The designer is referred to Refs. 53 and 54 for a discussion of design recommendations relative to strength and stability using both the elastic and plastic design methods. The stability is concerned with lateral buckling, vertical buckling of the compression tee, web buckling, web crippling and hole spacing. Concentrated loads should not be placed over the hole. Distributed loads, e.g. through a concrete slab, should be capable of bridging the hole. It is recommended that concentrated loads or reaction points should be distant from the opening edge by half the beam depth, as shown in Fig. 7-49. Corner radius in the hole should be at least ⅝-in., or twice the web thickness.

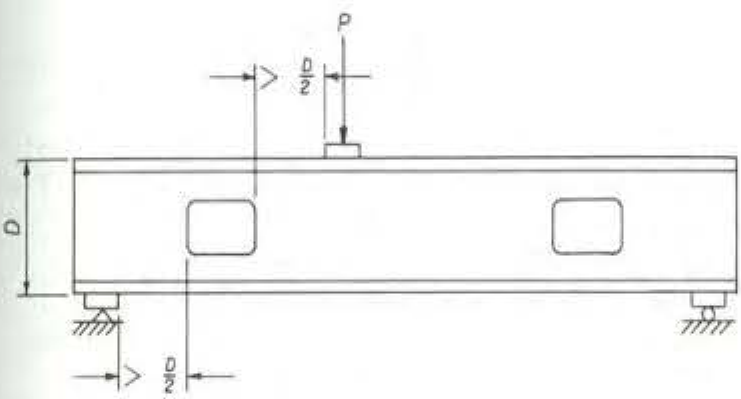

Figure 7-49

Reference 53 discusses the details and the method of attaching the horizontal bar type reinforcing and makes the following recommendations:

1. Reinforcement may be placed on one side of the web only under certain conditions.
2. Fillet welds may be placed on only one side of the bar. This will permit the maximum depth of tee section to be used. If weld is required on the hole side, the bar should be displaced only for enough to make the minimum required weld.
3. The weld should be continuous, since the bar will be in compression over some, or all, of its length.
4. The weld within the length of the opening should develop a force equal to twice the yield strength of the reinforcement. This is because, at failure of the beam at the hole, the reinforcing bar will be fully yielded in tension at one end of the hole and fully yielded in compression at the other end. This capacity need not, however, exceed the yield capacity of the beam web of $2atF_y/\sqrt{3}$, where t is the web thickness and $2a$ is the length of hole as shown in Fig. 7-48.
5. The weld anchoring the reinforcing bar beyond the hole should permit the full normal yield force of the reinforcement area to be transferred. The shear capacity of the web along these welds should not be exceeded. It is recommended that the length of this weld be at least equal to one-quarter of the opening length.

Thus far, only the horizontal reinforcing bars have been considered, since they are efficient and simple to fabricate. This procedure will accommodate the moment end shear forces encountered in most building applications. In many cases, it may be more economical to select a heavier beam section in order to utilize this method of reinforcement or to avoid reinforcing entirely.

If very high shear force is required, it may be necessary to utilize web doubler plates or some other type detail that will transfer these forces. Reference 59 discusses the results of a series of full-size tests on configurations similar to Figs. 7-47b, 7-47c, 7-47d and 7-50, where additional shear plates are shown between the horizontal reinforcement and the beam flange.

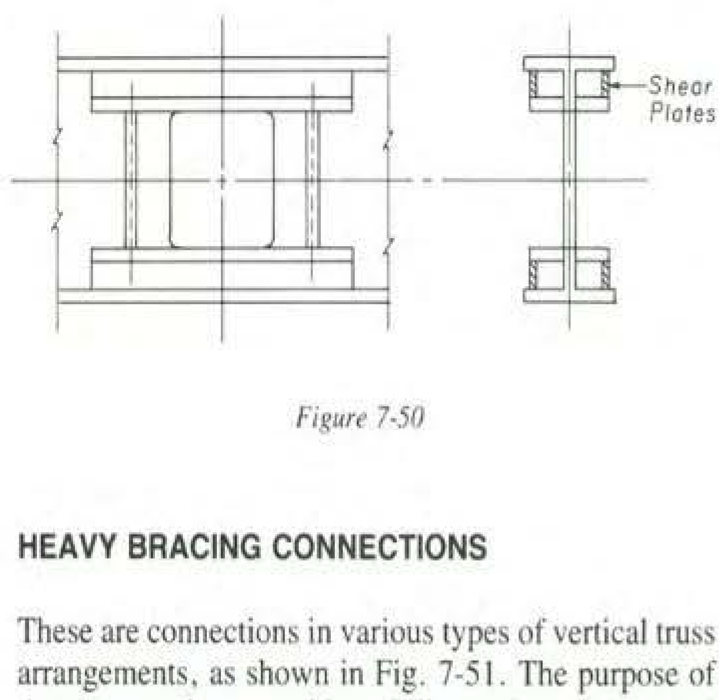

Figure 7-50

HEAVY BRACING CONNECTIONS

These are connections in various types of vertical truss arrangements, as shown in Fig. 7-51. The purpose of these trusses is to provide stability to the structure and to resist wind and seismic forces. Figures 7-51a and 7-51b show vertical bracing, composed of members subjected to tension and compression, occupying a single building bay. Figure 7-51c shows tension/compression bracing occupying two adjacent building bays. Figure 7-51d shows tension only bracing in a single bay, and Fig. 7-51e shows a common type of K-bracing. Other arrangements are possible, such as shown in Fig. 7-51f. The braces themselves may be single or double angle, WT or W sections, or tube

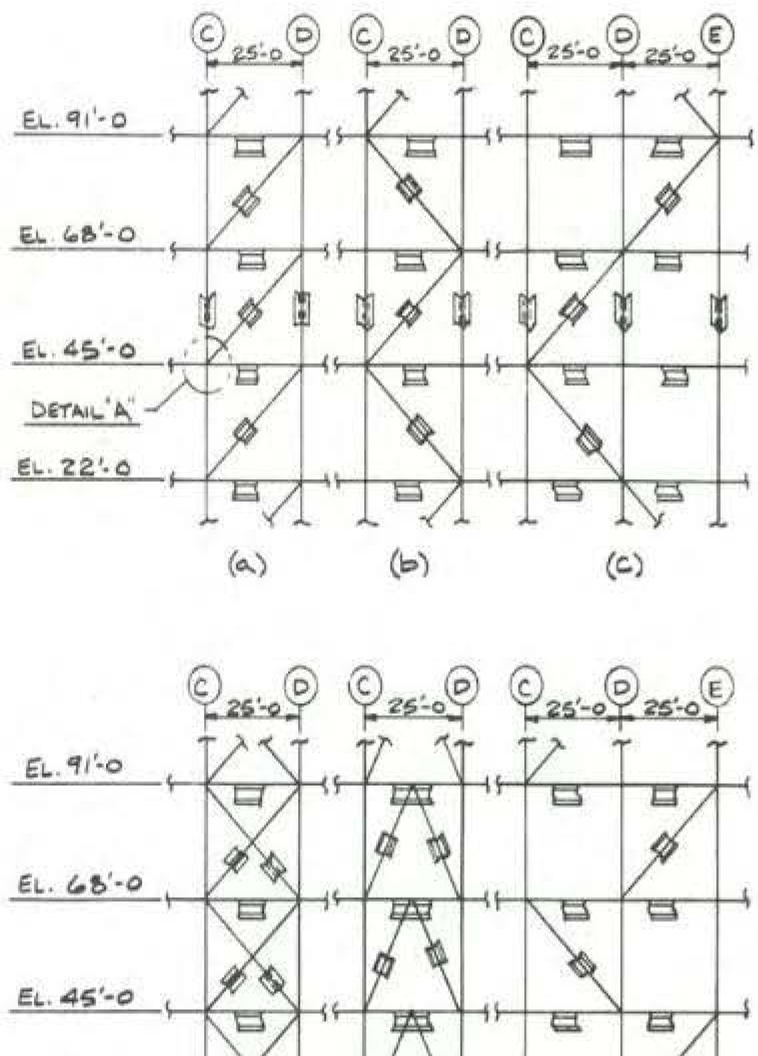

Figure 7-51

sections. Figure 7-52 shows a typical bracing detail where the gusset is prepared to connect a wide-flange member, web to view.

All of the vertical truss arrangements shown in Fig. 7-51 share a common feature not usually present in trusses designed primarily to carry gravity loads. That is, the columns and beams of the building which form the chords and "verticals" of the trusses, respectively, are designed primarily to carry gravity floor and roof loads, and only secondarily are incorporated into the vertical truss. Thus, these columns and beams will generally be much larger, relative to the diagonal brace members, than would normally be the case in gravity loaded trusses. For this reason, some relaxation of the usual requirement for intersection of member gravity axes at a common working point is often permissible in these vertical trusses. The induced secondary stresses in the columns and beams due to non-intersecting gravity axes is usually small compared to the primary gravity stresses for which these members were designed. In cases in which the brace forces become large, secondary stresses should be checked; this is easily done. Also, in this latter case, the designer may prefer to design his columns and beams for these secondary stresses, rather than depend on the connection to develop the secondary stresses. This approach will result in much more compact and visually aesthetic connnections, which are less likely to interfere with building function, i.e., equipment, access, etc., and will also be more economical because, for a small increase in member weight, connection weight will be significantly reduced and, more important, the expensive drilling, punching, cutting, welding and bolting operations will be greatly reduced.

Figure 7-52 shows a beam-to-column web bracing connection which would commonly occur on the perimeter of a building. The column is a W14×211 and the beam is a W24×55, as shown. The beam carries a floor load which results in an end shear reaction of 36.4 kips. The W14×68 brace force is 150 kips due to wind. An additional 40 kips of wind load is added

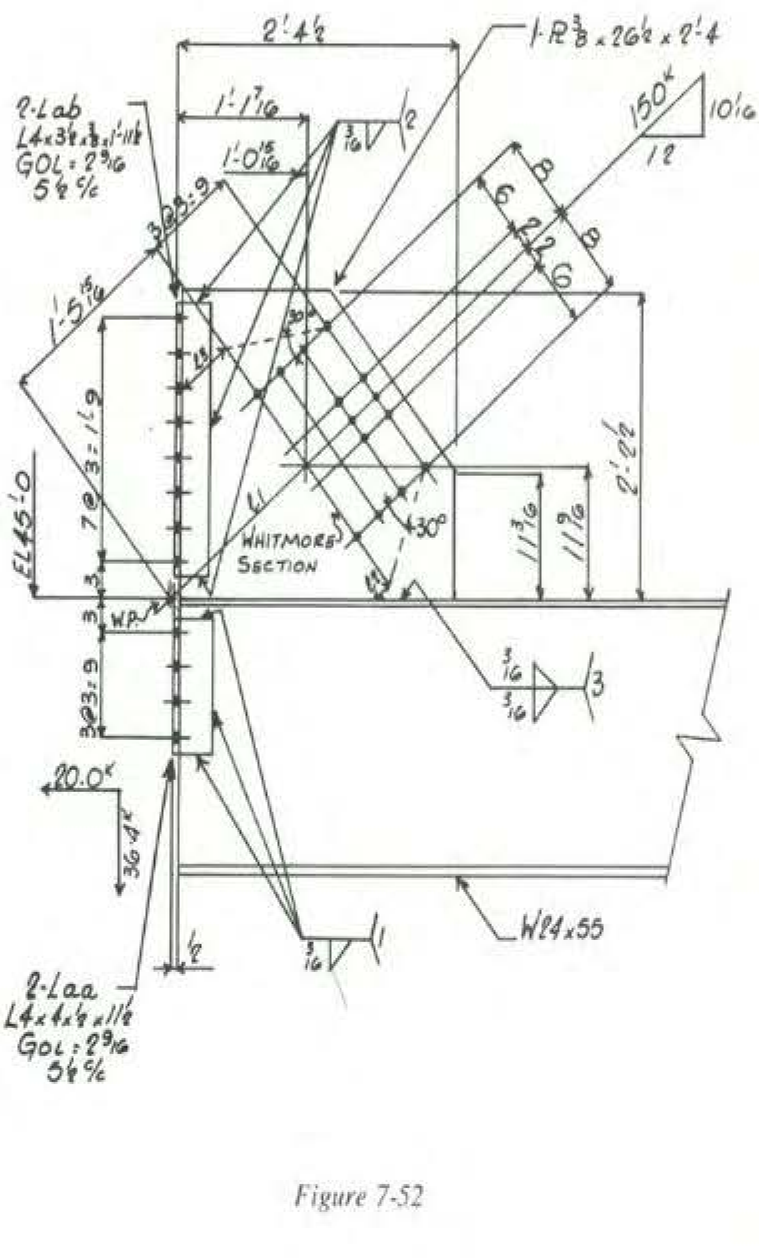

Figure 7-52

to the bracing system from the floor at Elev. 45′0. One half of this wind force is assumed to enter the braced bay at column line C, as shown. The other 20 kips enter at column line D in the same manner. Note that the working point is positioned at the column web center line and the beam top flange. This will produce a compact connection, but will induce a bending moment in the beam which should be checked and, if necessary, the beam size should be increased to accommodate the extra stresses. This check is described below.

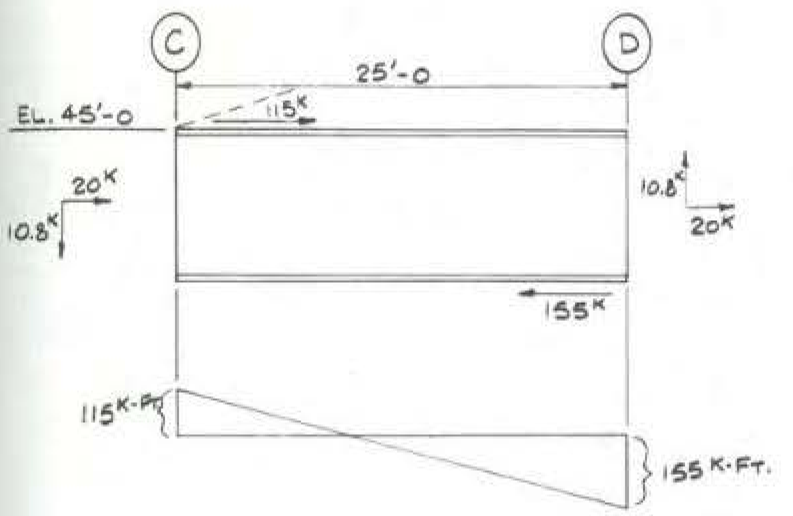

Figure 7-53

Figure 7-53 shows the beam subjected to the eccentric bracing loads. The moment induced in the beam from the lateral loads is largest at the ends where the gravity design moment will be zero. Thus, there will be little interaction between the two and each can be checked separately. The beam is assumed to be subjected to a uniform gravity load which results in the gravity end reaction of 36.4 kips, and a gravity bending moment at the center of the beam of 228 kip-ft, which is equal to the resisting moment (M_R) of this beam (the beam is fully stressed under gravity load alone). Since $0.75 \times 155 = 116.25 < 228$ kip-ft, the beam is satisfactory under wind loads. It can be verified that there is no need in this case to check combined wind and gravity at a location at or near the beam center line, because of the ⅓ increase in allowable stresses (or ¼ reduction in loads) permitted by the AISC Specification when gravity and wind or seismic loads are considered acting simultaneously. The 10.8-kip vertical reaction shown in Fig. 7-53 is caused by the moment and is necessary to keep the beam in equilibrium; it should be considered for inclusion in the design of the beam end connections. The beam end connection must therefore be designed for the greater shear value—36.4 kips gravity shear—and for $0.75(36.4 + 10.8) = 35.4$ kips combined wind and gravity shear. In this case, 36.4 kips is the design shear.* Again, the eccentric brace forces have no effect on the design of the connection. A simple rule of

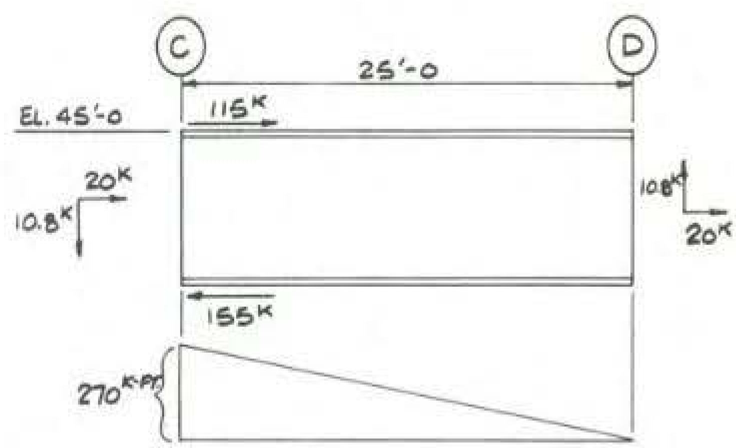

Figure 7-54

thumb worthy of note is that wind or seismic loads plus gravity loads will not be more critical than gravity loads alone unless the wind or seismic load exceeds one-third of the gravity load. Since, in this case 10.8 kips $< 36.4/3 = 12.13$ kips, the eccentric effect of the brace force can be neglected.

Before proceeding to the design of the elements of the connection shown in Fig. 7-52, let us consider the effect of eccentric bracing forces in another bracing arrangement. Consider the beam at Elev. 45′0 in Fig. 7-51b. Let the beam again be under uniform gravity load and the same bracing loads that were used in the previous discussion. Figure 7-54 shows the beam and bending moment diagram. Checking the bending moment at the left end of the beam, $0.75 \times 270 = 202.5$

**Note that in this problem there is a 15-kip (= 0.75 × 20) axial wind load to be considered along with the 35.4-kip shear force.*

< 228 kip-ft. **o.k.** Checking the moment at the center,* 0.75 [(0.5 × 270) + 228] = 272 > 228 kip-ft **n.g.** The lightest W24 which will be satisfactory is a W24×68 with M_R = 308 kip-ft.

Some comments on the above results are in order. First, it can be seen that a rearrangement of the bracing can have a significant effect on the stresses in the main members (floor beams in the present examples) when work points do not correspond to gravity axis intersections. Second, these examples are assumed fully stressed under gravity loads to demonstrate that quite large eccentric bracing forces can nevertheless be accommodated. In actual buildings, members are seldom chosen to be fully stressed under gravity loads, because of deflection limits, member size groupings, etc. It is therefore expected that the members chosen in the gravity load portion of the design process would be able to carry the extra wind loads without overstress.

In any case, in keeping with the general principle that equilibrium and yield must be satisfied, effects of eccentric brace forces must be accounted for to provide a path for the loads to ground. Therefore, the latter arrangement requires a W24×68 beam, or the working points can be moved to gravity axis intersections. Figure 7-55 shows what happens to the connection of Fig. 7-52 if this is done.

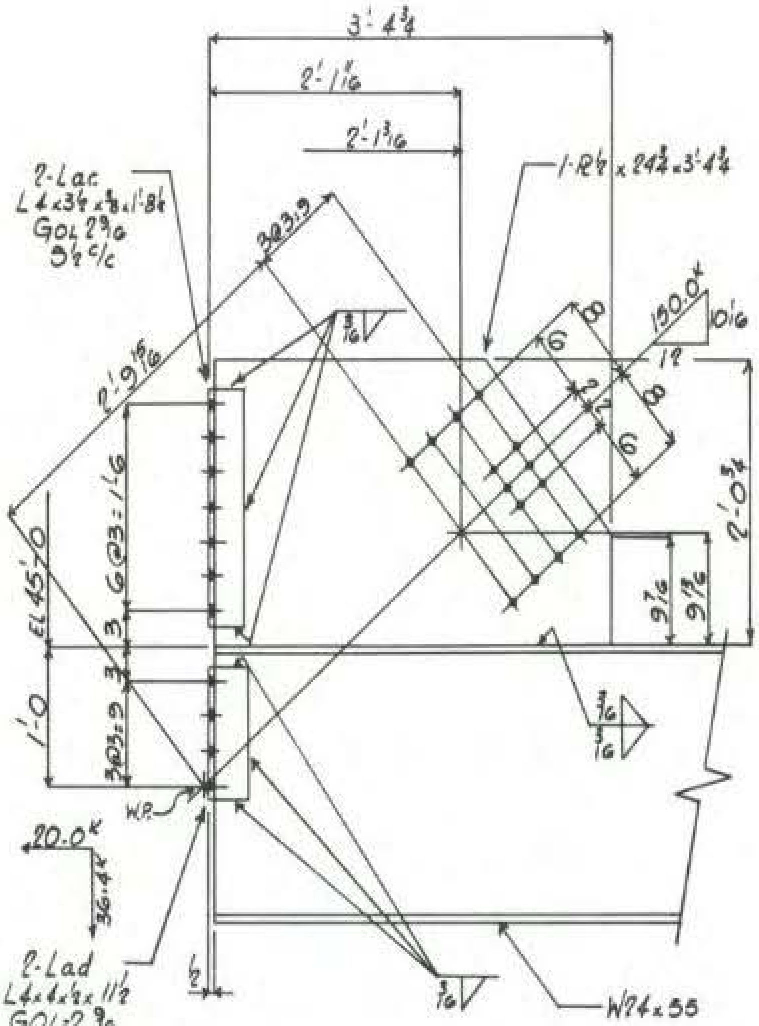

Figure 7-55

Following is the design of the various elements of the connection of Fig. 7-52.

Weld 1:

This is a "C" shaped weld subjected to tension or compression and shear, as shown in Fig. 7-56. Since

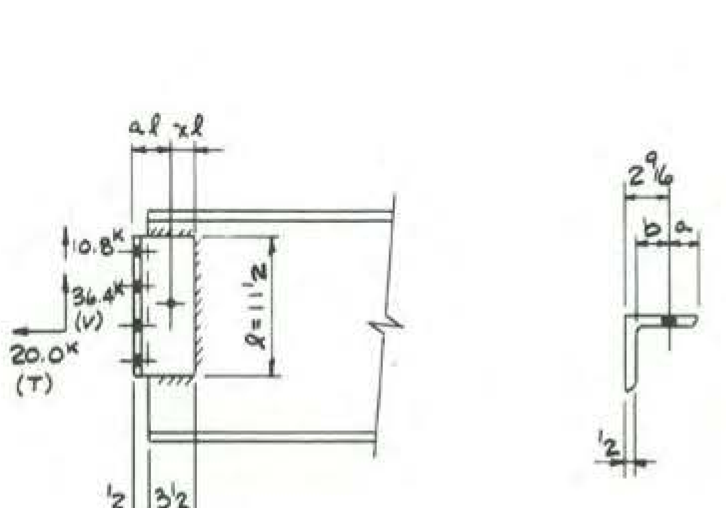

Figure 7-56

there are no tables for this case in the Manual, and the inelastic method is not amenable to simple manual calculations, the classical elastic method (Manual Part 4) will be used.**

l = 11.5 in.

kl = 3.5 in.

k = 3.5/11.5 = 0.304

*Actually, maximum moment will occur at $\bar{x} = (L/2) + (m/WL)$, where L = length, $\bar{x}$ = distance from left end, m = eccentric moment at left end, and WL = total gravity load. In the present case M_{max} at $\bar{x}$ = 383 kip-ft, which when reduced for wind, becomes 0.75 × 383 = 287 kip-ft. This is only 5% larger than the moment at center. Considering that the beam is not actually simply supported and that the actual beam length is reduced due to connection size, this 5% can be neglected.

**The inelastic method can handle the problem, using a computer. See AISC Engineering Journal, Third Quarter, 1982, Vol. 19, No. 3, pp. 150-159.

$xl = (3.5)^2/18.5 = 0.662$ in.

$al = 4.0 - 0.662 = 3.338$ in.

$$I_p = (11.5)^3 \times \left[\frac{(1 + 0.608)^3}{12} - \frac{(0.304)^2 + (1.304)^2}{1 + 0.608}\right]$$
$$= 378.3 \text{ in.}^4$$

Stress due to vertical load V:

$$F_y = \frac{47.2}{18.5} \times 0.75 = 1.914 \text{ ksi}$$

Stress due to horizontal load T:

$$f'_x = \frac{20}{18.5} \times 0.75 = 0.811 \text{ ksi}$$

Stress due to couple $M = V \times al = 47.2 \times 3.338 = 157.6$ kip-in:

$$f'_x = \frac{157.6 \times 5.75 \times 0.75}{378.3} = 1.796 \text{ ksi}$$

$$f'_x = \frac{157.6 \times 5.75 \times 0.75}{378.3} = 1.796 \text{ ksi}$$

$$f'_y = \frac{157.6 \times (3.5 - 0.622)}{378.3} \times 0.75$$
$$= 0.899 \text{ ksi}$$

$$f_R = \sqrt{(f_x + f'_x)^2 + (f_y + f'_y)^2}$$
$$= [(0.811 + 1.796)^2 + (1.914 + 0.899)^2]^{1/2}$$
$$= 3.83 \text{ ksi} < 0.928 \times 3 \times 2 = 5.568$$

Weld **o.k.** for combined tension and shear (wind and gravity loads).

For gravity load alone, Table XXIII, Manual Part 4, can be used. Thus, for $k = 0.304$ and $a = 3.338/11.5 = 0.29$, coefficient $C = 1.06$. Then, the weld capacity $= 2 \times 1.06 \times 3 \times 11.5 = 73.1$ kips $>$ 36.4, **o.k.** for gravity load alone. The 3/16-in. fillet weld is used because it is the smallest permitted by AISC Specification Sect. 1.17.2

To complete the check of the weld, the W24×55 web must be checked to determine if it is heavy enough to support a 3/16-in. fillet weld on both sides. A simple but very conservative way to do this is to assume that the shear in the weld at the most critical point produces a local shear stress in the beam web on a plane coinciding with the direction of the maximum weld stress, and to require that this "point" web stress does not exceed $0.4F_y$. In this case, f_R = 3.83 kips/in. for combined wind and gravity loads and $(36.4/73.1) \times 3 \times 2 \times 0.928 = 2.77$ kips/in. for gravity loads alone. Use 3.83 kips/in.

Then, $3.83 \leq 0.4 \times 36 \times t_w$. Required $t_w = 0.266$ in. Since t_w of the W24×55 is 0.395 in., the 3/16-in. fillet is fully effective. If the required t_w exceeded the actual t_w, the weld capacity would be reduced by the ratio of the actual value to the required value.

The above method for checking the web thickness is quite conservative, because it uses the maximum stress at one point (at most, four points) to determine the capacity of the entire connection, which consists of an infinity of points which integrate to a connection length of 18½ in. in this case.

It should be noted that Weld **1** could be replaced by shop bolts. In that case, the design shear in the bolts will be based upon either the gravity load acting alone (36.4 kips in this example) or the combined gravity and wind loads $0.75[(36.4 + 10.8)^2 + (20)^2]^{1/2} = 38.4$ kips, whichever is larger. All of the various edge, end, spacing, net and block shear and bearing checks can be simply and conservatively performed, using the larger of the two loads as the design load.

Angles **aa**—Field Bolted Connection:

Let the field bolts used in this connection be 7/8-in. dia. A325-N high-strength bolts. These bolts are subjected to a combination of tension and shear. Prying action, caused by bending of the outstanding legs of angles **aa**, must also be considered. This type of connection is analyzed as follows (see Manual p. 4-89 for design method). Figure 7-56b gives the geometry of the clip angle.

$b = 2.5625 - 0.5 = 2.0625$ in.

$b' = 2.0625 - (0.875/2) = 1.6250$ in.

$a = 4 - 2.5625$
$= 1.4375 < 1.25b = 2.5781$ in. **o.k.**

$a' = 1.4375 + 0.875/2 = 1.8750$

$d' = 15/16 = 0.9375$ in.

$\delta = 1 - 0.9375/3 = 0.6875$

$T = 0.75 \times 20/8 = 1.875$ kips

$M = 3 \times 0.5^2 \times (36/8) = 3.3750$ kip-in.

$\alpha = [(1.875 \times 1.625/3.375) - 1]/0.6875$
$= -0.141$ (use $\alpha = 0$)

$B_c = 1.875\ (1 + 0) = 1.875$ kips

$V_b = 35.4/8 = 4.43$ kips/bolt

$B = 33.07 - 1.8V_b = 33.07 - (1.8 \times 4.43)$
$= 25.10 < 26.5$ kips

Since $B_c = 1.875 < B = 25.10$, bolts are **o.k.** in tension and shear.

Check angle leg thickness:

$$t_f\,(\text{req}) \geq \left[\frac{8 \times 1.875 \times 1.6250}{3 \times 36 \times 1.0}\right]^{1/2} = 0.4751 \text{ in.}$$

Since $0.4751 < 0.5$, the $4 \times 4 \times \frac{1}{2}$ clip angles are satisfactory. Therefore, use the $4 \times 4 \times \frac{1}{2}$ clips.

Weld **2**:

Like Weld **1**, this is a "C"-shaped weld. It is subjected to the vertical component of the brace force, which is 96 kips, as shown in Fig. 7-57. (Note that the gusset plate shown in Fig. 7-57 is in equilibrium under the loads and reactions shown.) Because Weld **2** is subjected only to the vertical shear of 96 kips, Weld **A** of Table III, Manual Part 4, can be used to size the weld. From Table III, a 3/16-in. fillet weld on a connection angle 1 ft-11½ in. long has a capacity of 133 kips. Since $133 > 0.75 \times 96 = 72$ kips, a 3/16-in. fillet weld is satisfactory.

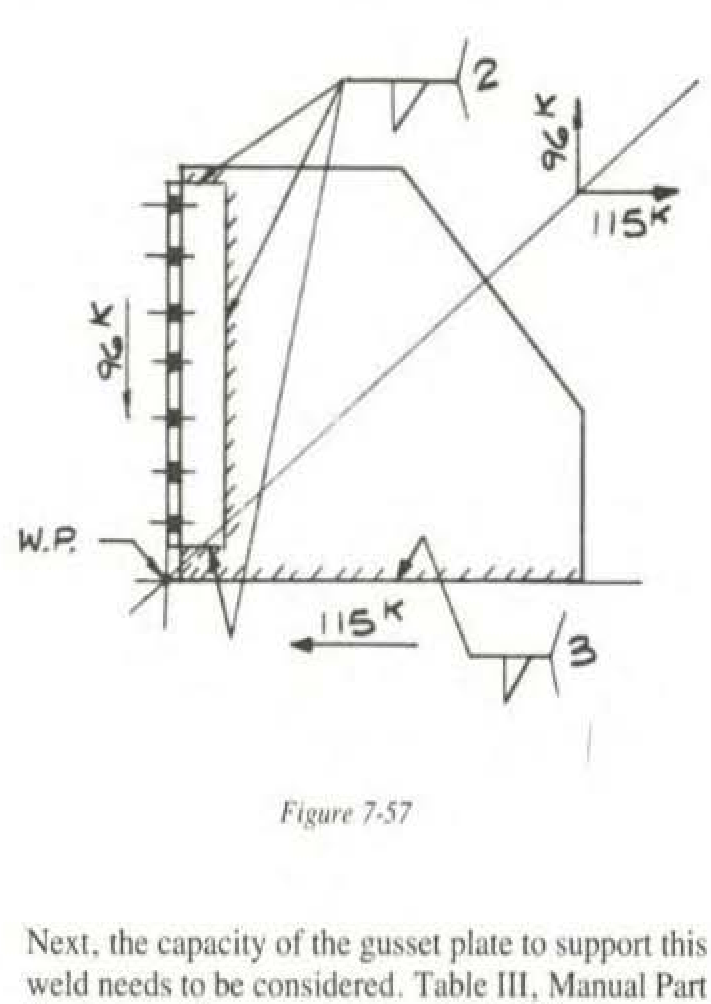

Figure 7-57

Next, the capacity of the gusset plate to support this weld needs to be considered. Table III, Manual Part 4, gives minimum web (or plate) thicknesses required based on the point stress method. Thus, a 3/16-in. fillet weld requires a web thickness of 0.38 in. to fully develop the fillet weld. The ⅜-in. gusset plate is close enough to 0.38 in. to do this. In the present problem, however, only 72 kips of the 133-kip weld capacity is required. Thus, a plate thickness of (72/133)0.38 = 0.206 in. is all that is required to support the weld. Again, a 3/16-in. fillet weld is the minimum size allowed by AISC Specification Sect. 1.17.2

As noted for Weld **1**, Weld **2** could be replaced by shop bolts. In this case, the bolted connection would be designed in exactly the same way as a beam web shop bolted shear connection. The equilibrium diagram of Fig. 7-57 would be exactly the same with Weld **2** replaced by bolts.

Angles **ab**—Field Bolted Connection:

This connection is analyzed in the same manner as a field bolted beam web connection.

Weld **3**:

As shown in Fig. 7-57, Weld **3** is subjected to a horizontal shear of 115 kips. The length of the weld is 28 in. on each side of the plate. Thus, the force f_R per inch of weld is:

$$f_R = \frac{0.75 \times 115}{2 \times 28} = 1.54 \text{ kips/in.}$$

The required weld size is 1.54/0.928 = 1.66 or ⅛-in. The W24×55 has a ½-in. flange, so AISC Specification Sect. 1.17.2 requires a 3/16-in. fillet weld each side. The weld is shown in Fig. 7-52.

Next, consider a horizontal section cut through the gusset plate just above Weld **3**. The stress on this section is:

$$f_R = \frac{0.75 \times 115}{28 \times 0.375} = 8.21 < 14.5 \text{ ksi}$$

The gusset plate can carry the shear load of 115 kips.

Consider now Weld **3** for the gusset plate of Fig. 7-55. A free body diagram for this gusset plate is shown in Fig. 7-58. Because the working point is now at the beam center line, 11 13/16 in. below the top flange, the gusset is not in equilibrium unless a couple of magnitude 115 × 11 13/16 = 1358 kip-in. is applied to the horizontal lower edge of the 40.25-in. long plate in the direction shown. Applying the couple to satisfy equilibrium of the gusset, Weld **3** will now be subjected to the following shear and tension forces:

$$f_v = \frac{0.75 \times 115}{2 \times 40.25} = 1.07 \text{ kips/in.}$$

$$f_t = \frac{3 \times 1358 \times 0.75}{(40.25)^2} = 1.89 \text{ kips/in.}$$

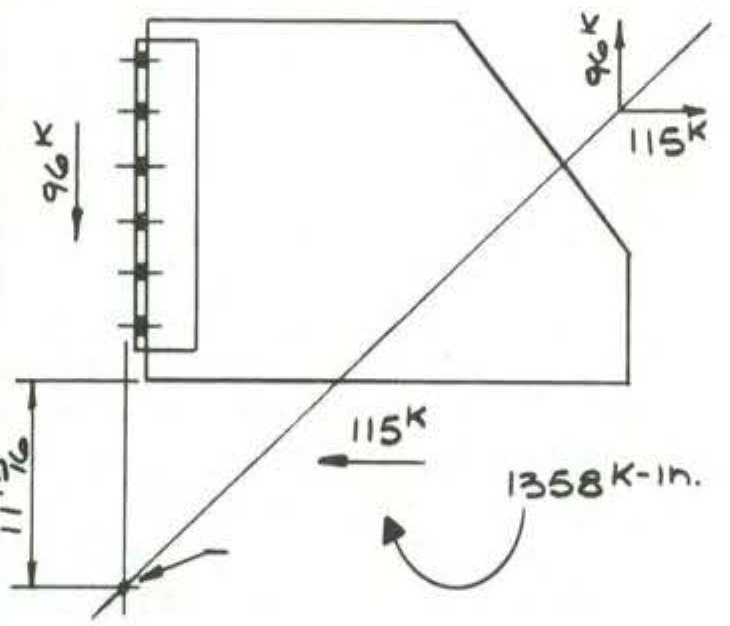

Figure 7-58

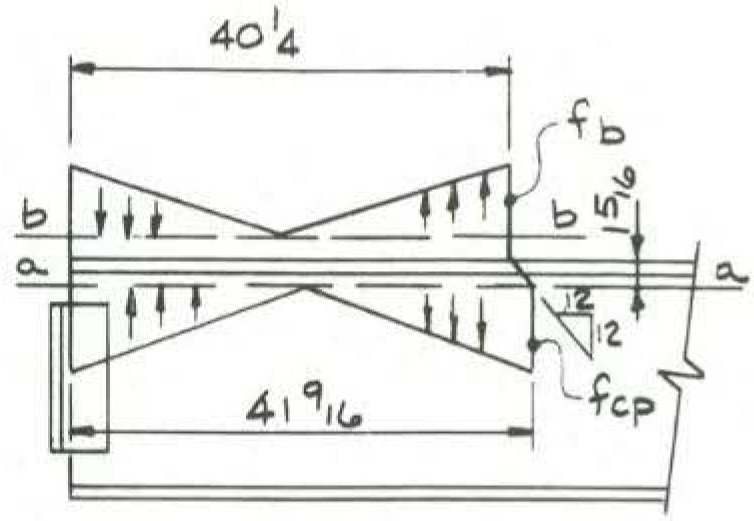

Figure 7-59

Now, the resultant force per inch of weld is:

$$f_R = (1.07^2 + 1.89^2)^{1/2} = 2.17 \text{ kips/in.}$$

The required weld size is 2.17/0.928 = 2.34 or 3/16-in. Table XIX, Manual Part 4, can be used as an alternate method to check Weld **3**. Using the "special case" of Table XIX, al = 11 13/16 in., l = 40.25 in., and a = 11 13/16/40.25 = 0.2935. Interpolating in Table XIX for $k = 0$ and $a = 0.2935$, $C = 1.16$. Thus, the number of 16ths of an inch of weld required is:

$$D = \frac{0.75 \times 115}{1.16 \times 40.25} = 1.85$$

Therefore, a 1/8-in. fillet weld is required. As before, a 3/16-in. fillet weld must be used.

Next, consider the previously discussed horizontal section in the gusset plate just above the weld. The shear stress on this section is:

$$f_v = \frac{0.75 \times 115}{40.25 \times 0.5} = 4.29 < 14.4 \text{ ksi} \quad \textbf{o.k.}$$

The bending stress is:

$$f_b = \frac{6 \times 1358 \times 0.75}{(40.25)^2 \times 0.5} = 7.54 < 22 \text{ ksi} \quad \textbf{o.k.}$$

Therefore, Weld **3** and the gusset plate are satisfactory.

A further check for beam web crippling should be made for the gusset plate configuration of Figs. 7-55 and 7-58 when the gusset is much thicker than the beam web. This will now be illustrated, even though it is not required in the present case because gusset and web thickness differ by only 1/8-in. Consider horizontal Section a-a in the beam web just below the toe of the flange to web fillet, as shown in Fig. 7-59. Because of the gusset bending stress of f_b = 7.54 ksi, which occurs on Section b-b, a similar stress f_{cp} will occur in the beam on Section a-a. If the beam web is very thin compared to the gusset, stress f_{cp} could be large enough to cause beam web crippling. In this case,

$$f_{cp} = \frac{6 \times 1358 \times 0.75}{0.395(41.5625)^2} = 8.95 < 0.75F_y = 27 \text{ ksi}$$

Therefore, beam web crippling will not occur.

Except for the above discussion regarding Weld **3** and beam web crippling, the connection of Fig. 7-55 is designed in the same fashion as the connection of Fig. 7-52.

Additional Gusset Plate Checks:

Gusset Plate Tear-Out:

This check is related to the block shear/net shear requirements of the 1978 AISC Specification. Figure 7-60 shows the tear-out section for the 3/8-in. gusset plate. The capacity is based on net section with hole size taken as bolt diameter plus 1/16-in., as in the block shear calculations. The net shear area is:

$$A_v = [10.75 - (3.5 \times 0.935)] \times 0.375 \times 2 = 5.608 \text{ in.}^2$$

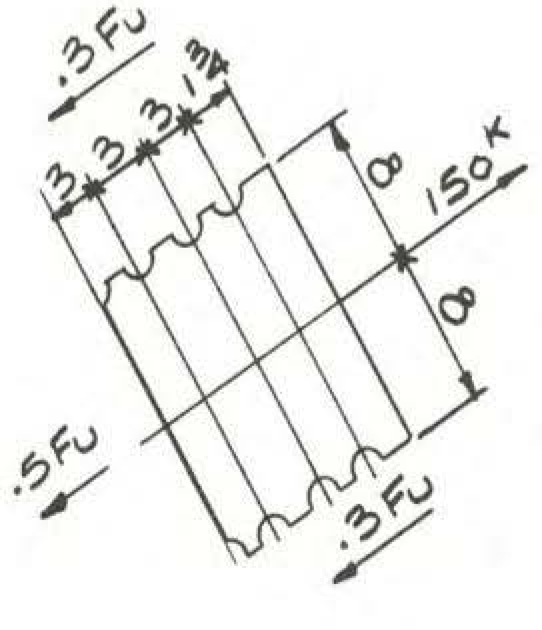

Figure 7-60

The net tension area is:

$$A_t = (16 - 0.9375) \times 0.375 = 5.648 \text{ in.}^2$$

Thus, the allowable brace force is:

$$P_{allow} = (5.608 \times 0.3 \times 58) + (5.648 \times 0.5 \times 58) = 261 > 0.75 \times 150 = 113 \text{ kips}$$

The gusset is satisfactory.

Gusset Plate Buckling:

The Whitmore Section (Ref. 8), as shown in Fig. 7-52, is a reasonable section to use as a basis for checking gusset stability. The stress on this section is:

$$f_a = \frac{150 \times 0.75}{26.39 \times 0.375} = 11.37 \text{ ksi}$$

where the gross area is used. The Whitmore Section stress is a fairly crude approximation to gusset stress which does not seem to justify the precision implicit in subtracting out the holes. When the brace force is tension, $f_a = 11.37 \text{ ksi} < 0.6F_y = 22$ ksi satisfies the stress requirement. However, $0.6F_y$ may be too high an allowable stress when the brace force is compression. To determine a conservative allowable compression stress, consider a 1-in. wide strip of gusset plate from the Whitmore Section to the working point along the line of action of the brace. The length l_1 of this strip of plate is approximately 1 ft-5 in. Now consider this 1-in. strip of plate to be a fixed-fixed column ($K = 0.65$) of slenderness ratio:

$$\frac{Kl_l}{r} = \frac{0.65 \times 17}{0.108} = 102$$

Then the allowable compressive stress F_a from Specification Table 3-36 is 12.72 ksi. Since 11.37 ksi < 12.72 ksi, the gusset will not buckle under the design load.

The method presented to determine an allowable buckling stress by using a strip is conservative, because it ignores plate action and the great post-buckling strength of plates. In the plate, Fig. 7-52, it is conservative also because the strip length taken is the maximum unsupported length of plate between the Whitmore Section and the supported edges of the plate. A shorter length, such as the average of the lengths l_1, l_2 and l_3 of Fig. 7-52, would appear to give a more reasonable approximation of buckling strength. Note, however, that using the average of l_1, l_2 and l_3 will not always result in a length less than l_1. This can be seen by reference to Fig 7-55. In this case using l_1 as the strip length may be unconservative.

Summary—Heavy Bracing Connections

The approach presented here for the analysis and design of vertical bracing connections is based on the dual requirements that equilibrium be satisfied for all parts of the connection and yield be satisfied for all cut sections and connecting elements on the boundaries of the parts. Because of the Lower Bound Theorem of Limit Analysis, this approach will produce a conservative connection, provided also that due consideration has been given to stability requirements.

The location of working points was also considered. It was found that positioning working points to simplify connection geometry can be achieved with no effect on main member sizes.