A W21 × 55 with F_y = 50 ksi is used for the beam and loads of Fig. 10.4. Check its adequacy in shear.

MACormac Example 10-2

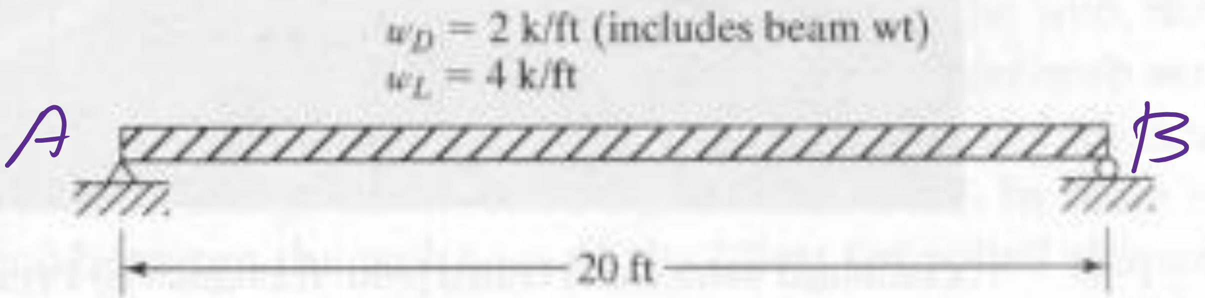
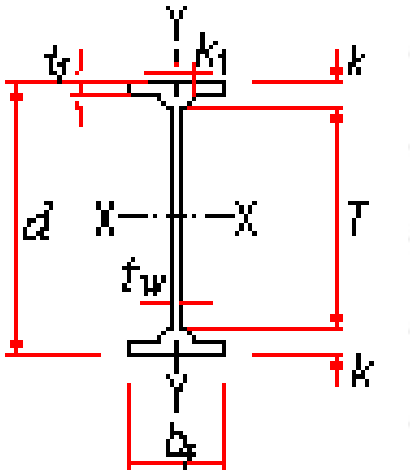


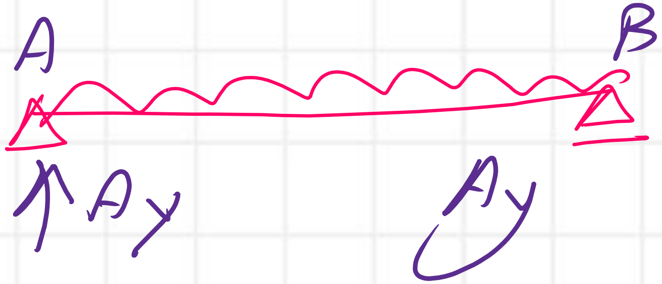
FIGURE 10.4

@ LRFD design

$\ell = 20'$

$$W_{ult} = 1.2\, w_D + 1.60\, w_L$$
$$= 1.2\,(2) + 1.6\,(4)$$
$$= 2.4 + 6.40 = 7\ k/F$$





$$A_y = w_u \cdot \frac{L}{2} = 7\,(10)$$
$$= 70\ kips$$

Prepared by Eng.Maged Kamel.

A W21 × 55 with F_y = 50 ksi is used for the beam and loads of Fig. 10.4. Check its adequacy in shear.

MACormac Example 10-2

w_D = 2 k/ft (includes beam wt)
w_L = 4 k/ft

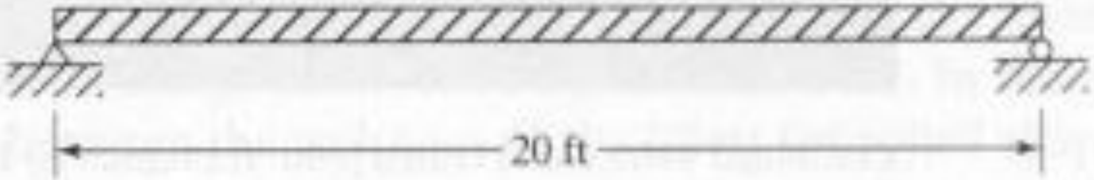
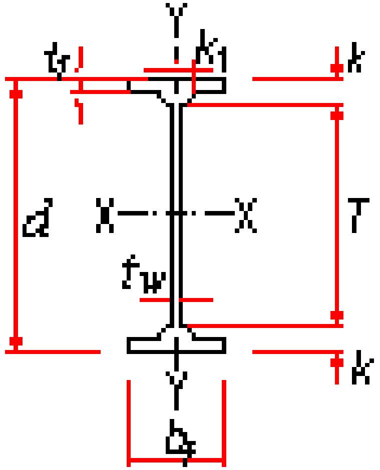


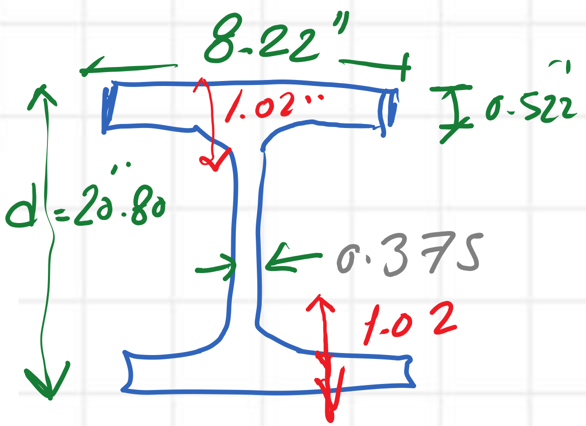
FIGURE 10.4

k design   k detailing

Solution

W21 x 55





Shape	Area, A	Depth, d		Web Thickness, t_w		$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		k k_{des}	k k_{det}	k_1	T	Workable Gage
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W21×93	27.3	21.6	21⅝	0.580	9/16	5/16	8.42	8⅜	0.930	15/16	1.43	1⅝	15/16	18⅜	5½
×83c	24.4	21.4	21⅜	0.515	½	¼	8.36	8⅜	0.835	13/16	1.34	1½	7/8		
×73c	21.5	21.2	21¼	0.455	7/16	¼	8.30	8¼	0.740	¾	1.24	1 7/16	7/8		
×68c	20.0	21.1	21⅛	0.430	7/16	¼	8.27	8¼	0.685	11/16	1.19	1⅜	7/8		
×62c	18.3	21.0	21	0.400	⅜	3/16	8.24	8¼	0.615	⅝	1.12	1 5/16	13/16		
×55c	16.2	20.8	20¾	0.375	⅜	3/16	8.22	8¼	0.522	½	1.02	1 3/16	13/16		
×48c,f	14.1	20.6	20⅝	0.350	⅜	3/16	8.14	8⅛	0.430	7/16	0.930	1⅛	13/16		

$$h_w = 20.80 - 2(1.02) = 18.76''$$

Prepared by Eng.Maged Kamel.

Relation between F_y, $\frac{h}{t_w}$ for Zone I

$E = 29 \times 10^6$ ksi

Different Grades of steel

$E = 29 \times 10^6$ psi $\qquad \frac{h}{t_w} = 2.24\sqrt{\frac{E}{F_y}}$

$F_y = 36$ ksi $\Rightarrow \frac{h}{t_w} = 2.24\sqrt{\frac{29\,000}{36}} = 63.576$

$F_y = 42$ ksi $\rightarrow \frac{h}{t_w} = 2.24\sqrt{\frac{29000}{42}} = 58.86 \Rightarrow 58.90$

$F_y = 45$ ksi $\rightarrow \frac{h}{t_w} = 2.24\sqrt{\frac{29000}{45}} = 56.86 \Rightarrow 56.90$

$F_y = 50$ ksi $\rightarrow \frac{h}{t_w} = 2.24\,(29000/50)^{1/2} = 53.90$

$F_y = 60$ ksi $\rightarrow \frac{h}{t_w} = 2.24\,(29000/60)^{1/2} = 49.25$

$F_y = 65$ ksi $\rightarrow \frac{h}{t_w} = 2.24\left(29000/65\right)^{1/2} \simeq 47.30$

$F_y = 100$ ksi $\rightarrow h/t_w = 2.24\,(29000/100)^{1/2} = 38.15$

Prepared by Eng.Maged Kamel.

Nominal shear value

G2. I-SHAPED MEMBERS AND CHANNELS

1. Shear Strength of Webs without Tension Field Action

The nominal shear strength, V_n, is:

$$V_n = 0.6F_yA_wC_{v1} \quad \text{(G2-1)}$$

where

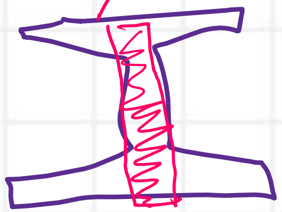
F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)

A_w = area of web, the overall depth times the web thickness, dt_w, in.^{2} (mm^2)

$E = 29{,}000$ $\quad F_y = 50\,ksi$, $t_w = 0.375''$ $\quad d = 20.8''$

$h_w = 18.76''$

For $2.24\sqrt{\dfrac{E}{F_y}} \Rightarrow 53.90$ $\qquad A_w = d\,t_w$



$\boxed{C_v = 1}$

our $\dfrac{h}{t_w}$ For $W_{21} \times 55$ $= 18.76/0.375 = 50.03 < 53.90$

shear stren

$V_n = C_v\, t_w\, d\, (0.6 F_y) = 1(0.375)(20.80)(0.6)(50$

$= 234\ K$

Spec 2010

CM #14

Prepared by Eng.Maged Kamel.

ⓐ LRFD

For G.2.1a

$\phi = 1$

$\phi V_n = 1(234) = 234$ kips

$V_u = 70$ kips $\Rightarrow$ $70^k < 234^k$

Since $V_u < \phi V_n \Rightarrow$ Section is adequate for shear

A W21 × 55 with F_y = 50 ksi is used for the beam and loads of Fig. 10.4. Check its adequacy in shear.

MACormac Example 10-2

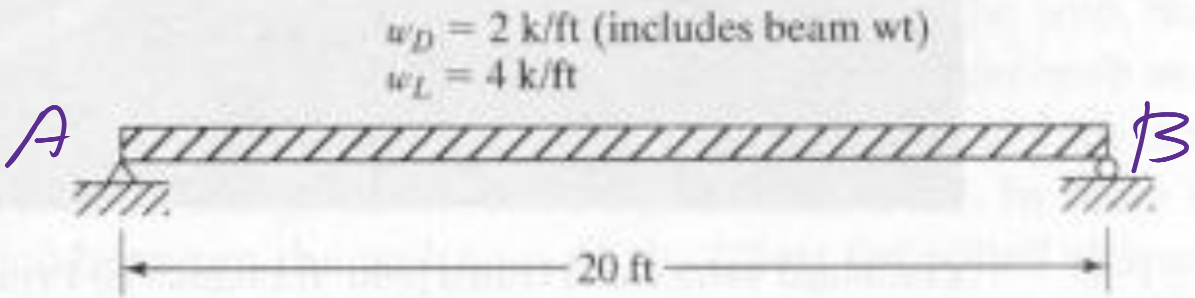


FIGURE 10.4

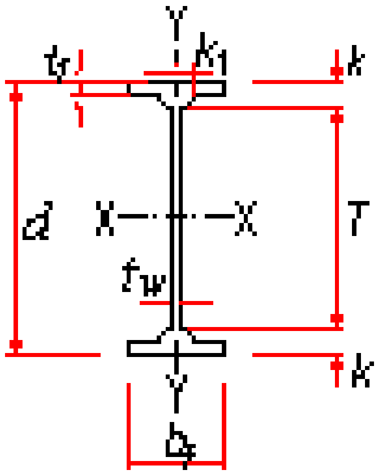
(b) ASD

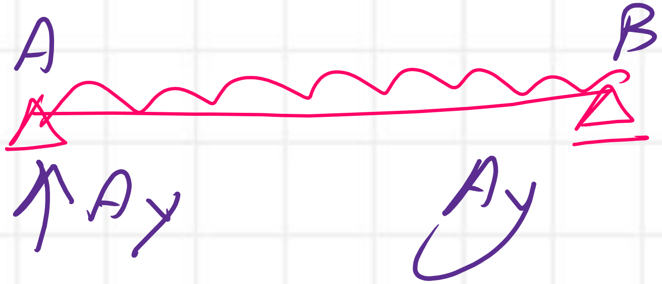
design

$\ell = 20'$

$W = W_D + W_L$

$= 2 + 4 = 6 \ k/FT$





$Ay = W_T \cdot \frac{L}{2} = 6\,(10)$

$= 60 \ kips$

Prepared by Eng.Maged Kamel.

(b) ASD

For 6.2.1a

$\Omega = 1.5$

$V_n = \frac{1}{1.5}(234) = 156$ kips

$V_T = 60$ kips $\Rightarrow 60^k < 156^k$

Since $V_T < \frac{1}{\Omega} V_n \Rightarrow$ Section is adequate for shear

CM # 14 → specification # 2010

16.1-67

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR

This chapter addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and *HSS* sections, and shear in the weak direction of singly or doubly symmetric shapes.

The chapter is organized as follows:

G1. General Provisions

G2 → G2. Members with Unstiffened or Stiffened Webs

G3. Tension Field Action

G4. Single Angles

G5. Rectangular HSS and Box-Shaped Members

G6. Round HSS

G7. Weak Axis Shear in Doubly Symmetric and Singly Symmetric Shapes

G8. Beams and Girders with Web Openings

User Note: For cases not included in this chapter, the following sections apply:

- H3.3 Unsymmetric sections
- J4.2 Shear strength of connecting elements
- J10.6 Web *panel zone* shear

Prepared by Eng.Maged Kamel.

Specs 2010 # CM # 14

G2-1a

(a) For webs of rolled I-shaped members with $h/t_w \leq 2.24\sqrt{E/F_y}$:

$\phi_v = 1.00$

$$\phi_v = 1.00 \text{ (LRFD)} \qquad \Omega_v = 1.50 \text{ (ASD)}$$

For $\frac{h}{t_w} \leq 2.24 \sqrt{\frac{E}{F_y}}$

and

Term is C_v

$\Omega_v = 1.50$

$$C_v = 1.0 \tag{G2-2}$$

User Note: All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for F_y = 50 ksi (345 MPa).

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, C_v, is determined as follows:

(i) When $h/t_w \leq 1.10\sqrt{k_v E/F_y}$

k_v

$$C_v = 1.0 \tag{G2-3}$$

$V_n = C_v A_w (0.6 F_y)$

Prepared by Eng.Maged Kamel.

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, C_v, is determined as follows:

How to find Cv values ?
when $\frac{h}{t_w}$ varies

(i) When $h / t_w \leq 1.10\sqrt{k_v E / F_y}$

$$C_v = 1.0 \tag{G2-3}$$

Provision in specs 2010

(ii) When $1.10\sqrt{k_v E / F_y} < h / t_w \leq 1.37\sqrt{k_v E / F_y}$

$$C_v = \frac{1.10\sqrt{k_v E / F_y}}{h / t_w} \tag{G2-4}$$

(iii) When $h / t_w > 1.37\sqrt{k_v E / F_y}$

$$C_v = \frac{1.51 k_v E}{(h / t_w)^2 F_y} \tag{G2-5}$$

where

A_w = area of web, the overall depth times the web thickness, dt_w, in.^{2} (mm^2)

h = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)

= for built-up welded sections, the clear distance between flanges, in. (mm)

= for built-up bolted sections, the distance between *fastener* lines, in. (mm)

= for tees, the overall depth, in. (mm)

specs . 2010

The web plate *shear buckling* coefficient, k_v, is determined as follows:

(i) For webs without *transverse stiffeners* and with $h/t_w < 260$:

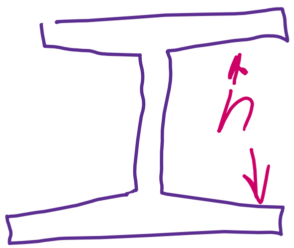
$$k_v = 5$$

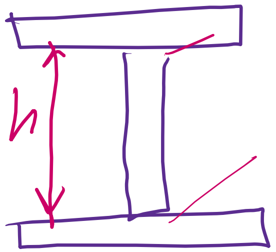
except for the stem of tee shapes where $k_v = 1.2$.

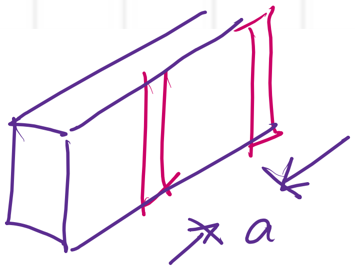
CM #14 specs - P-69

(ii) For webs with transverse stiffeners:

For h







clear distance rolled

Built up

$$k_v = 5 + \frac{5}{(a/h)^2} \tag{G2-6}$$

$$= 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[\frac{260}{(h/t_w)}\right]^2$$

where

a = clear distance between transverse stiffeners, in. (mm)

User Note: For all ASTM A6 W, S, M and HP shapes except M12.5×12.4, M12.5×11.6, M12×11.8, M12×10.8, M12×10, M10×8 and M10×7.5, when F_y = 50 ksi (345 MPa), C_v = 1.0.

Specs 2010

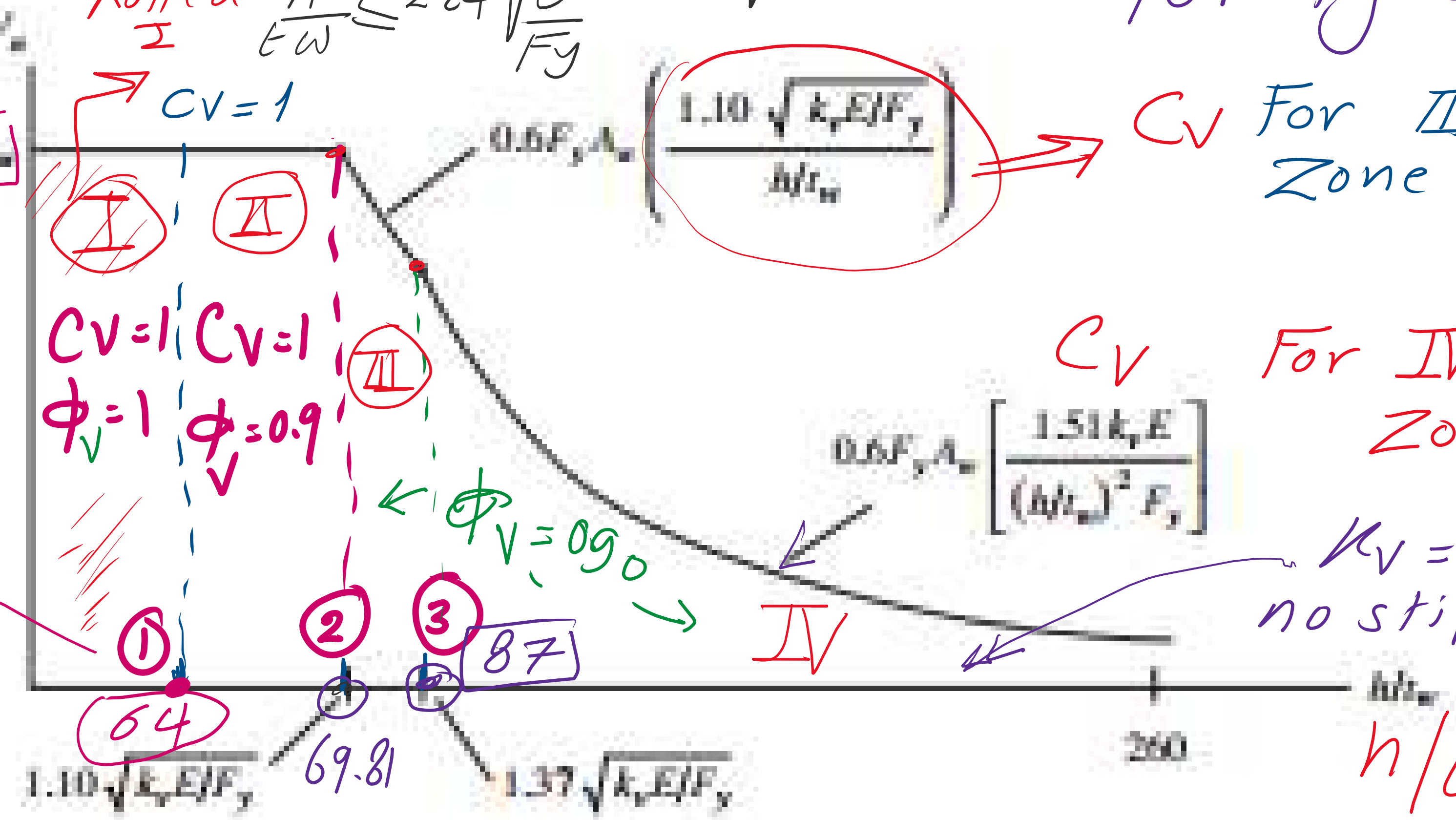
SEGui - 5th edition

$k_v = 5$

For $F_y = 36$ ksi

FIGURE 5.19

Rolled I $\frac{h}{t_w} \le 2.24\sqrt{\frac{E}{F_y}}$



Cv For III Zone

Cv For IV Zone

$k_v = 5$ no stiffners

h/t_w

Point # 1

$F_y = 36$ ksi, h/t_w

$2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29000}{36}} = 63.57$

Point . 2 ; $k_v = 5$

$h/t_w = 1.1\sqrt{\frac{k_v E}{F_y}}$

$\frac{h}{t_w} = 1.1\sqrt{\frac{5(29000)}{36}} = 69.81$

Point-3

$h/t_w = 1.37\sqrt{k_v . E/F_y}$

$1.37\sqrt{\frac{5(29000)}{36}} = 86.95 \simeq 87$

Prepared by Eng.Maged Kamel.

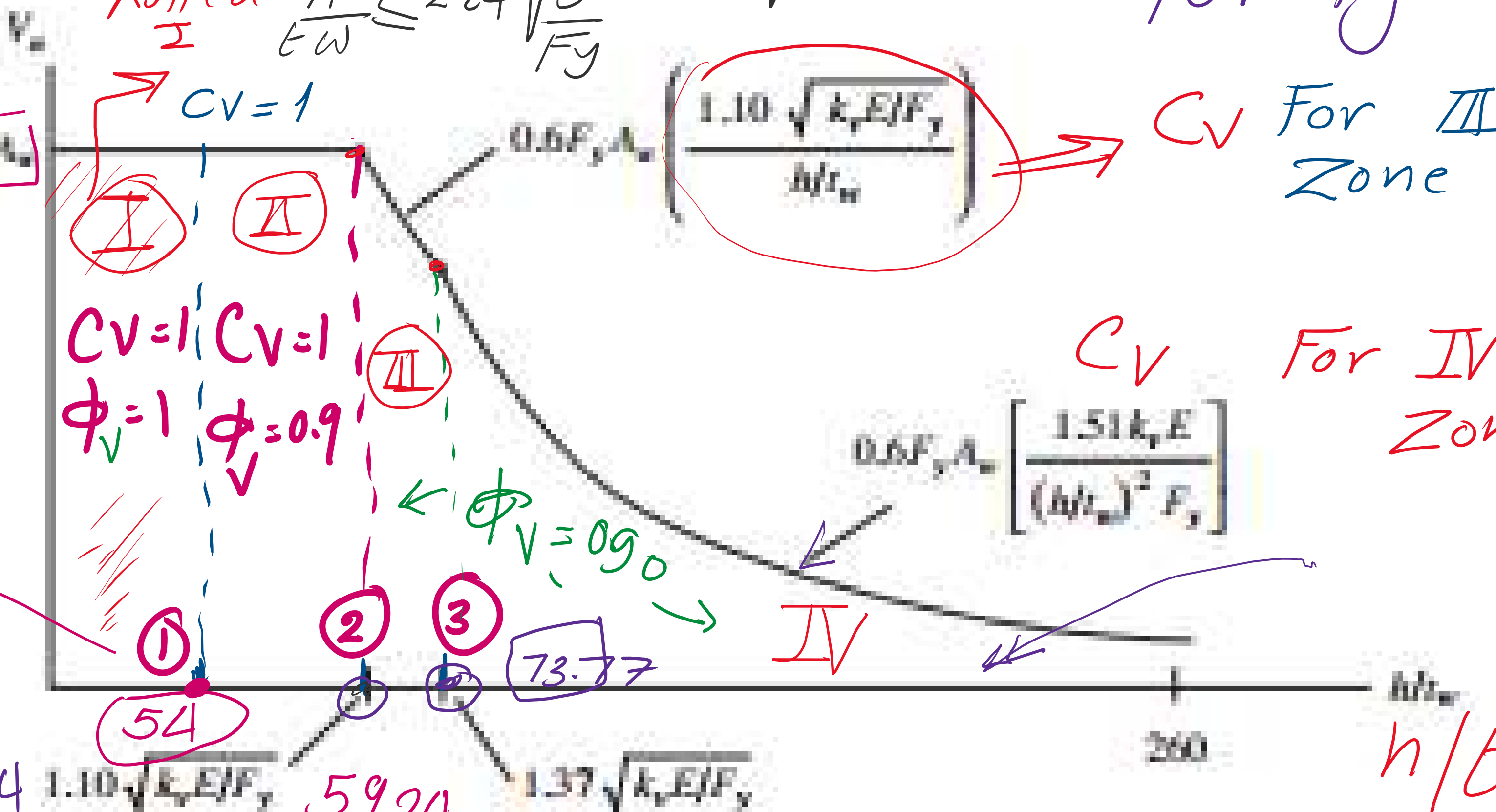
Specs 2010

SEGui - 5th edition

$K_v = 5$

For $F_y = 50$ ksi

FIGURE 5.19



Cv For III Zone

Cv For IV Zone

Point # 1

$F_y = 50$ ksi, h/t_w

$2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29000}{50}}$

$= 53.95 \simeq 54$

Point . 2 ; $K_v = 5$

$h/t_w = 1.1\sqrt{\frac{K_v E}{F_y}}$

$\frac{h}{t_w} = 1.1\sqrt{\frac{5(29000}{50}} = 59.24$

Point-3 $1.37\sqrt{\frac{5(29000)}{50}} = 73.77$

$> \frac{h}{t_w} < 1.1\sqrt{\frac{K_v E}{F_y}}$

$h/t_w = 1.37\sqrt{K_v . E/F_y}$

h/t_w

Prepared by Eng.Maged Kamel.

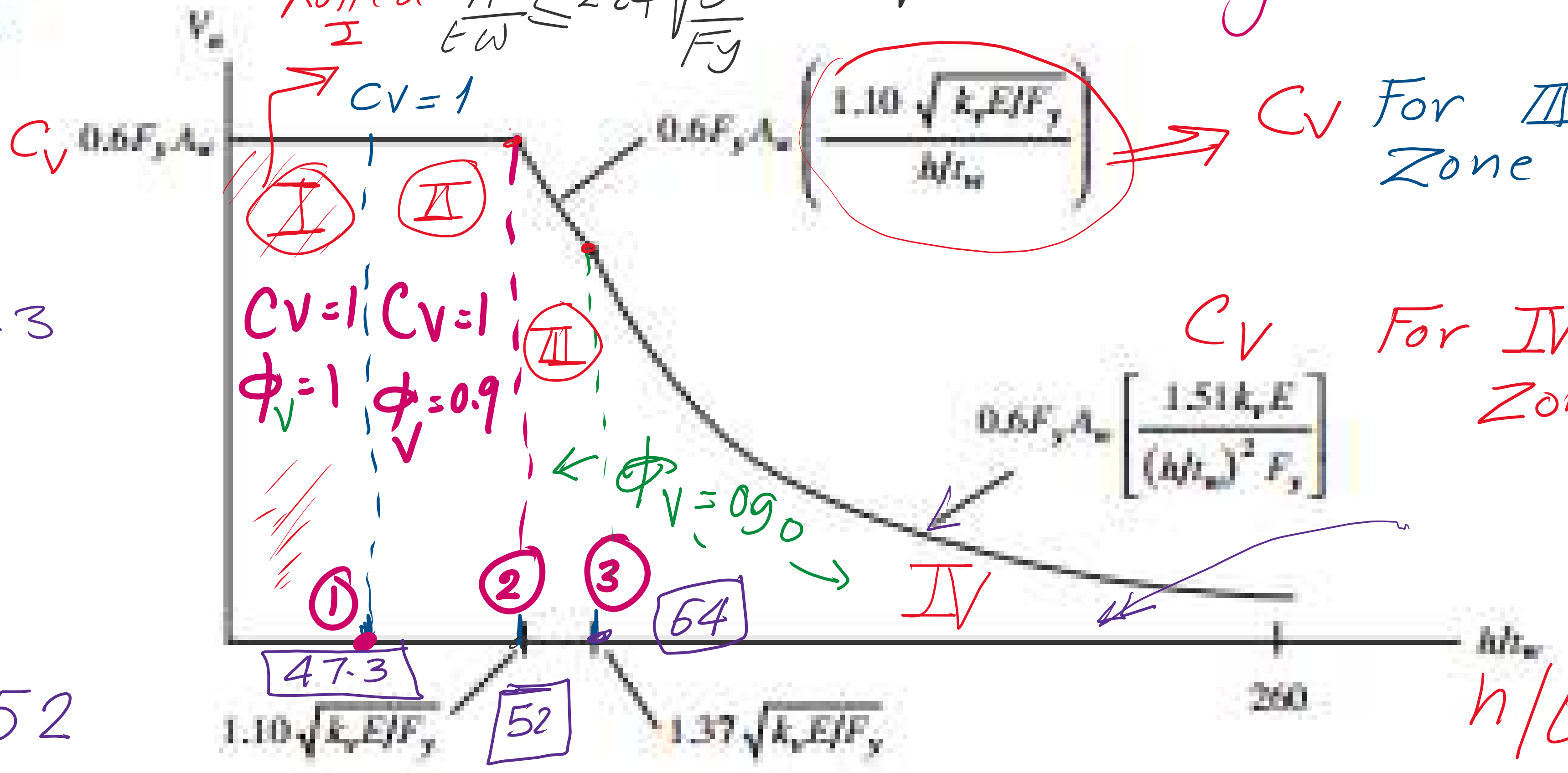
Specs 2010

SEGui - 5th edition

$K_v = 5$

$F_y = 65$ ksi

FIGURE 5.19



C_v For III Zone

C_v For IV Zone

h/t_w

Point # 1

$F_y = 65$ ksi

$2.24\sqrt{\frac{E}{F_y}} = 47.3$

Point-2

$1.1\sqrt{\frac{5(29000)}{65}} = 52$

Point-3

$1.37\sqrt{\frac{5(29000)}{65}} \simeq 64$

Prepared by Eng.Maged Kamel.