(1) Local buckling for Columns

Equation Used for Various types of steel sections

A given HSS Column (short Column)

Example 6-8 Mccormac

Cm #14 AISC-360-10

$F_y = 46\ ksi$

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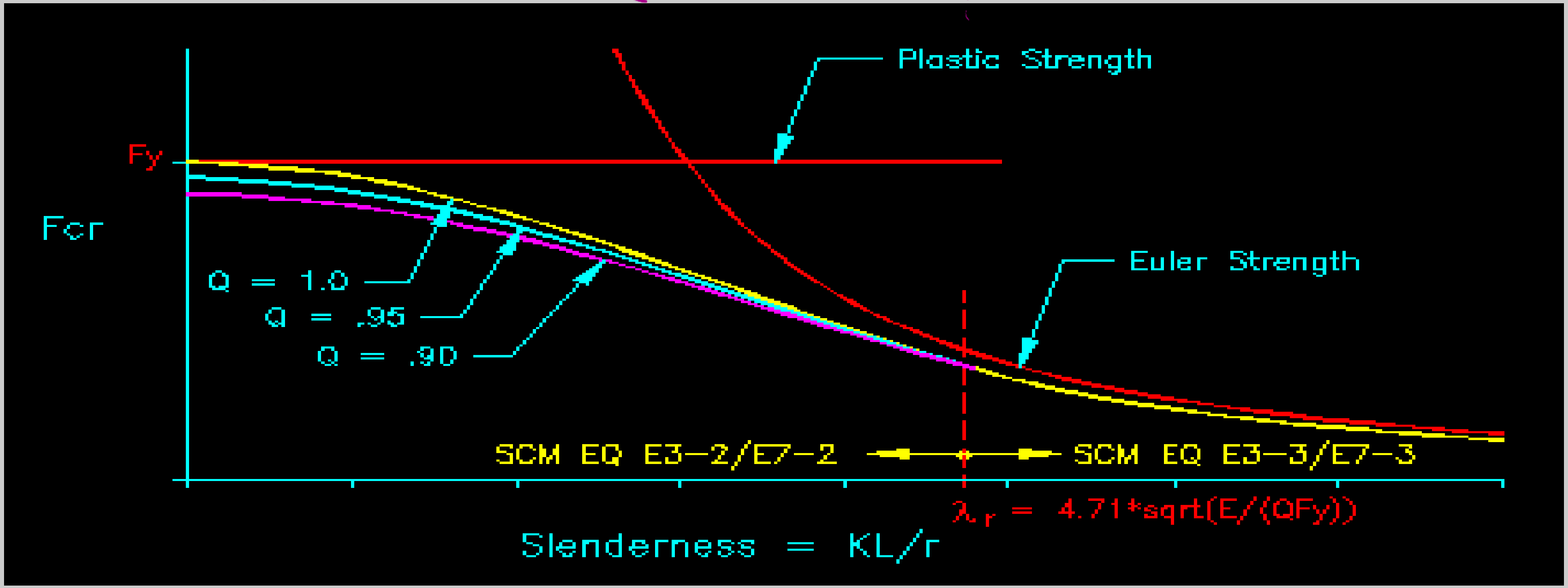
"2010" $\varphi = \varphi_a \cdot \varphi_s$

Figure 7.4.1

F_{cr} for Slender Sections

Click on image for larger view

Short ← → Long



E3-2 Short not slender

E7-2 short slender

$\lambda_r = 4.71 \sqrt{\dfrac{E}{\varphi F_y}}$

E3-3 Long not slender

E7-3 Long slender

Chapter E

This chapter addresses members subject to axial compression through the centroidal axis.

The chapter is organized as follows:

E3 ⇒ Short or Long } Not slender

E7 → slender members

$\lambda_F > \lambda_{Fr}$

$\lambda_w > \lambda_{wr}$

Short or Long

TABLE USER NOTE E1.1
Selection Table for the Application of Chapter E Sections

Cross Section	Without Slender Elements: Sections in Chapter E	Without Slender Elements: Limit States	With Slender Elements: Sections in Chapter E	With Slender Elements: Limit States
 TB	E3 E4	FB TB	E7	LB FB TB
	E3 E4	FB FTB	E7	LB FB FTB

LB: Local buckling

FB: Flange buckling

TB Torsional buckling

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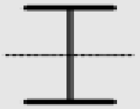
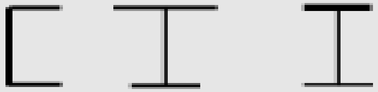
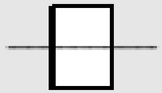
TABLE USER NOTE E1.1 Selection Table for the Application of Chapter E Sections				
	Without Slender Elements		With Slender Elements	
Cross Section	Sections in Chapter E	Limit States	Sections in Chapter E	Limit States
	E3 E4	FB TB	E7	LB FB TB
	E3 E4	FB FTB	E7	LB FB FTB
	E3	FB	E7	LB FB

if we have I section with no TB

Either

Long or Short

Equations in E3

Follow No slender

slender

Long or short

Follow

Equations in E 7

Check LB Parameters

FB

Tables

Unstiffened table 1-4

E7- Part #1

Chapter-B

Limiting r For no slender

Unstiffened Flange 4 cases

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_r (nonslender/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	$0.64\sqrt{\frac{k_cE}{F_y}}$ [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	$0.75\sqrt{\frac{E}{F_y}}$	

Rolled I — b/t

Built up — b/t

Angles — b/t

T — d/t

$0.56\sqrt{\frac{E}{f_y}}$

$0.64\sqrt{\frac{k_c E}{f_y}}$

$0.45\sqrt{\frac{E}{f_y}}$

$0.75\sqrt{\frac{E}{f_y}}$

Limiting r values

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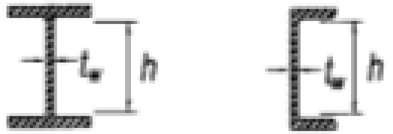
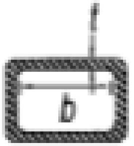
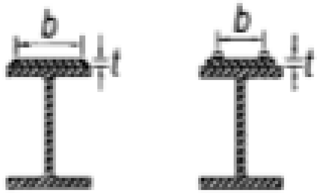
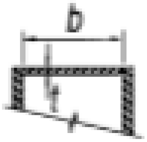
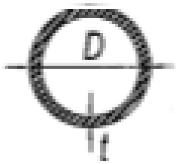
Stiffened Elements   5 → 9   16.1 – 17   AISC
14 ths

Stiffened Elements	5	Webs of doubly-symmetric I-shaped sections and channels	h/t_w	$1.49\sqrt{\frac{E}{F_y}}$	
	6	Walls of rectangular HSS and boxes of uniform thickness	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	$0.11\frac{E}{F_y}$	

webs

walls

Flange Cover

Other stiffened

Round HSS

↓ Flexure

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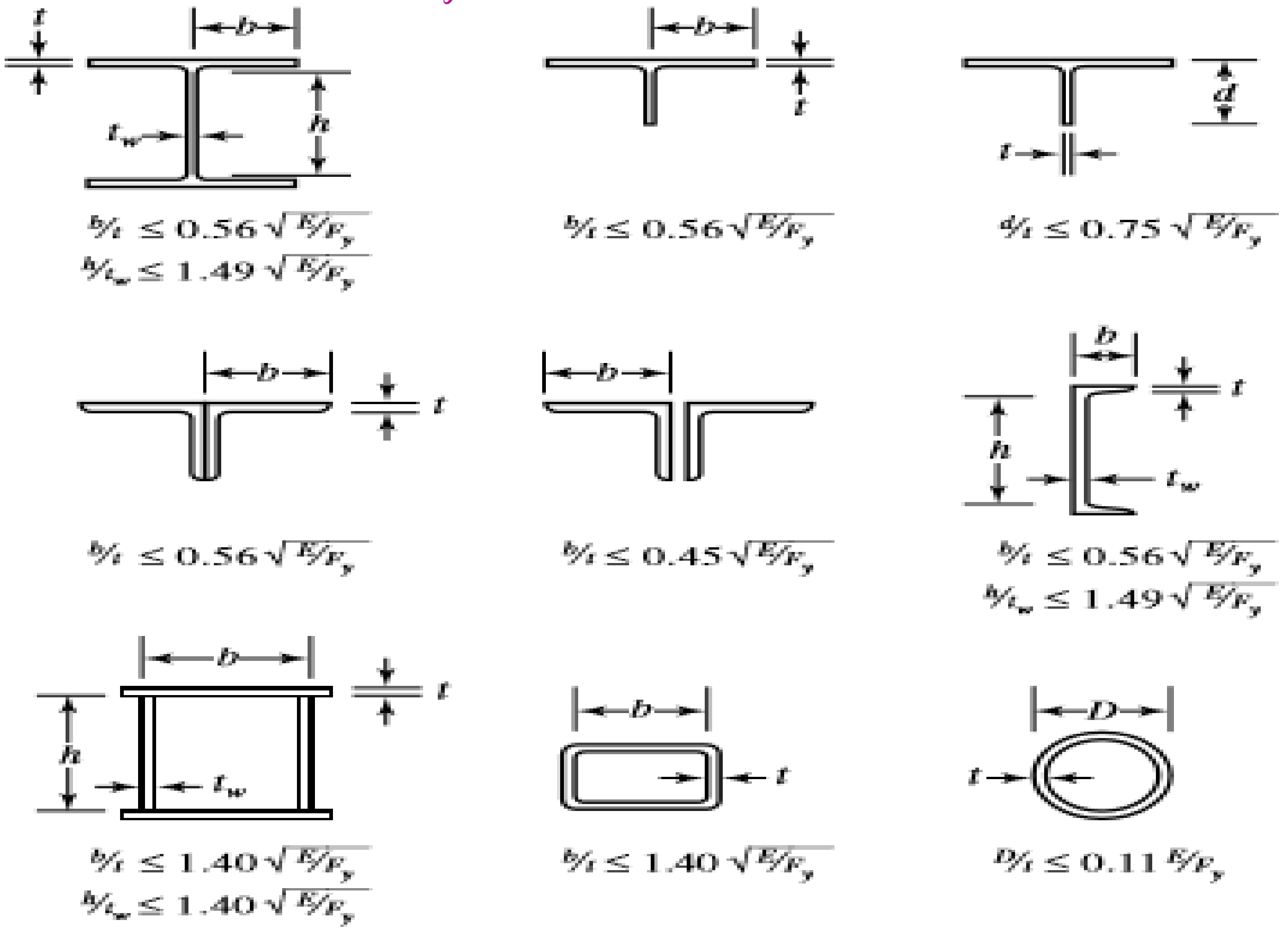
Brief description of Sections

Prof SEGui

No slender Columns

FIGURE 4.9



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

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

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

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

$>$ than

For. Slender Columns

$Q \to Q_s$

$\quad \to Q_a$

where

F_e = elastic *buckling stress*, calculated using Equations E3-4 and E4-4 for doubly symmetric members, Equations E3-4 and E4-5 for singly symmetric members, and Equation E4-6 for unsymmetric members, except for single angles with $b/t \leq 20$, where F_e is calculated using Equation E3-4, ksi (MPa)

Q = net reduction factor accounting for all slender compression elements;

= 1.0 for members without slender elements, as defined in Section B4.1, for elements in uniform compression

= Q_sQ_a for members with *slender-element sections*, as defined in Section B4.1, for elements in uniform compression.

User Note: For cross sections composed of only unstiffened slender elements, $Q = Q_s$ ($Q_a = 1.0$). For cross sections composed of only stiffened slender elements, $Q = Q_a$ ($Q_s = 1.0$). For cross sections composed of both stiffened and unstiffened slender elements, $Q = Q_sQ_a$. For cross sections composed of multiple unstiffened slender elements, it is conservative to use the smaller Q_s from the more slender element in determining the member strength for pure compression.

Qa : Slender stiffened → webs, $\square$

2. **Slender Stiffened Elements, Q_a**

The reduction factor, Q_a , for slender *stiffened elements* is defined as follows:

$$Q_a = \frac{A_e}{A_g} \qquad \text{(E7-16)}$$

where

A_g = gross cross-sectional area of member, in.^{2} (mm^2)

A_e = summation of the effective areas of the cross section based on the reduced *effective width*, b_e, in.^{2} (mm^2)

All Sections Except HSS

The reduced effective width, b_e, is determined as follows:

(a) For uniformly compressed slender elements, with $\frac{b}{t} \geq 1.49\sqrt{\frac{E}{f}}$, except flanges of square and rectangular sections of uniform thickness:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \qquad \text{(E7-17)}$$

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$

(b) For flanges of square and rectangular *slender-element sections* of uniform thickness with $\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}}$:

item 7 Table B4-1

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \qquad \text{(E7-18)}$$

Case of HSS

where

$f = P_n/A_e$

User Note: In lieu of calculating $f = P_n/A_e$, which requires iteration, f may be taken equal to F_y. This will result in a slightly conservative estimate of *column available strength*.

Example 6-8

#2010

Determine the axial compressive design strength $\phi_c P_n$ and the allowable design strength $\frac{P_n}{\Omega_c}$ of a 24-ft HSS $14 \times 10 \times \frac{1}{4}$ column section. The base of the column is considered to be fixed, while the upper end is assumed to be pinned. $F_y = 46$ ksi.

Solution

Table 1-11

$K_x = 0.80$

$K_y = 0.80$

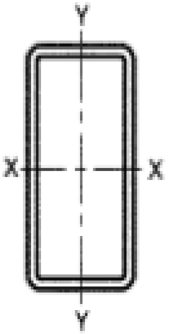


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	b/t	h/t	Axis X-X			
						I	S	r	Z
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS14×10×5/8	0.581	93.34	25.7	14.2	21.1	687	98.2	5.17	120
×1/2	0.465	76.07	20.9	18.5	27.1	573	81.8	5.23	98.8
×3/8	0.349	58.10	16.0	25.7	37.1	447	63.9	5.29	76.3
×5/16	0.291	48.86	13.4	31.4	45.1	380	54.3	5.32	64.6
×1/4	0.233	39.43	10.8	39.9	57.1	310	44.3	5.35	52.4

Area = 10.80 inch^2

$\frac{b}{t} = 39.90$ Flang part

$h/t = 57.1$ web part

$I_x = 310$ inch^4

$r_x = 5.35''$

Prepared by Eng.Maged Kamel.

$HSS \Rightarrow 14'' \times 10'' \times \frac{1}{4}''$

Second Part of Table 1-11

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS14-HSS12

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	S	r	Z	Depth	Width	J	C	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS14×10×5/8	407	81.5	3.98	95.1	11 3/16	7 3/16	832	146	3.83
×1/2	341	68.1	4.04	78.5	11 3/4	7 3/4	685	120	3.87
×3/8	267	53.4	4.09	60.7	12 5/16	8 5/16	528	91.8	3.90
×5/16	227	45.5	4.12	51.4	12 9/16	8 9/16	446	77.4	3.92
×1/4	186	37.2	4.14	41.8	12 7/8	8 7/8	362	62.6	3.93

$r_y = 4.14''$

$I_y = 186$ inch4

① Check whether Column is Long or Short

$F_y = 46$ ksi

$K = 0.80$

$L = 24' \Rightarrow 288''$

$r_y = 4.14$

$r_x = 5.35''$

r_y Governs

$r_y > r_x$

$\frac{b}{t}$ h/t

$$\left(\frac{KL}{r}\right)_y = \frac{(0.80)(288)}{4.14} = 55.652$$

$$4.71\sqrt{\frac{E}{F_y}} \Rightarrow 4.71\sqrt{\frac{29000}{46}} = 118.26$$

Since $\left(\frac{KL}{r_y}\right)_y < 4.71\sqrt{\frac{E}{F_y}}$

Column is Short

Find Euler stress $F_E = \dfrac{\pi^2 E}{\left(\frac{KL}{r}\right)_y^2} = \dfrac{\pi^2(29000)}{(55.65)^2}$

$F_e = 92.42$ ksi

$\pi = 3.14159$, $E = 29000$ ksi

$K = 0.80$

$L = 24' \Rightarrow 288''$

$r_y = 4.14$

$r_x = 5.35''$

r_y Governs

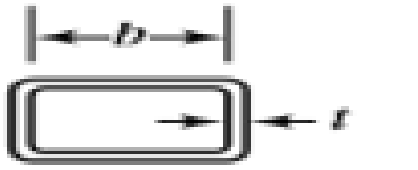
$r_y > r_x$

Short → Slender
or no slender

$\lambda_r = 1.40\sqrt{\dfrac{E}{F_y}} = 1.4\sqrt{\dfrac{29000}{46}} = 35.15$

$h/t = 57.10 > 35.15$

$Q_A < 1$



$b/t \le 1.40\sqrt{E/F_y}$

No slender

$> 1.4\sqrt{E/F_y}$ Slender

Refer to B4.1B → Stiffened

How to get b, h

(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.

$\left\{ \begin{array}{l} b \Rightarrow \text{in Estimate} = b - 3t_w \\ h \rightarrow \qquad\qquad\quad = h - 3t_w \end{array} \right.$

outside dimensions

b/t = 39.9 & h/t = 57.10 , t = 0.233"

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$

(b) For flanges of square and rectangular *slender-element sections* of uniform thickness with $\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}}$:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1-\frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \qquad \text{(E7-18)}$$

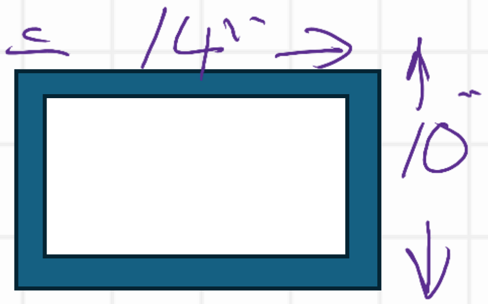
where

$f = P_n/A_e$

b/t = 39.9

User Note: In lieu of calculating $f = P_n/A_e$, which requires iteration, f may be taken equal to F_y. This will result in a slightly conservative estimate of *column available strength*.

(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.



$t_w = \frac{1}{4}"$

$t_d = 0.233"$

$h = 14 - 3(0.233) = 13.301"$

$b = 10 - 3(0.233) = 9.301"$

$$b_e = 1.92(0.233)\sqrt{\frac{29000}{46}}\left[1 - \frac{0.38}{39.9}\sqrt{\frac{29000}{46}}\right]$$

$= 8.55" < 9.30"$

$h = 13.30"$

$b = 9.30"$

$t_d = 0.233$ $\quad b/t = 39.9$ $\quad \left\{ h = 13.30'', \; t_d = 0.233'' \right.$

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$

(b) For flanges of square and rectangular *slender-element sections* of uniform thickness with $\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}}$:

$h/t = 57.9 \Rightarrow$

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \qquad \text{(E7-18)}$$

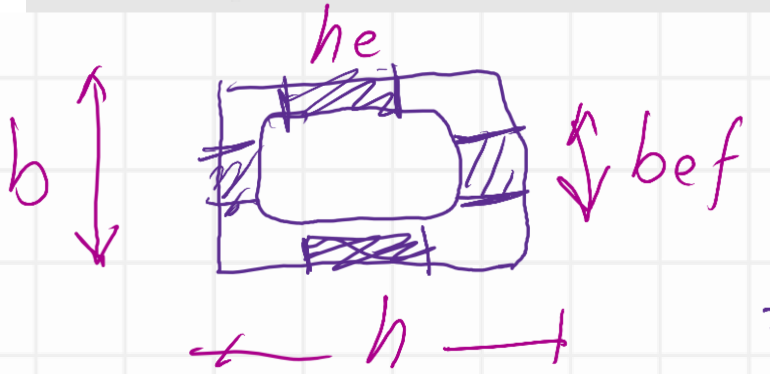
$$h_e = 1.92(0.233)\left(\sqrt{\frac{29000}{46}}\right)\left[1 - \frac{0.38}{57.9}\sqrt{\frac{29000}{46}}\right]$$

$$h_e = 9.36'' < 13.30'' \Rightarrow (h - t_d)$$

where

$f = P_n/A_e$

User Note: In lieu of calculating $f = P_n/A_e$, which requires iteration, f may be taken equal to F_y. This will result in a slightly conservative estimate of *column available strength*.



Area not used

$$2(b - b_e)\,t_d = 2[9.3 - 8.55)(0.233)$$
$$+\,2(h - h_e)\,t_d = 2[13.3 - 9.36)(0.233)$$
$$= 0.3495 + 1.836 \Rightarrow 2.1855 \text{ inch}^2$$

$A_g = 10.80 \text{ inch}^2$

$A_{not\ used} = 2.1855 \text{ inch}^2$

$A_e = 10.8 - 2.1855 \simeq 8.61 \text{ inch}^2 \Big\} \quad Q_s = 1, \quad Q = Q_s Q_A$

$$Q = Q_a = \frac{A_e}{A_g} = \frac{8.61}{10.80} = 0.7972$$

$F_e = 92.42$ Ksi

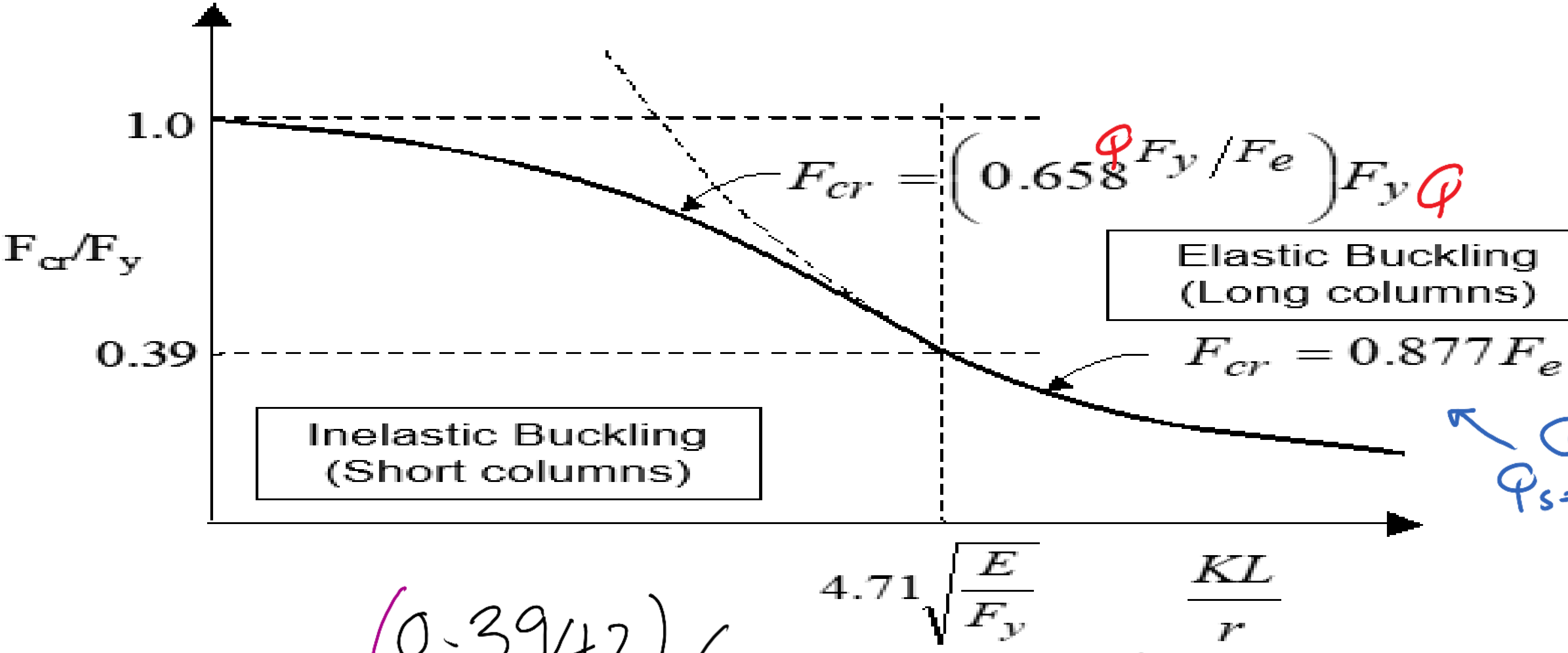
$F_y = 46$ Ksi

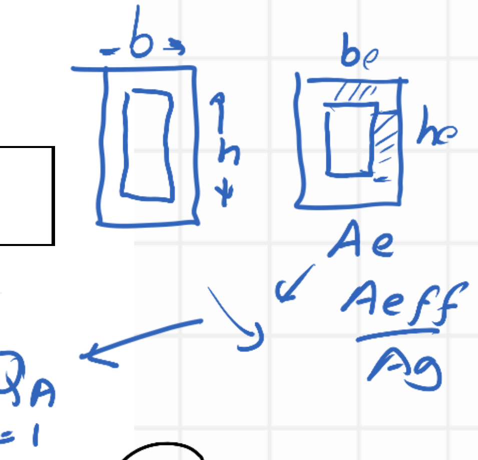
$$\Big\} \; Q \frac{F_y}{F_e} = 0.7972\left(\frac{46}{92.42}\right) = 0.3968$$

$Q = 0.7972$, $F_{cr} = 0.658^{Q F_y/F_e}(Q F_y)$

$F_e = 92.42$ ksi

$F_y = 46$ ksi





$Q_s = 1$

$F_{cr} = 0.658^{(0.3942)}(0.7972)(46) = 31.06$ ksi

$$P_n = F_{cr} \cdot A_g$$

$$= 31.06\,(10.80) = 335.45$$

$$P_n \simeq 335.50 \text{ Kips}$$

$$LRFD = \phi P_n = 0.90(335.5) = 301.90 \text{ Kips}$$

$$ASD = \frac{P_n}{\Omega_c} = 335.5/1.67 = 200.9 \text{ Kips}$$

Prepared by Eng.Maged Kamel.