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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.

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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.

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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

structural STEEL

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Fundamentals of Stability for Steel Design

Session 4
Design of Flexural Members

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

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Structural Stability Research Council

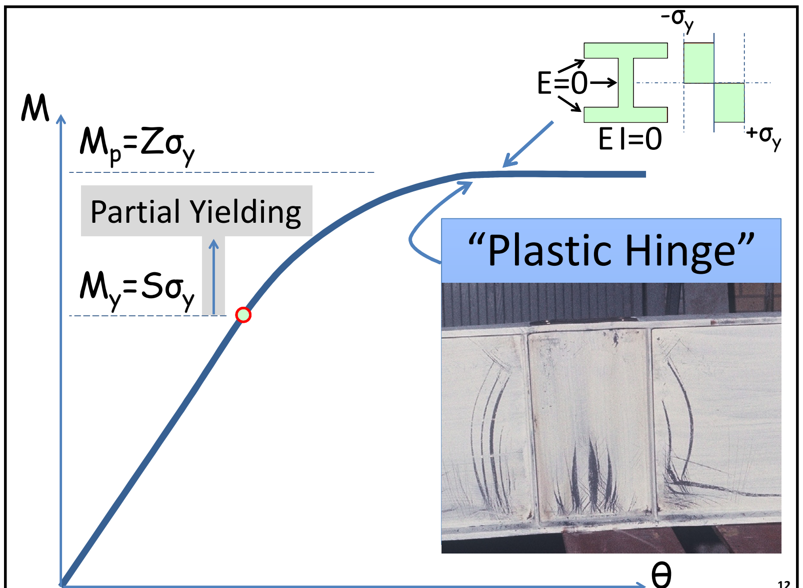
- SSRC values being a part of the AISC Night School series. Many thanks to Brent Leu, Christina Harber, and Nancy Gavlin for all the work they do with the speakers to develop a successful program.
- SSRC is an organization consisting of educators, practicing engineers, and industry professionals interested 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 :

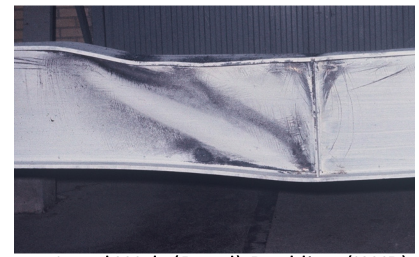
- Local Web Buckling (LWB)
- Local Flange Buckling (LFB)
- Lateral Torsional Buckling (LTB)

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)



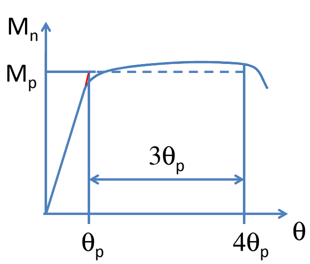
Local Web (Bend) Buckling (LWB)

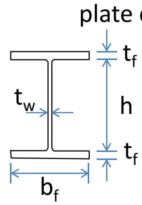
14

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: $\lambda_f = b_f/2t_f$

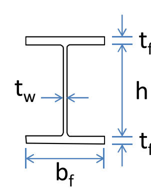
The compact section requirements only address local buckling modes. The compact section requirements have nothing to do with lateral torsional buckling (LTB).

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

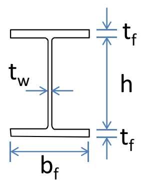
Consider the webs of all rolled wide flange shapes:

$$Web: \lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}} = \frac{640}{\sqrt{F_y}}$$

US Units

For Grade 50 Steel, λ_{pw} =90.5

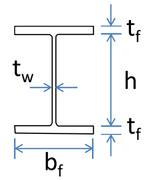
The rolled W-shape with the most slender web is a W30x90 with h/t_w = 57.5<<90.5=λ_{pw}.
Note: The W30x90 would need to have F_y = 124 ksi to have a non-compact web



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

Compact Section Requirements

Considering flanges of all rolled wide flange shapes:



$$Flange: \lambda_{pf} = {}^{b_f}\!/_{2t_f} \leq 0.38\sqrt{\frac{E}{F_y}} = \frac{65}{\sqrt{F_y}}$$

US Units

For Grade 50 Steel, λ_{pw} =9.2

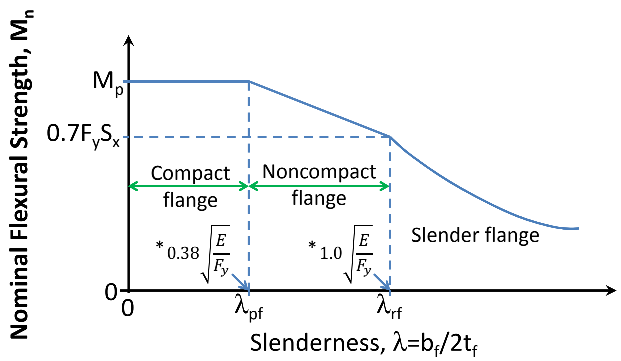
11 sections do not have compact flanges (F_y = 50 ksi).

Consider User Note: Section F2 - Page 16.1-47

User Note: All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5 and M4×6 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.

Effect of Local Flange Buckling on Beam Strength



*From Table B4.1b

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

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	M_{px}/Ω_b	ϕ_bM_{px}	M_{rx}/Ω_b	ϕ_bM_{rx}	BF/Ω_b	ϕ_bBF	L_p	L_r	I_x	V_{nx}/Ω_v	ϕ_vV_{nx}
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	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[f]	173	430	646	274	412	4.91	7.36	13.5	45.3	1110	138	207

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
[v] 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.

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

Z_x

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

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W24×84	**224**	**559**	**840**	**342**	**515**	**16.2**	**24.2**	**6.89**	**20.3**	**2370**	**227**	**340**
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ASD	LRFD
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[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
[v] 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.

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

Z_x = 173 in^3

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

$$\phi M_p = \frac{0.9x173x50ksi}{12\,{}^{in}/_{ft}}$$

$$\phi M_p = 649\ k - ft$$

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

21

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)

22

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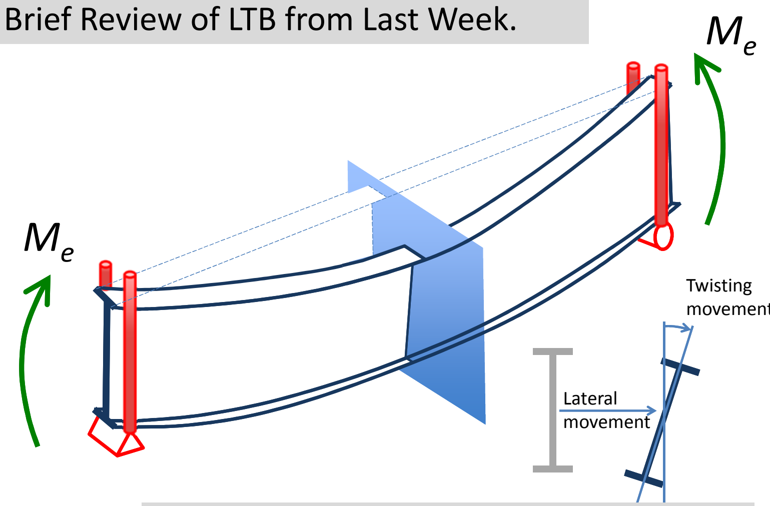
C) The section is not likely to fail due to local flange buckling (LFB)
D) (b) and (c), only
E) (a), (b), and (c)

23

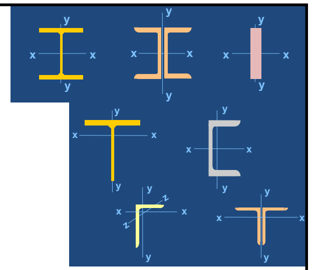
Brief Review of LTB from Last Week.



"Lateral torsional buckling (LTB)"

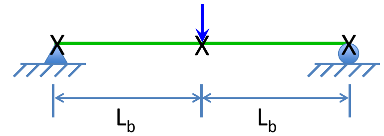
24

Member instability: Lateral Torsional Buckling



25

Definition of L_b – Unbraced Length for Beams



X - Braced Point

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)

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

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

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

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:

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

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

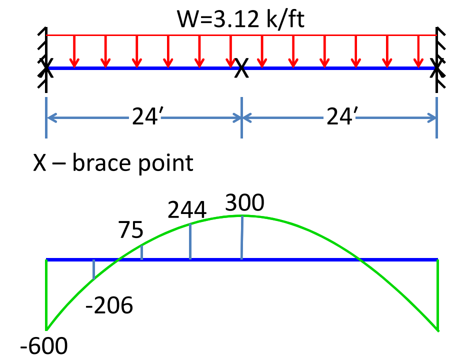
M_A = Moment at $L_b/4$

M_B = Moment at $L_b/2$

M_C = Moment at $3L_b/4$

29

Effect of Moment Gradient



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

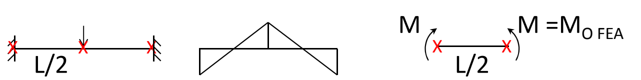
30

Calculating C_b factors using FEA programs

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

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

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

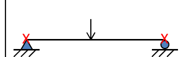
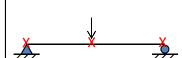
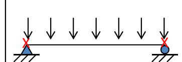
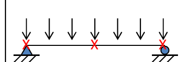


$M_{max} = P_{cr}L/8 = M_{cr\ FEA}$

X – Brace Point

31

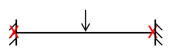
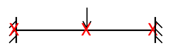
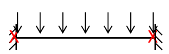
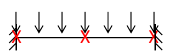
C_b – FEA versus Equation Comparison

	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

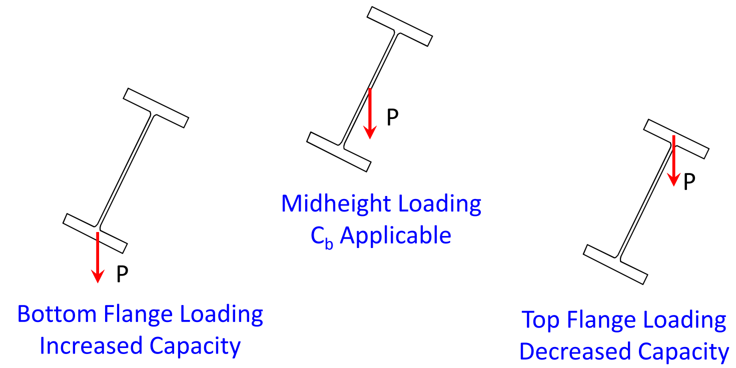
	BASP - C_b W16x26 – L/d = 30	AISC - C_b
	1.76	1.92
	2.70	2.27
	2.66	2.38
	3.39	2.38

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

X – Brace Point

33

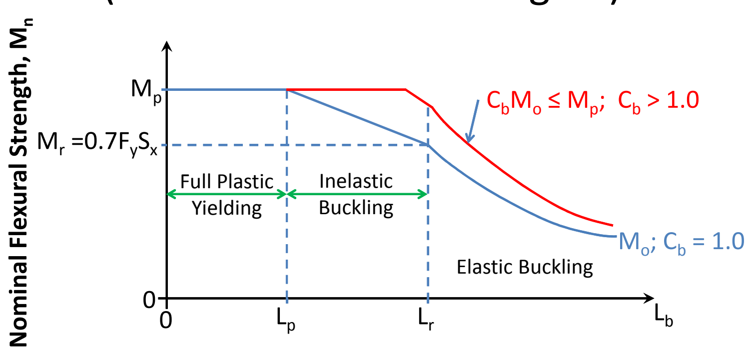
Load Position on the Cross Section



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)

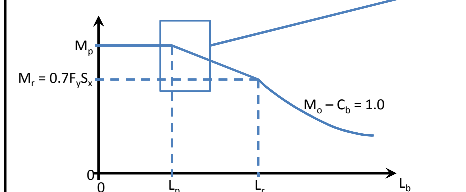


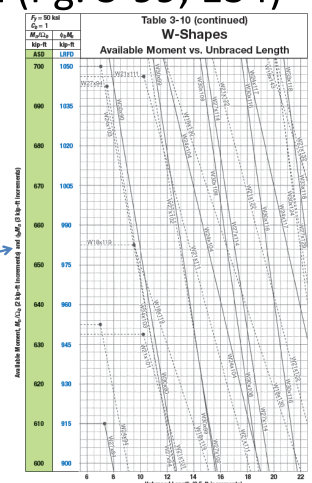
To design for LTB, it is imperative that we have a good understanding of the above curve.

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 $M_p/\Omega > M_a$)

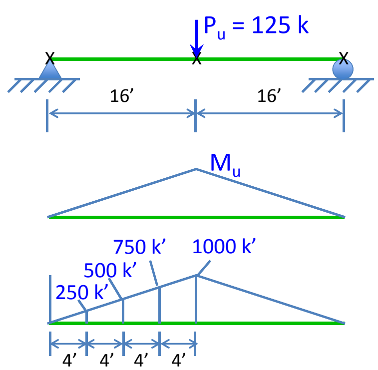
37

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.

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

Find the lightest W-shape that supports M_u=1000 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)

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

F_y = 50 ksi

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W30×90^v	**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 — Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.³	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.⁴	ASD	LRFD
W30×116	**378**	**943**	**1420**	**575**	**864**	**24.8**	**37.4**	**7.74**	**22.6**	**4930**	**339**	**509**
W21×147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3630	318	477
W24×131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445
W18×158	356	888	1340	541	814	10.5	15.9	9.68	42.8	3060	319	479
W14×193	355	886	1330	541	814	5.30	7.93	14.3	79.4	2400	276	414
W12×210	348	868	1310	510	767	4.25	6.45	11.6	95.8	2140	347	520
W30×108	**346**	**863**	**1300**	**522**	**785**	**23.5**	**35.5**	**7.59**	**22.1**	**4470**	**325**	**487**
W27×114	343	856	1290	522	785	21.7	32.8	7.70	23.1	4080	311	467
W21×132	333	831	1250	515	774	13.2	19.9	10.3	34.2	3220	283	425
W24×117	327	816	1230	508	764	15.4	23.3	10.4	30.4	3540	267	401
W18×143	322	803	1210	493	740	10.3	15.7	9.61	39.6	2750	285	427
W14×176	320	798	1200	491	738	5.20	7.83	14.2	73.2	2140	252	378
W30×99	**312**	**778**	**1170**	**470**	**706**	**22.2**	**33.4**	**7.42**	**21.3**	**3990**	**309**	**463**
W12×190	311	776	1170	459	690	4.18	6.33	11.5	87.3	1890	305	458
W21×122	307	766	1150	477	717	12.9	19.3	10.3	32.7	2960	260	391
W27×102	305	761	1140	466	701	20.1	29.8	7.59	22.3	3620	279	419
W18×130	290	724	1090	447	672	10.2	15.4	9.54	36.6	2460	259	388
W24×104	289	721	1080	451	677	14.3	21.3	10.3	29.2	3100	241	362
W14×159	287	716	1080	444	667	5.17	7.85	14.1	66.7	1900	224	335
W30×90[v]	**283**	**706**	**1060**	**428**	**643**	**20.6**	**30.8**	**7.38**	**20.9**	**3610**	**249**	**374**
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W21×111	279	696	1050	435	654	12.4	18.9	10.2	31.2	2670	237	355
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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

Pg. 3-23 of AISC Manual

M_u = 1000 k'

A W30x90 is the lightest section with ϕM_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 ϕ_v=0.9.

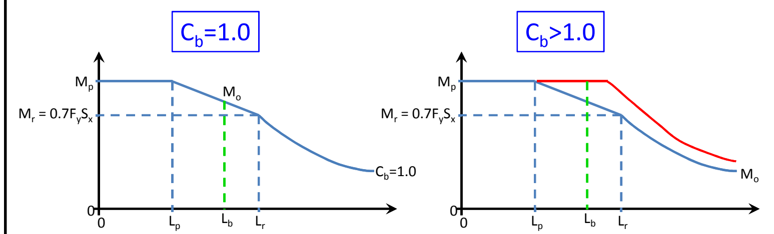
Hypothetical: What if, due to architectural limitations we had a depth limit for the beam equal to 18 inches?

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: ϕM_n= $C_b \phi M_o$= 1.67x ϕM_o > 1000 k' = M_u



42

Design Example 1 (continued)

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

F_y = 50 ksi — Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.³	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.⁴	ASD	LRFD
W30×116	**378**	**943**	**1420**	**575**	**864**	**24.8**	**37.4**	**7.74**	**22.6**	**4930**	**339**	**509**
W21×147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3630	318	477
W24×131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445
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W18×143	322	803	1210	493	740	10.3	15.7	9.61	39.6	2750	285	427
W14×176	320	798	1200	491	738	5.20	7.83	14.2	73.2	2140	252	378
W30×99	**312**	**778**	**1170**	**470**	**706**	**22.2**	**33.4**	**7.42**	**21.3**	**3990**	**309**	**463**
W12×190	311	776	1170	459	690	4.18	6.33	11.5	87.3	1890	305	458
W21×122	307	766	1150	477	717	12.9	19.3	10.3	32.7	2960	260	391
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W14×159	287	716	1080	444	667	5.17	7.85	14.1	66.7	1900	224	335
W30×90[v]	**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
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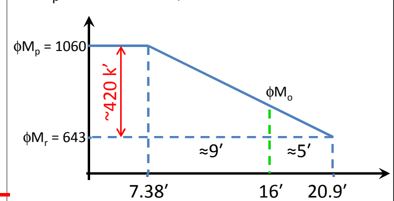
W12×152	243	606	911	365	549	4.06	6.10	11.3	70.6	1430	238	358

Pg. 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. ϕM_p=1060 k', ϕM_r=643 k', L_p=7.38', and L_r=20.9'

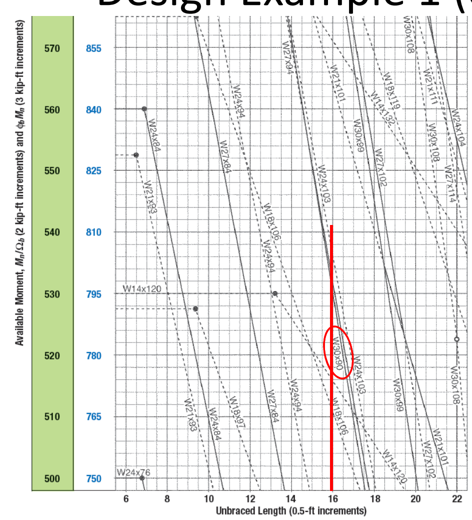


$$\phi M_0 = 643 + \frac{5}{14}(420) \approx 780$$

Find chart with ϕM_o≈780 and L_b=16'

43

Design Example 1 (continued)



Page 3-118:

L_b = 16'

ϕM_o = 795 k'

Considering Moment Gradient:

ϕM_n = 1.67x 795 k'

ϕM_n = 1328k' >1060k' = ϕM_p

ϕM_n = 1060k' = ϕM_p >1000 k'

OK

How much does self-weight add?

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

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

44

Other Solutions to Design Example 1

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

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W30×116	**378**	**943**	**1420**	**575**	**864**	**24.8**	**37.4**	**7.74**	**22.6**	**4930**	**339**	**509**
W21×147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3630	318	477
W24×131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445
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W30×108	**346**	**863**	**1300**	**522**	**785**	**23.5**	**35.5**	**7.59**	**22.1**	**4470**	**325**	**487**
W27×114	343	856	1290	522	785	21.7	32.8	7.70	23.1	4080	311	467
W21×132	333	831	1250	515	774	13.2	19.9	10.3	34.2	3220	283	425
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W18×143	322	803	1210	493	740	10.3	15.7	9.61	39.6	2750	285	427
W14×176	320	798	1200	491	738	5.20	7.83	14.2	73.2	2140	252	378
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W14×159	287	716	1080	444	667	5.17	7.85	14.1	66.7	1900	224	335
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W24×103	280	699	1050	428	643	18.2	27.4	7.03	21.9	3000	270	404
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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 $C_b\phi M_o > M_u$.

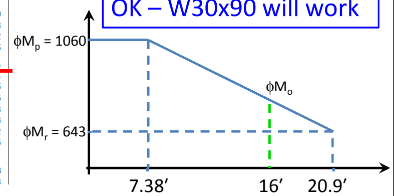
For our problem:

$C_b\phi M_r$= 1.67 x 643

$C_b\phi M_r$= 1074k'>1060 k' = ϕM_p.

ϕM_n = ϕMp = 1060k' > 1000k' = M_u.

OK – W30x90 will work



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

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

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: ϕM_p=1060 k', ϕM_r=643 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: ϕM_p=1060 k', ϕM_r=643 k', L_p=7.38', and L_r=20.9'

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

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

Therefore: $\phi M_n = 1060\,k' = \phi M_p$>1000 k' = M_u

As before, the W30x90 works.

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 ($M_p = Z_xF_y$):

A) True

B) False

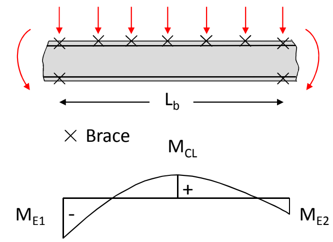
Beams Braced on One Flange With Reverse Curvature Bending

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)



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

M_{E2}= Other End Moment

M_{CL}= Midspan Moment

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.

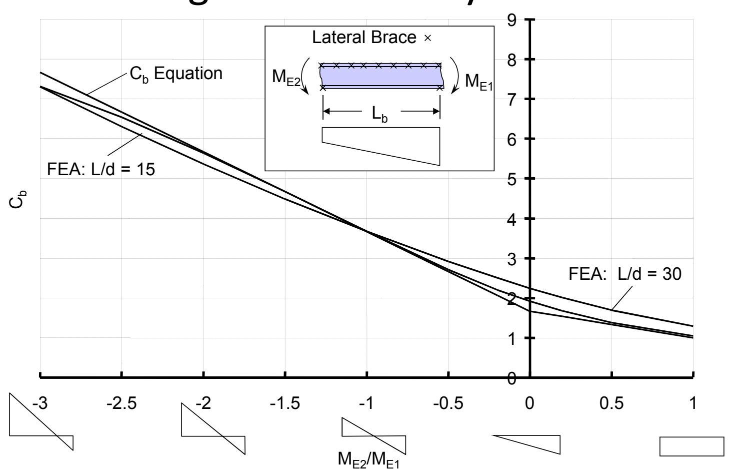
$$C_b = 3.0 - \frac{2}{3}\left(\frac{M_{E2}}{M_{E1}}\right) - \frac{8}{3}\frac{M_{CL}}{(M_{E1} + M_{E2})^*}$$

*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.

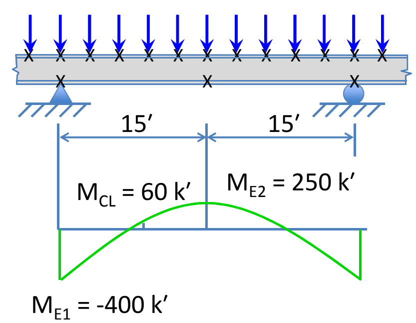
50

Moment Gradient Factors with Top Flange Continuously Braced



51

Design Example 2



Consider the beam that is continuous over the supports as shown. The top flange is continuously braced by closely spaced joists connected to a floor system. The bottom flange is restrained at the supports and at midspan.

Factored moments are shown. Select the lightest W-shape for the beam.

Moments shown are factored. M_u = 400 k′

52

Design Example 2 (Continued)

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

Pg. 3-25

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.[3]	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.[4]	ASD	LRFD
W21×55	**126**	**314**	**473**	**192**	**289**	**10.8**	**16.3**	**6.11**	**17.4**	**1140**	**156**	**234**
W14×74	126	314	473	196	294	5.31	8.05	8.76	31.0	795	128	192
W18×60	123	307	461	189	284	9.62	14.4	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.78	5.67	10.8	39.9	662	117	175
W14×68	115	287	431	180	270	5.19	7.81	8.69	29.3	722	116	174
W10×88	113	282	424	172	259	2.62	3.94	9.29	51.2	534	131	196
W18×55	**112**	**279**	**420**	**172**	**258**	**9.15**	**13.8**	**5.90**	**17.6**	**890**	**141**	**212**
W21×50	**110**	**274**	**413**	**165**	**248**	**12.1**	**18.3**	**4.59**	**13.6**	**984**	**158**	**237**
W12×72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159
W21×48[f]	**107**	**265**	**398**	**162**	**244**	**9.89**	**14.8**	**6.09**	**16.5**	**959**	**144**	**216**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.93	7.48	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192
W10×77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169
W12×65[f]	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142

Try the W21x50 with ϕM_p = 413 k' > 400 k' = M_u

L_r = 13.6' < 15' = L_b – Elastic LTB

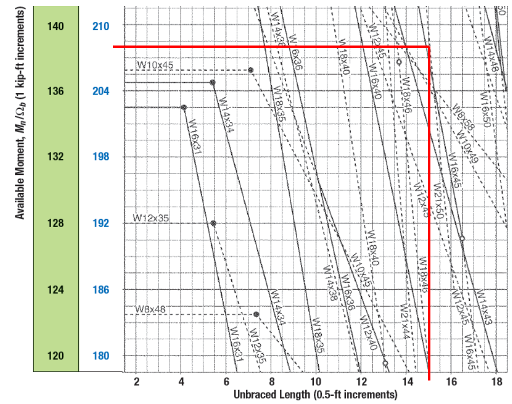
ϕM_o < 248 k' = ϕM_r……. Look for page with ϕM_o~200 k' at L_b=15'

53

Design Example 2 (Continued)

ϕM_o < 248 k' = ϕM_r……. Look for page with ϕM_o~200 k' at L_b=15'

Page 3-128



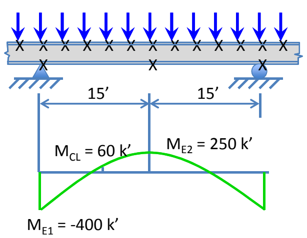
From Chart:
ϕM_o=208 k'

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

54

Design Example 2 (Continued)

From Chart: ϕM_o=208 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

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

$$C_b = 3.82$$

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

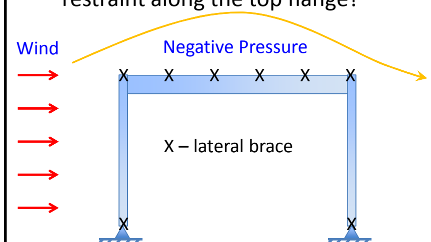
$$\phi M_n = \phi M_p = 413k' > 400\ k'$$

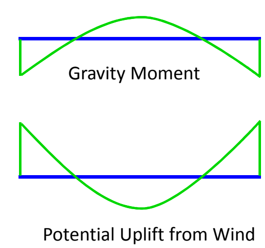
OK - Use W21x50
(Check Shear and LL Deflections)

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?





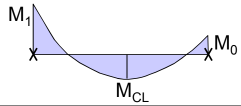
If the negative pressure doesn't cause compression in the bottom flange there is no beam buckling issue since top flange is continuously braced.

56

Moment Gradient Factors with Top Flange Continuously Braced (Uplift)

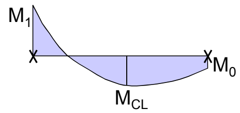
CASE A: Both end moments are positive or zero

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



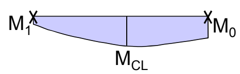
CASE B: One end moment is negative

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



CASE C: Both end moments are negative

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

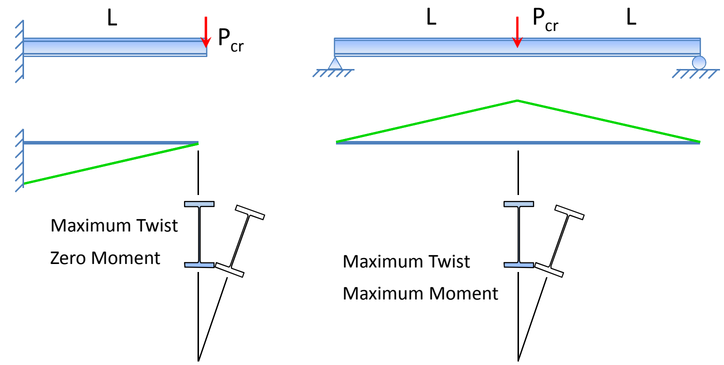


These equations are in Chapter F Commentary (Page 16.1-307).

57

Buckling of Unbraced Cantilevers

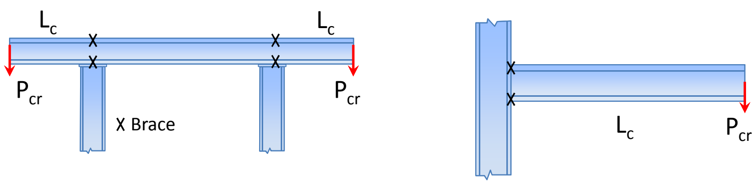
Do not use "$L_b = 2L$" for unbraced cantilevers.



58

Buckling of Unbraced Cantilevers

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

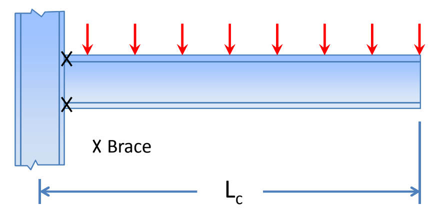


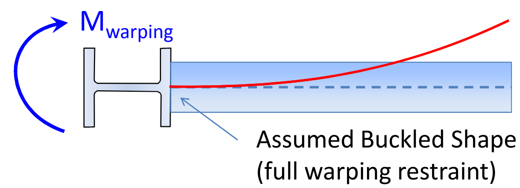
The AISC Specification (Section F1) recommends to use $C_b = 1.0$ for unbraced cantilevers. Lower Bound C_b values (from Timoshenko):

59

Effect of Fixed End Condition

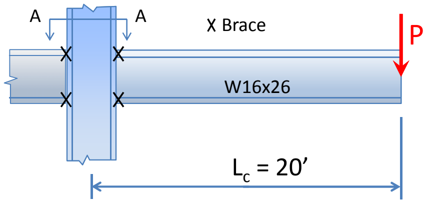
Assumption when using $L_b = L_c$ for unbraced cantilevers:



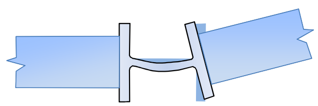


60

Effect of Fixed End Condition



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

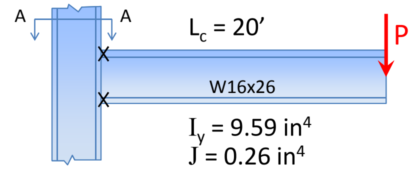


Section A-A

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

61

Approximation for Unbraced Cantilevers



I_y = 9.59 in^4
J = 0.26 in^4

Neglect the warping term in the calculation of the buckling capacity

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

$$M_{cr} = C_b \frac{\pi}{L_b}\sqrt{EI_yGJ} = 475\ k''$$

1.28 (C_b)   240 in. (L_b)

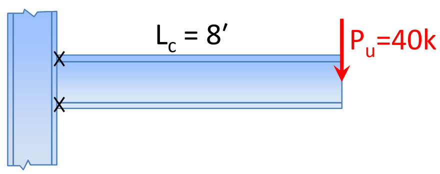
$$P_{cr} = (475k'')/240'' = \underline{\underline{1.98k}}$$

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

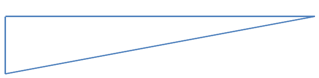
Column Size	P_{cr}
"0"	2.11 min
W8x24	2.18
W14x211	3.08
Fixed	3.61 max

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)x8′ = 320 k′

Check the Z_x Tables to find lightest W-shape with ϕM_p>320 k′

63

Design Example 3 (Cantilever in Inelastic Buckling Range)

Page: 3-25

F_y = 50 ksi

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

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W21×55	**126**	**314**	**473**	**192**	**289**	**10.8**	**16.3**	**6.11**	**17.4**	**1140**	**156**	**234**
W14×74	126	314	473	196	294	5.31	8.05	8.76	31.0	795	128	192
W18×60	123	307	461	189	284	9.62	14.4	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.78	5.67	10.8	39.9	662	117	175
W14×68	115	287	431	180	270	5.19	7.81	8.69	29.3	722	116	174
W10×88	113	282	424	172	259	2.62	3.94	9.29	51.2	534	131	196
W18×55	**112**	**279**	**420**	**172**	**258**	**9.15**	**13.8**	**5.90**	**17.6**	**890**	**141**	**212**
W21×50	**110**	**274**	**413**	**165**	**248**	**12.1**	**18.3**	**4.59**	**13.6**	**984**	**158**	**237**
W12×72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159
W21×48[f]	**107**	**265**	**398**	**162**	**244**	**9.89**	**14.8**	**6.09**	**16.5**	**959**	**144**	**216**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.93	7.48	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192
W10×77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169
W12×65[f]	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.1**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154

The W21x44 has
ϕM_p = 358 k′ >320k′

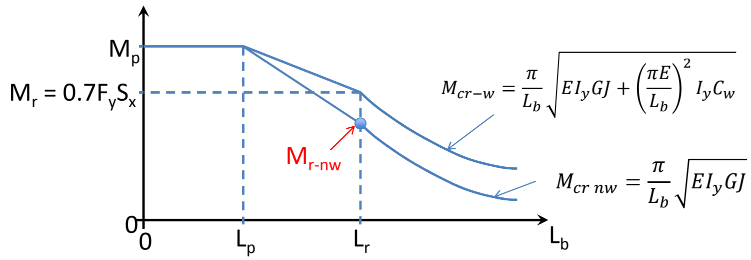
Check LTB:
L_r = 13′ > 8′ = L_b

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?

64

Design Example 3 (Cantilever in Inelastic Buckling Range)

We need to find $M_{r\text{-}nw}$ (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: $I_y = 20.7\ in^4$, $J = 0.77\ in^4$, $L_r = 13'$, $L_p = 4.45'$, $\phi M_p = 358\ k'$

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

$$\phi\ M_{r\ nw} = 0.9\left(\frac{\pi}{13x12}\right)\sqrt{29000x20.7x11200x0.77} = 1304\ k" = 109\ k'$$

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

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

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

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 ksi	F_y = 50 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

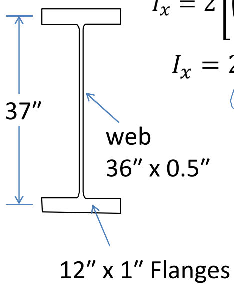
Consider the performance of the material in the cross section of an I-shape from the perspective of the moment of inertia:

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

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

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

Flange — Web

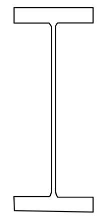


For this numerical example, the flange Ad^2 term contributes about 80% of the total moment of inertia and the web contributes the other 20%.

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.



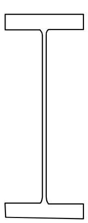
If we wanted to be able to reach the plastic bending capacity, M_p, from an efficiency perspective we would do better to have a web slenderness closer to λ_p.

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.

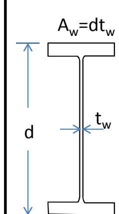


From an efficiency perspective, we often want to design the web to be able to “just” support applied shear and the size the flanges to support the bending moment working with the web. In many situations, the resulting web is relatively slender, resulting in a moment capacity that is often closer to the 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 = $V_n = 0.6F_yA_wC_v$

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

Eqn. G2-1, Pg. 16.1-67

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

$$\tau_{cr} = \frac{V_{cr}}{A_w} = \frac{\pi^2 E k_v}{12(1-\upsilon^2)\left({}^{h}/{}_{t_w}\right)^2}$$

υ = Poisson’s Ratio = 0.3

$$\tau_y = 0.6F_y$$

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

Eqn. G2-5, Pg. 16.1-68

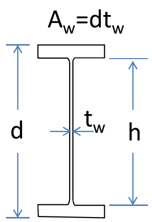
72

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):

$$V_u \le \phi V_n = 0.9x0.6F_yA_wC_v = 0.54F_y(dt_w)\left(\frac{1.51Ek_v}{\left(h/t_w\right)^2F_y}\right)$$



$d \approx h$
k_v=5
E=29000 ksi

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

$$t_w \ge \left(\frac{\sqrt[3]{V_uh}}{49.1}\right)$$

What should we use for h?

73

Preliminary Sizing of Built-Up Sections

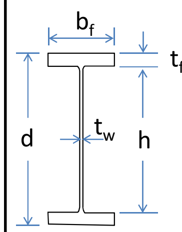
There are a number of approaches to get preliminary sizes for girder depth and flange widths.

Girder Depth ($d \approx h$):

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

Simply – Supported: L/d ≤ 25

Continuous Girder: L/d ≤ 30 - 35



Flange Width, b_f:

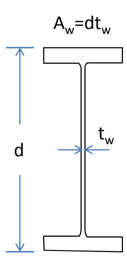
Rolled beam shapes often have $b_f \approx 0.33d$

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

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

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

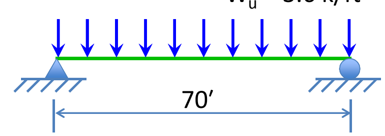
ASD Wt./ft $= \frac{(\Omega M_r)^{2/3}}{\sqrt{F_y}}$ LRFD Wt./ft $= \frac{(M_r)^{2/3}}{\sqrt{F_y}}$

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 k-ft, V_r = 126 k
Gr. 50 Steel: F_y = 50 ksi

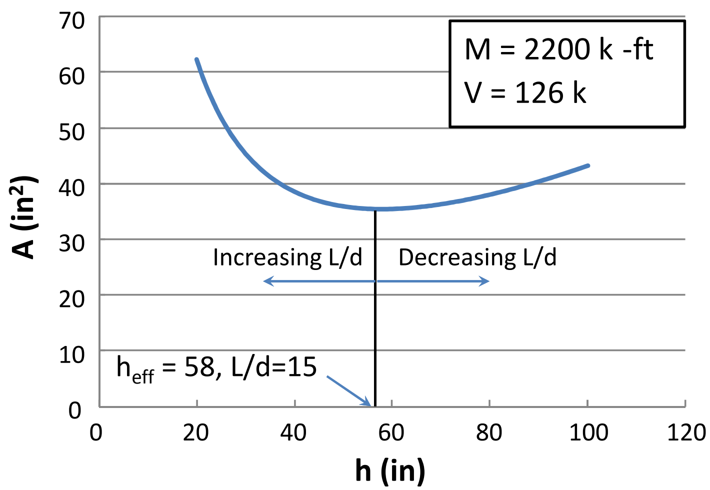
$$h_{eff} = \frac{7.6\left[M_r/\phi F_y\right]^{3/7}}{(V_r)^{1/7}} = 58\ in.$$

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

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.

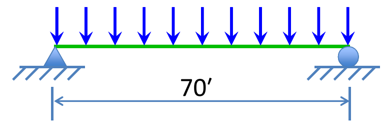
76

Preliminary Sizing of Built-Up Sections



77

LRFD Preliminary Sizing Example



M_r = 2200 k-ft, V_r = 126 k

F_y = 50 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 $\phi 0.6F_y A_{web}$ 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 :

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

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

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

78

LRFD Preliminary Sizing Example

Try both girders to get an indication of the difference in Efficiency (M_r = 2200 k-ft = 26400 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 in²
B	5/16" x 28"	93	<90.5? No	>74? Yes	129 k	8.8 in²

Girder A:

Size the Flanges: (Non-compact girder: λ_p = 90.5 < 133 < 137 = λ_r)

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

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

Use 12" x 0.75" flanges - ϕM_n = 2170 k-ft ≈ 2200 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 (in⁴)
A	3/8" x 48"	12" x 0.75"	36 in²	123	14150
B	5/16" x 28"	18" x 1.062"	47 in²	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 ($I_{x\ Gir.\ B} << I_{x\ Gir.\ A}$).
- Although Section B has more steel – how does it compare to a rolled section? From Z-tables, a W30x173 has ϕM_p = 2270 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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Within 2 business days…

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don’t forget to check the boxes next to each attendee’s name!

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8-Session Registrants

Access to the quiz: Information for accessing the quiz will be emailed to you by Wednesday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG

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Reasons for quiz:

•EEU – must take all quizzes and final to receive EEU
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•REINFORCEMENT – Reinforce what you learned tonight. Get more out of the course.

NOTE: If you attend the live presentation, you do not have to take the quizzes to receive CEUs/PDHs.

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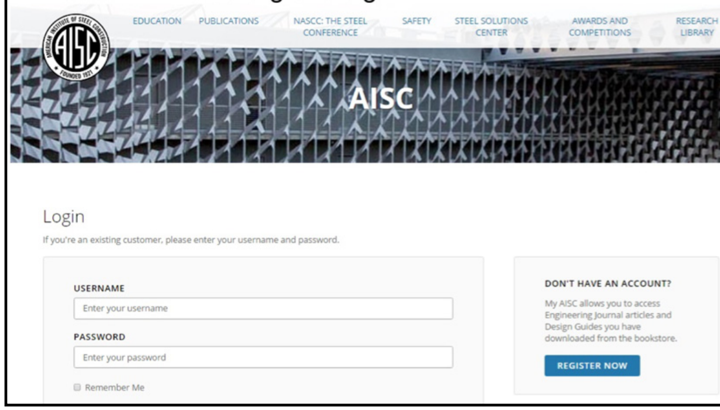

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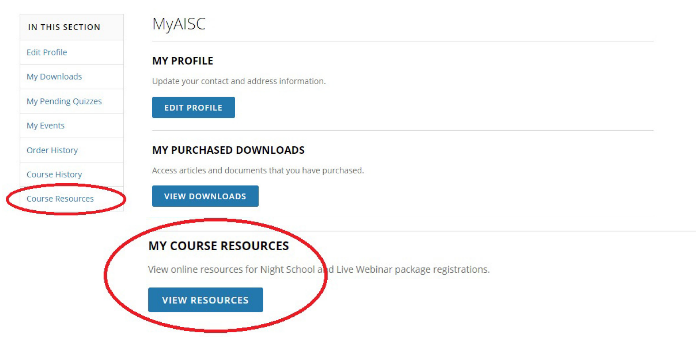

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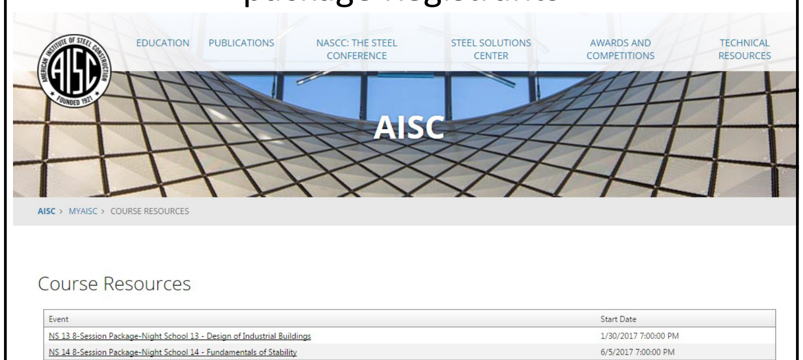

Night School Resources for 8-session package Registrants

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 03/01/2017 5pm EST	Available 03/01/2017 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 Dsn	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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- Webinar connection information:
  - Found in your registration confirmation/receipt.
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- Link to handouts also found here.

There's always a solution in steel.

Thank You

Please give us your feedback!
Survey at conclusion of webinar.

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