Design of Members for Shear

Shear stresses are usually not a controlling factor in the design of steel beams except for the following case:-

1-The beam is very short

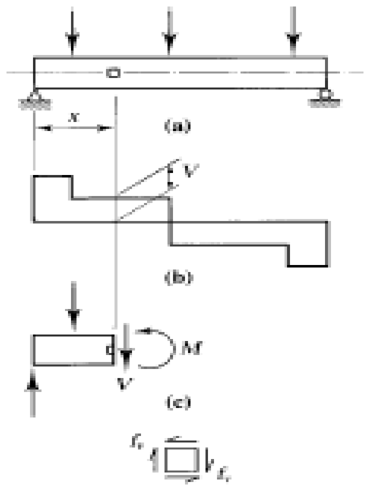
2-There are holes in web of the beam

3-The beam is subjected to a very heavy concentrated load near one the supports

4-The beam is coped.

Before covering the AISC provisions for shear strength we will first review some basic concept from mechanics pf material.

$$fv = \frac{VQ}{Ib} \qquad equ.1$$



Where

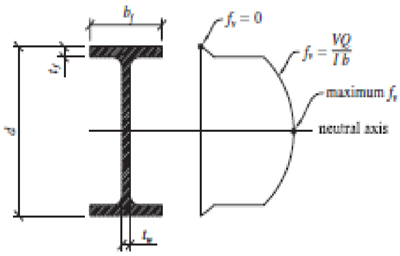
fv = vertical shearing stress at the point of interest

V = vertical shear force at the section under consideration

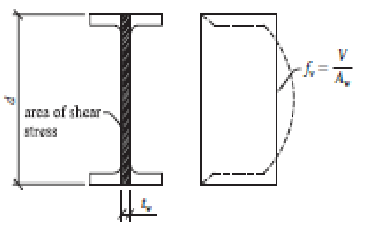
Q = first moment, about the neutral axis, of the area of the cross section between the point of interest and the top or bottom of the cross section

I = moment of inertia about the neutral axis

b = width of the cross section at the point of interest



a. actual shear stress



b. approximated shear stress

if the load has been increased on I shape section until the bending yield stress is reach in flange, then the flange will be unable to resist shear stress and it will be carried to the web. If the moment is further increased the bending yield stress will penetrate farther down into the web and the area of web that can resist will be further reduced.

The AISC specifications assume that shear is resisted by the entire web thickness. Web area (Aw) is equal to the overall depth with the web thickness

$Aw = dxtw$

Where

Aw=web area

d=overall depth of the I shape

tw=thickness of the web

Superimposed on the actual distribution is the average stress in the web, V/Aw, which does not differ much from the maximum web stress. Clearly, the web will completely yield long before the flanges begin to yield. Because of this, yielding of the web represents one of the shear limit states. Taking the shear yield stress as 60% of the tensile yield stress, we can write the equation for the stress in the web at failure as

$$Fv = \frac{V_n}{A_w} \qquad equ.2$$

$$0.6fy = \frac{V_n}{A_w}$$

Where
Aw = area of the web. The nominal strength corresponding to this limit state is therefore
$Vn = 0.6FyAw$ 3)
and will be the nominal strength in shear provided that there is no shear buckling of the web. Whether that occurs will depend on ($\frac{h}{tw}$), the width-to-thickness ratio of the web. If this ratio is too large⎯lead to be the web is too slender and the web can buckle in shear, either in elastically or elastically.

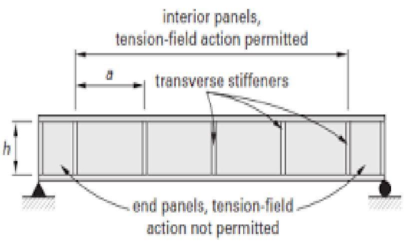
Beam shear strength is covered in Chapter G of the AISC Specification, "Design of Members for Shear"

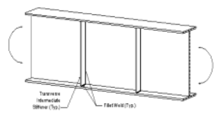
G1. GENERAL PROVISIONS
Two methods of calculating shear strength are presented below. The method presented in section G2 does not utilize the post buckling strength of the member (tension field action). The method presented in Section G3 utilizes tension field action.
The design shear strength, ϕVn, and the allowable shear strength, Vn/Ω , shall be determined as follows.
For all provisions in this chapter except Section G2.1a:
ϕ= 0.90 (LRFD) ,Ω = 1.67 (ASD)

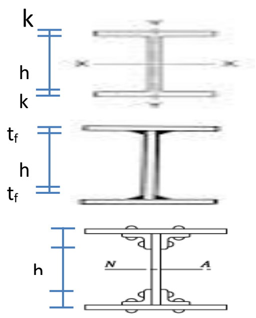
G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS
1. Nominal Shear Strength
This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web. The nominal shear strength, Vn, of unstiffened or stiffened webs, according to the limit states of shear yielding and shear buckling, is
$Vn = 0.6Fy\,AwCv$ (G2-1)
$Aw = dxtw$
(a) For webs of rolled I-shaped members with h/tw ≤ $2.24\sqrt{E/Fy}$
ϕ= 1.00 (LRFD)
Ω= 1.50 (ASD)
Cv = 1.0
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 Fy ≤ 50 ksi

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round HSS, the web shear coefficient, Cv, is determined as follows(i.e. with h/tw > $2.24\sqrt{E/Fy}$)

(i) For h/tw ≤ $1.10\sqrt{kv\,E/Fy}$
Cv = 1.0 (G2-3)`
(ii) For $1.10\sqrt{kvE/Fy}$ < h/tw ≤ $1.37\sqrt{kv\,E/Fy}$
Cv = $\frac{1.10\,kv\sqrt{(E/Fy)}}{h/tw}$ (G2-4)
(iii) For h/tw > $1.37\sqrt{kv\,E/Fy}$
Cv = $\frac{1.51Ekv}{fy(\frac{h}{tw})^2}$ (G2-5)







Where

Aw = the overall depth times the web thickness, , in.^{2}

The web plate buckling coefficient, kv, is determined as follows:

(i) For unstiffened webs with h/tw < 260

kv = 5 , except for the stem of tee shapes where kv = 1.2.

(ii) For stiffened webs,

kv = 5 + 5 /(a/h)^2

= 5 when a/h > 3.0 or a/h >[260/(h/tw)]^2

Where

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

h = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in = d-2kdes.

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

=d-2t_f

h= for built-up bolted sections, the distance between fastener lines, in.

= for tees, the overall depth, in.

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 Fy ≤ 50 ksi , Cv = 1.0

h/tw

If $h/tw \le 2.24\sqrt{E/Fy}$

Yes:

ϕ= 1.00 (LRFD)
Ω= 1.50 (ASD)
Cv = 1.0

No:

ϕ= 0.9 (LRFD)
Ω= 1.67 (ASD)
Cv = ?

Find Cv

If $h/tw \le 1.1\sqrt{kv\ E/Fy}$	$1.1\sqrt{kv\ E/Fy} < h/tw \le 1.37\sqrt{kv\ E/Fy}$	If $h/tw > 1.37\sqrt{kv\ E/Fy}$
Cv = 1.0	Cv = $\frac{1.10\ kv\sqrt{(E/Fy)}}{h/tw}$	Cv = $\frac{1.51Ekv}{fy(\frac{h}{tw})^2}$

Where

The web plate buckling coefficient, kv, is determined as follows:

(i) For unstiffened webs with h/tw < 260

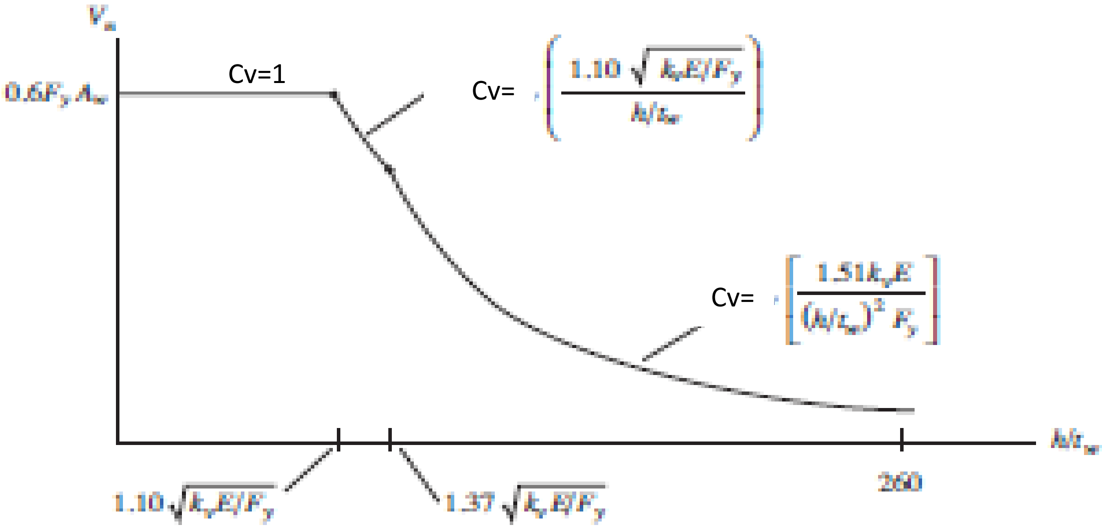
kv = 5 , except for the stem of tee shapes where kv = 1.2.

(ii) For stiffened webs,

kv = 5 + 5 /(a/h)^2

= 5 when a/h > 3.0 or a/h > [260/ (h/tw)]^2

Assume a= 5ft if it not given



Cv value with h/tw > 2.24$\sqrt{E/Fy}$

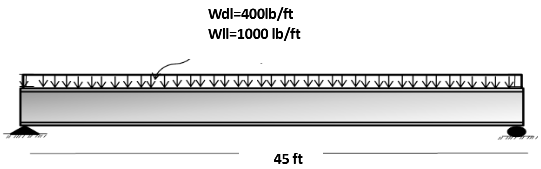
Example

W14x90 A992 steel material is used for simply supported beam with 45 ft is laterally supported at its ends and subjected to $W_{d.l}$ =400 lb/ft (including of self-weight) and $W_{l.l}$= 1000 lb/ft. Check the adequacy of the beam according to shear requirements Use LRFD method.

<u>Solution</u>

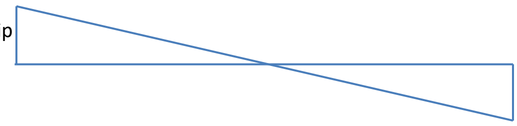
<u>Steel</u>	<u>fy</u>	<u>fu</u>
A992	50	65

<u>Section</u>	<u>h/tw</u>	<u>d</u>	<u>tw</u>	<u>k_{des}</u>
W14x90	25.9	14	0.44	1.31



Wu= 1.2W $_{d.l}$+ 1.6 W $_{l.l}$

=1.2x400/100+1.6x1000/1000= 2.08 k/ft



$V_{max}=\frac{WaxL}{2}=\frac{2.08x45}{2}$=46.8 kip

h=14-2x1.13=11.38

h/tw=11.38/0.44=25.86=25.9

h/tw ≤ 2.24$\sqrt{E/Fy}$

25.9 ≤ 2.24$\sqrt{29000/50}$

25.9< 53.94

Cv=1 ,ϕ=1

$Vn = 0.6x50x14x0.44x1 = 184.8\ kip$

$\phi Vn = 1x184.8 = 184.8\ kip > 46.8$

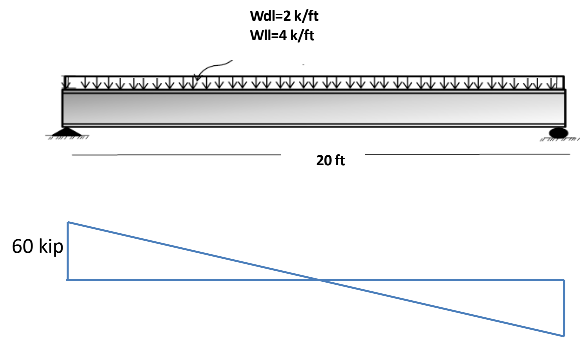
The section is adequate

Example

W21x55 A992 steel material is used for simply supported beam with 20 ft is laterally supported at its ends and subjected to $W_{d.l}$ = 2k/ft (including of self-weight) and $W_{l.l}$= 4 k/ft. Check the adequacy of the beam according to shear requirements. Use ASD method

Solution

<u>Steel</u>	<u>fy</u>	<u>fu</u>		
A992	50	65		
<u>Section</u>	<u>h/tw</u>	<u>d</u>	<u>tw</u>	<u>k_{des}</u>
W21x55	50	20.8	0.375	1.02



Wa= $W_{d.l}$+ $W_{l.l}$

=2 +4 = 6 k/ft

$V_{max}=\frac{WaxL}{2}=\frac{6x20}{2}$=60 kip

h=20.8-2x1.02=18.76

h/tw=18.76/0.375=50.02=50

h/tw $\leq 2.24\sqrt{E/Fy}$

50 $\leq 2.24\sqrt{29000/50}$

50< 53.94

Cv=1 ,Ω=1.5

$Vn = 0.6Fy\ AwCv$

$Vn = 0.6x50x20.8x0.375x1 = 234\ kip$

$\frac{Vn}{\Omega} = \frac{234}{1.5} = 156\ kip > 60$

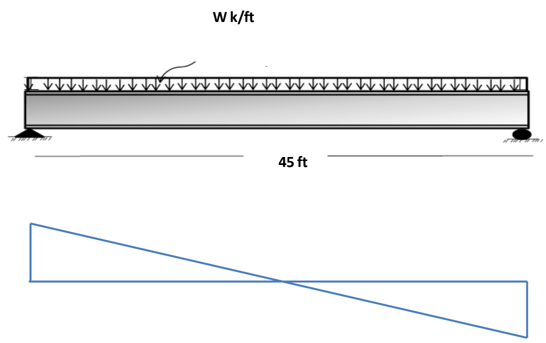
The section is adequate

Example

Calculate the Max. Uniform load control by shear requirements only assume a= 5ft, use M12x10 and A572G50 steel material

<u>Solution</u>

<u>Steel</u>	<u>fy</u>	<u>fu</u>	
A992	50	65	
<u>Section</u>	<u>h/tw</u>	<u>d</u>	<u>tw</u>
M12x10	74.4	12	0.149



h/tw > $2.24\sqrt{E/Fy}$

74.4 >$2.24\sqrt{29000/50}$

74.4 > 53.94

So ϕ &Ω will be

ϕ= 0.9 (LRFD)

Ω= 1.67 (ASD

Find Cv

h/tw > $1.1\sqrt{kvE/Fy}$

Find Kv

Kv =5+5/$(a/h)^2$

a/h =5x12/12= 5>3

So, kv =5

$h/tw > 1.1\sqrt{kvE/Fy}$

$74.7 > 1.1\sqrt{5x29000/50}$

74.7>59.23

$h/tw > 1.37\sqrt{kvE/Fy}$

$74.7 > 1.37\sqrt{5x29000/50}$

74.7>73.776

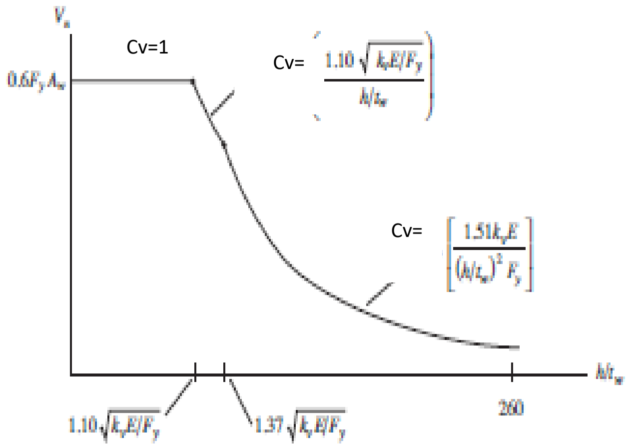
$Cv = \frac{1.51Ekv}{fy(\frac{h}{tw})^2}$

$Cv = \frac{1.51x29000x5}{50x(74.7)^2}$

=0.785

$Vn = 0.6Fy\ AwCv$

=0.6x50x12x0.149x0.785=42.1 kip



LRFD	ASD
$\phi vn = 0.9x42.1 = 37.89\ kip$ ϕvn =WuL/2 37.89=Wux20/2 Wu=3.7892 k/ft	$\frac{Vn}{\Omega} = \frac{42.1}{1.67} = 25.2$kip $\frac{Vn}{\Omega}$ =WaL/2 25.2=Wax20/2 Wa=2.52 k/ft

Shear Design by using AISC Tables (Table (3.2) (Zx Table) for W-shapes)

W-shape are stored in descending order by strong-axis flexural strength and then grouped in ascending order by weight with the lights W-shape in each rang in bold. Strong-axis available strength in flexure and shear are given for W-shapes with fy =50.

Limitation of using table (3.2) (Zx table)

- The W shape is available in table
- Yielding strengthen for steel material must be equal to $(fy = 50)$

F_y = 50 ksi

Table 3–2
W Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	ϕ_bM_{px}	M_{rx}/Ω_b	ϕ_bM_{rx}	BF		L_p	L_r	I_x	V_{nx}/Ω_v	ϕ_vV_{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
W36×800^h	3650	9110	13700	5310	7980	47.5	71.4	14.9	94.8	64700	2030	3040
W36×652^h	2910	7260	10900	4300	6460	46.8	70.4	14.5	77.8	50600	1620	2430
W40×593^h	2760	6890	10400	4090	6140	55.5	83.5	13.4	63.8	50400	1540	2310
W36×529^h	2330	5810	8740	3480	5220	46.5	70.0	14.1	64.4	39600	1280	1920
W40×503^h	2310	5760	8660	3460	5200	54.7	82.2	13.1	55.3	41600	1290	1940
W36×487^h	2130	5310	7990	3200	4800	46.1	69.3	14.0	60.0	36000	1180	1770
W40×431^h	1960	4890	7350	2950	4440	53.6	80.6	12.9	49.0	34800	1110	1660
W36×441^h	1910	4770	7160	2880	4330	45.2	68.0	13.8	55.5	32100	1060	1590
W27×539^h	1890	4720	7090	2740	4120	26.1	39.2	12.9	88.6	25600	1280	1920
W40×397^h	1800	4490	6750	2720	4100	52.3	78.7	12.9	46.6	32000	999	1500
W40×392^h	1710	4270	6410	2510	3780	60.4	90.8	9.33	38.3	29900	1180	1760
W36×395^h	1710	4270	6410	2600	3910	44.7	67.1	13.7	51.0	28500	937	1410
W40×372^h	1680	4190	6300	2550	3830	51.6	77.6	12.7	44.5	29600	943	1410
W14×730^h	1660	4140	6230	2240	3360	7.37	11.1	16.6	275	14300	1380	2060

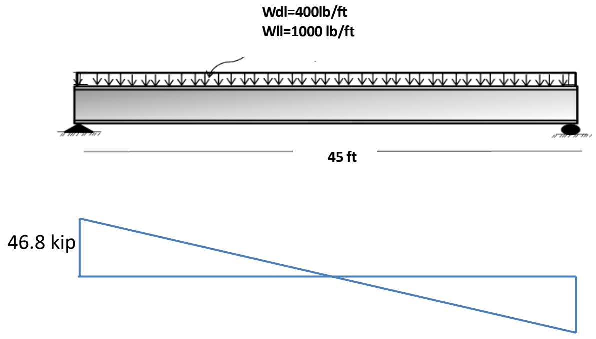
Example

W14x90 A992 steel material is used for simply supported beam with 45 ft is laterally supported at its ends and subjected to $W_{d.l}$ =400 lb/ft (including of self-weight) and $W_{l.l}$= 1000 lb/ft. Check the adequacy of the beam according to shear requirements Use LRFD method.

Solution

Steel	fy	fu
A992	50	65

Section	h/tw	d	tw	ϕvn
W14x90	25.9	14	0.44	185



$W_u = 1.2W_{d.l} + 1.6\, W_{l.l}$

=1.2x400/100+1.6x1000/1000= 2.08 k/ft

$V_{max} = \frac{WaxL}{2} = \frac{2.08x45}{2}$ =46.8 kip

46.8 <185 ok
The section is adequate

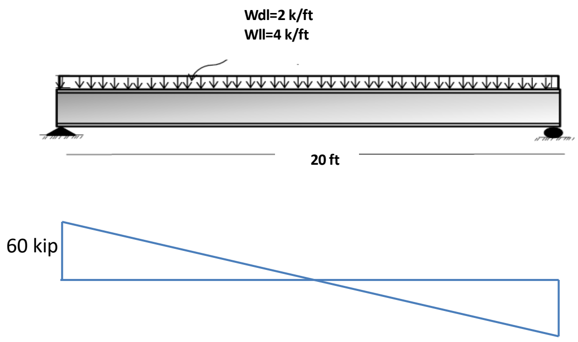
Example

W21x55 A992 steel material is used for simply supported beam with 20 ft is laterally supported at its ends and subjected to $W_{d.l}$ = 2k/ft (including of self-weight) and $W_{l.l}$= 4 k/ft. Check the adequacy of the beam according to shear requirements. Use ASD method

Solution

Steel	fy	fu
A992	50	65

Section	h/tw	d	tw	Vn/Ω
W21x55	50	20.8	0.375	156



$W_a = W_{d.l} + W_{l.l}$

=2 +4 = 6 k/ft

$V_{max} = \frac{WaxL}{2} = \frac{6x20}{2}$ = 60 kip

60<156 ok
The section is adequate

Home work in Shear Strength of a Steel Shape

Determine the design shear strength of the following, using A992 steel material for the W-shapes and A36 steel material for the C-shape:

1. W16 x 26
2. W18 x 50
3. C12 x 20.7