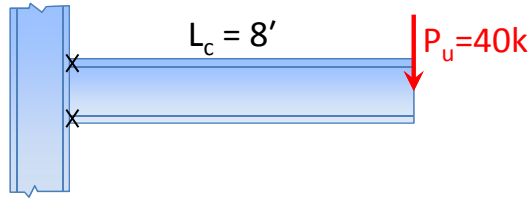
$$P_{cr} = (475 \text{ k"}) / 240" = \underline{\underline{1.98 \text{ k}}}$$

Note: If warping term is not neglected, $P_{cr} = 2.78 \text{ k}$

Column Size	P_{cr}
"0"	2.11 min
W8x24	2.18
W14x211	3.08
Fixed	3.61 max

62

Design Example 3 (Cantilever in Inelastic Buckling Range)



Consider the 8' long cantilever with a 40 kip load at tip. Tip is unbraced. Find lightest W-shape.



$$M_u = (40k) \times 8' = 320 \text{ k'}$$

Check the Z_x Tables to find lightest W-shape with $\phi M_p > 320 \text{ k'}$

Design Example 3 (Cantilever in Inelastic Buckling Range)

Page: 3-25

$F_y = 50 \text{ ksi}$
Table 3-2 (continued)
W-Shapes
Selection by Z_x

Z_x

Shape	Z_x in. ³	M_p/Ω_b		$\phi_p M_{px}$		M_{rx}/Ω_b		$\phi_p M_{rx}$		BF/Ω_b		$\phi_p BF$		L_p ft	L_r ft	I_x in. ⁴	V_{nx}/Ω_v		$\phi_p V_{nx}$	
		kip-ft	ASD	kip-ft	LRFD	kip-ft	ASD	kip-ft	LRFD	ASD	LRFD	kip	LRFD				kip	LRFD		
W21x55	126	314	473	192	289	10.8	16.3	6.11	17.4	1140	156	234								
W14x74	126	314	473	196	294	5.31	8.05	8.76	31.0	795	128	192								
W18x60	123	307	461	189	284	9.62	14.4	5.93	18.2	984	151	227								
W12x79	119	297	446	187	281	3.78	5.67	10.8	39.9	662	117	175								
W14x68	115	287	431	180	270	5.19	7.81	8.69	29.3	722	116	174								
W10x88	113	282	424	172	259	2.62	3.94	9.29	51.2	534	131	196								
W18x55	112	279	420	172	258	9.15	13.8	5.90	17.6	890	141	212								
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	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								
W14x61	102	254	383	161	242	4.93	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								
W10x77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169								
W12x65	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142								
W21x44	95.4	238	358	143	214	11.1	16.8	4.45	13.0	843	145	217								
W16x50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186								
W18x46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195								
W14x53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154								

The W21x44 has $\phi M_p = 358 \text{ k'}$ $> 320 \text{ k'}$

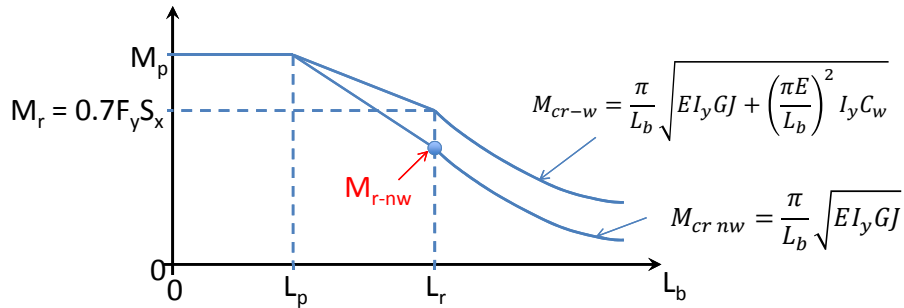
Check LTB:

$$L_r = 13' > 8' = L_b$$

This section is in the inelastic range. If we were to use the beam charts, they include the warping stiffness. So how can we check LTB without including the warping term?

Design Example 3 (Cantilever in Inelastic Buckling Range)

We need to find M_{r-nw} (neglecting warping)



Since the buckling curve in the inelastic range is a straight line, we just need to figure out M_r from the elastic equation, neglecting warping. This can be found by using the $M_{cr\ nw}$ equation above and just inserting $L_b = L_r$.

65

Design Example 3 (Cantilever in Inelastic Buckling Range)

W21 x 44: $I_y = 20.7 \text{ in}^4$, $J = 0.77 \text{ in}^4$, $L_r = 13'$, $L_p = 4.45'$, $\phi M_p = 358 \text{ k}'$

$$\phi M_{r\ nw} = 0.9 \frac{\pi}{L_r} \sqrt{EI_y GJ}$$

$$\phi M_{r\ nw} = 0.9 \left(\frac{\pi}{13 \times 12} \right) \sqrt{29000 \times 20.7 \times 11200 \times 0.77} = 1304 \text{ k}'' = 109 \text{ k}'$$

Use Inelastic Buckling Equation (F2-2) Page 16.1-47:

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

$$\phi M_n = 1.28 \left[358 - (358 - 109) \frac{(8 - 4.45)}{(13 - 4.45)} \right] = 326 \text{ k}' > 320 \text{ k}' \text{ OK}$$

Lower Bound C_b from Timoshenko.

Check LL Deflections and Shear

OK – Use W21 x 44

66

Built-Up Sections

The AISC Manual provides a number of tables and charts that allow the designer to quickly select an economical rolled section to satisfy the basic design requirements. In many instances, a designer may opt to design a built-up flexural member. Two of the primary reasons that an engineer may decide to design a built up section include the following:

- The necessary clear span is larger than what can be efficiently spanned with rolled shapes. For example, considering some approximate maximum desirable span/depth for W44 shapes (deflections become an issue):
 Simple Supports: 90'~100' Continuous: 115'~130'
- The established geometry of rolled sections often may result in relatively inefficient use of the material.

67

Built-Up Sections

From the perspective of efficiency, rolled wide flange shapes have relatively stocky webs. Recall that the most slender-webbed wide flange shape is a W30x90 that has an $h/t_w = 57.5$.

For Grade 50 Steel, recall the following web slenderness limits from Table B4.1b on Page 16.1-17:

	$F_y = 36 \text{ ksi}$	$F_y = 50 \text{ ksi}$	Rolled Shapes
$\lambda_{pw} = 3.76 \sqrt{\frac{E}{F_y}} = \frac{640}{\sqrt{F_y}}$	106.7	90.5	$\lambda = \frac{h}{t_w} < 57.5$
$\lambda_{rw} = 5.70 \sqrt{\frac{E}{F_y}} = \frac{970}{\sqrt{F_y}}$	161.7	137	

68



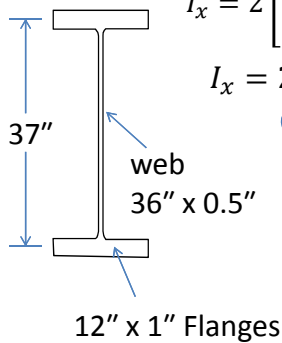
Built-Up Sections

Consider the performance of the material in the cross section of an I-shape from the perspective of the moment of inertia:

$$I_x = \sum (I_o + Ad^2)$$

$$I_x = 2 \left[\left(\frac{1}{12} (12'')(1'')^3 \right) + (12in^2) \left(\frac{37}{2} \right)^2 \right] + \frac{1}{12} (0.5)(36)^3$$

$$I_x = 2[(1in^4) + (4107in^4)] + 1944in^4 = 10160in^4$$



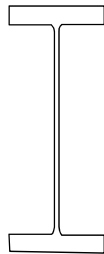
Flange Web

For this numerical example, the flange Ad^2 term contributes about 80% of the total moment of inertia and the web contributes the other 20%.

69

Built-Up Sections

While the moment of inertia is more related to the “flexural stiffness” of the cross-sectional shape, the impact on the “flexural strength” is similar. Most of the bending strength comes from the flanges. The web primarily carries the shear. Therefore, adding extra material to the web, is relatively inefficient at improving the flexural strength.



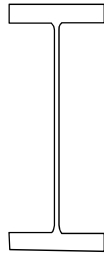
If we wanted to be able to reach the plastic bending capacity, M_p , from an efficiency perspective we would do better to have a web slenderness closer to λ_p .

A significant amount of the material is in the girder web. We often may be better off to size the section with a more slender web ($\lambda \geq \lambda_p$) resulting in a moment capacity that may be below M_p .

70

Preliminary Sizing of Built-Up Sections

One of the nice aspects with built-up sections is that we have a great deal of freedom in proportioning the section; however we often may not have a feel for what is efficient.

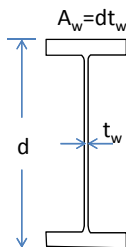


From an efficiency perspective, we often want to design the web to be able to “just” support applied shear and the size the flanges to support the bending moment working with the web. In many situations, the resulting web is relatively slender, resulting in a moment capacity that is often closer to the yield moment, M_y .

The resulting web is often in the elastic shear buckling range.

71

Preliminary Sizing of Built-Up Sections



In AISC: Nominal Shear Strength = $V_n = 0.6F_y A_w C_v$

$$C_v = \frac{\tau_{cr}}{\tau_y}$$

Eqn. G2-1,
Pg. 16.1-67

Elastic Shear Buckling: ($h/t_w > 74$ for unstiffened web and Gr. 50)

$$\tau_{cr} = \frac{V_{cr}}{A_w} = \frac{\pi^2 E k_v}{12(1 - \nu^2) \left(\frac{h}{t_w} \right)^2} \quad \nu = \text{Poisson's Ratio} = 0.3$$

$$\tau_y = 0.6F_y$$

$$C_v = \frac{\tau_{cr}}{\tau_y} = \frac{\pi^2 E k_v}{12(1 - \nu^2) \left(\frac{h}{t_w} \right)^2 (0.6F_y)} = \frac{1.51 E k_v}{\left(\frac{h}{t_w} \right)^2 F_y}$$

Eqn. G2-5,
Pg. 16.1-68

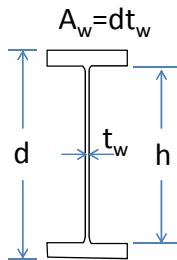
72

Preliminary Sizing of Built-Up Sections

Most economical built-up shapes have non-compact or slender webs, which puts them in the elastic shear buckling region.

Elastic Shear Buckling Resistance (unstiffened web):

$$V_u \leq \phi V_n = 0.9 \times 0.6 F_y A_w C_v = 0.54 F_y (d t_w) \left(\frac{1.51 E k_v}{\left(\frac{h}{t_w} \right)^2 F_y} \right)$$



$$\begin{aligned} d &\approx h \\ k_v &= 5 \\ E &= 29000 \text{ ksi} \end{aligned}$$

$$V_u \leq \left(\frac{118,200 (t_w)^3}{h} \right)$$

$$t_w \geq \left(\frac{\sqrt[3]{V_u h}}{49.1} \right)$$

What should we use for h?

73

Preliminary Sizing of Built-Up Sections

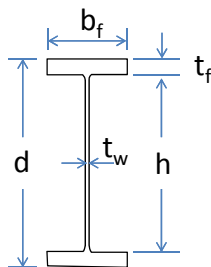
There are a number of approaches to get preliminary sizes for girder depth and flange widths.

Girder Depth ($d \approx h$):

We can start with maximum span to depth ratios (L/d):

Simply – Supported: $L/d \leq 25$

Continuous Girder: $L/d \leq 30 - 35$



Flange Width, b_f :

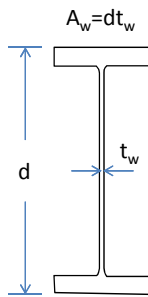
Rolled beam shapes often have $b_f \approx 0.33d$

In many cases a more practical value is probably $b_f \approx 0.25d$

74

Preliminary Sizing of Built-Up Sections

Yura developed an expression for an efficient girder depth based upon the elastic shear capacity and first yield of a doubly symmetric shape (See SSRC Guide Chapter 6). Depending on LRFD or ASD formats, a starting point for an efficient web is as follows:



ASD	LRFD
$h_{eff} = \frac{7.6 \left[\frac{\Omega M_r}{F_y} \right]^{3/7}}{(\Omega V_r)^{1/7}}$	$h_{eff} = \frac{7.6 \left[\frac{M_r}{\phi F_y} \right]^{3/7}}{(V_r)^{1/7}}$
$t_w = \frac{(\Omega V_r)^{1/3} (h_{eff})^{1/3}}{50.3}$	$t_w = \frac{(V_r)^{1/3} (h_{eff})^{1/3}}{49.1}$

The slightly different constants on the web thickness expressions are due to a $\phi=0.9$ for LRFD

Approximate Girder Weight: (Units: wt./ft. - lbs./ft; M_r : k-in, F_y : ksi)

$$\text{ASD Wt./ft} = \frac{(\Omega M_r)^{2/3}}{\sqrt{F_y}}$$

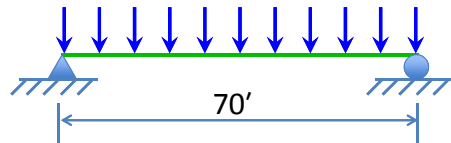
$$\text{LRFD Wt./ft} = \frac{(M_r)^{2/3}}{\sqrt{F_y}}$$

75

Preliminary Sizing of Built-Up Sections

In reality, the economy of the girder is highly dependent on the plate thickness values that we can get in practice. For example, we may start with a specific "h" and solve for the required web thickness based upon the maximum shear.

LRFD Example: $w_u = 3.6$ k/ft



$$M_r = 2200 \text{ k-ft}, V_r = 126 \text{ k}$$

$$\text{Gr. 50 Steel: } F_y = 50 \text{ ksi}$$

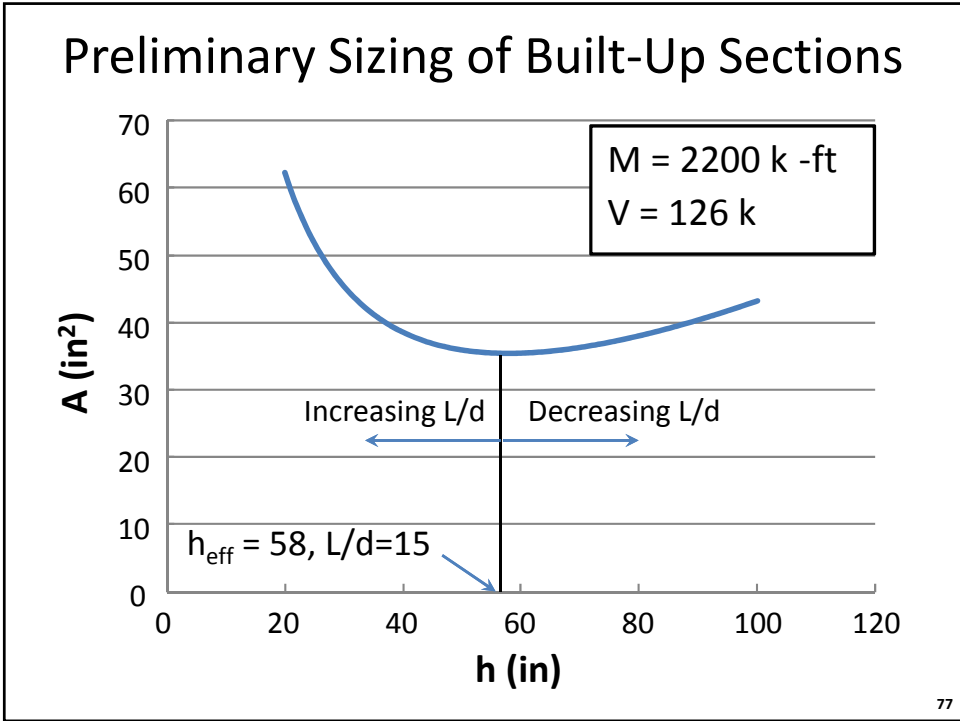
$$h_{eff} = \frac{7.6 \left[\frac{M_r}{\phi F_y} \right]^{3/7}}{(V_r)^{1/7}} = 58 \text{ in.}$$

$$t_w = \frac{(V_r)^{1/3} (h_{eff})^{1/3}}{49.1} = 0.39''$$

$t_w = 0.39''$ is not a realistic thickness. We have to select between $3/8 = 0.375''$ and $7/16 = 0.432''$.

As we change t_w the required h will also change.

76



LRFD Preliminary Sizing Example

$M_r = 2200 \text{ k-ft}, V_r = 126 \text{ k}$
 $F_y = 50 \text{ ksi}$

If we pick $t_w = 0.375''$, the limiting depth for elastic shear buckling changes. Our web will have exactly the adequate shear strength. Note: We tend to think of shear design in terms of web shear yielding where we have $\phi 0.6F_y A_{\text{web}}$ and a thinner web requires a deeper web to maintain A_{web} . However, because this is a buckling problem a thinner web requires a more shallow section :

$$h = \left(\frac{118,200(t_w)^3}{V_r} \right) = \left(\frac{118,200(0.375'')^3}{126 \text{ k}} \right) = 50'' \quad (L/d = 17)$$

Or Try $L/d \approx 25$: $h = 28''$ (it's actually $L/d \approx 30$ to get a practical thickness)

$$t_w = \frac{(126 \text{ k})^{1/3}(28'')^{1/3}}{49.1} = 0.31'' \quad \text{Try } 5/16'' = 0.3125''$$

78

LRFD Preliminary Sizing Example

Try both girders to get an indication of the difference in Efficiency ($M_r = 2200 \text{ k-ft} = 26400 \text{ k-in}$):

Girder	Web	h/t	Compact?	Elast. Shear?	ϕV_n	A_{web}
A	3/8" x 48"	133	<90.5? No	>74? Yes	130 k	18 in ²
B	5/16" x 28"	93	<90.5? No	>74? Yes	129 k	8.8 in ²

Girder A:

Size the Flanges: (Non-compact girder: $\lambda_p = 90.5 < 133 < 137 = \lambda_r$)

$$A_f = \frac{M_r}{\phi F_y h} - \frac{A_w}{4} + \frac{A_w}{12} \left[\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right] = 9.1 \text{ in}^2$$

Rearranged Eqs.
F4-1 and F4-9b
Pgs. 16.1-50,51

Use 12" x 0.75" flanges - $\phi M_n = 2170 \text{ k-ft} \approx 2200 \text{ k-ft}$.

Compact Flange (Close enough for this example)

Total Area = 36 in² - Section weighs 123 lbs/ft

79

LRFD Preliminary Sizing Example

Girder	Web	Flange	Total Area	Lbs/ft	I_x (in ⁴)
A	3/8" x 48"	12" x 0.75"	36 in ²	123	14150
B	5/16" x 28"	18" x 1.062"	47 in ²	160	8650

- If depth is not a limitation – clearly the deeper girder is much more efficient (Girder B is 30% heavier). Girder B is also more likely to be controlled by LL Serviceability ($I_{x \text{ Gir. B}} \ll I_{x \text{ Gir. A}}$).
- Although Section B has more steel – how does it compare to a rolled section? From Z-tables, a W30x173 has $\phi M_p = 2270 \text{ k-ft}$, which is a little over 8% heavier.
- For a 70 ft. span, we would probably typically use a rolled section due to simplified fabrication. However for longer spans or cases with special demands, a built-up section will often be necessary and we can be more efficient.

80

Summary

- The AISC Manual has a number of very useful tables and charts that provide an efficient means of evaluating the buckling capacity of beams.
- There are a number of “methods” that you can use to efficiently evaluate the beams buckling capacity. Understanding the shape of the curve of the buckling strength versus the unbraced length is important in order to efficiently apply the different methods as well as understanding when a given method may not be applicable.
- Moment gradient can result in a substantial increase in the lateral-torsional buckling strength. There are a number of useful C_b expressions provided in the AISC Commentary that can be used to account for various bracing and load cases.

81

Summary

- The unbraced length for cantilevers with bracing only at the support should be taken as the length of the cantilever. Moment gradient factors for unbraced cantilevers were presented.
- Using the actual length of the cantilever essentially assumes that full warping restraint will be achieved at the support. Distortion in the supporting column or warping deformation in overhanging beams can substantially reduce the buckling capacity of the beam. In these cases, the warping term should be neglected.
- Some preliminary sizing expressions were briefly discussed that can be used in the design of built-up sections.

82



Up Next...

- Session 5: July 17
Stability of Structural Systems
by R. Ziemian, PE, PhD
- This lecture will begin with a review of basic concepts related to the stability of structural systems. With an eye towards design, the difference between a bifurcation or critical load analysis and the loss in stiffness due to second-order effects and material yielding, as the maximum resistance of physical structures is approached, will be emphasized. The lecture will conclude with an overview of the direct analysis and effective length methods.

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Course Resources

Event	Start Date
NS 13 8-Session Package-Night School 13 - Design of Industrial Buildings	1/30/2017 7:00:00 PM
NS 14 8-Session Package-Night School 14 - Fundamentals of Stability	6/5/2017 7:00:00 PM

Night School Resources for 8-session package Registrants



Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	View Passcode: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 03/02/2017 5pm EST	Available 03/02/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longitudinal Bldg Bracing Dn	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending
NS13 - Final Exam	4/10/2017 7:00:00 PM			Available 04/12/2017 5pm EST	

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- Weekly “quiz and recording” email.
- Weekly updates of the master Quiz and Attendance record found at www.aisc.org/nightschool. Scroll down to Quiz and Attendance records.
 - Updated on Tuesday mornings.



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