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

by using table 3-2

MACormac Example 10-2

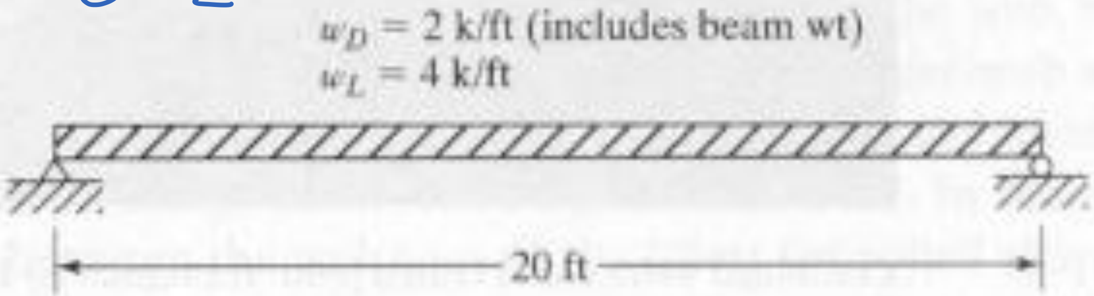
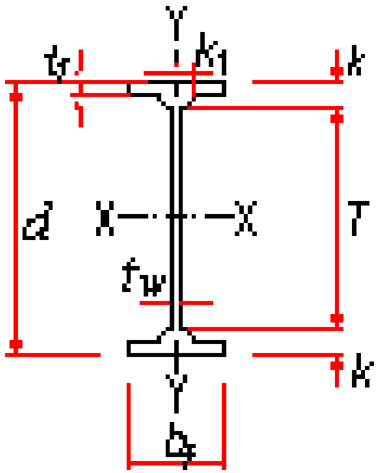
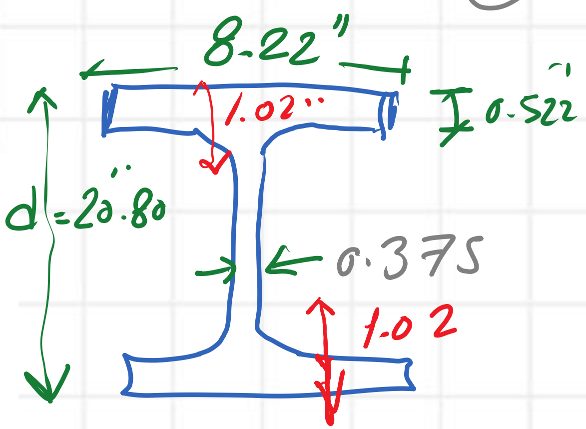


FIGURE 10.4

Solution

W21 x 55





① We need Z_x for W21 x 55

② Table 3-2 Sorted by Z_x → select W21 x 55

③ check ϕV_n, $\frac{V_n}{\Omega}$

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

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1-21

Table 1-1 (continued)
W-Shapes
Properties

W21 X 55

W24-W21

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}
93	4.53	32.3	2070	192	8.70	221	92.9	22.1	1.84	34.7	2.24	20.7	6.03	9940
83	5.00	36.4	1830	171	8.67	196	81.4	19.5	1.83	30.5	2.21	20.6	4.34	8630
73	5.60	41.2	1600	151	8.64	172	70.6	17.0	1.81	26.6	2.19	20.5	3.02	7410
68	6.04	43.6	1480	140	8.60	160	64.7	15.7	1.80	24.4	2.17	20.4	2.45	6760
62	6.70	46.9	1330	127	8.54	144	57.5	14.0	1.77	21.7	2.15	20.4	1.83	5960
55	7.87	50.0	1140	110	8.40	126	48.4	11.8	1.73	18.4	2.11	20.3	1.24	4980
48	9.47	53.6	959	93.0	8.24	107	38.7	9.52	1.66	14.9	2.05	20.2	0.803	3950

$Z_x = 126$ inch^3

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$F_y = 50$ ksi

Table 3-2 (continued)
W-Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W21×55	126	314	473	192	289	10.8	16.3	6.11	17.4	1140	156	234

$\phi_v V_{nx} = 234$ kips

$\frac{V_{nx}}{\Omega} = 156$ kips

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Specification

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CM # 15

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:

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

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CM # 15

G1. GENERAL PROVISIONS

Two methods of calculating shear strength are presented below. The method presented in Section G2 does not utilize the post *buckling strength* of the member (*tension field action*). The method presented in Section G3 utilizes tension field action.

The *design shear strength*, $\phi_v V_n$, and the *allowable shear strength*, V_n/Ω_v, shall be determined as follows:

G2.1a

For all provisions in this chapter except Section G2.1(a):

$$\phi_v = 0.90 \text{ (LRFD)} \qquad \Omega_v = 1.67 \text{ (ASD)}$$

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

1. Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, V_n, of unstiffened or stiffened webs according to the *limit states* of *shear yielding* and *shear buckling*, is

V_n

$$V_n = 0.6 F_y A_w C_v \tag{G2-1}$$

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G2.1a CM #15 #2016

G2. I-SHAPED MEMBERS AND CHANNELS

1. Shear Strength of Webs without Tension Field Action

The nominal shear strength, V_n, is:

$$V_n = 0.6 F_y A_w C_{v1} \qquad \text{(G2-1)}$$

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)

C_{v1} Term

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

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

and

$$C_{v1} = 1.0 \qquad \text{(G2-2)}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

h = clear distance between flanges less the fillet at each flange, in. (mm)

t_w = thickness of web, in. (mm)

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).

specs 2016

$k_v = 5.34$

(b) For all other I-shaped members and channels

(1) The web shear strength coefficient, C_{v1}, is determined as follows:

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

$\frac{h}{w} \le 1.10\sqrt{\frac{k_v E}{F_y}}$

$$C_{v1} = 1.0 \quad \text{(G2-3)}$$

where

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

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

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

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \quad \text{(G2-4)}$$

(2) The web plate shear buckling coefficient, k_v, is determined as follows:

(i) For webs without transverse stiffeners

$$k_v = 5.34$$

(ii) For webs with transverse stiffeners

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

$$= 5.34 \text{ when } a/h > 3.0$$

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_{v1} = 1.0.

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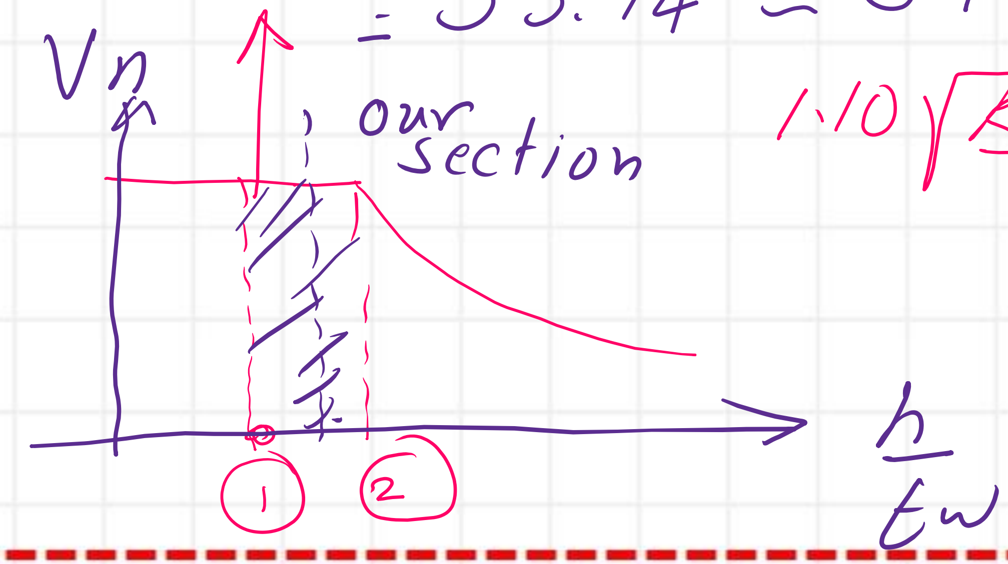
Check $W_{40}X149 \Rightarrow Z_x = 598\ inch^3$

$\frac{h}{tw} = 54.3 \Rightarrow F_y = 50\ ksi;\quad k_v = 5.34$

$E = 29000$

Limiting $2.24\sqrt{\frac{E}{F_y}}$

Part-2 $= 53.94 \simeq 54.0 \rightarrow$ Point 1



$1.10\sqrt{\frac{K_v E}{F_y}} = 1.10\sqrt{\frac{(5.34)29000}{50}}$

$= 61.22,\ C_{v,}=1$

	W40X149	
A =	43.8	in.^2
d =	38.2	in.
tw =	0.63	in.
bf =	11.8	in.
tf =	0.83	in.
k(des) =	2.01	in.
k(det) =	2.125	in.
k1 =	1.5	in.
T =	34	in.
gage =	7.5	in.
wt./ft. =	149	plf.
bf/(2*tf)	7.11	
h/tw =	54.3	
Ix =	9800	in.^4
Sx =	513	in.^3
rx =	15	in.
Zx =	598	in.^3
Iy =	229	in.^4
Sy =	38.8	in.^3
ry =	2.29	in.
Zy =	62.2	in.^3
rts =	2.89	in.

$d = 38.2''$

W40 X 149

$\phi_V = 0.90$

$F_y = 50$ ksi

$\Omega_V = 1.67$

$t_w = 0.63$

$A_w = d\,t_w = 0.63(38.2) = 24.07 \text{ inch}^2$

$\phi V_n = C_{V1}(0.6 F_y) A_w \phi_V$

(1)

$= 1(0.6)(50)(24.07)(0.90) = 649.89 \simeq 650\text{ K}$

$\frac{V_n}{\Omega} = 1(0.6)(50)(24.07)\left(\frac{1}{1.67}\right)$

$= 432.40\text{ K}$

$\Rightarrow$

432 K

	W40X149	
A =	43.8	in.^2
d =	38.2	in.
tw =	0.63	in.
bf =	11.8	in.
tf =	0.83	in.
k(des) =	2.01	in.
k(det) =	2.125	in.
k1 =	1.5	in.
T =	34	in.
gage =	7.5	in.
wt./ft. =	149	plf.
bf/(2*tf)	7.11	
h/tw =	54.3	
Ix =	9800	in.^4
Sx =	513	in.^3
rx =	15	in.
Zx =	598	in.^3
Iy =	229	in.^4
Sy =	38.8	in.^3
ry =	2.29	in.
Zy =	62.2	in.^3
rts =	2.89	in.

W40 X 149 in Part 2

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

$F_y = 50$ ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W40×149^v ✓	598	1490	2240	896	1350	38.3	57.4	8.09	23.6	9800	432	650
W36×150	581	1450	2180	880	1320	34.4	51.9	8.72	25.3	9040	449	673

$$\phi_v V_n = 650\text{K}$$

$$\frac{V_n}{\Omega_v} = 432\text{K}$$

Matches with our Calculation

G2.1.b

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(b) For all other I-shaped members and channels

(1) The web shear strength coefficient, C_{v1}, is determined as follows:

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

Term is CV1

$$C_{v1} = 1.0 \quad \text{(G2-3)}$$

where

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

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

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

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \quad \text{(G2-4)}$$

(2) The web plate shear buckling coefficient, k_v, is determined as follows:

(i) For webs without transverse stiffeners

$$k_v = 5.34$$

(ii) For webs with transverse stiffeners

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

$$= 5.34 \text{ when } a/h > 3.0$$

where

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

check why

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_{v1} = 1.0.

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$M12.5 \times 12.4$

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), $C_{v1} = 1.0$.

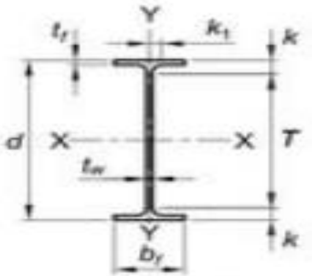

Table 1-2
M-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		$\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k	Distance k_1	Distance T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.
M12.5×12.4^{c,v}	3.63	12.5	12½	0.155	⅛	1/16	3.75	3¾	0.228	¼	9/16	⅜	11⅜	–
×11.6^{c,v}	3.40	12.5	12½	0.155	⅛	1/16	3.50	3½	0.211	3/16	9/16	⅜	11⅜	–

Nom-inal Wt.	Compact Section Criteria $\frac{b_f}{2t_f}$	Compact Section Criteria $\frac{h}{t_w}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis X-X Z	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z	r_{ts}	h_o	$\frac{J}{S_x h_o}$	Torsional Properties J	Torsional Properties C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.		in.^{4}	in.^{6}
12.4	8.22	74.8	89.3	14.2	4.96	16.5	2.01	1.07	0.744	1.68	0.933	12.3	0.000283	0.0493	76.0
11.6	8.29	74.8	80.3	12.8	4.86	15.0	1.51	0.864	0.667	1.37	0.852	12.3	0.000263	0.0414	57.1

$\frac{h}{t_w} = 74.80$, $d = 12.5''$

$K_v = 5.34$, $t_w = 0.155''$

$$1.10\sqrt{\frac{K_v E}{F_y}} = 1.1\sqrt{\frac{5.34(29000)}{50}} = 61.22$$

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

$C_{v1} \neq 1$

inch^2

$A_w = (12.5)(0.155) = 1.9375$

$E = 29000$ ksi

$F_y = 50$ ksi

$F_y = 50$ ksi

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$M12.5 \times 12.4$

h/t_w

Point-(1)

$$2.24\sqrt{\frac{E}{F_y}} = 2.24\sqrt{\frac{29000}{50}} = 53.94 \approx 54$$

Point-2

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

$$\frac{h}{t_w} = 1.1\sqrt{\frac{5.34(29000)}{50}} = 61.22$$

C_{v1}

$\frac{h}{t_w}$, $K_v = 5.34$

$$\frac{h}{t_w} = 74.80$$

$$C_{v1} = \frac{1.1\sqrt{\frac{5.34(29000)}{50}}}{74.80} = 0.818$$

$$V_n = \underbrace{0.60(50)}_{0.6F_y}\,\underbrace{(12.5)(0.155)}_{A_w}\,\underbrace{0.818}_{C_{v1}} = 47.55 \text{ kips}$$

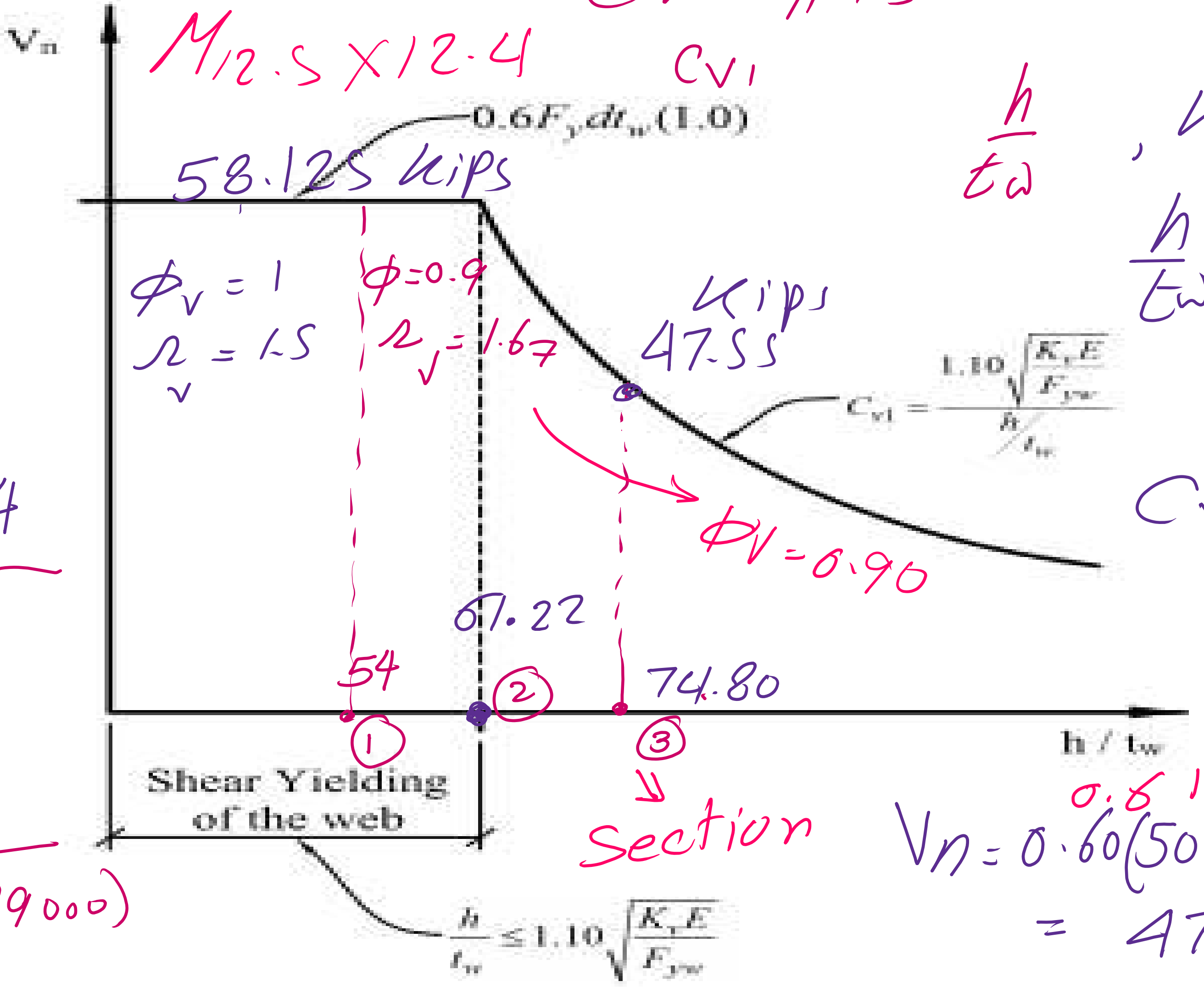

FIGURE 6-7c Shear strength variation in a flexural member

$\phi_v = 0.90$, $F_y = 50$ ksi ; $M12.5 \times 12.4$

$\Omega_v = 1.67$,

43 k

$\Uparrow$

$V_n = 47.55$ k $\Rightarrow \phi V_n = 0.90 (47.55) = 42.795$ k

$$\frac{V_n}{\Omega} = \frac{47.55}{1.67} = 28.47 \text{ k}$$

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