Bracing Connections for Heavy Construction

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INTRODUCTION AND PHILOSOPHY

The design of complex connections is not an exact science. Over the years, an intuitive approach to connection design has become widely accepted. This approach is based on the idea the structure (and parts thereof) will behave as the designer dictates, if he provides a path of adequate strength for the load (or loads) to follow. This "adequate strength path" is determined from the principles of statics and strength of materials.

About 30 years ago, this intuitive load path method of design was put on a rigorous basis with the development of the Theorem of Limit Analysis—specifically the Lower Bound Theorem of Limit Analysis,[1] which states: if a distribution of forces in the structure can be found which is in equilibrium with the applied loads, and if these forces everywhere within the structure are of such a magnitude that the yield stress (or yield criterion) is nowhere exceeded, then the applied loads are less than, or at most equal to, the loads required for collapse (unbounded yield deformations) to occur. Thus, if a load path is provided, the elements of which are in equilibrium with the applied loads, and if the stresses in these elements nowhere exceed the yield stress, a safe design will have been achieved. Also, the relative stiffness of the various connection elements should be considered in order to minimize the possibility of fracture.

In addition to strength, the stiffness of the structure and its connections must be considered. For the Lower Bound Theorem to be valid, a structure must be stiff enough to preclude buckling before yielding occurs. In connection design, this requirement can usually be met by consideration of appropriate width/thickness ratios and related local buckling formulations which force the elements to yield before they buckle.

The approach to connection design to be presented here will be based on satisfying the dual requirements of equilibrium and yield, with sufficient attention given to stiffness to preclude buckling and fracture. But, both equilibrium and yield are pointwise phenomena which theoretically must be satisfied at every point of the continuum which makes up the structure or connnection. This is an almost impossible task to achieve unless one resorts to some sort of numerical approach to the continuum such as the finite element method. However, as long as equilibrium is satisfied for all finite portions of the structure, it will also be satisfied at every point within these finite portions. Thus, we need only look at finite portions of the structure or connection and need not be concerned with equilibrium at every point.

Yield also is a pointwise function and must theoretically be considered at all points of the structure. Again, this is just about impossible to achieve in actual structures and connections, unless an advanced numerical method is used. The approach here instead will be to insure satisfaction of the yield criterion on all boundaries of the finite portions of the structure considered in the equilibrium analysis. This does *not* guarantee that yielding will not take place locally within some portion of the connection, but such possible yielding will be contained by the elastic boundaries of the portion and will be limited in magnitude by the elastic deformations of the boundaries. Thus, these possible local plastic deformations cannot cause a structural collapse.

Vertical Bracing

Let us now turn our attention to the design of vertical bracing connections.

These are connections in various types of vertical truss arrangements, as shown in Fig. 1. The purpose of these trusses is to provide stability to the structure and to resist wind and seismic forces. Figure 1a and 1b show vertical bracing, composed of members subjected to tension and compression, occupying a single building bay. Figure 1c shows tension/compression bracing occupying two adjacent building bays. Figure 1d shows tension only bracing in a single bay, and Fig. 1e shows a common type of K bracing. Other arrangements are possible, such as shown in Fig. 1f. The braces themselves may be single or double angle, WT or W sections, or tube sections. Figure 2 shows a typical bracing detail where the gusset is prepared to connect a wide-flange member, web to view.

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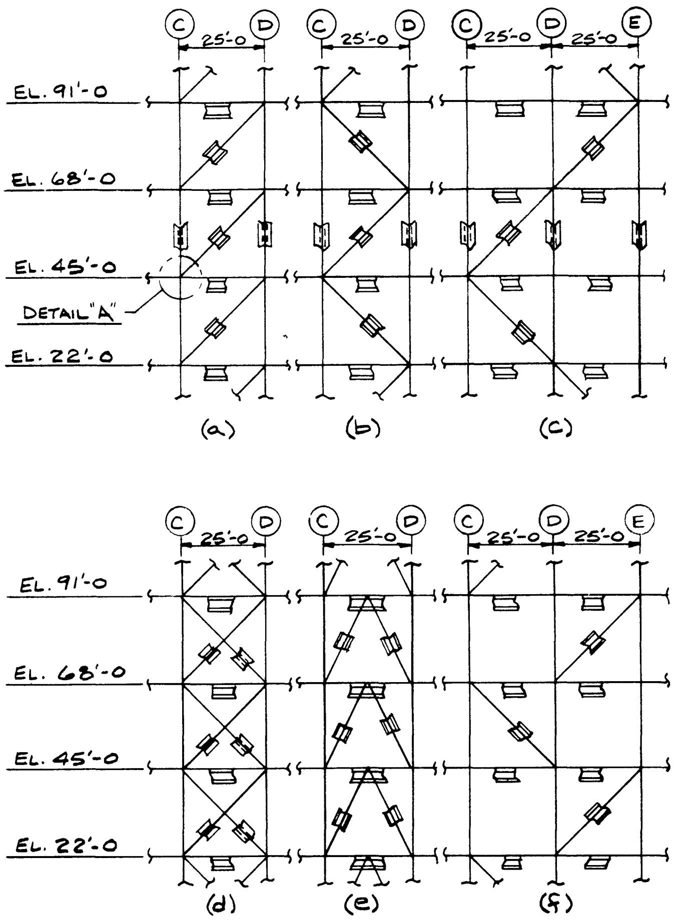

Fig. 1. Various vertical bracing arrangements

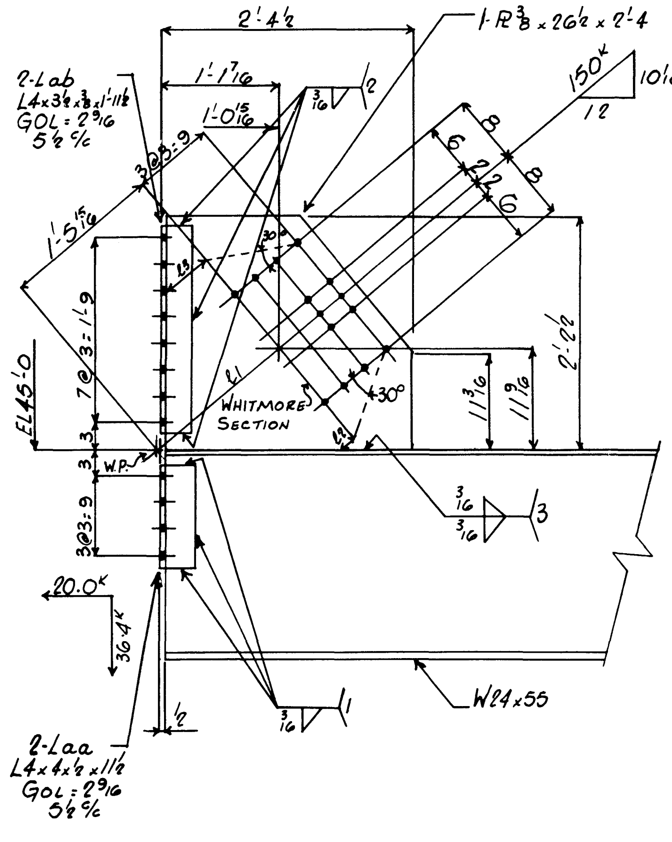

Fig. 2. Bracing connection (Detail "A" of Fig. 4a)

All of the vertical truss arrangements shown in Fig. 1 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 generally will 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 nonintersection gravity axes are 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 connections, 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 2 shows a beam-to-column web bracing connection which would commonly occur on the perimeter of a building. The column is a W14 $\times$ 211 and the beam is a W24$\times$55, as shown. The beam carries a floor load which results in an end shear reaction of 36.4 kips. The W12$\times$53 brace force is 150 kips due to wind. An additional 40 kips of wind load is added 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

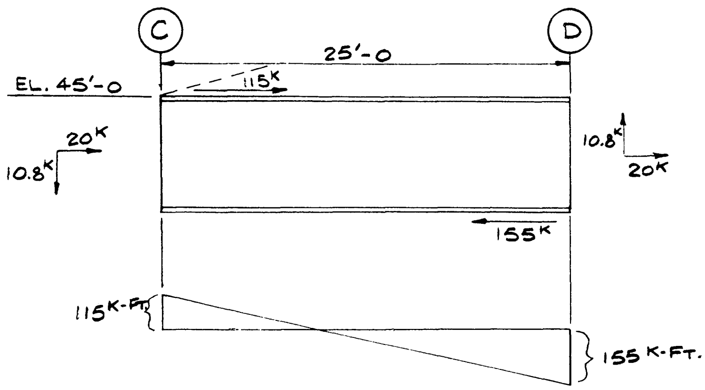

Fig. 3. Beam subjected to eccentric bracing loads (arrangement of Fig. 4a)

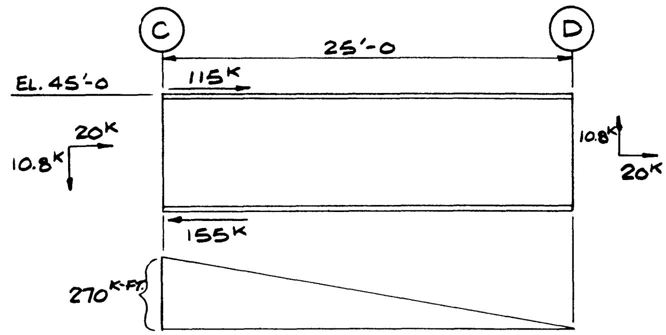

Fig. 4. Beam subjected to eccentric bracing loads (arrangement of Fig. 4b)

be checked and, if necessary, the beam size should be increased to accommodate the extra stresses. This check is described below.

Figure 3 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. 3 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 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. 2, let us consider the effect of eccentric bracing forces in another bracing arrangement. Consider the beam at Elev. 45′0 in Fig. 1b. Let the beam again be under uniform gravity load and the same bracing loads that were used in the previous discussion. Figure 4 shows the beam and bending moment diagram. Checking the bending moment at the left end of the beam, $0.75 \times 270 = 202.5 < 228$ kip-ft. **o.k.**

Checking the moment at the center,** $0.75\,[(0.5 \times 270) + 228] = 272 > 228$ 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 portions of the design process would be able to carry the extra wind loads without overstress.

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.

**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 $0.75 \times 383 = 287$ 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.*

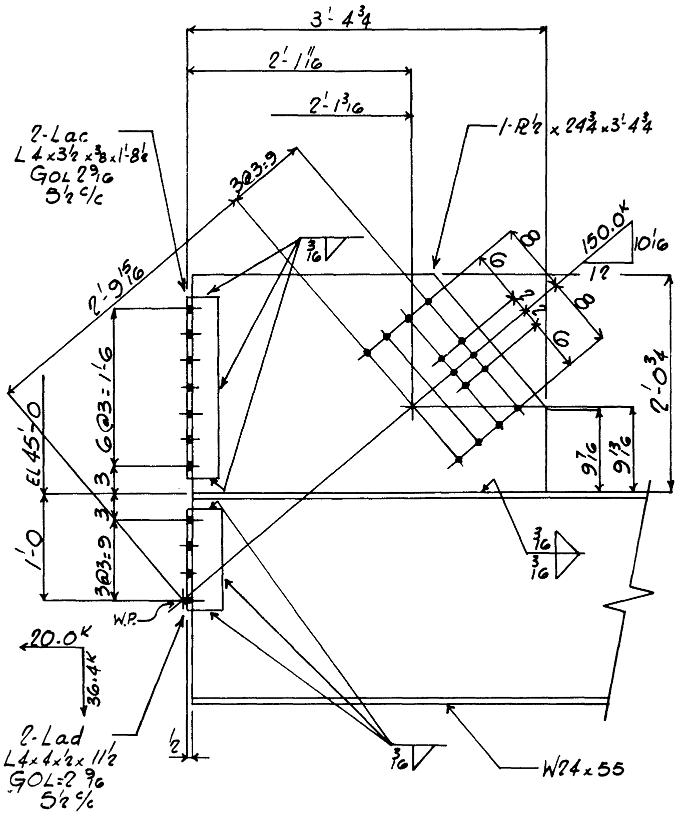

Fig. 5. Bracing connection (Detail "A" of Fig. 4a) working point at gravity axis intersection

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 5 shows what happens to the connection of Fig. 2 if this is done.

Following is the design of various elements of the connection of Fig. 2.

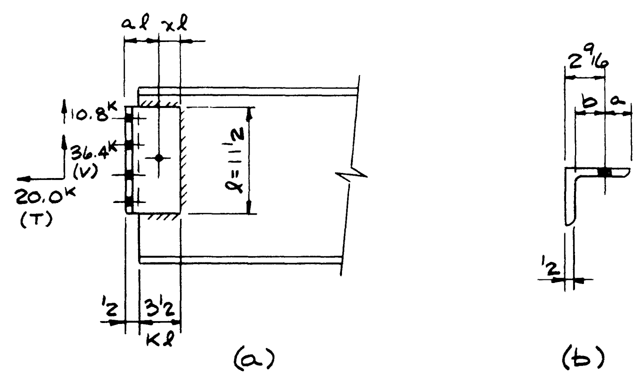

Figure 6

Weld 1:

This is a "C" shaped weld subjected to tension or compression and shear, as shown in Fig. 6. Since 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.
$k = 3.5/11.5 = 0.304$
$xl = (3.5)^2/18.5 = 0.662$ in.
$al = 4.0 - 0.662 = 3.338$ in.

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

$= 378.3$ 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:

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

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

$= 0.899$ ksi

$$f_R = \sqrt{(f_x + f'_x)^2 + (f_y + f'_y)^2}$$

$= [(0.811 + 1.796)^2 + (1.914 + 0.899)^2]^{1/2}$

$= 3.83 \text{ ksi} < 0.928 \times 3 \times 2 = 5.568$ ksi

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

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

To complete the check of the weld, 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.

Alternate Method to Check Web Strength

As mentioned, the above "point stress" method for checking the web is conservative. This is because it uses the maximum stress in the weld, which generally occurs at a single point, or over a relatively small portion of the total weld length when welds are loaded eccentrically, to estimate the load capacity of the web under the weld. Since the orientation of the plane in the web upon which the assumed web shear stress acts changes as the considered weld point changes, i.e. as the length of weld is traversed, there is no continuous critical or failure surface in the web associated with these shear stresses. Thus, there is little likelihood the point stress method provides an accurate assessment of web capacity. It is conservative because it assumes the least possible allowable stress, $0.4\ F_y$, occurs everywhere in the web along the length of the weld, and does not consider that these stresses do not occur on a possible failure surface.

An alternate method to check web capacity, based on recent research,[3,4] is to consider various tear-out modes of web failure, with relatively uniform stress distributions assumed on the failure surfaces which are in equilibrium with the applied loads. Thus, for vertical (or shear) load, Ref. 3 gives the model of Fig. 7, from which the allowable shear for the coped case is determined as:

$$V_{allow} = ½(.5F_u)\,(kl)t_w + (.4\ F_y)lt_w$$

For axial (or tension) load, Ref. 4 gives the model of Fig. 8, from which the allowable tension can be seen to be:

$$T_{allow} = (.5\ F_u)lt_w + .4\ F_y\ (2kl)t_w$$

The models of Figs. 7 and 8 and the resulting expressions for V_{allow} and T_{allow} have been verified by physical testing. In the case of V_{allow} it is to be noted that moment equilibrium is not satisfied by the model of Fig. 7. In the testing program the clip angle connection was determined to provide some rotation capacity. Thus, there is a couple applied to the web section in addition to the shear, but it is ignored in the capacity formula. Also, note that shear capacity is based on a coped beam, while the tension capacity is based on an uncoped beam. No research has yet been done on shear web capacity for uncoped beams and tension web capacity for coped beams,

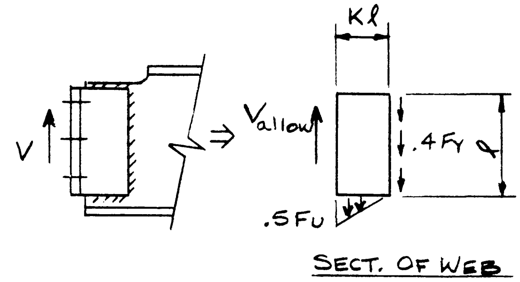

Figure 7

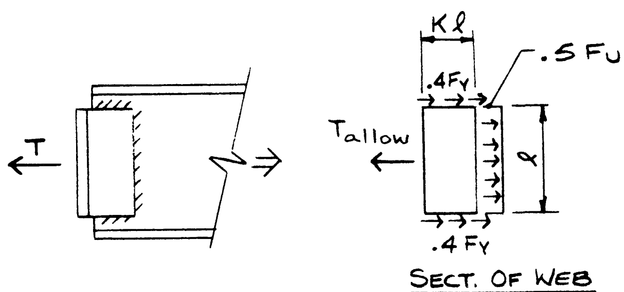

Figure 8

but Figs. 9 and 10 give logical extensions of Figs. 7 and 8 to these additional cases. For the present, it is recommended that the coped shear model of Fig. 7 be used for both coped and uncoped beams. However, the un-

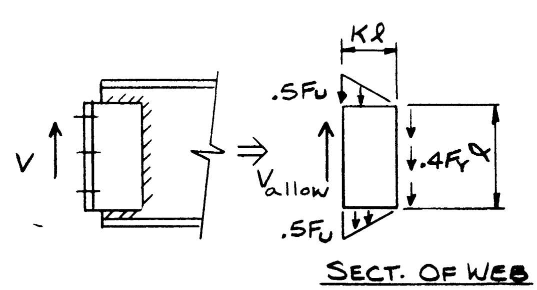

Figure 9

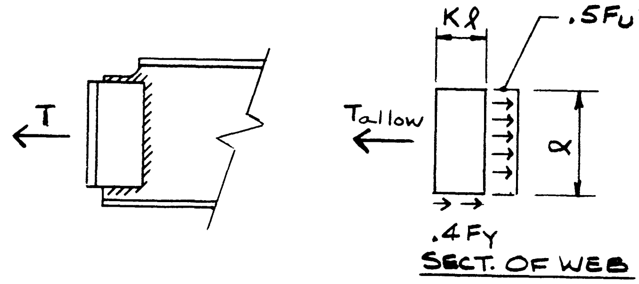

Figure 10

coped tension model of Fig. 8 may be unconservative if the beam is coped. Therefore, the coped tension model of Fig. 10 is recommended for coped beams.

Returning to the connection design problem, the required web thickness based on this alternate approach will now be determined. For shear alone, the required web thickness is:

$$t_{w_{reqd}} = \frac{V_{act}}{.25F_u kl + .4\,F_y l}$$

$$= \frac{36.4}{(.25)\,(58)\,(3.5) + (14.5)\,(11.5)}$$

$$= .167 \text{ in.} < .395 \text{ in.} \quad \textbf{o.k.}$$

For tension, the connection is subjected simultaneously to shear, so that an interaction expression is required, the simplest of which is a straight line interaction, as:

$$\frac{T}{T_{allow}} + \frac{V}{V_{allow}} \leq 1$$

Thus, we have:

$$\frac{15}{(.5)(58)(11.5)t_w + (14.5)(7)t_w} + \frac{35.4}{(.25)(58)(3.5)t_w + (14.5)(11.5)t_w} \leq 1$$

from which:

$$t_{w_{reqd}} \geq \frac{15}{435} + \frac{35.4}{217.5} = .197 \text{ in.} < .395 \text{ in.} \quad \textbf{o.k.}$$

The required web thickness by this method is thus .197 in., which can be compared to the .266 in. required by the "point stress" method. It can be seen the new alternate method requires a web only 74% as thick as the old point stress method. This is a significant reduction in required web thickness, which in many cases will remove the requirement for expensive web doubler plates which often results from using the point stress method. Note the 26% reduction in web thickness required was achieved here with the coped shear model. The actual beam is uncoped. Note also the reduction was achieved with the straight-line interaction equation, usually a conservative choice. With further research to eliminate these conservative assumptions, further reductions in required web thickness can be expected.

Before leaving the discussion of Weld **1**, it should be noted this weld could be replaced by shop bolts. In this case, the shear in the bolts will be checked for the 36.4 kips vertical load acting alone, and for the resultant .75 $[(36.4 + 10.8)^2 + (20)]^{1/2} = 38.4$ kips of the gravity and wind loads, 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, 36.4 kips or 38.4 kips, as a shear, i.e. vertical, load.

Angles aa—Field Bolted Connection

Let the field bolts used in this connection be ⅞-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 6b gives the geometry of the clip angle.

$b = 2.5625 - 0.5 = 2.0625$ in.
$b' = 2.0625 - (0.875/2) = 1.6250$ in.
$a = 4 - 2.5625$
$\quad = 1.4375 < 1.25b = 2.5781$ 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.
$\alpha = [(1.875 \times 1.625/3.375) - 1]/0.6875$
$\quad = -0.141$ (use $\alpha = 0$)
$B_c = 1.875\,(1 + 0) = 1.875$ kips
$V_b = 35.4/8 = 4.43$ kips/bolt
$B = 33.07 - 1.8V_b = 33.07 - (1.8 \times 4.43)$
$\quad = 25.10 < 26.5$ 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.

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. 11. (Note that the gusset plate

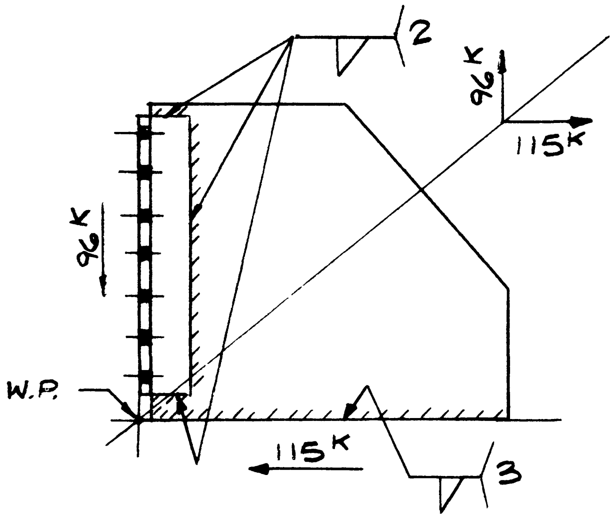

Figure 11

shown in Fig. 11 is in equilibrium under the loads and reactions shown. Note also that the entire 115^k horizontal force is assumed taken by the horizontal weld. This is because its stiffness relative to horizontal motion is much greater than that of the angles **ab**.) 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.

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 3/8-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 the AISC Specification Sect. 1.17.2.

For comparison, it is interesting to see what plate thickness would be required by the tear-out model of Fig. 7. Because the weld across the top of the connection angles may be too close to the top edge of the gusset plate to be effective, the coped top flange model will be used. Thus,

$$t_{Preqd} = \frac{72}{(.25 \times 58 \times 3 + .4 \times 36 \times 23.5)}$$
$$= .188 \text{ in.}$$

which is very similar to the .198 in. required by the simple point stress method.

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. 11 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. 11, 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 in. 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 1/8-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. 2.

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. 5. A free body diagram for this gusset plate is shown in Fig. 12. Because the working point is now at the beam center line, 11 3/16 in. below the top flange, the gusset is

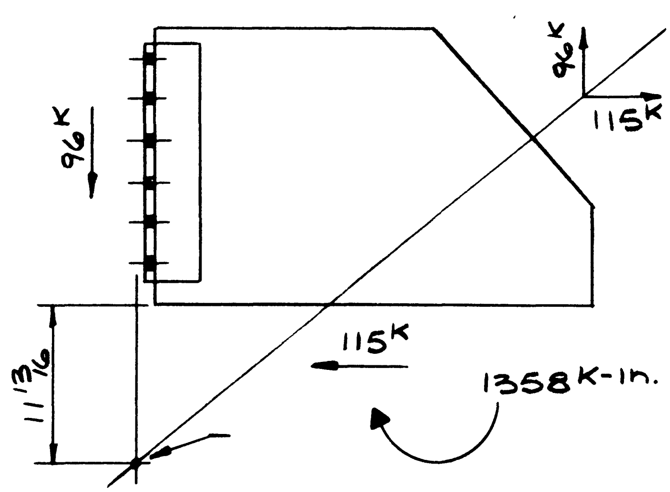

Figure 12

not in equilibrium unless a couple of magnitude 115 × 11$^{13}/_{16}$ = 1358 kip-in. is applied to 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:

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

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

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. 5 and 12 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. 13. 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,

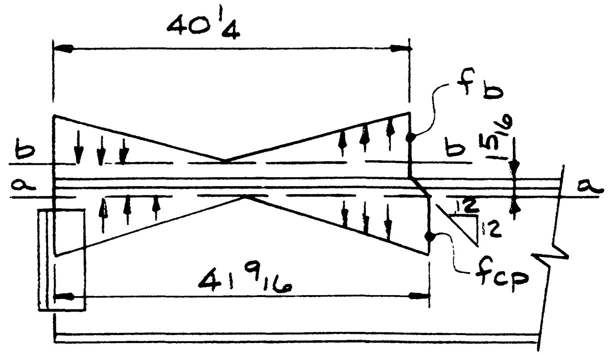

Figure 13

stress f_{cp} could be large enough to cause beam web crippling. In this case,

$$f_{cp} = \frac{6 \times 1358 \times 0.75}{0.395\ (41.5625)^2}$$

$$= 8.95 < 0.75\ F_y = 27 \text{ ksi}$$

Therefore, beam web crippling will not occur.

Except for the above discussion regarding Weld **3** and beam web crippling, the connection of Fig. 5 is designed in the same fashion as the connection of Fig 2.

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 14 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:

$$A_v = [10.75 - (3.5 \times 0.9375)] \times 0.375 \times 2$$
$$= 5.602 \text{ in.}^2$$

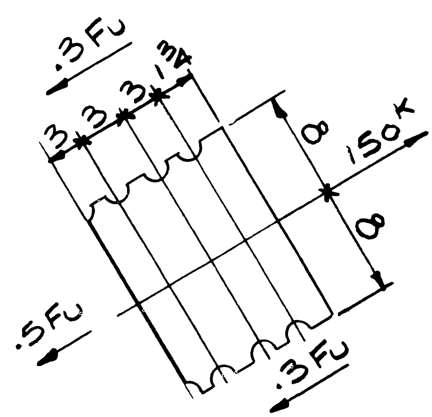

Figure 14

The net tension area is:

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

Thus, the allowable brace force is:

$$P_{allow} = (5.602 \times 0.3 \times 58) + (5.648 \times 0.5 \times 58)$$
$$= 261 > 0.75 \times 150 = 113 \text{ kips}$$

The gusset is satisfactory.

Gusset Plate Buckling:

The Whitmore Section (Ref. 5), shown in Fig. 2, 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, $f_a = 11.37$ 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 of 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:

$$\frac{Kl_1}{r} = \frac{0.65 \times 17}{0.108} = 102$$

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. 2, 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. 2, 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. 5. In this case using l_1 as the strip length may be unconservative.

CONNECTION OF BRACE TO GUSSET PLATE

Figures 15, 16 and 17 show the manner in which various types of braces can be connected to the gusset plate. The

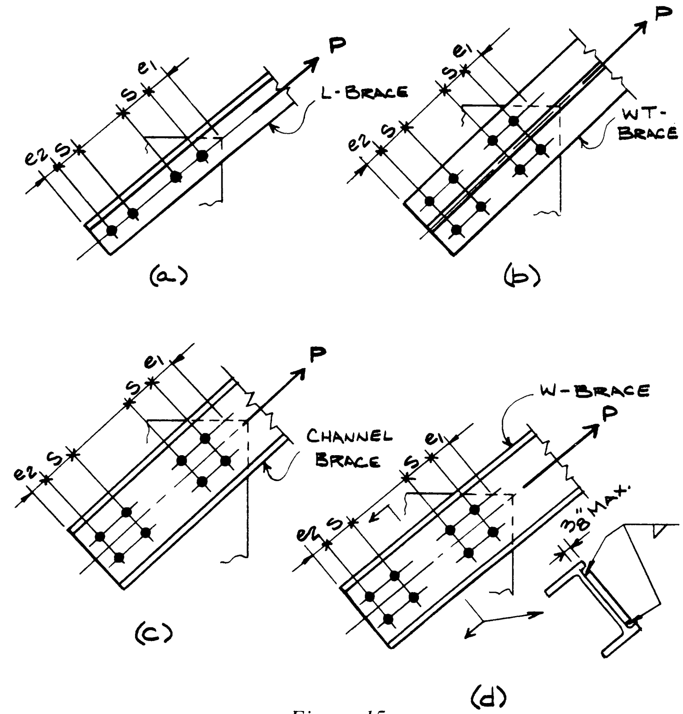

Figure 15

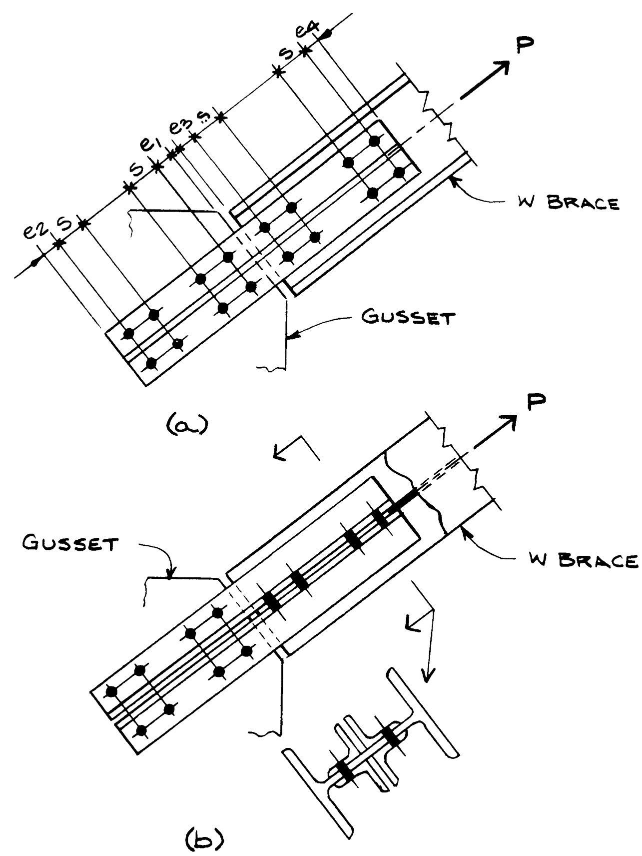

Figure 16

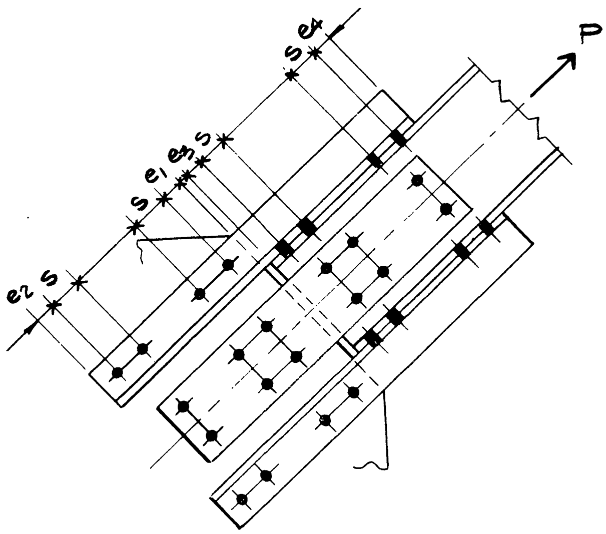

Figure 17

connection of Fig. 17 can also be used without the web connector plates. The connections of these figures are all designed in the same way. The detailed approach to those of Fig. 16 follows:

1. *Double shear in bolts.*
   $P_v = 2A_b F_v N \geq P$

2. *Bearing.* Let $t = \min(t_w, t_g, t_f)$, where t_w = W section web thickness, t_g = gusset thickness, t_f = WT flange or angle thickness.

   $P_p = 1.5\, F_u\, dt\, N \geq P$

3. *Spacing.*
   $P_s = .5\, F_u\, t(s - d/2)N \geq P$

4. *End distances.* Let $e' = \min\{e_2, e_4\}$
   $P_e = \min(.5\, F_u t_f e'N,\ .5\, F_u t_g e_1 N,\ .5\, F_u t_w e_3 N) \geq P$

5. *Gross and net sections.*

$$f_{gross} = \frac{P}{A_{gross}} \leq .6\, F_y$$

$$f_{net} = \frac{P}{A_e} \leq .5\, F_u$$

where $A_e = A_{net} \times C_t$. See AISC Specification Sect. 1.14.22 for definition of C_t.

6. *Brace web tear-out.* Similar to gusset plate tear-out discussed above.

SUMMARY

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 and to the relative stiffness of the various connection elements.

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.

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