Shear Design of Beams

CE 470 - Steel Design Class

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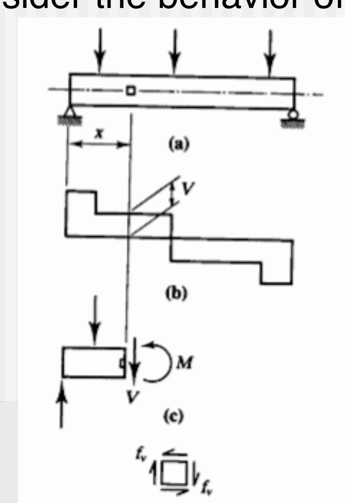
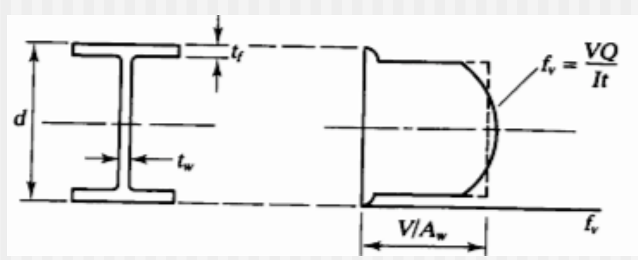
Shear Strength

- Beam shear strength is covered in Chapter G of the AISC specifications. Both rolled shapes and welded built-up shapes are covered.
- Rolled shapes is the focus here. Built-up shapes, commonly referred to as plate-girders are beyond the scope of our course.
- Consider the behavior of beams in shear





Shear Strength

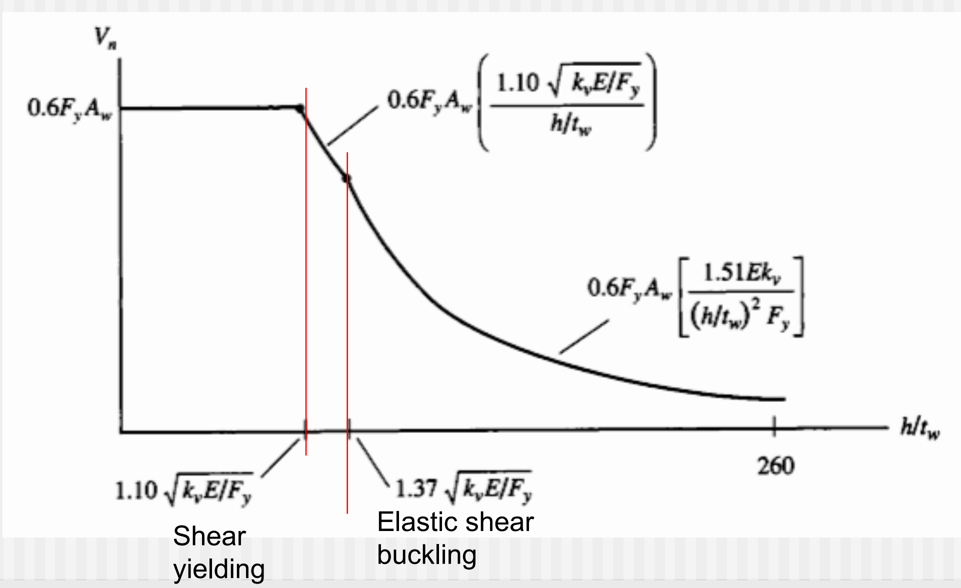
- The web will completely yield long before the flanges begin to yield. Because of this, yielding of the web represents one of the shear limit states.
- Taking the shear yield stress as 60% of the tensile yield stress.

$$f_v = \frac{V_n}{A_w} = 0.6\,F_y$$

$$\therefore V_n = 0.6\,F_y A_w$$

- This will be the nominal strength in shear provided there is no shear buckling of the web.
  - Shear buckling of the web depends on its h/t_w ratio.
  - If the ratio is too large, then then web can buckle in shear elastically or inelastically.

Shear Buckling



Shear Buckling

$\bullet$ *for unstiffened webs with* $h/t_w < 260$

$k_v = 5$

$\bullet$ *for stiffened webs*

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

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

where a = clear distance between transverse stiffners, in.

Shear Design Equations

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.

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

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

Shear Design Equations

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

$$V_n = 0.6 F_y A_w C_v \quad \text{(G2-1)}$$

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

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

and

$$C_v = 1.0 \quad \text{(G2-2)}$$

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).

Shear Design Equations

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

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

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

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

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

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

where

Shear Design Equations

A_w= area of web, the overall depth times the web thickness, dt_w, $in.^2$

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

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

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

= for tees, the overall depth, in.

t_w= thickness of web, in.

Shear Design Equations

The web plate shear buckling co-efficient, k_v, is determined as follows:

- For webs without transverse stiffeners and with $h/t_w < 260$:

$$k_v = 5$$

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

- For webs with transverse stiffeners:

$$k_v = 5 + \frac{5}{\left(\frac{a}{h}\right)^2} \qquad \text{(G2-6)}$$

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

where

a= clear distance between transverse stiffeners, in.

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.

Shear Design Equations

2. Transverse Stiffeners:

Transverse stiffeners are not required where ${}^{h}/_{t_w} \leq 2.46\sqrt{{}^{E}/_{F_y}}$, or where the available shear strength provided in accordance with Section G2.1 for $k_v = 5$ is greater than the required shear strength.

The moment of inertia, I_{st}, of the transverse stiffeners used to develop the available web shear strength, as provided in Section G2.1, about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners, shall meet the following requirement

$$I_{st} \geq b t_w^3 j \qquad \text{(G2-7)}$$

where

$$j = \frac{2.5}{\left({}^{a}/_{h}\right)^2} - 2 \geq 0.5 \qquad \text{(G2-8)}$$

and b is the smaller of the dimensions a and h.

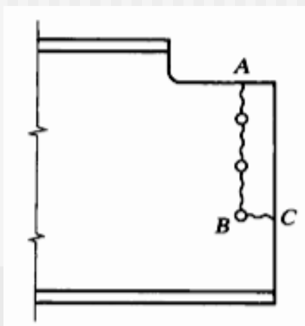
Shear Design Equations

Transverse stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which transverse stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe to the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange.

Bolts connecting stiffeners to the girder web shall be spaced not more than 12 in. (305 mm) on center. If intermittent *fillet weld*s are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 10 in. (250 mm).

Block Shear Failure of Beam

- Block shear failure was considered earlier in tension member connections.
- To facilitate the connections of beams to other beams so that the top flanges are at the same elevation, a short length of the top flange of one of the beams may be cut away, or *coped*.
- If a coped beam is connected with bolts as shown, segment ABC will tend to tear out.

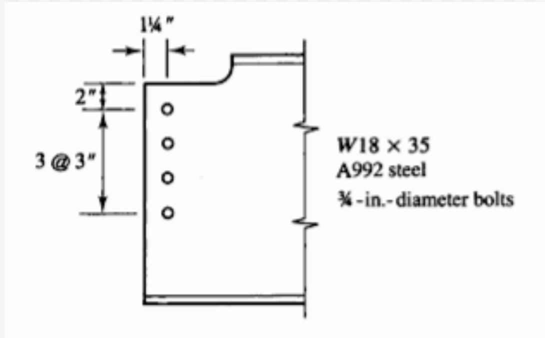


Block shear failure of beam ends

- The applied load in the case of the beam will be the vertical reaction, so shear will occur along line AB and there will be tension along BC. Thus, the block shear strength will be a limiting value of the reaction.
- Failure is assumed to occurred by rupture (fracture) on the tension area and rupture or yielding on the shear area.
- $R_n = 0.6\ F_u\ A_{nv} + U_{bs}\ F_u\ A_{nt} \leq 0.6\ F_y\ A_{gv} + U_{bs}\ F_u\ A_{nt}$
- Where, phi =0.75

Example

- Determine the maximum reaction, based on block shear, that can be resisted by the beam shown below. Treat the bolt end distance of 1.25 in. as standard.



- The effective hole diameter is 3/4 + 1/8 = 7/8 in.
- The shear areas are:
- $A_{gv} = t_w (2+3+3+3) = 0.30 (11) = 3.3 \text{ in}^2$
- $A_{nv} = 0.300 [11 - 3.5 (7/8)] = 2.381 \text{ in}^2$

Block shear example

- The net tension area is:
- $A_{nt} = 0.300 [1.25 - 1/2 (7/8)] = 0.2438 \text{ in}^2$
- Since the block shear will occur in a coped beam with standard bolt end distance $U_{bs} = 1.0$.
- $R_n = 0.6 F_u A_{nv} + F_u A_{nt} = 108.7$ kips
  - With an upper limit of
  - $R_n = 0.6 F_y A_{gv} + F_u A_{nt} = 114.85$ kips

- Therefore, nominal block shear strength = 108.7 kips
- Factored block shear strength for design = 0.75 x 108.7 = 81.5 kips.