Summary

Solved Problem 5-2

Short - non slender W Section CM#14

Use Table 4-22 & 4-1 } Parts a&b
For Factored Loads }

Verify by Using AISC E-3 ⇒ Part C

Prepared by Eng.Maged Kamel.

Example 5-2 JACK McCormack 5Th edition

(a) Using the column critical stress values in Table 4-22 of the Manual, determine the LRFD design strength $\phi_c P_n$ and the ASD allowable strength $\frac{P_n}{\Omega_c}$ for the column shown in Fig. 5.8, if a 50-ksi steel is used.

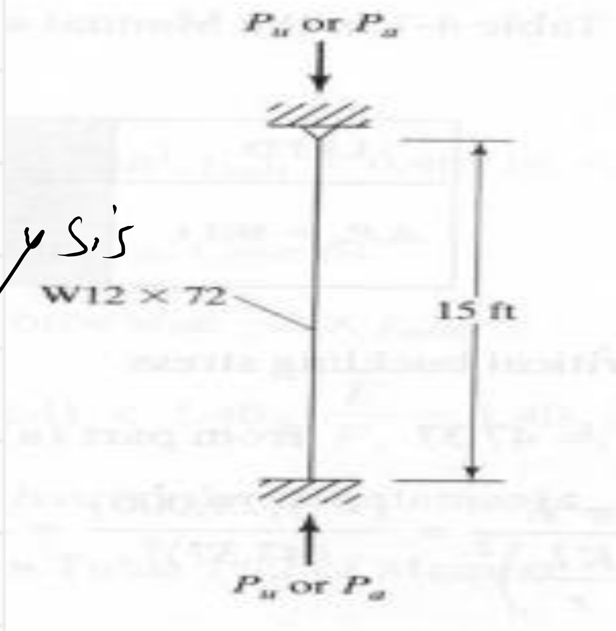
CM # 14

(b) Repeat the problem, using Table 4-1 of the Manual.

(c) Calculate $\phi_c P_n$ and $\frac{P_n}{\Omega_c}$, using the equations of AISC Section E3.

Solution

① This an analysis Example



Given W12 X 72 Column

check if W Section is short _ Long

Use Table 4-22

Use Table 4-1

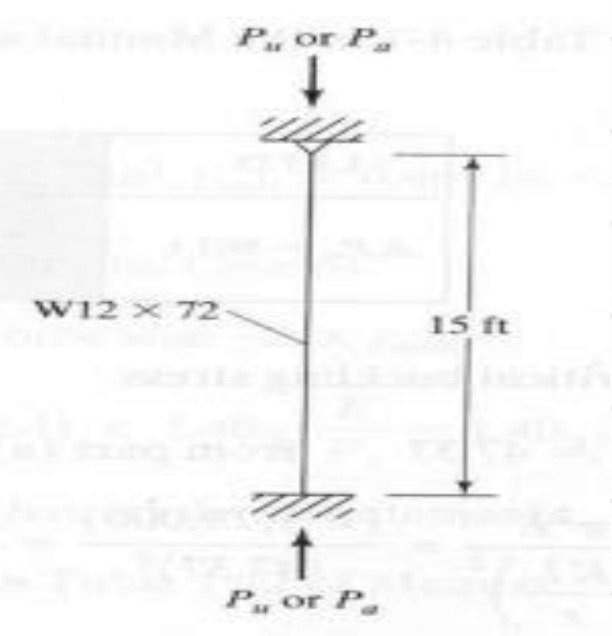
$\left.\begin{array}{l}\phi_c P_n \\ \frac{P_n}{\Omega_c}\end{array}\right\}$ $F_y = 50$ ksi

slender or not slender

Prepared by Eng.Maged Kamel.

What is the K value ?

Fixed at bottom
Hinged at top } $K = 0.70$



But K recommended = 0.80

Check Table 1-1 to get the data for W12X72 From Part-1 part-2

$K_x = K_y = 0.80$

$L = 15$ Ft

Prepared by Eng.Maged Kamel.

CM # 14

Table 1-1
Part-1
W 12 X 72

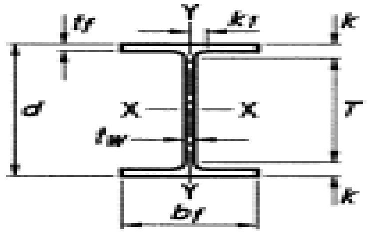


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		k_{des}	k_{det}	k_1	T	Workable Gage
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
×72	21.1	12.3	12 1/4	0.430	7/16	1/4	12.0	12	0.670	11/16	1.27	1 9/16	1 1/16	↓	↓
×65[f]	19.1	12.1	12 1/8	0.390	3/8	3/16	12.0	12	0.605	5/8	1.20	1 1/2	1		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W12 X 72

$A = 21.2$ inch²
$d = 12.3"$
$b_f = 12"$

$t_w = 0.43"$
$t_F = 0.67"$

Part-2 Table 1-1

Table 1-1 (continued)
W-Shapes
Properties

W14-W12

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
→ 72	8.99	22.6	597	97.4	5.31	108	195	32.4	3.04	49.2	3.41	11.6	2.93	6540
65	9.92	24.9	533	87.9	5.28	96.8	174	29.1	3.02	44.1	3.38	11.5	2.18	5780

W12 x 72 $\frac{bf}{2tf} = 8.99$ & $\frac{h}{tw} = 22.60$

$r_x = 5.31"$
$r_y = 3.04"$

Check whether Column W12x72 is Long or short

$E = 29000$ ksi

$F_y = 50$ ksi

$L = 15'$

$r_x = 5.31''$

$r_y = 3.04''$

$$\lambda_r = 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.19$$

Check $\left(\frac{KL}{r}\right)_x = \frac{0.8(15)(12)}{5.31} = 27.12$

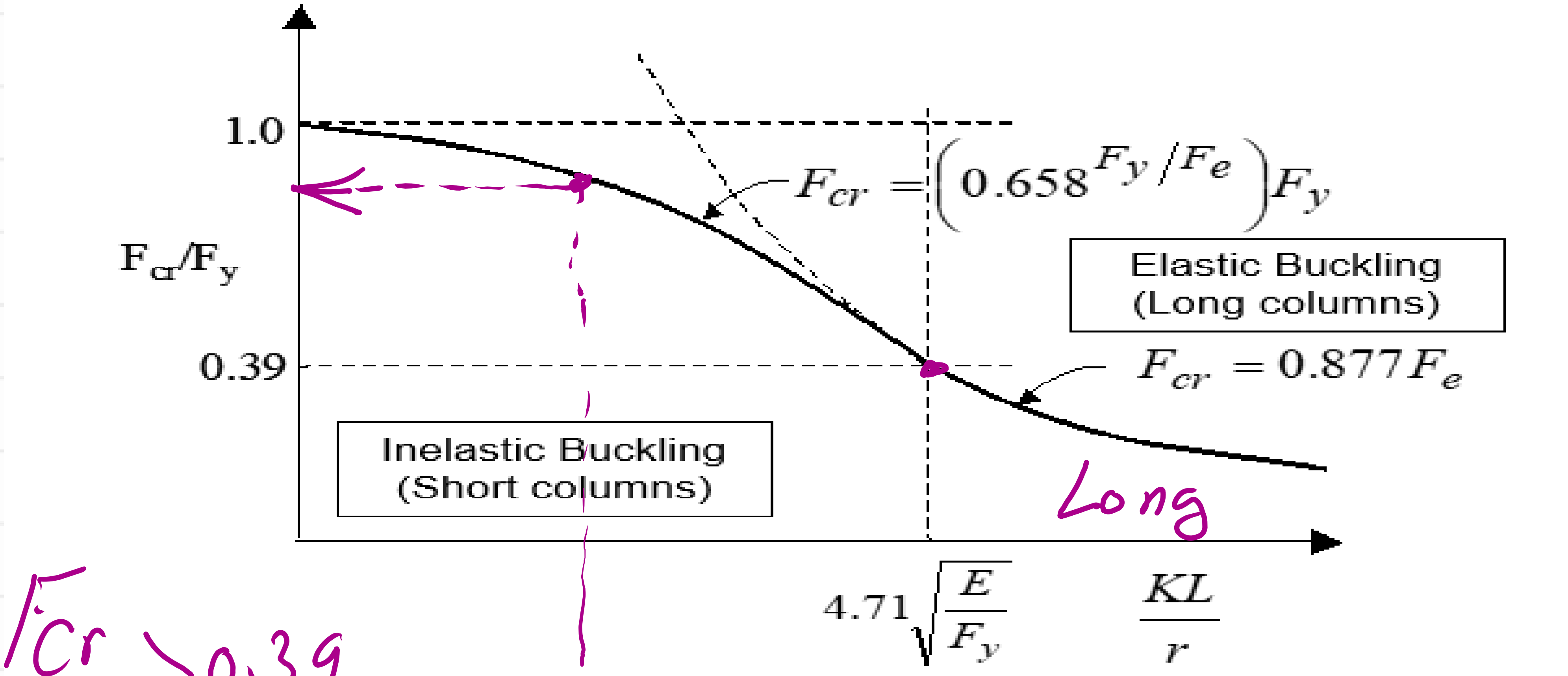
$\left(\frac{KL}{r}\right)_y = \frac{0.8(15)(12)}{3.04} = 47.36$

select max Value

Consider $\left(\frac{KL}{r}\right)_y = 47.36$

Prepared by Eng.Maged Kamel.

$\left(\frac{KL}{r}\right)_y = 47.36$ Column is short $F_y = 50ksi$



$\frac{F_{cr}}{F_y} > 0.39$
< 1

47.36

$F_y = 50 ksi$
$\lambda_r = 113.43$

① Column is Short since $\left(\frac{KL}{r}\right)_y < 4.71\sqrt{\frac{E}{F_y}}$

$47.36 < 113.43$

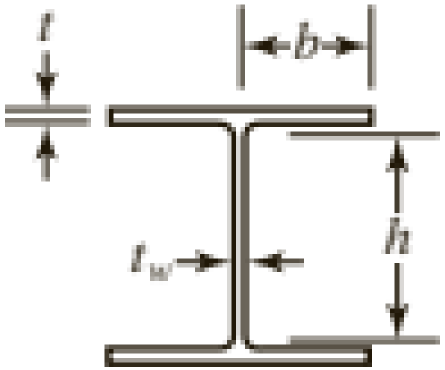
② Check whether $W_{12} \times 72$ is slender or non slender

$\frac{b}{t} \leq 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29000}{50}} = 13.48$

while $\frac{h}{t_w} \leq 1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29000}{50}}$

$= 35.88$

FIGURE 4.9



$b/t \leq 0.56\sqrt{E/F_y}$

$h/t_w \leq 1.49\sqrt{E/F_y}$

For $W_{12} \times 72$

$\frac{b_f}{2t_f} = 8.99 < 13.48$

$h/t_w = 22.6 < 35.88$

whole section non slender

Prepared by Eng.Maged Kamel.

CM#14 #2010 $Q_s = 1$ $b/t \leq 0.56\sqrt{\dfrac{E}{F_y}}$

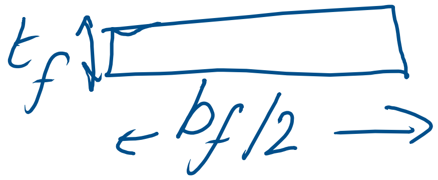
1. Slender Unstiffened Elements, Q_s

The reduction factor, Q_s, for slender *unstiffened elements* is defined as follows:

(a) For flanges, angles and plates projecting from rolled *columns* or other compression members:

check For flange

(i) When $\dfrac{b}{t} \leq 0.56\sqrt{\dfrac{E}{F_y}}$ ✓



in our example $\longrightarrow$

$$Q_s = 1.0 \tag{E7-4}$$

(ii) When $0.56\sqrt{\dfrac{E}{F_y}} < \dfrac{b}{t} < 1.03\sqrt{\dfrac{E}{F_y}}$

$$Q_s = 1.415 - 0.74\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \tag{E7-5}$$

(iii) When $\dfrac{b}{t} \geq 1.03\sqrt{\dfrac{E}{F_y}}$

$$Q_s = \frac{0.69E}{F_y\left(\dfrac{b}{t}\right)^2} \tag{E7-6}$$

Use Table 4-22 ⇒ part (a)

LRFD

$\frac{KL}{r_y} = 47.37$

$\phi F_{cr} = 38.3$ ksi

$\frac{KL}{r} = \downarrow 47$

Table 4-22 (continued)

Available Critical Stress for Compression Members

F_y = 35 ksi			F_y = 36 ksi			F_y = 42 ksi			F_y = 46 ksi			F_y = 50 ksi		
$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
41	19.2	28.9	41	19.7	29.7	41	22.7	34.1	41	24.6	37.0	41	26.5	39.8
42	19.2	28.8	42	19.6	29.5	42	22.6	33.9	42	24.5	36.8	42	26.3	39.5
43	19.1	28.7	43	19.6	29.4	43	22.5	33.7	43	24.3	36.6	43	26.2	39.3
44	19.0	28.5	44	19.5	29.3	44	22.3	33.6	44	24.2	36.3	44	26.0	39.1
45	18.9	28.4	45	19.4	29.1	45	22.2	33.4	45	24.0	36.1	45	25.8	38.8
46	18.8	28.3	46	19.3	29.0	46	22.1	33.2	46	23.9	35.9	46	25.6	38.5
47	18.7	28.1	47	19.2	28.9	47	22.0	33.0	47	23.8	35.7	47	25.5	38.3
48	18.6	28.0	48	19.1	28.7	48	21.8	32.8	48	23.6	35.4	48	25.3	38.0
49	18.5	27.9	49	19.0	28.5	49	21.7	32.6	49	23.4	35.2	49	25.1	37.7
50	18.4	27.7	50	18.9	28.4	50	21.6	32.4	50	23.3	35.0	50	24.9	37.5

$\phi_c P_n = A_g(\phi_c F_{cr})$ $21.10\left[38.3 - \frac{(0.30)}{}(0.37)\right]$

$= 21.10\,[33.189]$

$A_g = 21.20$ inch^2

$= 805.80$ kips

$\simeq 806$ kips

Prepared by Eng.Maged Kamel.

Use Table 4-22 ⇒ a

$\frac{KL}{r_y} = 47.37$

ASD

$(48 - 47.37) = 0.63$

Table 4-22 (continued)
Available Critical Stress for Compression Members

	$F_y = 35$ ksi			$F_y = 36$ ksi			$F_y = 42$ ksi			$F_y = 46$ ksi			$F_y = 50$ ksi	
$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
41	19.2	28.9	41	19.7	29.7	41	22.7	34.1	41	24.6	37.0	41	26.5	39.8
42	19.2	28.8	42	19.6	29.5	42	22.6	33.9	42	24.5	36.8	42	26.3	39.5
43	19.1	28.7	43	19.6	29.4	43	22.5	33.7	43	24.3	36.6	43	26.2	39.3
44	19.0	28.5	44	19.5	29.3	44	22.3	33.6	44	24.2	36.3	44	26.0	39.1
45	18.9	28.4	45	19.4	29.1	45	22.2	33.4	45	24.0	36.1	45	25.8	38.8
46	18.8	28.3	46	19.3	29.0	46	22.1	33.2	46	23.9	35.9	46	25.6	38.5
47	18.7	28.1	47	19.2	28.9	47	22.0	33.0	47	23.8	35.7	47	25.5	38.3
48	18.6	28.0	48	19.1	28.7	48	21.8	32.8	48	23.6	35.4	48	25.3	38.0
49	18.5	27.9	49	19.0	28.5	49	21.7	32.6	49	23.4	35.2	49	25.1	37.7
50	18.4	27.7	50	18.9	28.4	50	21.6	32.4	50	23.3	35.0	50	24.9	37.5

$A_g = 21.10 \text{ inch}^2$

$\frac{P_n}{\Omega} \simeq 536 \text{ kips}$

$\frac{1}{\Omega_c} P_n = A_g \cdot \frac{F_{cr}}{\Omega} = 21.10 \left[25.30 + \frac{0.20}{1}(0.63) \right]$

$\frac{P_n}{\Omega} = 536.49$

Prepared by Eng.Maged Kamel.

(b) Use Table 4-1 $\left(\frac{KL}{r}\right) = 47.37$ $(KL)_y = 0.8(15) = 12'$

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

$F_y = 50$ ksi

W12

Shape	W12×									
lb/ft	96		87		79		72		65	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
0	844	1270	766	1150	695	1040	632	949	572	859
6	811	1220	736	1110	667	1000	606	911	549	825
7	800	1200	726	1090	657	988	597	898	540	812
8	787	1180	714	1070	646	971	587	883	531	798
9	772	1160	700	1050	634	953	576	866	521	783
10	756	1140	685	1030	620	932	564	847	510	766
11	739	1110	670	1010	606	910	550	827	497	747
12	720	1080	653	981	590	887	(536)	(806)	484	728
13	701	1050	635	954	574	862	521	783	470	707
14	680	1020	616	925	556	836	505	759	456	685
15	659	990	596	896	538	809	489	735	441	663

$(KL)_y$ 12 →

$\phi_c P_n = 806$ Kips

$\frac{P_n}{\Omega_c} = 536$ kips

} matches with Results From Table 4-22

Prepared by Eng.Maged Kamel.

E-3 AISC For Part C

$$P_n = F_{cr}A_g \qquad \text{(AISC Equation E3-1)}$$

$$\phi_c P_n = \phi_c F_{cr} A_g = \text{LRFD compression strength } (\phi_c = 0.90)$$

$$\frac{P_n}{\Omega_c} = \frac{F_{cr}A_g}{\Omega_c} = \text{ASD allowable compression strength } (\Omega_c = 1.67)$$

The following expressions show how F_{cr}, the flexural buckling stress of a column, may be determined for members without slender elements:

(a) If $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$ $\left(\text{or } \frac{F_y}{F_e} \leq 2.25\right)$ → our case

$\left(\frac{KL}{r}\right) <$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y \qquad \text{(AISC Equation E3-2)}$$

(b) If $\frac{KL}{r} > 4.71\sqrt{\frac{E}{F_y}}$ $\left(\text{or } \frac{F_y}{F_e} \leq 2.25\right)$

$$F_{cr} = 0.877F_e \qquad \text{(AISC Equation E3-3)}$$

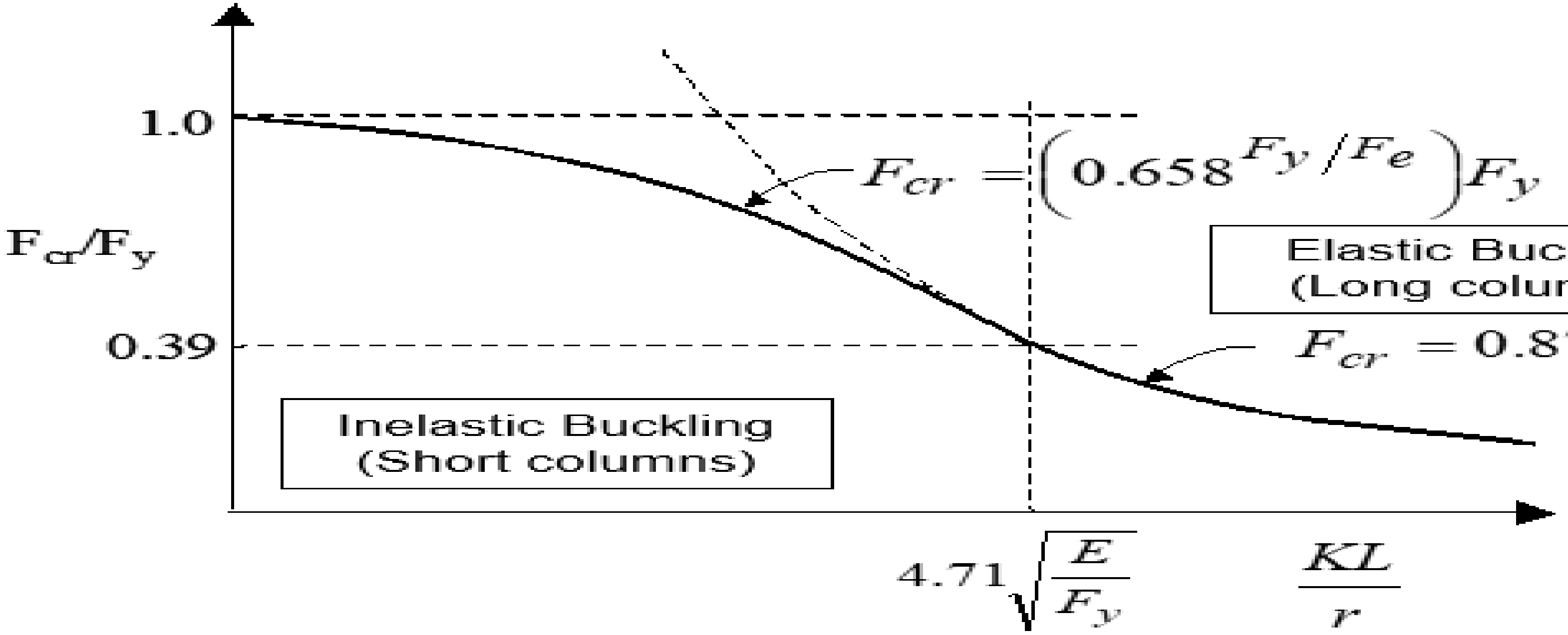
In these expressions, F_e is the elastic critical buckling stress—that is, the Euler stress—calculated with the effective length of the column KL.

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad \text{(AISC Equation E3-4)}$$

(C) check with AISC E-3

$\left(\frac{KL}{r}\right)_y = 47.37$

Find $F_E = \dfrac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \dfrac{(3.1459)^2(29000)}{(47.37)^2} = 127.553$ ksi



$$F_{cr} = 0.658^{\left(F_y/F_E\right)} F_y \qquad A_g = 21.10 \text{ inch}^2$$

$$= 0.658^{\left(\frac{50}{127.552}\right)} (50) = 42.434 \text{ ksi}$$

$$\frac{F_{cr}}{F_y} = \frac{42.434}{50} = 0.8487 > 0.39$$
$$< 1.00$$

$$\phi_c F_{cr} A_g = 0.9(42.434)(21.10) = 805.82 \text{ kips}$$

$$\frac{F_c A_g}{\Omega_c} = \frac{1}{1.67}(42.434)(21.10) = 536.14 \text{ kips}$$