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Basic Steel Design

Session L2: Compression Members
March 4, 2021



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Course Description

Compression Members

The design of columns – compression members, is the focus of this session. The session will review the strength of compression members as defined by the AISC *Specification*. The session will review steel shapes and their behavior in compression. The session will discuss the limit states of flexural buckling, local buckling, torsional buckling, and flexural-torsional buckling. Members with and without slender elements are reviewed. Design examples will be presented.



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Learning Objectives

- Describe the limit state of flexural buckling for the design of compression members.
- Describe the limit state of local buckling for the design of compression members.
- Describe the limit state of torsional buckling and flexural-torsional buckling for the design of compression members.
- List the design steps for members with and without slender elements.



Basic Steel Design: A review of the principles of steel design according to ANSI/AISC 360-16

Winter Webinar 2021
Lesson L2
Compression Members



Lesson L2 – Compression

- Compression Members
 - Strength
 - Flexural buckling
 - Effective length
 - Local buckling
 - Torsional and flexural-torsional buckling
 - Built-up shapes



L2.8

Compression Members

B3.1. For LRFD, design shall be performed in accordance with:

Required Strength \leq Available Strength

$$R_u \leq \phi R_n \quad (\text{B3-1})$$

where

R_u = required strength (LRFD) defined in Chapter C

R_n = nominal strength specified in **Chapter E**

ϕ = resistance factor specified in **Chapter E**

ϕR_n = design strength = resistance factor (nominal strength)



L2.9

Compression Members

B3.2. For ASD, design shall be performed in accordance with:

Required Strength \leq Available Strength

$$R_a \leq R_n / \Omega \quad (\text{B3-2})$$

where

R_a = required strength (ASD) defined in Chapter C

R_n = nominal strength specified in **Chapter E**

Ω = safety factor specified in **Chapter E**

R_n / Ω = allowable strength = $\frac{\text{nominal strength}}{\text{safety factor}}$



L2.10

Compression Members

E1. "The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, P_n / Ω_c , are determined as follows:

The nominal compressive strength, P_n , shall be the lowest value obtained based on the applicable limit states of **flexural buckling**, **torsional buckling**, and **flexural-torsional buckling**."

$$\phi_c = 0.90 \text{ (LRFD)} \quad \Omega_c = 1.67 \text{ (ASD)}$$



L2.11

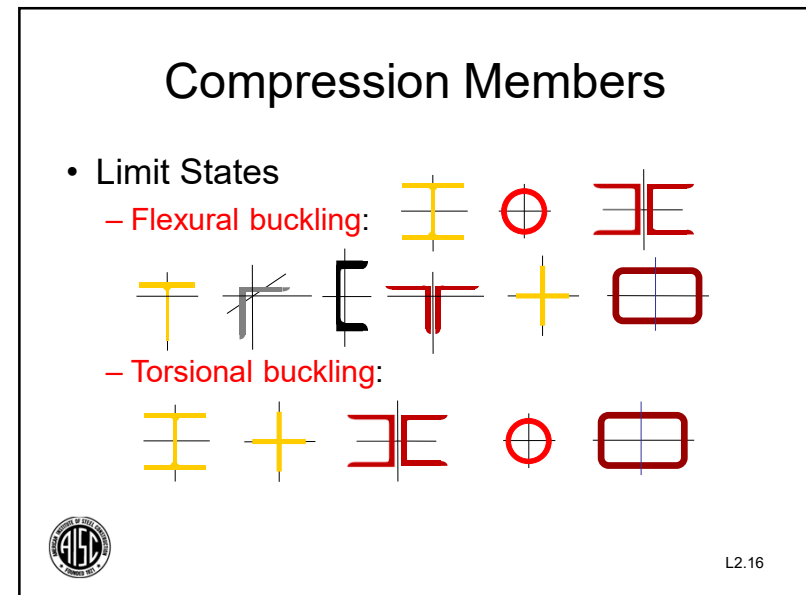
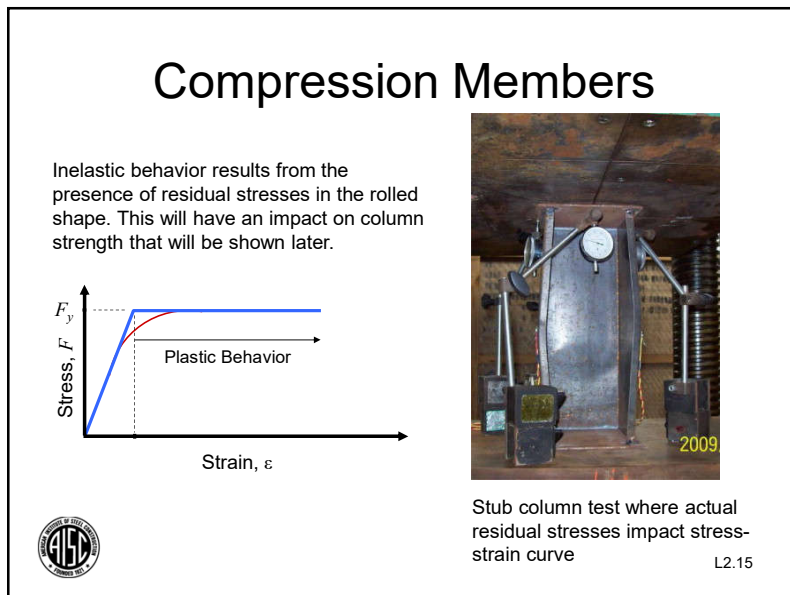
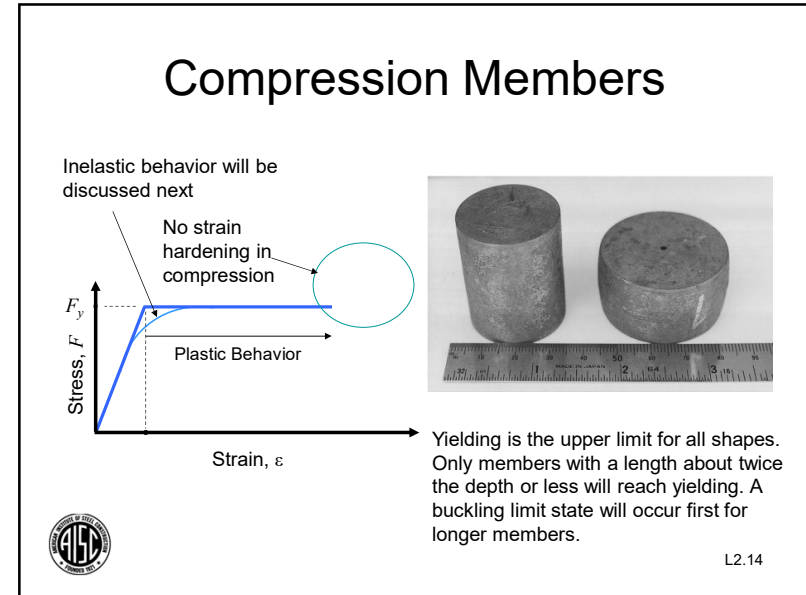
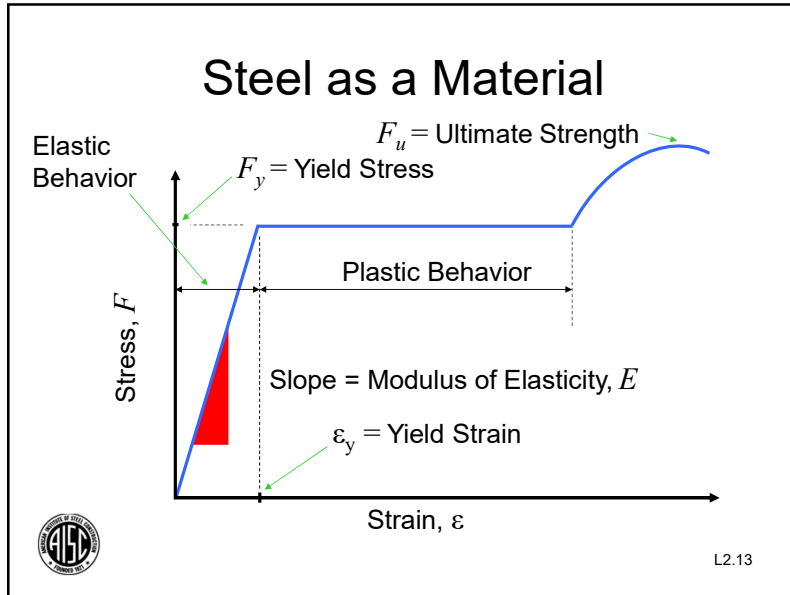
Compression Members

- Limit States

- **Yielding**: not mentioned in list of limit states to be checked. But, it is the upper limit for all shapes.
- **Flexural buckling**: lateral buckling about a geometric axis, Euler Buckling, considered for all shapes.
- **Torsional buckling**: Twist buckling of double symmetric shapes.
- **Flexural-Torsional buckling**: Combined twist and lateral buckling for singly- and non-symmetric shapes.



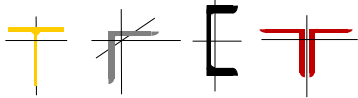
L2.12



Compression Members

- Limit States

- Flexural-Torsional buckling:



L2.17

Flexural Buckling

- Flexural buckling was first address by Leonhard Euler, a Swiss mathematician, about 1744. It is what we generally call Euler buckling.
- The theoretical derivation will not be addressed here but there are many references available.
- Remember Euler's Equation? It is given by

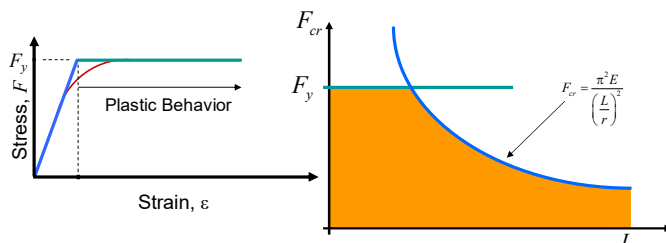
$$P_{cr} = \frac{\pi^2 EI}{L^2} \quad \text{or} \quad F_{cr} = \frac{\pi^2 E}{\left(\frac{L}{r}\right)^2}$$



L2.18

Flexural Buckling

- Yielding and elastic flexural buckling



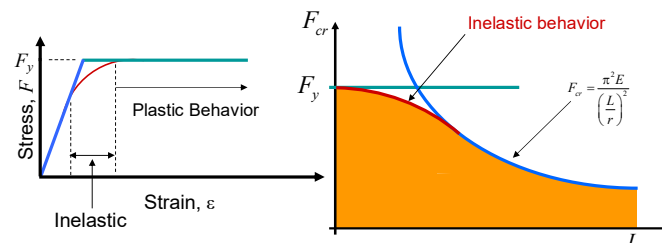
Note that although the Euler Buckling solution gives stresses greater than F_y , the column can not physically carry stresses that high, thus the curve is "cut off" at F_y .



L2.19

Flexural Buckling

- Inelastic flexural buckling



Now the Euler Buckling solution is again reduced because of the presence of residual stresses in the real compression member.



L2.20

Flexural Buckling

- Have already considered:
 - Yielding
 - Elastic flexural buckling
 - Inelastic flexural buckling
- Additional factors influencing column behavior that must be addressed to produce the *Specification* provisions:
 - End conditions: **the K -factor and effective length are introduced**
 - Out-of-straightness: **the 0.877 multiplier is used**



L2.21

Flexural Buckling

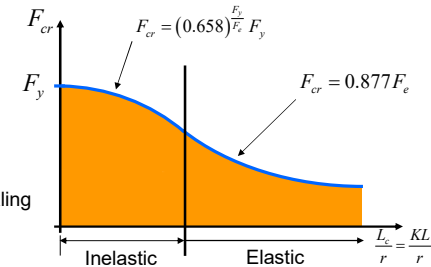
- Specification equations

Elastic buckling stress

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2}$$

Dividing line between inelastic and elastic buckling

$$\frac{KL}{r} = 4.71 \sqrt{\frac{E}{F_y}}$$



Both curves are reduced from theoretical to account for out-of-straightness



L2.22

Compression Members

E2. Effective Length

“The effective length, L_c , for calculation of member slenderness, L_c/r , shall be determined in accordance with Chapter C or Appendix 7,”

where

$L_c = KL$ = effective length of member, in. (mm)

K = effective length factor

L = laterally unbraced length of the member, in. (mm)

r = radius of gyration, in. (mm)

User Note: For members designed on the basis of compression, the effective slenderness ratio, L_c/r , preferably should not exceed 200.



L2.23

Effective Length Factor

TABLE C-A-7.1
 Approximate Values of Effective Length Factor, K

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.1	2.0
End condition code	Rotation fixed and translation fixed Rotation fixed and translation free Rotation free and translation fixed Rotation free and translation free Rotation fixed and translation free Rotation free and translation free					



L2.24

Effective Length Factor

Braced frame members: ends do not sway relative to each other

The graph shows the relationship between the effective length factor K and the relative stiffness of the frame members G_A and G_B . The x-axis represents K from 0.0 to 1.0. The y-axes represent G_A and G_B from 0.0 to 50.0. Red arrows indicate that for a fixed-fixed column ($K=0.5$), G_A and G_B must be 50.0. For a pinned-pinned column ($K=1.0$), G_A and G_B must be 1.0. The diagrams show that for a fixed-fixed column, the effective length is KL (where L is the actual length), and for a pinned-pinned column, the effective length is also KL .

L2.25

Effective Length Factor

- Compression member bracing

The diagram illustrates five different bracing conditions for a compression member of length L . The effective length factor K is determined by the bracing condition:

- $K = 1.0$: Pinned-pinned ends.
- $K = 0.5$: Fixed-fixed ends.
- $K = 0.33$: Fixed-pinned ends.
- $K = 0.25$: Pinned-fixed ends.
- $K = 0.67$: Fixed-free ends.

The definition of K is dependent on the definition of L .

L2.26

Effective Length Factor

- Compression member bracing

The diagram illustrates five different bracing conditions for a compression member of length L . The effective length factor K is determined by the bracing condition and the height of the brace:

- $K = 1.0$: Pinned-pinned ends.
- $K = 0.5$: Fixed-fixed ends.
- $K = 0.33$: Fixed-pinned ends.
- $K = 0.25$: Pinned-fixed ends.
- $K = 0.67$: Fixed-free ends.

The definition of K is dependent on the definition of L .

L2.27

Flexural Buckling

E3. Flexural Buckling of Members (Without Slender Elements)

Nominal Compressive Strength

$$P_n = F_{cr} A_g \quad (E3-1)$$

L2.28

Flexural Buckling

- Elastic buckling stress is based on

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \quad (\text{E3-4})$$

or Appendix 7.2.3(b) where F_e shall be determined from a sidesway buckling analysis for moment frames.



L2.29

Flexural Buckling

- Inelastic Response $\frac{L_c}{r} \leq 4.71 \sqrt{\frac{E}{F_y}}$ or $\frac{F_y}{F_e} \leq 2.25$

$$F_{cr} = (0.658)^{\frac{F_y}{F_e}} F_y \quad (\text{E3-2})$$

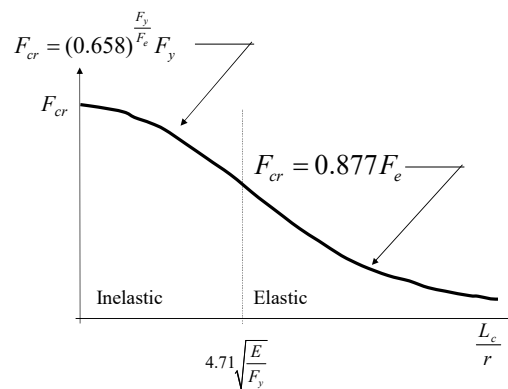
- Elastic Response $\frac{L_c}{r} > 4.71 \sqrt{\frac{E}{F_y}}$ or $\frac{F_y}{F_e} > 2.25$

$$F_{cr} = 0.877 F_e \quad (\text{E3-3})$$



L2.30

Flexural Buckling



L2.31

Flexural Buckling

- ASD

$$\frac{P_n}{\Omega_c} = \frac{F_{cr} A_g}{1.67} = 0.6 F_{cr} A_g$$

Allowable Stress

- LRFD

$$\phi_c P_n = 0.90 F_{cr} A_g$$

These are the available stress values tabulated in Table 4-14

Design Stress



L2.32

Flexural Buckling

Table 4-14
 Available Critical Stress for
 Compression Members


$\frac{L_c}{r}$	$F_y = 35$ ksi		$F_y = 36$ ksi		$F_y = 46$ ksi		$F_y = 50$ ksi		$F_y = 65$ ksi		$F_y = 70$ ksi	
	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	21.0	31.5	21.6	32.4	27.5	41.4	29.9	44.9	38.9	58.3	41.9	63.0
2	21.0	31.5	21.6	32.4	27.5	41.4	29.9	44.9	38.9	58.3	41.9	63.0
3	20.9	31.5	21.5	32.4	27.5	41.4	29.9	44.9	38.9	58.4	41.9	62.9
4	20.9	31.5	21.5	32.4	27.5	41.4	29.9	44.9	38.9	58.4	41.8	62.9
5	20.9	31.5	21.5	32.4	27.5	41.3	29.9	44.9	38.8	58.4	41.8	62.8
6	20.9	31.4	21.5	32.3	27.5	41.3	29.9	44.9	38.8	58.3	41.8	62.8
7	20.9	31.4	21.5	32.3	27.5	41.3	29.8	44.8	38.7	58.2	41.7	62.7
35	19.7	29.6	20.2	30.4	25.4	38.1	27.4	41.2	34.6	52.1	37.0	55.6
36	19.6	29.5	20.1	30.3	25.2	37.9	27.2	40.9	34.4	51.7	36.7	55.2
37	19.5	29.4	20.1	30.1	25.1	37.8	27.1	40.7	34.2	51.4	36.4	54.8
38	19.5	29.3	20.0	30.0	25.0	37.6	26.9	40.6	33.9	51.0	36.2	54.3
39	19.4	29.1	19.9	29.9	24.9	37.4	26.8	40.3	33.7	50.6	35.9	53.9
40	19.3	29.0	19.8	29.8	24.7	37.2	26.6	40.0	33.4	50.2	35.6	53.5
	ASD	LRFD										
	$\Omega_c = 1.67 \quad \phi_c = 0.90$											



L2.33

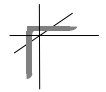
Flexural Buckling

- A992 Wide flange members, $F_y = 50$ ksi



$$\frac{L_c}{r} = 4.71 \sqrt{\frac{E}{F_y}} = 113$$

- A36 Angles, $F_y = 36$ ksi



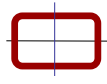
$$\frac{L_c}{r} = 4.71 \sqrt{\frac{E}{F_y}} = 134$$



L2.34


Flexural Buckling

- A500 Gr C Rectangular HSS, $F_y = 50$ ksi



$$\frac{L_c}{r} = 4.71 \sqrt{\frac{E}{F_y}} = 113$$

- A500 Gr C Round HSS, $F_y = 46$ ksi

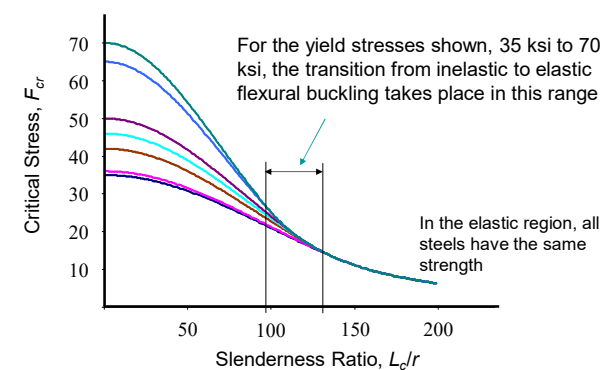


$$\frac{L_c}{r} = 4.71 \sqrt{\frac{E}{F_y}} = 118$$



L2.35

Flexural Buckling



L2.36

Flexural Buckling

Table 4-14 (continued)
Available Critical Stress for
Compression Members

F _y	50			60			70			80			90		
	ASD	LRFD	φ _c P _n	ASD	LRFD	φ _c P _n	ASD	LRFD	φ _c P _n	ASD	LRFD	φ _c P _n	ASD	LRFD	φ _c P _n
139	7.78	11.7	139	7.78	11.7	139	7.78	11.7	139	7.78	11.7	139	7.78	11.7	139
140	7.67	11.5	140	7.67	11.5	140	7.67	11.5	140	7.67	11.5	140	7.67	11.5	140
141	7.56	11.4	141	7.56	11.4	141	7.56	11.4	141	7.56	11.4	141	7.56	11.4	141
142	7.45	11.2	142	7.45	11.2	142	7.45	11.2	142	7.45	11.2	142	7.45	11.2	142
143	7.35	11.0	143	7.35	11.0	143	7.35	11.0	143	7.35	11.0	143	7.35	11.0	143
144	7.25	10.9	144	7.25	10.9	144	7.25	10.9	144	7.25	10.9	144	7.25	10.9	144
145	7.15	10.7	145	7.15	10.7	145	7.15	10.7	145	7.15	10.7	145	7.15	10.7	145
146	7.05	10.6	146	7.05	10.6	146	7.05	10.6	146	7.05	10.6	146	7.05	10.6	146

F_y has no impact on available strength

L2.37

Flexural Buckling

Table 4-1a (continued)
Available Strength in
Axial Compression, kips

F_y = 50 ksi

Table 4-1b
F_y = 65 ksi

Table 4-1c
F_y = 70 ksi

Shape lb/ft	W14s											
	145		132		120		109		99		90	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	0	1280	1920	1160	1750	1060	1590	958	1440	871	1310	793
7	1250	1880	1130	1700	1030	1550	932	1400	848	1270	772	1160
8	1230	1840	1110	1660	1010	1510	913	1370	830	1250	755	1140
9	1210	1800	1090	1620	990	1470	894	1350	810	1230	746	1120

Effective length, L_e (ft), with respect to least radius of gyration, r_y

In respect to	W14s											
	145		132		120		109		99		90	
to	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	15	1100	1650	982	1480	892	1340	808	1210	733	1100	667
16	1080	1620	960	1440	872	1310	789	1190	716	1080	652	979
17	1060	1590	937	1410	850	1280	770	1160	696	1050	635	955
18	1030	1550	913	1370	829	1240	750	1130	680	1020	618	928
19	1010	1510	888	1330	805	1210	729	1100	661	994	601	903

For W-shapes, r_y is the least radius of gyration

Shape length	W14s											
	145		132		120		109		99		90	
to	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	26	816	1224	702	1007	630	905	574	803	519	761	472
28	759	1140	648	974	586	800	529	736	478	719	434	653
30	703	1060	594	893	537	707	485	729	438	656	397	597
32	647	973	542	814	489	735	441	663	398	598	361	543

L2.38

Example 1

- Consider a W14 x 120 column (A992)
 - Shape without slender elements
- Determine the available compressive strength by ASD and LRFD

L = 30 ft
F_y = 50 ksi
K_x = 1.0
K_y = 0.5

L2.39

Example 1

Critical Slenderness

$$\frac{L_c}{r_y} = \frac{0.5(30)(12)}{3.74} = 48.1 \quad \frac{L_c}{r_x} = \frac{1.0(30)(12)}{6.24} = 57.7$$

★

$$F_c = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} = \frac{\pi^2 (29,000)}{(57.7)^2} = 86.0 \text{ ksi}$$

L2.40

Example 1

$$\frac{L_c}{r} = 57.7 < 113 \quad \text{therefore use Eq. E3-2}$$

$$F_{cr} = (0.658)^{\frac{50}{39.2}} (50) = 39.2 \text{ ksi}$$

$$P_n = (39.2)(35.3) = 1380 \text{ kips}$$



L2.41

Example 1

- ASD $\frac{P_n}{\Omega_c} = \frac{1380}{1.67} = 826 \text{ kips}$

To use Table 4-1a, when x-axis is critical we must determine an equivalent L_{cy}

$$\frac{(L_{cy})_{\text{equivalent}}}{r_y} = \frac{L_{cx}}{r_x}$$

$$\frac{(L_{cy})_{\text{equivalent}}}{r_x/r_y} = \frac{L_{cx}}{r_x/r_y}$$

$$= \frac{30}{1.67} = 18.0 \text{ ft}$$

Table 4-1a (continued)
Available Strength in Axial Compression, kips $F_y = 50 \text{ ksi}$

W14

Shape	145		132		120		109		99		90	
	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1280	1920	1160	1740	1060	1590	958	1440	871	1310	793	1190
6	1250	1880	1138	1700	1030	1550	932	1400	848	1270	772	1160
7	1240	1860	1130	1690	1020	1538	923	1390	839	1260	764	1150
8	1230	1840	1120	1680	1010	1526	914	1380	830	1250	756	1140
9	1210	1820	1100	1660	994	1490	901	1350	819	1230	745	1120
10	1200	1800	1080	1620	980	1470	888	1340	807	1210	735	1100
11	1180	1770	1060	1600	965	1450	874	1310	794	1190	723	1090
12	1160	1750	1040	1570	948	1430	859	1290	780	1170	710	1070
13	1140	1720	1020	1540	931	1400	843	1270	765	1150	697	1050
14	1120	1690	1000	1510	912	1370	826	1240	750	1130	682	1030
15	1100	1650	982	1480	892	1340	808	1210	733	1100	667	1000
16	1080	1620	965	1460	872	1310	790	1190	716	1080	652	979
17	1060	1590	947	1440	852	1280	770	1160	698	1050	635	955
18	1030	1550	913	1370	828	1240	750	1130	680	1020	618	929
19	1010	1510	888	1330	805	1210	729	1100	661	994	601	903
20	980	1470	862	1300	782	1180	708	1060	642	964	583	877

r_x , in: 3.98, 3.76, 3.74, 3.73, 3.71, 3.70
 r_x/r_y : 1.59, 1.67, 1.67, 1.67, 1.66, 1.66
 F_c/Ω_c , k/in²: 4800, 4380, 3950, 3500, 3180, 2800
 F_c/ϕ , k/in²: 19400, 15700, 14200, 12800, 11500, 10400

ASD LRFD
 $\Omega_c = 1.67$ $\phi_c = 0.90$ L2.42



L2.42

Example 1

- LRFD $\phi_c P_n = 0.9(1380) = 1240 \text{ kips}$

To use Table 4-1a, when x-axis is critical we must determine an equivalent L_{cy}

$$\frac{(L_{cy})_{\text{equivalent}}}{r_y} = \frac{L_{cx}}{r_x}$$

$$\frac{(L_{cy})_{\text{equivalent}}}{r_x/r_y} = \frac{L_{cx}}{r_x/r_y}$$

$$= \frac{30}{1.67} = 18.0 \text{ ft}$$

Table 4-1a (continued)
Available Strength in Axial Compression, kips $F_y = 50 \text{ ksi}$

W14

Shape	145		132		120		109		99		90	
	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n	P_n/Ω_c	ϕP_n
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1280	1920	1160	1740	1060	1590	958	1440	871	1310	793	1190
6	1250	1880	1138	1700	1030	1550	932	1400	848	1270	772	1160
7	1240	1860	1130	1690	1020	1538	923	1390	839	1260	764	1150
8	1230	1840	1120	1680	1010	1526	914	1380	830	1250	756	1140
9	1210	1820	1100	1660	994	1490	901	1350	819	1230	745	1120
10	1200	1800	1080	1620	980	1470	888	1340	807	1210	735	1100
11	1180	1770	1060	1600	965	1450	874	1310	794	1190	723	1090
12	1160	1750	1040	1570	948	1430	859	1290	780	1170	710	1070
13	1140	1720	1020	1540	931	1400	843	1270	765	1150	697	1050
14	1120	1690	1000	1510	912	1370	826	1240	750	1130	682	1030
15	1100	1650	982	1480	892	1340	808	1210	733	1100	667	1000
16	1080	1620	965	1460	872	1310	790	1190	716	1080	652	979
17	1060	1590	947	1440	852	1280	770	1160	698	1050	635	955
18	1030	1550	913	1370	828	1240	750	1130	680	1020	618	929
19	1010	1510	888	1330	805	1210	729	1100	661	994	601	903
20	980	1470	862	1300	782	1180	708	1060	642	964	583	877

r_x , in: 3.98, 3.76, 3.74, 3.73, 3.71, 3.70
 r_x/r_y : 1.59, 1.67, 1.67, 1.67, 1.66, 1.66
 F_c/Ω_c , k/in²: 4800, 4380, 3950, 3500, 3180, 2800
 F_c/ϕ , k/in²: 19400, 15700, 14200, 12800, 11500, 10400

ASD LRFD
 $\Omega_c = 1.67$ $\phi_c = 0.90$ L2.43



L2.43

Compression Member Design

- For a compression member design
 - We likely know
 - Required strength
 - Member length
 - Some idea of effective length factor
 - Unlike for tension members we don't know
 - The critical stress
 - However, we could estimate
 - Radius of gyration which leads to a slenderness ratio which leads to critical stress
 - Or rely on the design tables from the Manual



L2.44



Example 2 (ASD)

Select a column section by **ASD**

$$P_D = 275 \text{ kips} \quad L_{cx} = L_{cy} = 18 \text{ ft}$$

$$P_L = 600 \text{ kips}$$

ASD load combination (D+L)

$$P_a = 275 + 600 = 875 \text{ kips}$$



L2.45

Example 2 (ASD)

- Select W14x132
- Required Strength $P_a = 875 \text{ kips}$
- Available strength $\frac{P_n}{\Omega} = 913 \text{ kips}$

- Therefore the column is adequate

Table 4-1a (continued)
 Available Strength in Axial Compression, kips $F_y = 50 \text{ ksi}$
 W-Shapes

Shape lb/ft	145		132		120		109		99		90	
	P_n	ϕP_n	P_n	ϕP_n	P_n	ϕP_n	P_n	ϕP_n	P_n	ϕP_n	P_n	ϕP_n
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1200	1620	1160	1550	1060	1390	965	1240	865	1100	790	1000
4	1260	1680	1190	1570	1090	1410	992	1260	892	1120	810	1020
7	1290	1710	1200	1580	1100	1420	1000	1270	900	1130	820	1030
8	1290	1710	1180	1560	1090	1410	992	1260	892	1120	810	1020
9	1210	1600	1160	1530	1070	1390	970	1240	870	1100	790	1000
10	1200	1560	1150	1510	1060	1380	960	1230	860	1090	780	990
11	1180	1530	1140	1490	1050	1370	950	1220	850	1080	770	980
12	1160	1500	1130	1470	1040	1360	940	1210	840	1070	760	970
13	1140	1470	1120	1450	1030	1350	930	1200	830	1060	750	960
14	1120	1440	1110	1430	1020	1340	920	1190	820	1050	740	950
15	1100	1410	1100	1410	1010	1330	910	1180	810	1040	730	940
16	1080	1380	1090	1390	1000	1320	900	1170	800	1030	720	930
17	1060	1350	1080	1370	990	1310	890	1160	790	1020	710	920
18	1040	1320	1070	1350	980	1300	880	1150	780	1010	700	910
20	980	1260	1020	1300	930	1230	830	1080	730	960	650	850
22	920	1200	970	1250	880	1170	780	1030	680	910	600	800
24	870	1140	920	1200	830	1110	730	970	630	850	550	750
26	810	1080	870	1150	780	1050	680	910	580	790	500	690
28	750	1020	820	1100	730	1000	630	860	530	740	450	640
30	700	960	770	1050	680	950	580	810	480	690	400	590
32	640	900	720	1000	630	900	530	760	430	640	350	540



L2.46

Example 2 (LRFD)

Select a column section by **LRFD**

$$P_D = 275 \text{ kips} \quad L_{cx} = L_{cy} = 18 \text{ ft}$$

$$P_L = 600 \text{ kips}$$

LRFD load combination (1.2D+1.6L)

$$P_u = 1.2(275) + 1.6(600) = 1290 \text{ kips}$$



L2.47

Example 2 (LRFD)

- Select W14x132
- Required Strength $P_u = 1290 \text{ kips}$
- Available strength $\phi P_n = 1370 \text{ kips}$

- Therefore the column is adequate

Table 4-1a (continued)
 Available Strength in Axial Compression, kips $F_y = 50 \text{ ksi}$
 W-Shapes

Shape lb/ft	145		132		120		109		99		90	
	P_n	ϕP_n	P_n	ϕP_n	P_n	ϕP_n	P_n	ϕP_n	P_n	ϕP_n	P_n	ϕP_n
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1200	1620	1160	1550	1060	1390	965	1240	865	1100	790	1000
4	1260	1680	1190	1570	1090	1410	992	1260	892	1120	810	1020
7	1290	1710	1200	1580	1100	1420	1000	1270	900	1130	820	1030
8	1290	1710	1180	1560	1090	1410	992	1260	892	1120	810	1020
9	1210	1600	1160	1530	1070	1390	970	1240	870	1100	790	1000
10	1200	1560	1150	1510	1060	1380	960	1230	860	1090	780	990
11	1180	1530	1140	1490	1050	1370	950	1220	850	1080	770	980
12	1160	1500	1130	1470	1040	1360	940	1210	840	1070	760	970
13	1140	1470	1120	1450	1030	1350	930	1200	830	1060	750	960
14	1120	1440	1110	1430	1020	1340	920	1190	820	1050	740	950
15	1100	1410	1100	1410	1010	1330	910	1180	810	1040	730	940
16	1080	1380	1090	1390	1000	1320	900	1170	800	1030	720	930
17	1060	1350	1080	1370	990	1310	890	1160	790	1020	710	920
18	1040	1320	1070	1350	980	1300	880	1150	780	1010	700	910
20	980	1260	1020	1300	930	1230	830	1080	730	960	650	850
22	920	1200	970	1250	880	1170	780	1030	680	910	600	800
24	870	1140	920	1200	830	1110	730	970	630	850	550	750
26	810	1080	870	1150	780	1050	680	910	580	790	500	690
28	750	1020	820	1100	730	1000	630	860	530	740	450	640
30	700	960	770	1050	680	950	580	810	480	690	400	590
32	640	900	720	1000	630	900	530	760	430	640	350	540



L2.48



Slender Elements

- An element is slender if it would buckle locally before it is able to reach yield.
- Two types of elements
 - *Unstiffened* elements: those supported along only one edge parallel to the direction of the compression force; such as flanges
 - *Stiffened* elements: those supported along two edges parallel to the direction of the compression force; such as webs



L2.49

Unstiffened Elements

$$\frac{b}{t} = \frac{b_f}{2} \left(\frac{1}{t_f} \right) = \frac{b_f}{2t_f}$$

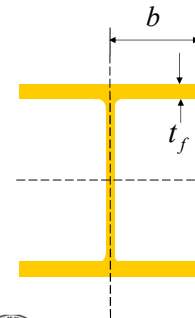


TABLE B4.1a
Width-to-Thickness Ratios: Compression Elements
Members Subject to Axial Compression

Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_p (non-slender/slender)	Examples
Unstiffened Elements	1 Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections, outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees	b/t	$0.56 \sqrt{\frac{E}{F_y}}$	
	2 Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	$0.64 \sqrt{\frac{K_c E}{F_y}}$ (a)	
	3 Legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	$0.45 \sqrt{\frac{E}{F_y}}$	
	4 Stems of tees	d/t	$0.75 \sqrt{\frac{E}{F_y}}$	

L2.50

Stiffened Elements

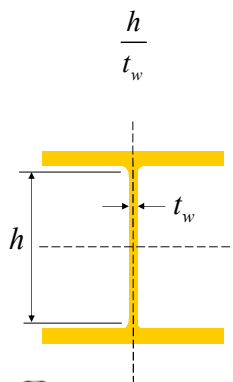


TABLE B4.1a
Width-to-Thickness Ratios: Compression Elements
Members Subject to Axial Compression

Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_p (non-slender/slender)	Examples
Stiffened Elements	5 Webs of doubly symmetric rolled and built-up I-shaped sections and channels	h/t_w	$1.49 \sqrt{\frac{E}{F_y}}$	
	6 Walls of rectangular HSS	b/t	$1.40 \sqrt{\frac{E}{F_y}}$	
	7 Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.40 \sqrt{\frac{E}{F_y}}$	
	8 All other stiffened elements	b/t	$1.49 \sqrt{\frac{E}{F_y}}$	
	9 Round HSS	D/t	$0.11 \frac{E}{F_y}$	

(a) $K_c = 4\sqrt{h/t_w}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.



L2.51

Slender Elements

Unstiffened Elements W-shape Flange - Case 1

$$A992, F_y = 50 \text{ ksi}$$

$$\frac{b_f}{2t_f} = \frac{b}{t} \leq \lambda_{rf} = 0.56 \sqrt{\frac{E}{F_y}}$$

$$\lambda_{rf} = 13.5$$



L2.52

Slender Elements

Stiffened Element
 W-shape Web - Case 5

A992, $F_y = 50$ ksi

$$\frac{h}{t_w} \leq \lambda_{rw} = 1.49 \sqrt{\frac{E}{F_y}}$$

$$\lambda_{rw} = 35.9$$



L2.53

Slender Elements

Table 1-1 (continued)
W-Shapes
 Dimensions

Shape	Area, A	Depth, d	Web		Flange		Distance				Workable Edge		
			Thickness, t_w	L_z/2	Width, b_f	Thickness, t_f	k	k	k_1	r			
	in. ²	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	
W27x129 ^a	37.8	27.6	27%	0.810	3/8	10.0	10	1.10	1 1/8	1.70	2 1/2	1 1/2	5 1/2
>114 ^b	33.6	27.3	27%	0.570	3/8	10.1	10 1/8	0.830	1 1/8	1.53	2 1/8	1 1/2	
>102 ^c	30.0	27.1	27%	0.515	3/8	10.0	10	0.830	1 1/8	1.43	2 1/8	1 1/2	
>94 ^d	27.6	26.9	26 7/8	0.490	1/2	10.0	10	0.745	3/4	1.34	1 7/8	1 1/2	
>84 ^e	24.7	26.7	26 3/4	0.460	3/8	10.0	10	0.640	3/8	1.24	1 7/8	1 1/2	
W24x370 ^a	109	28.0	28	1.52	1 1/2	13.7	13 3/8	2.72	2 3/4	3.22	4	2	20
>335 ^b	98.3	27.5	27 1/2	1.38	1 1/8	13.5	13 1/2	2.48	2 1/2	2.98	3 3/4	1 7/8	
>300 ^c	89.7	27.1	27 1/8	1.26	1 1/8	13.4	13 1/8	2.28	2 1/4	2.78	3 1/8	1 13/16	
>270 ^d	81.9	26.7	26 3/4	1.16	1 1/8	13.3	13 1/4	2.09	2 1/8	2.59	3 1/8	1 11/16	
>250	73.5												
>229	67.2												
>207	60.7												
>192	56.5	25.5	25 1/2	0.810	1 1/8	13.0	13	1.46	1 3/8	1.96	2 3/4	1 5/8	
>176	51	25.2	25 1/4	0.750	3/4	12.9	12 3/4	1.34	1 1/4	1.84	2 1/2	1 3/8	
>162	47.8	25.0	25	0.705	1 1/8	13.0	13	1.22	1 1/4	1.72	2 1/2	1 3/8	
>146	45.0	24.7	24 3/4	0.650	3/8	12.9	12 3/4	1.09	1 1/8	1.59	2 1/2	1 3/8	
>131	38.6	24.5	24 1/2	0.605	3/8	12.9	12 3/4	0.960	1 1/8	1.46	2 1/2	1 3/8	
>117 ^e	34	24.3	24 1/4	0.550	3/8	12.8	12 3/4	0.850	3/4	1.35	2 1/2	1 3/8	
>104 ^f	30.7	24.1	24	0.500	1/2	12.8	12 3/4	0.750	3/4	1.25	2 1/2	1 3/8	
W24x103 ^a	30.3	24.5	24 1/2	0.550	3/8	9.00	9	0.880	1	1.48	2 1/4	2 1/8	5 1/2

Note the footnote on the weight, 117^c



L2.54

Slender Elements

Table 1-1 (continued)
W-Shapes
 Properties

Nominal Depth, h	Web Thickness, t_w	Axis X-X			Axis Y-Y			r_x	r_y	Torsional Properties				
		I	S	Z	I	S	Z			J	C_w			
in.	in.	in. ⁴	in. ³	in. ⁴	in. ⁴	in. ³	in. ⁴	in.	in.	in. ⁶	in. ⁶			
129	4.50	39.7	4760	345	11.2	395	184	36.8	2.21	57.6	2.66	28.5	11.1	32500
114	5.41	42.5	4080	299	11.0	343	159	31.5	2.18	49.3	2.65	26.4	7.33	27600
102	6.03	47.1	3620	267	11.0	305	139	27.8	2.15	43.4	2.62	26.3	5.28	24000
94	6.09	46.5	3210	243	10.9	278	124	24.8	2.12	38.9	2.59	26.2	4.03	21300
84	7.78	52.7	2650	213	10.7	244	106	21.2	2.07	33.2	2.54	26.1	2.81	17900
370	2.51	14.2	13400	957	11.1	1130	1160	170	3.27	267	3.92	25.3	201	186000
335	2.73	15.6	11900	864	11.0	1020	1030	152	3.23	238	3.86	25.0	152	161000
306	2.94	17.1	10700	789	10.9	922	919	137	3.20	214	3.81	24.8	117	142000
279	3.18	18.8	9500	718	10.8	835	823	124	3.17	193	3.78	24.6	90.5	125000
250	3.49	20.7	8300	650	10.8	750	738	112	3.14	172	3.73	24.4	65.6	109000
229	3.79	22.5	7200	585	10.8	675	663	100	3.11	151	3.68	24.2	51.3	96100
207	4.14	24.8	6200	525	10.8	600	588	88	3.08	130	3.63	24.0	38.3	84100
192	4.43	26.8	5300	470	10.8	530	518	77	3.05	110	3.58	23.8	30.8	76300
176	4.81	28.7	4500	420	10.8	465	453	67	3.02	90	3.53	23.6	23.9	68400
162	5.31	30.6	3700	370	10.8	405	393	58	2.99	70	3.48	23.4	18.5	62600
146	5.90	33.2	2900	320	10.8	350	338	50	2.96	50	3.43	23.2	13.4	54600
131	6.70	35.6	2200	270	10.8	300	288	43	2.93	30	3.38	23.0	9.50	47100
117	7.70	39.2	1600	220	10.8	250	238	37	2.90	10	3.33	22.8	6.72	40900
104	8.90	43.1	1100	170	10.8	200	188	31	2.87	0	3.28	22.6	4.72	35200
103	4.59	39.2	3000	245	10.0	280	119	26.5	1.99	41.5	2.40	23.5	7.07	16600

Note that h/t_w exceeds 35.9



L2.55

Slender Elements

- All W-shapes have nonslender flanges for compression with $F_y < 68$ ksi.
- Only one "column" section has a slender web for compression with A992 steel;
W14x43
- Many W-shapes, meant to be used as beams, have slender webs for uniform compression. For example those just shown.



L2.56

Chapter E

- E7. Members with Slender Elements
 - Stiffened and unstiffened elements treated similarly (same effective width equation)
 - The critical stress is the same, regardless of element slenderness (E3-2, E3-3)
 - Slender element comes into play through the effective area

$$P_n = F_{cr} A_e \quad (E7-1)$$



L2.57

Chapter E

- E7. Members with Slender Elements

– when $\frac{h}{t_w} = \lambda \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$

Web of an I-shape
 $\lambda_r = 1.49 \sqrt{\frac{E}{F_y}}$

$$b_e = b \quad (E7-2)$$

$$\frac{h}{t_w} = \lambda \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}} = 1.49 \sqrt{\frac{E}{F_y}} \sqrt{\frac{F_y}{F_{cr}}} = 1.49 \sqrt{\frac{E}{F_{cr}}}$$



L2.58

Chapter E

- E7. Members with Slender Elements

– when $\frac{h}{t_w} = \lambda > \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$

Web of an I-shape
 $\lambda_r = 1.49 \sqrt{\frac{E}{F_y}}$

$$b_e = b \left(1 - c_1 \sqrt{\frac{F_{el}}{F_{cr}}} \right) \sqrt{\frac{F_{el}}{F_{cr}}} \quad (E7-3)$$

Elastic local buckling stress $F_{el} = \left(c_2 \frac{\lambda_r}{\lambda} \right)^2 F_y \quad (E7-5)$



L2.59

Chapter E

Table E7.1
 Effective Width Imperfection Adjustment Factor, c_1
 and c_2 Factor.

Case	Slender Element	c_1	c_2
(a)	Stiffened elements except walls of square and rectangular HSS	0.18	1.31
(b)	Walls of square and rectangular HSS	0.20	1.38
(c)	All other elements	0.22	1.49

Round HSS are treated differently

$$c_2 = \frac{1 - \sqrt{1 - 4c_1}}{2c_1}$$



L2.60



Example 3

- Determine the compressive strength of a built-up slender flange I-shape. $L_c = KL = 20$ ft



Flange: 24 x 0.5 in.
Web: 24 x 0.75 in.
 $r_y = 5.24$ in.

Web slenderness, Case 5
 $h/t_w = 24.0/0.75 = 32$
 $\lambda_{rf} = 1.49\sqrt{E/F_y} = 35.9$

Thus, the web is not slender

Flange slenderness, Case 2

$$k_c = \frac{4}{\sqrt{h/t_w}} = \frac{4}{\sqrt{24.0/0.75}} = 0.707$$

$$\lambda_{rf} = 0.64\sqrt{k_c E/F_y} = 0.64\sqrt{0.707(29,000)/50} = 13.0 < b_f/2t_f = 24$$

Thus, the flange is slender



L2.61

Example 3

- Determine the compressive strength of a built-up slender flange I-shape. $L_c = KL = 20$ ft



Flange: 24 x 0.5 in.
Web: 24 x 0.75 in.
 $r_y = 5.24$ in.

$$\frac{L_c}{r_y} = \frac{20(12)}{5.24} = 45.8$$

$$F_e = \frac{\pi^2 E}{(L_c/r)^2} = 136 \text{ ksi}$$

$$\frac{F_y}{F_e} = \frac{50}{136} = 0.368 < 2.25$$

$$F_{cr} = 0.658^{(0.368)} (50) = 42.9 \text{ ksi}$$



L2.62

Example 3

Table E7.1 Effective Width Imperfection Adjustment Factor, c_1 and c_2 Factor.

Case	Slender Element	c_1	c_2
(a)	Stiffened elements except walls of square and rectangular HSS	0.18	1.31
(b)	Walls of square and rectangular HSS	0.20	1.38
(c)	All other elements	0.22	1.49

We know that the flange will act as a slender element if $F_{cr} = F_y$.

But, at the actual compression stress will it act as a slender element?



L2.63

Example 3

- Determine if the flange will actually act slender.

$$\lambda_{rf} = 13.0 \text{ and } F_{cr} = 42.9 \text{ ksi}$$

$$b_f/2t_f = 24.0 > \lambda_{rf} \sqrt{F_y/F_{cr}} = 13.0 \sqrt{50/42.9} = 14.0$$

Thus, the flange will behave as a slender element for a column with a stress of 42.9 ksi

- Determine the effective width

$$F_{el} = \left(c_2 \frac{\lambda_{rf}}{\lambda} \right)^2 F_y \quad (E7-5)$$

$$= \left(1.49 \left(\frac{13.0}{24.0} \right) \right)^2 (50)$$

$$= 32.6$$

$$b_e = b \left(1 - c_1 \sqrt{\frac{F_{el}}{F_{cr}}} \right) \sqrt{\frac{F_{el}}{F_{cr}}} \quad (E7-3)$$

$$= 12 \left(1 - 0.22 \sqrt{\frac{32.6}{42.9}} \right) \sqrt{\frac{32.6}{42.9}} = 8.45$$

Note there is no upper limit on b_e since it will always be less than b



L2.64

Example 3

- Then determine the effective area

$$A_g = 24(0.75) + 2(12.0)(0.5) + 2(12.0)(0.5) = 42.0 \text{ in.}^2$$

web flange flange

$$A_e = 24(0.75) + 2(8.45)(0.5) + 2(8.45)(0.5) = 34.9 \text{ in.}^2$$

- Using the effective area, determine the nominal strength

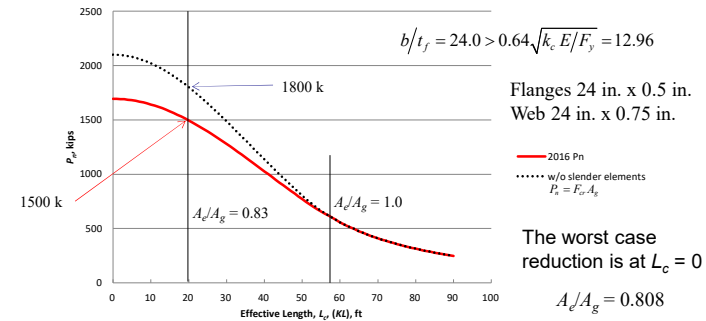
$$P_n = 42.9(34.9) = 1500 \text{ kips}$$



L2.65

Example 3

Built-up I-shape with Slender Flange,
 $F_y = 50 \text{ ksi}$



L2.66

Example 3

- At what effective length, L_c , will there be no reduction in effective area?

when $\frac{b_f}{2t_f} = \lambda \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$, $b_e = b$

- therefore

$$\frac{b_f}{2t_f} = 24 = 13 \sqrt{\frac{50}{F_{cr}}} \quad \text{thus, } F_{cr} = 14.7 \text{ ksi}$$

We could solve for L_c/r but we will use Manual Table 4-14.



L2.67

Example 3

$$\phi F_{cr} = 0.9(14.7) = 13.2 \text{ ksi}$$

$$\frac{L_c}{r} = 131$$

thus,

$$L_c = 131r_y = 131(5.24) = 686 \text{ in.} = 57.2 \text{ ft}$$

Table 4-14 (continued)
Available Critical Stress for
Compression Members

L_c/r	$F_y = 35 \text{ ksi}$		$F_y = 36 \text{ ksi}$		$F_y = 46 \text{ ksi}$		$F_y = 50 \text{ ksi}$		$F_y = 65 \text{ ksi}$		$F_y = 70 \text{ ksi}$	
	F_{cr}/k_s	ϕF_{cr}	F_{cr}/k_s	ϕF_{cr}	F_{cr}/k_s	ϕF_{cr}	F_{cr}/k_s	ϕF_{cr}	F_{cr}/k_s	ϕF_{cr}	F_{cr}/k_s	ϕF_{cr}
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
121	9.91	14.9	10.0	15.0	10.3	15.4	10.3	15.4	10.3	15.4	10.3	15.4
122	9.79	14.7	9.85	14.9	10.1	15.2	10.1	15.2	10.1	15.2	10.1	15.2
123	9.67	14.5	9.72	14.6	9.94	14.9	9.94	14.9	9.94	14.9	9.94	14.9
124	9.55	14.3	9.59	14.4	9.78	14.7	9.78	14.7	9.78	14.7	9.78	14.7
125	9.43	14.2	9.47	14.2	9.62	14.5	9.62	14.5	9.62	14.5	9.62	14.5
126	9.31	14.0	9.35	14.0	9.47	14.2	9.47	14.2	9.47	14.2	9.47	14.2
127	9.19	13.8	9.22	13.9	9.32	14.0	9.32	14.0	9.32	14.0	9.32	14.0
128	9.07	13.6	9.10	13.7	9.17	13.8	9.17	13.8	9.17	13.8	9.17	13.8
129	8.95	13.4	8.98	13.5	9.03	13.6	9.03	13.6	9.03	13.6	9.03	13.6
130	8.83	13.3	8.86	13.3	8.89	13.4	8.89	13.4	8.89	13.4	8.89	13.4
131	8.71	13.1	8.73	13.1	8.76	13.2	8.76	13.2	8.76	13.2	8.76	13.2
132	8.60	12.9	8.61	12.9	8.63	13.0	8.63	13.0	8.63	13.0	8.63	13.0
133	8.48	12.7	8.49	12.8	8.50	12.8	8.50	12.8	8.50	12.8	8.50	12.8
134	8.37	12.6	8.37	12.6	8.37	12.6	8.37	12.6	8.37	12.6	8.37	12.6
135	8.25	12.4	8.25	12.4	8.25	12.4	8.25	12.4	8.25	12.4	8.25	12.4
136	8.13	12.2	8.13	12.2	8.13	12.2	8.13	12.2	8.13	12.2	8.13	12.2



L2.68

Torsional Buckling

- Doubly symmetric members may exhibit buckling in a torsional mode.



These shapes are arranged in order of increasing torsional strength

Strength of the cruciform is very likely to be controlled by the limit state of torsional buckling while strength of the closed shapes will not.



L2.69

Torsional Buckling

- The elastic torsional buckling stress for doubly symmetric members is a function of two types of torsion, pure torsion and warping torsion. The Specification gives:

$$F_e = \left[\frac{\pi^2 EC_w}{L_c^2} + GJ \right] \frac{1}{I_x + I_y} \quad (E4-2)$$

Warping Torsion Pure Torsion

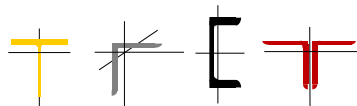
- This elastic torsional buckling stress is then used, like the elastic flexural buckling stress, to obtain the critical stress for this limit state.



L2.70

Flexural-Torsional Buckling

- Singly symmetric members can buckle in a mode that combines torsional and flexural buckling which we call flexural-torsional buckling.



L2.71

Flexural-Torsional Buckling

- For singly symmetric members the elastic flexural-torsional buckling stress.

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H} \right) \left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}} \right] \quad (E4-3)$$

The y-axis is the axis of symmetry and the z-axis represents the torsional axis



L2.72

Flexural-Torsional Buckling

- The elastic torsional buckling stress for a singly symmetric member is given by

$$F_{ez} = \left[\frac{\pi^2 EC_w}{L_{cz}^2} + GJ \right] \frac{1}{A_g \bar{r}_o^2} \quad (\text{E4-7})$$

- The elastic flexural buckling stress is given by

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{L_{cy}}{r_y} \right)^2} \quad (\text{E4-6})$$

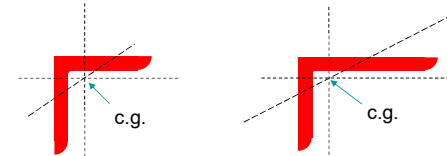


We will look at an example when we address built-up members

L2.73

Single Angle Compression Members

- These members may be singly symmetric (equal legs) or non-symmetric (unequal legs)
- To be axially loaded, they must be loaded at the centroid (unlikely)



L2.74

Single Angle Compression Members

- E5. Single-Angle Compression Members

- May consider only flexural buckling, if

$$b/t \leq 0.71 \sqrt{E/F_y}$$

This is $b/t = 20$ for $F_y = 36$ ksi

This limit is met by all currently produced angles.

- Otherwise must consider flexural-torsional buckling
- There is also a special case described in Section E5 for when the single angle is not loaded at the centroid but eccentricity may be neglected.



L2.75

Single Angle Compression Members

- If the member is
 - loaded at its ends through same leg
 - attached by welding or a minimum of two bolts
 - has no intermediate transverse loads
 - L/r determined here does not exceed 200
 - long leg/short leg ≤ 1.7

- Then

Use the modified slenderness ratio and ignore eccentricity



L2.76

Single Angle Compression Members

- As an example, for equal leg angles that are individual members or webs of planer trusses

$$\text{when } \frac{L}{r_a} \leq 80: \quad \frac{L_c}{r} = 72 + 0.75 \frac{L}{r_a} \quad (\text{E5-1})$$

$$\text{when } \frac{L}{r_a} > 80: \quad \frac{L_c}{r} = 32 + 1.25 \frac{L}{r_a} \quad (\text{E5-2})$$



L2.77

Single Angle Compression Members

- For the same equal leg angle that is part of a box or space truss

$$\text{when } \frac{L}{r_a} \leq 75: \quad \frac{L_c}{r} = 60 + 0.8 \frac{L}{r_a} \quad (\text{E5-3})$$

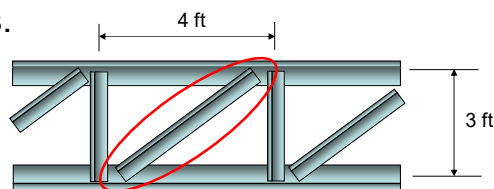
$$\text{when } \frac{L}{r_a} > 75: \quad \frac{L_c}{r} = 45 + \frac{L}{r_a} \quad (\text{E5-4})$$



L2.78

Example 4

- Determine the available compressive strength of a 5 x 3 x 1/2 A36 angle used as a web member of a truss. The web member is 5 ft long and welded to the chords.



L2.79

Example 4

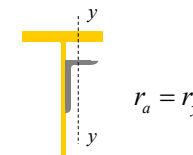
- The angle is attached through its 5 in. leg at each end. There are no intermediate transverse loads. It satisfies the requirements of Section E5

$$A_g = 3.75 \text{ in.}^2$$

$$r_x = 1.58 \text{ in.}$$

$$r_y = 0.824 \text{ in.}$$

$$r_z = 0.642 \text{ in.}$$



L2.80

Example 4

- Determine the effective slenderness

$$\frac{L_c}{r_a} = \frac{L}{r_y} = \frac{5.0(12)}{0.824} = 72.8 < 80$$

- Therefore use Eq. E5-1

$$\frac{L_c}{r} = 72 + 0.75 \left(\frac{L}{r_a} \right)$$

$$\frac{L_c}{r} = 72 + 0.75(72.8) = 127 < 4.71 \sqrt{\frac{E}{F_y}} = 134 \quad \text{Use Eq. E3-2}$$

The limit for determining use of Eq. E3-2 or E3-3



L2.81

Example 4

- Determine the elastic buckling stress from Eq. E3-4

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r} \right)^2} = \frac{\pi^2 (29000)}{(127)^2} = 17.7$$

and the critical stress from Eq. E3-2

$$F_{cr} = (0.658)^{\frac{36}{17.7}} (36) = 15.4 \text{ ksi}$$



L2.82

Example 4

- Nominal strength

$$P_n = F_{cr} A_g = 15.4(3.75) = 57.8 \text{ kips}$$

- ASD $\frac{P_n}{\Omega_c} = \frac{57.8}{1.67} = 34.6 \text{ kips}$

- LRFD $\phi_c P_n = 0.9(57.8) = 52.0 \text{ kips}$



L2.83

Example 4

- To use the concentrically loaded single angle tables, determine the effective L_c with respect to the z-axis based on the slenderness ratio already determined

$$L_{c \text{ eff}} = \left(\frac{L_c}{r} \right) r_z = \frac{127(0.642)}{12} = 6.79 \text{ ft}$$



L2.84

Example 4

Table 4-11 (continued)
 Available Strength in Axial Compression, kips
 $F_y = 36$ ksi
 Centrally Loaded Single Angles

Shape	L5 x 3 1/2 x 5						L5 x 3 x 5					
	1/4"		1/2"		7/16"		3/8"		5/16"		1/4"	
lb/ft	7.00		12.8		11.3		9.80		8.20		6.60	
Design	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$
	ASD	LFRD	ASD	LFRD	ASD	LFRD	ASD	LFRD	ASD	LFRD	ASD	LFRD
0	37.0	55.6	60.8	122	71.4	107	60.8	91.4	47.8	71.8	35.1	52.8
1	30.7	55.1	78.4	119	70.1	100	59.9	90.0	47.1	70.8	34.0	52.0
2	35.7	53.7	75.1	113	66.3	98.7	57.3	86.0	45.1	67.8	33.2	48.9
3	34.2	51.4	68.0	103	60.5	91.0	52.4	78.7	41.9	63.0	30.9	46.5
4	32.0	48.1	60.2	90.5	53.3	80.0	46.1	69.3	37.9	56.9	28.0	42.1
5	29.1	43.8	51.0	76.7	45.2	67.9	39.1	58.8	33.1	49.8	24.0	37.0
6	25.9	38.9	41.7	62.7	37.0	55.5	32.1	48.2	27.2	40.8	21.0	31.6
7	22.5	33.8	32.8	49.3	29.1	43.8	25.3	38.0	21.5	32.3	17.4	26.1
8	19.1	28.7	25.2	37.9	22.4	33.7	19.5	29.3	16.6	24.9	13.5	20.2
9	15.4	23.2	19.9	29.9	17.7	26.6	15.4	23.1	13.1	19.7	10.6	16.0
10	12.5	18.6	16.1	24.2	14.3	21.5	12.5	18.7	10.6	15.9	8.61	12.9
11	10.3	15.5										
12	8.69	13.1										

aspect to least radius of gyration, r_z

Interpolating
 $P_{cr} = 34.7$ kips
 $P_u = 52.1$ kips

L2.85

Single Angle

- If the requirements of Section E5 are not met, eccentricity must be considered

Eccentricity will induce bending moments. Combined axial force and bending will be addressed in Lesson L4.

Table 4-12 (continued)
 Available Strength in Axial Compression, kips
 $F_y = 36$ ksi
 Eccentrically Loaded Single Angles

Shape	L5 x 3 1/2 x 5						L5 x 3 x 5					
	1/4"		1/2"		7/16"		3/8"		5/16"		1/4"	
lb/ft	7.00		12.8		11.3		9.80		8.20		6.60	
Design	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$	P_n/A_g	$\phi_t P_n$
	ASD	LFRD	ASD	LFRD	ASD	LFRD	ASD	LFRD	ASD	LFRD	ASD	LFRD
0	32.1	48.3	36.2	54.4	34.7	52.2	34.2	51.4	31.9	47.9	30.0	45.1
1	32.0	48.5	35.3	53.0	34.0	51.1	33.4	50.0	31.1	46.9	29.1	43.9
2	32.4	48.6	33.0	49.8	31.8	48.0	31.2	47.1	28.9	43.7	26.8	40.6
3	33.1	49.1	29.9	45.3	28.6	43.4	27.6	42.2	25.7	38.0	25.5	36.6
4	30.0	46.8	26.3	40.1	25.0	36.1	24.0	36.5	22.1	33.9	20.0	30.5
5	25.0	38.5	22.7	34.7	21.4	32.7	20.3	31.0	18.7	28.6	16.6	25.6
6	20.6	31.9	18.3	28.6	17.1	27.7	16.9	26.0	15.4	23.7	13.7	21.2
7	17.0	26.5	14.8	23.0	14.1	23.2	14.0	21.6	12.7	19.6	11.3	17.5
8	14.1	22.9	12.6	20.9	12.8	19.4	11.6	17.9	10.4	16.1	9.23	14.2
9	11.7	18.2	11.5	17.8	10.6	16.4	9.79	15.0	8.72	13.4	7.64	11.8
10	9.79	15.2	9.95	15.2	9.12	14.0	8.93	12.8	7.98	11.3	6.43	9.94
11	8.31	12.0										
12	7.14	11.0										

ecc to least radius of gyration, r_z

L2.86

Built-Up Members

- Built-up members are composed of two shapes interconnected with bolts or welds or with at least one open side interconnected by plates or lacing.
- The key to determining the strength of built-up members is determining the correct slenderness ratio.



L2.87

Built-Up Members

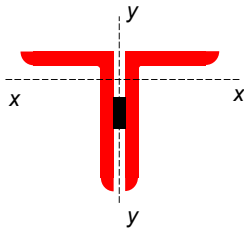
- Section E6 requires
 - “The end connection shall be welded or connected by means of pretensioned bolts with Class A or B faying surfaces.”
- Slip in the end connection could cause the built-up member to lose strength and behave as individual members.



L2.88

Built-Up Members

- Nominal strength is determined using a modified slenderness ratio if connectors are in shear in the buckling mode.



Buckling about the x-axis treat as two single angles

Buckling about y-axis treat as a built-up member



L2.89

Built-Up Members

- When intermediate connectors are snug-tight bolts,

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \quad (E6-1)$$

$\left(\frac{L_c}{r}\right)_m$ = modified slenderness ratio

$\left(\frac{L_c}{r}\right)_o$ = built-up member acting as a unit

a = distance between connectors

r_i = minimum radius of gyration of component



L2.90

Built-Up Members

- When intermediate connectors are welds or pretensioned bolts

$$\text{for } \frac{a}{r_i} \leq 40 \quad \left(\frac{L_c}{r}\right)_m = \left(\frac{L_c}{r}\right)_o \quad (E6-2a)$$

$$\text{for } \frac{a}{r_i} > 40 \quad \left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \quad (E6-2b)$$



L2.91

Built-Up Members

- Definitions:

$\left(\frac{L_c}{r}\right)_m$ = modified slenderness ratio

$\left(\frac{L_c}{r}\right)_o$ = slenderness ratio of built-up member acting as a unit

$K_i = 0.50$ for angles back-to-back

$= 0.75$ for channels back-to-back

$= 0.86$ for all other cases

a = distance between connectors

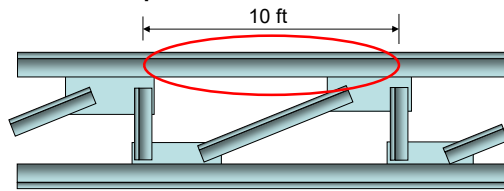
r_i = minimum radius of gyration of component



L2.92

Example 5

- Determine the available compressive strength of 2-L5 x 3 x 1/2 LLBB A36 angles used as the top chord of a truss. The angles are attached with welds at two intermediate points and at the ends.



L2.93

Example 5

- Determine the available compressive strength of 2-L5 x 3 x 1/2 LLBB A36 angles used as the top chord of a truss.

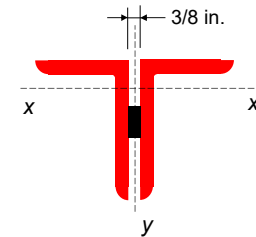
Single Angle Table 1-7

$$A_g = 3.75 \text{ in.}^2 \quad I_x = 9.43 \text{ in.}^4$$

$$r_x = 1.58 \text{ in.} \quad I_y = 2.55 \text{ in.}^4$$

$$r_y = 0.824 \text{ in.} \quad J = 0.322 \text{ in.}^4$$

$$r_z = 0.642 \text{ in.} \quad C_w = 0.444 \text{ in.}^6$$



L2.94

Example 5

- Combined properties, Table 1-15 in red here

$$A_g = 2(3.75) = 7.50 \text{ in.}^2 \quad H = 0.646$$

$$I_x = 2(9.43) = 18.9 \text{ in.}^4 \quad \bar{r}_o = 2.51 \text{ in.}$$

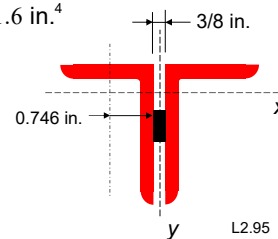
$$r_x = 1.58 \text{ in.}$$

$$I_y = 2 \left(2.55 + 3.75 \left(0.746 + \frac{3}{16} \right)^2 \right) = 11.6 \text{ in.}^4$$

$$r_y = \sqrt{\frac{11.6}{7.50}} = 1.24 \text{ in.}$$

$$J = 2(0.322) = 0.644 \text{ in.}^4$$

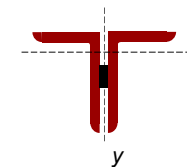
$$C_w = 2(0.444) = 0.888 \text{ in.}^6$$



L2.95

Example 5

- For a singly symmetric compression member we will see that the applicable limit states are flexural-torsional buckling about the axis of symmetry, y-axis, and flexural buckling about the other axis, x-axis.



L2.96

Example 5

- Since the double angle compression member is a built-up member,
 - if the buckling mode involves relative deformation that produces shear forces in the connectors between individual shapes, the modified slenderness ratio as a function of connector spacing must be determined according to Section E6.
 - If the buckling mode does not involve relative deformation, the slenderness ratio using the actual effective length and radius of gyration is used.



L2.97

Example 5

- For the x-axis (no connector shear)
 - The slenderness ratio is

$$\frac{L_c}{r_x} = \frac{10(12)}{1.58} = 75.9$$

- and the elastic buckling stress is

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{L_c}{r_x}\right)^2} = \frac{\pi^2 (29,000)}{(75.9)^2} = 49.7 \text{ ksi} \quad (\text{E3-4})$$



L2.98

Example 5

- For the y-axis (connectors in shear)
 - Buckling produces shear forces in the connectors between individual shapes.
 - For our example, place pretensioned connectors at the 1/3 points of the column

$$a = 40.0 \text{ in.}$$



L2.99

Example 5

- For the y-axis (connectors in shear)
 - As a single unit

$$\left(\frac{L_c}{r_y}\right)_o = \frac{10(12)}{1.24} = 96.8$$

- Between connectors

$$\frac{a}{r_i} = \frac{a}{r_z} = \frac{40}{0.642} = 62.3 > 40$$

- Thus,

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{K_1 a}{r_i}\right)^2} = \sqrt{(96.8)_o^2 + \left(\frac{0.5(40)}{0.642}\right)^2} = 102 \quad \text{E6-2b}$$



L2.100

Example 5

- For the y-axis (connectors in shear)
 - With the slenderness ratio

$$\frac{L_c}{r_y} = \left(\frac{L_c}{r}\right)_m = 102$$

- the elastic buckling stress is

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{L_c}{r_y}\right)^2} = \frac{\pi^2 (29,000)}{(102)^2} = 27.5 \text{ ksi} \quad (\text{E3-4})$$



L2.101

Example 5

- E4. Torsional and Flexural-torsional Buckling
 - E4.(b) for singly symmetric members twisting about the shear center where y is the axis of symmetry

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right) \left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right] \quad (\text{E4-3})$$

If x is the axis of symmetry replace F_{ey} by F_{ex}



L2.102

Example 5

- For torsional buckling

$$F_{ez} = \left[\frac{\pi^2 EC_w}{L_{cz}^2} + GJ\right] \frac{1}{A_g \bar{r}_o^2} \quad (\text{E4-7})$$

$$F_{ez} = \left[\frac{\pi^2 E (0.888)}{(12(10))^2} + 11,200(0.644)\right] \frac{1}{7.50(2.51)^2}$$

$$= [17.5 + 7213] \frac{1}{47.3} = 153 \text{ ksi}$$



L2.103

Example 5

- For flexural-torsional buckling

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right) \left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right] \quad (\text{E4-3})$$

$$F_e = \left(\frac{27.5 + 153}{2(0.646)}\right) \left[1 - \sqrt{1 - \frac{4(27.5)(153)(0.646)}{(27.5 + 153)^2}}\right]$$

$$= 25.7 \text{ ksi}$$

This is less than F_{ey} , thus flexural-torsional buckling will control.



L2.104

Example 5

- Determine the critical stress

$$\frac{F_y}{F_e} = \frac{36.0}{25.7} = 1.40 < 2.25$$

This is the "other way" to determine which equation to use in determining F_{cr} .

- Therefore use Eq. E3-2

$$F_{cr} = (0.658)^{\frac{36}{25.7}} (36) = 20.0 \text{ ksi}$$



L2.105

Example 5

- The nominal strength is then,

$$P_n = F_{cr} A_g = 20.0(7.50) = 150 \text{ kips}$$

- The available strength is

For LRFD

$$\phi P_n = 0.9(150) = 135 \text{ kips}$$

For ASD

$$\frac{P_n}{\Omega} = \frac{150}{1.67} = 89.8 \text{ kips}$$



L2.106

Example 5

If back-to-back spacing is greater than 3/8 in., the table values are conservative.

$$\phi P_n = 136 \text{ kips}$$

$$\frac{P_n}{\Omega} = 90.2 \text{ kips}$$

Note 2 intermediate connectors required.

Shape	2L5x3x					c
	1/2	3/8	1/2	3/8	1/2	
Design	P_n/Ω	ϕP_n	P_n/Ω	ϕP_n	P_n/Ω	ϕP_n
0	162	243	143	214	122	183
4	140	211	121	181	89.9	150
6	129	194	111	167	82.8	140
8	109	165	94.7	142	79.4	119
10	90.2	136	79.0	117	65.4	90.2
12	71.1	107	61.3	92.1	51.3	77.1
14	53.8	80.8	46.3	69.6	38.8	58.4
16	41.5	62.4	35.9	53.9	30.2	45.4
18	33.0	49.6	28.5	42.8	24.4	35.2
20	26.8	40.1	23.2	34.7	19.6	28.5
A_g , in. ²	7.50	6.62	5.72	4.82	3.88	
r_x , in.	1.58	1.59	1.60	1.61	1.62	
r_y , in.	1.24	1.23	1.22	1.21	1.19	
r_x , in.	0.642	0.644	0.646	0.649	0.652	
ASD	ϕP_n	136	136	136	136	
LRFD	ϕP_n	90.2	90.2	90.2	90.2	



L2.107

Example 5

x-axis strength is independent of intermediate connectors

$$\phi P_n = 179 \text{ kips}$$

$$\frac{P_n}{\Omega} = 119 \text{ kips}$$

y-axis strength includes flexural-torsional buckling strength

$$\phi P_n = 136 \text{ kips}$$

$$\frac{P_n}{\Omega} = 90.2 \text{ kips}$$

Shape	2L5x3x					c
	1/2	3/8	1/2	3/8	1/2	
Design	P_n/Ω	ϕP_n	P_n/Ω	ϕP_n	P_n/Ω	ϕP_n
0	162	243	143	214	122	183
4	140	211	121	181	89.9	150
6	129	194	111	167	82.8	140
8	109	165	94.7	142	79.4	119
10	90.2	136	79.0	117	65.4	90.2
12	71.1	107	61.3	92.1	51.3	77.1
14	53.8	80.8	46.3	69.6	38.8	58.4
16	41.5	62.4	35.9	53.9	30.2	45.4
18	33.0	49.6	28.5	42.8	24.4	35.2
20	26.8	40.1	23.2	34.7	19.6	28.5
A_g , in. ²	7.50	6.62	5.72	4.82	3.88	
r_x , in.	1.58	1.59	1.60	1.61	1.62	
r_y , in.	1.24	1.23	1.22	1.21	1.19	
r_x , in.	0.642	0.644	0.646	0.649	0.652	
ASD	ϕP_n	136	136	136	136	
LRFD	ϕP_n	90.2	90.2	90.2	90.2	



L2.108

Summary

- Looked at the limit states for compression members
- Addressed flexural buckling
- Considered design of compression members
- Treated members with slender elements
- Discussed torsional and flexural-torsional buckling
- Treated the special case of single angles
- Addressed built-up members



L2.109

Lesson L3

- The next lesson will look at the principles of design for flexural members, including shear
- We will look at the material in Chapters F and G of the *Specification* and Part 3 of the Manual



L2.110



Thank You

American Institute of Steel Construction
130 East Randolph St., Suite 2000
Chicago, IL 60601



L2.111

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Single-Session Registrants

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Course Resources

Event	Start Date
Seismic Design of Joist	1/12/2019 12:00:00 AM
4-Session Package Design of Facade Attachments	5/9/2019 1:00:00 PM
05_20_8-Session Package High School 13 - Fundamentals of Connection Design	10/5/2017 7:00:00 PM
05_20_8-Session Package High School 14 - Seismic Design of Joist	2/9/2018 7:00:00 PM
05_21_8-Session Package High School 15 - Design of Facade Attachments	7/26/2018 7:00:00 PM
05_20_8-Session Package High School 18 - Steel Construction: Mid To Tallness Dist	10/5/2018 7:00:00 PM
05_20_8-Session Package High School 19 - Connections Design	2/4/2019 7:00:00 PM
05_20_8-Session Package High School 20 - Classical Methods of Structural Analysis	6/9/2019 7:00:00 PM
8-Session Package Seismic Design of Steel - Concepts & Examples	7/16/2018 1:00:00 PM

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Design of Facade Attachments

4-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
R3 Facade Fundamentals	N/A	Download	Video	Pass	N/A
L1 Facade Attachments Part 1	May 9 2019 1:00PM EDT	Download	Available 05/11/2019 5:00PM EDT	Available 05/11/2019 5:00 PM EDT	Pending
L2 Facade Attachments Part 2	May 14 2019 1:00PM EDT	Download	Available 05/16/2019 5:00PM EDT	Available 05/16/2019 5:00 PM EDT	Pending
L3 Facade Attachments - Building Lateral Drift	May 21 2019 1:00PM EDT	Download	Available 05/23/2019 5:00PM EDT	Available 05/23/2019 5:00 PM EDT	Pending
Total Event	N/A			Available 5/27/2019 5:00 PM EDT	



