Beam-to-Column Continuous Connections

1. INTRODUCTION

Welding is most efficient in structures designed for full continuity. This type of design builds into the structure the inherent strength which comes from continuous action of all members. Loads are easily redistributed when overloading occurs on certain members.

This type of design realizes a weight saving in the beams since a negative moment acts at the supports, thus reducing the positive moment at the center of the span by the same amount.

Continuous connections also take advantage of what used to be a 20% increase in the allowable bending stress in the negative moment region near the support. This is accomplished through a 10% increase in bending allowables for "compact" sections, and using a 10% reduction in the negative moment. This reduction in negative moment is allowed for "compact" sections, provided the section modulus here is not less than that required for the positive moments in the same beam and provided the compression flange is regarded as unsupported from the point of support to the point of contraflexure.

Examples of Continuous Connections

In Figure 1, a beam frames into the web of a column. A seat, made of a plate with a stiffener, is shop welded to the column. This stiffener carries the beam reaction. A pair of flange connecting plates are field welded to the column and the beam. By using two plates instead of one, fillet welds on the flange can be of greater length to transmit the required load.

In Figure 2, flange and web plates are shop welded into the column. Usually these plates are of the same thickness as the corresponding part of the beam which they connect. An additional plate, fastened to the lower flange plates, serves as a seat plate. The flanges of the beam are beveled for downhand groove welding in the field. The web plate laps the web of the beam and is connected by a fillet weld.

In Figure 3, the beam frames to the column flange. The erection seat, with stiffener, and the web connecting plate are shop welded to the column flange. Also, column flange stiffeners if needed are shop welded. A flange connecting plate is field welded in place, being groove welded to the column flange and fillet welded to the upper beam flange. Usually a backing strip is

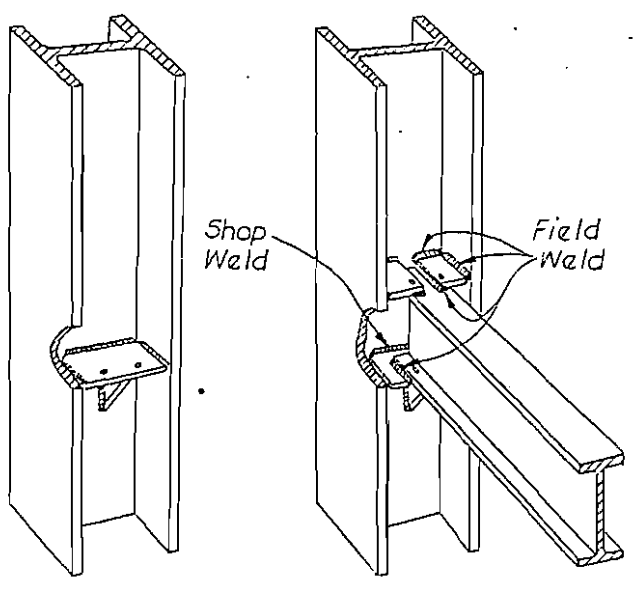


FIGURE 1

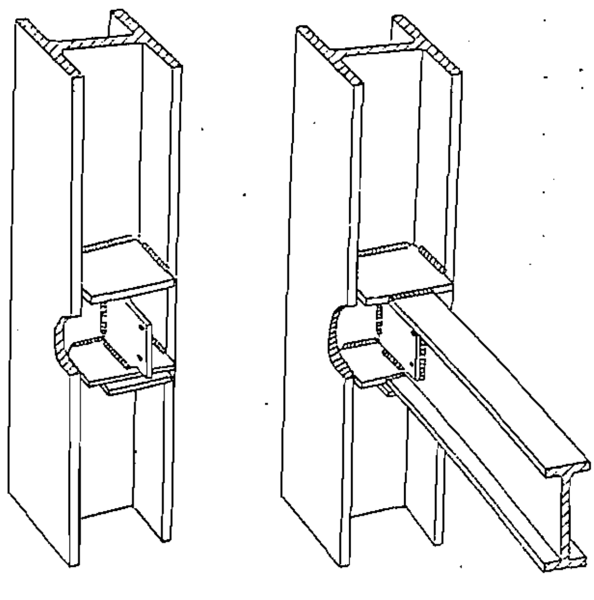


FIGURE 2

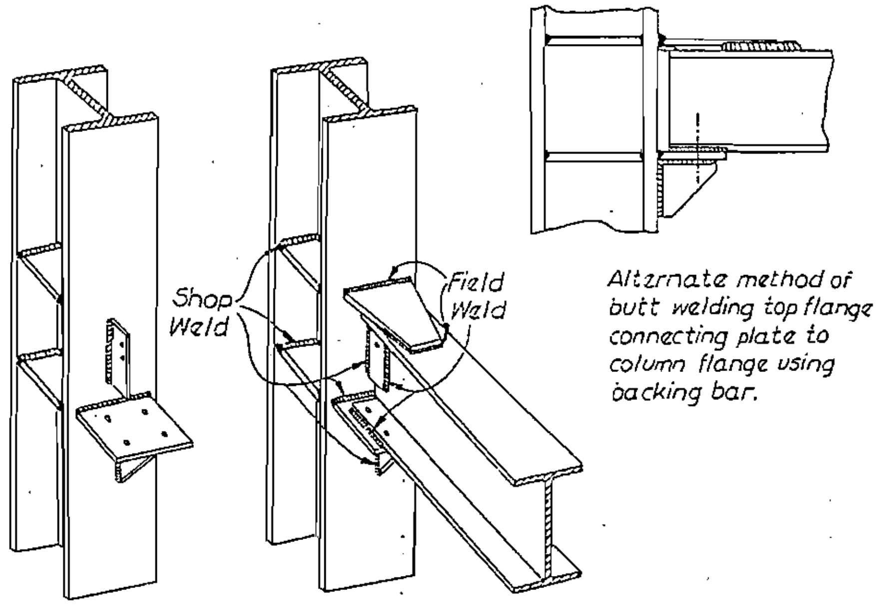


FIGURE 3

placed between the connecting plate and the beam flange to ensure a complete-penetration groove weld to the column. This eliminates back gouging and welding an overhead pass on the other side.

Reducing Welding Requirements

It is possible to design the seat stiffener to carry all of the end reaction, eliminating any vertical welding in the field. This reduces the field welding to just downhand groove welding of the beam flanges to the column.

Where good fit-up can be assured, the beam flanges are beveled from the top side and groove welded in the field directly to the column flange. The beam web is cut back about 1″ and fillet welded to the web connecting plate.

Some fabricating shops have jigs so that columns can be elevated into a vertical position. This allows much of the shop welding on the connecting plates to be made in the downhand position.

Cover Plates

When added at ends of beams to carry the extra negative moment, cover plates must be welded to the column for continuity; Figure 4.

Shop welding the cover plates to the beam, with the lower beam flange and the upper cover plate left

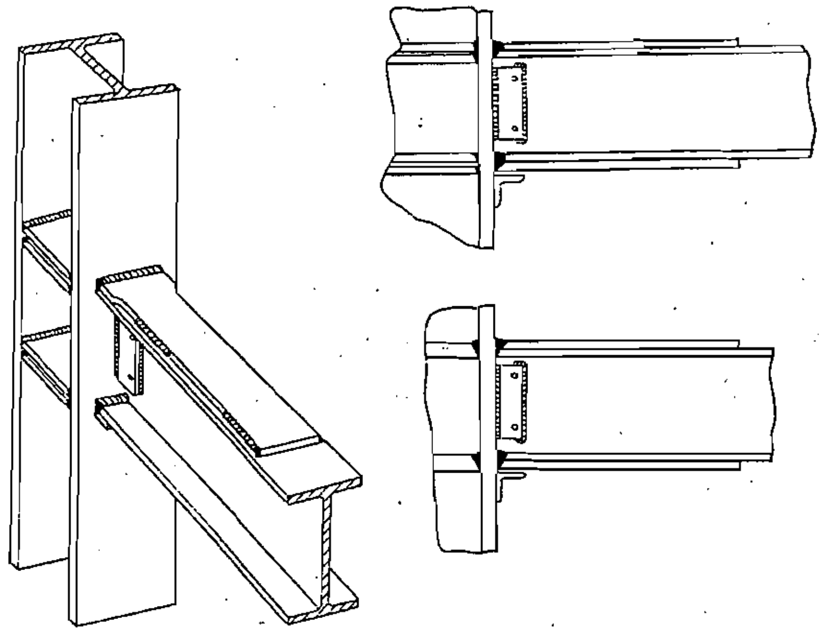


FIGURE 4

unbeveled, produces a type of "J" groove for the weld connecting them to the column flange.

If column-flange stiffener plates are needed in this case, they should be of about the same thickness as the beam flange and cover plate combined. The usual single thick stiffener in line with each beam flange can be replaced with two plates, each having half the required thickness. This means working with lighter connecting material and using two groove welds, each being half the size of the original single groove weld, which reduces the amount of welding on the stiffeners by half.

2. ANALYZING NEED FOR COLUMN STIFFENERS

If the flange of the supporting column is too flexible, the forces transmitted by the connecting flanges will load the outstanding portion of the column flange as a cantilever beam and cause it to deflect slightly; Figure 5. As this deflection takes place it reduces the stress in the outer ends of the beam-to-column connecting weld, thereby loading up the center portion of the weld in line with the column web.

It was previously thought that unless the column flange is extremely rigid (thick), flange stiffeners must be added to the column in line with the beam's top and bottom flanges (or their connecting plates). Such stiffeners keep the column flange from deflecting and load the weld uniformly.

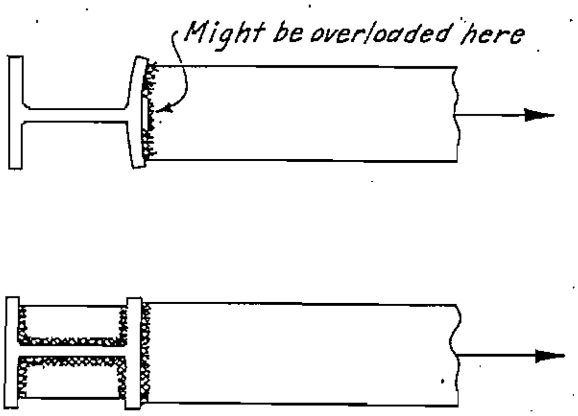


FIGURE 5

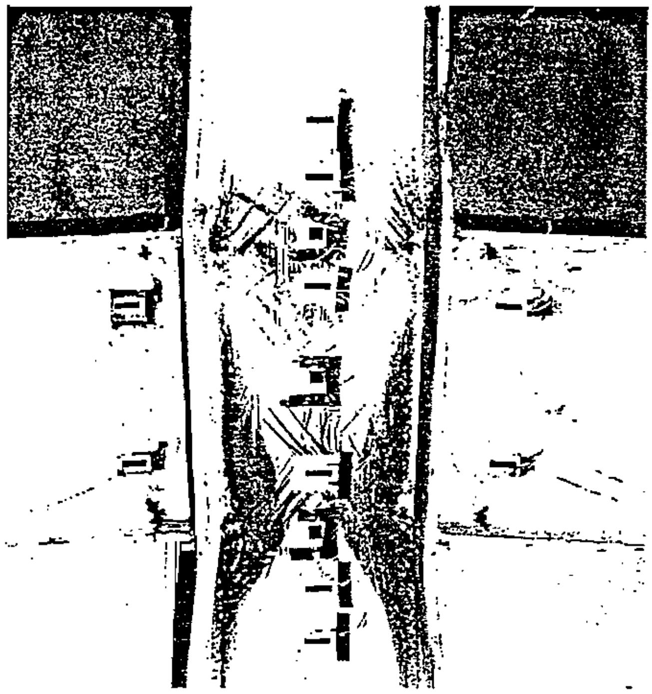


FIGURE 6

However, recent research at Lehigb University indicates that in most cases the deciding factor is a crippling of the column web; Figure 6. If the column web is thick enough, stiffeners are not required.

Buckling of Column Web Due to Compressive Force of Lower Beam Flange

A test was set up, Figure 7, to evaluate effects of the lower flange of the beam in compression against the column. Two bars, one on each side of the column, represented the cross-section of the beam flange. The test member was placed in a testing machine and loaded under compression.

In all cases, yielding began in the fillet of the

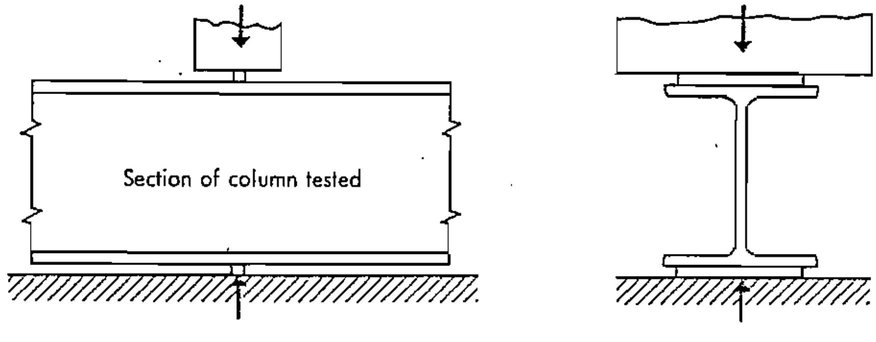


FIGURE 7

column just inside the column flange, and directly beneath the bars. Yielding progressed into the column web by means of lines radiating from this point to the column "K" line, at a maximum slope of 1 to 2½. This progressed for some distance. A slight bending of the column flanges was noticed at about 80% of the failure load. Figure 8 shows an analysis of this.

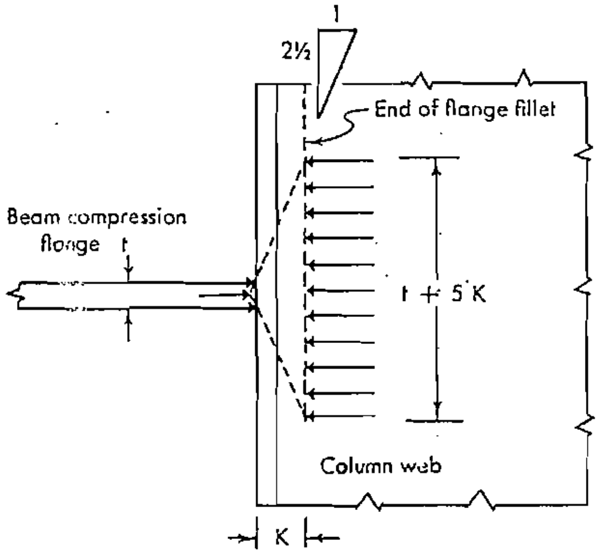


FIGURE 8

Overloading of Column Flange Due to Tension Force of Upper Beam Flange

A test was set up, Figure 9, to evaluate effects of the upper flange of the beam in tension against the column. Two plates, one on each side of the column and welded to it, represented the cross-section of the beam flange. The member was pulled in a tensile testing machine.

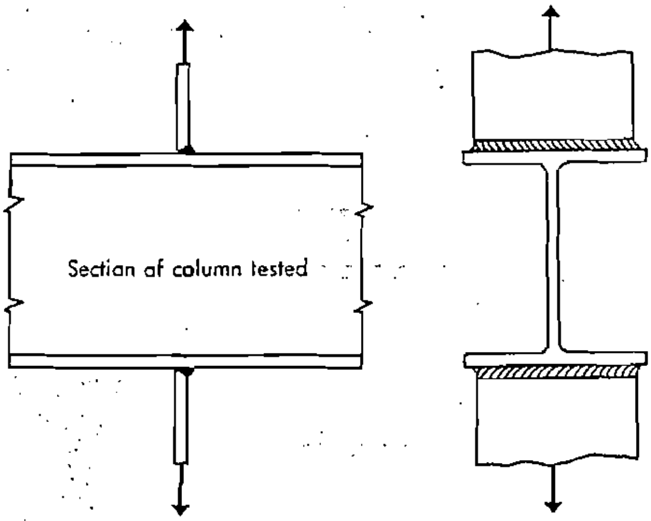


FIGURE 9

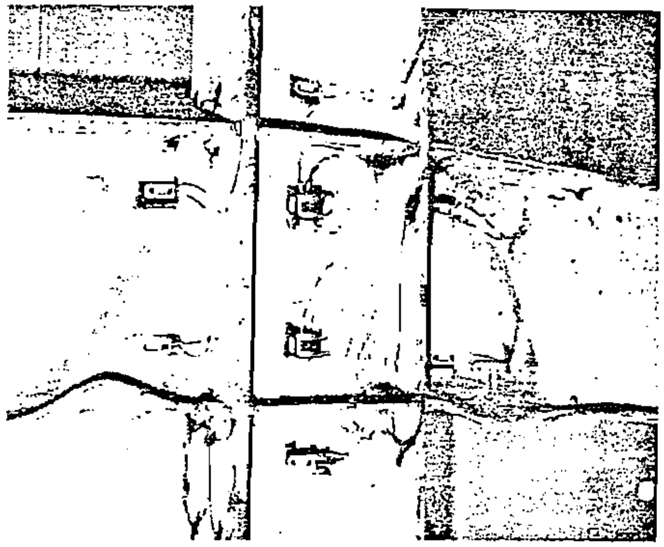


FIGURE 10

Dimensions of both the column flange and the connecting plates were varied in order to study the effect of different combinations of columns and beams.

First yielding was noticed in the fillet of the column just inside the column flange, and directly beneath the attaching plates, at about 40% of the ultimate load. With further loading, yielding proceeded into the column web, underneath the column flange parallel to the attaching plate, and into the column flange from the center of the connecting welds, and parallel to the column web. After ultimate loading, some members failed by cracking of the central portion of the connecting weld directly over the column web, some by cracking in the inside fillet of the column, and some by cracking in the inside fillet of the column, and some by a tearing out of material in the column flange.

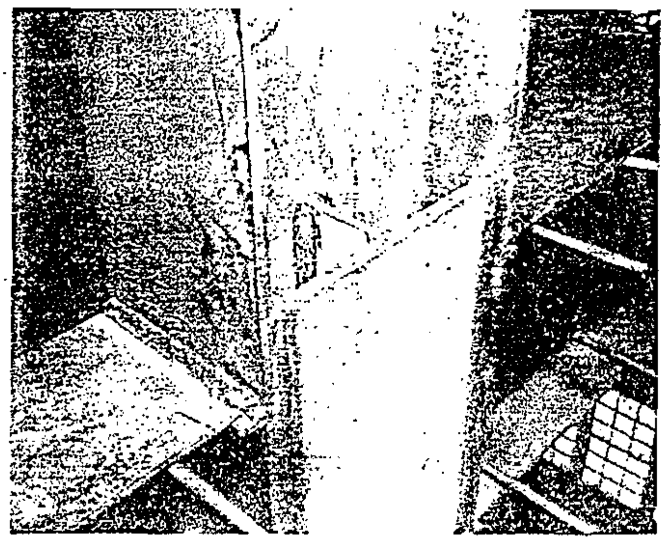


FIGURE 11

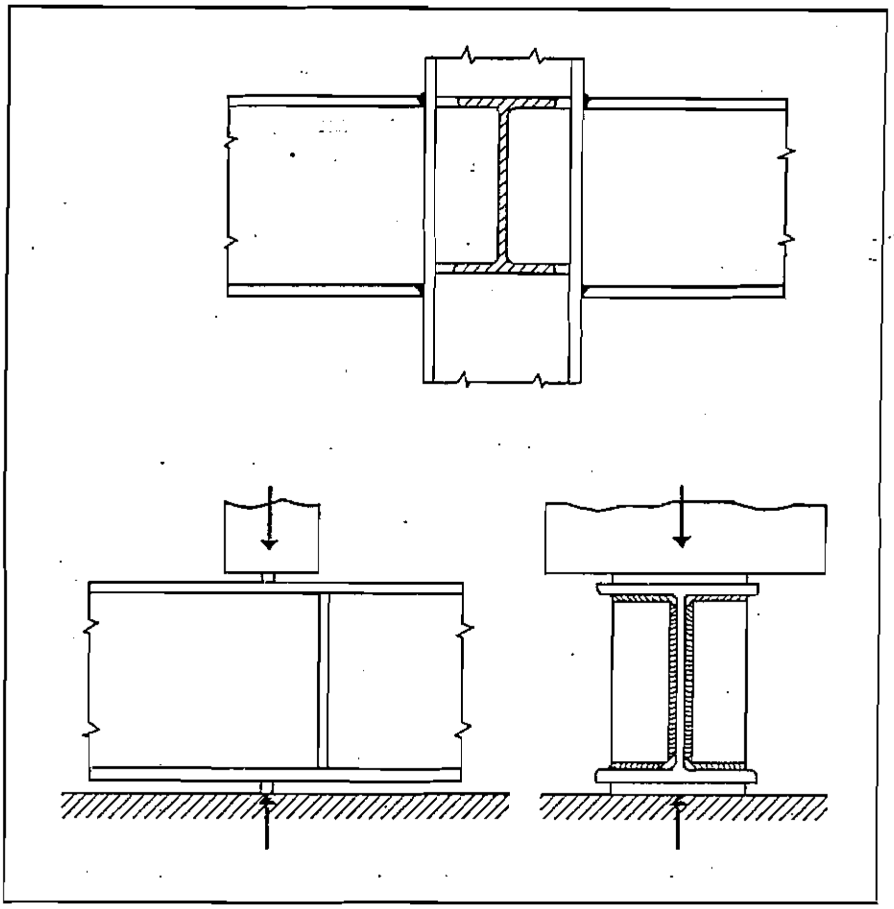


FIGURE 12

Standard Stiffeners

When some type of web stiffening is required, the standard horizontal flange stiffeners are an efficient way to stiffen the column web. Figure 10 shows this type under test.

A Tee section flame cut from a standard wide-flange section may be used for stiffening, Figure 11. The stem of the Tee section is welded to the column web for a short distance in from the ends. This could be entirely shop welded, all of it being done in the flat position, possibly using a semi-automatic welder. This type stiffener would have numerous advantages in four-way beam connections. The beams normally framing into the column web would now butt against this flat surface with good accessibility. The flanges of the beam could be beveled 45° and then easily groove welded in the field to this surface, using backing straps. There would be no other connecting or attaching plates to be used. In effect this part of the connection would be identical to the connection used for beams framing to column flanges.

See Figures 28, 29 and 30 and related text for specifications of stiffeners applicable to elastic design.

Effect of Eccentric Stiffeners

In a four-way beam-to-column connection, the column flanges may be stiffened by the connecting plates of the beam framing into the column web. It may be that the beam framing to the column flange is of a different depth. This in effect will provide eccentric stiffeners, Figure 12.

The lower part of Figure 12 shows how this was tested. It was found that an eccentricity of 2″ provided only about 65% of the stiffening provided by concentric stiffeners, and an eccentricity of 4″ provided less than 20%.

Three methods of framing beams of different depths on opposite flanges of columns are shown in Figure 13.

3. TEST COMPARISON OF STIFFENER TYPES

The following is adapted from "Welded Interior Beam-To-Column Connections", AISC 1959, which summarized tests on various connections.

Figure 14 represents a direct beam-to-column connection. Here the column has no stiffening and is not as stiff against rotation as the 16″ WF 36# beams which frame to the column.

This arrangement showed high stress concentrations at the center of the beam tension flanges, and therefore at the center of the connecting groove weld.

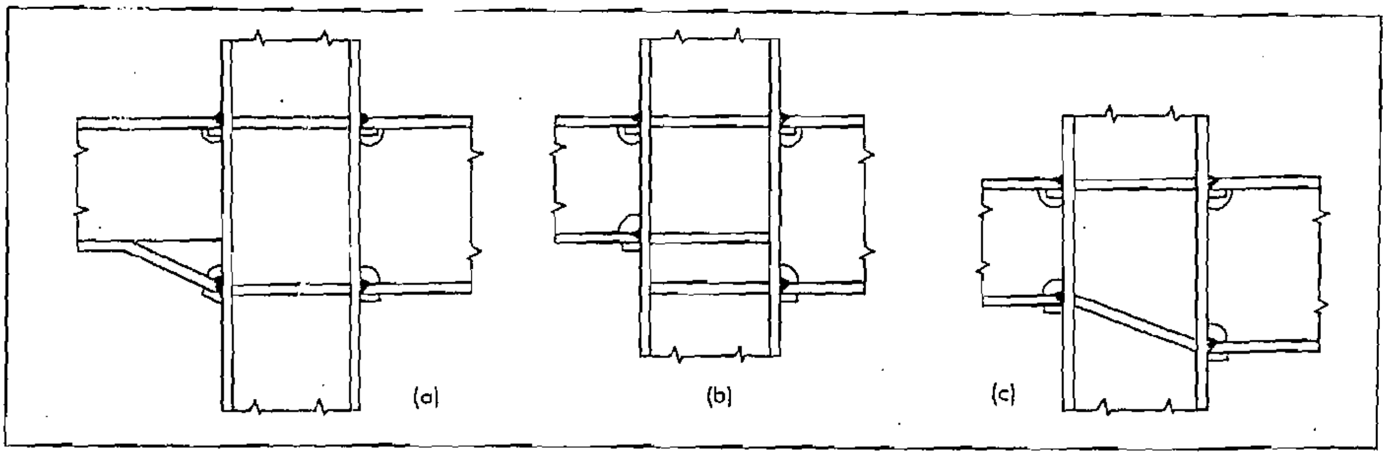


FIGURE 13

However, it was noted that no weld failures occurred until after excessive rotation had taken place.

The stiffeners here in Figure 15 provide the equivalent of beam flanges to the columns, and the columns become as stiff against rotation as the beams framing to the column.

The stress distribution on the compression flanges were uniform on the whole, while in the tension areas the stresses were somewhat higher in the center.

In Figure 16 the column is shown stiffened by a pair of wide-flange Tee sections. As a result the columns are as stiff against rotation as the beams framing into the columns.

From strain gage readings it was calculated that each of the vertical plate stiffeners in the elastic range transmitted only about $3/16$ of the forces coming from the beam flanges and the column web transmitted $5/8$ of the forces.

Placing these stiffener plates closer to the column web might have improved the distribution. However, since the prime purpose of this type of connection is to afford a convenient four-way connection, the plate usually needs to be positioned flush with the edge of the column flange.

The stress distribution was uniform in both flanges at the working load. At 1.5 of the working load, high

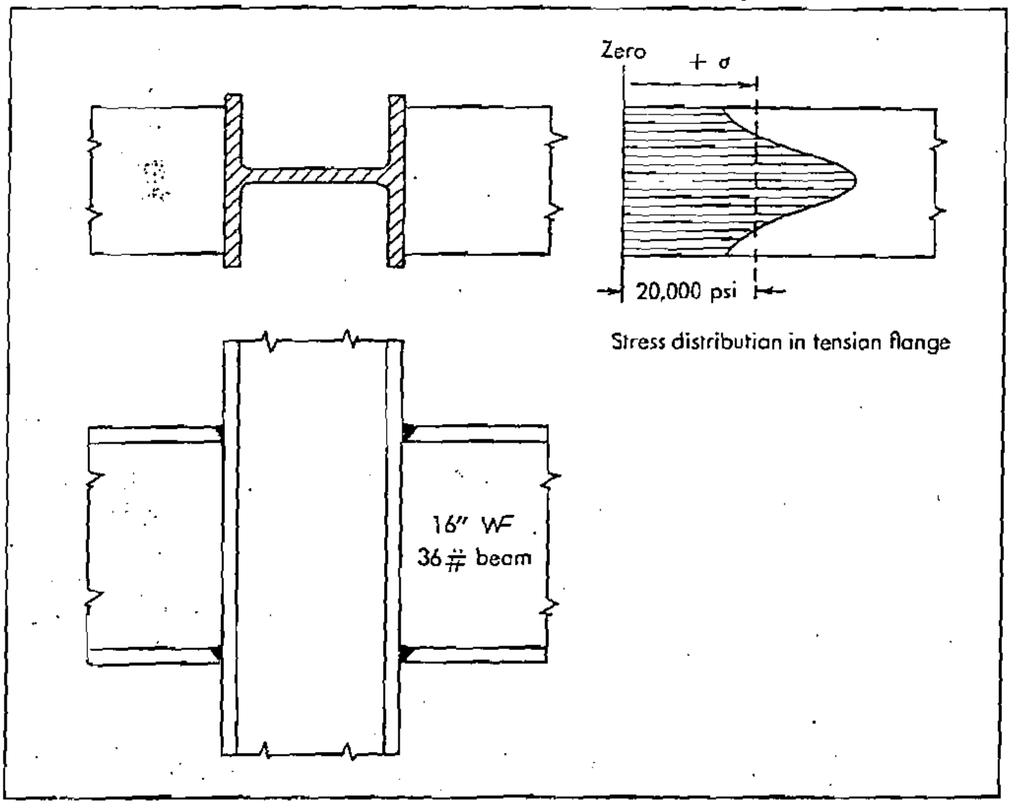


FIGURE 14

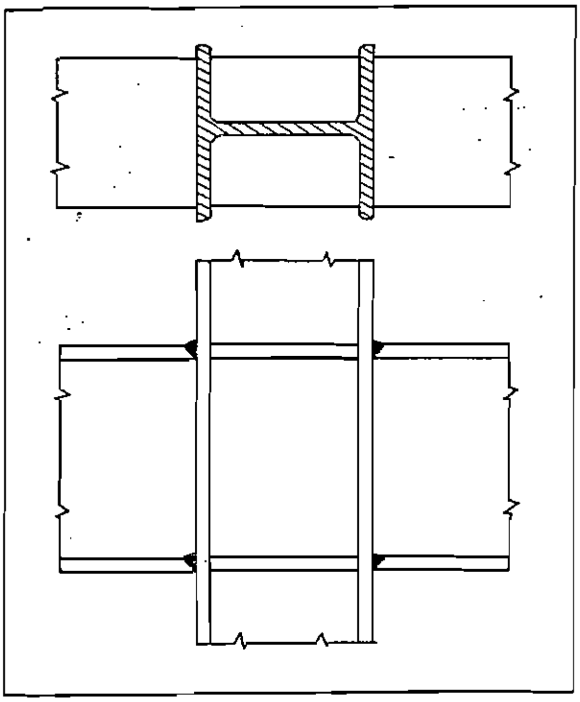


FIGURE 15

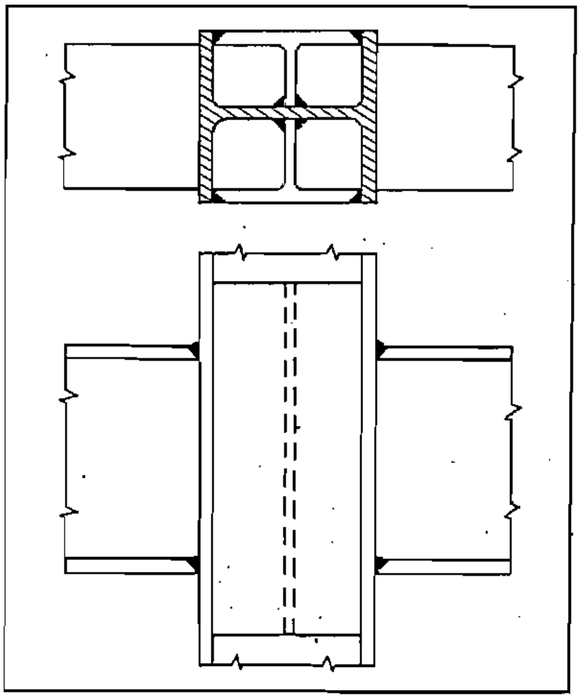


FIGURE 16

tensile stresses occurred at midflange.

The connection in Figure 17 was stronger than its two-way counterpart. This evidently shows that the stiffening action provided by two beams framing into the column web strengthens the connection more than it is weakened by the triaxial stresses.

The connections of Figure 18 involving (East-West) beams welded directly to the column flanges proved stiffer than the connection of (North-South) beams to the Tee stiffeners.

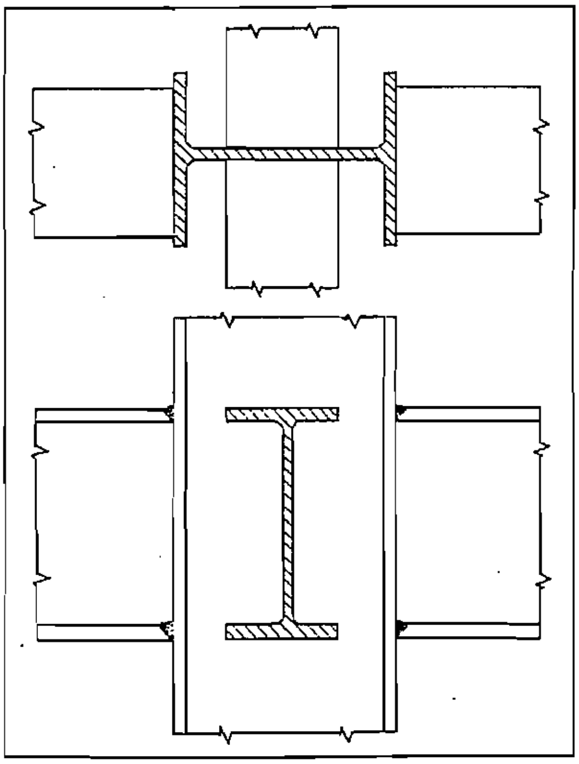


FIGURE 17

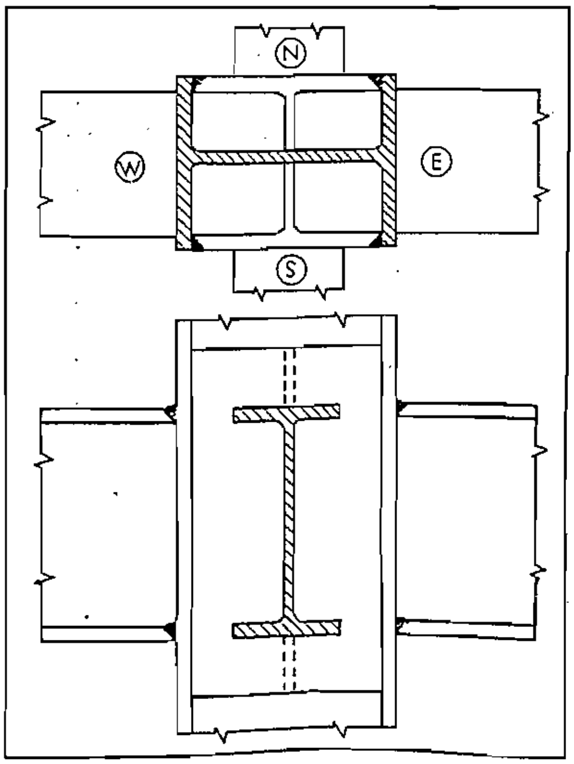


FIGURE 18

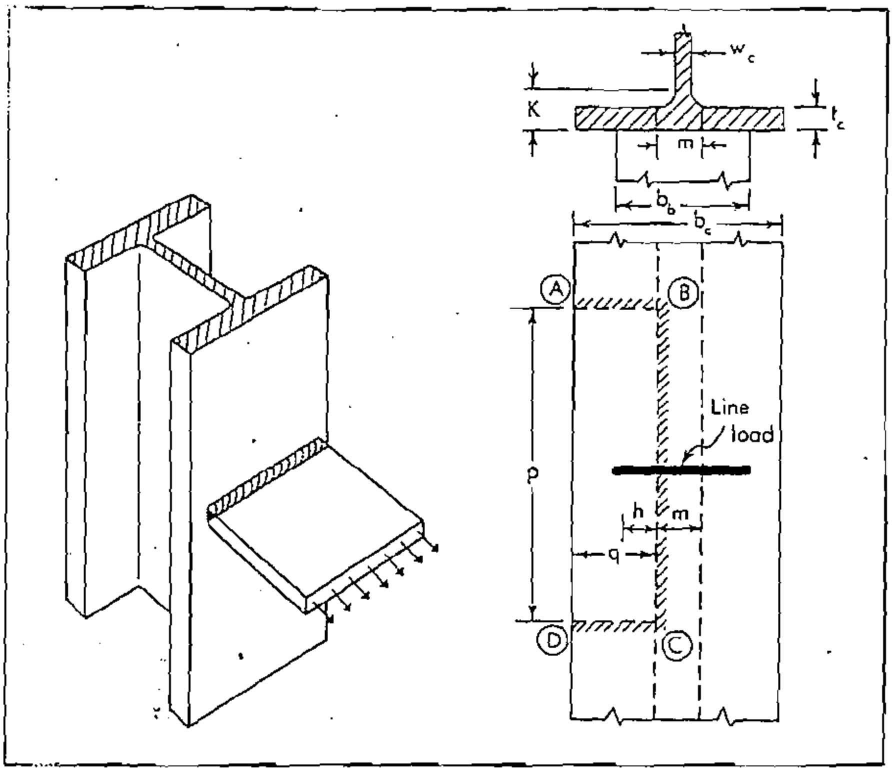


FIGURE 19

The stiffening of the latter connection is mainly dependent on the thickness of the stem of the Tee stiffener, the flanges of the column being too far away to offer much resistance.

The column web is ably assisted in preventing rotation at the connection by the flanges of the split-beam Tee stiffeners.

4. ANALYSIS OF STIFFENER REQUIREMENTS IN TENSION REGION OF CONNECTION (Elastic Design)

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

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.

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—

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

where:

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

Let:

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

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

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

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:

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

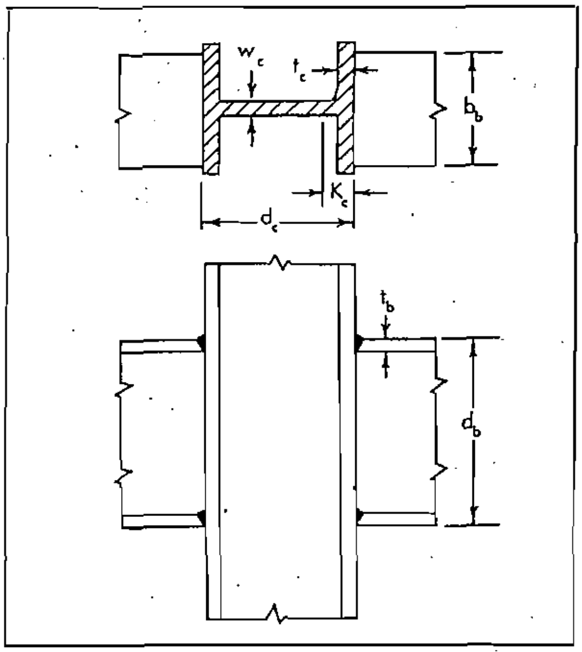


FIGURE 20

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

$$\boxed{t_c \geqq 0.4\, \sqrt{A_f}}$$

If the thickness of the column flange (t_c) meets the above requirement, column stiffeners are not needed in line with the tension flanges of the beam.

If the actual thickness of the column flange (t_c) is less than this value, stiffeners are needed.

5. ANALYSIS OF STIFFENER REQUIREMENTS IN COMPRESSION REGION OF CONNECTION (Elastic Design)

It is assumed the concentrated compression force from the beam flange spreads out into the column web at a slope of 1 in 2½ until it reaches the K line or web toe of the fillet; see Figure 8.

Equating the resisting force of the column web to the applied force of the beam flange, assuming yield stress—

$$w_c\ (t_b + 5\ K_c)\ \sigma_y \geqq A_f\ \sigma_y \qquad \text{or}$$

$$\boxed{w_c \geqq \frac{A_f}{t_b + 5\ K_c}}$$

If the thickness of the column web (w_c) meets the above requirement, column stiffeners are not needed in line with the compression flanges of the beam.

If the actual thickness of the column web (w_c) is less than this value, the web must be stiffened in some manner.

6. HORIZONTAL STIFFENERS

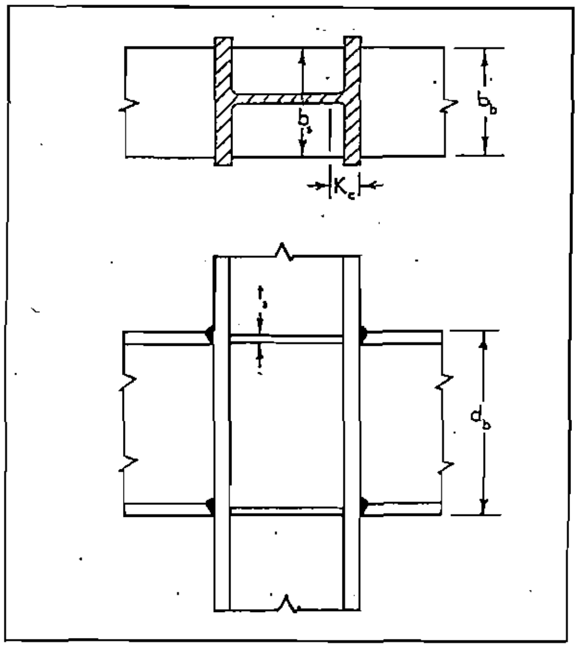


FIGURE 21

Equating the resisting force of the column web and a pair of horizontal plate stiffeners to the applied force of the beam flange at yield stress—

$$w_c\ (t_b + 5\ K_c)\ \sigma_y + A_s\ \sigma_y \geqq A_f\ \sigma_y \qquad \text{or}$$

$$\boxed{A_s \geqq A_f - w_c\ (t_b + 5\ K_c)}$$

where:

A_s = total cross-sectional area of pair of stiffeners

To prevent buckling of the stiffener—

$$\boxed{\frac{b_s}{t_s} \leqq 16}$$

where:

b_s = total width of pair of stiffeners

If the stiffener is displaced not more than 2″ from alignment with the adjacent beam flange (as in Fig. 12), it may still be used if considered about 60% as effective as when in direct line. The stiffener thickness (t_s) found from the above formula should then be multiplied by 1.70 to give the actual required value.

7. VERTICAL STIFFENERS

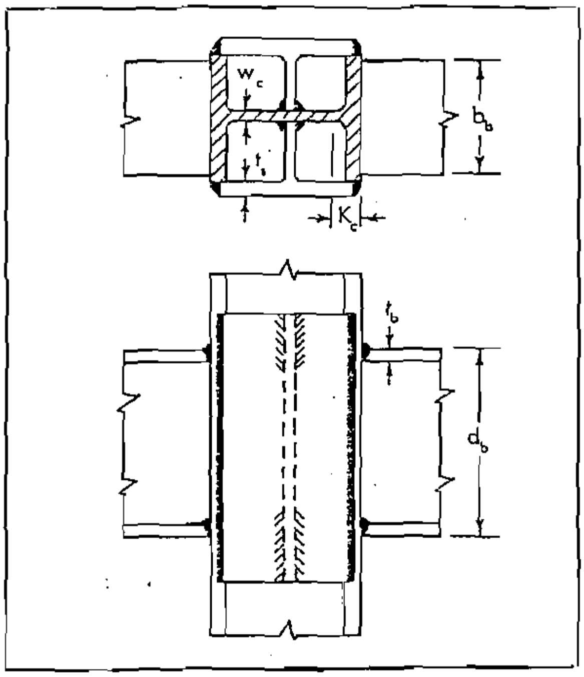


FIGURE 22

Because the vertical stiffeners (usually Tees) are placed at the outer edges of the column flange, they are assumed to be half as effective as though placed near the column web. It is assumed the concentrated beam flange force spreads out into the vertical stiffener in the same manner as the column web.

Equating the resisting force of the column web and a pair of vertical Tee stiffeners to the applied force of the beam flange at yield stress—

$$w_c\ (t_b + 5\ K_c)\ \sigma_y + 2 \times \tfrac{1}{2}\ t_s\ (t_b + 5\ K_c)\ \sigma_y = A_f\ \sigma_y \quad \text{or}$$

$$\boxed{t_s \geqq \frac{A_f}{t_b + 5\ K_c} - w_c}$$

To prevent buckling of the stiffener—

$$\boxed{\frac{d_c}{t_s} \geqq 30}$$

Problem 1

As an example of applying the preceding analysis of the tension region of a connection, we will analyze a connection which, when tested to failure, performed well; see Figure 23.

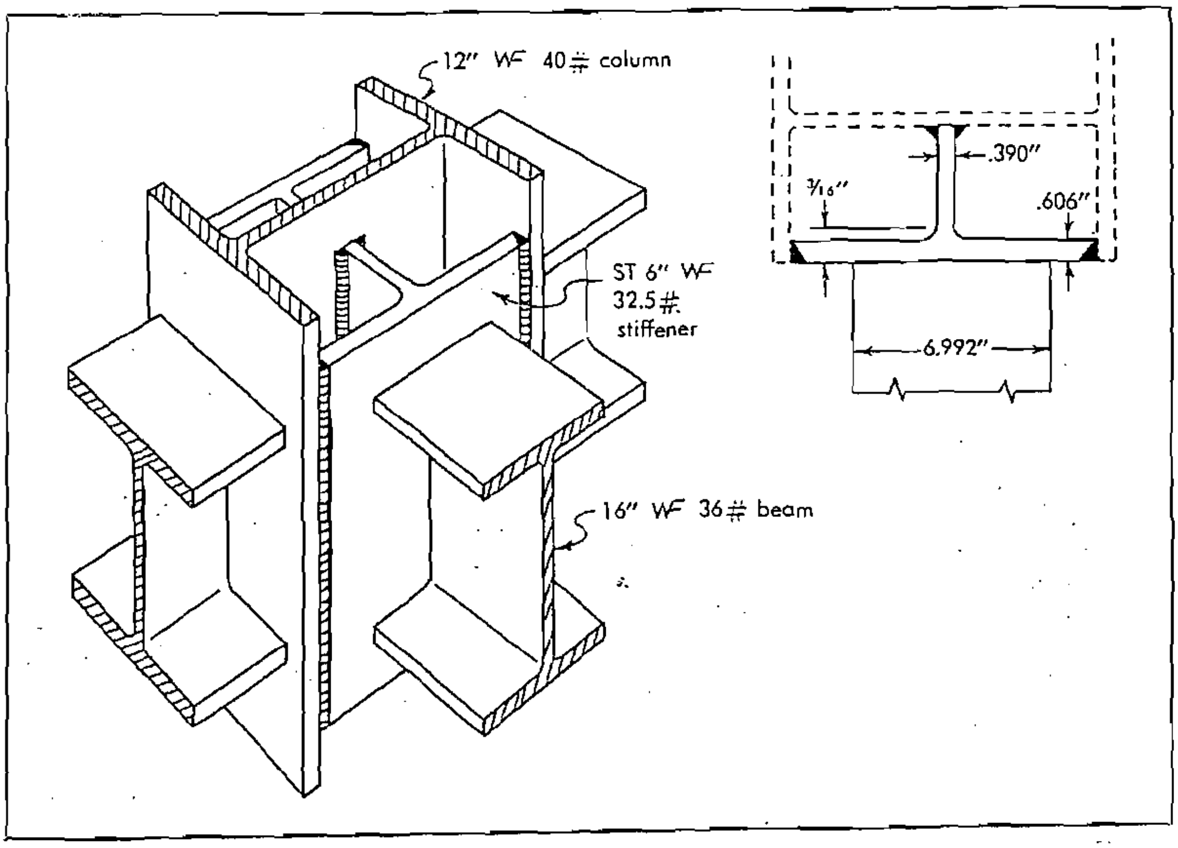


FIGURE 23

where:

$$m = w_c + 2\ (K - t_c)$$
$$= (.390) + 2\ [(1\ {}^3\!/_{16}) - (.606)]$$
$$= 1.553''$$
$$q = \frac{b_c - m}{2}$$
$$= \frac{(10.92) - (1.55)}{2}$$
$$= 4.69''$$
$$h = \frac{b_b - m}{2}$$
$$= \frac{(6.99) - (1.55)}{2}$$
$$= 2.72''$$
$$p = 12\ t_c$$
$$= 12\ (.606)$$
$$= 7.27''$$

Since:

$$\lambda = \frac{h}{q}$$
$$= \frac{(2.72)}{(4.69)}$$
$$= .58$$
$$\beta = \frac{p}{q}$$
$$= \frac{(7.27)}{(4.69)}$$
$$= 1.55$$

and:

$$\eta = \frac{\beta}{4}\left[\sqrt{\beta^2 + 8\ \lambda} - \beta\right]$$
$$= \frac{(1.55)}{4}\left[\sqrt{(1.55)^2 + 8\ (.58)} - (1.55)\right]$$
$$= .387$$
$$\therefore\ c_1 = \frac{\frac{4}{\beta} + \frac{\beta}{\eta}}{2 - \frac{\eta}{\lambda}}$$
$$= \frac{\frac{4}{(1.55)} + \frac{(1.55)}{(.387)}}{2 - \frac{(.387)}{(.58)}}$$
$$= 4.94$$

The total force which can be carried by the tension region of the column stiffener's flange must equal or exceed the force of the beam's tension flange, or:

$$\sigma_y\ t_b\ m + 2\ c_1\ \sigma_y\ t_c^2 \geqq \sigma_y\ A_f$$

Provided both column stiffener and beam have same yield strength:

$$(.428)(1.553) + 2\ (4.94)(.606)^2 \geqq (6.992)(.428)$$
$$4.28 \geqq 3.00 \qquad \text{O.K.}$$

If we used the conservative formula:

$$t_s \geqq 0.4\ \sqrt{A_f}$$
$$\geqq .40\ \sqrt{(6.992)(.428)}$$
$$\geqq .692''$$

but the initial design called for $t_c = .606''$ and the connection tested O.K.

8. CONNECTIONS THROUGH VERTICAL TEE STIFFENERS

Tests have shown that when the beam flange extends the full width of the connecting plate, Figure 24, about ⅝ of the flange force is carried by the central portion of the plate. Each of the two outer edges carry about $\frac{3}{16}$ of this force.

Figure 25 comes from test data of Lehigh University. Notice in the East-West beams, the flange of which extends almost the full width of the column

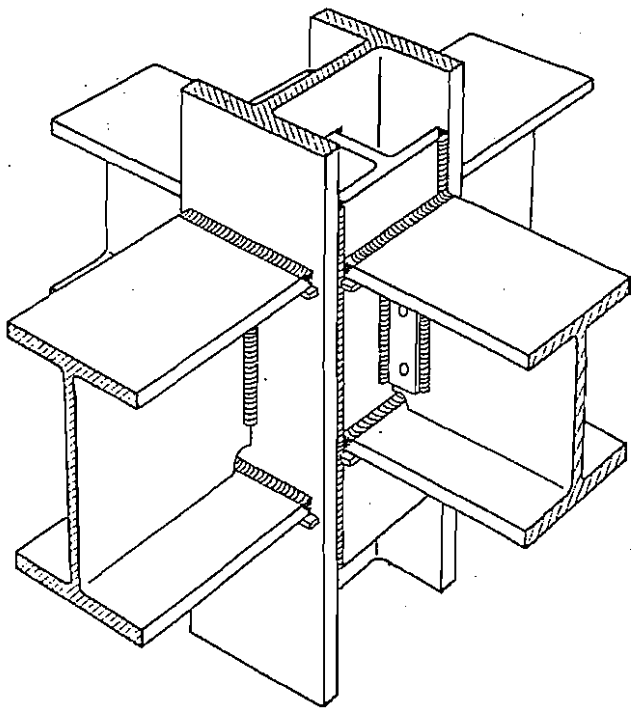


FIGURE 24

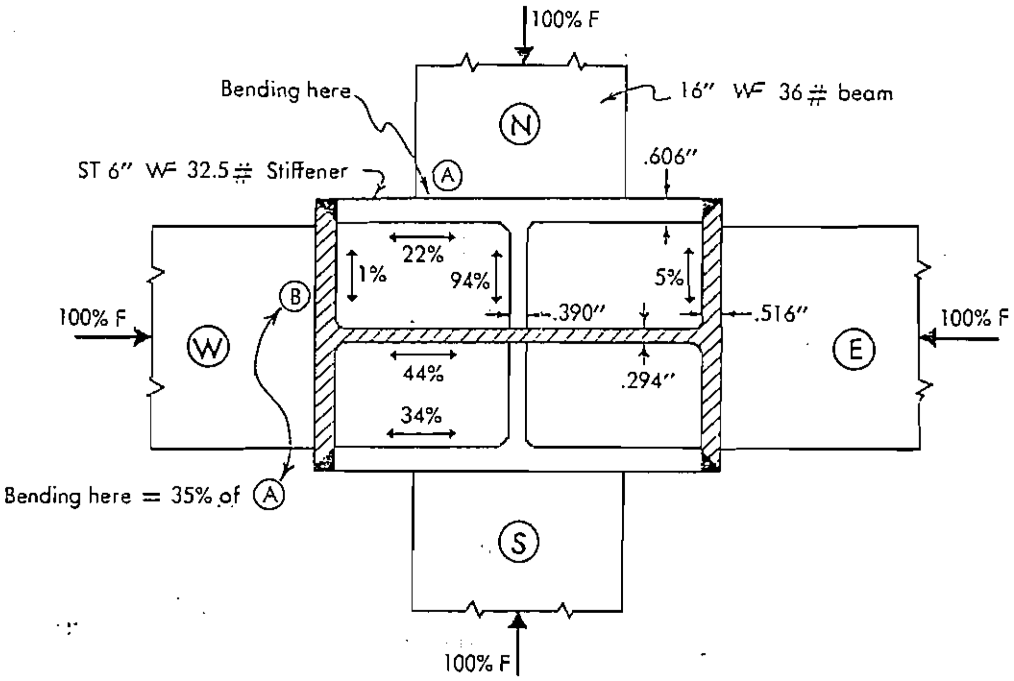


FIGURE 25

flange, 44% of the force is transferred through the web of the connection even though it is only about half as thick as the stiffener plates. This corresponds well with the idea that the flange of the column in this region is similar to a two-span beam on three supports with a uniform load; in this case the center reaction is ⅝ of the total load, and the two outer supports each carry $\frac{3}{16}$ of the load.

The report "Welded Interior Beam-To-Column Connections", AISC 1959, mentions that "from strain gage readings it was calculated that the vertical plate stiffeners in the elastic range each transmitted only about $\frac{3}{16}$ths of the forces coming from the beam flanges and the web transmitted ⅝ths."

Of course, the same would not be true in the North-South beams because they do not extend the full width of the flange of the Tee stiffener. As a result, most of this force must be transferred into the web or stem of the Tee stiffener since any portion of this force reaching the outer edges of the column flange must be transferred as bending out along the flange of the Tee section.

Weld Size: Stiffener Stem to Column Web

On the basis of these tests at Lehigh University, on connections where the beam flange extends the full

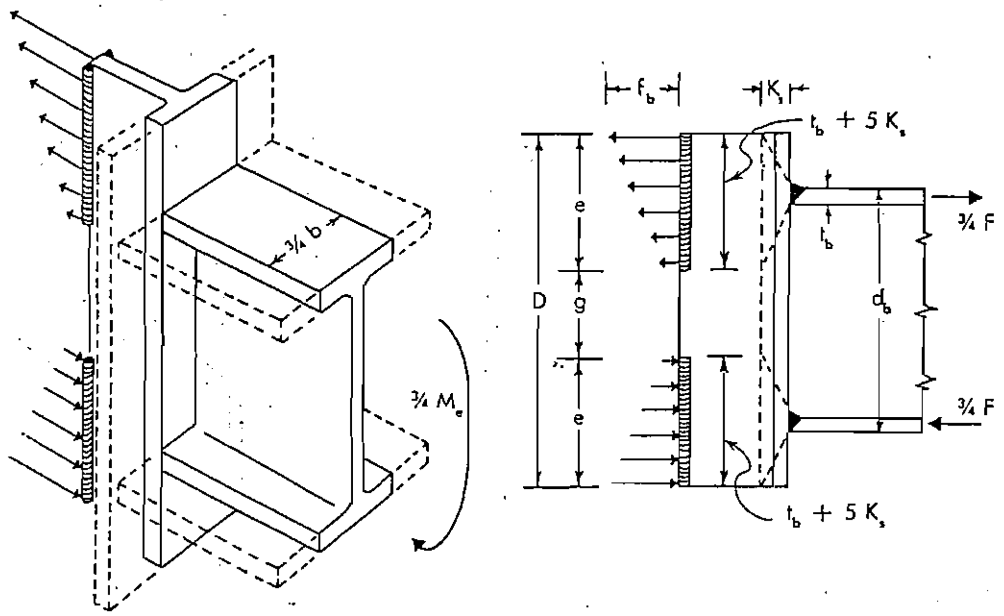


FIGURE 26

width of the stiffener flange, we will assume that ¾ of the beam flange force is carried by the stem portion of the connection. See Figure 26.

Because of the stiffening effect of the beam web and the stem of the connecting plate, this central (stem) portion of the connection will load up in bending. This assumes it rotates as a unit about a point at mid-height. The bending force on the weld is zero at this neutral axis and increases linearly to a maximum value at the upper and lower edges of the connection.

Treating the weld group as a line, the section modulus is equal to—

$$S_w = \frac{(D^3 - g^3)}{3\ D}$$

The resulting maximum unit bending force at the top portion of the weld on the stem is—

$$f_b = \frac{M}{S_w} = \frac{\tfrac{3}{4}\ M\ D\ 3}{(D^3 - g^3)}$$

The leg size of this weld would be found by dividing this value by the allowable for the particular weld metal.

A7, A373 Steel; E60 Welds
$f = 9600\ \omega$
A36, A441 Steel; E70 Welds
$f = 11{,}200\ \omega$

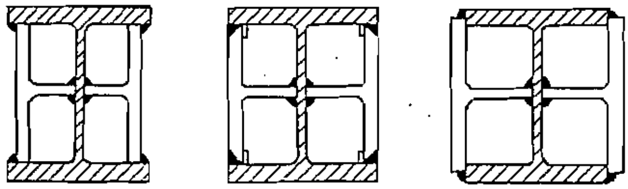
Here:

$$\boxed{D = d_b + 5\ K_s}$$

$$\boxed{g = d_b - 2\ t_b - 5\ K_s}$$

Weld Size: Stiffener Flange to Column Flange

The Tee stiffeners may be joined to the column flanges by a) fillet welds, b) groove welds, or c) corner welds. The groove welds (b) were used in the Lehigh Research of this connection.



(a) (b) (c)

FIGURE 27

Since tests on full-width flanges showed that the two outer edges of the connection carry about $\frac{3}{16}$ of the flange force, we will assume that each outer weld must carry ⅓ of the flange force. See Figure 28.

These welds will be pulled with an axial force of ⅓ F. We may assume the same distribution of force through the connecting plate at a slope of 1 to 2½ into the connecting welds. This will provide an effective length of weld of $t_b + 5\ t_s$ to carry this force.

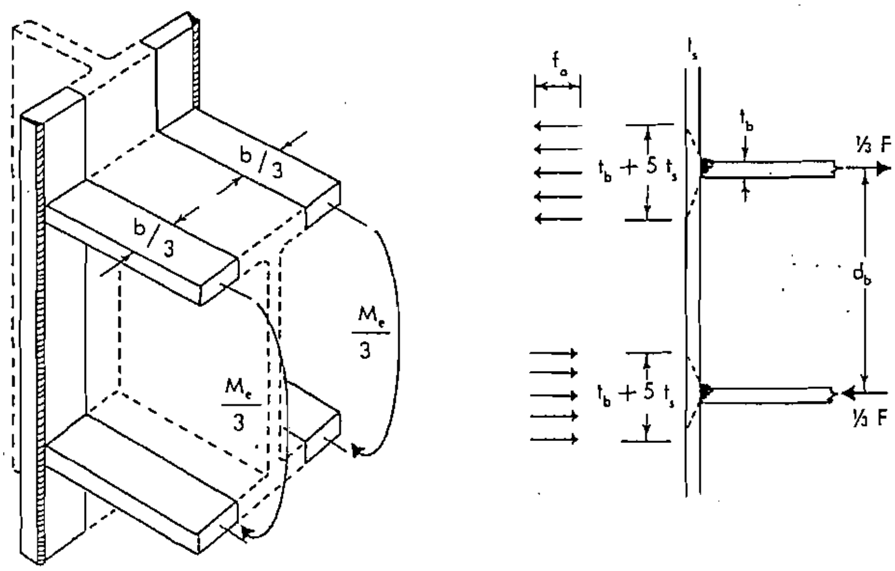
The unit force on this weld is—

$$f_a = \frac{M_e}{3(d_b - t_b)(t_b + 5\ t_s)}$$

The leg size of the fillet weld, or throat of groove weld, is determined by dividing this unit force by the suitable allowable.

The effect of the vertical shear load (V) on these

FIGURE 28



welds could be checked by using the entire length of the welds. However, this would represent little additional force on these welds.

Proportioning the Tee Stiffener

The following will be helpful in selecting a Tee stiffener section for this type of connection, where the beam flange equals the full width of the stiffener flange:

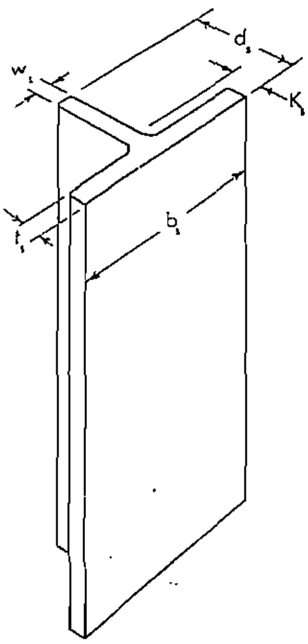


FIGURE 29

1. The thickness of the stiffener flange (t_s) must be sufficient to transfer the tensile force of the beam flange. In this case ¾ of the beam flange will be used.

$$\boxed{t_s \geqq .40 \sqrt{\tfrac{3}{4} b_b\, t_b}}$$

2. The width of the stiffener flange (b_s) must be sufficient for it to reach to the column flanges.

$$\boxed{b_s \geqq d_c - 2\, t_c}$$

3. The thickness of the stiffener stem (w_s) should be about the same as the beam flange thickness (t_b).

$$\boxed{w_s \geqq t_b}$$

4. The depth of the stiffener (d_s), as measured through the stem portion, must be sufficient for it to extend from the face of the column web to the outer edge of the column flange.

$$\boxed{d_s \geqq \frac{b_c - w_c}{2}}$$

5. As a guide, the stiffener should satisfy this condition:

$$\boxed{w_s\, (t_b + 5\, K_s) \geqq \tfrac{3}{4}\, b_b\, t_b}$$

or an approximation on the conservative side:

$$\boxed{w_s\, K_s = \frac{\tfrac{3}{4}\, b_b\, t_b}{5}}$$

Where Beam Flange Width < Stiffener Flange Width

Where the beam flange does not extend the full width of the connecting plate, the stem portion of the connection is assumed to carry the entire moment. Therefore the maximum bending force on the top portion of this weld will be—

$$f_b = \frac{3\, M\, D}{(D^3 - g^3)}$$

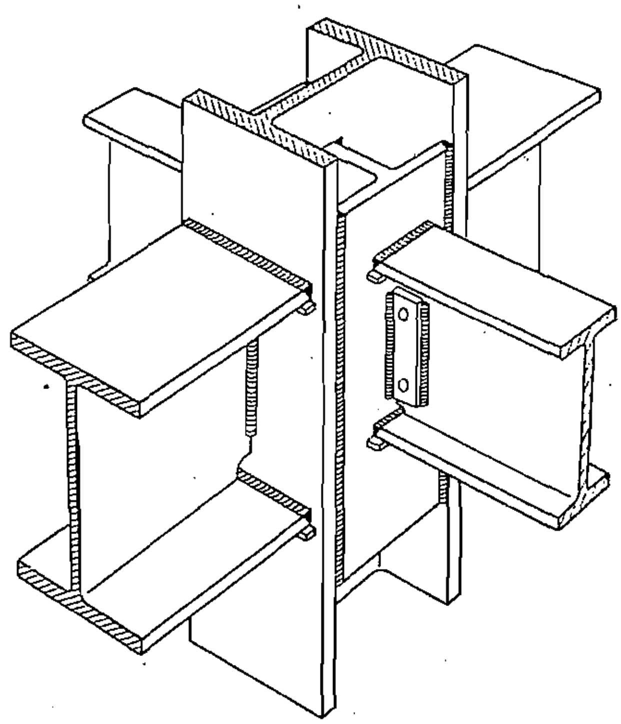


FIGURE 30

The same items as before are used to proportion the Tee stiffener, except in items 1 and 5 where the full value of the beam flange's section area is used instead of ¾ of this value. These formulas become—

1. $\boxed{t_s \geqq .40 \sqrt{b_b\, t_b}}$

5. $\boxed{w_{R}\ K_{s} \geqq \frac{b_b\ t_b}{5}}$

Problem 2

To design a fully welded beam-to-column connection for a 14" WF beam to an 8" WF column to transfer an end moment of M. = 1100 in.-kips and a vertical shear of V = 20 kips. The solution of this problem will be considered with seven variations. Use A36 steel and E70 welds.

1

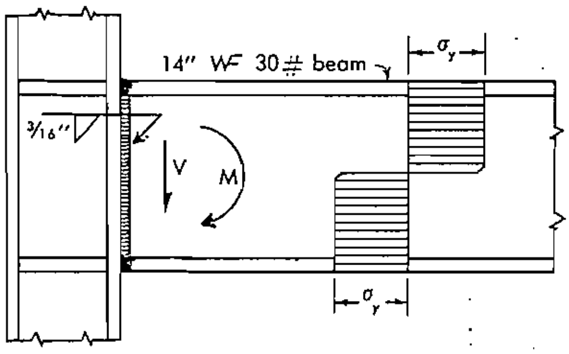


FIGURE 31

Here:

$$M = 1100 \text{ in.-kips}$$
$$V = 20 \text{ kips}$$

The welding of both the flanges and the web along its full depth enables the beam to develop its full plastic moment, thus allowing the "compact" beam to be stressed 10% higher in bending, or $\sigma = .66\ \sigma_y$. This also allows the end of the beam, and its welded connection, to be designed for 90% of the end moment due to gravity loading. (AISC Sec 1.5.1.4.1 and Sec 2.6)

$$\sigma = \frac{.9\ M}{S}$$
$$= \frac{.9(1100 \text{ in.-kips})}{(41.8 \text{ in.}^3)}$$
$$= 23{,}700 \text{ psi} < .66\ \sigma_y < 24{,}000 \text{ psi} \qquad \underline{\text{OK}}$$

TABLE 1—Properties of Beams Used in Problem 2

	b	d_b	t_f	t_w	S	K
14" WF 30# beam	6.733"	13.86"	.383"	.270"	41.8 in.³	
14" WF 34# beam	6.75"	14.00"	.453"	.287"	48.5 in.³	
14" WF 38# beam	6.776"	14.12"	.513"	.313"	54.6 in.³	1.0"

The weld on the beam's web must be able to stress the web in bending to yield (σ_y) throughout its entire depth; see the bending stress distribution in Figure 25. The weld must also be able to transfer the vertical shear.

unit force on this weld from the vertical shear

$$f_w = \frac{V}{2\ L_v}$$
$$= \frac{(20 \text{ kips})}{2(13.86 - 2 \times .387)}$$
$$= 770 \text{ lbs/in.}$$

leg size of fillet weld

$$\omega = \frac{\text{actual force}}{\text{allowable force}}$$
$$= \frac{(770)}{(11{,}200)}$$
$$= .069''$$

However, since the beam web is welded to a .433" thick flange of the column, the minimum size for this fillet weld would be 3/16"; see Section 7.4, Table 3.

WELD SIZE TO DEVELOP ULTIMATE LOAD

The next question is what size fillet weld would be required to develop the beam web to yield stress. The force in question results from bending, so it is transverse to the weld.

The AWS allowables for fillet welds are based on parallel loading, AWS has not set up any allowable values for transverse loading.

weld vs *web plate*

(parallel load) (transverse load-tension)

$$2(11{,}200\ \omega) \geqq t_w\ (.60\ \sigma_y) = t_w\ 22{,}000$$
$$\omega \geqq .982\ t_w$$

or $\boxed{\omega \geqq t_w}$ (AWS Code)

However it has been known for several years through testing and theory that a fillet weld loaded transversely is ⅓ stronger than when loaded parallel. Accordingly this ratio would become—

weld vs *web plate*

(transverse load) (transverse load-tension)

$$2(11{,}200\ \omega)\ 1\tfrac{1}{3} \geqq t_w\ (.60\ \sigma_y) = t_w\ 22{,}000$$
$$\omega \geqq .736\ t_w$$

or $\boxed{\omega \geqq \tfrac{3}{4}\ t_w}$

For plastic design concepts, based on ultimate loading, the allowable for the fillet weld would be increased by the factor 1.67 (AISC Sec 2.7). This is the same increase used for the member ($.60\ \sigma_y$ up to σ_y), hence the same relationship between weld size and plate thickness will still hold.

Based on AWS Code allowables (for parallel loading), this fillet weld on the web of the beam would have to be equal to the web thickness.

$t_w = .270''$ or use $\omega = 1/4''$

However since it is known a fillet weld ($\omega = 3/4\ t_w$) will outpull the web, a $3/16''$ fillet weld will be used here.

(2)

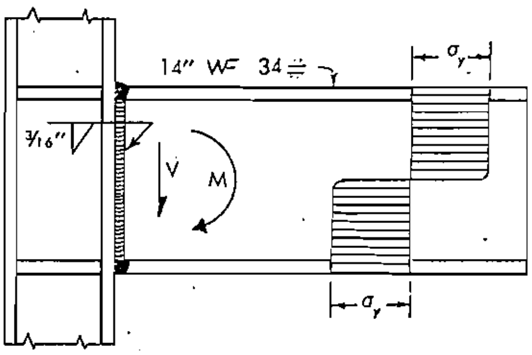


FIGURE 32

Here:

$M = 1100$ in.-kips

$V = 20$ kips

The welding of the flanges and full depth of the web enables the beam to develop its full plastic moment, allowing the "compact" beam to be stressed 10% higher in bending, or $\sigma = .66\ \sigma_y$. In this case the beam cantilevers out from the support so that no 10% reduction in the negative moment can be made.

$$\sigma = \frac{M}{S}$$

$$= \frac{(1100 \text{ in.-kips})}{(48.5 \text{ in.}^3)}$$

$$= 22{,}700 \text{ psi} < .66\ \sigma_y < 24{,}000 \text{ psi} \qquad \underline{\text{OK}}$$

The fillet weld on the web of the beam is figured as in method (1.)

(3)

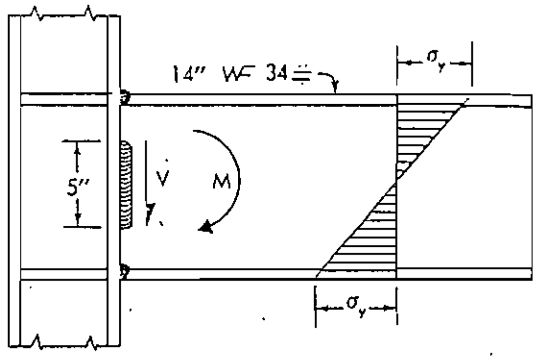


FIGURE 33

Here:

$M = 1050$ in.-kips

$V = 20$ kips

If this cantilever beam had an end moment of $M = 1050$ in.-kips instead of the previous 1100 in.-kips:

$$\sigma = \frac{M}{S}$$

$$= \frac{(1050 \text{ in.-kips})}{(48.5 \text{ in.}^3)}$$

$$= 21{,}600 \text{ psi} < .60\ \sigma_y < 22{,}000 \text{ psi} \qquad \underline{\text{OK}}$$

In this case the bending stress is within $.60\ \sigma_y$, and the beam and connection must be able to develop a bending resistance equal to the product of the beam's section modulus and yield point stress (see Fig. 27) rather than the full plastic moment. As a result it is not necessary to weld the web for its full depth.

For determining the minimum length of the fillet weld on the web, assume the leg size to not exceed $2/3\ t_w = 2/3\ (.287'') = .192''$. This will provide sufficient length of weld so the beam web at the connection will not be overstressed in shear. (AISC Sec 1.17.5)

The minimum length of fillet weld on each side of the web is—

$$L_r = \frac{V}{2\ f_w}$$

$$= \frac{(20 \text{ kips})}{2(11{,}200\ \omega)} = \frac{20 \text{ kips}}{2(11{,}200)(.192)}$$

$$= 4.65''$$

If $3/16''$ fillet welds are used (next size smaller than $.192''$), their length would be—

$$L_v = \frac{V}{2\ f_w}$$

$$= \frac{(20 \text{ kips})}{2(11{,}200)(3/16)}$$

$$= 4.75''$$

Hence use 3/16″ fillet 5″ long on both sides < 4.65″. OK

Since the size of this weld used in determining its length was held to ⅔ of the web thickness, it is unnecessary to check the resulting shear stress in the web at this connection. However, to illustrate this, it will be checked here:

$$\tau_{web} = \frac{V}{A_w}$$

$$= \frac{(20 \text{ kips})}{(5)\ (.287)}$$

$$= 14{,}000 \text{ psi} < .40\ \sigma_y < 14{,}500 \text{ psi} \quad \text{OK}$$

4

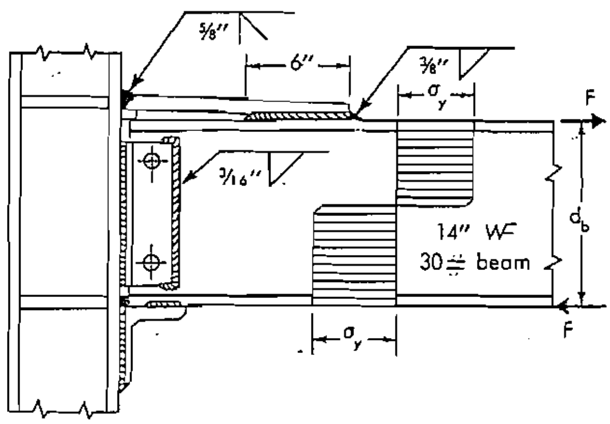


FIGURE 34

Here:

$$M = 1100 \text{ in.-kips}$$
$$V = 20 \text{ kips}$$

The welding of the flanges and full depth of the web enables the beam to develop its full plastic moment, allowing the "compact" beam to be stressed 10% higher in bending, or $\sigma = .66\ \sigma_y$. This also allows the end of the beam, and its welded connection, to be designed for 90% of the end moment due to gravity loading. (AISC Sec 1.5.1.4.1 and Sec 2.6)

bending stress in beam

$$\sigma = \frac{.9\ M}{S}$$

$$= \frac{.9\ (1100 \text{ in.-kips})}{(41.8 \text{ in.}^3)}$$

$$= 23{,}700 \text{ psi} < .66\ \sigma_y < 24{,}000 \text{ psi} \quad \text{OK}$$

bending force on top connecting plate

$$F = \frac{.9\ M}{d_b}$$

$$= \frac{.9\ (1100 \text{ in-kips})}{13.86''}$$

$$= 71.5 \text{ kips}$$

section area of top connecting plate

$$A_p = \frac{F}{\sigma}$$

$$= \frac{(71.5 \text{ kips})}{(24{,}000 \text{ psi})}$$

$$= 2.98 \text{ in.}^2$$

or use a 5½″ x ⅝″ plate, the section area of which is—

$$A_p = 3.44 \text{ in.}^2 > 2.98 \text{ in.}^2 \quad \text{OK}$$

If ⅜″ fillet welds are used to connect top plate to upper flange of beam:

$$f_w = 11{,}200\ (3/8)$$
$$= 4200 \text{ lbs/linear inch}$$

length of fillet weld

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

$$= \frac{(71.5 \text{ kips})}{(4200 \text{ lbs/in.})}$$

$$= 17''$$

or use 5½″ of weld across the end, and return 6″ along each side, for a total weld length of 17½″.

The lower flange of the beam is groove butt welded directly to the column flange; and, since the web framing angle carries the shear reaction, no further work is required on this lower portion of the connection. The seat angle simply serves to provide temporary support for the beam during erection and a backing for the flange groove weld.

The fillet weld on the web of the beam is figured as in method 1.

5

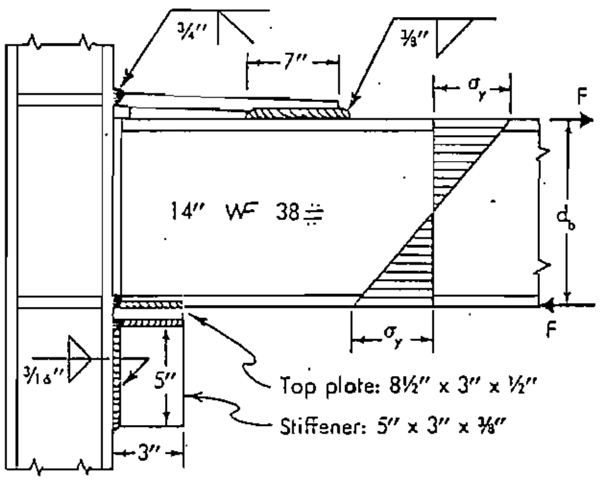


FIGURE 35

Here:

$M = 1100$ in.-kips

$V = 20$ kips

In this particular connection, the shear reaction is taken as bearing through the lower flange of the beam.

There is no welding directly on the web. For this reason it cannot be assumed that the web can be stressed (in bending) to yield through its full depth. Since full plastic moment cannot be assumed, the bending stress allowable is held to $\sigma = .60\ \sigma_y$ or $\sigma = 22{,}000$ psi for A36 steel. (AISC Sec 1.5.1.4.1)

bending stress in beam

$$\sigma = \frac{M}{S}$$

$$= \frac{(1100 \text{ in.-kips})}{(54.6 \text{ in.}^3)}$$

$$= 20{,}200 \text{ psi} < .60\ \sigma_y < 22{,}000 \text{ pso} \qquad \underline{\text{OK}}$$

bending force in top connecting plate

$$F = \frac{M}{d}$$

$$= \frac{(1100 \text{ in.-kips})}{(14.12'')}$$

$$= 78.0 \text{ kips}$$

section area of top connecting plate

$$A_p = \frac{F}{\sigma}$$

$$= \frac{(78.0 \text{ kips})}{(22{,}000 \text{ psi})}$$

$$= 3.54 \text{ in.}^2$$

or use a 5″ x ¾″ plate, the section area of which is—

$$A_p = 3.75 \text{ in.}^2 > 3.54 \text{ in.}^2 \qquad \underline{\text{OK}}$$

If ⅜″ fillet welds are used to connect the top plate to the upper flange of the beam:

$$f_w = 11{,}200\ (⅜)$$

$$= 4200 \text{ lbs/linear inch}$$

length of fillet weld

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

$$= \frac{(78.0 \text{ kips})}{(4200 \text{ lbs/in.})}$$

$$= 18.6''$$

or use 5″ of weld across the plate end and return 7″ along each side, to give a total weld length of 19″ > 18.6″ OK

DESIGN OF BOTTOM SEAT

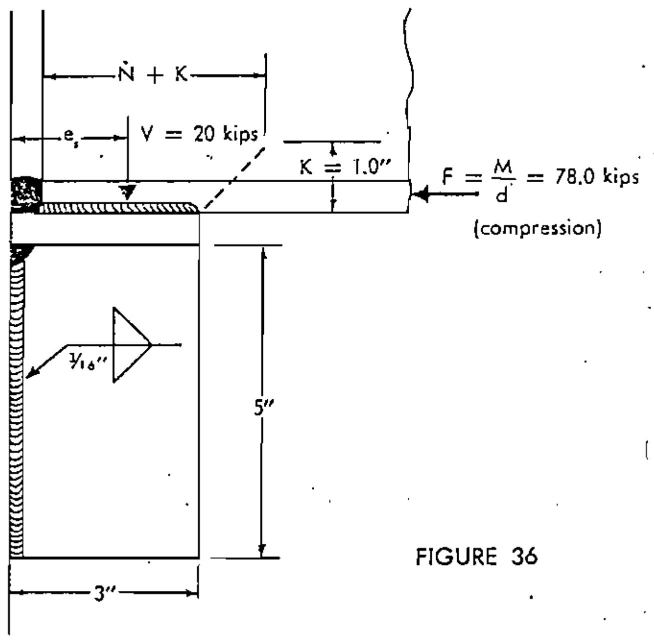


FIGURE 36

The shear reaction (V) by itself, applied to the bracket, produces a bending moment in the seat. This causes a tensile force in the seat bracket's top plate and connecting welds.

In the usual simple beam type construction, this moment must be considered in addition to the shear reaction when determining the required size of connecting weld on the seat.

In a continuous beam, the negative moment produces a compressive force in the lower flange which, in most cases, will offset the tensile force mentioned above.

As a result, the welds connecting the seat bracket will be designed only to resist the vertical shear force (V).

web crippling from end reactions

$$\frac{R}{t_w\ (N + K)} = .75\ \sigma_y \qquad \text{(AISC Sec 1.10.10)}$$

or:

$$N = \frac{R}{.75\ \sigma_y\ t_w} - K$$

$$= \frac{(20 \text{ kips})}{.75(36{,}000 \text{ psi})(.313'')} - 1.0''$$

$$= 1.37''$$

Hence the top plate of the seat must extend to at least ½″ gap + 1.37″ = 1.87″ and have a width at least 1″ greater than the beam's flange width (b) = 1″ + 6.776″ = 7.776″; or use an 8½″ x 3″ x ½″ plate. The 3″ dimension would allow room for erection bolt.

seat stiffener

The thickness of the seat stiffener (t_s) should be slightly greater than that of the beam web (t_w = .313″), or use a ⅜″ plate.

For determining the minimum length of the fillet weld on the stiffener, assume the leg size to not exceed ⅔ t_s = ⅔ (⅜) = ¼″. This keeps the stiffener at the connection from being overstressed in shear. (AISC Sec 1.17.5)

Thus, the minimum length of fillet weld on each side of the stiffener is—

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

$$= \frac{(20 \text{ kips})}{2(11{,}200)(¼)}$$

$$= 3.57''$$

Because the column flange to which this weld is placed is .433″ thick, the minimum fillet weld size would be $\frac{3}{16}$″.

Hence, use:

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

$$= \frac{20 \text{ kips}}{2(11{,}200)(\frac{3}{16})}$$

$$= 4.76'' > 3.57'' \qquad \underline{\text{OK}}$$

or use welds of $\frac{3}{16}$″ leg size and 5″ long, and of course the stiffener must be 5″ deep.

In this case, the lower flange of the beam will not be groove welded to the column flange. Instead, the top plate of the seat bracket will be extended to provide sufficient length of fillet weld.

If ⅜″ fillet welds are used along the edge of the .513″ thick beam flange:

$$L = \frac{F}{2\ f_w}$$

$$= \frac{(78.0 \text{ kips})}{2(11{,}200)(⅜)}$$

$$= 9.3'' \text{ or use } 9½''$$

Therefore, allowing for ½″ fit-up gap, use a 10″ x 8½″ x ½″ top plate for the seat.

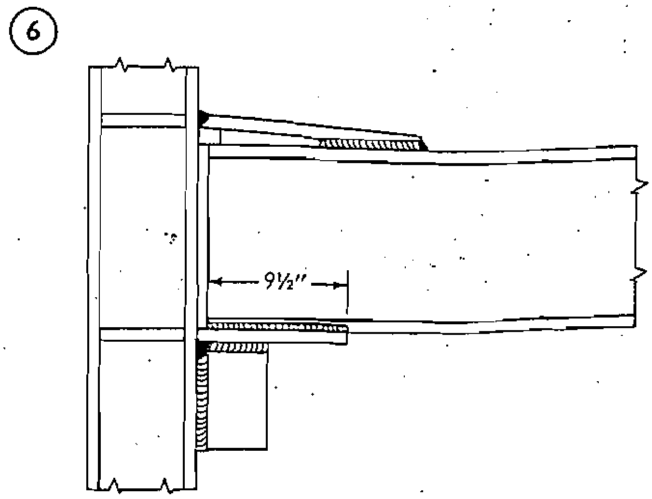


FIGURE 37

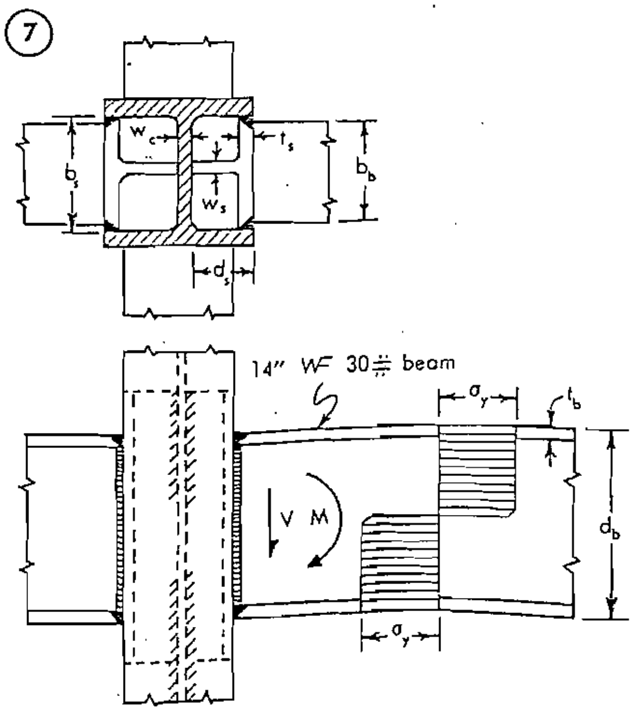


FIGURE 38

In this case the connection is made through the Tee stiffeners of the column. Since the beam flange is nearly as wide as the stiffener flange, the central stem portion of the stiffener is designed for ¾ of the moment and each outer edge of the stiffener flange for ⅛ of the moment.

The welding of the upper and lower portions of the stem to the column web is sufficient to stress the beam web up to yield (in bending) through its full depth. Thus, the beam may develop its full plastic moment. This allows the "compact" beam to be stressed at $\sigma = .66\ \sigma_y$, and also to be designed for only 90% of the end moment. (AISC Sec 1.5.1.4.1 and Sec 2.6)

DETAIL THE TEE STIFFENER

1. $$t_s \geqq .40\sqrt{\tfrac{3}{4}\, b_b\, t_b}$$
$$\geqq .40\sqrt{\tfrac{3}{4}\,(6.733)(.387)}$$
$$\geqq \underline{.56''}$$

2. $$b_s \geqq d_c - 2\ t_c$$
$$\geqq (8.0) - 2\ (.433)$$
$$\geqq \underline{7.13''}$$

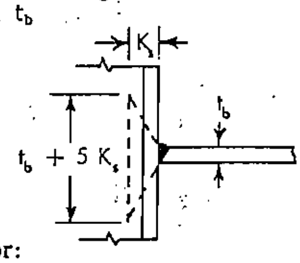
3. $$w_s \geqq t_b$$
$$\geqq \underline{.433''}$$

4. $$d_s \geqq \frac{b_c - w_c}{2}$$
$$\geqq \frac{(8.0) - (.288)}{2}$$
$$\geqq \underline{3.86''}$$

5.* $$w_s K_s \geqq \frac{\tfrac{3}{4}\, b_b\, t_b}{5}$$
$$\geqq \frac{\tfrac{3}{4}(6.733)(.387)}{5}$$
$$\geqq \underline{.39}$$

---

* $$w_s\,(t_b + 5\ K_s) = \tfrac{3}{4}\ \text{beam flange area} = \tfrac{3}{4}\ b_b\ t_b$$



For simplicity, use a conservative value:

$$w_s\ 5\ K_s = \tfrac{3}{4}\ b_b\ t_b \qquad \text{or:}$$

$$w_s\ K_s = \frac{\tfrac{3}{4}\ b_b\ t_b}{5}$$

On this basis use Tee section cut from an 8″ WF 48# beam; see Figure 39.

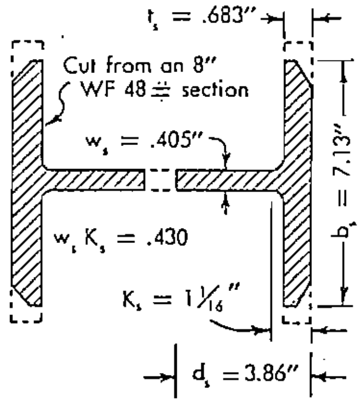
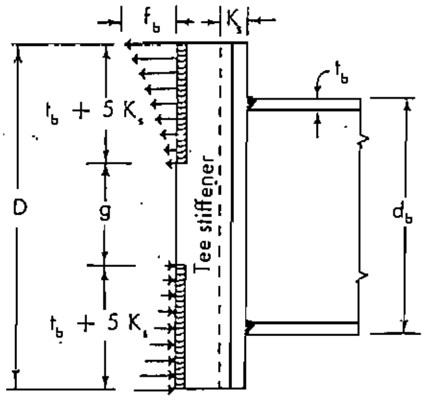


FIGURE 39

CHECK SIZE OF WELDS ON STIFFENER STEM



$$D = d_b + 5\ K_s$$
$$= (13.86) + 5\ (1\tfrac{1}{16})$$
$$= 19.18''$$
$$g = d_b - 2\ t_b - 5\ K_s$$
$$= (13.86) - 2\ (.387) - 5\ (1\tfrac{1}{16})$$
$$= 7.77''$$

maximum bending force

At top of weld on stem. Use ¾ of the moment (M).

$$f_b = \frac{\tfrac{3}{4}\ M\ 3\ D}{D^3 - g^3}$$
$$= \frac{\tfrac{3}{4}\ (.90 \times 1100)\ 3\ (19.18)}{(19.18)^3 - (7.77)^3}$$
$$= 6500\ \text{lbs/linear inch}$$

leg size of fillet weld

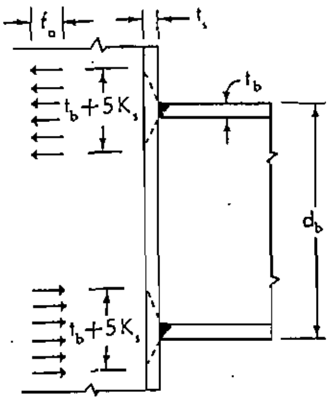
$$\omega = \frac{\text{actual force}}{\text{allowable force}}$$

$$= \frac{(6500)}{(11{,}200)}$$

$$= .58'' \text{ or use } \tfrac{9}{16}''$$

CHECK WELDS AT OUTER EDGES OF STIFFENER

Use ⅓ of the moment (M).



force on weld

$$f_a = \frac{\tfrac{1}{3}\,M}{(d_b - t_b)(t_b + 5\ t_s)}$$

$$= \frac{\tfrac{1}{3}\ (.90 \times 1100)}{(13.86 - .387)(.387 + 5 \times .683)}$$

$$= 6270 \text{ lbs/linear in.}$$

if fillet welds, leg size

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

$$= \frac{6270}{11{,}200}$$

$$= .56'' \text{ or use } \tfrac{9}{16}''$$

if partial-penetration single-bevel groove welds, throat size

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

$$= \frac{6270}{15{,}800}$$

$$= .397''$$

actual throat is—

$$t = t_e + \tfrac{1}{4}''$$

$$= .397'' + \tfrac{1}{4}''$$

$$= .647'' \text{ or use } \tfrac{11}{16}''$$

CHECK EFFECT OF SHEAR

The vertical shear of 20 kips was not considered on the welds because of the great length of welding. This could be checked out.

assumed total length of welding

$$L = 2\ D + 4\ (t_b + 5\ K_s)$$

$$= 2\ (19.18) + 4\ (.387 + 5 \times 1\tfrac{1}{16})$$

$$= 61.2''$$

unit shear force on weld

$$f_s = \frac{V}{L}$$

$$= \frac{(20)}{(61.2)}$$

$$= 327 \text{ lbs/linear inch}$$

For fillet welds, this would represent an additional leg size of—

$$\omega = \frac{327}{11{,}200}$$

$$= .029''$$

For partial-penetration groove welds, this would represent an additional throat of—

$$t = \frac{327}{15{,}800}$$

$$= .021''$$

These additional weld sizes are neglected in this example. If they had been appreciably larger, they would have been added to the weld sizes already obtained for bending.

9. LARGE HEAVILY LOADED BEAM-TO-COLUMN CONNECTION

It might be well to consider the basic transfer of forces through a beam-to-column connection.

A force applied transverse or at right angles to a member is transferred almost wholly into the portions of that member which lie parallel to this force. See Figure 40.

In the design of some connections, the portion of this force (F) transferred into any given element of the built-up member has been assumed to be proportionate to the stiffness or moment of inertia of this element compared to the total. See Figure 41.

An axial force in a member can transfer out at one end either as an axial force (normal stress, either tensile or compressive) or out sideways into an adjacent member as shear.

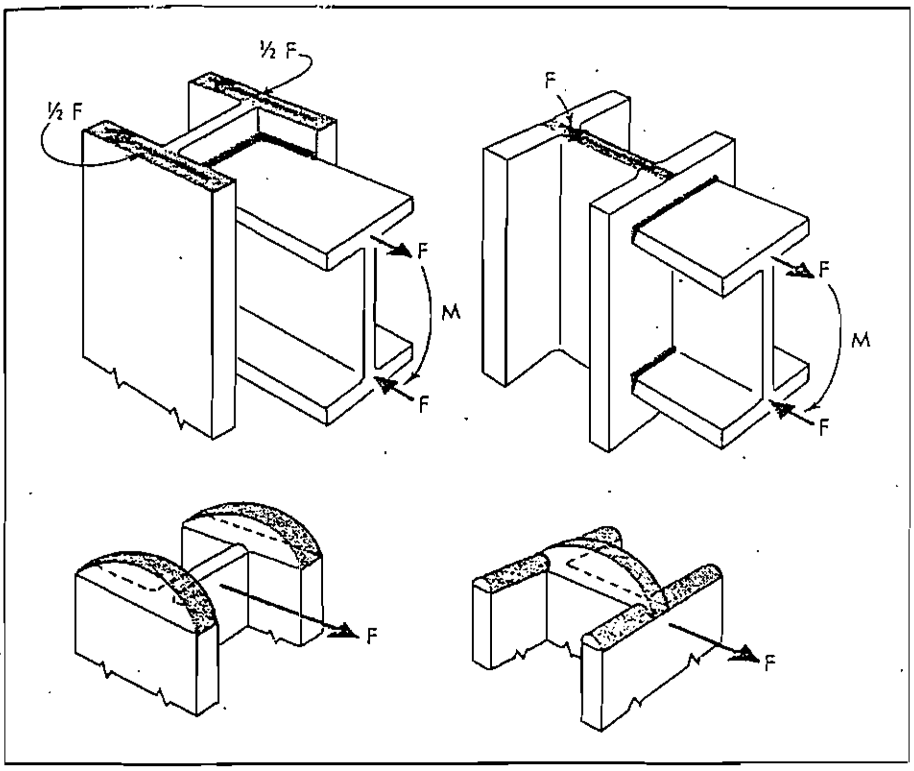


FIGURE 40

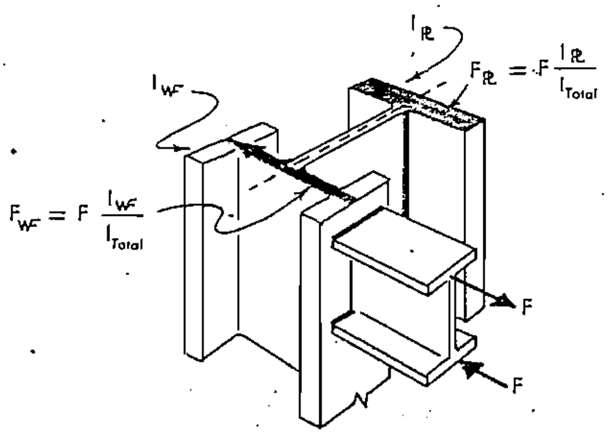


FIGURE 41

Tensile Transfer

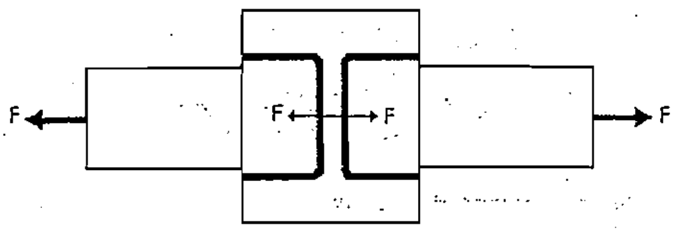


FIGURE 42

Tensile force from right-hand beam flange transfers directly as tension through the right-hand stiffener, column web, left-hand stiffener, and into flange of opposing beam.

Welds to column web and flange must be designed for this force. Although the total length of welding on the stiffener would be figured for this force, actually most of the force would be carried by the transverse weld between the stiffener and the column web. Under ultimate loading, we can assume the transverse portion will have yielded and the force will be uniformly distributed.

Shear Transfer

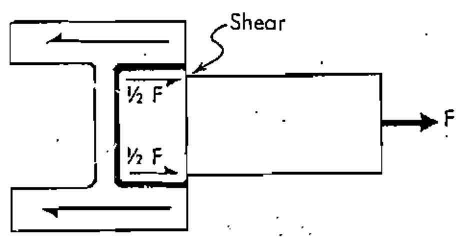


FIGURE 43

Tensile force from beam flange transfers directly as tension into stiffener and then out as shear into the column flanges.

Parallel welds to column flanges must be designed for this force, unless another stiffener is placed on the opposite side of the column web to back up this stiffener.

Tensile Transfer

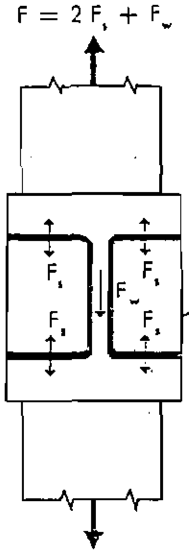


FIGURE 44

Tensile force from beam flange transfers directly as tension through both stiffeners and web of column into other beam flange.

Transverse welds between column flanges and stiffeners must be designed for this force (F) less that which passes directly into the web from the flange.

Parallel welds between stiffeners and column web transfer no force. Compression portion of beam connection would keep stiffener from buckling.

Shear Transfer

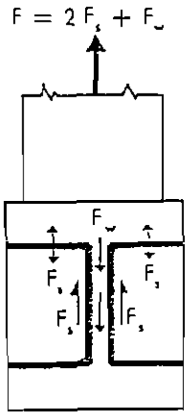


FIGURE 45

Tensile force from beam flange transfers directly as tension into stiffeners and column web. The tensile force in the stiffeners then transfers out as shear through the parallel welds into column web.

Transverse welds between column flanges on the beam side and stiffeners must be designed for this force (F) less that which passes directly into the web from the flange. Parallel welds to column web must be designed for this same force.

Any unbalanced moment ($M = M_1 - M_2$) entering the column must be transferred into the column flanges as a shear transfer. Assume $M_1 > M_2$.

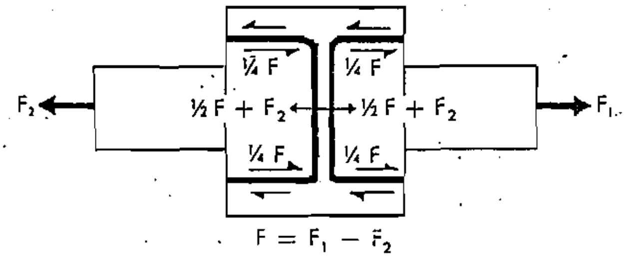


FIGURE 46

The tensile force F_2 of the flange of the left-hand beam will transfer as tension into the stiffener, then through the transverse welds along the column web into the other stiffener, and into the flange of the other beam.

The unbalanced tensile force ($F_1 - F_2$) of the flange of the right-hand beam will transfer as tension into the right-hand stiffener, and half of this through the transverse welds of the column web into the left-hand stiffener. This unbalanced tensile force in these stiffeners now transfers through the parallel welds as shown into the flanges of the columns.

Welds to column web must be designed for the balanced force, or $\frac{1}{2} F + F_2 = \dfrac{F_1 + F_2}{2}$.

Welds to column flange must be designed for the unbalanced force or $F_1 - F_2$.

Distribution of Tensile Force

There is some problem in estimating the portion of the tensile force in the beam flange transferring directly into the web of the column and into the column stiffeners.

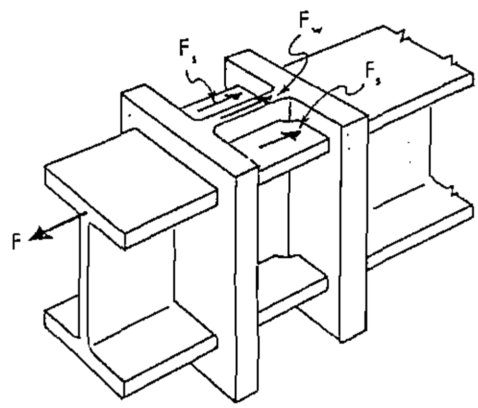


FIGURE 47

At first glance it would seem reasonable to assume this force would be divided according to the width of the stiffeners (b_s) and thickness of column web (t_w).

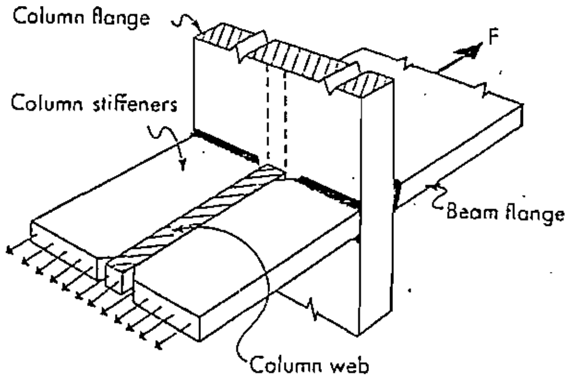


FIGURE 48

However, this column web section is not limited to the thickness of the beam flange since there is some spreading out of this force in the web. This might be assumed to occur at a slope of 1 to 2½.

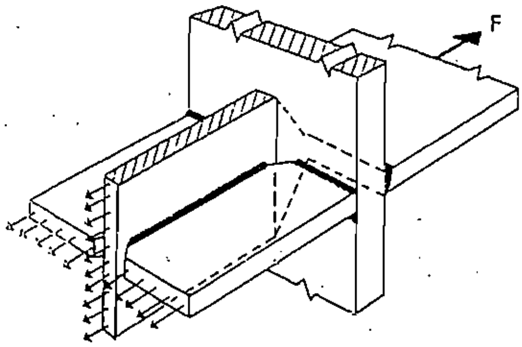


FIGURE 49

The effective depth of the column web through which force is distributed, is obtained as follows:

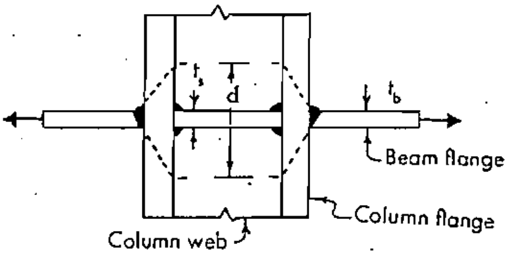
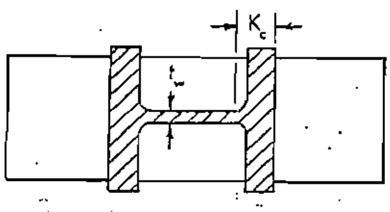


FIGURE 50

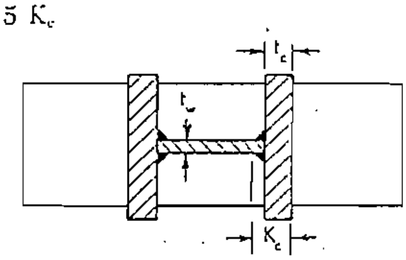
rolled column

$$d = t_b + 5\ K_c$$



fabricated column

$$d = t_b + 5\ K_c$$



Since:

A_w = area of column web over which force is distributed = $d\ t_w$

A_s = area of one stiffener (there is a pair)

$$(\text{web})\quad F_w = F\left(\frac{A_w}{A_w + 2\ A_s}\right)$$

$$(\text{stiffener})\quad F_s = F\left(\frac{A_s}{A_w + 2\ A_s}\right)$$

Combined Stress in Stiffener (See Figure 51.)

On the left-hand figure, the shear stress (τ_{xy}) results from the unbalanced East-West moments. This causes the difference in tensile beam flange force (F_1-F_2) to be transferred as shear in the stiffeners into the column flanges.

Although conservative in this particular analysis, it is assumed the small section in the stiffener to be checked lies outside of the path which the East-West tensile flange force will travel; hence $\sigma_x = 0$. Actually some of this tensile force will spread out into this region, and this would result in lower principal stress. In either case, it would be checked by the following formula:

$$\sigma_{max} = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

or

$$\boxed{\sigma_{max} = \frac{\sigma_y}{2} + \sqrt{\left(\frac{\sigma_y}{2}\right)^2 + \tau_{xy}^2}}$$

On the right-hand figure, it is assumed the small section to be checked is not subjected to any shear stress, just biaxial tensile stress. In this case, the use of the formula results in the principal stresses being equal to the applied tensile stresses. This does not result in any higher stress.

$$\sigma_{max} = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$$

or

$$\boxed{\sigma_{max} = \sigma_x \text{ or } \sigma_y}$$

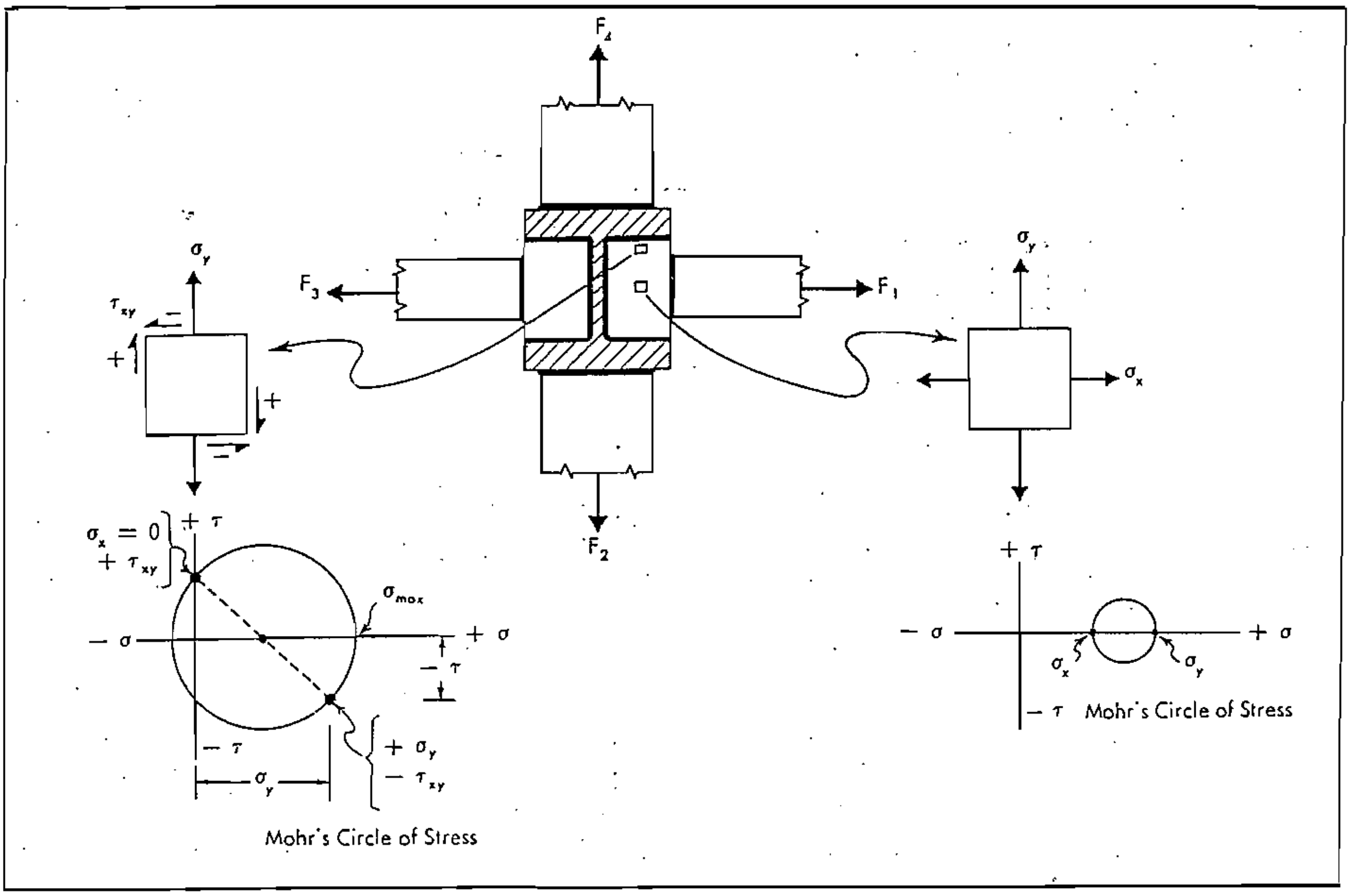
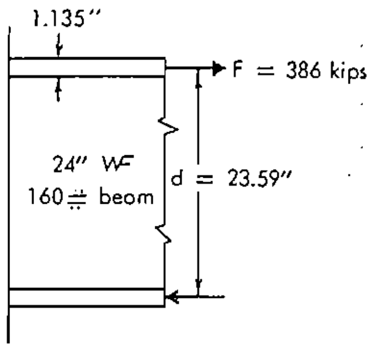


FIGURE 51

Problem 3

To check beam-to-column connection shown in Figure 52 (next page) for weld sizes.

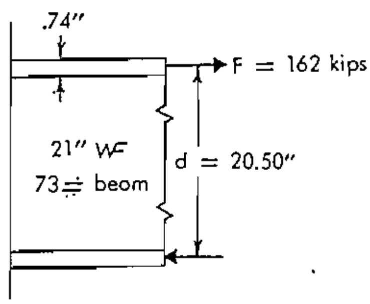
flange force: 24″ WF 160# beam



$$M = \sigma S$$
$$= (22{,}000 \text{ psi})(413.5 \text{ in.}^3)$$
$$= 9097 \text{ in.-kips}$$
$$d = 24.72'' - 1.135''$$
$$= 23.59''$$

$$F = \frac{M}{d}$$
$$= \frac{(9097 \text{ in-kips})}{(23.59'')}$$
$$= 386 \text{ kips}$$

flange force: 21″ WF 73# beam



$$M = \sigma S$$
$$= (22{,}000 \text{ psi})(150.7 \text{ in.}^3)$$
$$= 3315 \text{ in.-kips}$$
$$d = 21.24'' - .74''$$
$$= 20.50''$$

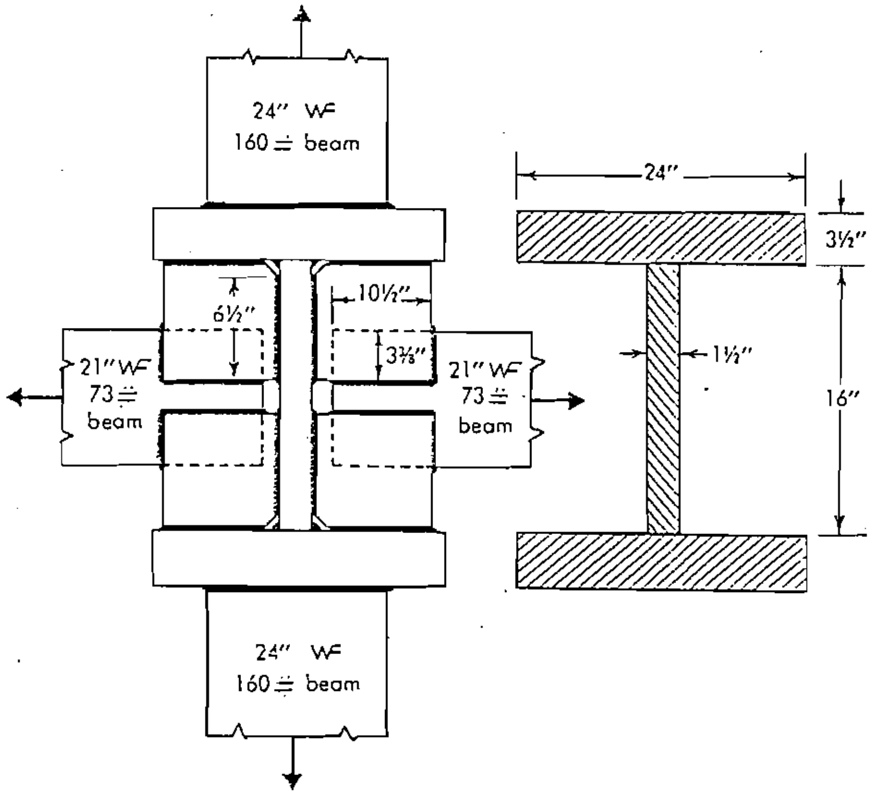


FIGURE 52

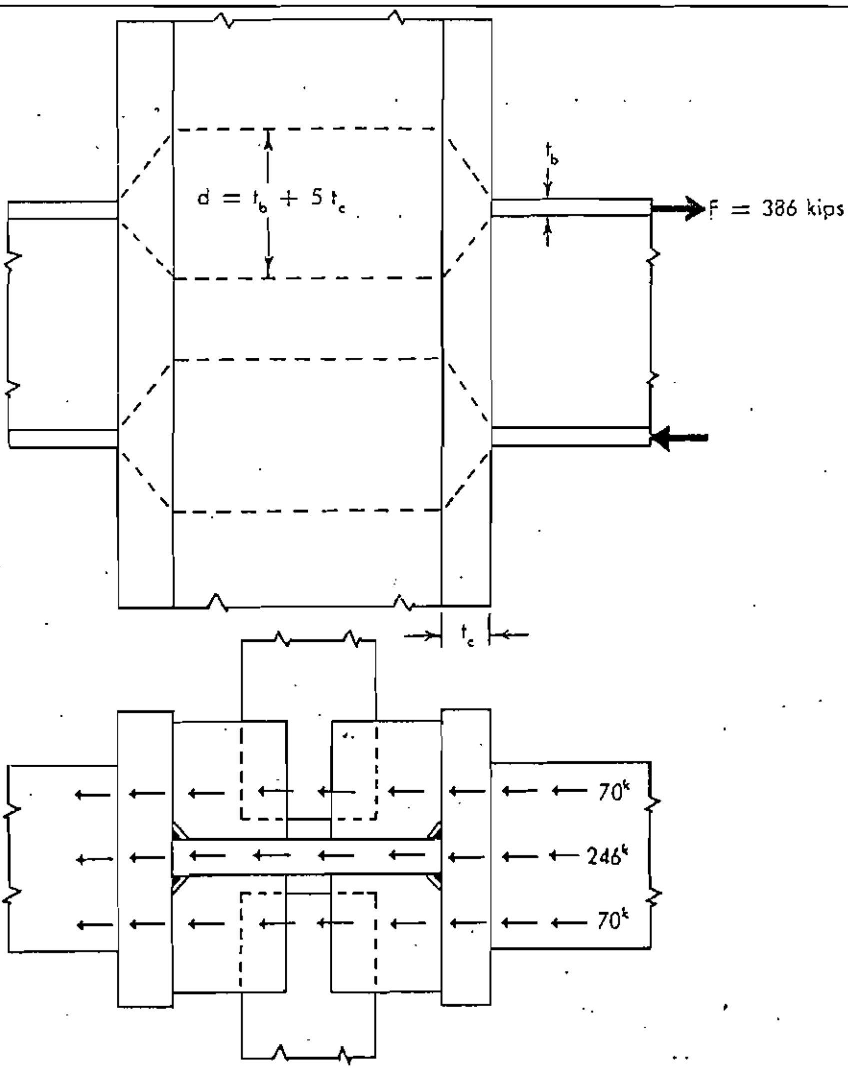


FIGURE 53

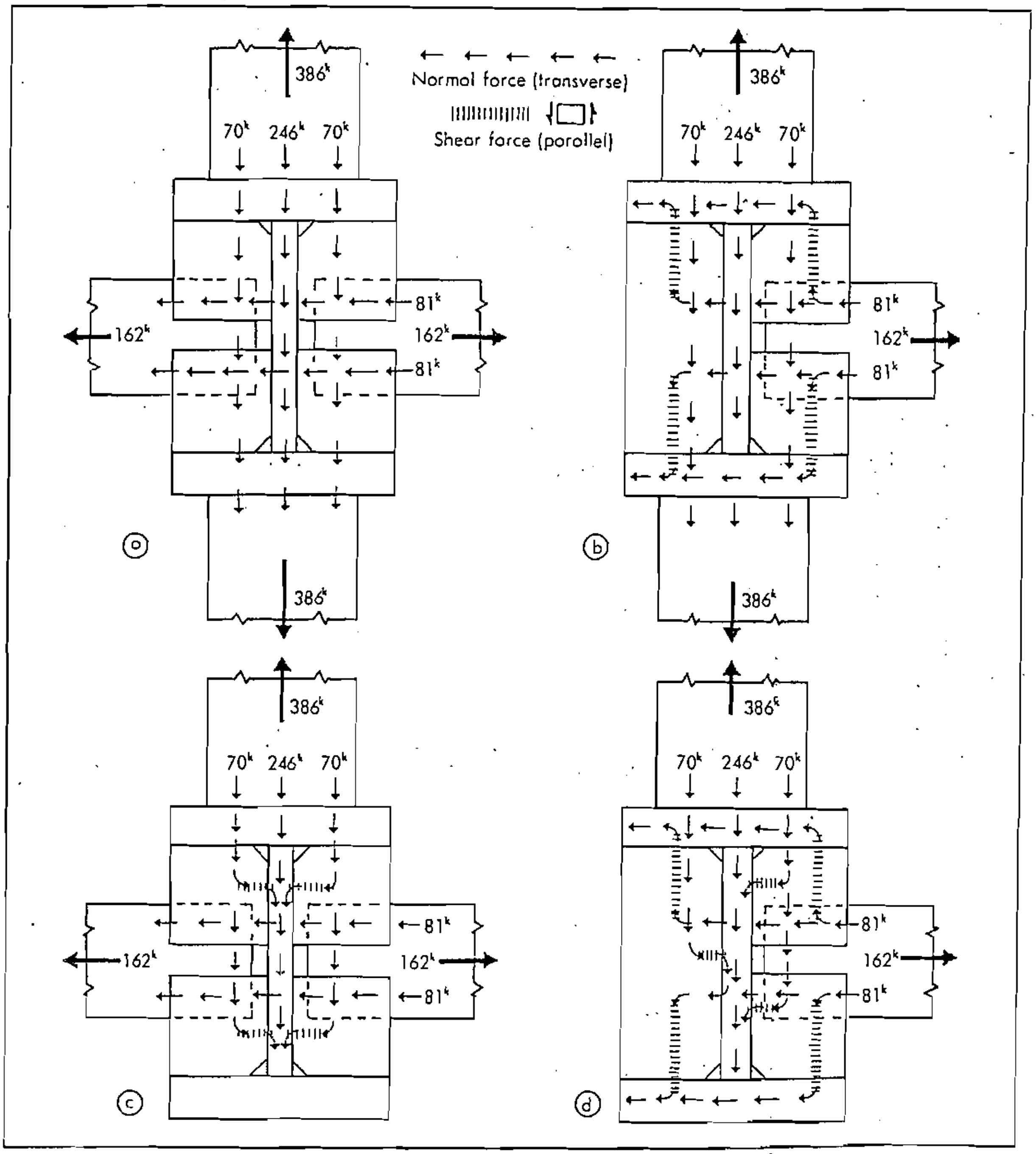


FIGURE 54

$$F = \frac{M}{d}$$

$$= \frac{(3315 \text{ in.-kips})}{(20.50'')}$$

$$= 162 \text{ kips}$$

distribution of beam force (See Figure 53.)

Depth of column web through which beam force is transferred is—

$$d = t_b + 5\ t_c$$

$$= (1.135'') + 5\ (3\tfrac{1}{2}'')$$

$$= 18.64''$$

If 1″ horizontal plate stiffeners are used—

$$A_s = (10\tfrac{1}{2})(1)$$

$$= 10.5 \text{ in.}^2$$

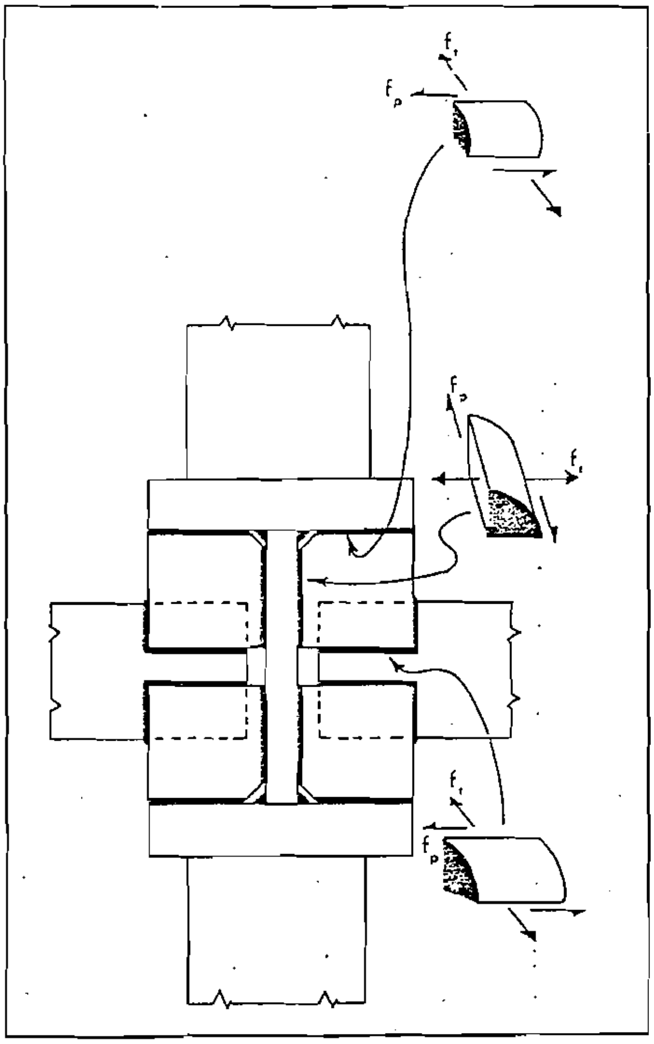


FIGURE 55

$$A_w = (18.64)(2) = 37.28 \text{ in.}^2$$

$$F_w = F\left(\frac{A_w}{A_w + 2\,A_s}\right) = 386\left(\frac{37.28}{37.28 + 21}\right) = 246 \text{ kips}$$

$$F_s = F\left(\frac{A_s}{A_w + 2\,A_s}\right) = 386\left(\frac{10.5}{37.28 + 21}\right) = 70 \text{ kips}$$

Figure 54 diagrams this distribution of beam force for four situations. Only one need be considered for any one problem. However, in this example we will detail the welds so they can carry any combination of forces from any of these four situations.

Figure 55 shows the forces on the various welds for which size must be determined.

weld size: stiffener to column flange; case b and d

$$f_t = \frac{70^k}{2\,(10\tfrac{1}{2}'')} = 3.33 \text{ kips/in.}$$

$$f_p = \frac{81^k}{4\,(10\tfrac{1}{2}'')} = 1.93 \text{ kips/in.}$$

$$f_r = \sqrt{f_t^2 + f_p^2} = \sqrt{3.33^2 + 1.93^2} = 3.87 \text{ kips/in.}$$

$$\omega = \frac{3.87}{11.2} = .344'' \text{ or } \tfrac{3}{8}'' \text{ if shop weld,}$$

but 3¾″ plate would need ½″

In the shop, fillet welds would be used, because they can be made on both sides of the stiffener.

For field welding, use 45° single bevel groove weld because it would be difficult to weld underside overhead.

weld size: stiffener to column web; case c and d

$$f_t = \frac{81^k}{2(6\tfrac{1}{2}'')} = 6.23 \text{ kips/in.}$$

$$f_p = \frac{70^k}{4(6\tfrac{1}{2}'')} = 2.69 \text{ kips/in.}$$

$$f_r = \sqrt{f_t^2 + f_p^2} = \sqrt{6.23^2 + 2.69^2} = 6.78 \text{ kips/in.}$$

$$\omega = \frac{6.78}{11.2} = .605'' \text{ or } \tfrac{5}{8}'' \text{ if shop weld}$$

(2″ plate needs min. of ⅜″).

For field weld, use 45° single bevel groove weld.

weld size: beam flange to stiffener; case a and b

$$f_t = \frac{70^k}{(10\tfrac{1}{2}'') + (3\tfrac{3}{8}'')} = 5.04 \text{ kips/in.}$$

$$f_p = \frac{81^k}{(10\tfrac{1}{2}'') + (3\tfrac{3}{8}'')} = 5.84 \text{ kips/in.}$$

$$f_r = \sqrt{f_t^2 + f_p^2}$$
$$= \sqrt{5.04^2 + 5.84}$$
$$= 7.72 \text{ kips/in.}$$
$$\omega = \frac{7.72}{11.2}$$
$$= .69'' \text{ or } 3/4''$$

check combined stress in stiffener; case (d)

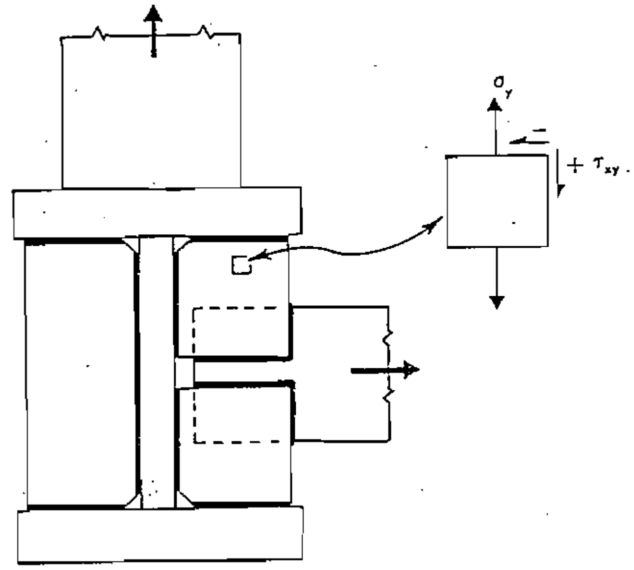


FIGURE 56

$$\sigma_y = \frac{70^k}{1''(10\tfrac{1}{2}'')}$$
$$= 6660 \text{ psi}$$
$$\tau_{xy} = \frac{81^k}{1''(2 \times 10\tfrac{1}{2}'')}$$
$$= 3860 \text{ psi}$$

$$\sigma_{max} = \frac{\sigma_y}{2} + \sqrt{\left(\frac{\sigma_y}{2}\right)^2 + \tau_{xy}^2}$$
$$= \frac{6660}{2} + \sqrt{\left(\frac{6660}{2}\right) + 3860^2}$$
$$= 8430 \text{ psi} \quad \underline{\text{OK}}$$
$$\tau_{max} = \sqrt{\left(\frac{\sigma_y}{2}\right)^2 + \tau_{xy}^2}$$
$$= \sqrt{\left(\frac{6600}{2}\right)^2 + 3860^2}$$
$$= 5100 \text{ psi} \quad \underline{\text{OK}}$$

Problem 4

To check the weld size joining the flange and web of the built-up welded column in Figures 57 and 58.

(1) *weld on column between floors*

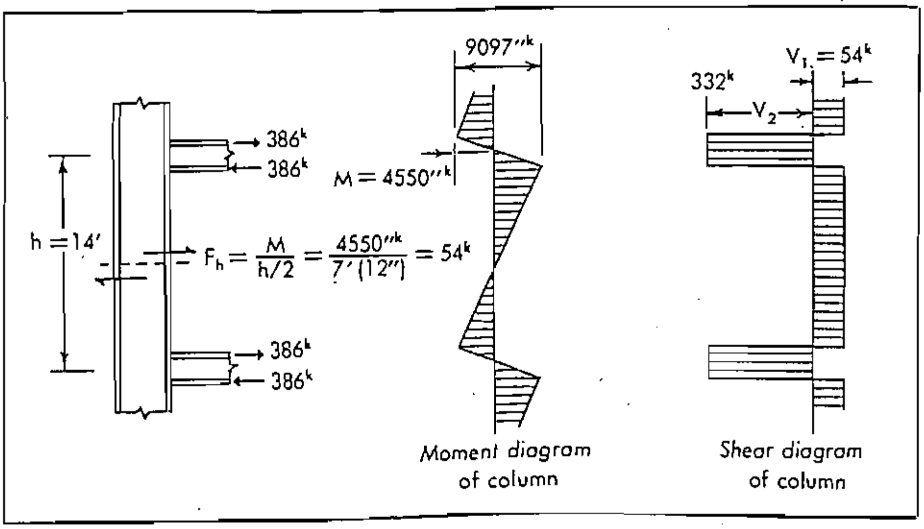
$$f_1 = \frac{V_1\, a\, y}{I\, n}$$
$$= \frac{(54^k)(84)(9\tfrac{3}{4})}{(16{,}815)(2)}$$
$$= 1310 \text{ lbs/in. longitudinal shear on weld}$$
$$\omega = \frac{1310}{11{,}200}$$
$$= .10''$$

but because of 3½″ plate, use ½″

(2) *weld on column within beam connection*

$$f_2 = \frac{V_2\, a\, y}{I\, n}$$
$$= \frac{(332^k)(84)(9\tfrac{3}{4})}{(16{,}815)(2)}$$
$$= 8090 \text{ lbs/in. longitudinal shear on weld}$$

FIGURE 57



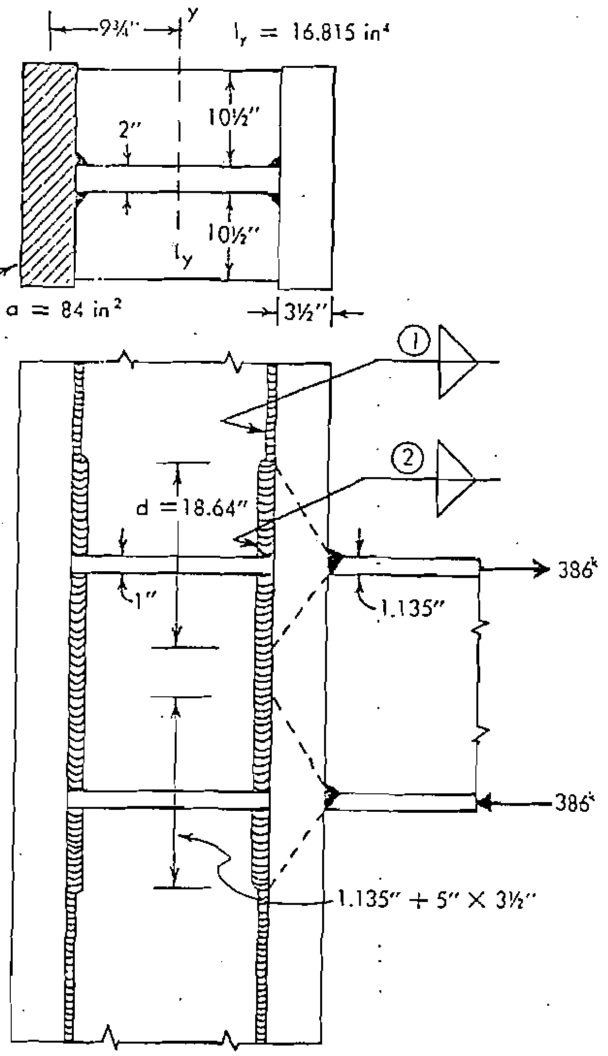


FIGURE 58

The transverse force must be added to this. A portion of the beam flange force must be transferred through this flange-to-web weld within the distance d = $t_b + 5\ K_s$ = 18.64″; the remainder of this force is transferred directly through the horizontal stiffeners:

$$F_{web} = F\ \left(\frac{A_w}{A_w + A_s}\right)$$

$$= 386^k\ \left(\frac{(18.64\ x\ 2)}{(18.64\ x\ 2) + 2(10½\ x\ 1)}\right)$$

$$= 247\ \text{kips}$$

This is a unit force on the weld of—

$$f_t = \frac{F_w}{2\ d}$$

$$= \frac{(247^k)}{2(18.64)}$$

$$= 6630\ \text{lbs/in.}$$

The resultant force on the weld is—

$$f_r = \sqrt{f_2^2 + f_t^2}$$

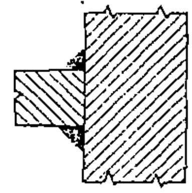
$$= \sqrt{8.09^2 + 6.63^2}$$

$$= 10{,}460\ \text{lbs/in.}$$

(a) If fillet welds are used, the required leg size is—

$$\omega = \frac{10{,}460}{11{,}200}$$

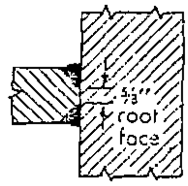
$$= .933''\ \text{or use}\ 1''$$



(b) If partial penetration J-groove welds are used, the required throat is—

$$t = \frac{10{,}460}{15{,}800}$$

$$= .662''$$



and the root face is—

$$2'' - 2(.662'') = .676''\ \text{or}\ ⅝''$$

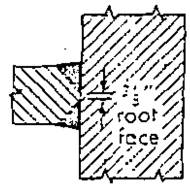
(c) If partial penetration bevel groove welds are used, the required throat is —

$$t_e = \frac{10{,}460}{15{,}800}$$

$$= .662''$$

$$t = t_e + ⅛''$$

$$= .787''$$



and the root face is—

$$2'' - 2(.787'') = .426''\ \text{or}\ ⅜''$$

10. ADDITIONAL STIFFENING OF WEB WITHIN BEAM-TO-COLUMN CONNECTION

In cases of unusually high unbalance of applied moments to a column, it might be well to check the resulting shear stresses in the web within the connection. See Figures 59 and 60.

Here the end moments (M_1 and M_2) of the beam due to a combination of the gravity load and wind, are resisted by the moments (M_3 and M_4) in the column. A good example of this occurs in multi-story buildings having no interior columns.

The forces in the beam flanges (F_1) resulting from the end moment (M_1), are transferred into the web of the connection as shear.

There are similar forces in the column flange (F_3 and F_4) from the same resisting moment. These forces

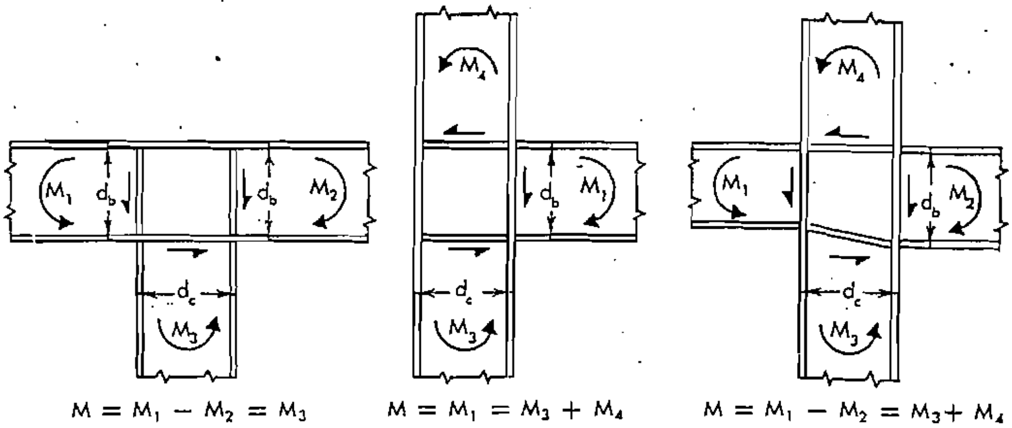


FIGURE 59

are transferred into the column web within the connection region as shear.

It can be assumed that most of the vertical shear force (V_1) of the beam web is transferred directly into the flange of the supporting column and does not enter the web of the connection.

The horizontal shear force (V_4) of the upper column will be transferred through the web of the connection into the lower column if caused by wind; ~~or~~ out across the beam to the adjacent column if caused by gravity load.

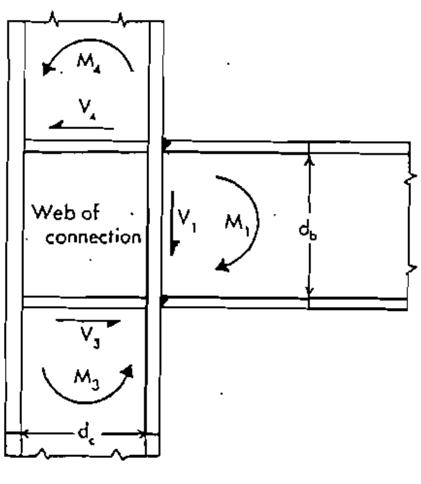


FIGURE 60

These resulting vertical and horizontal shear forces cause a diagonal compressive force to act on the web of the connection; and, if the web is too thin compared to its width or depth, it may suffer some buckling action. See Figure 61.

The following analysis, based on plastic design concepts, may be used to check this condition.

Analysis of Required Web Thickness

The unit shear force applied to the web of the connection is—

$$\nu = \frac{V}{d} = \frac{F_1 - V_4}{d_c} = \frac{M_1}{d_b d_c} - \frac{V_4}{d_c}$$

The resulting unit shear stress in the web of the connection is—

$$\tau = \frac{\nu}{w_1} = \frac{1}{w_c}\left(\frac{M_1}{d_b d_c} - \frac{V_4}{d_c}\right)$$

Using plastic design concepts, the applied moment (M_1) will become the plastic moment. For this value, the allowable shear stress (τ) will be based on the yield strength of the steel. The value for the shear

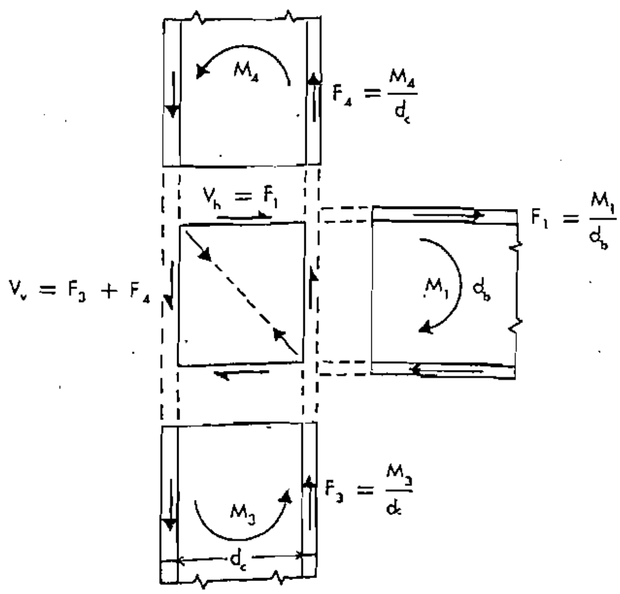


FIGURE 61

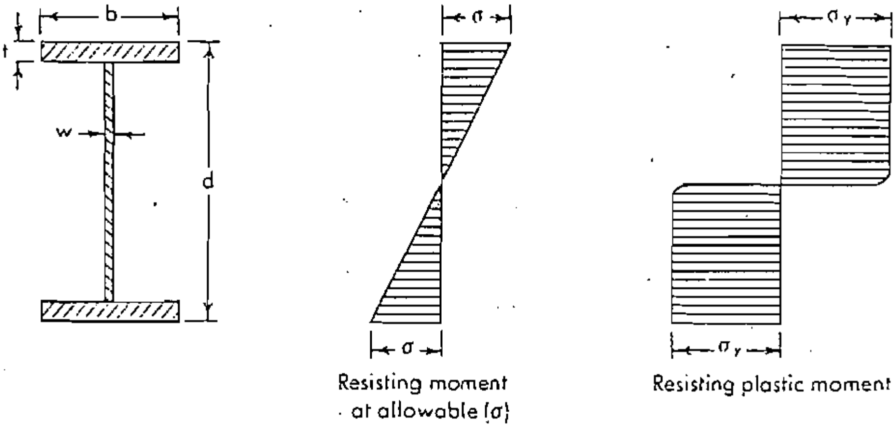


FIGURE 62

stress at yield (τ_y) may be found by using the Mises yield criterion:

$$\sigma_{cr} = \sqrt{\sigma_x^2 - \sigma_x\,\sigma_y + \sigma_y^2 + 3\,\tau_{xy}}$$

In this application of pure shear, σ_x and $\sigma_y = 0$, and setting the critical value (σ_{cr}) equal to yield (σ_y), we obtain:

$$\sigma_y = \sqrt{3\,\tau_{xy}} \qquad \text{or} \qquad \tau = \frac{\sigma_y}{\sqrt{3}}$$

hence:

$$\tau = \frac{\sigma_y}{\sqrt{3}} = \frac{1}{w_1}\left(\frac{M_1}{d_b d_c} - \frac{V_4}{d_c}\right)$$

or $$\boxed{w_1 = \frac{\sqrt{3}}{\sigma_y}\left(\frac{M_1}{d_b d_c} - \frac{V_4}{d_c}\right)} \quad \text{.............(1)}$$

The horizontal shear force (V_4) of the upper column acts in the opposite direction to (F_1) and thus reduces the shear value in the web of the connection; so this portion could be neglected for simplicity. This formula then becomes:

$$\boxed{w_1 = \frac{\sqrt{3}\;\; M_1}{d_b d_c\;\; \sigma_y}} \quad \text{........................(2)}$$

The plastic moment (M_1) is obtained by multiplying the plastic section modulus (Z) of the beam by the yield strength (σ_y) of the steel.

The plastic section modulus for all rolled sections is available in several steel manuals.

The plastic section modulus of a welded plate girder (Fig. 62) is obtained from the following formula:

$$\boxed{Z = b\,t\,(d - t) + \frac{w}{4}\,(d - 2t)^2} \quad \text{....(3)}$$

Or assuming that a conservative shape factor,

$$f = \frac{M_p}{M_y} = \frac{Z}{S} = 1.12$$

$$M_p = 1.12\,M_y \qquad\qquad \text{and } M_y = \sigma_y\,S$$

Formula 2 may be reduced to—

$$\boxed{w_1 = \frac{1.94\;\,S}{d_b\;\;d_c}} \quad \text{..........................(4)}$$

If the actual thickness of the web in the connection (w_2) is equal to or greater than this required value (w_1), no additional stiffening of the web would be necessary.

If the web thickness is less than this value, it must be stiffened by some method.

Methods of Stiffening Web in Connection

A web doubler plate could be added to make up this difference between actual and required values of web thickness.

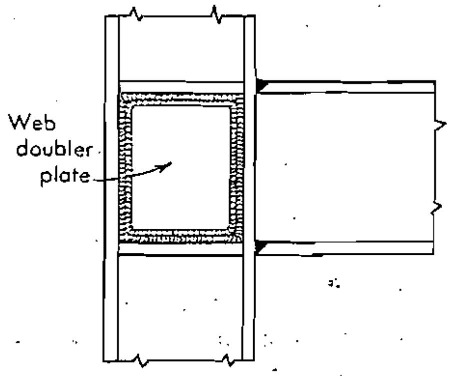


FIGURE 63

The most common solution is to use a pair of diagonal stiffeners. Their cross-sectional area would

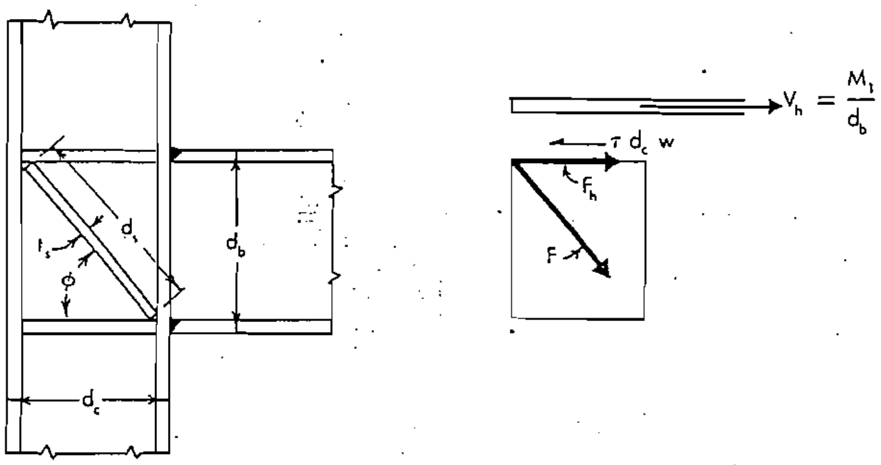


FIGURE 64

depend on the compressive force they must carry, over and above that carried by the web. See Figure 64.

The horizontal force applied to the connection is—

$$V_h = F_1 = \frac{M_1}{d_b}$$

The horizontal shear force resisted by the web is—

$$\tau\ d_c\ w_2 = \frac{\sigma_y}{\sqrt{3}}\ d_c\ w_2$$

The resulting horizontal component applied to the diagonal stiffener is—

$$F_h = \frac{M_1}{d_b} - \frac{\sigma_y}{\sqrt{3}}\ d_c\ w_2$$

The force on the diagonal stiffener is—

$$F = \frac{F_h}{\cos\ \theta} = \frac{1}{\cos\ \theta}\left(\frac{M_1}{d_b}\ \frac{\sigma_y}{\sqrt{3}}\ d_b\ w_2\right)$$

and the required total area of both stiffeners is—

$$\boxed{A_s = \frac{F}{\sigma_y} = \frac{1}{\cos\ \phi}\left(\frac{M_1}{\sigma_y d_b} - \frac{w_2 d_c}{\sqrt{3}}\right)} \quad \ldots\ldots(5)$$

also

$$\boxed{A_s = \sqrt{d_b^2 + d_c^2}\left(\frac{M_1}{\sigma_y\ d_b d_c} - \frac{w_2}{\sqrt{3}}\right)} \quad .(6)$$

also

$$\boxed{A_s = \frac{\sqrt{d_b^2 + d_c^2}}{\sqrt{3}}\ (w_1 - w_2) = \frac{d_s\ (w_1 - w_2)}{3}}$$

$$\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7)$$

where:

w_1 = minimum required web thickness, from Formula 2 or 4

w_2 = actual web thickness of connection

d_s = length of diagonal of connection area

11. COPE HOLES

When beam flanges will be field (groove) welded to the column, cope holes are quite often provided in the beam web to aid the welding operator in making the best possible groove weld across the flange where the web intersects it. See Figure 65.

This design decision is more important in bridge construction because of the possibility of fatigue or repeated loading. For steady loads, or even fatigue loads if the range of stress fluctuation is not very much, the requirement for a perfect groove weld is less important. This does not mean we should not try to get a good sound weld.

Although a cope hole in the web should provide a better groove weld, there is some concern with the notch effect of the hole when subjected to fatigue loading. In some fatigue testing of groove welds of beam flanges, with and without cope holes, it was found that the hole reduced the fatigue strength about 10% between the ranges of 100,000 cycles and 2,000,000 cycles. This was for a fatigue stress range of

$$K = \frac{\sigma_{min}}{\sigma_{max}} = 0$$

in other words going from a given stress down to zero, etc. For a more narrow range of stress, for example K = ½, going from a given stress down to just one-half, etc., there was almost no difference with or without cope holes.

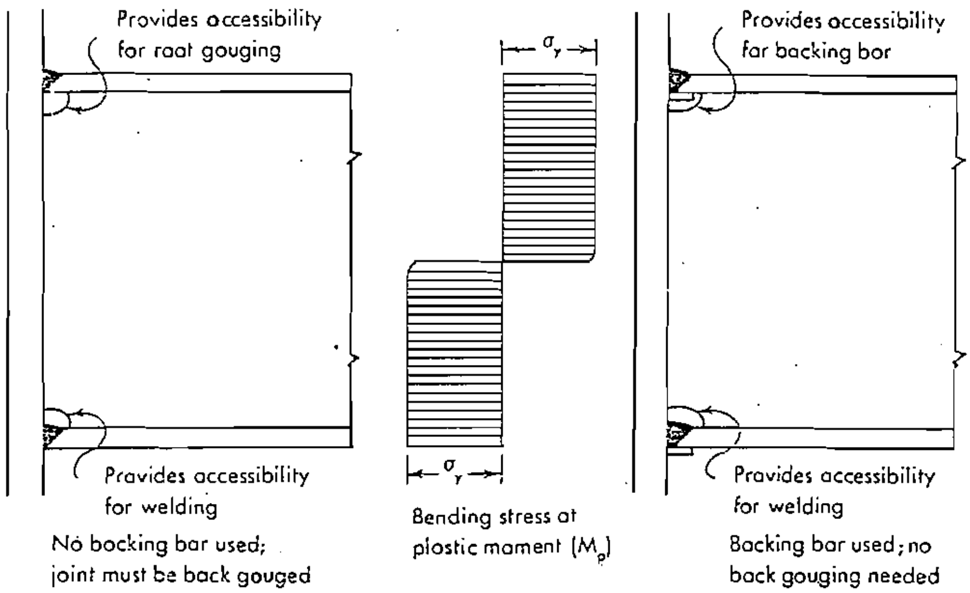


FIGURE 65

Plastic design is not used under fatigue loading conditions, so there should be less concern here about the need for cope holes and their resulting effect on the connection's strength. Cope holes would probably not result in any appreciable loss in plastic strength. The additional moment brought about by allowing the web to be stressed to yield strength after the outer fibers once reach yield is about 10%, and the cope hole represents a very small portion of this web section. Hence, the reduction in strength caused by the cope hole should be only a small fraction of the 10%.

Along the same line of thought, any minor lack of weld penetration due to this lack of accessibility with no cope hole would not be as critical.

In going through the original test reports of welded connections for plastic design, there are many beam-to-column connections or knees in which no cope holes were used. In the AISC report, "Welded Interior Beam-To-Column Connections" cope holes were used and a detail of these shown; see Figure 66. Notice that backing bars were used and the holes were not later filled with weld metal.

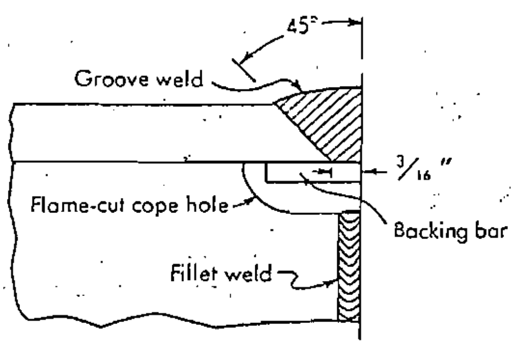


FIGURE 66

In plastic design, cope holes are not required to provide the weld quality required, although they would make it easier for the welding operator. And, if they are used, they won't have a detrimental effect on the strength of the connection if left unfilled.

The cope hole helps more for accessibility of the groove weld on the lower flange if welded in position. In most cases this would be an area of negative moment and this weld would be under compression, so this should not be as critical as the tension weld on the upper flange.

If the member could be turned over for shop welding, both flanges could be beveled from the outside and cope holes would not be needed; see Figure 67.

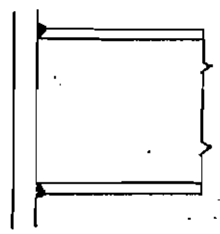


FIGURE 67

12. BEAMS CONTINUOUS THROUGH COLUMN (COLUMN CUT OFF)

On one-story construction, it is quite common to obtain continuity of the beam by allowing it to run continuously over the top of the column for two or more spans. Frequently the splice in the beam is carried out to the point of contraflexure.

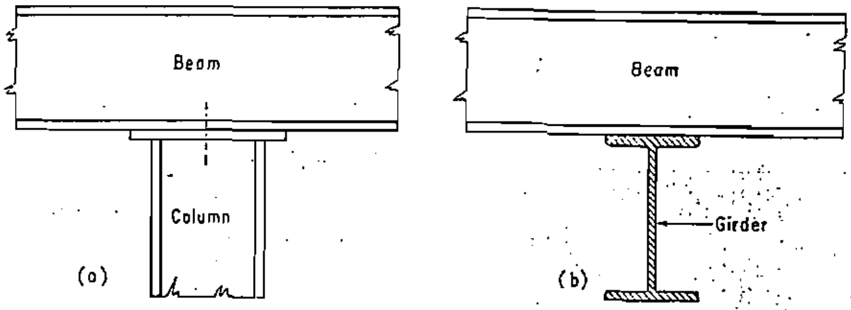


FIGURE 68

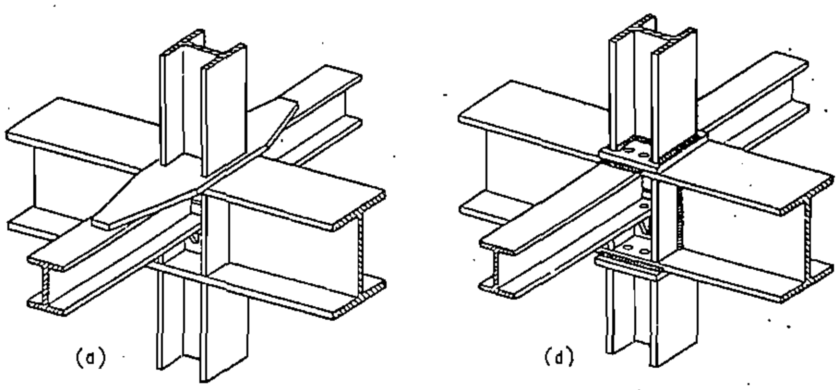


FIGURE 69

Figure 68 (a) shows the beam resting on a plate shop welded to the top of the column. In most cases fillet welds made in the downhand or flat position will be sufficient, since there is usually very little moment which must be transferred from the beam into the column.

Figure 68 (b) shows a similar connection made in the beam and the girder which supports it.

Figures 69 (a) and (b) show this method extended to multi-story construction. In both cases, stiffening plates are shop welded in between the flanges of the beam, in line with the column flanges, so that the compressive load may be transferred directly from one column flange to the other.

13. COVER PLATES FOR CONTINUOUS FRAMING

Cover plates are sometimes used in connection with rolled beams in order to increase the strength (S) or stiffness (I) properties of the beam.

Unless minimum weight is a real factor, the use of cover plates on simply supported beams might not be justified in building construction since the savings in steel might not offset the additional cost of fabricating and welding the cover plate to the beam. This is because the cover plate must extend quite a distance to both sides of the beam centerline. Notice in the example shown for uniform loading, Figure 70 (a), that the cover plate must extend 70.7% of the beam's length (c). Because of this great length, the weight reduction is only 8.7%.

On continuous girders and beams, however, there is a real advantage in using cover plates since the increased section produced needs to extend only a very short distance in from each end of the beam, Figure 70 (d). In the example shown, the total length of cover plate is just 18.3% of the length of the beam (f). Here weight reduction in applying cover plates to the continuous beam is 29.8%.

Additional weight reduction is achieved in going from the simply supported beam to the continuous beam with fixed ends. When considering this in the example below, of going from a simply supported beam to the continuous beam with cover plates, the over-all weight reduction in the beam becomes 35.8%.

Constants to Help Calculate Final Moments

Charts have been developed by which the designer can readily find constants to use in determining stiffness factors, carry-over factors, and fixed-end moments for beams in which there are abrupt changes in moment of inertia due to welded cover plates.

Sources include:

(1) Bull. 176, R. A. Caughy and R. S. Cebula: Iowa Engineering Experiment Sta., Iowa State College, Ames, Iowa. 36 charts for beams with cover plates at ends. Also reprinted as Structural Study 1302.150, The Lincoln Electric Co.

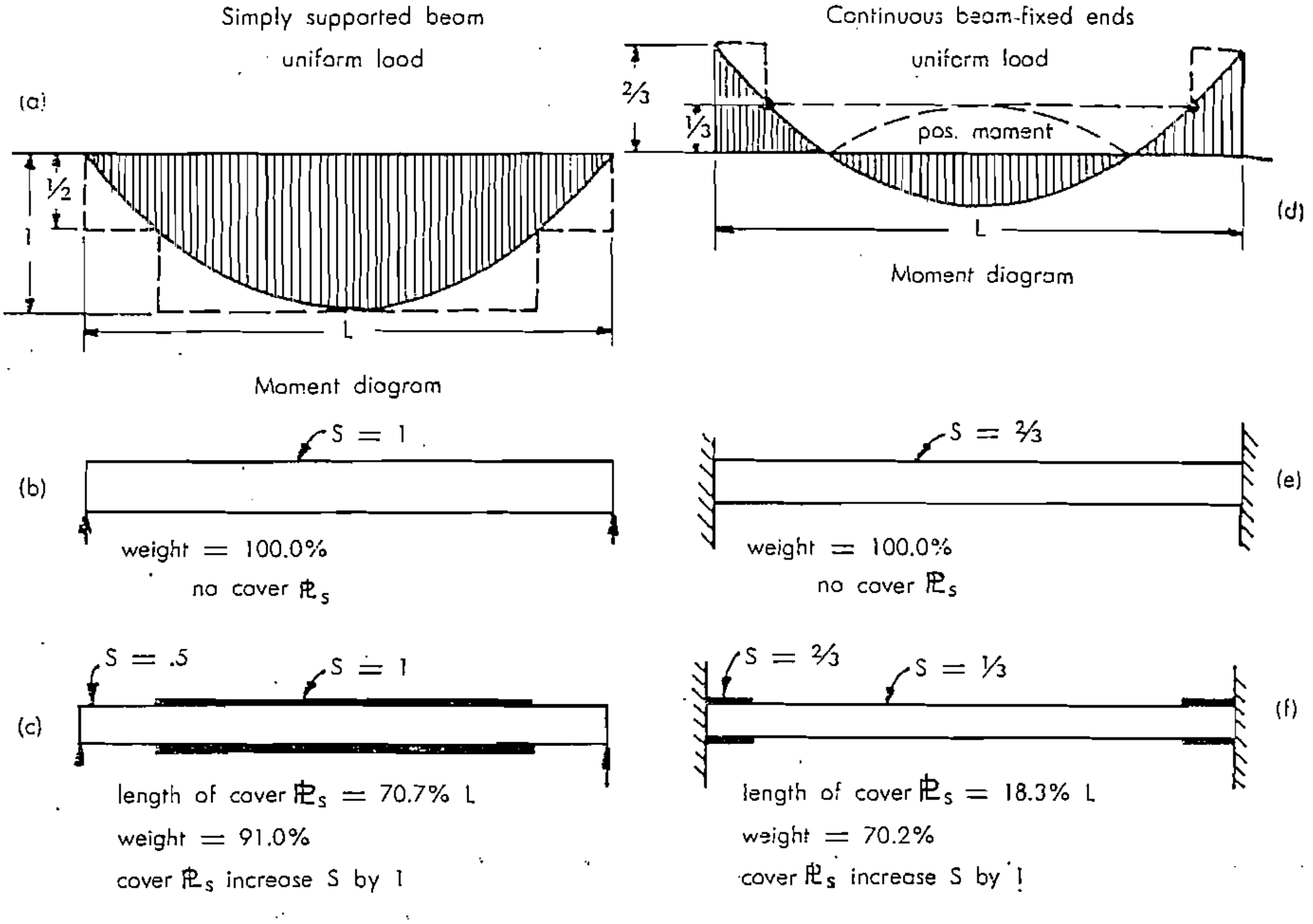


FIGURE 70

(2) "Moment Distribution", J. M. Gere, 1963; D. Van Nostrand Co. 29 charts for beams with cover plates at ends; 42 charts for tapered beams.

For methods of calculating these design factors, see Section 6.1, on Design of Rigid Frames.

Example

A frame is to be designed to support a uniform load of 2.4 kips/ft. Three spans of 20′ each are supported by four columns, 12′ high. The beams are 12″ WF 27# beams, reinforced with ⅜″ x 5″ cover plates for a distance of 2′ on each side of the interior supports. The columns are 8″ WF 31# sections. See Figure 71.

The section properties of the rolled beam, Figure 72, without and with cover plates are as follows:

beam only

$$I_1 = 204.1 \text{ in.}^4$$

$$S_1 = 34.1 \text{ in.}^3$$

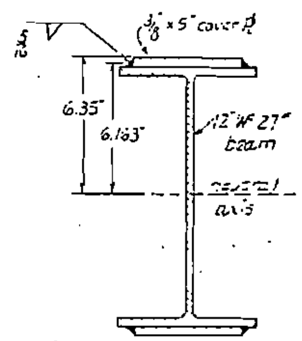


FIGURE 72

beam with cover plates

$$I_2 = 204.1 + 2\ (\tfrac{3}{8} \times 5)(6.163)^2$$

$$= 346 \text{ in.}^4$$

$$S_2 = \frac{I_2}{c_2}$$

$$= \frac{346}{6.35}$$

$$= 56 \text{ in.}^3$$

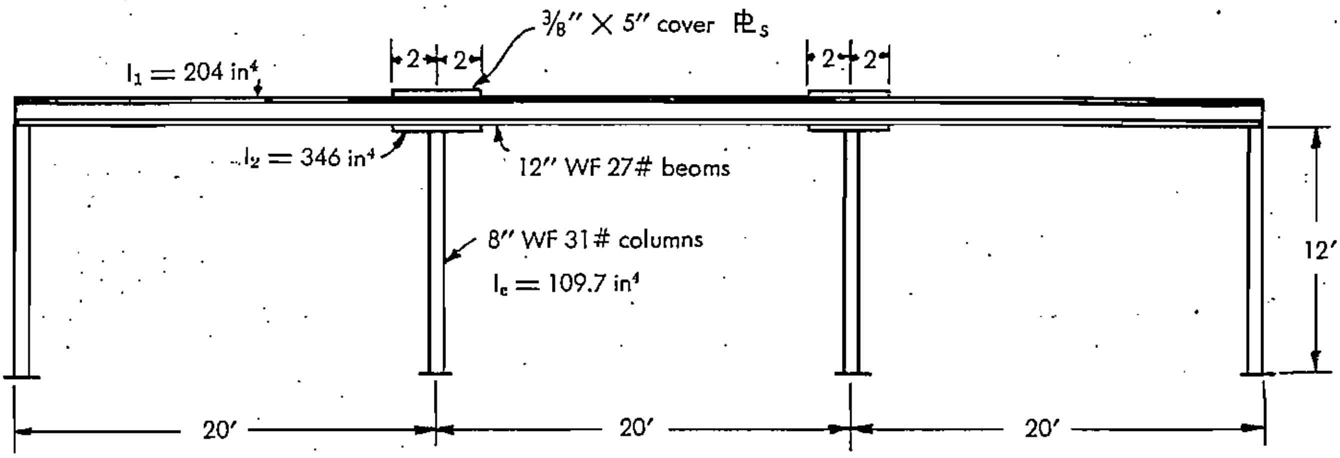
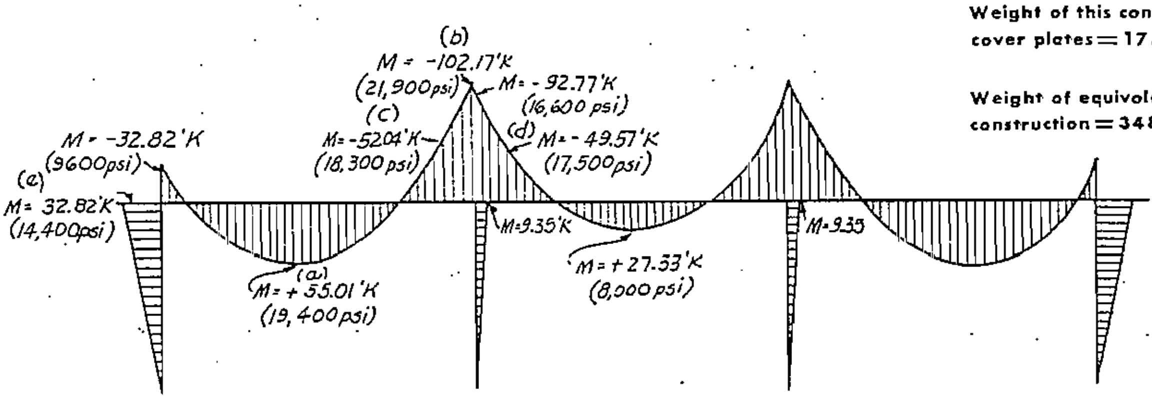


FIGURE 71



Weight of this continuous beam with cover plates = 1750 lbs.

Weight of equivalent simple beam construction = 3480 lbs.

FIGURE 73

14. EXAMPLES OF CONTINUOUS CONNECTIONS

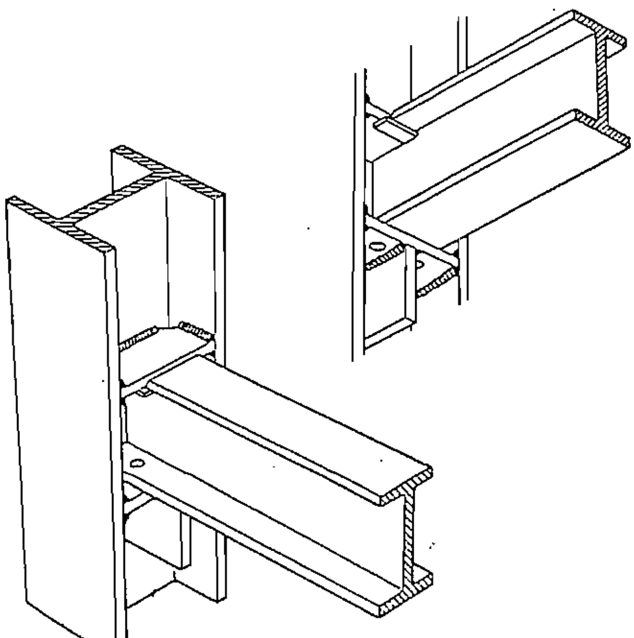


FIGURE 74 (a)

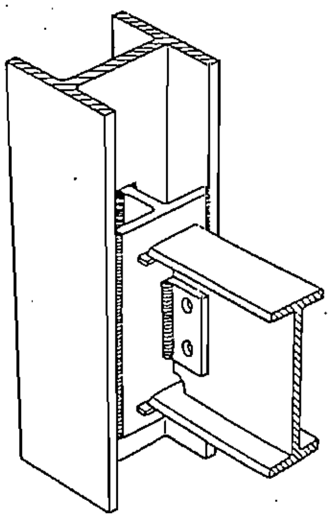


FIGURE 74 (b)

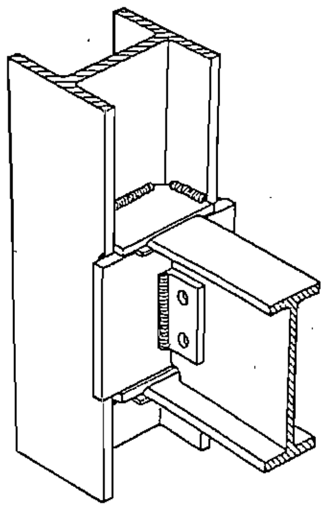


FIGURE 74 (c)

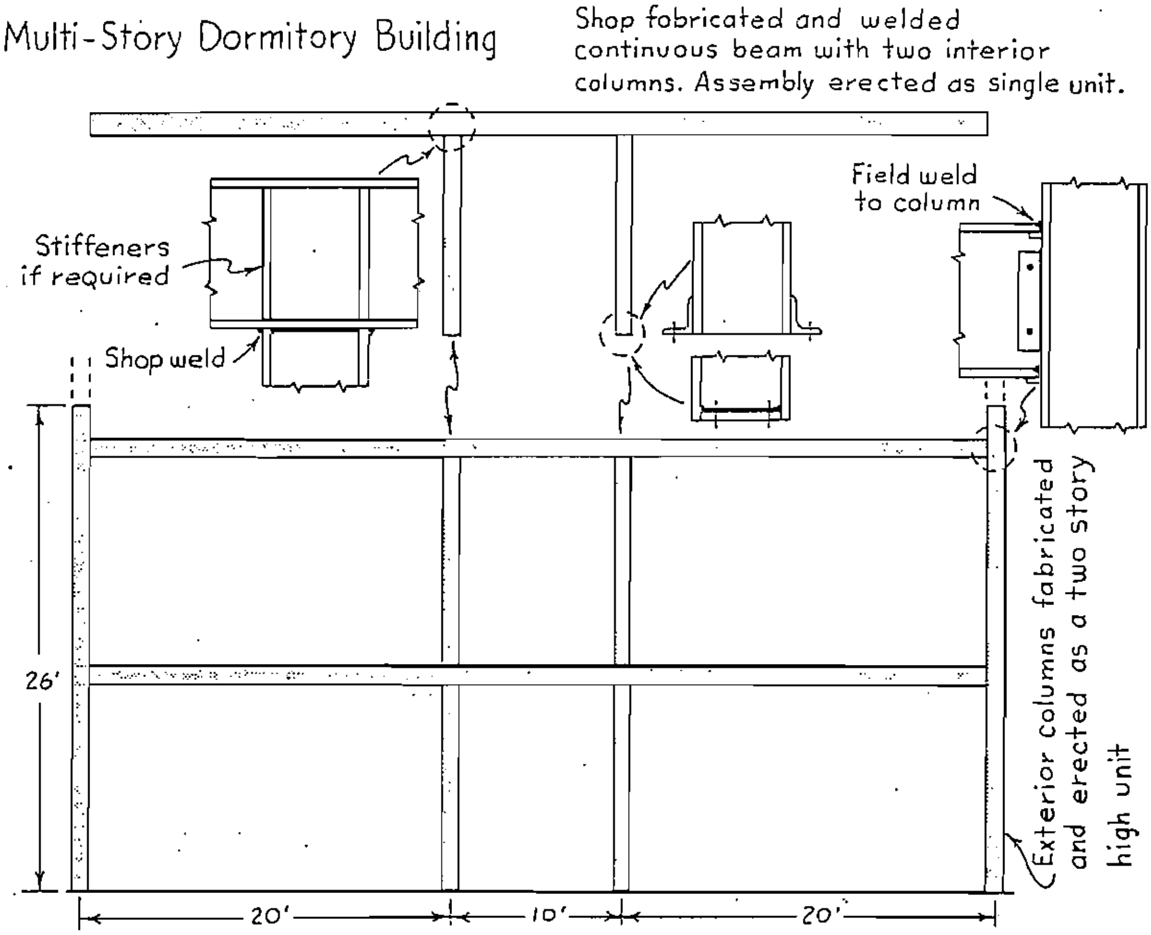


FIGURE 75

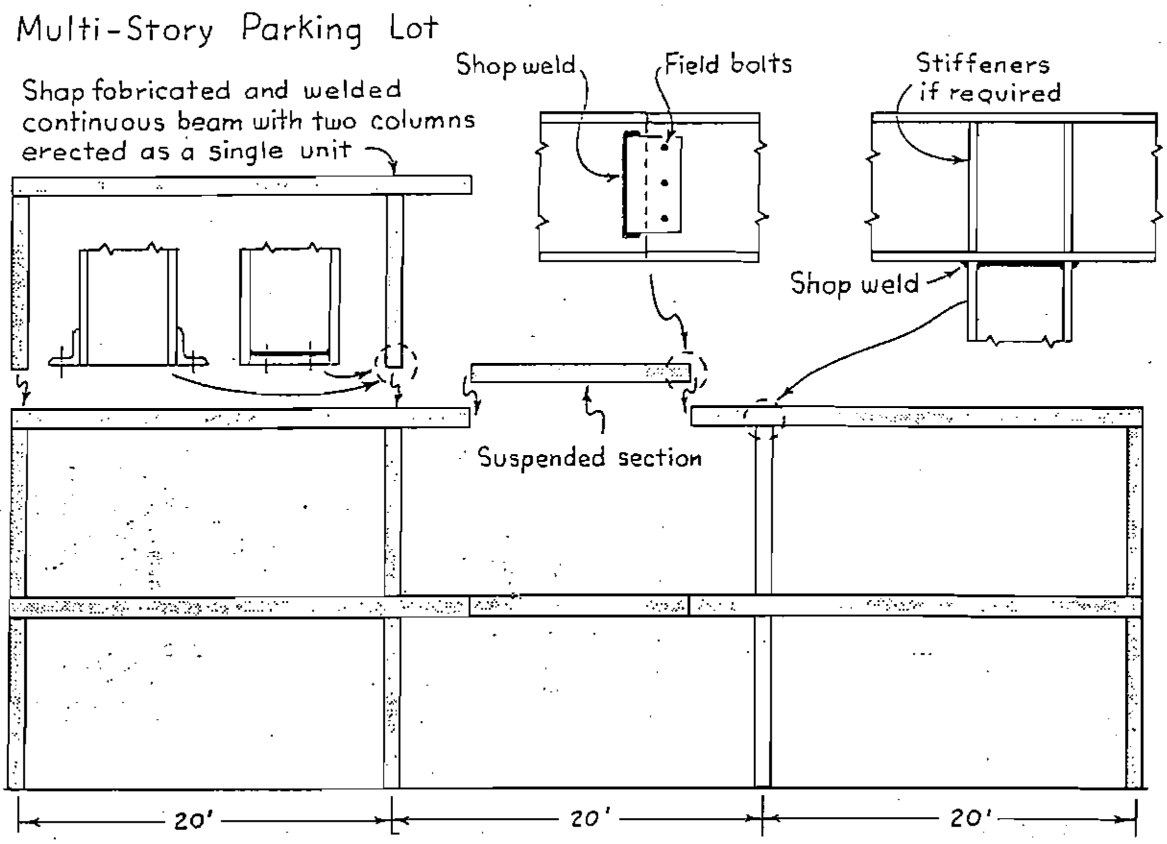
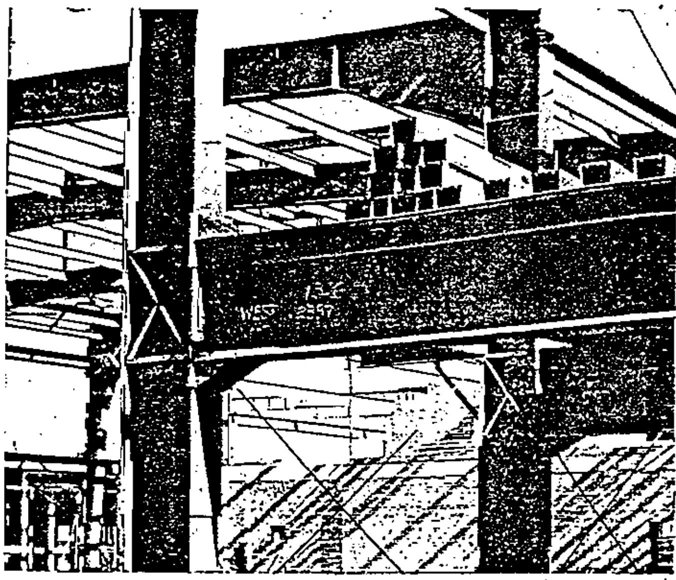
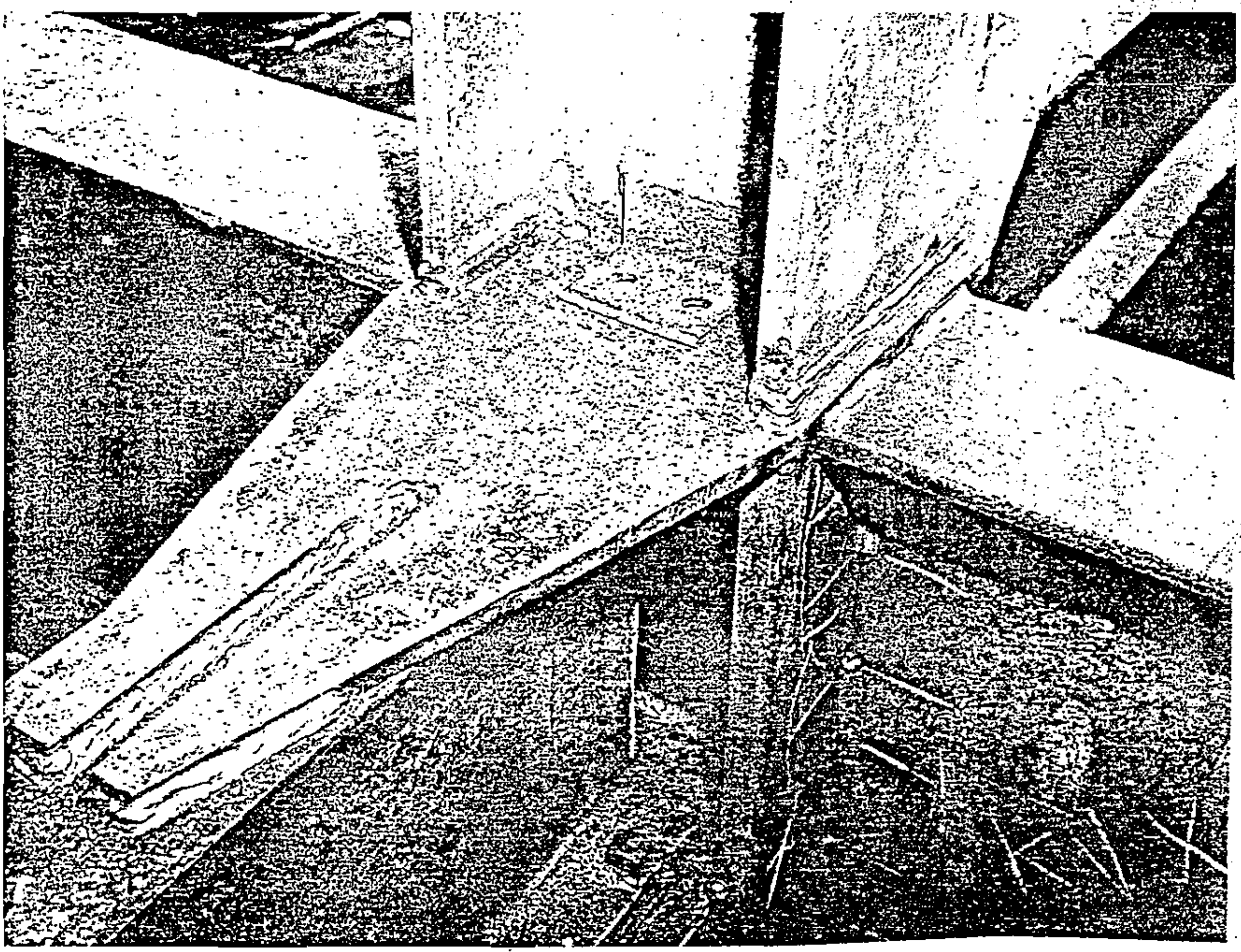


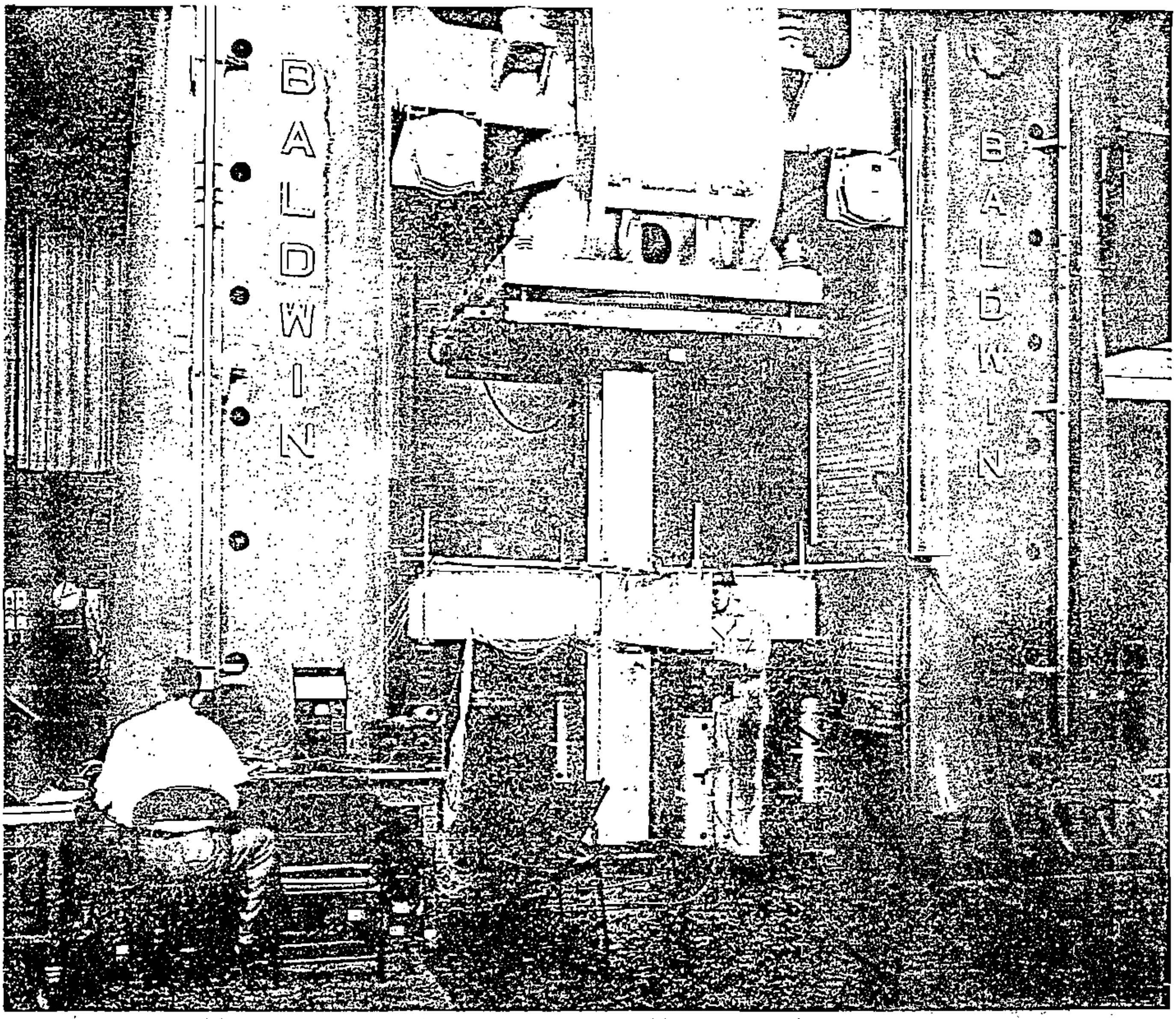
FIGURE 76



Girder terminating at a column and not continuing through loads the column web in shear in the region of the beam connection. This causes high diagonol compressive stresses, and diagonal stiffeners are used to resist the tendency of the web to buckle.



Typical column joint to develop continuity in both directions. The column is cut off at this point. The moin girder (left to right) has 100% continuity, no joint; column stiffeners on girder webs are shop welded. The cross beams are provided continuity by the use of a welded top plate extending right ocross the upper girder flange. The column for the floor above is positioned on top of this connecting plate, temporarily held by angles shop-welded to the column web, and then permanently field welded along the flanges to the connecting plate.



Actual service conditions on beam-to-column continuous connections were simulated in this experimental setup at Lehigh University's Fritz Engineering Laboratories. Here, the column is subjected to compressive axial load by the main press ram while the beam stubs are loaded individually by means of hydraulic cylinders.