1 Local buckling for Columns

Equation Used for Various types of steel sections

A given HSS Column (short Column)

Solved Problem 5-3 → Short non-slender

Square HSS Section CM#14

a Use Manual Calculation

b Table 4-22

c Table 4-4

Prepared by Eng.Maged Kamel.

CM #14 #2010

E7. MEMBERS WITH SLENDER ELEMENTS

This section applies to slender-element compression members, as defined in Section B4.1 for elements in uniform compression.

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

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

The critical *stress*, F_{cr}, shall be determined as follows:

(a) When $\dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{QF_y}}$ $\left(\text{or } \dfrac{QF_y}{F_e} \le 2.25\right)$ ≤ For short Columns

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y \quad \text{(E7-2)}$$

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}}$ $\left(\text{or } \dfrac{QF_y}{F_e} > 2.25\right)$ ⇒ Long Columns

> than

$$F_{cr} = 0.877 F_e \quad \text{(E7-3)}$$

Q For. Slender Columns

See B-3

(b) For webs of *built-up sections*, h is the distance between adjacent lines of *fasteners* or the clear distance between flanges when welds are used, and h_c is twice the distance from the center of gravity to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.

(c) For flange or *diaphragm plates* in built-up sections, the width, b, is the distance between adjacent lines of fasteners or lines of welds.

Hollow

(d) For flanges of rectangular hollow structural sections (*HSS*), the width, b, is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, h is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, b and h shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, t, shall be taken as the *design wall thickness*, per Section B4.2.

Prepared by Eng.Maged Kamel.

Example 5-3

An HSS $16 \times 16 \times \frac{1}{2}$ with $F_y = 46$ ksi is used for an 18-ft-long column with simple end supports.

(a) Determine $\phi_c P_n$ and $\frac{P_n}{\Omega_c}$ with the appropriate AISC equations.

(b) Repeat part (a), using Table 4-4 in the AISC Manual.

Solution

HSS ⇒ 16x16 x 1/2

Part ⓐ

b/t = 31.40

Check Table 1-12

Table 1-12
Square HSS
Dimensions and Properties

HSS16-HSS8

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	b/t	h/t	I	S	r	Z	Workable Flat	Torsion J	Torsion C	Surface Area
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS16×16×5/8	0.581	127.37	35.0	24.5	24.5	1370	171	6.25	200	13 3/16	2170	276	5.17
×1/2	0.465	103.30	28.3	31.4	31.4	1130	141	6.31	164	13 3/4	1770	224	5.20
×3/8	0.349	78.52	21.5	42.8	42.8	873	109	6.37	126	14 5/16	1350	171	5.23
×5/16	0.291	65.87	18.1	52.0	52.0	739	92.3	6.39	106	14 5/8	1140	144	5.25

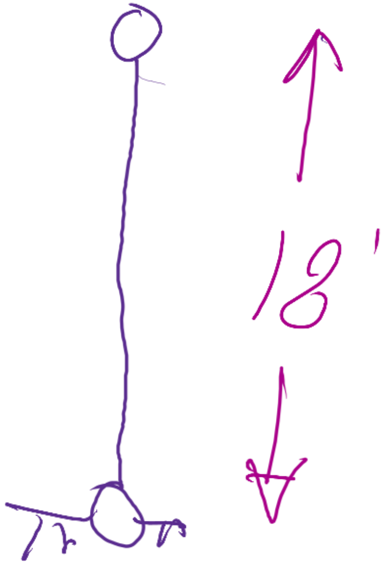
$A = 28.30$ inch^2

$t_{design} = 0.465''$

$r_x = r_y = 6.31''$

Prepared by Eng.Maged Kamel.

Column



HSS → $16'' \times 16'' \times \frac{1}{2}''$

$K = 1$

$L = 18' \Rightarrow 216''$

$r_x = r_y = 6.31''$

$\frac{b}{t}$: From Table ⇒ 31.40

$E = 29000$ ksi

$F_y = 46$ ksi

Check whether Long or Short

$$\left(\frac{KL}{r}\right) = 4.71\sqrt{\frac{E}{F_y}} = 4.71\left(\frac{29000}{50}\right)^{\frac{1}{2}} = 118.26$$

$$\left(\frac{KL}{r}\right)_x = \left(\frac{KL}{r}\right)_y = \frac{(1)(216)}{6.31} = 34.23$$

< 118.26 Column is short

* (b) Check whether slender or not

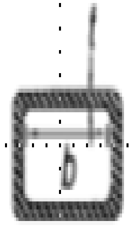
6	Walls of rectangular HSS and boxes of uniform thickness	b/t	$1.40\sqrt{\frac{E}{F_y}}$	

Table B4.1

$$\lambda = \frac{b}{t} = 1.4\sqrt{\frac{29000}{46}} = 35.15$$

short
no slender

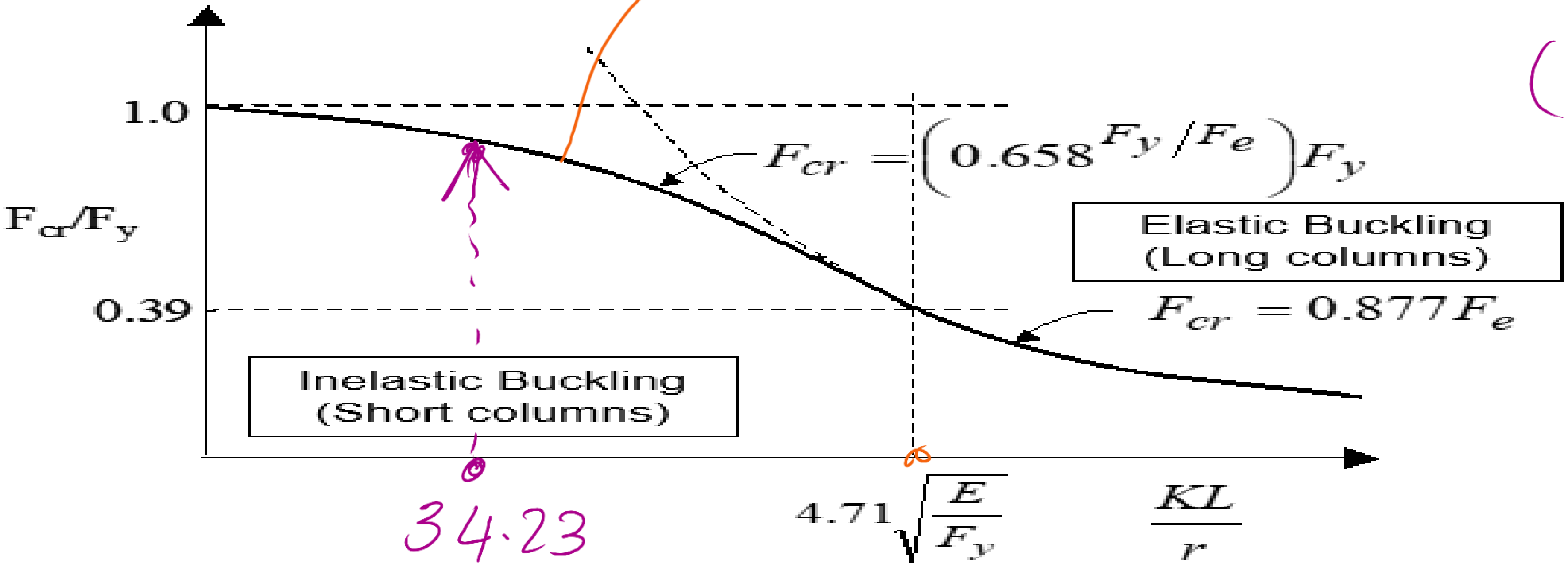
We have b/t For Section $= 34.23 < \lambda_r$

Short

$E = 29000$ ksi

$F_y = 46$ ksi

$\left(\frac{KL}{r}\right)_y = 34.23$



118.26

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(3.14159)^2(29000)}{(34.23)^2} = 244.27 \text{ ksi}$$

$$F_{cr} = \left(0.658^{(F_y/F_E)}\right) F_y \Rightarrow F_y = 46 \text{ ksi}$$

$$F_e = 244.27 \text{ ksi}$$

$$A_g = 28.3 \text{ inch}^2$$

$$F_{cr} = \left(0.658^{\left(\frac{46}{244.27}\right)}\right)(46)$$

$$= 42.51 \text{ Ksi}$$

$$P_n = F_{cr} A_g = (42.51)(28.3)$$

$$= 1203 \text{ Kips}$$

$$\text{LRFD} = \phi_c P_n = 0.9(1203) = 1082.73 \simeq 1083 \text{ Kips}$$

$$\text{ASD} \quad \frac{P_n}{\Omega_c} = 1203/1.67 = 720.35 \simeq 720 \text{ Kips}$$

Table 4-22. Available Critical Stress for Compression Members

Table 4-22 provides the available critical stress for various ratios of Kl/r, for materials with a minimum specified yield strength of 35 ksi, 36 ksi, 42 ksi, 46 ksi and 50 ksi.

Given data $F_y = 46$ ksi $\frac{KL}{r} = 34.32$

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
30	20.0	30.1	30	20.6	30.9	30	23.8	35.8	30	25.9	39.0	30	28.0	42.1
31	20.0	30.0	31	20.5	30.8	31	23.7	35.6	31	25.8	38.8	31	27.9	41.9
32	19.9	29.9	32	20.4	30.7	32	23.6	35.5	32	25.7	38.6	32	27.8	41.8
33	19.8	29.8	33	20.4	30.6	33	23.5	35.4	33	25.6	38.5	33	27.7	41.6
34	19.8	29.7	34	20.3	30.5	34	23.4	35.2	34	25.5	38.3	34	27.5	41.4
35	19.7	29.6	35	20.2	30.4	35	23.3	35.1	35	25.4	38.1	35	27.4	41.2
36	19.6	29.5	36	20.1	30.3	36	23.2	34.9	36	25.2	37.9	36	27.2	40.9
37	19.5	29.4	37	20.1	30.1	37	23.1	34.8	37	25.1	37.8	37	27.1	40.7
38	19.5	29.3	38	20.0	30.0	38	23.0	34.6	38	25.0	37.6	38	26.9	40.5
39	19.4	29.1	39	19.9	29.9	39	22.9	34.4	39	24.9	37.4	39	26.8	40.3
40	19.3	29.0	40	19.8	29.8	40	22.8	34.3	40	24.7	37.2	40	26.6	40.0

LRFD

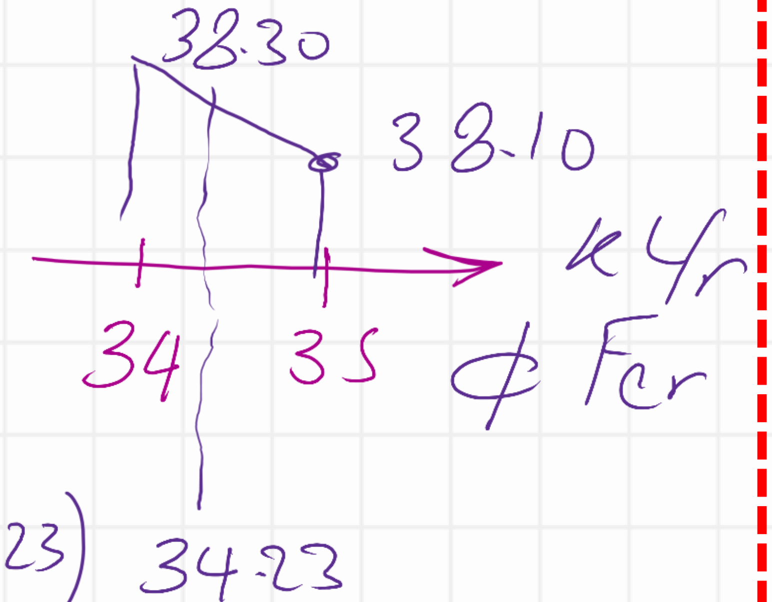
$\left(\frac{KL}{r}\right) = 34.23$

$\phi_c F_{cr}$ at $\frac{KL}{r} = 34$

$= 38.30$ ksi

while $\phi_c F_{cr}$ at $\frac{KL}{r} = 35$

$= 38.10$ ksi



$\phi_c F_{cr} = 38.10 + \underline{(0.20)}(35 - 34.23)$

$= 38.254$ ksi

$\phi_c F_{cr} A_g = 38.254(28.3) \Rightarrow 1082.59$ kips

ASD

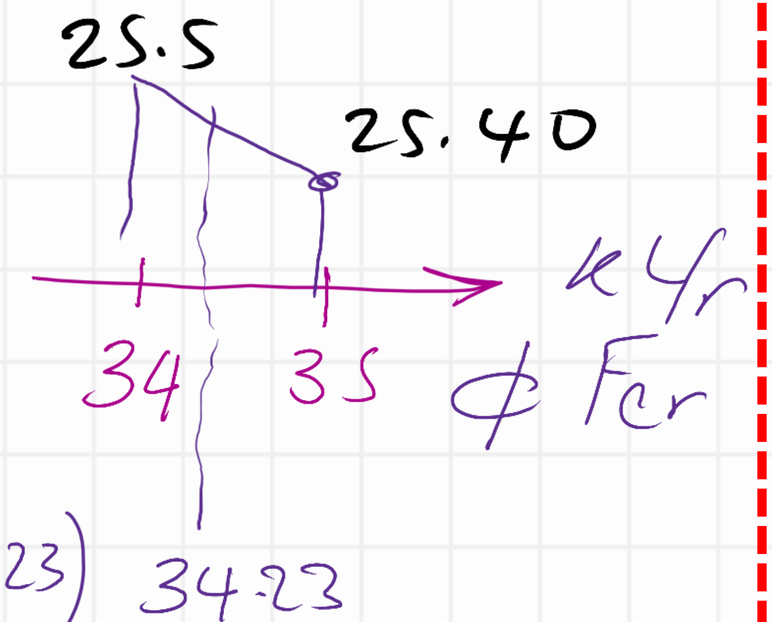
$\left(\frac{KL}{r}\right) = 34.23$

$\frac{1}{\Omega_c} F_{cr}$ at $\frac{KL}{r} = 34$

$= 25.50$ ksi

while $\frac{1}{\Omega_c} F_{cr}$ at $\frac{KL}{r} = 35$

$= 25.40$ ksi



$\phi_c F_{cr} = 25.40 + (0.10)(35 - 34.23)$

$= 25.477$ ksi

$\phi_c F_{cr} A_g =$ $(28.3) \Rightarrow \boxed{721}$ kips

Table 4-3. Rectangular HSS in Axial Compression

Available strengths in axial compression are given for rectangular HSS with $F_y = 46$ ksi (ASTM A500 Grade B). The tabulated values are given for the effective length with respect to the y-axis, $(KL)_y$. However, the effective length with respect to the x-axis $(KL)_x$ must also be investigated. To determine the available strength in axial compression, the table should be entered at the larger of $(KL)_y$ and $(KL)_{y\,eq}$, where

$$(KL)_{y\,eq} = \frac{(KL)_x}{\frac{r_x}{r_y}} \qquad (4\text{-}1)$$

Values of the ratio r_x/r_y and other properties useful in the design of rectangular HSS compression members are listed at the bottom of Table 4-3.

Table 4-4. Square HSS in Axial Compression

Table 4-4 is similar to Table 4-3, except that it covers square HSS.

Use $(KL)_{y\,req} = \dfrac{(KL)_x}{\frac{rx}{ry}}$

in our Example $r_x = r_y = 6.31$

$(KL)_y = \dfrac{(KL)_x}{1}$ $\qquad (KL)_x = (KL)_y =$

Given data

HSS 16x16x $\frac{1}{2}$"

$(KL)_y = 18'$

Table 4-4
Available Strength in Axial Compression, kips
Square HSS

HSS16–HSS14

F_y = 46 ksi

Fy

$\phi_c P_n = 1080$ kips

Shape	HSS16×16× 1/2		HSS16×16× 3/8[c]		HSS16×16× 5/16[c]		HSS14×14× 5/8		HSS14×14× 1/2		HSS14×14× 3/8[c]	
t_{design}, in.	0.465		0.349		0.291		0.581		0.465		0.349	
lb/ft	103		78.5		65.9		110		89.7		68.3	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
10	761	1140	513	772	376	566	808	1210	656	986	487	733
11	757	1140	512	769	375	564	802	1210	652	980	485	729
12	753	1130	510	767	374	563	796	1200	647	972	483	726
13	748	1120	508	764	373	561	790	1190	642	965	480	722
14	743	1120	506	761	372	559	783	1180	636	956	477	718
15	738	1110	504	758	371	557	775	1170	630	947	474	713
16	732	1100	502	755	370	555	768	1150	624	938	471	708
17	727	1090	500	751	368	553	759	1140	618	928	468	703
18	720	1080	497	747	367	551	751	1130	611	918	464	697
19	714	1070	495	743	365	549	742	1110	603	907	460	691
20	707	1060	492	739	363	546	732	1100	596	896	454	683

spect to least radius of gyration, r_y

$P_n/\Omega_c = 720$ kips