Design of Trusses

1. INTRODUCTION

In trusses of proper arc welded design, gusset plates are generally eliminated. Tension members in the welded design are lighter because the entire cross-section is effective, and the amount of extraneous detail metal is reduced to a minimum.

Welded trusses may be designed in various ways, using T shapes, H and WF sections, etc. for chords. The diagonal members are usually angles. Various types of welded truss designs are illustrated in the following:

1. Perhaps the simplest type of truss construction is made of angle shapes and Tee's. In this example, the bottom and top chords are made of T sections, with angle sections for the diagonals. This is easy to fabricate and weld because the sections lap each other and fillet welds are used, Figure 1.

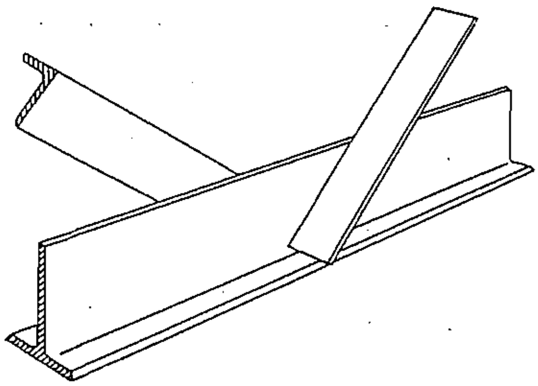


FIGURE 1

2. For a heavier truss, the vertical member can be an I or WF section. The web of this member, in the example illustrated, is slotted to fit over the stem of the T section. The T section is used for both the top and bottom chord members. The diagonal members are made of a double set of angles, Figure 2.

3. Some trusses make use of T sections for their diagonal members. The flanges of the diagonal members must be slotted to fit over the stem of the T section used for the top and bottom chords. The stem of the diagonal is also cut back and butt welded to the stem of the top and bottom chords, Figure 3.

4. Quite a few trusses are made of WF sections completely: both top and bottom chords as well as

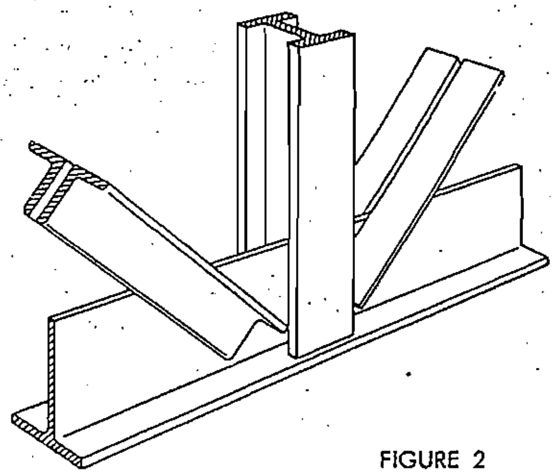


FIGURE 2

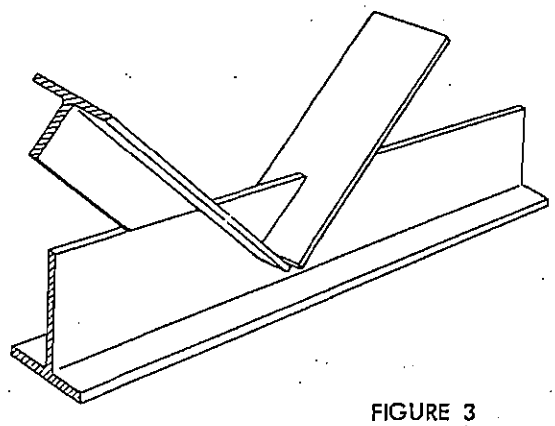


FIGURE 3

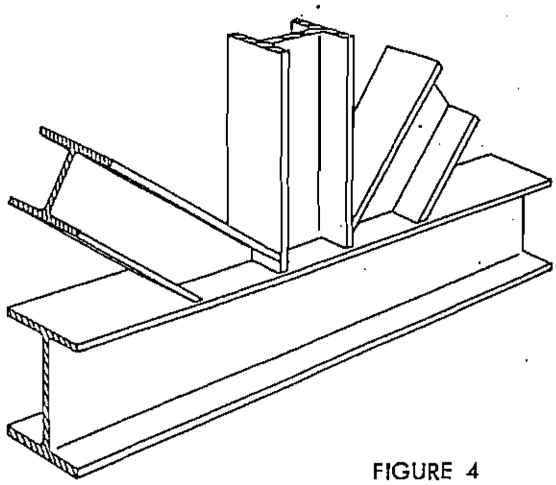


FIGURE 4

diagonal and vertical members. This allows loads to be placed anywhere along the top and bottom chords because of their high bending strength. (With the conventional truss design, loads must be placed only at points where diagonal or vertical members connect to the chord members.) Almost all of the welds are on the flanges of the top and bottom chords, and since these are flat surfaces, there is no difficult fitting of the members to make these connections, Figure 4.

5. Where longer lengths of connecting fillet welds are required, a simple flat plate may be butt welded directly to the stem of the horizontal T chord, without any joint preparation. This weld is then chipped or ground flush in the area where web members will connect, Figure 5.

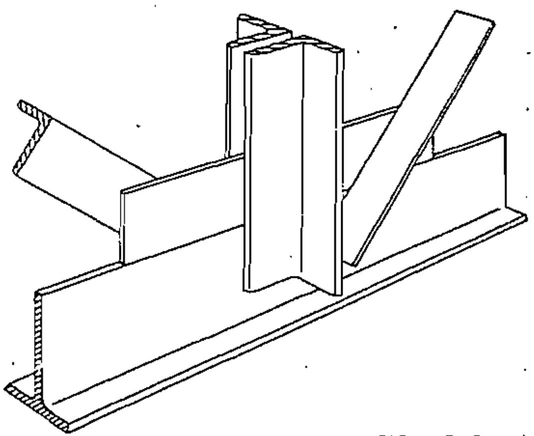


FIGURE 5

6. Sometimes heavier trusses are made of WF sections with the web of the top and bottom chords in the horizontal position. The welding of these members would consist mainly of butt welding the flanges together. Under severe loading, gusset plates may be added to strengthen the joint and reduce the possibility of concentrated stresses, Figure 6.

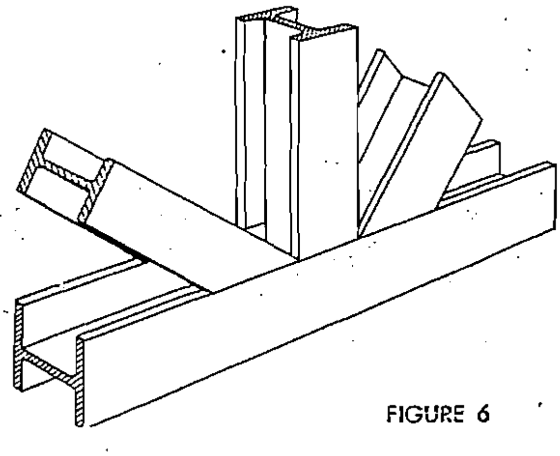


FIGURE 6

7. It is now possible to obtain hot-rolled square and rectangular tubular sections in A36 steel at about the same price as other hot-rolled sections. This type of section has many advantages. It has good resistance to bending, and has high moment of inertia and section modulus in both directions. It offers good strength in compression because of high radius of gyration in both directions. It is very easy to join by welding to other similar sections because of its flat sides. For lighter loads, fillet welds are sufficient. These sections offer good torsional resistance; this in turn provides greater lateral stability under compression, Figure 7.

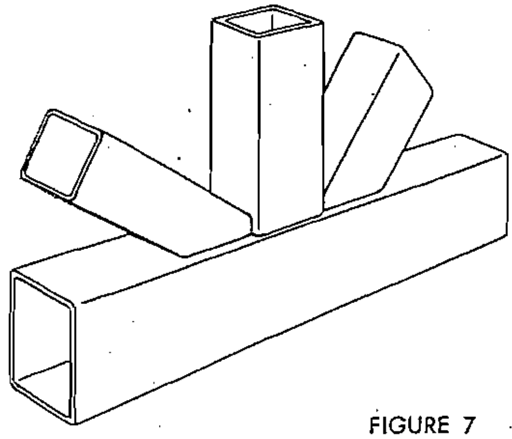


FIGURE 7

8. Round tubular sections or pipe have certain advantages in truss construction: good bending resistance, good compressive strength, and good torsional resistance. There is no rusting problem on the inside if they are sealed at the ends by welding, hence only the outside must be painted. Although it is more difficult to cut, fit, and weld the pipe sections together, this is not a problem for fitters and weldors experienced in pipe fabrication and welding. Pipe is used extensively in Europe for trusses. In this country it has been used for some mill buildings, special trusses for material handling bridges, extremely large dragline booms, off-shore drilling rigs, etc., Figure 8.

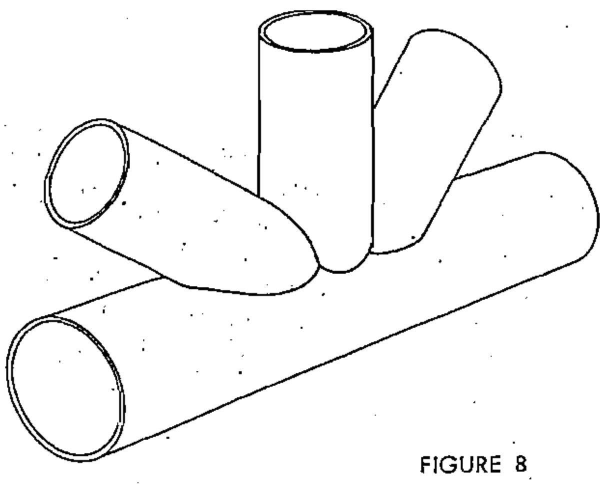
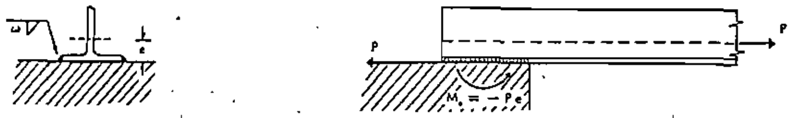
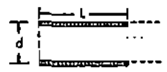
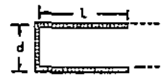
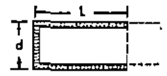


FIGURE 8

TABLE 1—Effect of Eccentric Loading



Welded connection	If consider moment $M_e = -Pe$	If neglect moment
	$P = \dfrac{2\,L^2\,f}{\sqrt{L^2 + 36\,e^2}}$	$P = 2\,L\,f$
	$P = \dfrac{f}{\sqrt{\left(\frac{3e}{L}\right)^2 \left(\frac{1}{2d + L}\right)^2 + \left(\frac{1}{d + 2L}\right)^2}}$	$P = (d + 2L)\,f$
	$P = \dfrac{f}{\left(\frac{3e}{L}\right)^2 \left(\frac{1}{6d + L}\right)^2 + \left(\frac{1}{2(d + L)}\right)^2}$	$P = 2(d + L)\,f$

$f = 9{,}600\ \omega$ A7, A373 steel & E60 welds
$f = 11{,}200\ \omega$ A36 steel & E70 welds

There are many methods by which to join the various pipe sections together in a truss. In this case, the pipe is cut back and a gusset plate is used to tie them together. A gusset plate also provides additional stiffness to the pipe within the connection area. However, they tend to cause an uneven stress distribution within the pipe, with rather high stresses in line with the gusset plate. See Figure 9.

These closed sections, with less surface area exposed to the elements, are less subject to corrosion than are open sections; in practically all cases they are left unpainted on the inside. It is only necessary to see that the ends are sealed by welding.

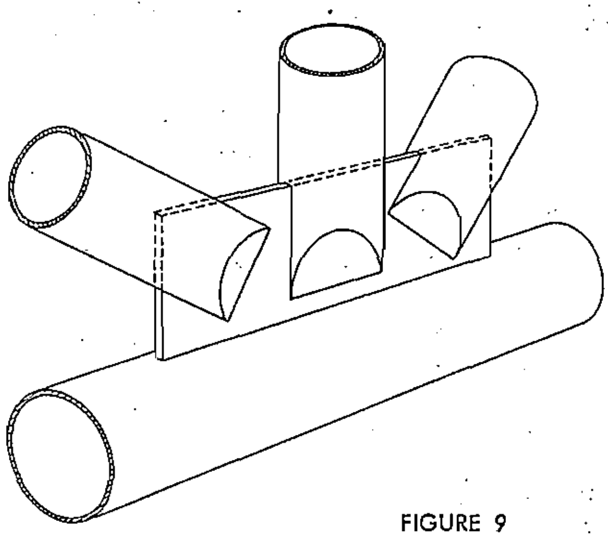


FIGURE 9

2. EFFECT OF ECCENTRIC LOADING

It can be shown that, with members back to back, or separated with a gusset plate, the connections will supply a restraining end moment:

$$M_e = -P\,e$$

Since this moment is equal and opposite to the moment due to the eccentric loading ($M = P\,e$), they will cancel. As a result there will be no moment through-

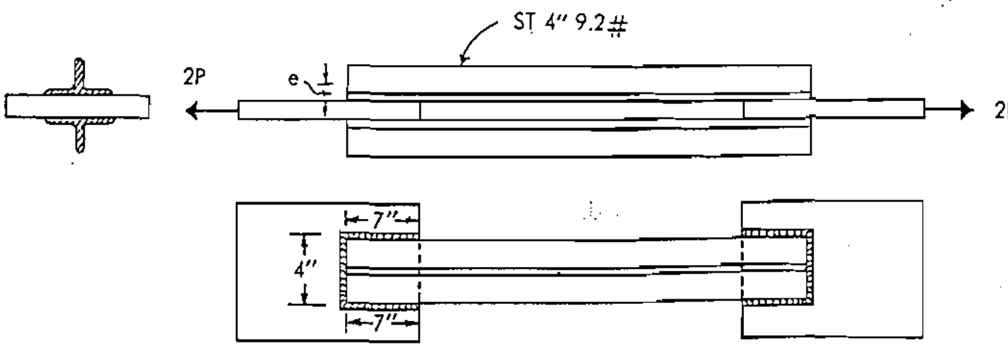


FIGURE 10

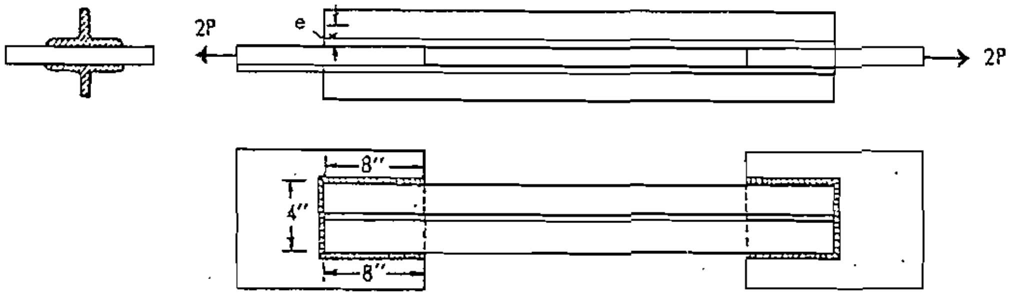


FIGURE 11

out the length of the member and it will remain straight.

However, this moment (M_e) is carried by the connecting welds in addition to their axial load (P). This moment is usually neglected in the design of the welded connection, because of the difficulty in determining the length of weld (L) when it is considered. Further, there usually is not much difference in the actual length of the required weld whether it is considered or not.

(a) If the moment (M_e) is neglected:

(See Figure 10.)

Assuming A373 steel and E60 welds,

$$A_T = 2.67 \text{ in.}^2$$

$$P = \sigma\ A_T$$
$$= (20{,}000)(2.67)$$
$$= 53.4 \text{ kips}$$

leg size of fillet weld

$$\omega = \tfrac{3}{4}\ t_f$$
$$= \tfrac{3}{4}\ (.425)$$
$$= .3185'' \text{ or } \tfrac{5}{16}'' \triangle$$

total length of weld

$$L_T = \frac{P \text{ kips}}{\tfrac{5}{16}\ (9.6) \text{ kips/in.}}$$
$$= \frac{(53.4)}{(3)}$$
$$= 17.8''$$

This would be distributed 4″ across the end, returning 6.9″ on the sides, or use 7″ long on each side. This would give a total length of 18″ of $\tfrac{5}{16}'' \triangle$ weld.

(b) *If the moment (M_e) is considered:*

(See Figure 11.)

Here:

$$e = y = .94''$$
$$d = 4''$$
$$\omega = \tfrac{5}{16}''$$
$$P = 53.4 \text{ kips}$$

since:

$$P = \frac{f}{\sqrt{\left(\frac{3e}{L}\right)^2 \left(\frac{1}{2d + L}\right)^2 + \left(\frac{1}{d + 2L}\right)^2}}$$
$$= \frac{9600\ (\tfrac{5}{16})}{\sqrt{\left(\frac{3 \times .94}{L}\right)^2 \left(\frac{1}{8 + L}\right)^2 + \left(\frac{1}{4 + 2L}\right)^2}}$$
$$= 53.4 \text{ kips}$$

and from this we find L = 8″. (This value was found by plotting several values of L on graph paper and selecting that L value which gave the closest value of P = 53.4 kips.) This would give a total length of 20″ of $\tfrac{5}{16}'' \triangle$ weld.

In this case, the extra work involved in considering the moment did not pay for the very slight overstress in the weld when the moment was neglected.

If only one member is used, and the plate to which it is attached is not very rigid, this restraining end moment will not be set up. The member will then have a moment due to the eccentric load (M = P e), in addition to its axial load (P). See Figure 12.

axial tensile stress in member

$$\sigma = \frac{P}{A}$$

bending stress

$$\sigma = \frac{M\ c}{I} = \frac{P\ y^2}{I}$$

Since the distance to the outer tensile fiber (c) and the distance of the section's center of gravity from the base line (y) are equal, and since the eccentricity of

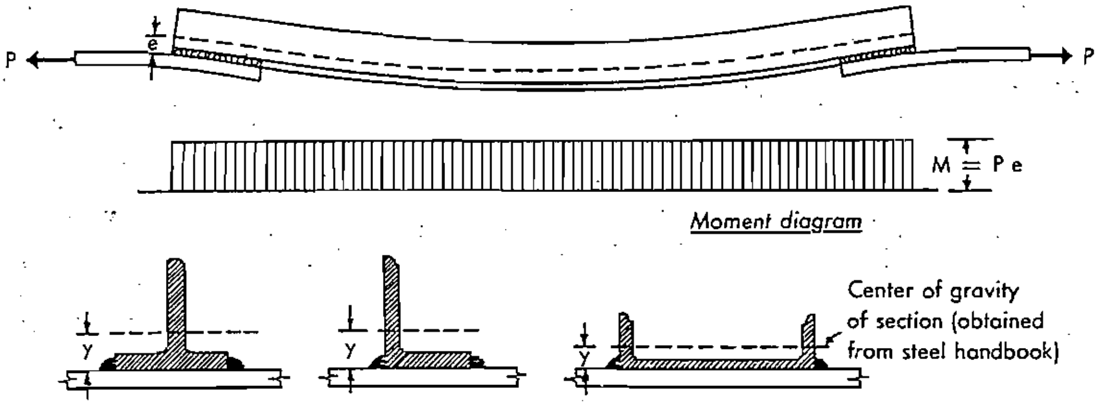


FIGURE 12

loading (e) is nearly equal to these, it is assumed for simplicity that c = e = y. Therefore, the total (maximum) stress is—

$$\boxed{\sigma = \frac{P}{A} + \frac{P\ y^2}{I}} \qquad (1)$$

or the maximum axial load (P) for a given allowable stress (σ) is—

$$\boxed{P = \frac{\sigma}{\frac{1}{A} + \frac{y^2}{I}}} \qquad (2)$$

For the ST 4″ I 9.2# member used in the previous example, Figure 10, this additional moment due to eccentricity of loading would reduce the member's allowable axial tensile force to:

$$P = \frac{\sigma}{\frac{1}{A} + \frac{y^2}{I}} = \frac{(20{,}000)}{\frac{1}{2.67} + \frac{(.94)^2}{(3.50)}}$$

$$= 32 \text{ kips}$$

In this particular case, the additional moment due to the eccentrically applied axial load reduces the member's allowable load carrying capacity by 40%. This far exceeds any reduction in the strength of the welded connection due to this moment. Thus, the connection will be on the conservative side.

Conclusions:

(a) If the attaching plate is very flexible and offers no restraining action at the end of the member, the full moment (M = P e) must be added to the member and no moment added to the connection. In other words, the connection is designed for the transfer of the axial force only.

(b) If the attaching plate is rigid enough so there is no end rotation of the member, this moment is not added to the member, but must be added to the connection.

Even in this example, if the moment were also figured to be added to the connection, at the reduced load of P = 32 kips, it would not require as much weld as in the previous case:

Here:

$\omega = 5/16''$ $\qquad$ $d = 4''$

$e = .94''$ $\qquad$ $P = 32$ kips

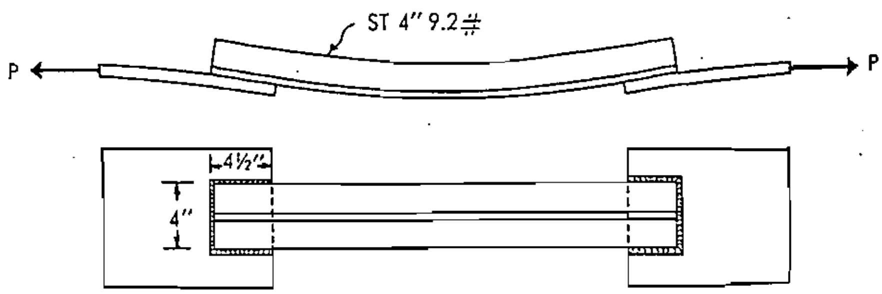


FIGURE 13

since:

$$P = \frac{f}{\sqrt{\left(\frac{3e}{L}\right)^2 \left(\frac{1}{2d + L}\right)^2 + \left(\frac{1}{d + 2L}\right)^2}}$$

$$= \frac{9600\ (5/16)}{\sqrt{\left(\frac{3 \times .94}{L}\right)^2 \left(\frac{1}{8 + L}\right)^2 + \left(\frac{1}{4 + 2L}\right)^2}}$$

$$= 32 \text{ kips}$$

From this we find L = 4.4″ or = 4½″. (This value was found by plotting several values of L on graph paper and selecting that which gave the closest value of P = 32 kips.) This would give a total length of 13″ of 5/16″ ◺ weld.

This is another case where theory would indicate a much higher reduction in the carrying capacity of a connection than actual testing shows. The following lap joints were welded and pulled to failure.

(a) *calculated allowable load:*

$$P = \frac{2\ L^2\ f}{\sqrt{L^2 + 36\ e^2}}$$

$$= \frac{19{,}200\ (2)^2\ (1/4)}{\sqrt{2^2 + 36\ (1/4)^2}}$$

$$= \underline{7500 \text{ lbs}}$$

(b) *calculated allowable load:*

$$P = \frac{2\ L^2\ f}{\sqrt{L^2 + 36\ e^2}}$$

$$= \frac{19{,}200\ (2)^2\ (1/4)}{\sqrt{2^2 + 36\ (1)^2}}$$

$$= \underline{3040 \text{ lbs}}$$

Theory would indicate that, in the above samples, increasing the eccentricity (e) from ¼″ up to 1″ would decrease the strength of the welds by 60%.

Yet, the actual test results showed:

(a) f = 11,260 lbs/in.

(b) f = 10,280 lbs/in.

or that this large increase in eccentricity (e), from ¼″ to 1″, only decreased the strength by 8.7%:

The reasons for neglecting this eccentricity in the detailing of most connections may be summarized as follows:

1. In the usual welded connection, the eccentricity is not very large, and in these cases the theoretical reduction in strength due to the additional moment induced by the eccentricity is not very much.

2. Actual test results indicate a much smaller decrease in strength due to this eccentricity than theory would indicate. Also these test pieces were very short; the usual member would be much longer and, if any-

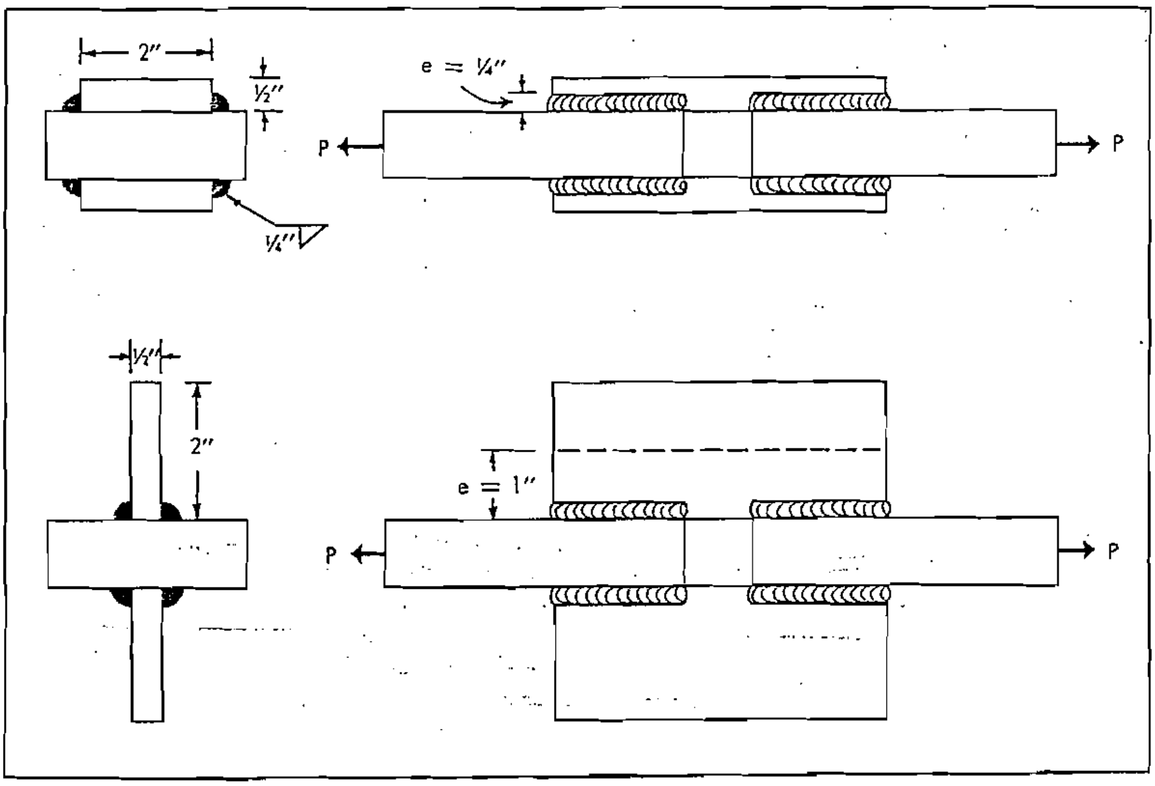


FIGURE 14

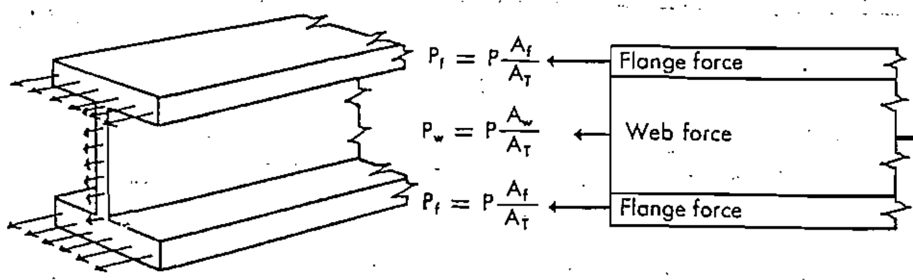


FIGURE 15

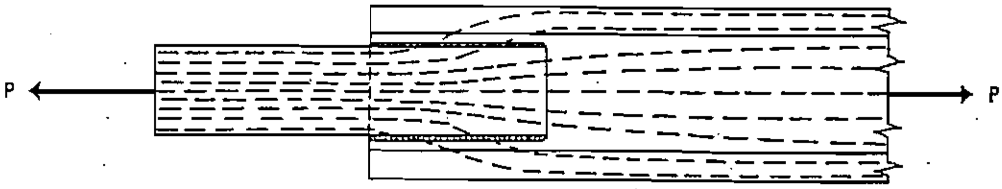


FIGURE 16

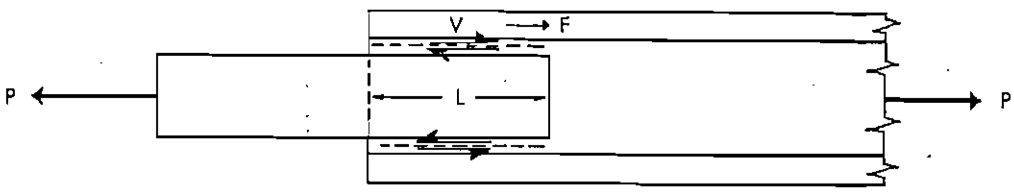


FIGURE 17

thing, would minimize this problem.

3. The eccentric loading would effect a reduction in strength of the member several times greater than any reduction in the strength of the welded connection.

4. It is very time-consuming to include this moment in consideration of the connection.

AISC Sec 1.15.3 requires that welds at the ends of any member transmitting axial force into that member shall have their center of gravity line up with the gravity axis of the member unless provision is made for the effect of the resulting eccentricity. However, except for fatigue loading conditions, fillet welds connecting the ends of single angles, double angles, and similar types of members (i.e. having low center of gravity or neutral axis, relative to attaching surface) need not be balanced about the neutral axis of the member.

3. DISTRIBUTION AND TRANSFER OF FORCES

It is assumed that the axial forces in a member are uniformly distributed throughout the various elements of the cross-section.

See Figure 15, where:

A_f = area of flange

A_w = area of web

A_T = total area of section

If the force in some element of a member cannot be transferred directly through the connection, this portion of the force must work its way around into another element of the member which can provide this transfer. See Figure 16.

This decrease in axial force (F) of one element of a member is accomplished through a transfer in shear (V) into another element. See Figure 17.

The length of this shear transfer (L) must be sufficient so that the resulting shear stress (τ) within this area does not exceed the allowable. This area may also have to be reinforced with doubler plates so it can safely carry this increased axial force.

If we assume uniform distribution of axial stress through the cross-section of the following member, then the web area has a force of P_w.

(See Figure 18.)

Shear transfer from web:

$V_w = P_w = \sigma\ A_w$ and

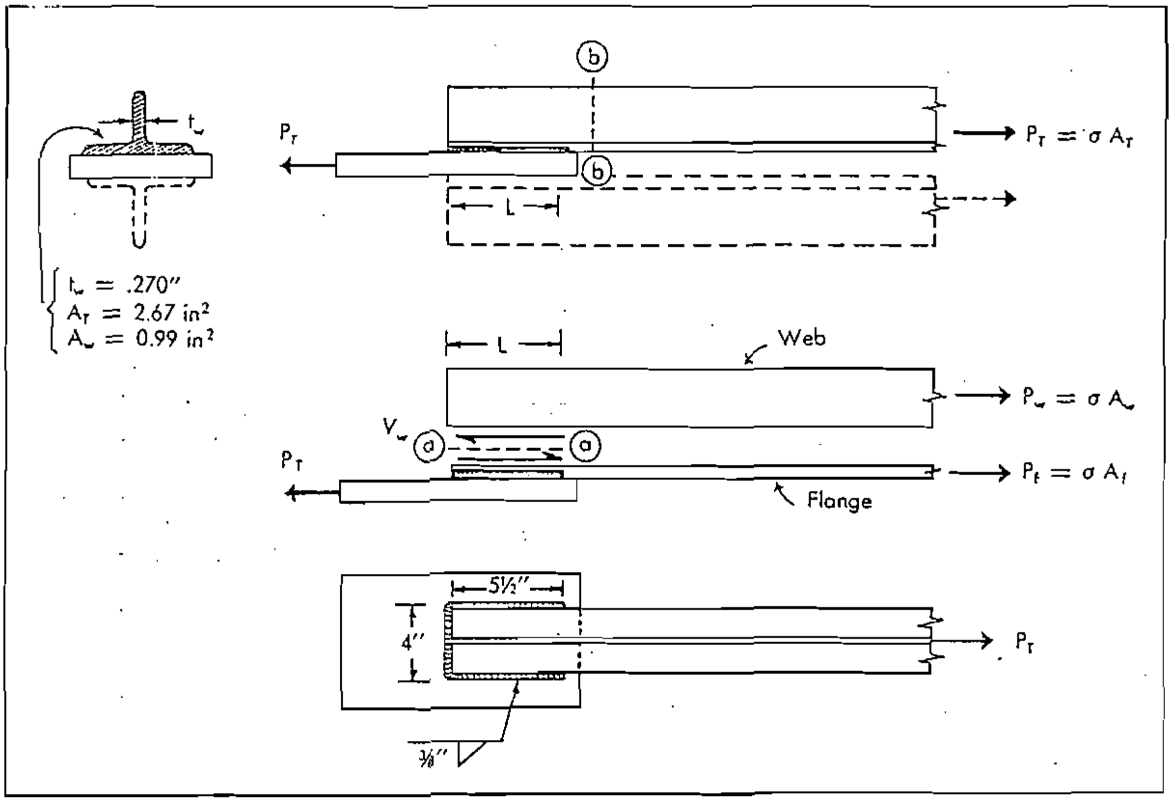


FIGURE 18

$$P_w = \sigma A_w$$
$$= (20{,}000)(.99)$$
$$= 19.8 \text{ kips}$$

This force in the web area ($P_w = 19.8$ kips) must be transferred down into the flange by shear (V_w), and out into the connection.

Theoretically, if the section is not to be stressed above its allowable, this shear transfer (V_w) must take place within a length bounded by the connecting welds.

If this is true, then this 19.8-kip force in the web, transferred as shear through a length of 5½″ where the flange joins the web, causes a shear stress in the section (a-a) of:

$$\tau = \frac{P_w}{A_w}$$
$$= \frac{(19.8 \text{ kips})}{(.270)(5\tfrac{1}{2})}$$
$$= 13{,}330 \text{ psi} > 13{,}000 \text{ psi (A373 steel)}$$

This is close enough. However, if it were higher, it would indicate that one of the following conditions exists:

a. The shear transfer takes place over a greater distance and, beyond the welds, must travel this short distance in the flange as additional tension until the weld is reached. It thus slightly overstresses the section (b-b) in tension.

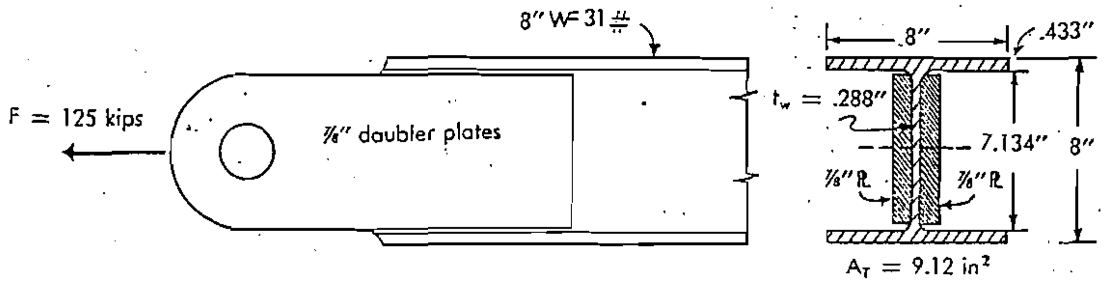


FIGURE 19

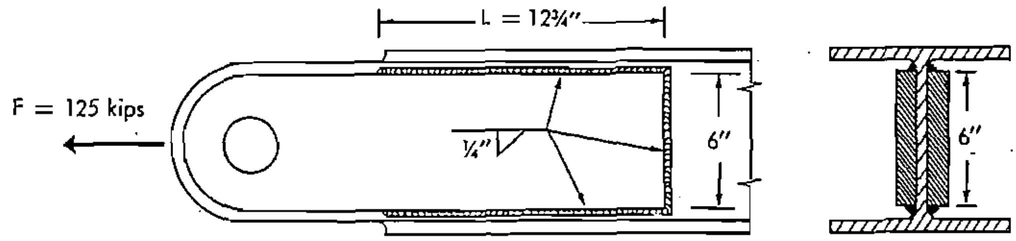


FIGURE 20

b. The shear transfer does take place within this 5½″ length, and slightly overstresses this section (a-a) in shear.

In most cases the welded connection will provide sufficient length (a-a) for the proper transfer of these forces from one portion of the member to another.

Problem 1

To detail an attachment to the tension member shown in Figure 19.

If we assume the total axial tensile force (F = 125 kips) is divided among the two flanges and web of the beam by the ratio of their areas to the total area, then the force in the flange which must be transferred out is—

$$F_f = F \frac{A_f}{A_T}$$

$$= (125)\ \frac{(.433 \times 8)}{(9.12)}$$

$$= 47.5 \text{ kips}$$

(a) If the doubler plates are 6″ wide, this flange force (F_f = 47.5 kips) must first transfer into the beam web along the length (L) as shear, V = 47.5 kips.

This length (L) must be—

$$L = \frac{V}{t_w\ \tau} \qquad \text{(See Figure 20.)}$$

$$= \frac{(47.5 \text{ kips})}{(.288)\ (13{,}000)}$$

$$= 12.7'' \text{ or } \underline{12\tfrac{3}{4}''}$$

The leg size of these parallel welds would be based upon the force on the weld:

$$f = \frac{V}{2\ L}$$

$$= \frac{(47.5 \text{ kips})}{2(12\tfrac{3}{4})}$$

$$= 1865 \text{ lbs/in.}$$

$$\omega = \frac{\text{actual force}}{\text{allowable force}}$$

$$= \frac{(1865)}{(9600)}$$

$$= .194'' \text{ or } \underline{\text{use } \tfrac{1}{4}''} \text{ (A373 steel; E60 weld)}$$

(b) If the doubler plates are 7″ wide and are welded directly to the inside of the flanges of the WF section, the flange force (F_f = 47.5 kips) will transfer directly through the parallel welds. See Figure 21.

If the leg size of these parallel fillet welds is ω = ½″, the length of these welds would be—

$$L = \frac{F_f}{2(9600\ \omega)}$$

$$= \frac{(47.5 \text{ kips})}{2(9600)(\tfrac{1}{2})}$$

$$= 4.95'' \text{ or } \underline{\text{use } 5''}$$

Transverse Forces

Any transverse component of a force applied to a member is carried by those elements of the member which lie parallel to this force. In other words, a vertical force applied to an I beam with the web vertical is carried as

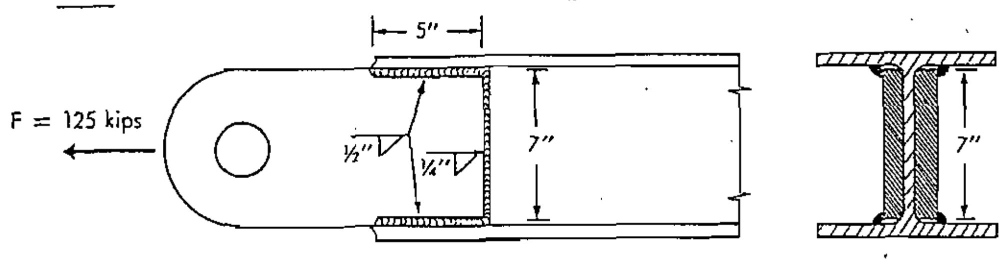


FIGURE 21

shear almost entirely by the web. If the web is horizontal, this force is carried as shear almost entirely by the two flanges. See Figure 22.

In a truss connection subject to a moment (for example, a Vierendeel Truss), the applied moments, if unbalanced, cause shear forces (V) around the periphery of the connection web. The resulting diagonal compression from these shear forces can buckle the web if it is not thick enough. See Figure 23.

The Law of Force and Reaction states that in a member constrained by its supports, an applied force at any point sets up at this point an equal, collinear, opposite reaction. This of course assumes the member to be a rigid body, that is one which does not change its shape or dimensions.

In the following member which is supported, the applied force (F) has two components: horizontal (F_h) and vertical (F_v). The result is two reactions in the member: vertical (R_v) in the web stiffener, and horizontal (R_h) for the most part in the lower flange. See Figure 24.

In order for one of these components of the applied force to be transferred into another member, it is necessary for the other component to be transferred also.

Figure 25 illustrates this. If either one of the force components cannot be carried (F_v in this example, because there is no stiffener), there will be little or no transfer of the other component (here F_h) even though there is a member or element present to do this. In other words the amount of a force component (here F_h) which may be transferred into the member depends on the ability of the connection to transfer the

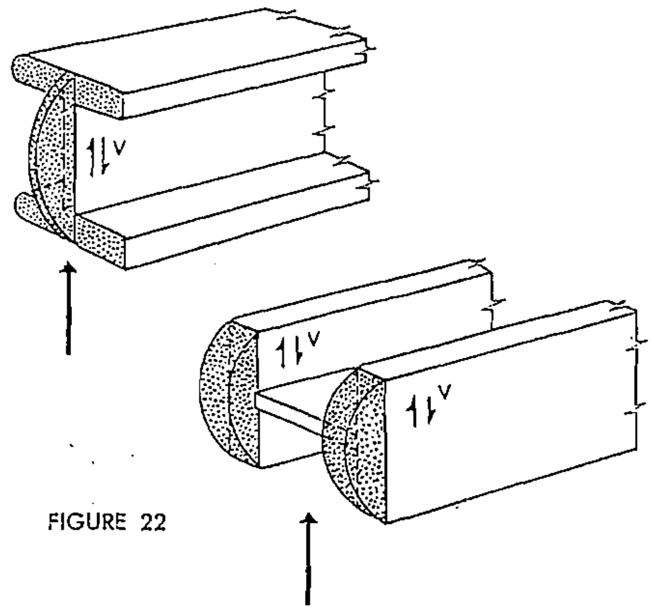
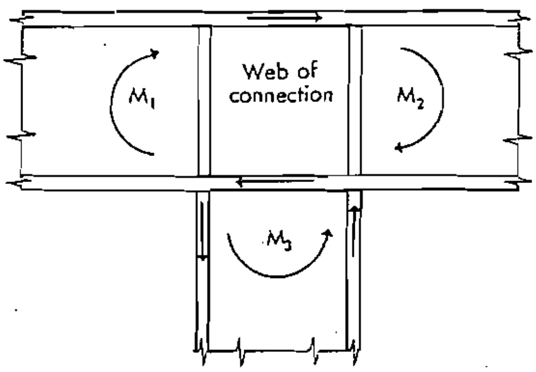


FIGURE 22



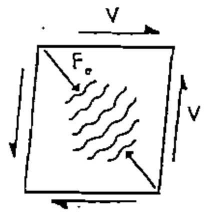


FIGURE 23

Diagonal compression on web of connection due to shear forces from unbalanced moment

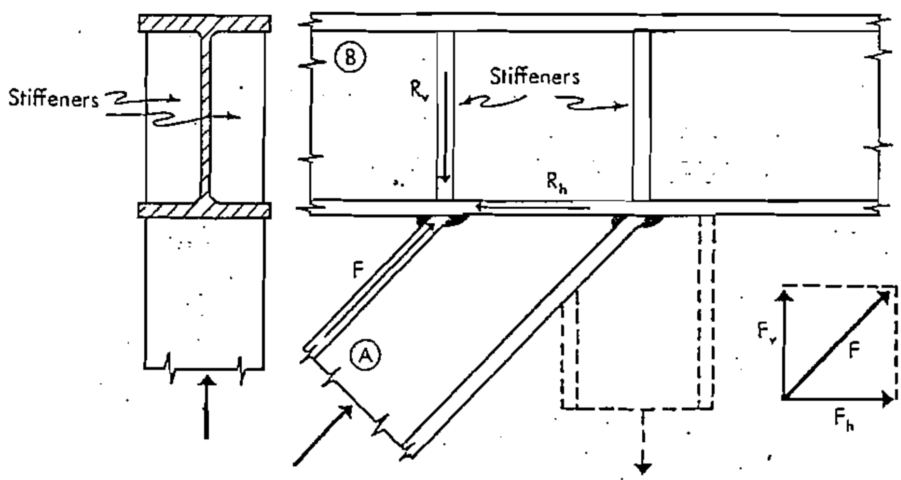


FIGURE 24

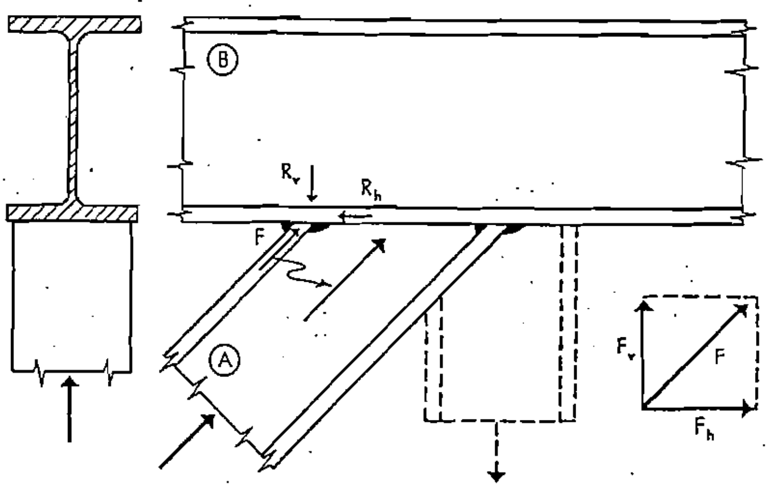


FIGURE 25

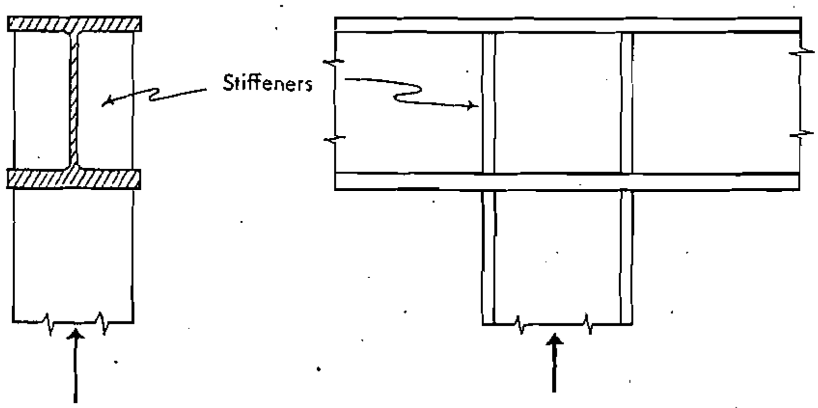


FIGURE 26

other component (here F_v). Of course the applied force (F) will be reduced also, and under these conditions some other portion of this member must transfer it. In this case the web of member A will transfer the balance of the force (F).

Determining Need for Stiffeners

Normally stiffeners would be added to a member in which large concentrated transverse forces are applied.

However, for smaller members with lower forces, these stiffeners are sometimes left off in truss connections. It is difficult to know under what conditions this might have to be stiffened.

In recent research at Lehigh University on "Welded Interior Beam-to-Column Connections", short sections were tested under transverse compression as well as tension, with and without stiffeners. See Figure 27.

It was found that the compressive force applied over a narrow section (t_f) of member's flange spread out over a wide section of the web by the time the net web thickness was reached. A conservative value for this distance is given as:

$$(t_f + 5K)$$

where K = the distance from the outer face of the flange to the web toe of the fillet. This value for all rolled sections may be found in any steel handbook.

t_f = thickness of the flange of the connecting member which supplies the compressive force.

Although there was no axial compression applied to the member in this test, on subsequent work involving actual beam-to-column connections, axial compression was simultaneously applied. See Figure 28.

It was found that an axial compressive stress of about 1.65 times the working stress (14,500 psi), or $\sigma = 24{,}000$ psi, had little effect on the strength of the connection. At the end of each test with the final loads left on the beams, this axial compressive stress was increased to twice the working stress or $\sigma = 29{,}000$ psi with no indication of trouble in the connection.

From this, they concluded that the minimum web thickness of the column for which stiffeners are not required is found from the following:

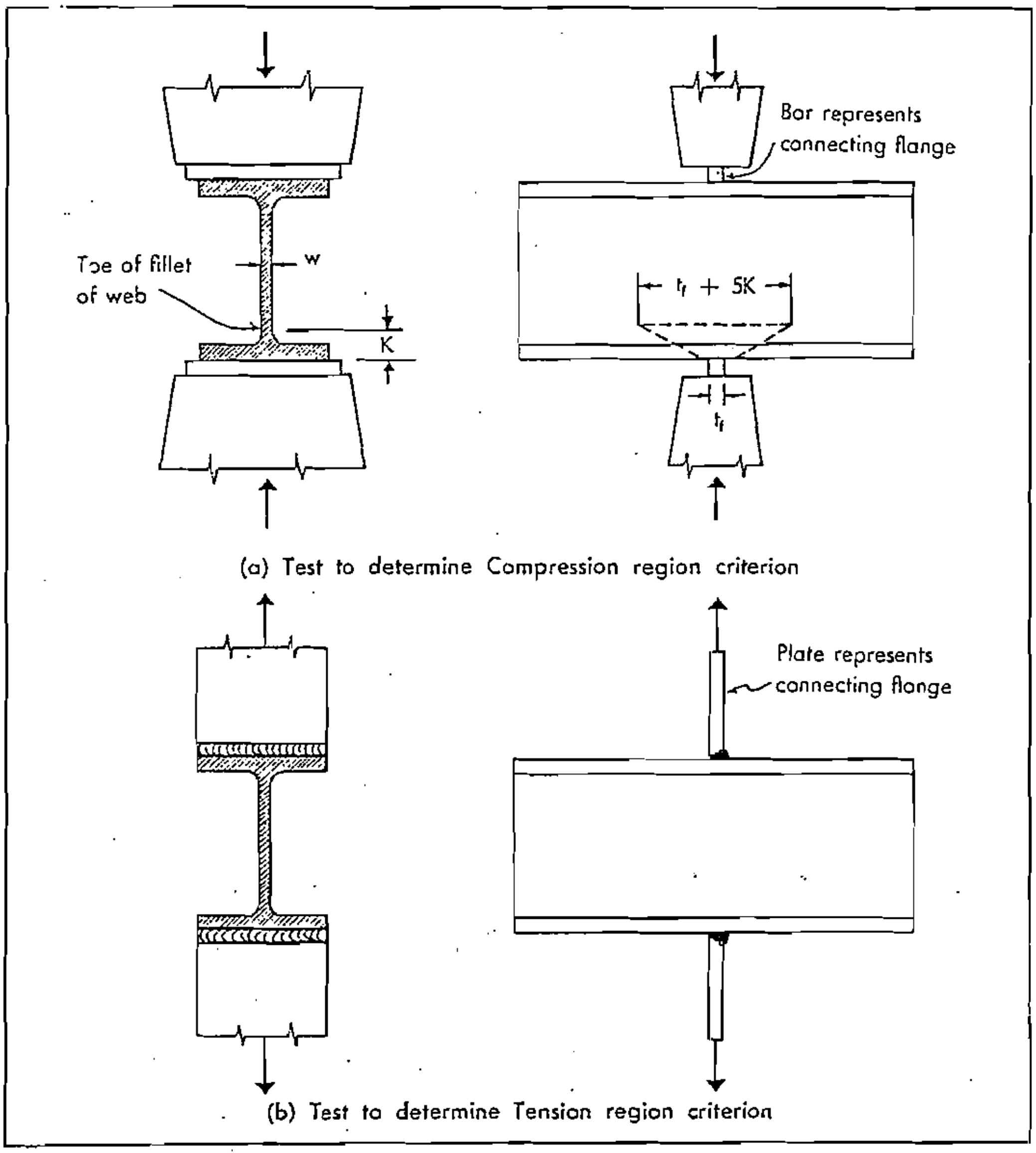


FIGURE 27

$$w \geqq \frac{t_f \, b_b}{t_f + 5K}$$

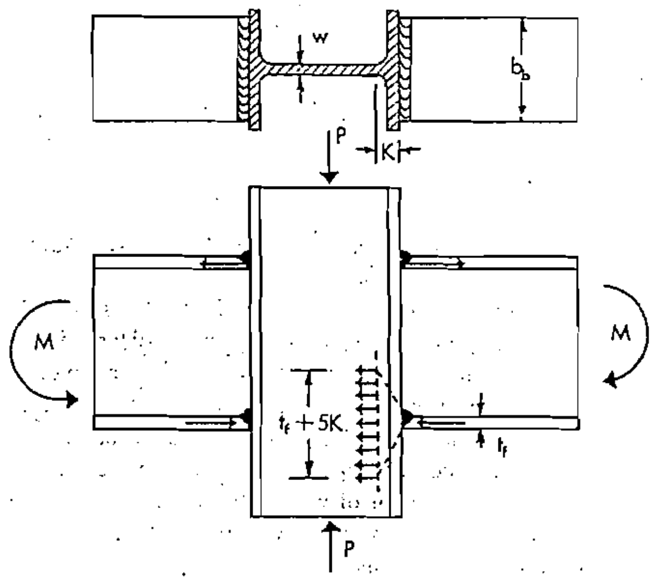


FIGURE 28

This research, concerned with the application of concentrated flange forces applied to flanges of WF members, was directed toward beam-to-column connections. However, it does seem reasonable to use this as a guide for the distribution of flange forces in truss connections. This will then provide an indication of the stresses in the chord resulting from the flange force of the connecting member.

In the test of the tension area, they found that the thickness of the column flange (t_c) determined whether stiffeners were required. On the basis of their tests, they made the following analysis.

Analysis of Tension Region of Connection

The following is adapted from "Welded Interior Beam-to-Column Connections", AISC 1959.

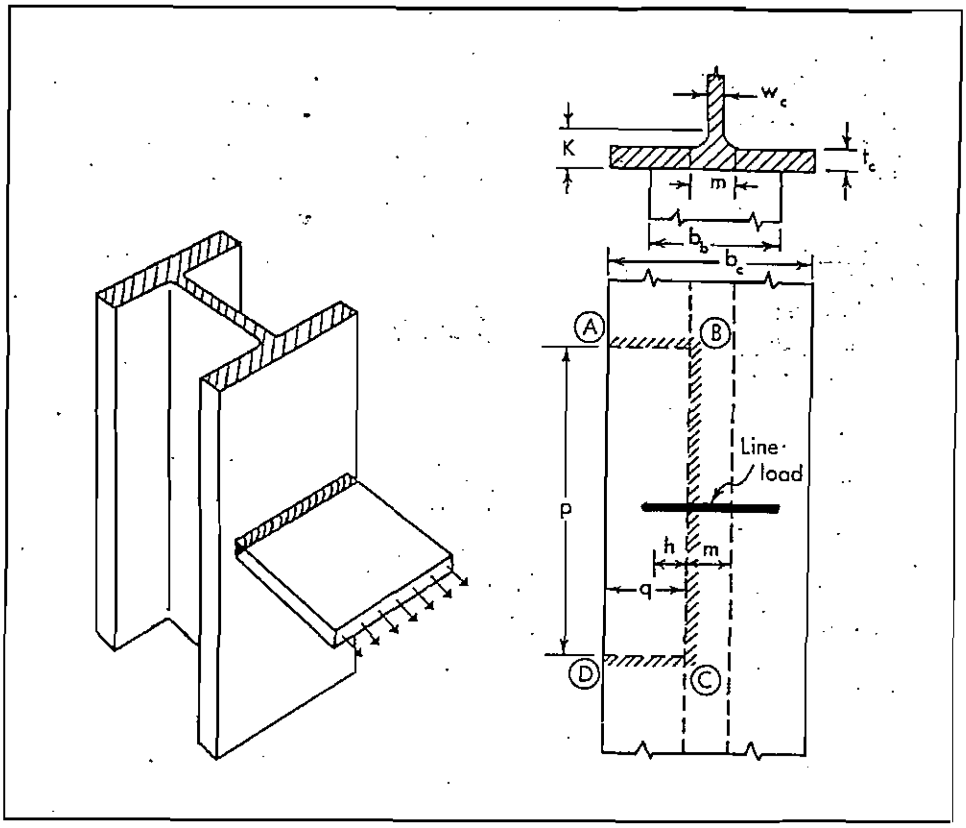


FIGURE 29

The column flange can be considered as acting as two plates, both of type ABCD; see Figure 19. The beam flange is assumed to place a line load on each of these plates. The effective length of the plates (p) is assumed to be 12 t_c and the plates are assumed to be fixed at the ends of this length. The plate is also assumed to be fixed adjacent to the column web.

See Figure 29, where:

$$m = w_c + 2(K - t_c)$$

$$q = \frac{b_c - m}{2}$$

$$h = \frac{b_b - m}{2}$$

$$p = 12\ t_c$$

Analysis of this plate by means of yield line theory leads to the ultimate capacity of this plate being—

$$Pu = c_1\ \sigma_y\ t_c^2$$

where:

$$c_1 = \frac{\frac{4}{\beta} + \frac{\beta}{\eta}}{2 - \frac{\eta}{\lambda}}$$

$$\eta = \frac{\beta}{4}\left[\sqrt{\beta^2 + 8\lambda} - \beta\right]$$

$$\beta = \frac{p}{q}$$

$$\lambda = \frac{h}{q}$$

For the wide-flange columns and beams used in practical connections, it has been found that c_1 varies within the range of 3.5 to 5. A conservative figure would be—

$$P_u = 3.5\ \sigma_y\ t_c^2$$

The force carried by the central rigid portion of the column in line with the web is—

$$\sigma_y\ t_b\ m$$

Setting this total force equal to that of the beam's tension flange:

$$\sigma_y\ t_b\ m + 7\ \sigma_y\ t_c^2 = \sigma_y\ b_b\ t_b$$

Reducing the strength of this column region by 20% and making the conservative assumption that $m/b_b = .15$, this reduces to the following:

$$(.80)\ \sigma_y\ t_b\ (.15\ b_b) + (.80)\ 7\ \sigma_y\ t_c^2 = \sigma_y\ b_b\ t_b$$

$$t_c^2 = \frac{b_b\ t_b - .12\ b_b\ t_b}{5.6}$$

or $\boxed{t_c \geqq .40\sqrt{b_b\ t_b}}$ ....(3)

If the column flange has this thickness, stiffeners are not required as far as the tension area is concerned.

We might carry this thought one step further and apply it to a tension flange which connects to the member at an angle other than 90°, such as in a truss connection. See Figure 30.

resistance of supporting flange (t_c)

$$P = (.80)\ \sigma_y\ t_b\ (.15\ b_b) + (.180)\ 7\ \sigma_y\ t_c^2$$

pull of tension flange (t_b)

$$P_1 = b_b\ t_b\ \sigma_y$$

$$\therefore\ (.80)\ \sigma_y\ t_b\ (.15\ b_b) + (.80)\ 7\ \sigma_y\ t_c^2 = b_b\ t_b\ \sigma_y \sin\alpha$$

or $\boxed{t_c \geqq \sqrt{\dfrac{b_b\ t_b\ (\sin\alpha - .12)}{5.6}}}$ ....(4)

Application to Truss Connections

This Lehigh work for beam-to-column connections will now be applied as a guide for determining the distribution of compressive forces in a truss connection.

It is assumed that this transfer of the flange force of (A) occurs in the web of member (B) within distance of $(t + 5K)$. See Figure 31.

Here:

$$t = \frac{t_b}{\sin\phi}$$

The vertical component of the web force of member (A) transfers directly into the web of member (B) within the distance of $\dfrac{d}{\sin\phi}$

Within the region b–c, these compressive stresses in the web of member (B) overlap and would be added.

$$\sigma = \frac{F_f \sin\phi}{\left(\dfrac{t_b}{\sin\phi} + 5K\right)w} + \frac{F_w \sin\phi}{\left(\dfrac{d}{\sin\phi}\right)w}$$

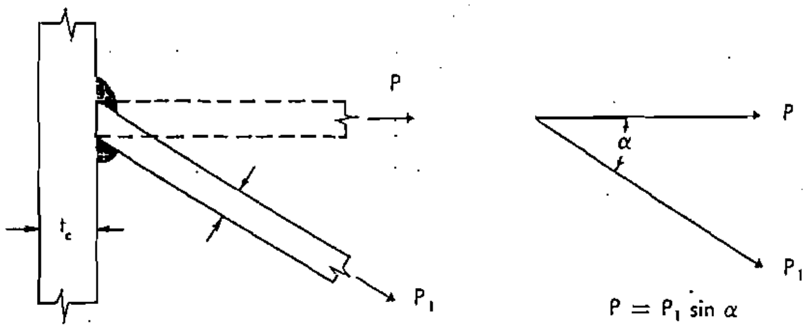


FIGURE 30

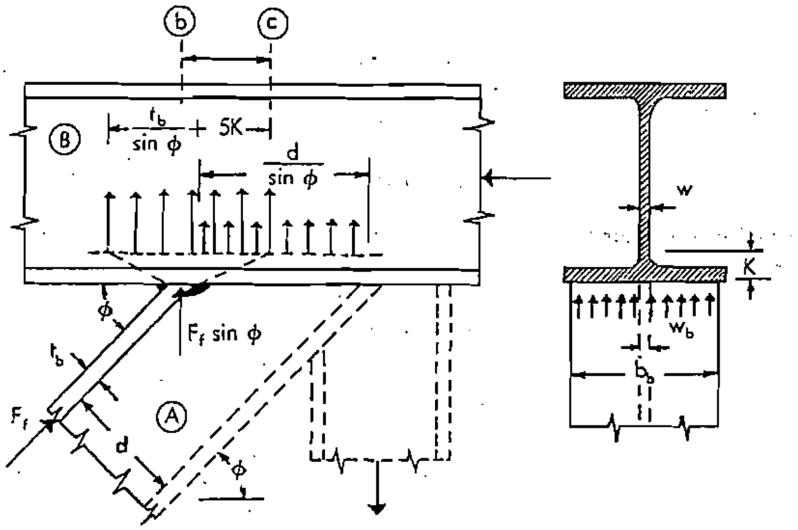


FIGURE 31

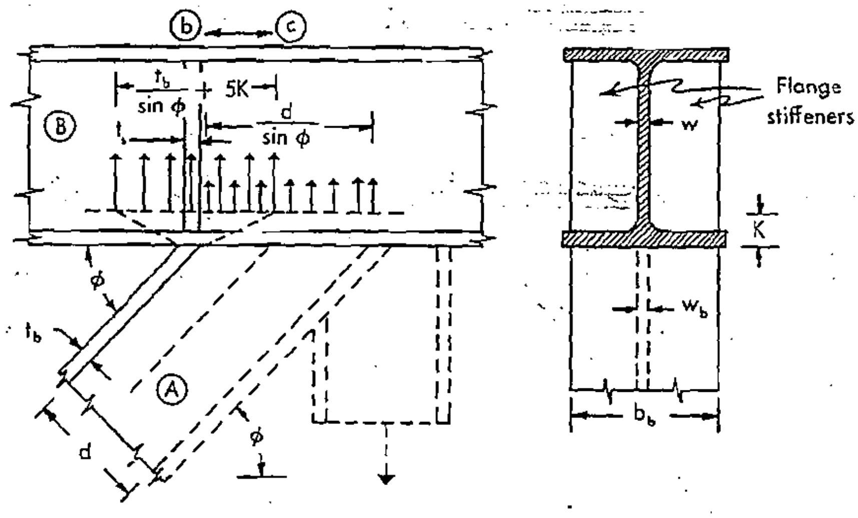


FIGURE 32

$$\text{or} \quad \boxed{\sigma = \frac{\sin^2\phi}{w}\left[\frac{F_f}{t_b + 5K \sin\phi} + \frac{F_w}{d}\right]} \quad \ldots\ldots(5)$$

Another method would be to assume ultimate load conditions, with all parts involved, stressed to yield. Using the previous formula (5):

where:

$F_f = b_b\ t_b\ \sigma_y$

$F_w = d\ w_b\ \sigma_y$

$$\sigma_y = \frac{\sin^2\phi}{w}\left[\frac{b_b\ t_b\ \sigma_y}{t_b + 5K \sin\phi} + \frac{d\ w_b\ \sigma_y}{d}\right]$$

$$\text{or} \quad \boxed{w \geqq \sin^2\phi\left[\frac{b_b\ t_b}{t_b + 5K \sin\phi} + w_b\right]} \quad \ldots\ldots(6)$$

If the thickness of the web (w) of member B satisfies this formula, stiffeners are not required. Normally, member A will not be stressed up to its allowable in compression, so that this shorter method of checking stiffener requirements is on the conservative side.

4. VERTICAL STIFFENERS

If Formula 6 should indicate that stiffeners are required, the same method of analysis may be extended to get an expression for the cross-sectional area of the vertical stiffeners. See Figure 32.

It is assumed the transfer of the flange force of member A occurs in the web of member B within the distance (t + 5K) as well as in the flange stiffeners. The compressive stress within this section would be—

$$\sigma_1 = \frac{\text{force}}{\text{area}} = \frac{F_f \sin\phi}{\left(\frac{t_f}{\sin\phi} + 5K\right)w + b_s\ t_s}$$

The vertical component of the web force of member A transfers directly into the web of member B within the distance $\frac{d}{\sin\phi}$.

The compressive stress within this section would be—

$$\sigma_2 \doteq \frac{\text{force}}{\text{area}} = \frac{F_w \sin\phi}{\frac{d}{\sin\phi} w}$$

Within the region (b–c), these compressive stresses in the member B overlap and would be added:

$$\boxed{\sigma = \frac{F_f \sin\phi}{\left(\frac{t_f}{\sin\phi} + 5K\right)w + b_s\ t_s} + \frac{F_w \sin^2\phi}{d\ w}} \quad (7)$$

Now if ultimate load conditions are assumed, that is all parts involved are stressed to yield:

where:

$F_f = b_b\ t_b\ \sigma_y$

$F_w = d\ w_b\ \sigma_y$

$$\sigma_y = \frac{b_b\ t_b\ \sigma_y \sin\phi}{\left(\frac{t_f}{\sin\phi} + 5K\right)w + b_s\ t_s} + \frac{d\ w_b\ \sigma_y\ \sin^2\phi}{d\ w}$$

and the required cross-sectional area of a pair of stiffeners becomes:

$$\boxed{b_s\ t_s \geqq \frac{w\ b_b\ t_b \sin\phi}{w - w_b \sin^2\phi} - \left(\frac{t_f}{\sin\phi} + 5K\right)w} \quad (8)$$

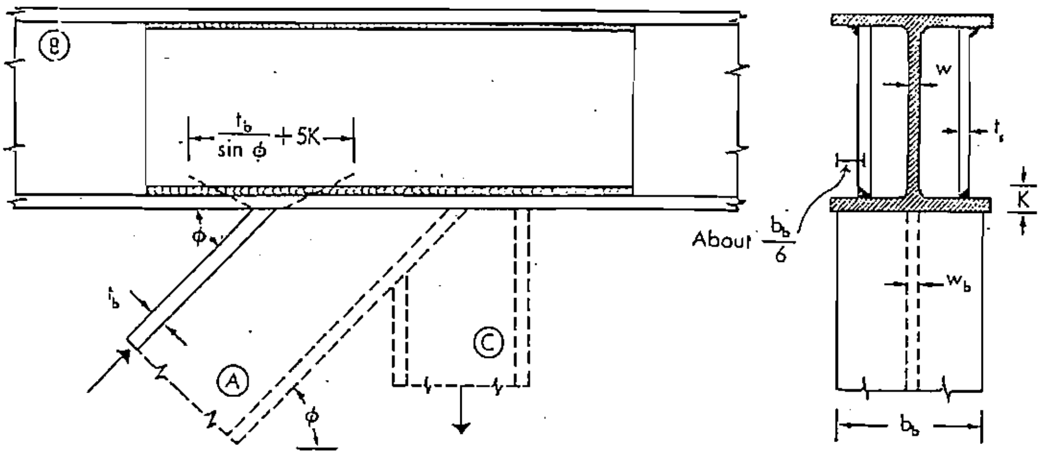


FIGURE 33

5. LONGITUDINAL STIFFENERS

The type of connection shown here may be reinforced with two stiffeners placed parallel to the web, and welded to the flanges of member (B). See Figure 33.

In the Lehigh test of this type of stiffening for beam-to-column connections, these plates were added along the outer edges of the flange so that beams framing in the other direction could be attached directly to them without extending within the column section. It was found that these plates each carried about $\frac{3}{16}$ of the applied compression, while the central web section loaded up and carried the remaining ⅝. For this reason the recommendation was made to assume these plates to be about half as effective.

It is interesting to remember that when a beam is supported at three points, the two ends and the center, the two outer supports each will carry only $\frac{3}{16}$ of the load and at center ⅝ of the load. If the outer supports are pushed in for $\frac{1}{6}$ of the beam length toward the center, all three reactions will be equal.

By setting the stiffening plates about $\frac{1}{6}$ b_b in from the edge of the flange of member (A), as shown above, it seems reasonable to assume they will carry a greater load and can be considered as effective as the web.

Although the K value applies only to the distribution in the web of member (B) and has nothing to do with these side plates, the Lehigh researchers for simplicity assumed the same distribution in the plates. The compressive stress in the web (B) and the two side stiffeners due to the vertical component of the flange force of member (A) is:

$$\sigma_1 = \frac{\text{force}}{\text{area}}$$

$$= \frac{F_f \sin\phi}{\left(\frac{t_b}{\sin\phi} + 5K\right) w + 2\left(\frac{t_b}{\sin\phi} + 5K\right) t_s}$$

$$= \frac{F_f \sin\phi}{\left(\frac{t_b}{\sin\phi} + 5K\right)\left(w + 2\, t_s\right)}$$

The compressive stress in the web of member (B) due to the vertical component of the web force of member (A) is:

$$\sigma_2 = \frac{\text{force}}{\text{area}} = \frac{F_w \sin\phi}{\frac{d}{\sin\phi}\, w}$$

$$= \frac{F_w \sin^2\phi}{d\, w}$$

These stresses are added together.

$$\boxed{\sigma = \frac{F_f \sin\phi}{\left(\frac{t_b}{\sin\phi} + 5K\right)\left(w + 2\, t_s\right)} + \frac{F_w \sin^2\phi}{d\, w}} \quad \ldots(9)$$

Now if ultimate load conditions are assumed, that is all parts involved are stressed to yield:

where:

$$F_f = b_b\, t_b\, \sigma_y$$
$$F_w = d\, w_b\, \sigma_y$$

$$\sigma_y = \frac{b_b\, t_b\, \sigma_y \sin\phi}{\left(\frac{t_b}{\sin\phi} + 5K\right)\left(w + 2\, t_s\right)} + \frac{d\, w_b\, \sigma_y \sin^2\phi}{d\, w}$$

and the required thickness of the two vertical plate stiffeners becomes:

$$\boxed{t_s \geq \frac{w\, b_b\, t_b \sin\phi}{2\left(\frac{t_b}{\sin\phi} + 5K\right)\left(w - w_b \sin^2\phi\right)} - \frac{w}{2}} \quad \ldots(10)$$

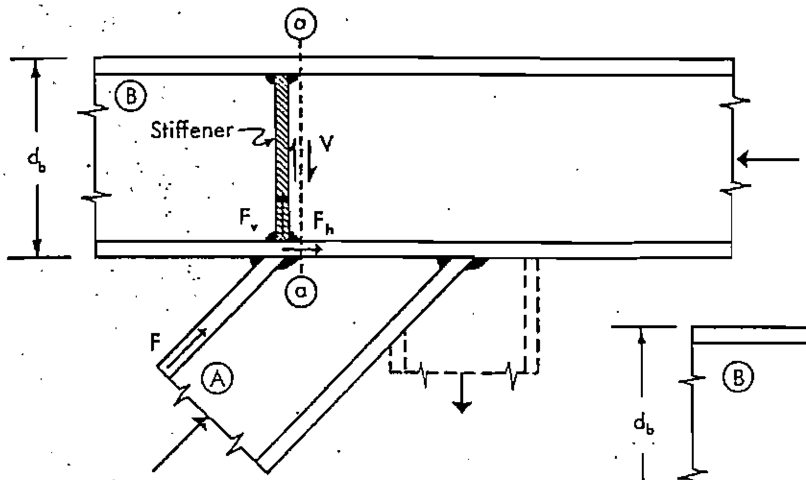


FIGURE 34

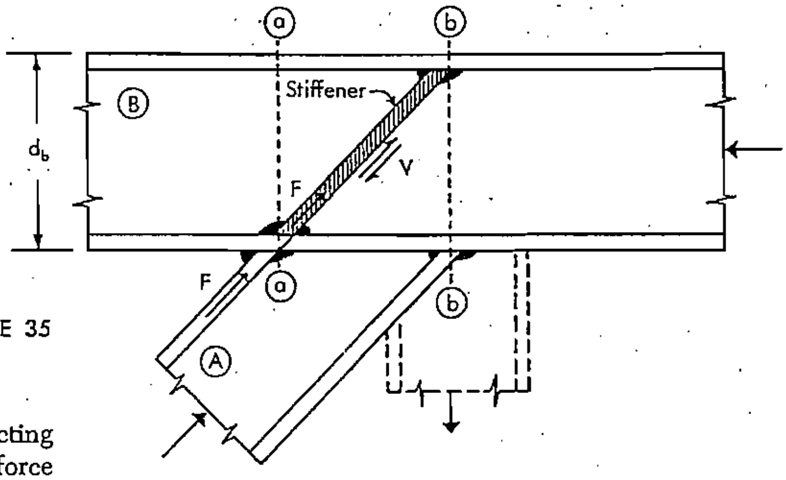


FIGURE 35

These plates must have sufficient welds connecting them to the lower flange because the compressive force of member (A) enters here. Since fillet welds cannot be placed on the inside, this would mean a rather large fillet weld on the outside. It may be more economical to bevel the plate and use a groove weld. In this example, the vertical compressive force is transferred from the plate down into the vertical member (C); thus a simple fillet weld along the top edge of the plate to the upper flange would be sufficient.

This discussion and resulting formulas will allow the connection to be detailed without computing the actual stresses. It is based on providing a connection as strong as the members.

Since member (A) will normally not be stressed to its full allowable compression, a more efficient connection would probably result if the actual stresses were computed, using these guides on distribution. Instead of providing full-strength welds, their size would then be determined from these computed forces.

These ideas will now be applied to various parts of a truss connection.

6. STIFFENING ACTUAL TRUSS CONNECTIONS

The vertical component (F_v) from the flange of (A) enters the stiffener and passes into the web of (B) as shear, $V = F_v$, along section a-a. The horizontal component (F_h) from the flange of (A) enters the lower flange of (B). The weld between stiffener and web of member (B) would be designed to transfer this shear force (V), Figure 34.

The force (F) from the flange of (A) transfers directly into the stiffener, leaving no horizontal component to enter the lower flange of (B). This force (F), now in the stiffener, gradually transfers into the web of (B) as shear, from section a-a to section b-b. This unit shear force is equivalent to $\nu = \frac{F_v}{d_b}$ The weld between stiffeners and web of member (B) would be designed to transfer this shear force (V), Figure 35.

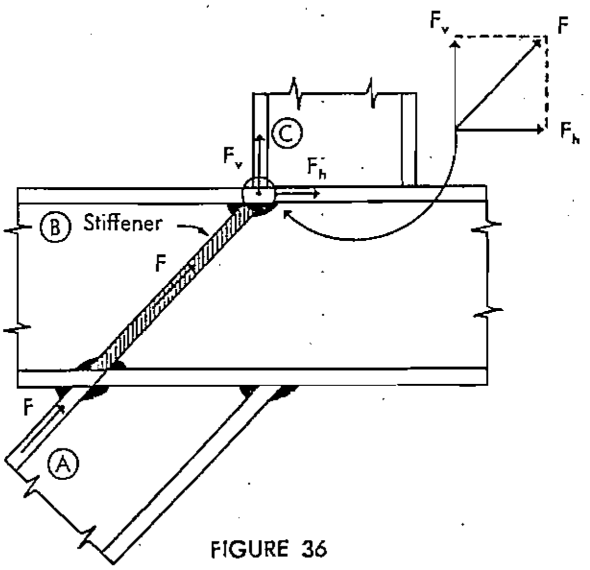


FIGURE 36

The force (F) from the flange of (A) enters the stiffener, and is transferred through to the opposite end. The vertical component (F_v) enters the flange of (C), and the horizontal component (F_h) enters the upper flange of (B). No shear force is transferred through the weld between stiffener and web of member (B). Only enough weld is required near midsection of stiffener to keep it from buckling, Figure 36.

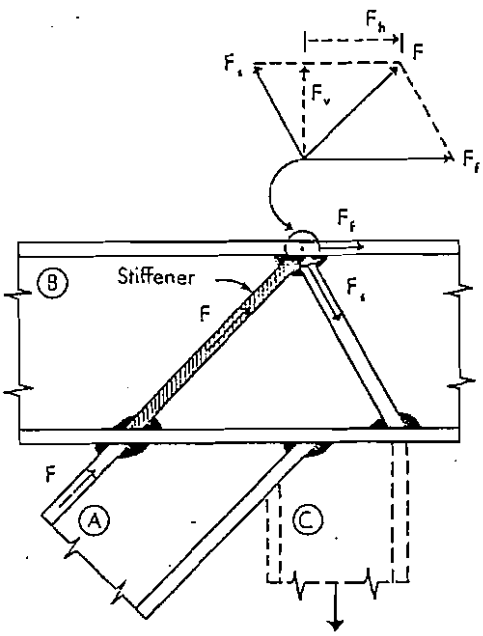


FIGURE 37

The force (F) from the flange of (A) enters the stiffener, and is transferred through to the opposite end. The vertical component (F_v) is taken by the second stiffener as (F_s), and the horizontal component (F_h) is taken by the upper flange of (B), Figure 37.

In these last two cases, it is assumed that no portion of the force (F) in the stiffener is transferred into the web of (B). The welding of the stiffener would be similar to the previous case, that is Figure 37.

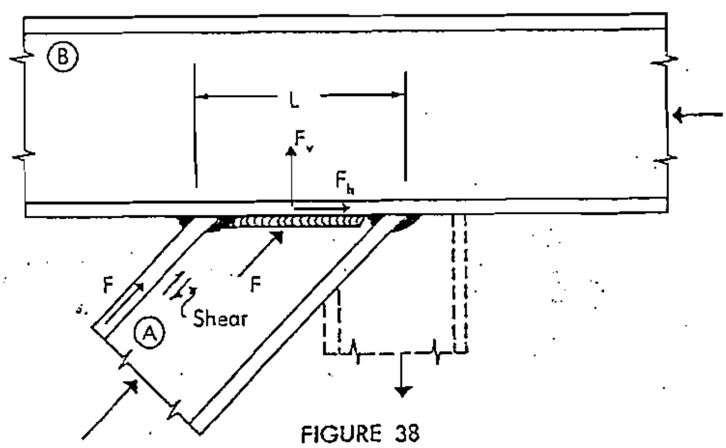


FIGURE 38

If there are no flange stiffeners on member B and no advantage of the preceeding distribution of the concentrated force into the web is to be taken, then the conservative method may be used. Thus, it is assumed that the flange force must first be transferred as shear into the web of the same member before it is transferred through the connecting weld into member (B). This weld may have to be made larger because of this additional force, Figure 38.

If this flange force (F) is high, a web doubler plate might have to be used so that these forces can be effectively distributed into the web of (A) without overstressing it.

Problem 2A

Consider the connection of Figure 39, using A373 steel and E60 welds.

In this case a portion of the vertical component of (A) is transferred directly into (C). It will be assumed that the vertical component of the left flange of (A) and the vertical force in the right flange of (C) will be transferred around through the web of (B) by means of two vertical stiffeners. See Figure 40.

(a) Check the size of the connecting welds on the flanges of (A).

unit force on flange fillet welds

$$f_f = \frac{F}{L}$$

$$= \frac{(138 \text{ kips})}{2(10)}$$

$$= 6.9 \text{ kips/linear inch}$$

leg size of flange fillet welds

$$\omega_f = \frac{6.9}{9.6}$$

$= .72''$ <u>or use ¾″</u> (or use a groove weld)

(b) Check the size of the connecting welds on the web of (A), which has a force of 74 kips.

unit force on web fillet welds

$$f_w = \frac{F}{L}$$

$$= \frac{(74 \text{ kips})}{2(17.5)}$$

$$= 2.11 \text{ kips/linear inch}$$

leg size of web fillet welds

$$\omega_w = \frac{2.11}{9.6}$$

$$= .22''$$

FIGURE 39

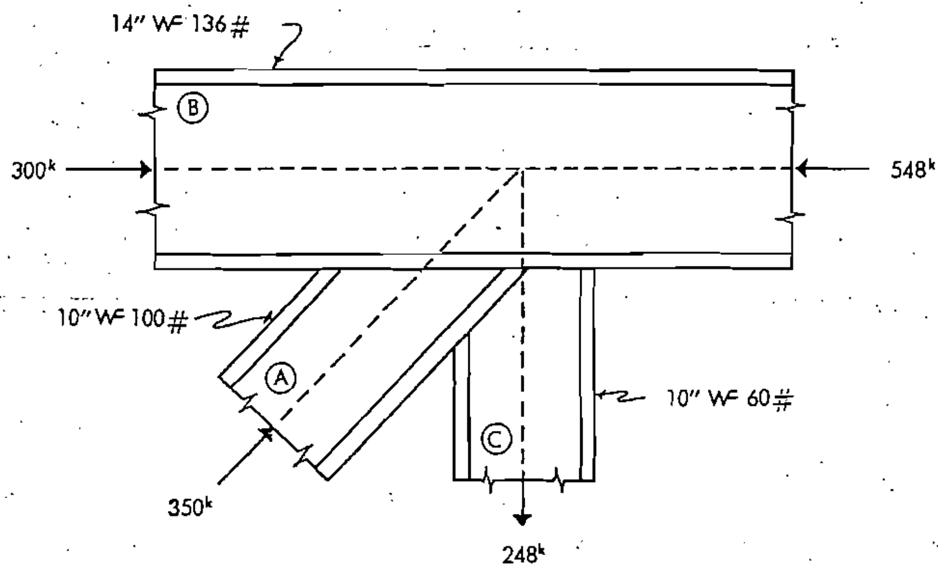
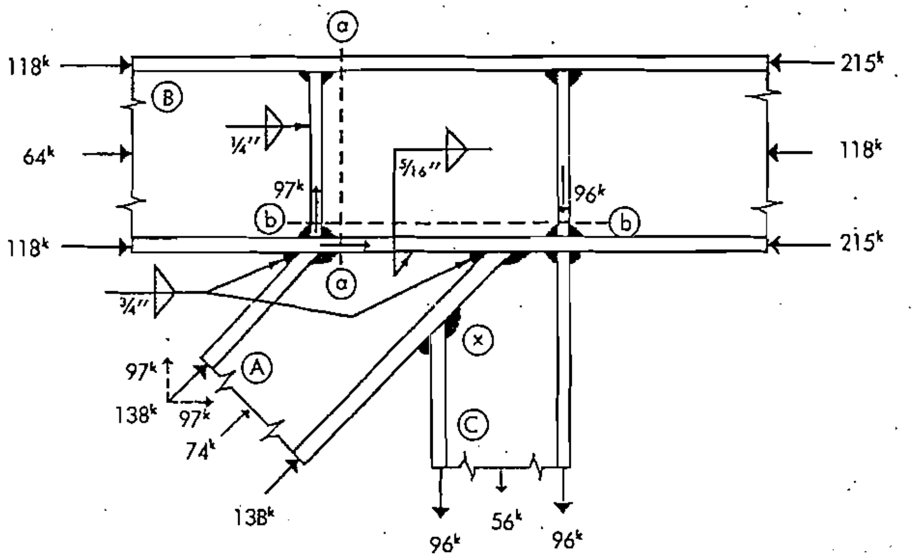


FIGURE 40



However, the minimum fillet weld to be attached to the 1.063″-thick flange would be $\omega_w = 5/16''$. (AISC Sec 1.17.4)

(c) Determine required sectional area of vertical stiffeners.

$$A_s = \frac{F_r}{.90\ \sigma_y}$$

$$= \frac{(97 \text{ kips})}{(29.7 \text{ ksi})} \qquad \text{(AISC Sec 1.5.1.5.2)}$$

$= 3.27$ in.², or use two ⅜″ x 5″ stiffeners

Their $A_s = 3.75$ in.² > 3.27 in.² OK

(d) Check the size of connecting welds to transfer this force (F_r) as shear into the web of B.

unit force on stiffener-to-web fillet welds

$$f = \frac{97 \text{ kips}}{4(12.6)}$$

$= 1.92$ kips/linear inch

leg size of fillet welds

$$\omega = \frac{1.92}{9.6}$$

$= .20''$ or use ¼″

(e) Check the vertical shear stress along a-a.

$$\tau = \frac{V}{A_w} \qquad \text{See Figure 41.}$$

$$= \frac{(97 \text{ kips})}{(.660)(12.62)}$$

$= 11{,}650$ psi $< 13{,}000$ psi $< .40\ \sigma_y$ OK (AISC Sec 1.5.1.2)

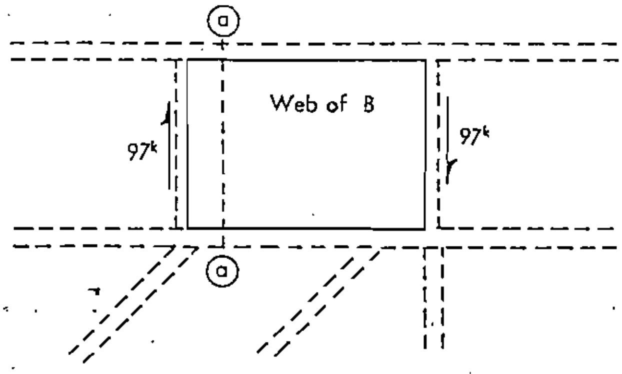


FIGURE 41

(f) Check the horizontal shear stress along b-b in the web of (B) parallel to the welded connection between (A) and (B). This length is about 20″.

The total horizontal component from (A) to be transferred into (B) is 248 kips. The lower flange of (B) has a compressive force of 215 kips on the right end and 118 kips on the left end. This means it will pick up 215 — 118 = 97 kips from (A).

Hence, a force of 248 — 97 = 151 kips is to be transferred into the web of (B) over a distance of 20″.

$$\tau = \frac{V}{A_w}$$

$$= \frac{(151 \text{ kips})}{(.660)(20)}$$

$$= 11{,}430 \text{ psi} < 13{,}000 \text{ psi} < .40\,\sigma_y \quad \underline{\text{OK}}$$

(AISC Sec 1.5.1.2)

As a result no stiffening of the web of (B) is required as far as shear is concerned. If these shear stresses exceed the allowable, the web of the connection could be reinforced with a doubler plate, either on the web itself, or separated slightly and welded to the edges of the upper and lower flanges of (B).

(g) There is one more item to check; consider point (x) in the figure below. It is necessary that the vertical component of the right flange of (A) be transferred into the left flange of (C), and yet its horizontal component be transferred into the lower flange of (B).

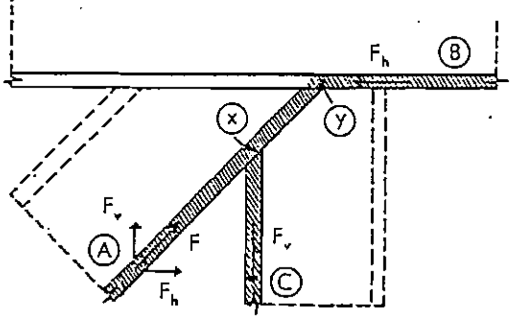


FIGURE 43

Theoretically, the flange of (A) can only transmit an axial force (F) between points (x) and (y). There would be no problem if these 3 flanges met at a common point.

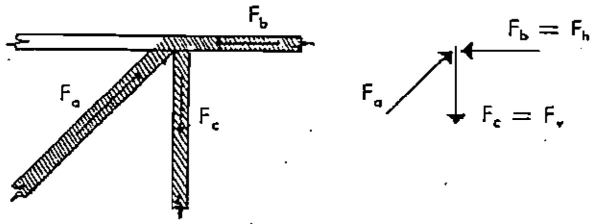


FIGURE 44

In order for the flange of (C) to take the vertical component (F_v) from the flange of (A) at (x), it is necessary that the horizontal component (F_h) also

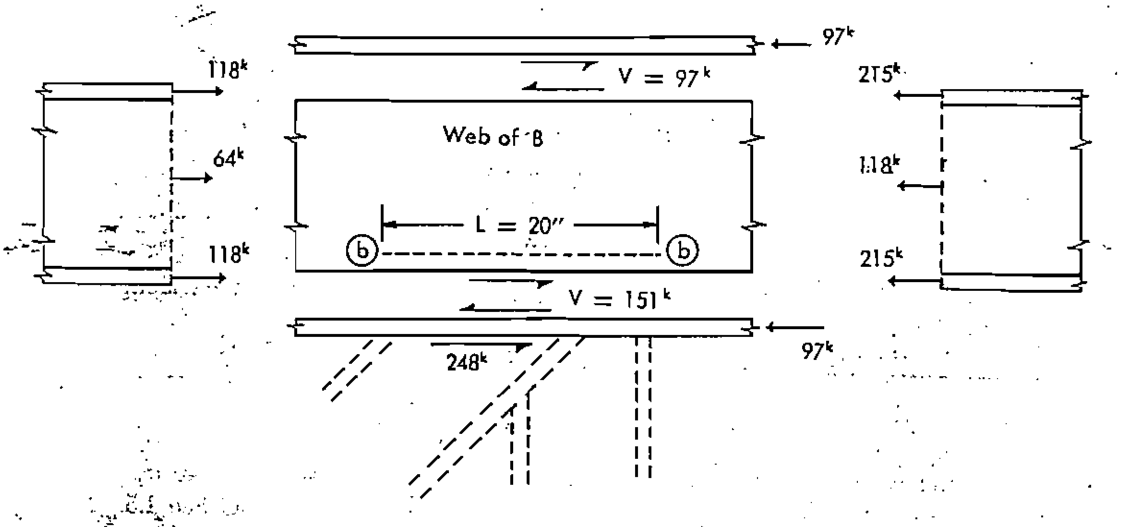


FIGURE 42

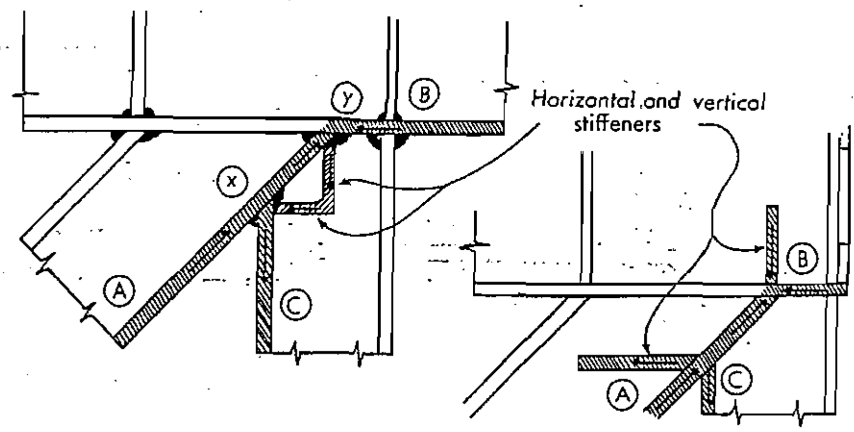


FIGURE 45

be taken at this point and somehow carried up into the lower flange of (B).

Likewise, in order for the flange of (B) to take the horizontal component (F_h) at (y), it is necessary that the vertical component (F_v) also be taken at this point and carried into the flange of (C). There are several methods by which this may be done.

(1) This could be accomplished with a vertical stiffener at (y) and a horizontal stiffener at (x). These would transfer the components into the web of (C) from where they could work their way back into the flange of (B) and the flange of (C). Two methods of using this are shown in Figure 45.

(2) This also could be accomplished with two sets of vertical stiffeners; see Figure 46. The left stiffener would transfer the vertical force of the flange of (C) up into the web of (B), where it would work its way over to the right stiffener through shear (V). The right stiffener would transfer the vertical component (F_v) of the flange of (A) into the web of (B) so that the horizontal component (F_h) could be transferred into the flange of (B).

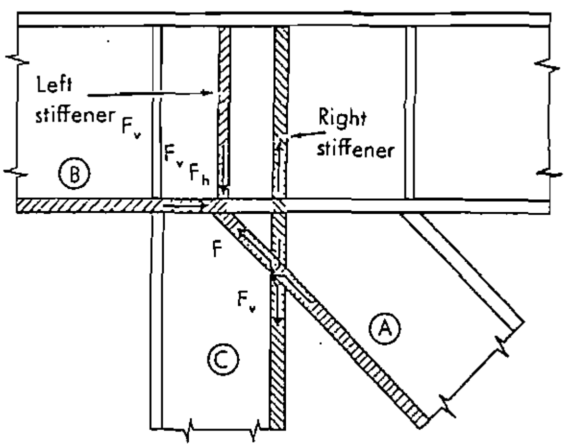
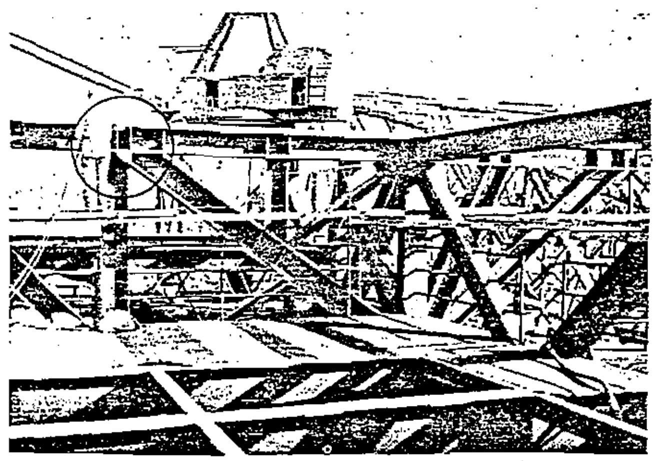
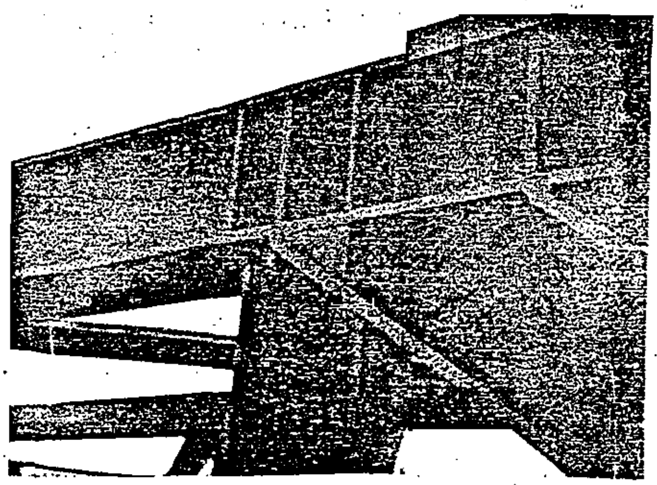


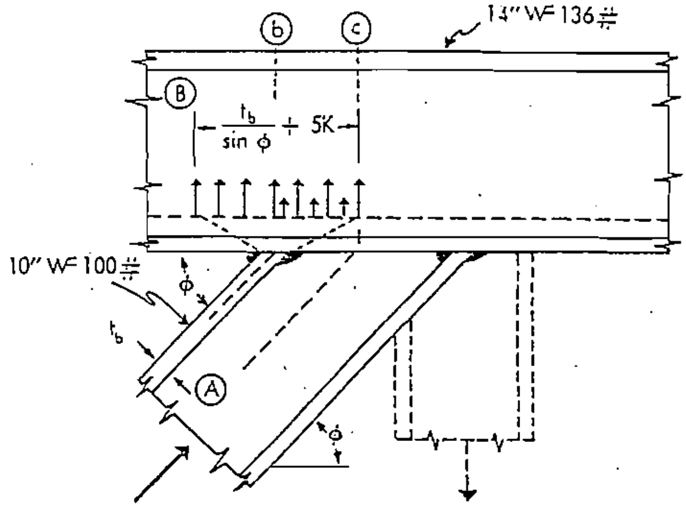
FIGURE 46

If the shear transfer (V) between these two stiffeners exceeds the allowable of the web of (B), a doubler plate may be added to the web; or a plate may be set out on each side to box in this area.





In this substructure for an offshore drilling rig, the truss connections carry large concentrated transverse forces. Vertical flange stiffeners are required to prevent web buckling. The triangular "gusset" is welded in to enclose the area for greater protection against corrosion in addition to stiffening.



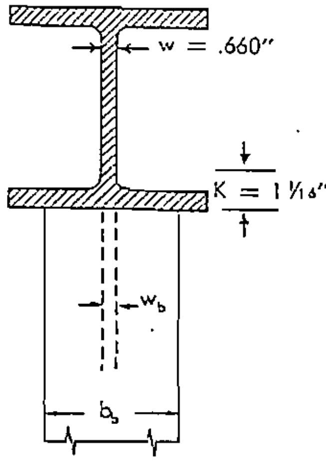


FIGURE 47

Problem 2B

Another solution of the same problem would be to check the stiffener requirements using the Lehigh research for beam-to-column connections as a guide for the distribution of the forces through the connection.

(a) See if the web thickness (w) of (B) is sufficient for stiffeners not to be required; Figure 47.

$$w \geqq \sin^2 \phi \left[\frac{b_b\, t_b}{t_b + 5K \sin \phi} + w_b \right]$$

$$\geqq (.707)^2 \left[\frac{(10.345)(1.118)}{1.118 + 5(1^{11}\!/_{16})(.707)} + .685 \right]$$

$w \geqq 1.16''$ required $> .660''$ actual

On this basis some stiffeners would be required.

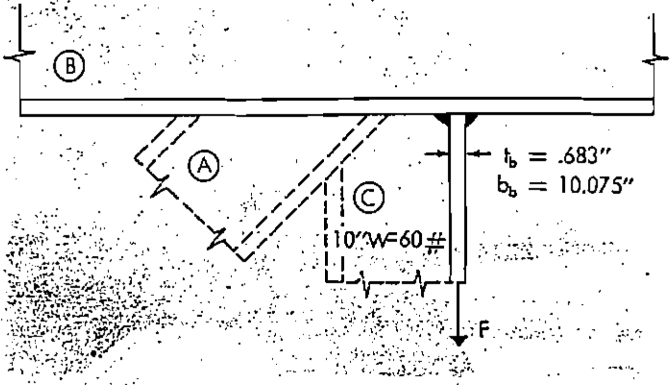


FIGURE 48

(b) Check the tension flange of (C) where it joins the flange of (B), as to the necessity of stiffeners to transfer the flange force; Figure 48.

$$t_c = .40 \sqrt{b_b\, t_b}$$

$$= .40 \sqrt{(10.075)(.683)}$$

$$= 1.05'' < 1.063'' \qquad \underline{\text{OK}}$$

On this basis, stiffeners would not be needed opposite this flange of (C) where it joins the Bottom flange of (B)

(c) Check the tension flange of (C) where it joins the flange of member (A); Figure 49.

$$t_c = \sqrt{\frac{b_b\, t_b\, (\sin \alpha - .12)}{5.6}}$$

$$= \sqrt{\frac{(.683)(10.075)(.707 - .12)}{5.6}}$$

$$= .85'' < 1.118'' \qquad \underline{\text{OK}}$$

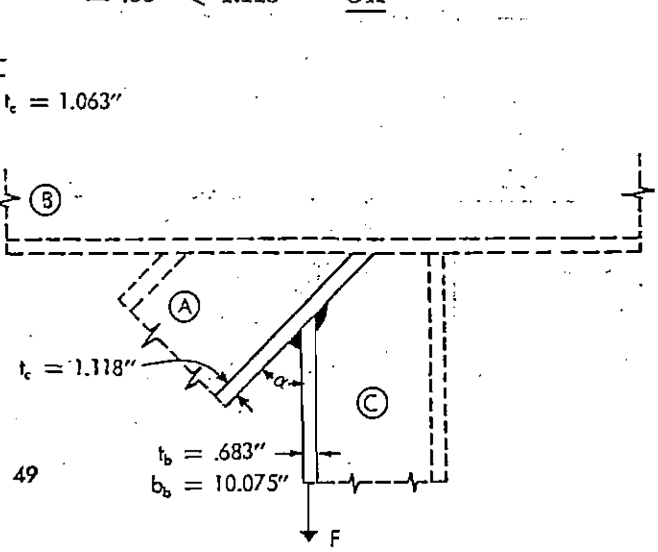


FIGURE 49

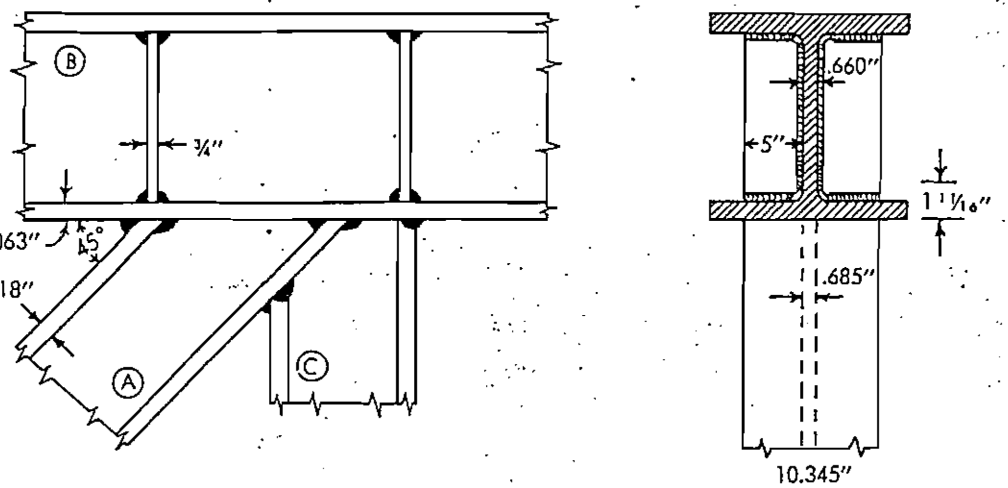


FIGURE 50

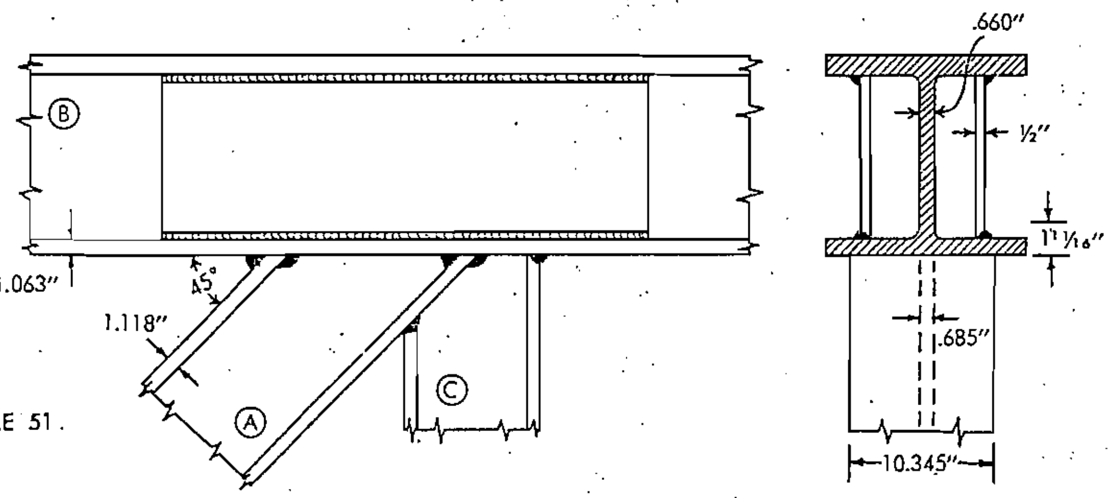


FIGURE 51.

On this basis, stiffeners would not be required on (A) opposite this flange of member (C).

Either vertical flange stiffeners or longitudinal flange stiffeners can be used to provide added stiffness for the compressive force of (A).

vertical flange stiffeners

$$b_b t_s \geqq \left(\frac{w\ b_b t_b \sin\phi}{w - w_b \sin^2\phi}\right) - \left(\frac{t_b}{\sin\phi} + 5K\right)$$

$$\geqq \frac{(.660)(10.345)(1.118)(.707)}{(.660)-(.685)(.707)^2} - \left(\frac{1.118}{.707} + 5 \times 1\frac{11}{16}\right)$$

$$\geqq 7.03 \text{ in.}^2$$

so use two pairs of ¾" x 5" stiffeners.

longitudinal flange stiffeners

$$t_s \geqq \frac{w\ b_b t_b \sin\phi}{2\left(\dfrac{t_b}{\sin\phi} + 5K\right)(w - w_b \sin^2\phi)} - \frac{w}{2}$$

$$\geqq \frac{(.660)(10.345)(1.118)(.707)}{2\left(\dfrac{1.118}{.707} + 5 \times 1\dfrac{11}{16}\right)\left[.660 - .685(.707)^2\right]} - \frac{.660}{2}$$

$$\geqq .53''$$

or use a pair of ½" x 12⅝" x 36" stiffeners.

7. TYPICAL TRUSS PROBLEMS

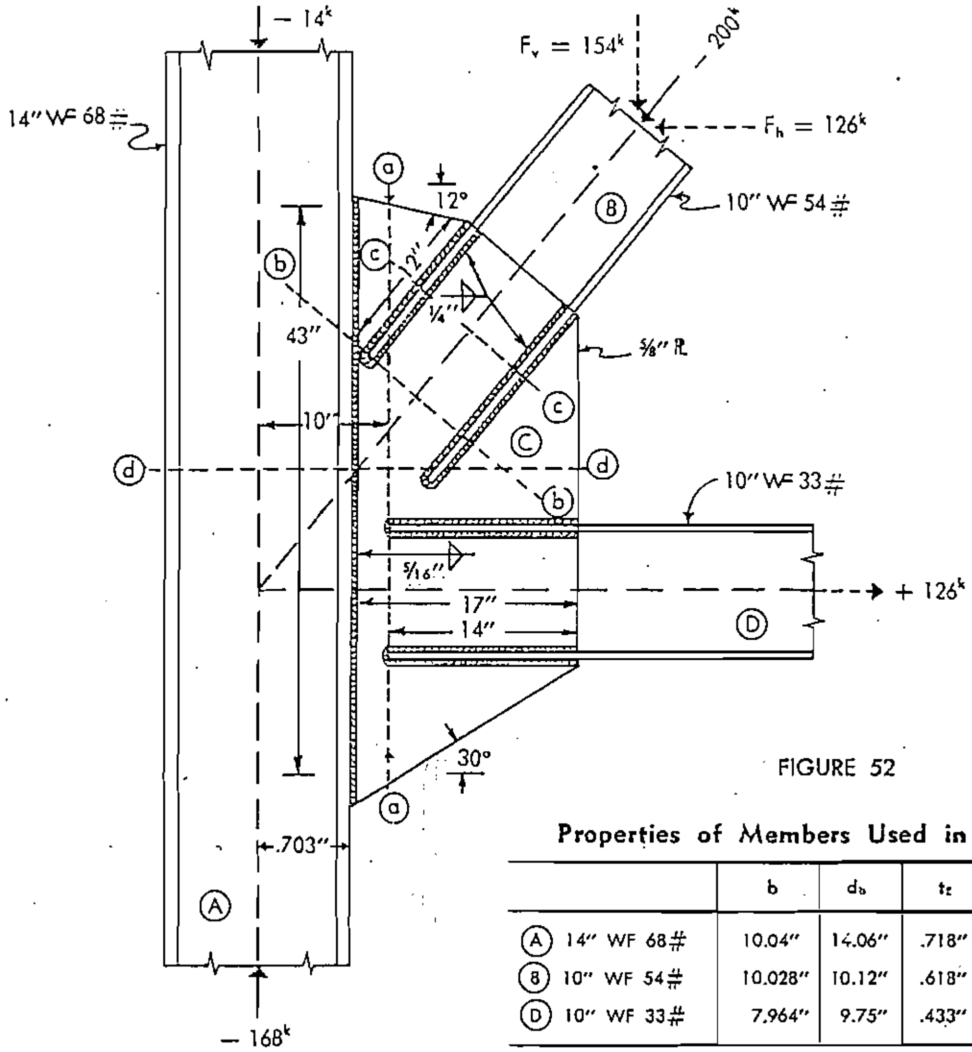


FIGURE 52

Properties of Members Used in Problem 3

	b	d_b	t_f	t_w	A_T
(A) 14" WF 68#	10.04"	14.06"	.718"	.418"	20.00 in.^{2}
(B) 10" WF 54#	10.028"	10.12"	.618"	.368"	15.88 in.^{2}
(D) 10" WF 33#	7.964"	9.75"	.433"	.292"	9.71 in.^{2}

Problem 3

Check the details of this connection, using A373 steel and E60 welds.

(a) Consider the moment and vertical shear on section a-a.

$$M = F\ d = (168^k - 14^k)(10'') = 1540 \text{ in-kips}$$

$$V = 154 \text{ kips}$$

section modulus of section a-a

$$S = \frac{(5/8)(43)^2}{6} = 192.5 \text{ in.}^3$$

bending

$$\sigma = \frac{M}{S} = \frac{(1540)}{(192.5)} = 8{,}000 \text{ psi}$$

shear

$$\tau = \frac{V}{A} = \frac{(154^k)}{(5/8)(43)} = 5{,}730 \text{ psi}$$

resulting maximum normal stress (See Figure 53.)

$$\sigma_{max} = \frac{\sigma}{2} \pm \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2}$$

$$= \frac{(8000)}{2} \pm \sqrt{\left(\frac{8000}{2}\right)^2 + 5730^2}$$

$$= 10{,}980 \text{ psi}$$

The resulting bending stress of $\sigma = 8{,}000$ psi at the outer fiber is for a horizontal edge. If this edge slopes (ϕ), the resulting fiber stress along this edge may be found from the following:

so: $\boxed{\sigma_\phi = \dfrac{\sigma_h}{\cos^2\phi}}$ (See Figure 54.)

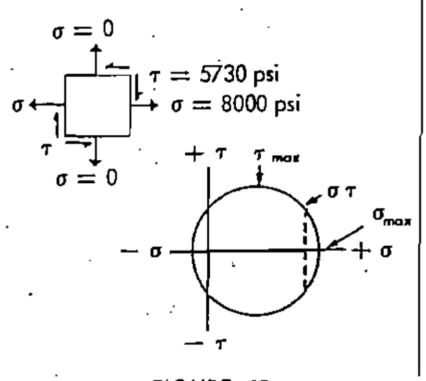


FIGURE 53

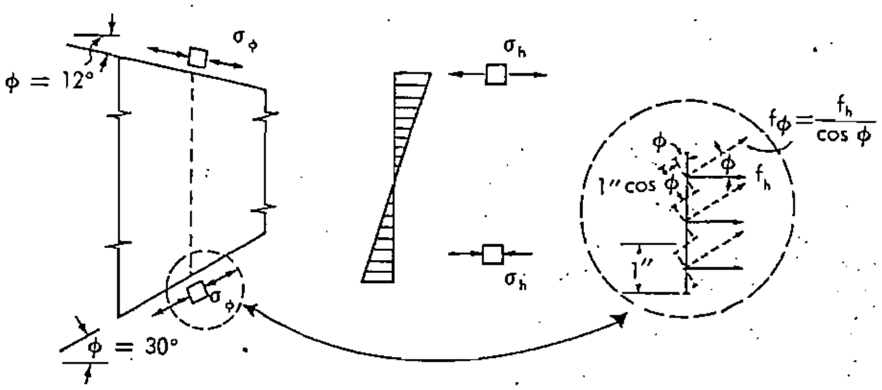


FIGURE 54

at top edge of gusset plate

$$\phi = 12^\circ \qquad \cos 12^\circ = .977$$

$$\sigma = \frac{8{,}000}{.977^2} = 8{,}390 \text{ psi (compression)}$$

at bottom edge of gusset plate

$$\phi = 30^\circ \qquad \cos 30^\circ = .865$$

$$\sigma = \frac{8{,}000}{.865^2} = 10{,}700 \text{ psi (tension)}$$

(b) Consider the transfer of the vertical component (F_v) of the truss members (B) and (D) through gusset plate (C) and into the web of column (A) within the connection length of 43″ as shear. From this vertical component (F_v), deduct the portion to be carried by the right flange of (A). (This does not have to enter the web of column (A).) This portion carried by the right flange can be determined by the ratio of the flange area to the total section area.

The force taken by this flange is—

$$F = 154\,\frac{(10.04)(.718)}{(20.00)}$$

$$= 55.5 \text{ kips}$$

This leaves 154 — 55.5 = 98.5 kips to pass into the web (some of which will enter into the left flange). The resulting shear stress within this 43″ length of web is:

$$\tau = \frac{(98.5)}{(43)(.418)}$$

$$= 5{,}490 \text{ psi} < 15{,}000 \text{ psi} < .40\,\sigma_y \quad \text{OK}$$

(AISC Sec 1.5.1.2)

This transfer can be made while still keeping the flange compressive stress within the uniform stress of—

$$\sigma = \frac{(168 \text{ kips})}{(20.00 \text{ in.}^2)} = 8400 \text{ psi}$$

(c) Consider the vertical weld between connection plate (C) and member (A). The forces applied on the left side of this weld are—

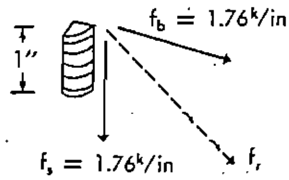


FIGURE 55

$$M = (168^k - 14^k)(7.03'') = 1082\text{-in.-kips}$$

$$V = 154 \text{ kips}$$

section modulus of weld connection

$$S_w = \frac{2\ d^2}{6} = \frac{43^2}{3} = 616.3 \text{ in.}^2$$

bending force on weld

$$f_b = \frac{M}{S_w} = \frac{(1082)}{(616.3)} = 1.76 \text{ kips/in.}$$

shear force on weld

$$f_s = \frac{V}{A_w} = \frac{(154)}{(2)(43)} = 1.79 \text{ kips/in.}$$

resultant force on weld

$$f_r = \sqrt{f_b^2 + f_s^2} = \sqrt{(1.76)^2 + (1.79)^2}$$

$$= 2.51 \text{ kips/in.}$$

leg size of fillet weld

$$\omega = \frac{(2.51)}{(9.6)} = .261'' \text{ or use } 5/16''$$

(d) Flange plates, ⅝″ by 4¾″, are welded onto (C) to extend the flange of (B) back a sufficient distance. The compressive force in the flange of (B) is—

$$F = 200^k\,\frac{(10.028)(.618)}{(15.88)} = 78.0 \text{ kips}$$

O. 'is basis, the stress in each of these flange plates i .

$$\sigma = \frac{(78 \text{ kips})}{(2'')(5/8'')(4\tfrac{3}{4}'')} = 13{,}100 \text{ psi} \quad \underline{OK}$$

The force from an adjacent pair of these plates is transferred into (C) as double shear.

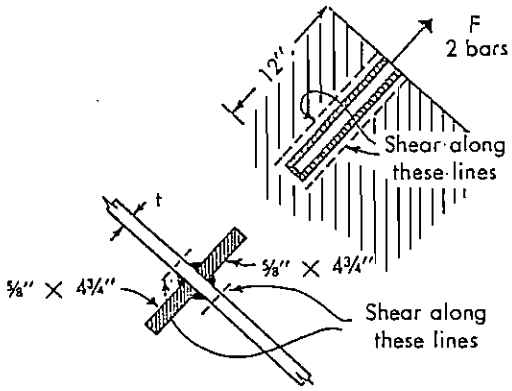


FIGURE 56

This shear stress in (C) is—

$$\tau = \frac{F}{4\ L\ t}$$

$$= \frac{(78.0 \text{ kips})}{4(12'')(5/8'')}$$

$$= 2600 \text{ psi} < 13{,}000 \text{ psi} \quad \underline{OK}$$

size of connecting welds

$$f = \frac{(37.4 \text{ kips})}{(2)(12'')} = 1.56 \text{ kips/in.}$$

$$\omega = \frac{(1.56 \text{ k/in.})}{(9.6 \text{ k/in.})} = .163'' \text{ or } \underline{\text{use } 3/16''}$$

However, the AWS as well as the AISC would require a ¼" fillet weld because of the ⅝" plate.

(e) At section b-b at the termination of the flange plates, we will assume the 200-kip compressive force must be taken by (C) alone. The cross-sectional area of (B) is A = 15.88 in.².

For the same stress in (C), this would require the same cross-sectional area, or 15.88 in.², and a net width of

$$W = \frac{15.88}{5/8} = 25.4''$$

There is sufficient width; see Figure 52.

(f) At section c-c halfway along the flange plates, it is assumed that half of the flange force of (B) has been transferred out into (C):

$$\tfrac{1}{2}\ (200^k)\ \frac{(10.028)(.618)}{(15.88)} = 39.0 \text{ kips}$$

For the two flange plates, this reduction would leave—

$(200^k) - 2\ (39.1^k) = 122.0$ kips to be taken by (C).

For the same stress, this would require an area of—

$$A = \frac{(122.0^k)}{(12.52 \text{ ksi})} = 9.74 \text{ in.}^2$$

and a net width of—

$$W = \frac{(9.74)}{(5/8)} = 15.6''$$

There is sufficient width; see Figure 52.

(g) Another section which might be checked is along d-d. The loads on this section are the direct compressive load of the column (A), a shearing force from the tension in the lower chord member (D), and a bending moment from the eccentricity of both the column (A) and the bottom chord (D). This critical section (d-d) is placed as high as possible above the lower chord (D) without intercepting the stiffening elements of the connection. In this case it is placed 9" above the centerline of member (D).

The properties of this built-up cross-section are

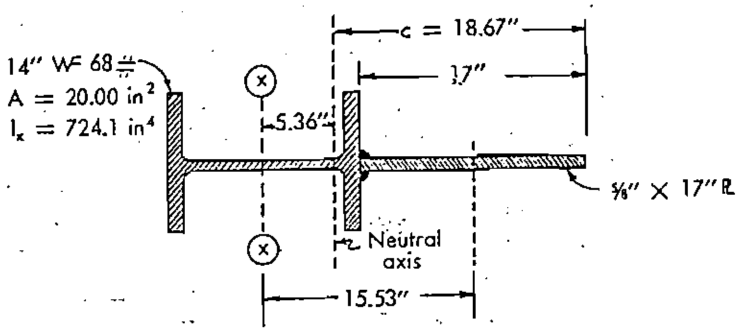


FIGURE 57

computed and the eccentricities determined. For simplicity in this computation, the reference axis (x-x) is placed along the centerline of the column (A).

Member	A	d	M = A d	I = M d	$I_{x..}$
14″ WF 68#	20.00	0	0	0	724.1
⅝ x 17″ ℞	10.52	+15.53	+163.5	+2675.	255.9
Total →	30.52		163.5	3655.0	

$$I_{NA} = I_x - \frac{M^2}{A} = (3655) - \frac{(163.5)^2}{(30.52)} = 2380 \text{ in.}^4$$

$$NA = \frac{M}{A} = \frac{(163.5)}{(30.52)} = 5.36''$$

From this:

$$c = (14.06 + 17) - (7.03 + 5.36) = 18.67''$$

$$S = \frac{I}{c} = \frac{(2380)}{(18.67)} = 127.5 \text{ in.}^3$$

Applied Loads

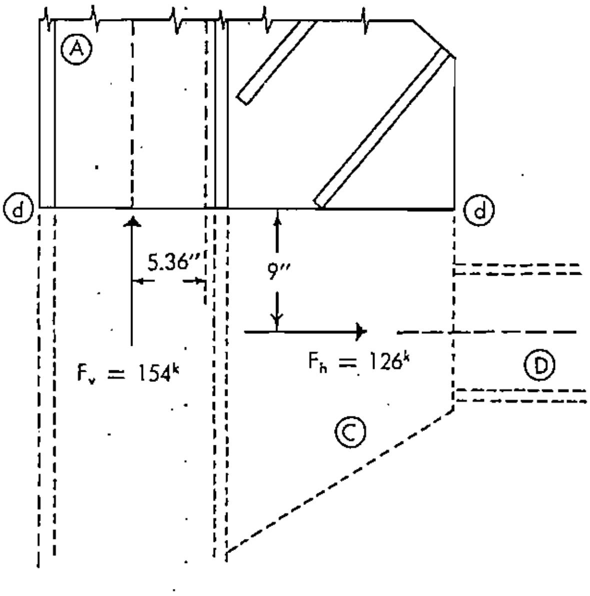


FIGURE 58

compression

$$F_v = 168^k - 14^k = 154 \text{ kips}$$

$$\sigma = \frac{F_v}{A} = 5050 \text{ psi (compression)}$$

shear

$$F_h = 126^k$$

$$= \frac{(126^k)}{(14.06)(.418) + (17)(⅝)} = 7640 \text{ psi}$$

bending

$$M = (126^k)(9'') - (154^k)(5.36) = 233.0 \text{ in.-kips}$$

$$\sigma = \frac{(233.0)}{(127.5)} = 2420 \text{ psi (compression)}$$

This is a total compressive stress of 5050 + 2420 = 7470 psi, and a shear stress of 7640 psi at the outer edge of the connection plate (C).

The resultant maximum normal (compressive) stress at the edge of the plate is—

$$\sigma_{max} = \frac{\sigma}{2} \pm \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2}$$

$$= \frac{(7470)}{2} + \sqrt{\frac{7470^2}{2} + 7640^2}$$

$$= 12{,}800 \text{ psi}$$

Check the outer edge of this plate (C) as a column.

radius of gyration

$$r = .289\ t = (.289)(⅝) = .181''$$

The unbraced length of this edge is L = 15″, and

$$\frac{L}{r} = \frac{15}{.181} = 83$$

and the corresponding allowable compressive stress is—

$$\sigma = 14{,}130 \text{ psi} > 12{,}285 \text{ psi} \quad \text{OK}$$

(AISC Sec 1.5.1.3.1)

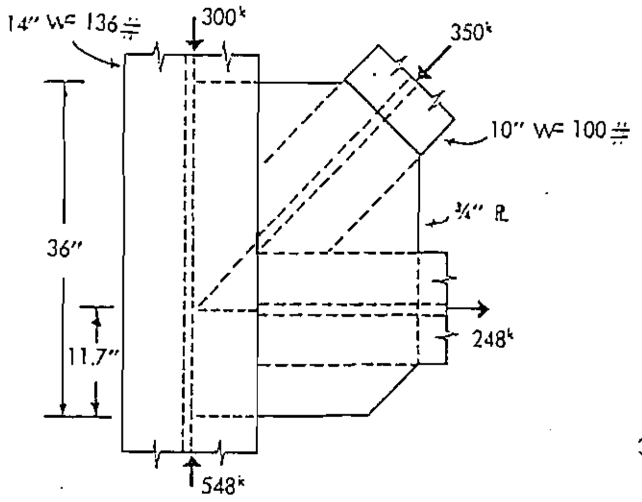
If the calculated compressive stress had exceeded this allowable, a flange could have been added along this one outer edge to give it sufficient stiffness against buckling.

Plate (C) will have 7⁄16″ by 4″ flange plates to extend the flanges of member (D) along a distance of 12″. ¼″ fillet welds will be sufficient to attach these plates, this size being required because of the ⅝″ plate. No further checking is necessary because, by observation, the 126-kip force is much less than the 200-kip force of member (B) and the same amount of plate (C) is available.

Problem 4

Determine the leg size of the four fillet welds connecting the two ¾″ gusset plates to the vertical leg of a tower. A373 steel, E60 welds. See Figure 59.

The horizontal component of the 350-kip force of the diagonal member (10″ WF 100#) is transferred back to the horizontal member (248 kips) through the



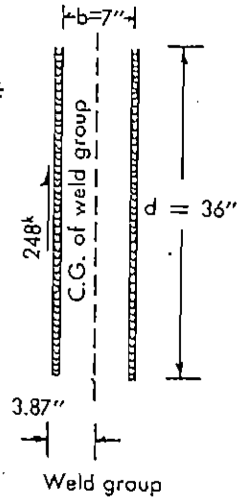
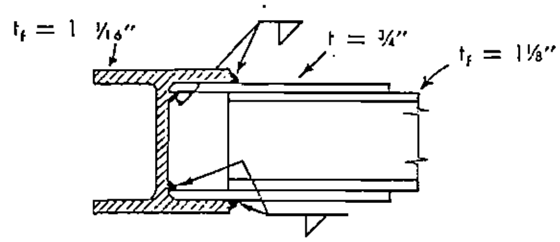


FIGURE 59



gusset plate. The only force transferred through this connecting weld to the vertical member (14″ WF 136#) connecting weld to the vertical member (14″ WF 136#) is the 248-kip vertical force acting 3½″ away from the center of gravity of the welded connection.

Treat the weld group as a line:

$$J_w = 2\,\frac{d(3b^2 + d^2)}{6}$$

$$= 2\,\frac{36}{6}\,(3 \times 7^2 + 36^2)$$

$$= 18.516 \text{ in.}^3$$

twisting action

vertical force:

$$f_v = \frac{T\,c}{J_w}$$

$$= \frac{(248 \times 3.87)(3\tfrac{1}{2})}{(18{,}516)}$$

$$= 181 \text{ lbs/in.}$$

horizontal force

$$f_h = \frac{T\,c}{J_w}$$

$$= \frac{(248 \times 3.87)(18)}{(18{,}516)}$$

$$= 933 \text{ lbs/in.}$$

vertical shear

$$f_{v1} = \frac{F}{L_w}$$

$$= \frac{(248)}{(4 \times 36)}$$

$$= 1720 \text{ lbs/in.}$$

resultant

$$f_r = \sqrt{f_h^2 + (f_v + f_{v1})^2}$$

$$= \sqrt{(933)^2 + (181 + 1720)^2}$$

$$= 2120 \text{ lbs/in.}$$

leg size of weld

$$\omega = \frac{\text{actual force}}{\text{allowable force}}$$

$$= \frac{(2120)}{(9600)}$$

$$= .220'' \text{ or } \underline{\tfrac{1}{4}''\,\triangle}$$

However, AWS and AISC would require $\underline{\tfrac{5}{16}''\,\triangle}$ because of 1 1/16″ flange.

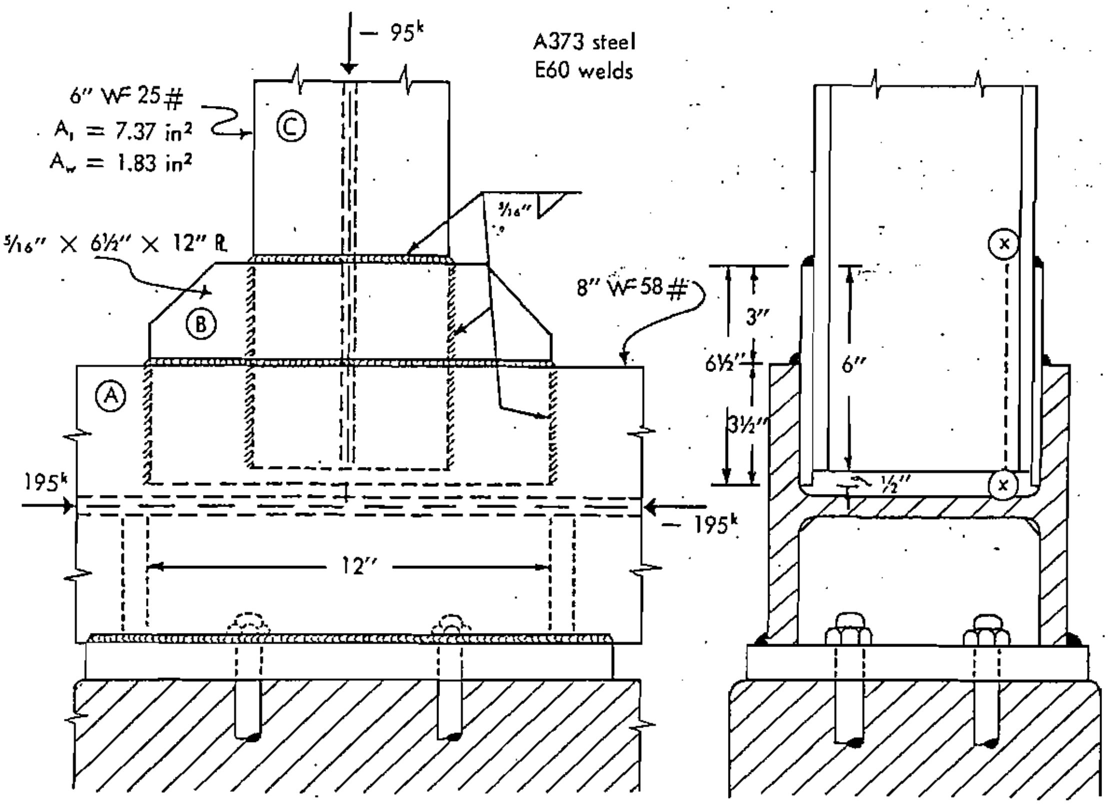


FIGURE 60

Problem 5

Determine the weld sizes on this connection. A373 steel, E60 welds.

(a) Find the required size of fillet weld between member (C) and connecting plates (B) The total length of connecting weld is—

$$L = 4(6'') + 2(6.08'') = 36.0''$$

force of weld

$$f = \frac{F}{L} = \frac{(95 \text{ kips})}{(36'')} = 2.64 \text{ kips/in.}$$

weld leg size

$$\omega = \frac{(2.64 \text{ k/in.})}{(9.6 \text{ k/in.})} = .275'' \text{ or } \underline{\text{use } 5/16''}$$

Check the length of web (C) within the connection along section x-x, required to transfer the force of the web (C) out into the flanges of (C) as shear.

force in web

$$F_w = 95^k \frac{(1.83)}{(7.37)} = 23.6 \text{ kips}$$

shear stress

$$\tau = \frac{F}{A_w} = \frac{½(23.6 \text{ kips})}{(.320)(L)} = 13{,}000 \text{ psi}$$

minimum weld length

$$L = \frac{½(23.6 \text{ kips})}{(.320)(13 \text{ k/in.}^2)} = 2.84'' < 6'', \text{ so is } \underline{\text{OK}}$$

(b) Find the required size of fillet weld between flanges of (A) and plates (B). The total length of connecting weld is—

$$L = 4(3½'') + 2(12'') = 38.0''$$

force on weld

$$f = \frac{(95 \text{ kips})}{(38'')} = 2.5 \text{ kips/in.}$$

weld leg size

$$\omega = \frac{2.5}{9.6} = .260'' \text{ or } \underline{\text{use } 5/16''}$$

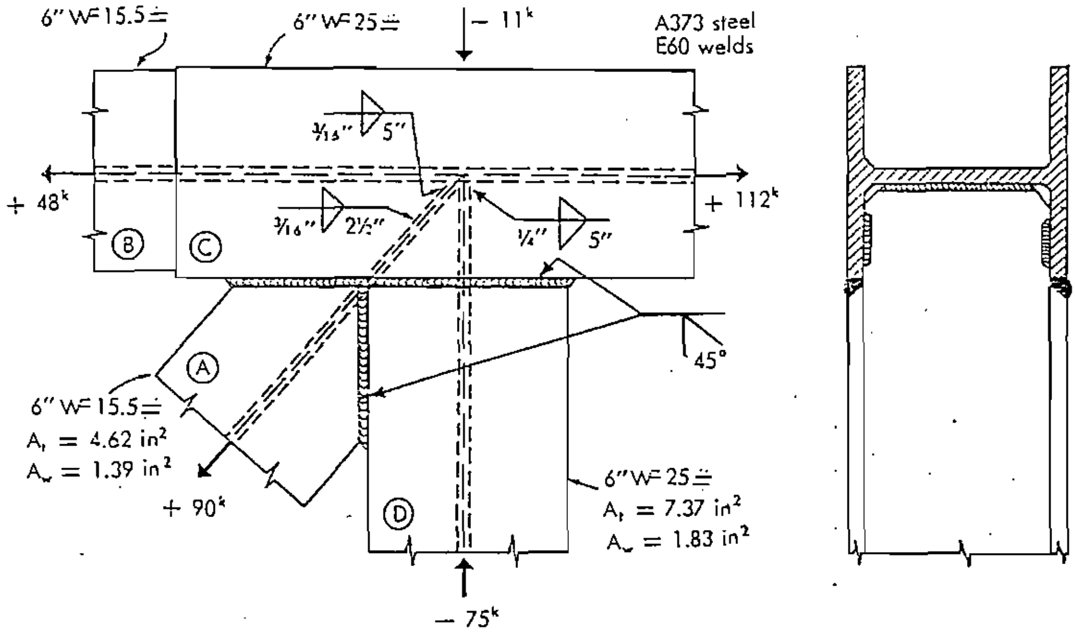


FIGURE 61

Problem 6

Check the weld sizes on this truss connection. A373 steel, E60 welds.

No calculations are required for the flange groove welds, since they will develop the full strength of the flanges.

The force in the web of diagonal member (A) is—

$$F_w = 90^k \frac{(1.39)}{(4.62)} = 27.1 \text{ kips}$$

The effective fillet weld size for this web (A) is—

$$\omega = \tfrac{3}{4}\, t_w = \tfrac{3}{4}(.240) = .180'' \text{ or } \tfrac{3}{16}''$$

The total length of this weld is—

$$L = \frac{(27.1 \text{ kips})}{(1.8 \text{ k/in.})} = 15.0''$$

If there is 5" of 3/16" fillet weld on each side of web (A) to web (C), this leaves 15" − 2(5") = 5" or 2½" on web of (A) to each flange of (C).

The force in the web of member (D) is—

$$F_w = 75^k \frac{(1.83)}{(7.37)} = 18.6 \text{ kips}$$

If there is 5" of ¼" fillet weld on each side of the web of (D) to the webs of (A) and (C), this will develop:

$F = 2 \times 5''(2.4 \text{ k/in.}) = 24.0 \text{ kips} > 18.6 \text{ kips}$, so is OK

* * *

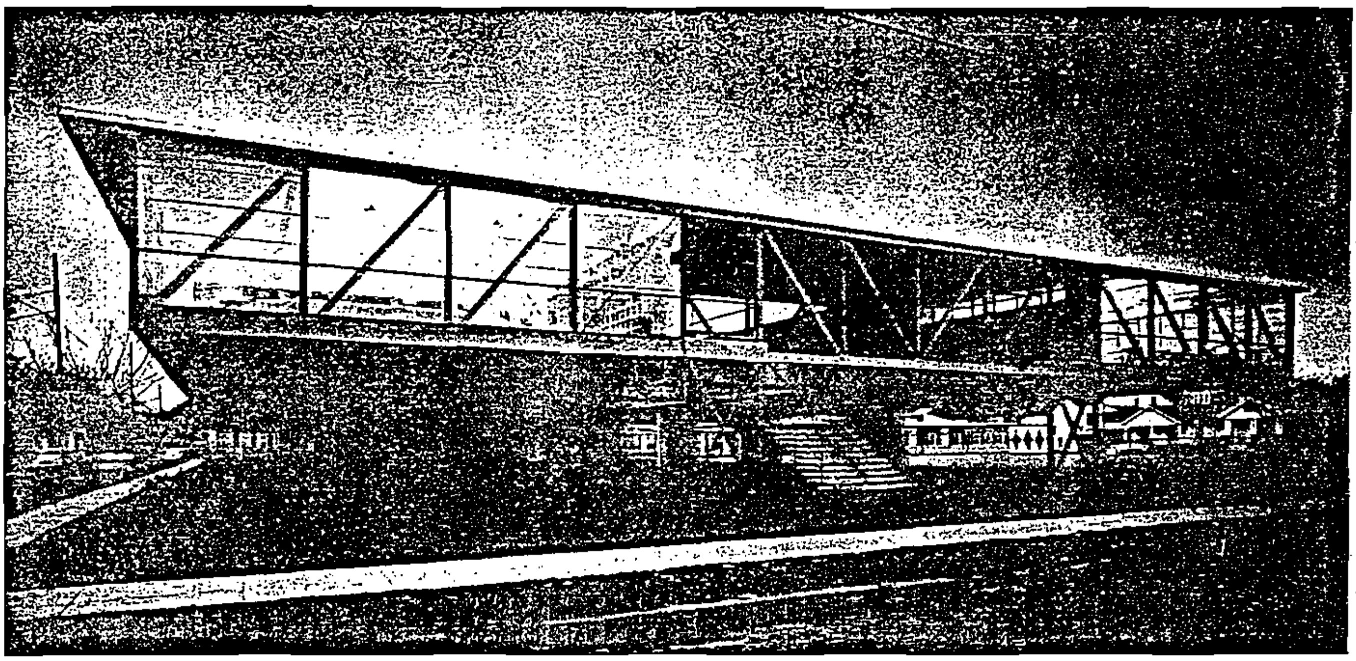
The solution of some truss connection problems can be arrived at by an approach often used in connection with the design of hangers and supports. See Section 6.8, Problem 3.

8. SECONDARY STRESSES IN TRUSSES

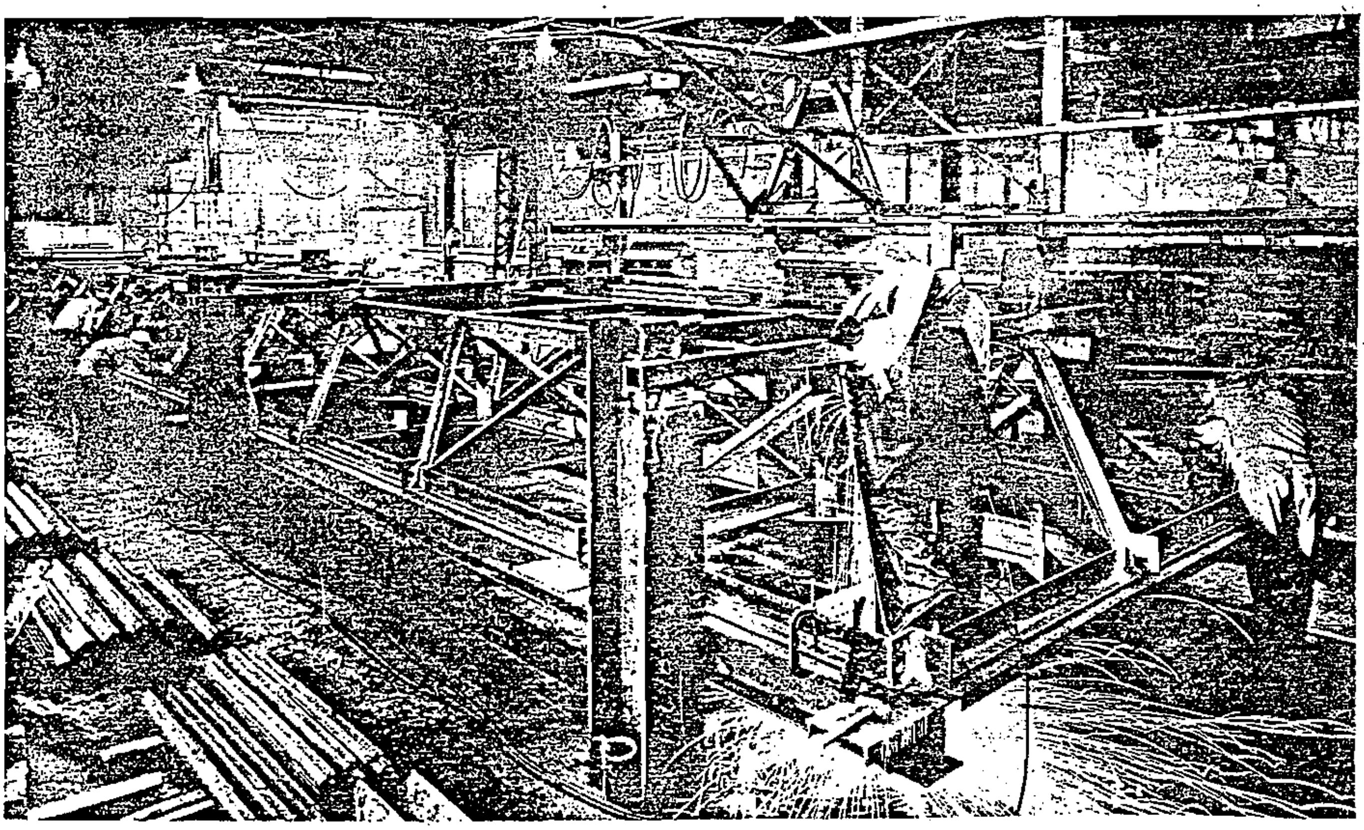
So-called "Secondary Stresses" may result from bending moments applied to the various chord members as the result of the truss deflecting under load. With the exception of large bridge trusses, these secondary stresses are usually ignored and only the primary or direct stresses are considered. If these must be determined, it would be possible to compute the actual deflection of the truss under load and from this condition to then compute the secondary stresses.

In Grinter's "Modern Steel Structures", Volume 1, page 51, he mentions that experiments have shown that these secondary stresses ordinarily do not exceed 30% of the primary stresses for a given member. If the engineer is concerned about this, he may reduce the working stress to allow for an assumed secondary stress of about ⅓ of the primary stress. This method will of course require additional steel, but it is easy to use and is reasonably safe.

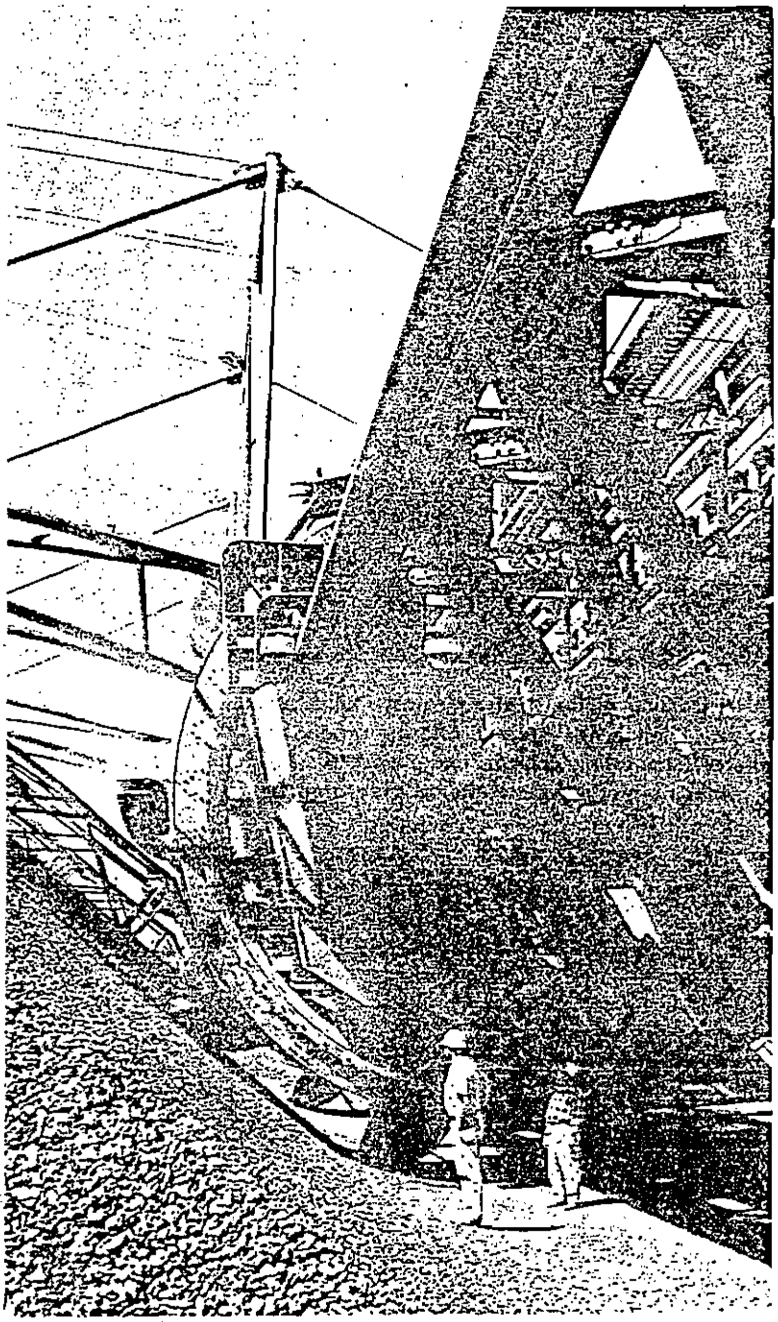
In order to take full advantage of possible economy in the design of large important structures, the secondary moments should be calculated and used in the design with increased working stresses.



Trusses were essential to the all welded framing of the steel and glass Phillis Wheatley Elementary School in New Orleans. The school was erected off the ground on two rows of concrete piers, plus exposed steel supporting columns under end trusses of the cantilevered classroom wings. This provides both open and sheltered play area beneath the structure.



The roof supporting space frame that tops the Upjohn Co.'s Kalamazoo office building is of welded angle construction. A system of subassembly jigs facilitated the holding of alignment during fabrication of the giant frame sections. Nearly all joints are welded downhand.



Main load-carrying element in the world's largest ore reclaimer, at Kaiser's Eagle Mountain mine in California, is a 170′ long welded truss of triangular cross-section. Tubular construction is used where practical for extra strength and torsional resistance, and in order to keep weight to a minimum. Closeup below shows welded cluster where vertical and diagonal members meet the top chord.