Beam-to-Column Connections

1. TYPES OF DESIGN

AISC Specifications permit four types of design and design assumptions in steel construction. Beam-to-column connections can be categorized accordingly; see Figure 1. The four types of design:

		Restraint (R)
(a) simple frame	AISC Type 2	below approx. 20%
(b) fully rigid frame	AISC Type 1	above approx. 90%
(c) semi-rigid frame	AISC Type 3	approx. 20 to approx. 90%
(d) plastic design	AISC Plastic Design	

Here the degree of restraint (R) is the ratio of the actual end moment (assuming no column rotation) to the end moment in a fully fixed end beam.

These various connections are discussed comparatively here in Section 5.1, but details of their design are presented more comprehensively in later sections.

2. SIMPLE BEAM CONNECTIONS

The most common types of simple beam connections use web framing angles, or a top connecting angle with the beam supported on a seat.

The connection is designed to transfer the vertical shear reaction only, it being assumed there is no bending moment present at the connection. However the simple beam, under load, will deflect, causing the ends to rotate slightly. The connection must be designed to rotate this amount without failure, and to flex enough to keep the end moment from building up to any appreciable amount. This is sometimes referred to as a "flexible connection."

A top connecting plate is sometimes used for simple beams. In this case the end of the plate is beveled and butt welded to the column, the plate and connecting weld being designed to develop about 25% of the resisting moment of the beam at the standard allowable bending stress. Just beyond this weld, the plate is reduced in cross-section to produce yield stress at this load. The length of the reduced section should be about 1.2 times its width to allow the plate to yield plastically without failure. This plate is attached to the beam flange with a continuous fillet weld across the end and returning for a sufficient length on both sides to develop the strength of the butt weld at standard allowables.

Wind moments applied to these connections present an additional problem. Some means to transfer these wind moments must be provided in a connection that is supposed to be flexible. Any additional restraint in the connection will increase the end moment resulting from the beam load. AISC (Sec. 1.2) provides for two approximate solutions:

Method 1. Designing the top plate for the force resulting from the moment caused by the combination of gravity and wind loads at a ⅓ increase in the stress allowables. This same ⅓ increase may also be applied to the connecting welds (AISC Sec. 1.5.3 and 1.5.6).

Method 2. Designing the top plate to carry the force resulting from the calculated wind moment at a ⅓ increase in the allowable stress. The top plate should be capable of safely yielding plastically within the unwelded length for any combination of gravity and wind loads causing it to be stressed above its yield point, thus relieving these additional moments. The connecting welds are designed for standard allowables when plate is at yield stress.

This type of connection may necessitate some non-elastic but self-limiting deformation of the connecting plate, but under forces which do not overstress the weld (AISC Sec. 1.2).

3. SEMI-RIGID CONNECTIONS

Semi-rigid connections are very intriguing, but unfortunately are many times misleading.

The disadvantage of a simply supported beam is that the entire moment requirement is at one portion of the beam, its central section having the greatest moment: $M = \frac{1}{8}\,W\,L$ (if uniformly loaded).

A rigid, fixed-end connection reduces the moment at the central portion of the beam, with a corresponding increase in the moment at the ends. The moment reduction at the center is added to the ends. As the degree of restraint or rigidity of the connection increases, the center moment decreases, and the end moments increase. Fully rigid connections for uniformly loaded beams result in a center moment of $M = \frac{1}{24}$

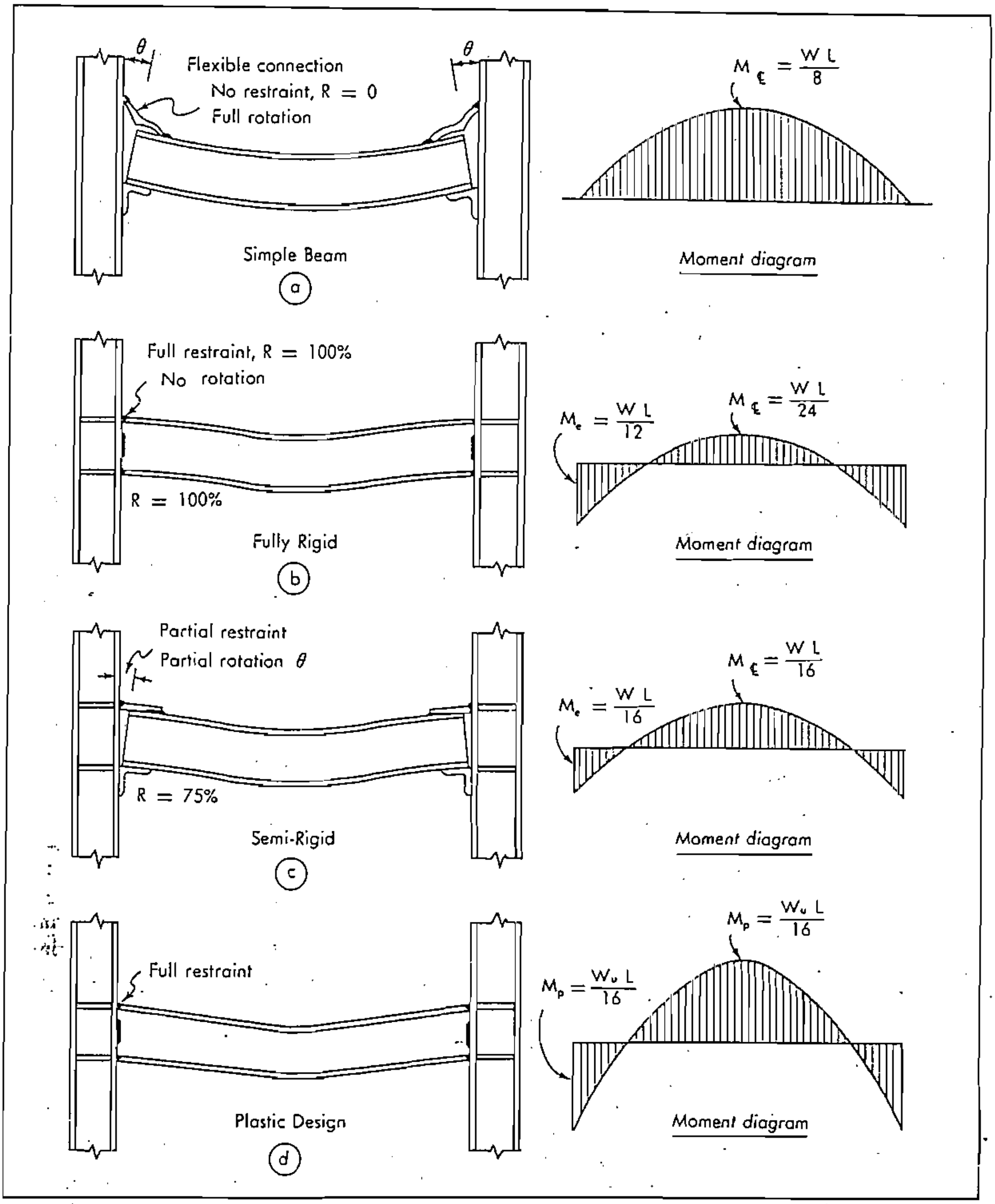

FIGURE 1

W L, and the end moment of $M = \frac{1}{12}$ W L. Thus, the beam needs a section modulus just ⅔ of that required for a simply supported beam using flexible connections.

Advocates of semi-rigid connections point out that the above redistribution of moment has been carried a little too far. They advise that if a semi-rigid connection is used instead, having an end restraint of R = 75%, both the center moment and the end moments would be equal, or $M = \frac{1}{16}$ W L. This would produce the least requirement for section modulus, being ½ of that needed for the original simply supported beam. This is true, but this ideal condition depends on two requirements:

1. The supports to which the connection joins the beam must be unyielding, i.e. absolutely rigid.
2. The beam must not be influenced by adjacent

spans from which additional moments might be carried over through the connection.

This condition of R = 75% restraint does produce the minimum section modulus for the uniformly loaded beam, but it does not offer any leeway or range of connection rigidity. If the resulting connection should be a little too rigid (anything over 75%), the end moment increases above the allowable; if the connection is a little too flexible (anything below 75%), the center moment increases above the allowable. To take care of this, it is usually suggested that the beam be designed for an end restraint of R = 50% (center moment of $\frac{1}{12}$ W L), and the connection for a restraint of R = 75% (end moment of $\frac{1}{16}$ W L). This appears to be good until it is remembered that this resulting design moment of $\frac{1}{12}$ W L is no lower than if fully rigid welded connections were used; so, there is no saving in beam requirements by using the semi-rigid connections.

It could be argued that this semi-rigid connection results in a slight reduction in the amount of connecting weld. This might be true if the fully rigid connection used a top connecting plate (groove weld to the column and fillet weld to the beam), but would not be true if the beam flanges for the rigid connection were groove welded directly to the supporting column without the additional fillet welds.

Although perhaps not intended, most structural texts, and other literature on the subject, imply that the engineer simply takes each span one at a time and designs the beam for an end restraint of R = 50% and the connection for a restraint of R = 75%. It wouldn't be difficult to calculate the cross-sectional area and the length of reduced section of the proper top connecting plate for this semi-rigid connection to arrive at the actual required restraint.

Those who voice the apparent advantages of semi-rigid connections seldom discuss how to apply them to actual frames of several spans and stories and different span loadings.

In frames using fully rigid, welded connections, the resulting moments must be found. For example, if the moment distribution method is used, the end moments must be determined for each span, treating each as an isolated, fixed-end beam. A distribution factor is required for each member so that the unbalanced moment at each joint may be properly distributed about the various members connecting at a given joint. Carry-over factors are needed to determine the amount of the unbalanced moment to be carried over to the opposite end of the member.

For a frame using semi-rigid connections, all of these factors (end moments, distribution factors, and carry-over factors) will be affected by the connection's degree of rigidity. This would make the analysis more complicated.

AISC allows semi-rigid connections only upon evidence that they are capable of resisting definite moments without overstressing the welds. Design of the members so connected shall be based on no greater degree of end restraint than the minimum known to be effective.

This type of connection may necessitate some non-elastic but self-limiting deformation, but under forces which do not overstress the weld (AISC Sec. 1.2).

4. RIGID CONNECTIONS (Elastic Design)

For fully rigid connections the actual moments must be found by one of several methods, and the beams and their connections designed for the proper moments and shear forces. The connections must have sufficient rigidity to hold virtually unchanged the original angles between connecting members.

The rigidity of a connection is also influenced by the rigidity of its support. For beams framing into column flanges, a decrease in rigidity will occur if the column flanges are too thin, or if stiffeners are not used between the column flanges in line with the beam flanges. For a single beam framing into a column web, a decrease in rigidity may occur unless the beam flange is also welded directly to the column flanges or attached with suitable connecting plates.

5. PLASTIC-DESIGN CONNECTIONS

The use of welded connections based on plastic design has several advantages:

1. A more accurate indication of the true carrying capacity of the structure.
2. Requires less steel than conventional simple beam construction. In many cases, there is a slight saving over conventional elastic design of rigid frames.
3. Requires less design time than does elastic design of rigid frames.
4. Tested by several years of research on full-scale structures.
5. Backed by the AISC.

So far, plastic design connections have been largely restricted to one-story structures, and to applications where fatigue or repeat loading is not a problem. See separate Sect. 5.12 in this manual for a full discussion of Welded Connections for Plastic Design.

6. BEHAVIOR OF WELDED CONNECTIONS

One way to better understand the behavior of a beam-to-column connection under load, and its load-carrying capacity, is to plot it on a moment-rotation chart; see Figure 2.

The vertical axis is the end moment of the beam,

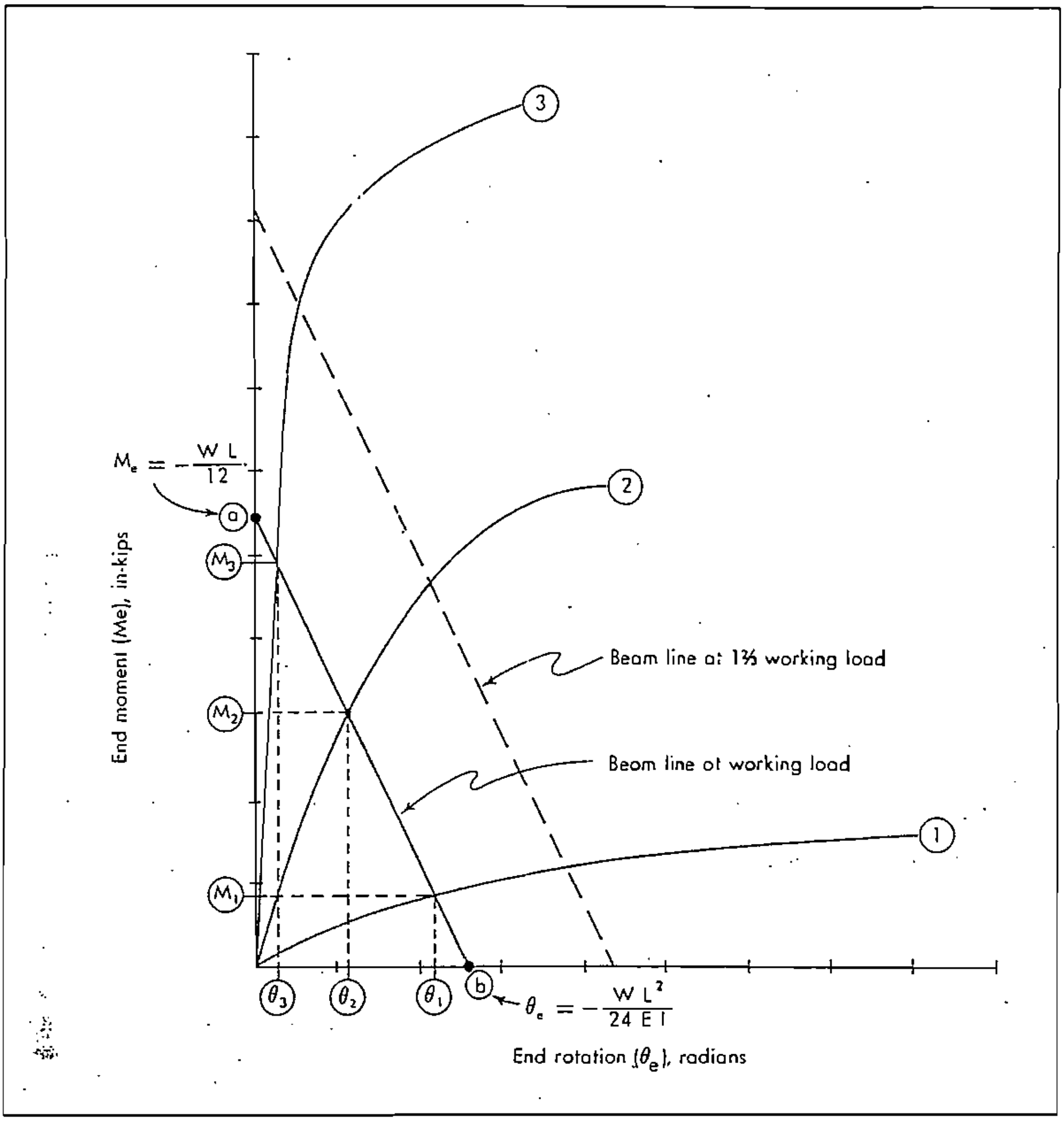

FIGURE 2

which is applied to the connection. The horizontal axis is the resulting rotation in radians. Basically this is another type of stress-strain diagram.

Superimposed upon this is the beam diagram. The equation expressing the resulting end moment (M_e) and end rotation (θ_e), for a uniformly loaded beam and any end restraint from complete rigid to simply supported, is:

$$\boxed{M_e = -\frac{2\,E\,I\,\theta}{L} - \frac{W\,L}{12}}$$

This is a straight line, having points *a* and *b* on the chart.

Point *a* is the end moment when the connection is completely restrained ($\theta_e = 0$), in other words a fixed-end beam, and is equal to—

$$(a)\quad M_e = -\frac{W\,L}{12}$$

Point *b* is the end rotation when the connection has no restraint ($M_e = 0$), in other words a simple beam, and is equal to—

$$(b)\quad \theta_e = -\frac{W\,L^2}{24\,E\,I}$$

For increased loads on the beam, the beam line moves out parallel to the first line, with correspondingly increased values of end moment (M_e) and the end rotation (θ_e). This (dashed) second beam line on the chart represents the addition of a safety factor, and is usually 1.67 to 2 times that of the first which is based on the working load.

The point at which the connection's curve intersects the beam line, gives the resulting end moment and rotation under the given load. From this it is seen how the beam's behavior depends on its connection.

It is assumed, in this case, the beam is symmetrically loaded and the two end connections are the same. In this way both ends will react similarly.

Curve 1 represents a flexible connection. At a very low moment it safely yields (M_1) and allows the connection to rotate (θ_1). This is typical of top angle connections, web framing angles, and top plate connections small enough to yield. Notice, even with these so-called flexible connections, some end moment does set up.

Curve 2 represents a semi-rigid connection. One type is the top connecting plate so detailed that under working load it elastically yields sufficiently to provide the necessary rotation of the connection, and yet has sufficient resistance to develop the proper end moment. Although thick top angles have been suggested for service as semi-rigid connections, they are impractical to design and fabricate with the desired built-in restraint.

Curve 3 represents a rigid connection, using a top connecting plate detailed to develop the full end moment. Since no elastic yielding is needed or desired, the plate is made as short as practical.

All three of these connections have ample reserve carrying capacity, as shown by where their curves intersect the beam line at 1⅔ load relative to their crossing of the beam line at working load.

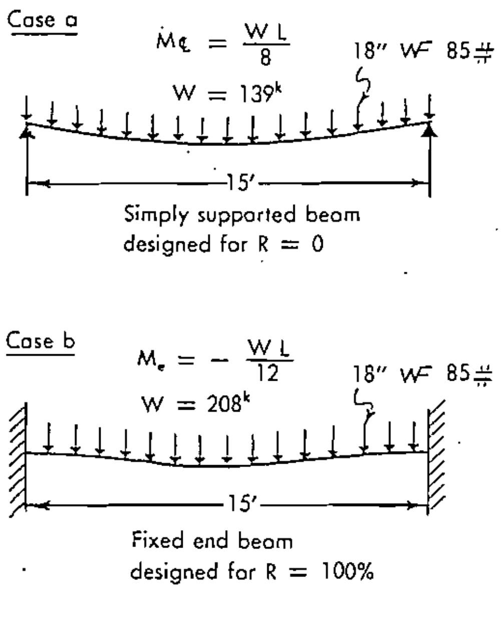

FIGURE 3

The actual results of testing three top plate connections on an 18″ WF 85# beam are shown in Figure 4. Two conditions are considered, as shown by the load diagrams, Figure 3.

Beam line a (in Figure 4) is based on a design moment of ⅛ W L at centerline, i.e. simply supported. Beam line a_1 is for a load 1⅔ times that of the working load.

Beam line b is based on a design moment of $\frac{1}{12}$ W L at the ends, i.e. fixed ends, and will support a 50% greater load. Beam line b_1 is for a load 1⅔ times that of the working load. Both of these two beam lines stop at R = 50%, because at this restraint the center of the beam now has this moment of $\frac{1}{12}$ W L and a restraint lower than this would overstress the central portion of the beam.

Top plate #1 is a $\frac{5}{16}$″ thick plate, 3″ wide at the reduced section, and has a cross-sectional area of $A_p = .94$ in.^{2} It is widened to 6″ at the butt-welded connection. This connection should reach yield at about $M = A_p\,\sigma_y\,d_b = (.94)\ (33{,}000)\ (18) = 558$ in.-kip. The actual value from the test is about M = 600 in.-kip. Above this moment, the plate yields and due to strain hardening will have increased resistance. The ultimate moment should be about twice this yield value, or about M = 1200 in.-kip. The resulting restraint is about R = 34.5%, a little too high for the beam to be classed as simply supported.

Top plate #2 has the same $\frac{5}{16}$″ thickness, but has a 6″ width throughout its length. It has double the cross-sectional area, $A_p = 1.88$ in.^{2} As expected, it is twice as rigid. It should reach yield at about M = 1110 in.-kip. The actual is about M = 1000 in.-kip. The restraint is about R = 58%. Notice if the beam had been designed for a moment of $\frac{1}{12}$ W L, i.e. a restraint of R = 100%, the connection's curve would have intersected the beam line b just short of the R = 50% value. There would then be a slight overstress of the beam at centerline.

Top plate #3 is ⅞″ thick and 7½″ wide, having a cross-sectional area of $A_p = 6.56$ in.^{2} This greater area produces a more rigid connection with greater restraint. The actual connection curve (solid) shows slightly more flexibility than the calculated curve (dotted). The extra flexibility probably comes from some movement of the lower portion of the connection which has just short parallel fillet welds joining the lower flange of the beam to the seat. A butt weld placed directly across the end of this lower flange to the column, undoubtedly would bring the rigidity of the connection curve up almost to that of the calculated curve.

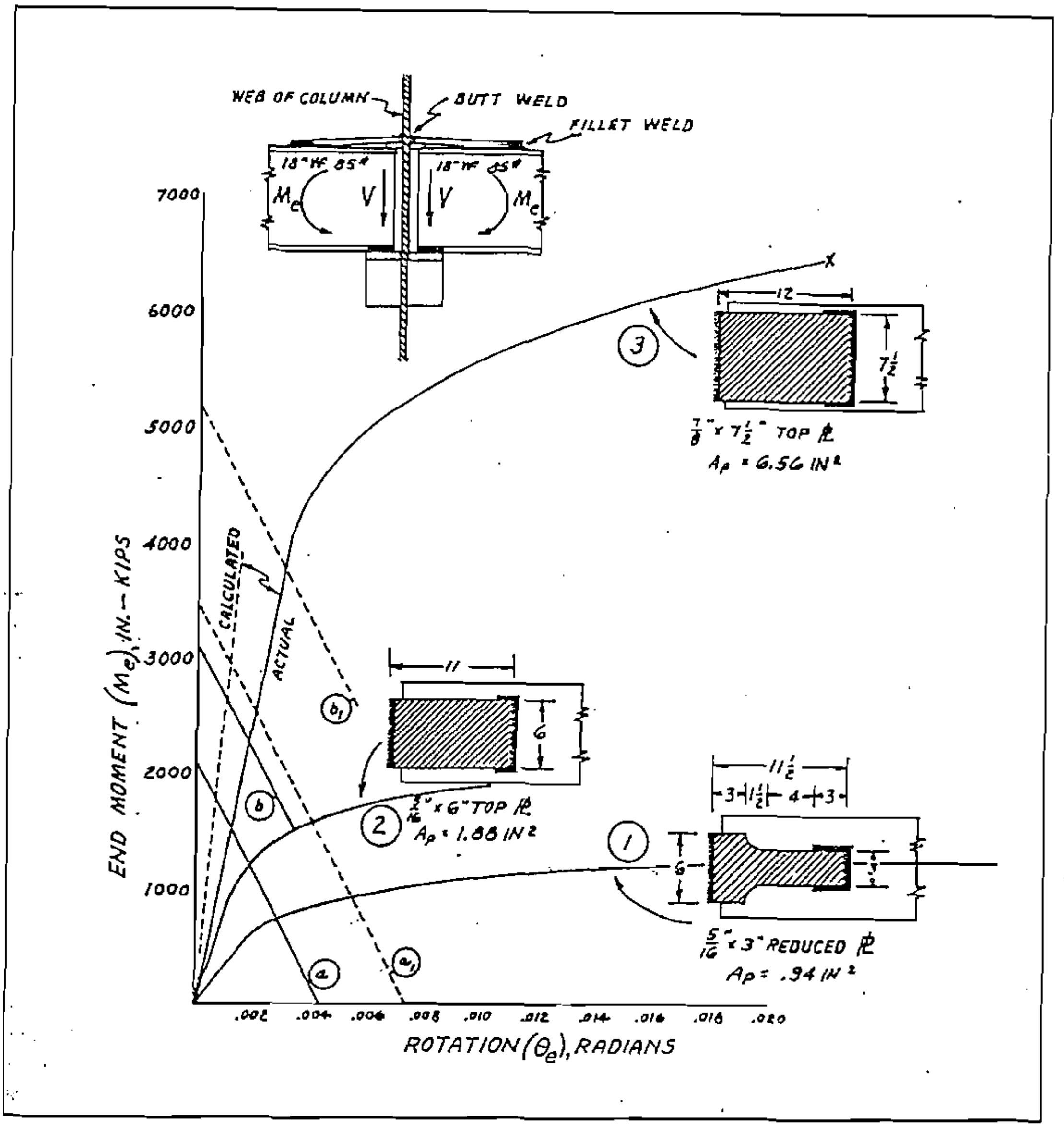

FIGURE 4

Figure 4 from: "Tests of Miscellaneous Welded Building Connections"; Johnston & Diets; AWS Welding Journal Jan. 1942 and "Report of Tests of Welded Top Plate and Seat Building Connections"; Brandes & Mains; AWS Welding Journal Mar. 1944

Figure 5 illustrates the additional restraining action provided by column flange stiffeners. Both connections use $\frac{5}{16}$" x 6" top plates.

Connection #1 has column stiffeners. In the case of the beam designed for a moment of $\frac{1}{12}$ W L (R = 100% down to R = 50%), it would supply a restraint of about R = 70.2%.

Connection #2 has no column stiffeners and loses sufficient rigidity so that the beam designed for a moment of $\frac{1}{12}$ W L (R = 100% down to R = 50%) will be overstressed. This is because the connection restraint would be only about R = 45%.

This shows the importance of proper stiffening.

7. FACTORS IN CONNECTION DESIGN

The following items greatly affect the cost of welded structural steel and cannot be overlooked. In order to take full advantage of welded construction, they must be considered.

Moment Transfer

The bending forces from the end moment lie almost entirely within the flanges of the beam. The most effective and direct method to transfer these forces is some type of flange weld. The relative merits of three types are discussed here.

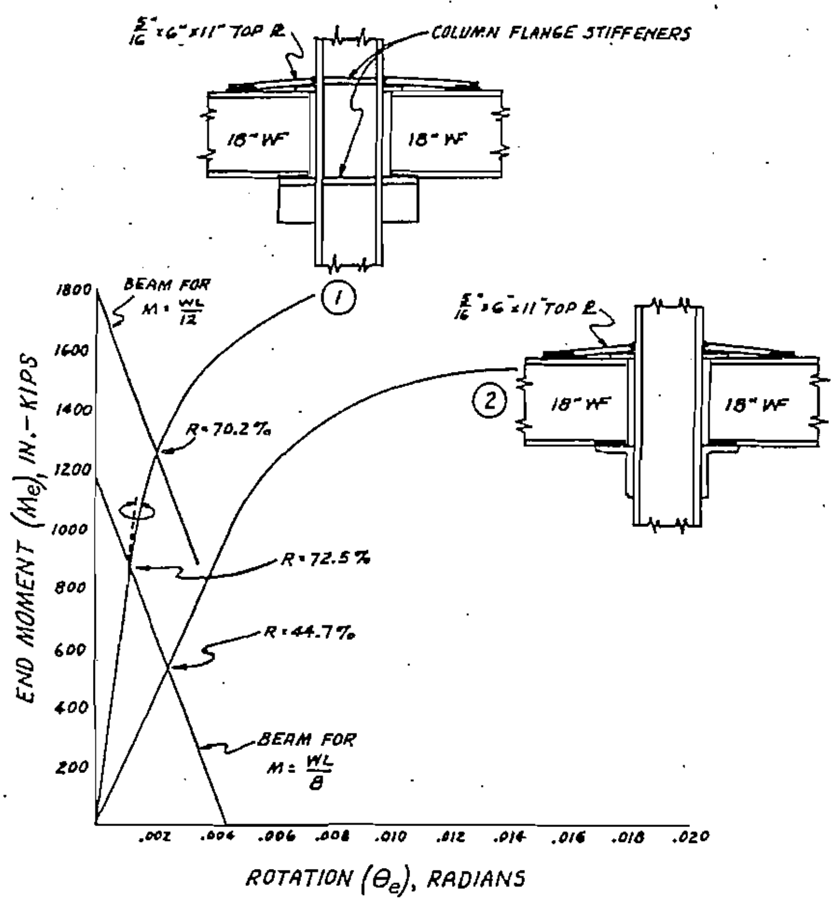

FIGURE 5

Figure 5: From "Tests of Miscellaneous Welded Building Connections"; Johnston & Deits; AWS Welding Journal, Jan. 1942

In Figure 6, the flanges are directly connected to the column by means of groove welds. This is the most direct method of transferring forces and requires the least amount of welding.

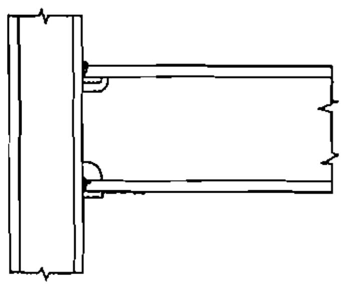

FIGURE 6

The backing strip just below each of the flanges allows the weld to be made within reasonable fit-up, as long as there is a proper root opening.

There is little provision for over-run of the column dimensions which may be as much as ± ⅛". For excessive over-run, the flanges of the beam may have to be flame-cut back, in the field, in order to provide the minimum root opening. For under-run, the excessive root opening will increase the amount of welding required, but the joint is still possible.

It is usually more costly to cut the beam to exact length; in addition there is the cost of beveling the flanges. Milling the beam to length is costly and not recommended because the over-run or under-run of the column having a tolerance of ± ⅛" would reduce this accuracy in fit-up.

In Figure 7, a top connecting plate is shipped loose and, for proper fit-up, is put in place by the weldor after the beam is erected. A greater tolerance can be allowed in cutting the beam to length, and any method can be used (circular cutoff saw, flame-cutting,

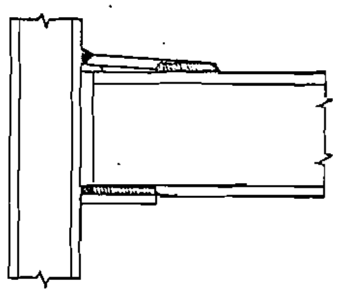

FIGURE 7

etc.) without subsequent beveling of the flanges. The beams frequently are ordered from the steel supplier cut shorter than required: ¾″ ± ¼″. Sometimes beams are ordered still shorter, allowing a cutting tolerance of ± ⅜″. This greatly reduces the cost of cutting and preparation.

This type of connection requires the extra connecting plate, which must be cut to size and beveled. It doubles the amount of field welding on the top flange. It also can interfere with metal decks placed on top of the beam. Occasionally, the top plate is shop welded to the top flange on one or both ends of the beam. This decreases the amount of field welding but eliminates the fit-up advantage.

The beam's bottom flange may be groove welded directly to the column if sufficient root spacing is obtained, even though the edge of the flange is not beveled. Although this is not an AWS Prequalified Joint, it is widely used, perhaps because the bottom flange weld is in compression. One disadvantage is that the beam length must be held accurately. As the beam flange increases in thickness, the required root spacing must increase. The bottom seat also serves as a backing strip. Sometimes for additional strength, the flange is fillet welded to this plate for a short distance along its

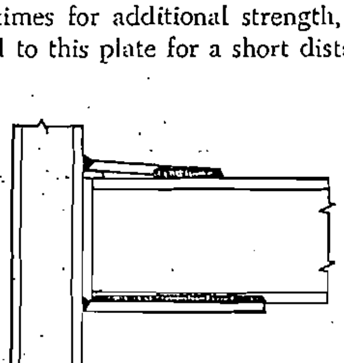

FIGURE 8

length. All of this field welding is done in the flat position.

In Figure 8, the lower flange is not groove welded directly to the column; instead, the bottom seat plate is extended farther along the beam, and is fillet welded to the beam flange. These welds are designed to transfer the compressive force of the flange back into the column. All of the field welding is done in the flat position. This connection requires a little more care in handling and shipping so that these longer plates are not damaged. This also requires a little more weight of connecting material.

A beam that is "compact" (AISC Sec. 2.6) permits a 10% higher bending stress, $\sigma_b = 0.66\ \sigma_y$. However, to take advantage of this higher bending allowable in the connection, it is necessary that the web be welded almost its full depth to the support. Thus, it might be possible to stress the entire depth of the web to yield (σ_y) in bending to develop the plastic moment (M_p), (AISC Sec. 1.5.1.4.1). These same beams if continuous over supports or rigidly framed to columns, may be proportioned for 90% of the negative moment provided the maximum positive moment is increased by 10% of the average of the two negative moments (AISC Sec 1.5.1.4.1).

Check to see if stiffeners between the column flanges are necessary. Recent research indicates web crippling is the deciding factor and if the column web has sufficient thickness, stiffeners are not required.

If flange stiffeners are required, consider whether they can be fillet welded to the column. Usually the groove type of T joint is detailed, i.e. the stiffener has a single 45° bevel all the way around three sides. If the fillet weld has a leg size of about ¾ the stiffener thickness, it will develop full plate strength. Both of these joints would require about the same amount of weld metal. The single bevel joint requires extra fitting, a lower welding current and smaller electrodes for the first few passes. The groove joint in this case is not very accessible for the weldor and presents an additional problem because it is difficult to get down in between the column flanges to do this welding. On this basis, fillet welds would probably cost less and be easier to use.

Double bevel joints require about half as much welding as the fillet welded joint; but unless the stiffeners are extremely thick, perhaps above 1½″, fillet welds would still be the lowest in cost and trouble.

Consider the use of iron powder manual electrodes or the semi-automatic submerged-arc process for flat welding in the field as well as in the shop.

Shear Transfer

The shear forces lie almost entirely within the web of the beam and must be 1) transferred directly out to the supporting column by means of a connection on the web, or 2) directly down to a supporting seat.

The web connection must have sufficient vertical weld length so as not to overstress the beam web in shear. The seat connection must have sufficient horizontal length so as not to overstress the beam web in compression or bearing.

In Figure 9, the vertical reaction of the beam is carried by a weld connecting the beam web to an attaching plate. This plate, which was shop welded to the column, is used also for the erection bolts. This method of shear transfer not only requires a field weld, but the weld must be made in the vertical position at lower speeds.

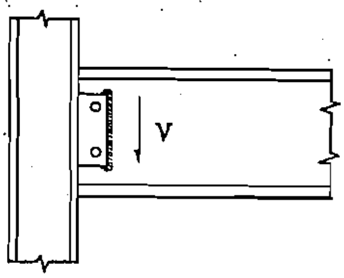

FIGURE 9

In Figure 10, the beam fits close enough for its web to be fillet welded on both sides directly to the column. The length of these welds is determined by the shear reaction to be transferred. This method of transfer also requires a field weld in the vertical position at lower welding speeds.

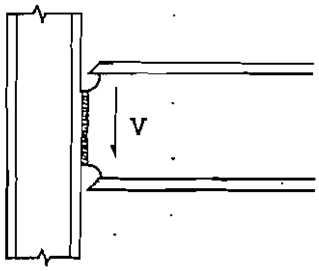

FIGURE 10

Vertical welding in the field increases the cost of the joint and should be eliminated if possible.

In Figure 11, the stiffened seat bracket has sufficient welding to transfer the shear reaction back into the column. This welding on the column is done in the shop in the flat position for the fastest welding speeds. It eliminates any out-of-position, more costly, welding in the field. The seat bracket serves as a support for the beam during erection, a place for the erection bolts through the bottom flange, and a means to carry the shear reaction. This bracket should not extend out too far, or it will interfere with any fireproofing or wall construction. The web of the beam sometimes is reinforced with an additional plate on the end to give it the necessary thickness for this reduced bearing length.

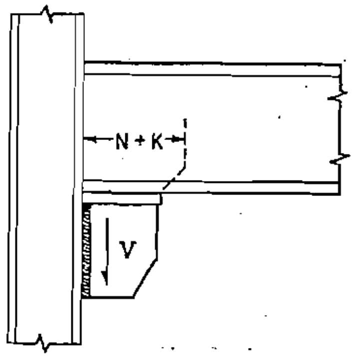

FIGURE 11

If some vertical welding in the field is still required, consider having one or more welding operators do this and other operators do the flat welding with the higher currents. This eliminates changing welding current and electrode size for the various positions of welding.

Erection Ease

The connection must allow rapid erection and fitting in place of the beam. It must provide temporary support for the dead load and some horizontal stability until the connection can be completed by welding.

If erection bolts are used, holes must be punched or drilled in the member. For beams and columns with thick flanges which exceed the capacity of the punch, these holes must be drilled; this is costly. It might be well to place these holes in the thinner web of the beam.

Where possible, use small attaching plates which may be punched while separate and then shop welded to the beam or column. This eliminates any need to move heavy members into another area for punching or drilling.

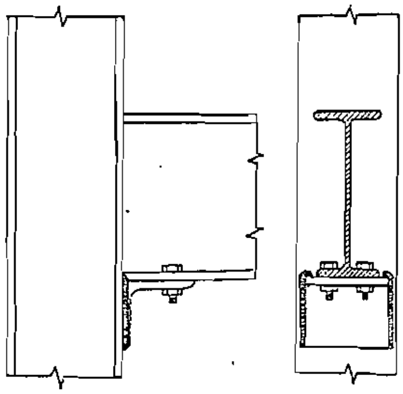

FIGURE 12

In Figure 12, a shop-welded seat provides support for the dead load of the beam. The beam is held in place by means of erection bolts through the bottom flange.

In Figure 13, a shop-welded plate on the column provides temporary support for the beam. Erection bolts

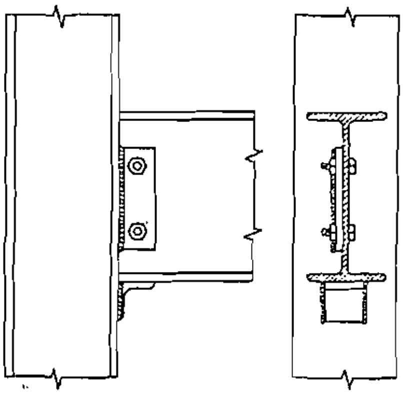

FIGURE 13

through the beam web hold the beam in position. An angle could be used instead of the plate. Although this would increase the material cost slightly, it would be easier to install and hold in proper alignment during welding. Sometimes a small seat is shop welded to the column, as shown, to give support while the erection bolts are being installed.

If the beam is supported on a seat, the elevation at the top of the beam may vary because of possible over-run or under-run of the beam. If the beam is supported by a web connection, this may be laid out from the top of the beam so as to eliminate this problem.

Saxe erection clips, Figure 14, are made of forged steel and are readily weldable. The clip is shop welded

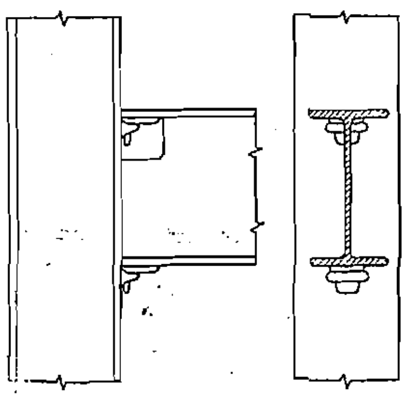

FIGURE 14

to the under side of the beam flange and the seat is shop welded in the proper position on the column. This allows the beam to slip easily into place during erection. One type of Saxe clip is adjustable and allows a movement of $\frac{3}{16}''$ as well as some rotation.

Consider the use of welded studs on main members in place of erection bolts; this will eliminate the punching of main members. These have already been accepted in the building and bridge fields for use as shear attachments, and an increasing number of fabricating shops have this equipment. See Figures 15, 16 and 17.

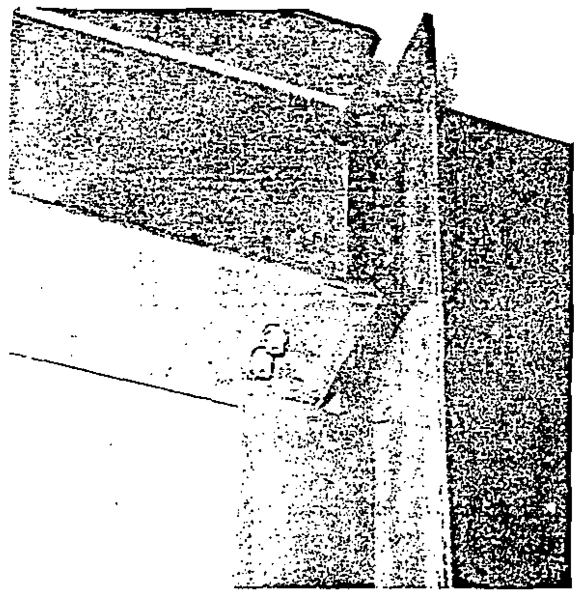

FIGURE 15

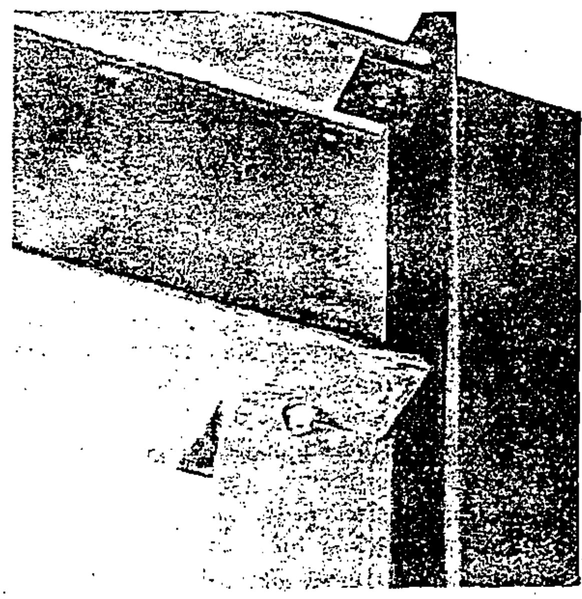

FIGURE 16

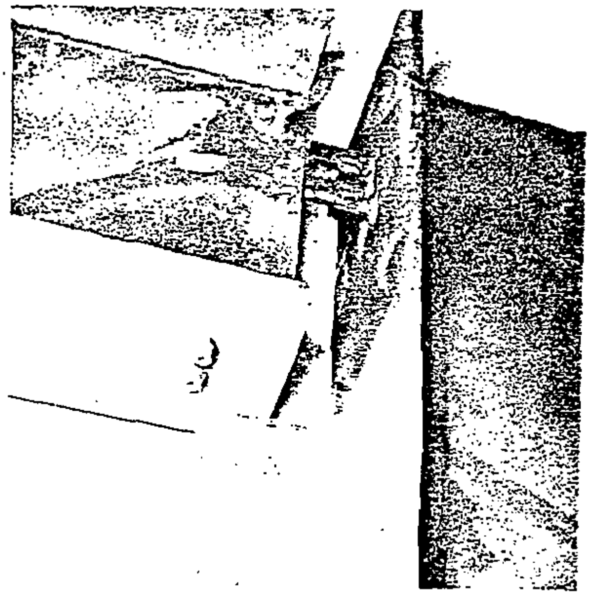

FIGURE 17

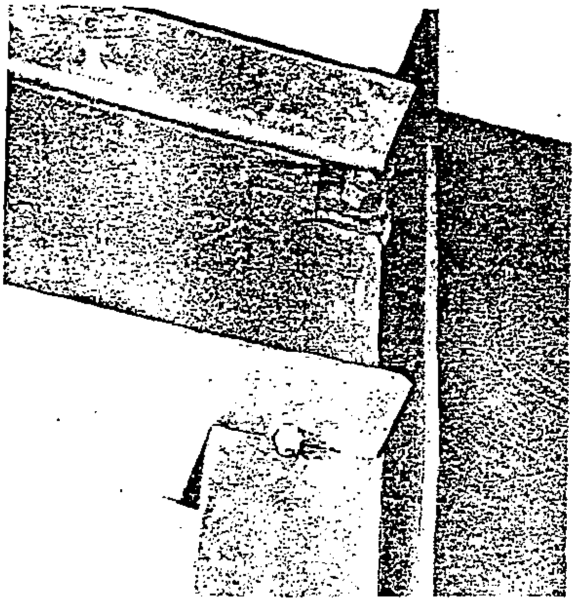

FIGURE 18

Between the time the beam is erected and the joint welded, the columns are pulled into proper alignment by cables. Careful layout in the fabricating shop and a positive location in the connection will facilitate this.

Any over-run or under-run of the column requires some adjustment of the connection. Otherwise the column would be pulled out of line in order to get the beam in place and the connection lined up.

General

Use the newer A36 steel for a 10% higher stress allowable and about 5 to 7% savings in steel at little additional unit price in steel. E70 welds have 16% higher allowable for fillet welds.

Use a 10% higher allowable bending stress for "compact beams", $\sigma = .66\ \sigma_y$ instead of $.60\ \sigma_y$, and for negative moment region at supports use only 90% of the moment (AISC Sec 1.5.1.4.1).

Many connections provide a direct and effective transfer of forces and yet are too costly in preparation, fitting and welding.

Maximum economy is obtained when a joint is designed for welding. It is not sufficient to apply welding to a riveted or bolted design.

Use rigid, continuous connections for a more efficient structure. This will reduce the beam weight and usually reduces the overall weight of the complete structure.

Use plastic design to reduce steel weight below that of simple framing, and reduce the design time below that of conventional elastic rigid design.

The greatest portion of welding on a connection should be done in the shop and in the flat position. As much as possible, miscellaneous plates used in connections, such as seat angles, stiffeners on columns, etc.. should be assembled, fitted, and welded in the shop in the flat position.

The connection must offer proper accessibility for welding, whether done in shop or field. This is especially true of beams framing into the webs of columns.

Proper fit-up must be obtained for best welding. Care must be used in layout of the connection, flame cutting the beam to the proper length, preparation of the joint, and erecting the member to the proper position and alignment. Good workmanship, resulting in good fit-up pays off.

Weldor makes continuous beam-to-column connection on Inland Steel Co.'s office building in Chicago. At this level, the column cross-section is reduced, the upper column being stepped back. Spandrel beam is here joined to column by groove welds. The weldor, using low-hydrogen electrodes, welds into a backing bar. *Run-off tabs* are used to assure full throat size from side to side of flange.

For New York's 21-story 1180 Avenue of the Americas Building, welded construction offered important weight reductions and economy, quiet and fast erection. Maximum use of shop welding on connections minimized erection time.