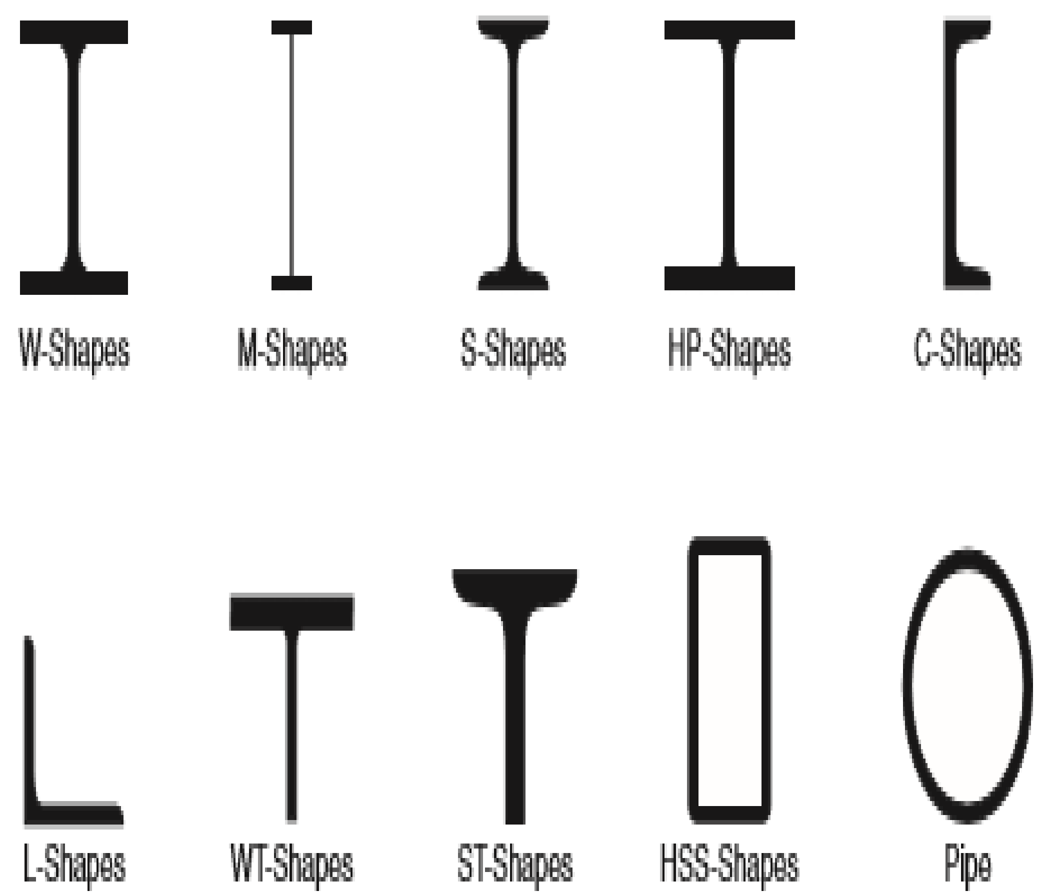
Shape	Steel Type		
	ASTM Designation	F_y, ksi	F_u, ksi
Wide flanged beams	A992	50–65	65
Miscellaneous beams	A36	36	58–80
Standard beams	A36	36	58–80
Bearing piles	A572 Gr. 50	50	65
Standard channels	A36	36	58–80
Miscellaneous channels	A36	36	58–80
Angles	A36	36	58–80
Ts cut from W-shapes	A992	50–65	65
Ts cut from M-shapes	A36	36	58–80
Ts cut from S-shapes	A36	36	58–80
Hollow structural sections, rectangular	A500 Gr. B	46	58
Hollow structural sections, square	A500 Gr. B	46	58
Hollow structural sections, round	A500 Gr. B	42	58
Pipe	A53 Gr. B	35	60

Note: F_y = specified minimum yield stress; F_u = specified minimum tensile strength



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Early editions of the LRFD *Specification* defined the exponent of Equation E3-2 in a slightly different form that makes the presentation a bit simpler. If a new term is defined such that

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

$\lambda_c^2 \rightarrow$ used as a ratio between F_y & F_e

$$\lambda_c^2 = \frac{F_y}{F_e}$$

$$F_e = \frac{\pi^2 E r^2}{L_e^2} = \frac{\pi^2 E (r^2)}{L_{cr}^2}$$

then the dividing point between elastic and inelastic behavior, where

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

when $L_e = L_{cr}$

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

becomes

$$\lambda_c = \frac{L_c}{\pi r}\sqrt{\frac{F_y}{E}} = \frac{4.71}{\pi} = 1.5$$

$$F_e = \frac{\pi^2 E}{1}\left(\frac{r^2}{L_c^2}\right) = \frac{\pi^2 E}{\left(4.71\sqrt{\frac{E}{F_y}}\right)^2}$$

By substituting $\lambda_c^2 = F_y/F_e$, the critical flexural buckling stress for $\lambda_c \leq 1.5$ becomes

$$F_c = 0.658^{\lambda^2} F_y$$

$$F_{cr} = \left(0.658^{\lambda_c^2}\right)F_y \qquad (5.10)$$

$$F_e = \frac{\pi^2 \cancel{E}}{(4.71)^2 \cancel{E}} \cdot F_y$$

and for $\lambda_c > 1.5$,

$$F_c = 0.877 F_e \rightarrow$$

$$F_{cr} = \frac{0.877}{\lambda_c^2} F_y \qquad (5.11)$$

$$= 0.877 \frac{F_y}{\lambda_c^2}$$

$$\lambda_c^2 = \frac{F_y}{F_e} = \frac{4.71^2}{\pi^2} = 2.25$$

$\pi = 3.14159 \rightarrow \lambda_c = 1.5$

Unified design of steel structure

Chapter 5

$\lambda_c^2 = \frac{F_y}{F_e} = 2.25$

$F_e = \frac{F_y}{\lambda_c^2}$

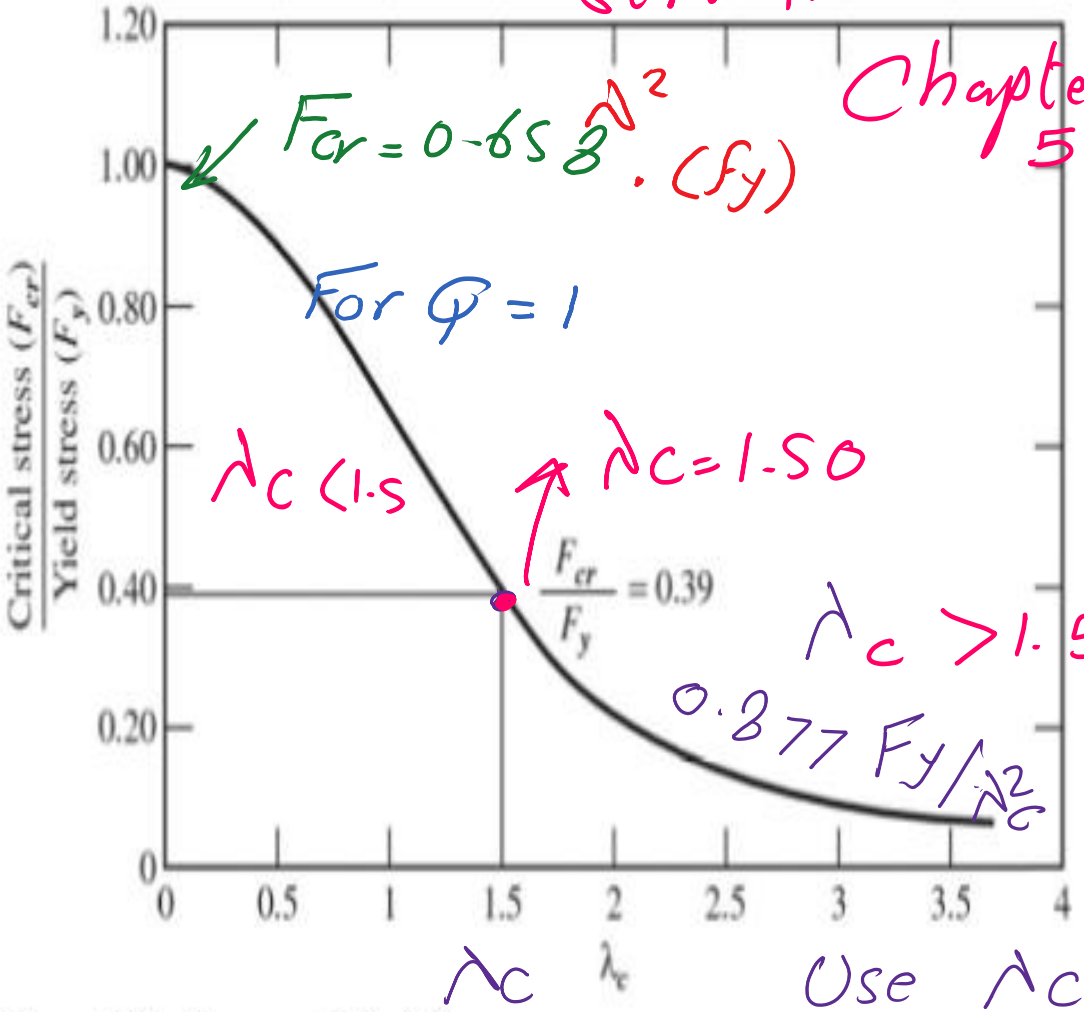
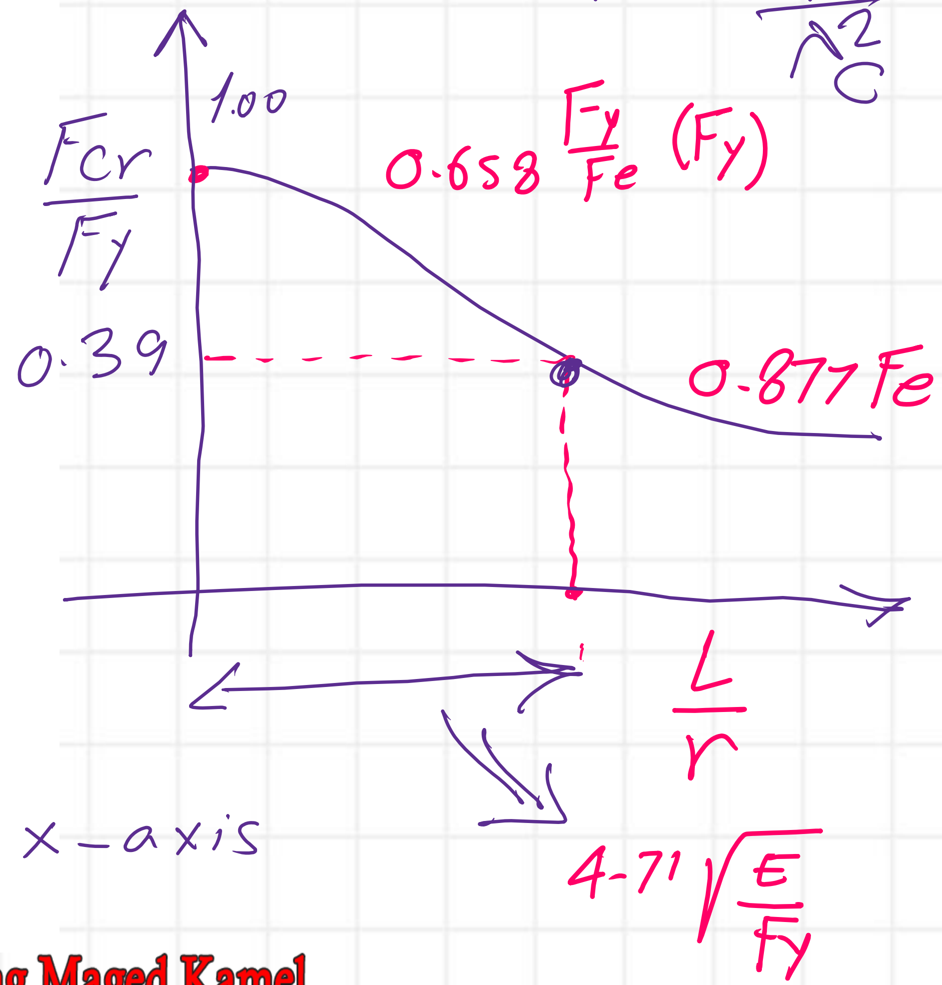


Figure 5.16 λ_c versus Critical Stress



Use λ_c as x-axis

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\# 2010 CM \# 14

EXAMPLE 5.10 ***Strength of Column with Slender Elements***	**Goal:**	Determine the available strength of a compression member with a slender web.
	Given:	Use a W16×26 as a column with L_{cy} = 5.0 ft.

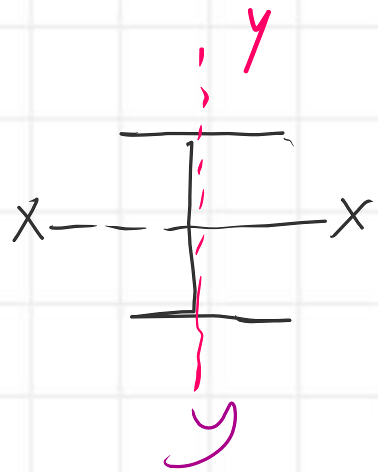
Solution :

A992 steel → $F_y = 50$ Ksi;

① Check Column whether → Elastic
→ inelastic

$$\frac{KL}{r} \Rightarrow \text{or } \frac{L_e}{r} = 4.71\sqrt{E/F_y} = 4.71\sqrt{\frac{29000}{50}} = 113.43$$

From Table 1-1 For W16x26



$L_{cy} = 5.00$ FT $r_y = 1.12$ inch^3

$$\left(\frac{KL}{r}\right)_y \rightarrow \frac{L_{cy}}{r_y} = \frac{5(12)}{1.12} = 53.57 \Rightarrow \text{inelastic}$$

< 113.43

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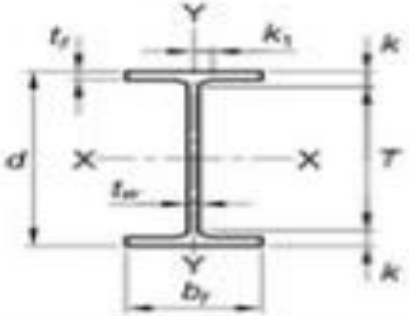

Table 1-1 (continued)
W-Shapes
Dimensions

Part-1

$A_g = 7.68$ inch2

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W16×31[c]	9.13	15.9	$15\frac{7}{8}$	0.275	$\frac{1}{4}$	$\frac{1}{8}$	5.53	$5\frac{1}{2}$	0.440	$\frac{7}{16}$	0.842	$1\frac{1}{8}$	$\frac{3}{4}$	$13\frac{5}{8}$	$3\frac{1}{2}$
×26[c,v]	7.68	15.7	$15\frac{3}{4}$	0.250	$\frac{1}{4}$	$\frac{1}{8}$	5.50	$5\frac{1}{2}$	0.345	$\frac{3}{8}$	0.747	$1\frac{1}{16}$	$\frac{3}{4}$	$13\frac{5}{8}$	$3\frac{1}{2}$

[c] Shape is slender for compression with $F_y = 50$ ksi. C

[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.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi.

W16×26 with Footnote C Shape is slender

section is slender For Compression

Prepared by Eng.Maged Kamel.

Table 1-1 (continued)
W-Shapes
Properties

W18–W16

d: 15.7"

h

t_w

$r_y = 1.12$ inch

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}
31	6.28	51.6	375	47.2	6.41	54.0	12.4	4.49	1.17	7.03	1.42	15.5	0.461	739
26	7.97	56.8	301	38.4	6.26	44.2	9.59	3.49	1.12	5.48	1.38	15.4	0.262	565

W16x26

$\frac{b_f}{2t_f} = 7.97$

$h/t_w = 56.80$

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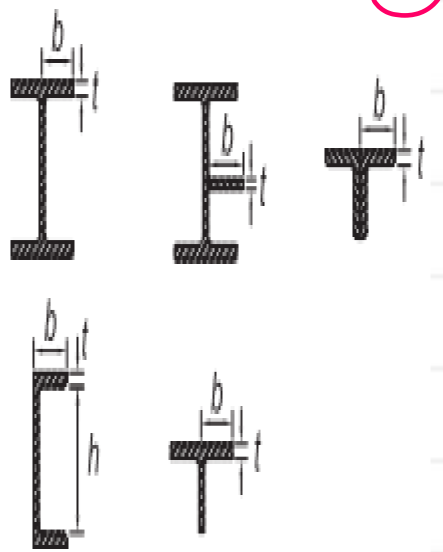
Case 1 For Flange

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

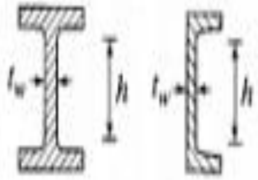
$\frac{b_f}{2t_f} = 7.97$ From Table section $\rightarrow$ $\frac{b}{t} = 0.56\sqrt{\frac{E}{F_y}}$

$\frac{b}{t} = 13.49$

$\frac{b_f}{2t_f} < b/t$ Table B4.1a

Non-Slender Flange

Case 5 For web

Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r (nonslender / slender)	Examples
5	Webs of doubly symmetric I-shaped sections and channels	h/t_w	$1.49\sqrt{\frac{E}{F_y}}$	

From Table 1-1 $\frac{h}{t_w} = 56.80$

$\lambda_{rw} = \left(\frac{h}{t_w}\right) = 1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29000}{50}} = 35.88$

$h/t_w >$ $\rightarrow$ slender web section

Prepared by Eng.Maged Kamel.

(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:

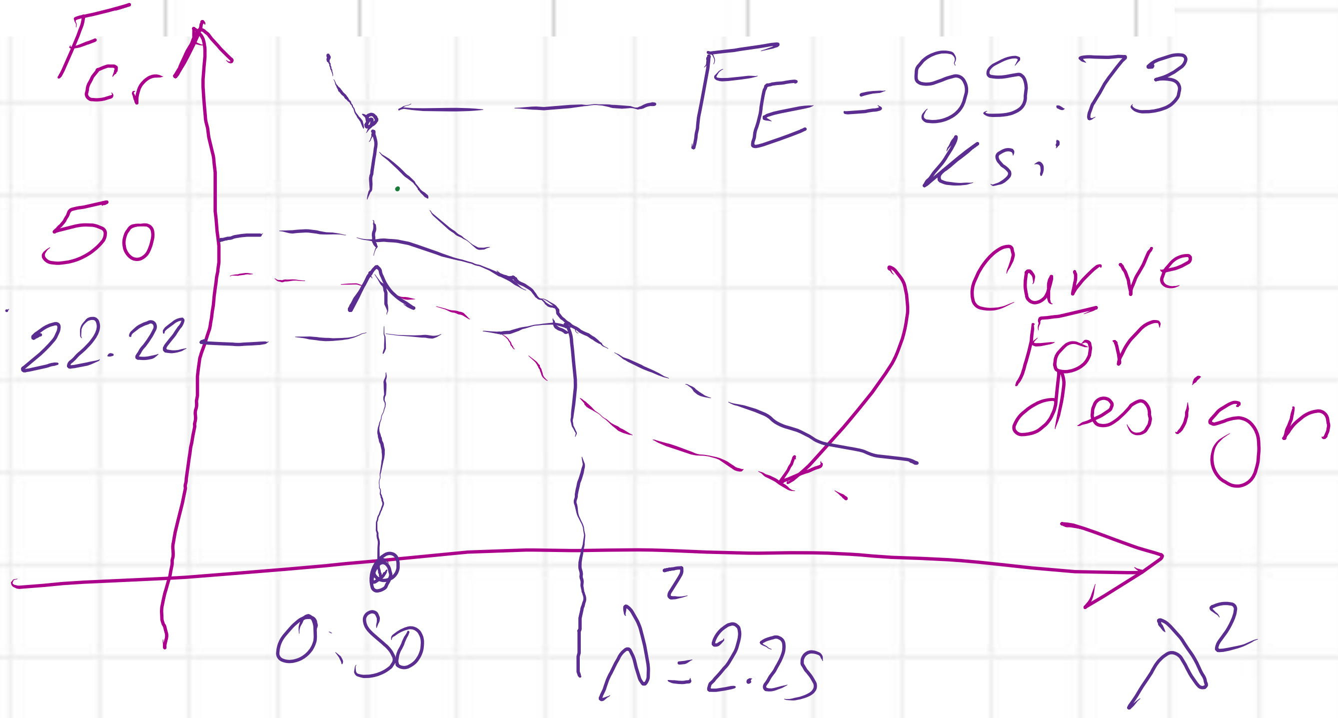
$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \qquad \text{(E7-17)}$$

$$F_E = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)_y^2} = \frac{\pi^2 (29000)}{(53.57)^2}$$

$$= 99.73 \text{ ksi}$$

$$\lambda_c^2 = \frac{F_y}{F_E}$$

$$= 50/99.73$$
$$= 0.50$$



(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:

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

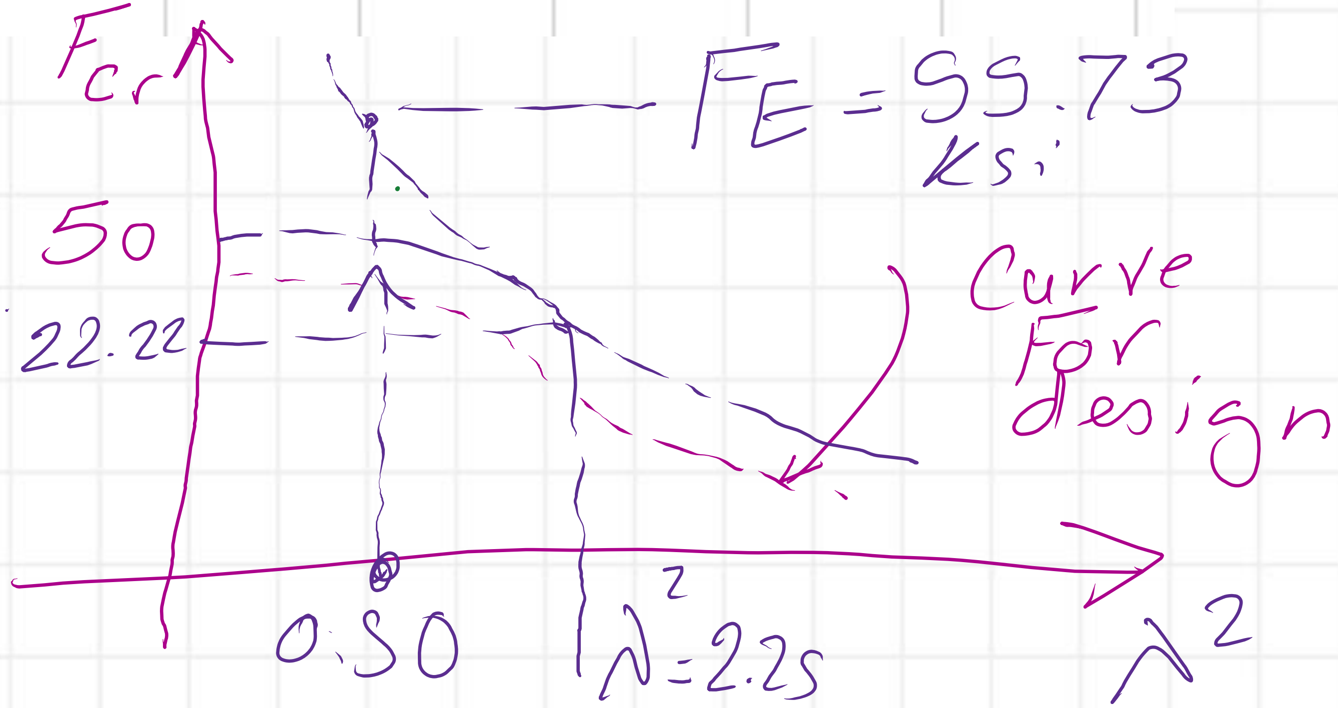
$Q = 1$

$\lambda_c^2 < 2.25$

$F_{cr} = 0.658^{\lambda_c^2} F_y Q$

$= 0.658^{(0.50)} (50)\,1$

$f_{cr} = 40.56$ ksi

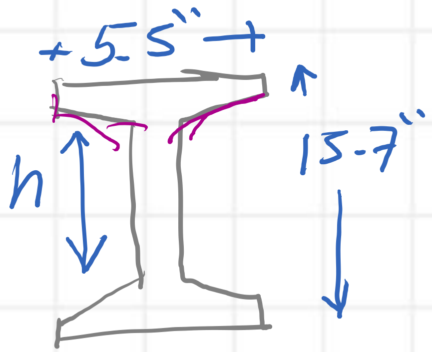


(a) For uniformly compressed slender elements, with $\frac{b}{t} \geq 1.49\sqrt{\frac{E}{f}}$, except flanges of square and rectangular sections of uniform thickness:

web

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

kdes = 0.747"



$E = 29000$ ksi

$t = t_{web}$

$f = f_{cr} = 40.56$ ksi

$\frac{b}{t} = \frac{h_w}{t_w}$

Find $h_w = d - 2\,kdes$

$= 15.7 - 2(0.747)$

$h_w = 14.21"$

$\frac{b}{t} = \frac{14.21}{0.25} = 56.80$

Prepared by Eng.Maged Kamel.

(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:

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

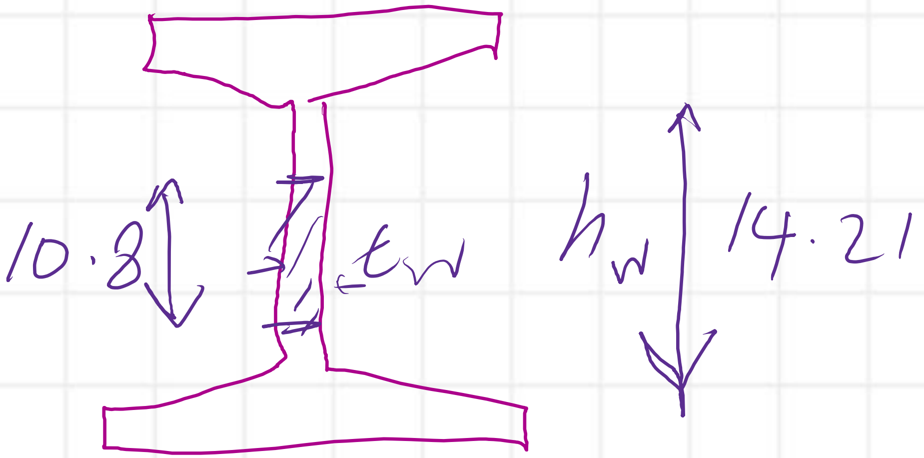
$$be = 1.92(0.25)\sqrt{\frac{29000}{40.56}}\left[1 - \frac{0.34}{56.8}\sqrt{\frac{29000}{40.56}}\right]$$

$$he = be = 12.835\,[0.83994]$$

$$= 10.78 \text{ inches}$$

$$\simeq 10.80 \text{ inches} < 14.21''$$

$$d - 2k_{des}$$



Find A_{Eff} web

Flange is fully active

$\lambda_F < \lambda_r$

Effective web area

$h_w = 14.21''$
$t_w = 0.25''$

$\Rightarrow A_w = 3.55 \text{ inch}^2$ reduced to

$h_e = 10.8''$
$t_w = 0.25''$

$\Rightarrow A_{ef} = 10.8(0.25)$
$= 2.70 \text{ inch}^2$

$$A_{sect} = A_g - (h_w)(t_w) + A_{ef\,w}$$

$$A_e = 7.68 - 3.55 + 2.70 = 6.83 \text{ inch}^2$$

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 $\frac{KL}{r} \le 4.71\sqrt{\frac{E}{QF_y}}$ $\left(\text{or } \frac{QF_y}{F_e} \le 2.25\right)$

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

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

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

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 $b/t \le 20$, 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 iF no Iteration $Q = 1$

$A_{eff} = 6.83\ inch^2$

$F_{cr} = 40.56\ ksi$

$A_g = 7.68\ inch^2$

LRFD $= \phi_c\ A_g (F_{cr}) Q =$

$0.9 (7.68)(40.56)(1) = 280\ kips$

ASD $= \frac{1}{\Omega_c} A_{eff} (F_{cr}) Q$

$= \frac{1}{1.67} (7.68)(40.56)(1) \simeq 187\ kips$