Detailing to Achieve Practical Welded Fabrication

OMER W. BLODGETT

The thousands of highway bridges, scores of high-rise buildings, and other impressive structures framed with welded steel certainly testify to the knowledge and skills of design engineers and structural steel fabricators.

But, even sophisticated engineers do make mistakes—and there are very few designers or fabricators who are completely up-to-date on modern welding technology or structural design. The author, in his consulting work, often sees the errors that engineers make and has observed their lack of awareness of improved principles in design and advances in the welding arts.

Before discussing examples of good and bad detailing, one point should be stressed: *Anything done five years ago in welding design or fabrication could be obsolete!* Every welded detail should be re-evaluated to determine (1) if it incorporates new knowledge about how to handle forces; (2) if it takes advantage of new provisions in governing codes; (3) if it is compatible with new advances in welding equipment, electrodes, processes, and procedures; and (4) if it offers opportunity to minimize costs through use of more precise determinations made possible with the advent of the programmable calculator.

To show how changes have come about in structural design, refer to Fig. 1, reproduced from *Design of Welded Structures,* published by The James F. Lincoln Arc Welding Foundation. The first edition of this book appeared in late 1966.

The photograph shows the welding of a connecting plate in a beam-to-column connection on a building in Miami. Obviously, this was considered a good technique in 1966—or it would not have been illustrated in the book. But that procedure is now obsolete, and the method of making the attachment is used sparingly. The welding was being done with a 5/32″ E6010 stick electrode. The electrode is good, but if one were to use this type of connection today and weld with a stick electrode, he would choose an electrode with a much faster deposition rate, probably an iron-powder type of a larger size. And with the welding done downhand, it would be much less costly to use the semiautomatic self-shielded flux-cored process, thereby achieving X-ray quality welds at still faster welding speeds.

Omer W. Blodgett is Design Consultant, The Lincoln Electric Company, Cleveland, Ohio.

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Figure 1

Obviously, in a revised edition, this photograph will not be used as an example of good practice.

To get into the subject of detailing, it would be well to start with a dictum that is paramount in welded steel design: *A path must be provided so transverse force can enter that part of the member (section) that lies parallel to the force.* Figure 2 is illustrative of this principle. While it seems so elementary it is hardly worthy of mention, failure to provide such a path possibly leads to more structural design problems than any other cause.

Figure 3a shows the simplest and most efficient way of welding a lug to a flanged beam so the force goes into the web, the part parallel to it.

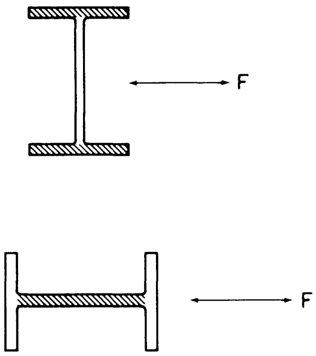


Figure 2

In Fig. 3b, the lug is placed across the bottom flange, which may require the use of either rectangular or triangular stiffeners to transfer the load to the web (see AISC Specification Sect. 1.15.5). Merely welding the lug across the more flexible flange would result in an uneven distribution of load to the weld and to the lug. Note the stiffeners are not welded to the top flange. There is no reason for such welds, since the flange will not take the force.

In Fig. 3c, the member is in a different position, and the lug is correctly welded to the flanges, which will take the load. It is not welded to the web, since such welding would serve no purpose in transferring force.

Figure 4 shows an actual problem that resulted because the designer overlooked the necessity of providing a proper

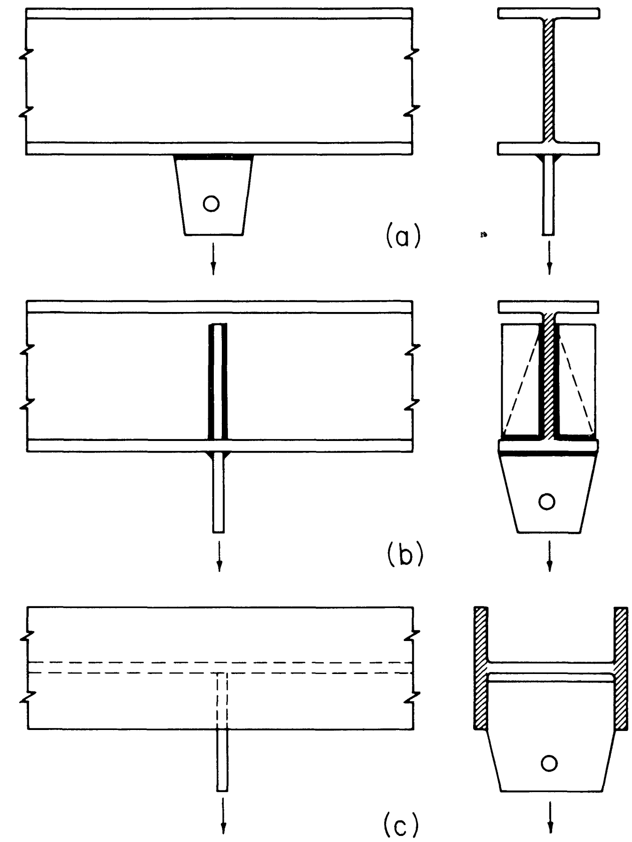


Figure 3

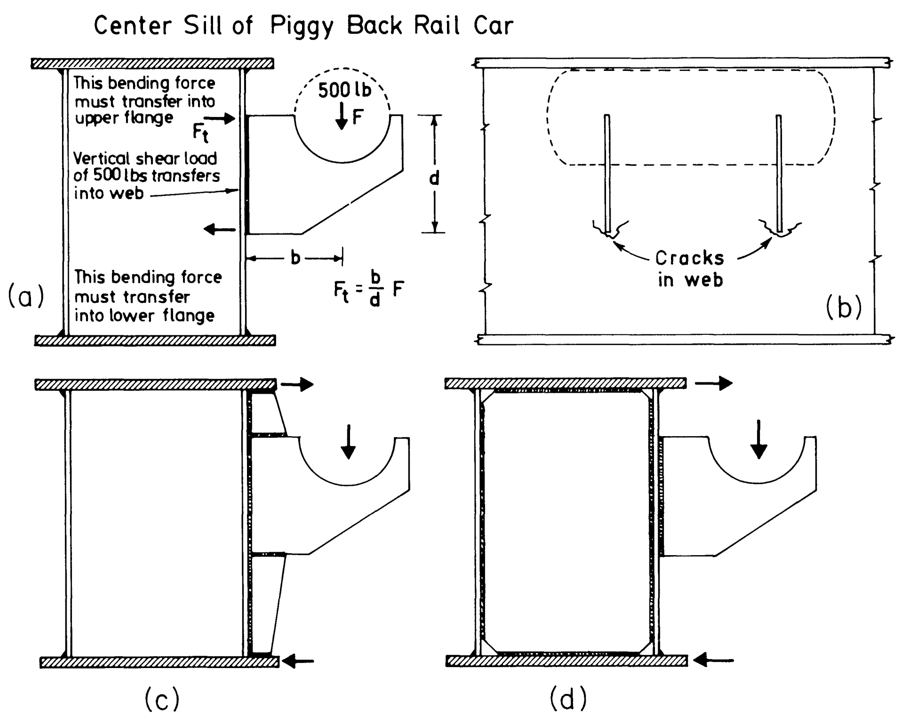


Figure 4

path for the transfer of forces. While the instance cited is not in the structural field, the example applies directly to attachments on structural box girders.

Figures 4a and 4b illustrate how a bracket to carry a 500-pound air compressor unit was welded to the center sill of a piggyback railroad car—and the cracks that resulted in service. Note that there are no interior diaphragms. The vertical force from the weight of the tank is transferred as moment into the bracket, creating out-of-plane bending at the web. The two horizontal bending forces must eventually transfer out to the parallel flanges, but with an open box section there are no ready pathways. As a result, the web flexes and fatigue cracks soon appeared in the web.

Figures 4c and 4d show two possible means suggested for correcting the faulty design. In one, an interior diaphragm is added before the web opposite the bracket side is welded into the assembly. The diaphragm is welded to both flanges and to one web. There are now paths for the bending forces to get to the flanges. The second way to correct the design is to shape the bracket so it can be welded directly to the center sill flanges in new fabrications, or to add pieces to the bracket on existing cars to accomplish the same purpose.

Figure 5 illustrates a similar example. A floor beam frames into a main girder on a railroad bridge. A bracket is welded to the top of the floor beam and bolted to the transverse stiffener of the girder. The stiffener is not welded to the flanges of the girder. When a load is applied to the floor beam, it deflects, causing end rotation and creating a moment made up of tension in the top flange of the floor beam and compression in the lower flange. The forces should eventually end up in the girder flanges—but there are no suitable paths. The tensile force is taken by the bracket, and then by the stiffener, after which only the web of the girder can provide reaction. The out-of-plane bending force on the web is too great, and it cracks under fatigue loading.

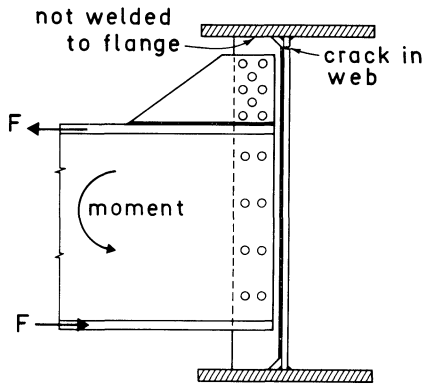


Figure 5

Structural designers occasionally overlook the need to provide a component when a force changes direction. Figure 6 shows a knee, such as used in building frames. Consider Fig. 6b. In order for the force F to change direction, a force component F_c is required. See the balloon with the force diagram. The forces are transmitted and balanced through the welds joining the diagonal to the flanges. The diagonal does not have to be welded to the web for transfer of forces. If there is concern about buckling, the stiffener should be designed as a column with appropriately spaced

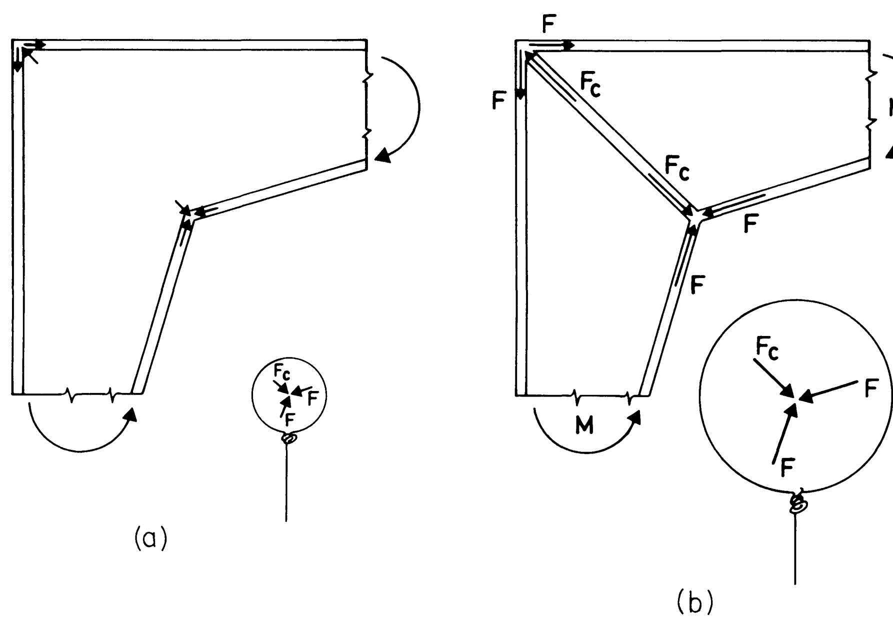


Figure 6

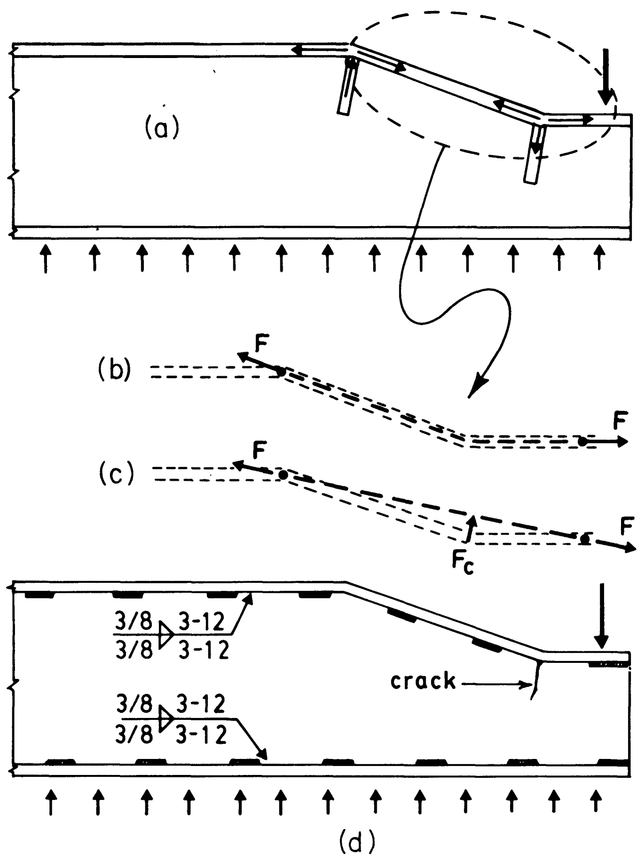


Figure 7

intermittent welds. If exposed, such as in a bridge, a continuous seal weld might be needed to keep out water.

In Fig. 6a no provision has been made for the component force F_c. Only the web can carry it. The force component F_c at the outer edge of the flange is zero, or almost so. "Letting the air out of the balloon" to decrease F_c will also decrease F. This means the force F in the flange at its outer edge is also zero, building up to a maximum in line with the web. Therefore, without a diagonal stiffener there will be an uneven distribution of force in the flange of the frame at this point.

A part of a structural frame for a cantilevered balcony is shown in Fig. 7. In Fig. 7a, brackets are shown where the flange changes direction and component forces are developed. The dotted line sketches illustrate how the forces F in Fig. 7b create the condition in Fig. 7c, resulting in the component force F_c at the outer point of taper of the member. Unfortunately, in fabricating the girder no thought was given to the component force and no stiffener was specified. To compound the error, when the weldor placed the intermittent welds, it just happened that no weld was placed at the point of change in direction of the flange. As a result, large tensile forces were induced in the edge of the web, which caused a crack to develop as shown in Fig. 7d.

Structural designers should always be aware of the different requirements for curved sections subject to moments.

The structural knee in Fig. 8a has a long radius of curvature, and the unit radial forces push the inner flange against the web. The welds attaching the inner flange to the web transfer little or no force at all; they merely hold the parts together. The unit radial forces go directly into the web, and they are relatively small, since they distribute the force F over a long curvature. The unit radial forces in the press in Fig. 8b, however, are relatively high, because of the lesser radius of curvature, and are in the opposite direction. The inner flange is now in tension rather than compression, and the weld must be adequate to hold it to the web at the curvature. Sometimes these welds break because they are undersized.

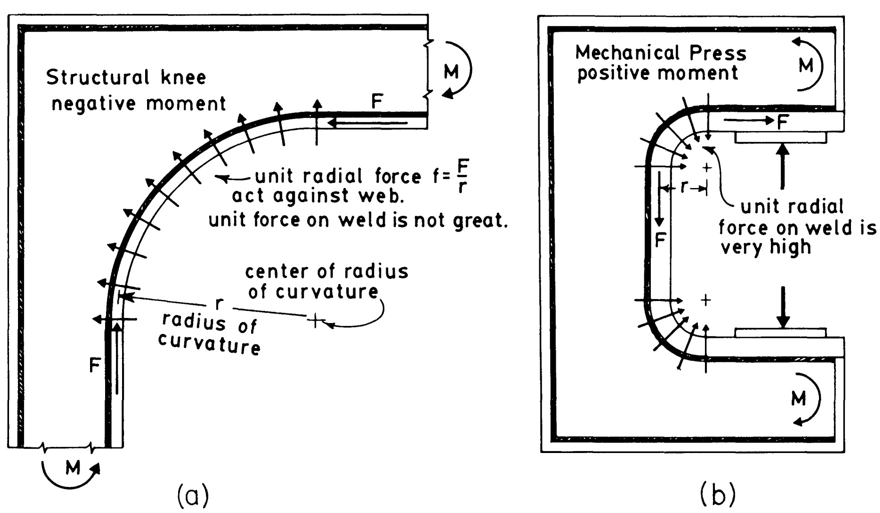


Figure 8

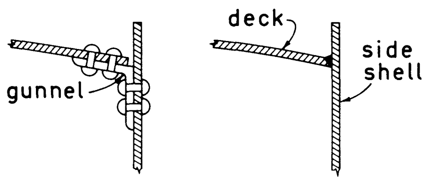


Figure 9

The need for careful detailing is especially acute when welding is used to replace riveting or bolting in joining members of a steel structure. If the old details for riveting or bolting are followed, weld and plate cracking are likely to occur. The reason is that riveted or bolted connections act as "crack arresters", whereas the continuous nature of a welded assembly facilitates crack propagation.

The experience during World War II in changing from riveted joints to welded joints in shipbuilding drove home to designers the need to change detailing concepts—and to follow carefully planned welding procedures. As Fig. 9 makes clear, if a crack should develop in the deck plate of a ship, it would be stopped at the first riveted joint—not proceed through the gunnel angle and into the side shell. But with a welded connection, there is a path for propagation from one plate to the other. Also, it was soon found that a riveted joint was more "forgiving" when the design was poor. Thus, the hatch openings with square corners, as shown at the left in Fig. 10, had to be made with rounded corners with welded design to prevent stress raisers and the initiation of cracks that might propagate the length of the deck. Corner stress raisers were of no major consequence with riveted decks; the crack would stop at the first riveted joint. The same considerations apply when changing joining methods in bridge and building construction.

Failure to recognize strain compatibility can cause the designer much trouble. Figures 11, 12, and 13 illustrate this problem.

In Figure 11, a stringer runs through and is supported by a box girder bent. A slot has been cut into the web of the box girder to receive the lower flange of the stringer, and the designer decides to weld this to the web of the box girder at the slot. If he does, he is virtually asking for a crack to form in the web of the box, as indicated in the lower view. Why? . . . The reason is that the web of the box girder is stressed from bending and will elastically strain. If the lower flange of the stringer is welded to the web plate of the box girder, it will be pulled along with the web plate as it is elastically strained.

Assume that the web in the region of the slot is stressed to 20 ksi. If the width of the flange of the stringer is 20 in., the slot will strain at the rate of 0.000667 in./in., or a total elastic elongation of 0.0133 in. The flange of the stringer where it passes through the slot in the web must also elongate this much if it is welded. The question then becomes: What tensile force will be required to elongate the flange 0.0133 in.?

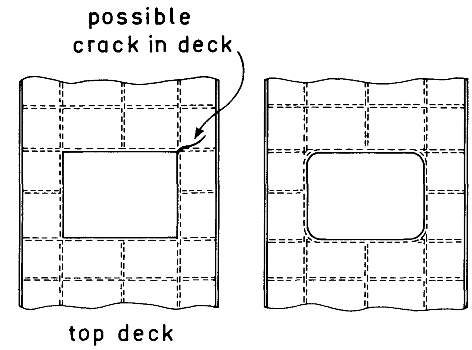


Figure 10

Referring to Fig. 12, a 1-in. section of flange 1-in. thick would require 20 kips of force to elongate 0.0133 in. If this section were welded at the ends, the stress on the welds would be 20 ksi. If a 3-in. wide section of flange is assumed, it is found that 60 kips of force are required to elongate the flange section 0.0133 in. The end welds are now stressed 60 ksi. For a 6-in. wide section, the force becomes 120 kips with 120 ksi stress on the welds. Carrying this procedure further would obviously impose forces and stresses that are unrealistic. It would be impossible for any weld to withstand the force when the web of the box section is stressed just 20 ksi in service. If the weld did not break, it would pull out of the web plate.

The solution to the problem would be to let the bottom flange of the stringer ride unattached to the web in a smooth flame-cut and ground slot, or to use an attachment as shown in Fig. 13, where a wide plate cut to a radius of 24 in. is welded between the web of the box and the flange of the stringer.

Another dictum the designer and, especially, the fabricator must keep in mind is: *There are no secondary members in a welded design. Even interrupted backing bars can cause main members to crack.*

The codes today specify that backing bars must be continuous for their full length (for example, see AWS 01.1, Sect. 3.13), but the designer can run into situations where continuity may not seem required. Figure 14 is an example. The orthotropic deck for a bridge was welded to stringers as shown in the figure. The deck was welded to the stringers first, and then the short pieces of backing bars tacked in to back the groove weld that would be placed the width of the deck. Shortly after commencing the groove weld in the field, it was noticed that transverse shrinkage cracks were occurring occasionally in the groove weld over the interrupted backing bars. Detecting the problem in time, the detail was changed to provide a continuous backing bar laid in slots cut in the stringer webs.

Figure 15 is a case where an interrupted backing bar could cause a fatigue crack in the flange of a deep girder. This detail was developed in the design of a bridge. An attachment was needed as near as possible to the lower tension flange of the girder, and by using the transverse

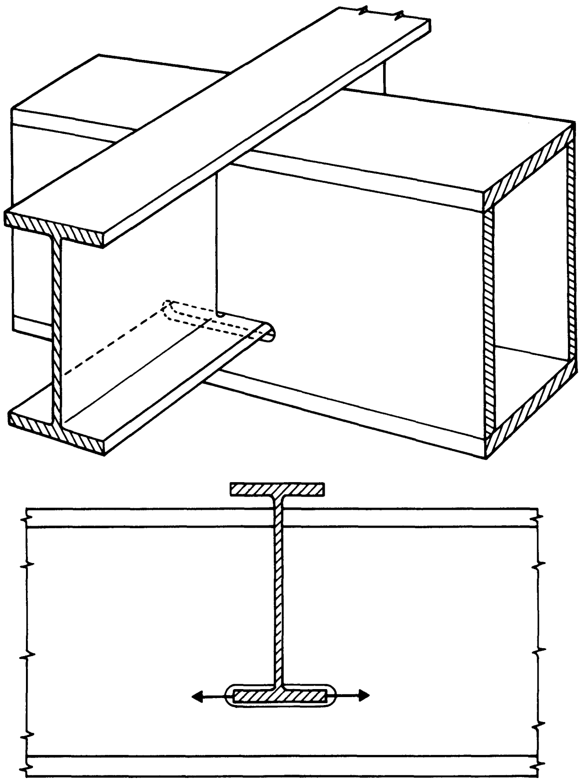


Figure 11

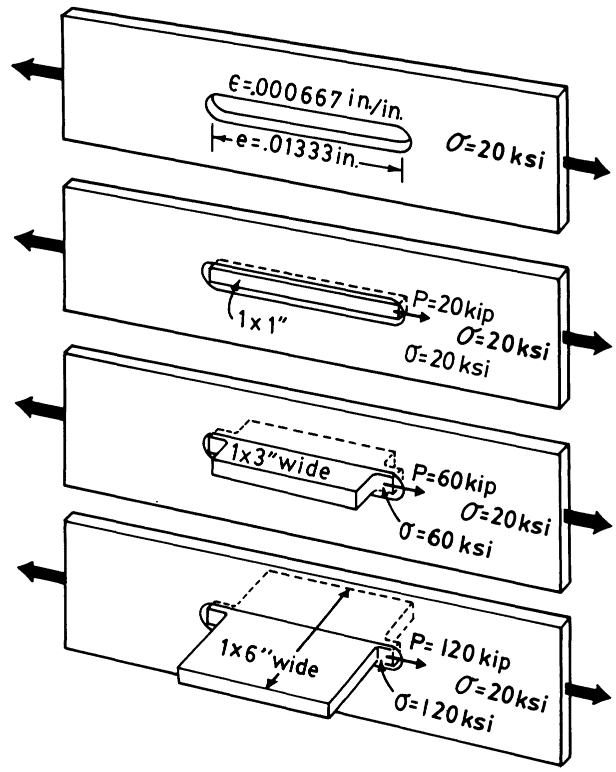


Figure 12

stiffener as shown it appeared feasible to place the attachment about 5 in. above this flange. This closeness didn't allow space for overhead fillet welding the attachment to the girder web, so a groove weld using a backing bar was chosen as an alternative. The backing bar was interrupted where it abutted the stiffener.

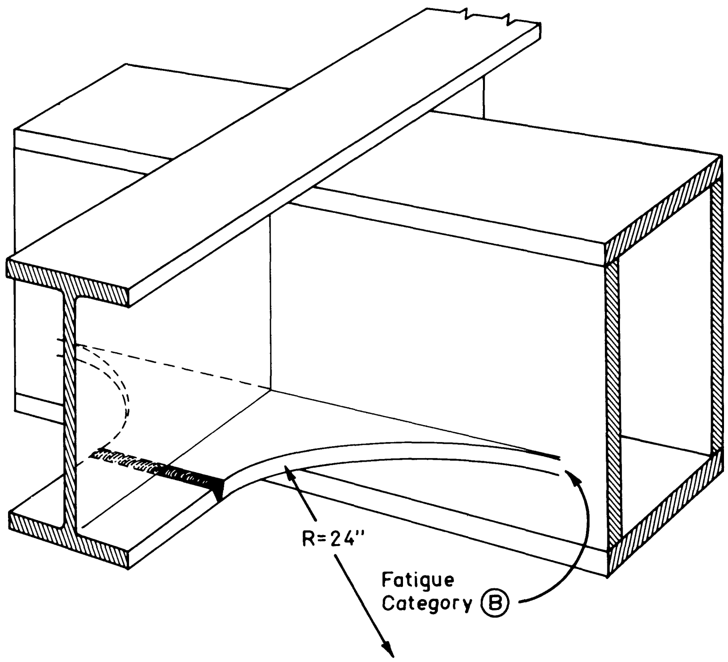


Figure 13

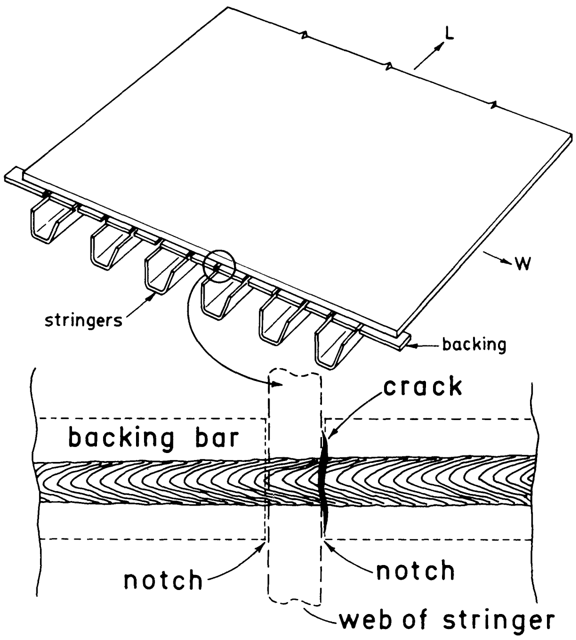


Figure 14

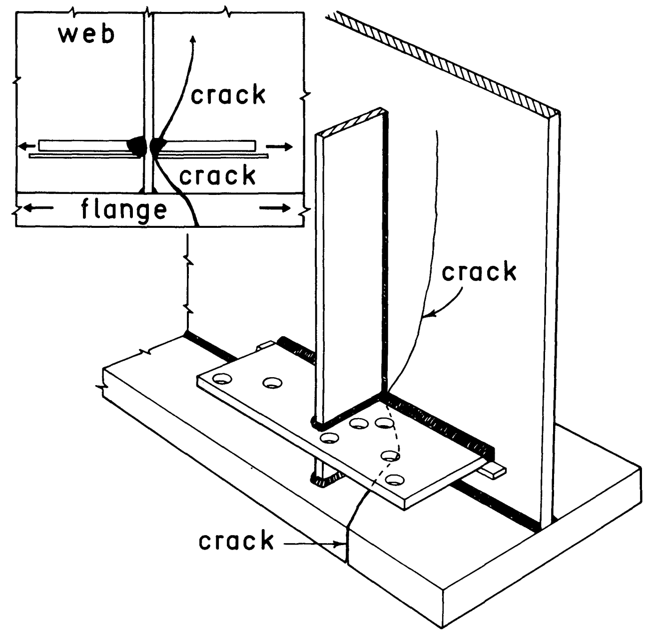


Figure 15

The error of the detail was, of course, the interrupted backing bar. With the attachment welded to the web close to the tension flange, there would be high bending stress in the web at the weld. The notches created by the interrupted backing bar under fatigue loading would cause the weld to crack, and that crack to propagate up into the web and down into the bottom flange. This detail produced a notch almost as critical as though a hack-saw cut $\frac{1}{4}$-in. deep had been made into the tension flange.

Figure 14 illustrated how transverse shrinkage cracks could result from interrupted backing bars, and Fig. 15 showed how fatigue cracking could develop. Brittle fracture is another consequence of welding over the notches created by interrupted backing bars. Figure 16 is illustrative.

The detail in Fig. 16 shows the method of joining two bent plates to form a box beam for a boom for a large earth-moving machine. There are two groove welds, made over backing bars. The design was good, and there would have been no trouble had the backing bars been continuous. But apparently someone in the shop decided "there was no reason for wasting these pieces of backing bar. They're just something to keep the weld metal from spilling through." So the pieces were used, and the finished boom was being transported down Michigan Avenue in Chicago on a flatbed trailer one February morning when the temperature was near 30 degrees below zero, when the savings in material was nullified. The boom, under its own weight, snapped in two. A notch, plus low temperature, had led to brittle fracture. Had the pieces of scrap backing bar been welded together and ground before use, there would have been no notches, and thus no problem.

Fabricated box girders, square, rectangular, or delta in section are popular in structural work. Smaller box girders can be made with a fillet weld between the webs and flanges. Larger box girders are sometimes made with a bevel groove weld, using an inside backing bar, as illustrated in Fig. 17a. The continuous backing bar is usually tack-welded in place so it fits tight against the plates, as shown in the sketch. While this is a practical shop procedure, it is not ideal. If the girder is fatigue loaded, which most are in highway structures, the intermittent tack welds reduce the fatigue strength from Category B down to E—which is fairly low.

It appears possible that a welding procedure now being used by the shipbuilding industry for the butt welding of plate from one side may be the solution to fabricating box girders without reducing fatigue strength. This procedure is called "one-sided" welding and is accomplished by using

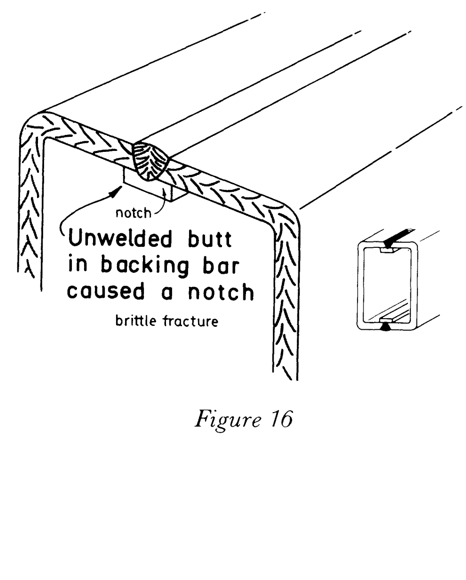


Figure 16

a grooved copper backing bar beneath the weld joint. Welding is done with special series-arc equipment. The copper bar does not adhere to the underside of the weld and is removed for reuse. The root of the groove weld is smooth. One can easily visualize adaptation of one-sided welding to box girder fabrication by appropriate shaping of the groove in the copper backing and positioning the girder for flat welding, Fig. 17b.

Any effort to minimize the amount of weld metal used will usually decrease the welding cost. Just as important, it will reduce the weld distortion and the locked-in stresses that might lead to lamellar tearing. It is important for the engineer to see if there are alternate weld joints he might use and then use the one which requires the least amount of weld metal.

Now that programmable calculators enable us to determine the stresses at weld joints, even in complex structures, and the required size of weld can be accurately determined, there are great opportunities for cost savings. However, before savings of pennies are sought by modern analytical methods, designers and fabricators would do well to start saving the dollars that are wasted by what might be called obsolete "hereditary" ways of weld sizing.

In a fabricating shop, the author saw a 60-degree included bevel being used for a butt weld in a 3-in. flange plate. It seemed reasonable to ask why such a wide angle was selected, instead of a 45-degree bevel with a 1/4-in. root opening or a 30-degree with a 3/8-in. root opening, or a 20-degree with a 1/2-in. root opening. The reply was: "I don't know. That's the way we have been doing it the 20 years I've been here, and that's the way Ole Joe did it before me."

Ole Joe, in his days, was probably doing the most economical job for his company—because he was butt-welding 1/2-in. plate, rather than 3-in. A glance at Table 1 shows that a 60-degree included bevel with 1/2-in. plate requires less weld metal than a lesser angle. But this does not hold for thicker plate. A 20-degree angle with 3-in. plate, using a backing bar, would cut weld metal costs 50%. This could have a pronounced effect on the fabricator's profit.

Over the years, thicker plate has come into greater use. Shop personnel—and designers—need to be alert to the wasteful traps one can fall into by sticking to old ways. Note in Table 1 that for T-joints the 30-degree with 3/8-in. root

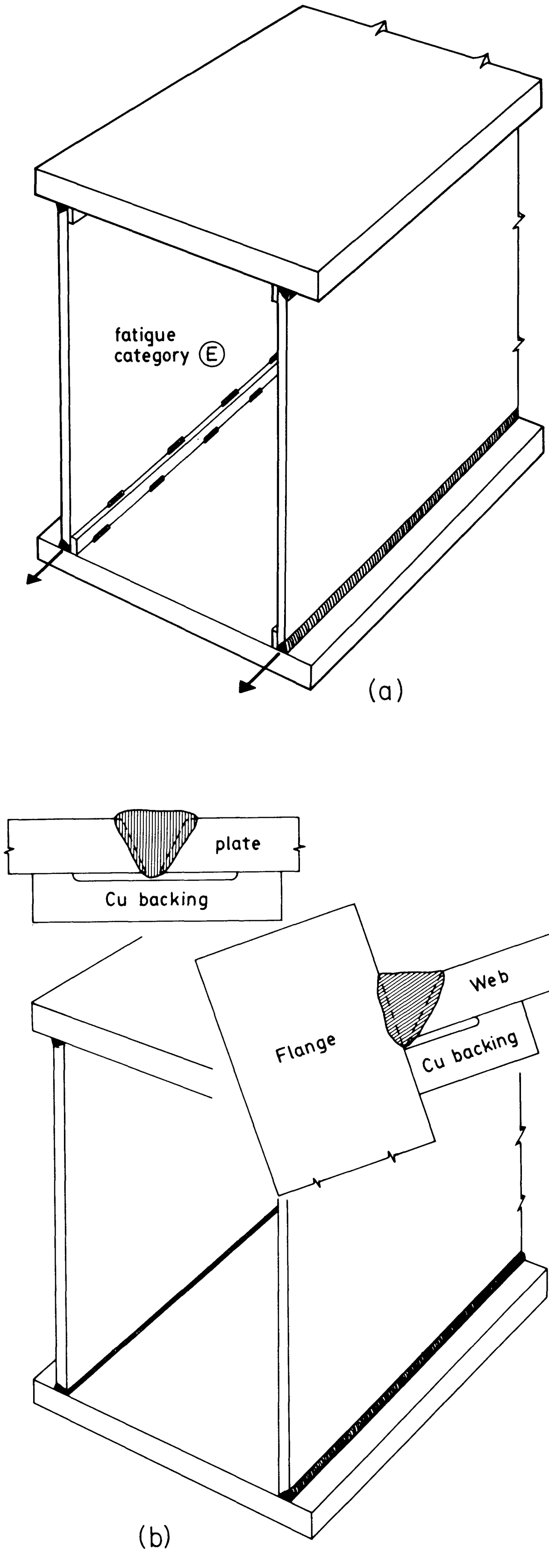


Figure 17

Table 1. Weight of Weld Metal

plate thickness	60° 1/8"	45° 1/4"	30° 3/8"	20° 1/2"	45° 1/4"	30° 3/8"
1/2"	.84	.90	.99	1.13	1.03	1.04
1"	2.69	2.50	2.38	2.47	2.93	2.51
2"	9.43	7.85	6.54	6.07	9.39	6.98
3"	20.21	16.08	12.57	10.88	19.39	13.46

AWS D1.1 Prequalified Joints

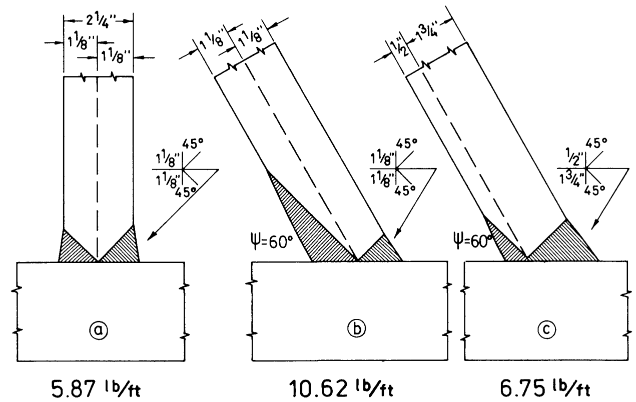


Figure 18

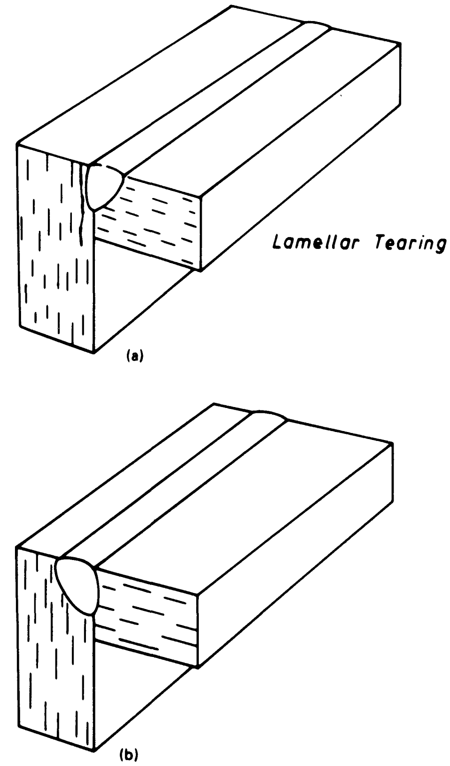


Figure 19

opening becomes less costly than the 45-degree with ½-in. root opening, once the plate thickness is more than ½-in. Other than providing slightly better accessibility for the welding operator, there is apparently no reason to use the 45-degree included joint.

One engineer had been detailing a T-joint made of 2½-in. thick plate. He correctly used a welding symbol indicating a double bevel joint in which the depth of bevel was equal on each side (1⅛-in.), as shown in Fig. 18a.

Without thinking, he used the identical symbol on a similar joint in which the angle between plates (dihedral angle) was 60 degrees. Figure 18b shows what he would have had if the shop had followed his symbol. The shop correctly changed the joint to Fig. 18c and reduced the amount of weld metal from 10.62 to 6.75 lbs/ft, with a similar reduction in distortion, and, if lamellar tearing had been a problem, a reduction in that risk.

Not only will reducing the amount of weld metal help in lamellar tearing, but sometimes a change in detail will help (see Fig. 19). In Fig. 19a, the transverse residual stress of the weld acts on a single line of inclusion which may open up into a lamellar tear. In Fig. 19b, this critical section is now skewed and passes through many lines of inclusions, hence decreasing the tendency for lamellar tearing.

Figure 20a shows a cantilevered beam welded to a column where stiffeners were required. This detail was included in the original plans for the construction of a stadium. The design consultants on the job were apprehensive about the detail. Lamellar tearing was being encountered in some welded structures, and supporting a balcony with a cantilever in such a way suggested the possibility of a problem because of lamellar tearing. If striations in the rolled plate forming the side of the column should exist and

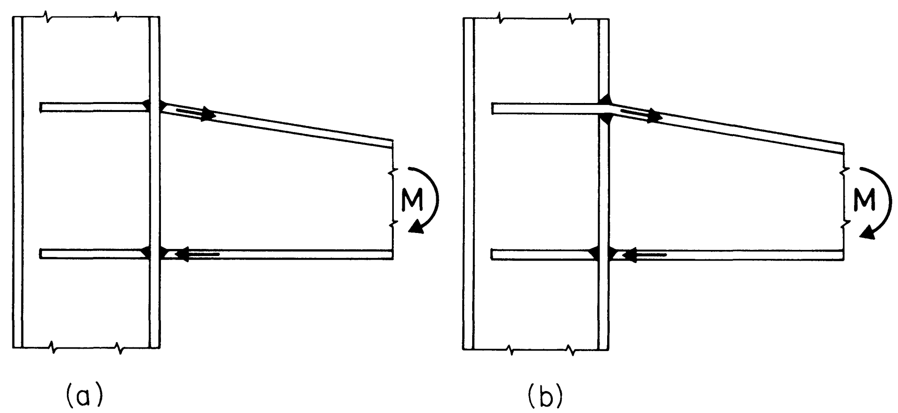


Figure 20

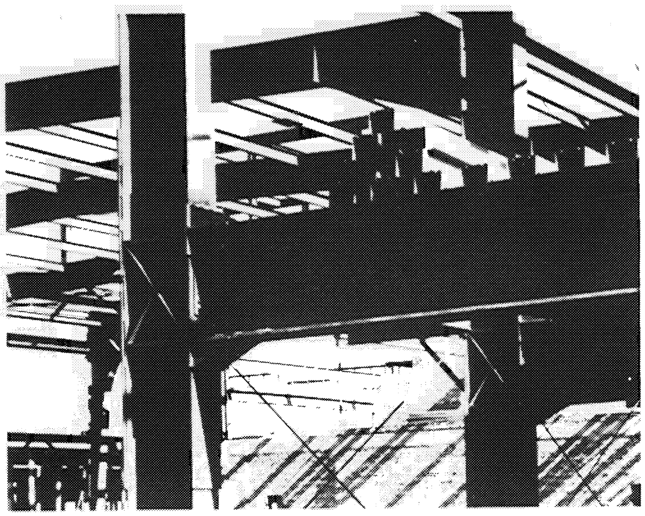


Figure 21

pull apart, there would be nothing to prevent the beam from separating from the column; there would be no other support to avoid collapse, such as with non-cantilevered beams.

The detail in Fig. 20b was recommended—and was used in construction of the stadium. Note that in this detail there is no possibility of lamellar tearing at the tension region. And while the bottom flange of the beam is still welded to the outside of the column plate, lamellar tearing should not be a problem because the bottom flange is in compression.

Redundancy is needed in space vehicles, but serves no useful purpose in a well designed ground structure. Adding unnecessary parts merely runs up costs and frequently actually weakens the member. Figure 21 shows a double diagonal in a framing of a building, which suggests that the designer was uncertain about the performance of diagonals in such details and threw in an extra diagonal "just to be sure". The situation is analogous to the man who wears both a belt and suspenders to be sure his pants stay up.

As Figure 22 shows, there are two ways to use a diagonal stiffener in transferring a moment from a beam into a column.

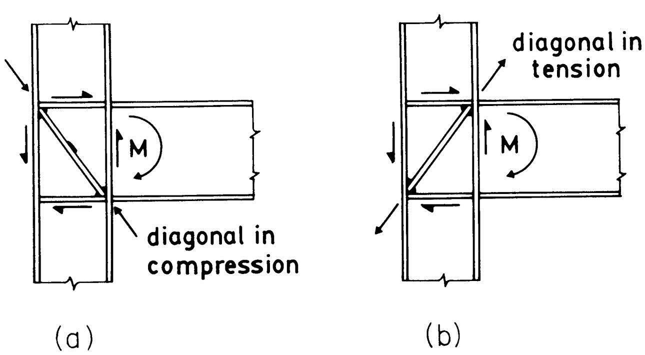


Figure 22

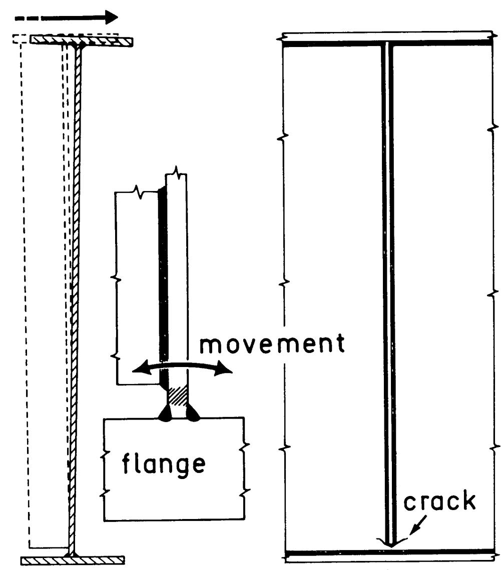


Figure 23

In Fig. 22a the diagonal is in compression, and in Fig. 22b in tension. Each method of placement transfers the moment effectively. Of the two details, welding is less critical in Fig. 22a, where the welds transfer compression.

The fitting and welding of a double diagonal brace greatly increases the cost. If the designer felt that one diagonal was inadequate for the moment, the logical step would have been merely to increase the thickness of that single diagonal. Note that in welding diagonals it is only necessary to weld where force enters and leaves the diagonal. A small weld may be used at the center of a diagonal in compression if there is concern about buckling.

What happens to a structural member before it is put into use can affect its design. The designer should be aware that all members are subjected to handling, loading, shipping, unloading, and erection operations. For example, refer to Fig. 23. The deep plate girder was shipped to the site tied down to a flat-bed railroad car. Before unloading, it was noticed that there were cracks in the web at the lower end of the transverse stiffener. It was concluded that cracking occurred because during shipment the deep girder had differential lateral displacements of the top flange with respect to the bottom flange. This caused bending stresses in the one-inch unstiffened portion of web between the bottom tension flange and the end of the stiffener. With all the flexing concentrated in a small section of web, it gave way—fatigue cracked—under the thousands of jolts encountered in transit. It was recommended that the stiffener stop back at least four to six times the web thickness from the tension flange to spread the flexing over more area. Curved or skewed bridges require special treatment (see

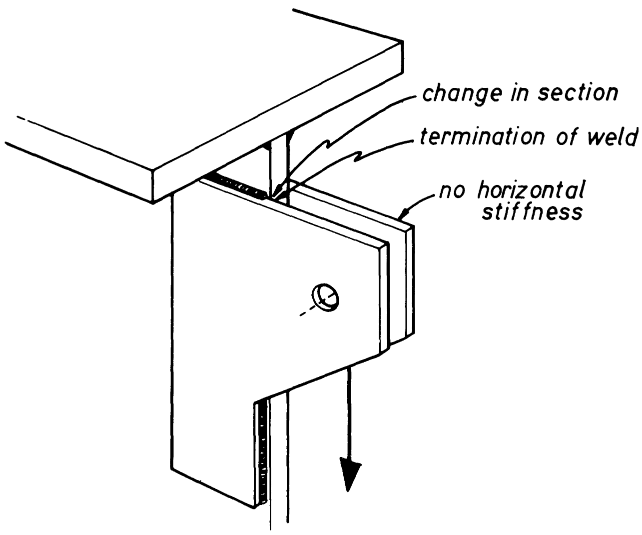


Figure 24

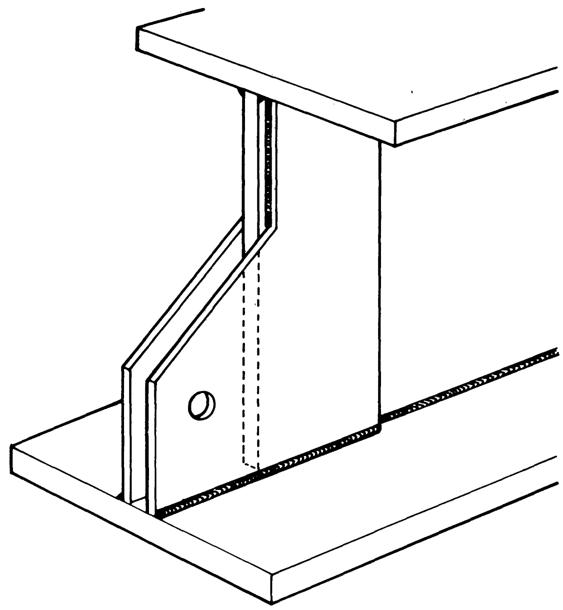


Figure 25

the AISC publication *Bridge Fatigue Guide/Design and Details*).

The designer must depend upon the skill of the weldor for integrity of the assembly, but he should not detail in such a manner that it is the weldor's skill that determines the strength and performance of the structure. At times, also, the designer should tell the weldor "this way, and only this way, it must be done".

Figure 24 illustrates a detail which leaves much to the weldor. Figure 25 is a substantial improvement in the detail, which does not leave the performance of the structure up to the knowledge, skill, and convenience of the weldor.

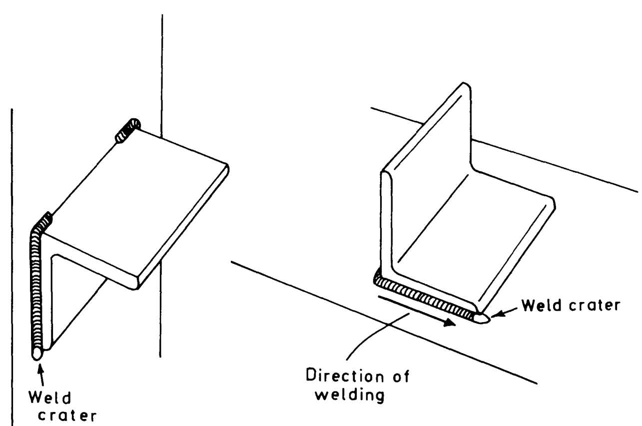
The detail is the support bracket for a short beam section. A two-span continuous plate girder cantilevers out beyond a pier and will support the short suspended beam. The bracket will be loaded in bending, and its top outer fiber will be in tension. The outer fiber in the detail in Fig. 24 is made up of the edge of flame-cut plate and either the start or stop of the fillet weld. There is an even chance that the weld will terminate with a crater at the point of high bending stress. Perhaps of minor importance is that the detail provides no horizontal stiffness for the bracket.

The detail in Fig. 25 requires a little more material and work, but eliminates the chance of design failure. The outer fiber in bending now becomes the top smooth surface of the flange plate. There is no flame-cut edge of a plate or weld crater to affect performance. There is good horizontal stiffness for the bracket.

Figure 26 is an example of a case where the designer should specify how the weldor should proceed with his work—namely where to start and stop his weld. When welding a seat to a column in the shop in the horizontal positions, the weld must start and stop so no crater is left at the point of high bending stress.

Run-off tabs or extension bars are used to start and finish automatic welds, thereby eliminating craters at the ends of welds. Butt joints of stress-carrying members should always, where possible, be welded with some type of run-off tab attached to the ends of the joint to make it easier to obtain good quality welds at the ends. The design engineer should specify the use of run-off tabs in critical work and designate the type. Figure 27 shows run-off tabs (extension bars) that will give the proper equivalent joint detail at the ends.

How a run-off tab is tacked to the work is important. Tack welds to hold run-off tabs must be made with proper preheat if they are not to be remelted and incorporated in



Welded in horizontal position to column flange.

Figure 26

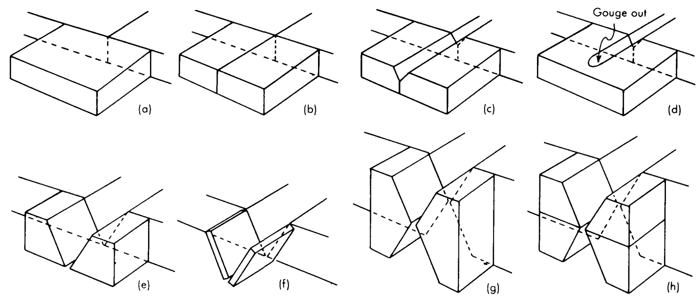


Figure 27

the final weld made by submerged-arc welding. Later, if the designer calls for removal of the tabs, the surface must be ground smooth (see Fig. 28).

With a tension flange in a fracture-critical member, it is better to tack weld the run-off tabs on the inside of the joint as shown in Fig. 29. Then, when the joint is submerged-arc welded, the tack weld will be incorporated in the final weld. After removing the tabs and grinding the edges of the flange plate smooth, the final joint will be as if no tack welds were used.

The designer should be very careful when he adds extra members to a designed member that he has determined satisfactory for his load. Sometimes the additions create undesirable side effects. A case in point is illustrated by Fig. 30. Here, the problem was to carry two large pipes between two buildings. The designer selected a fabricated girder as the load-carrying member. Its bending stress under load was calculated to be about 10 ksi. Then 1/4″ x 60″ x 48″ panels were welded to the girder as shown in the figure to enclose the structure. The designer forgot that the additions actually became part of the load-carrying member and neglected to recalculate for bending stress. If he had recalculated he would have found that stress was lowered from 10 ksi to 5 ksi. But the critical buckling stress of the 1/4″ x 60″ x 48″ panel was only 1.5 ksi, and during erection, the panels would buckle. The structure had adequate strength—didn't collapse—but the buckled plates are a constant reminder that there are no secondary members in a weld design.

The designer or fabricator should be familiar with prevailing codes, and especially any recent changes that simplify the design or fabrication tasks. A shop was fabricating a girder from A572 grade 65 steel. Table 4.1.1 of the AWS D1.1 *Structural Welding Code* indicated that the matching weld metal would be from an E80 electrode and the company was having difficulty getting electrode wire for either gas metal arc or flux-cored arc welding meeting this strength level. But E80 electrode was not really required. Because the girder was to be fillet welded, the very common and abundant E70 weld metal was all that was required. Not only did E70 metal fully meet codes, it would actually reduce the chance for cracking and other problems. The company had gone wrong by assuming AWS Table 4.1.1 *demanded* that matching weld metal be used with the A572 steel. The engineers should have taken a look at Table 8.4.1 for buildings or Table 9.3.1 for bridges. If they had, they would have found that matching weld metal is required *only* for complete-penetration groove welds with tension applied normal to the effective throat. This means that butt welds in the flanges of tension girders would require matching weld metal—but there were only fillet welds in the subject girder. According to AWS Table 8.4.1,

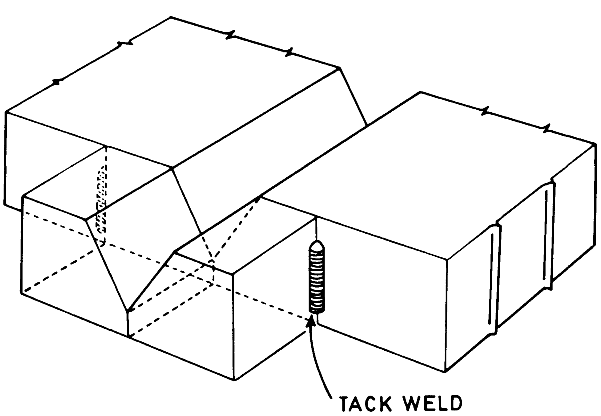


Figure 28

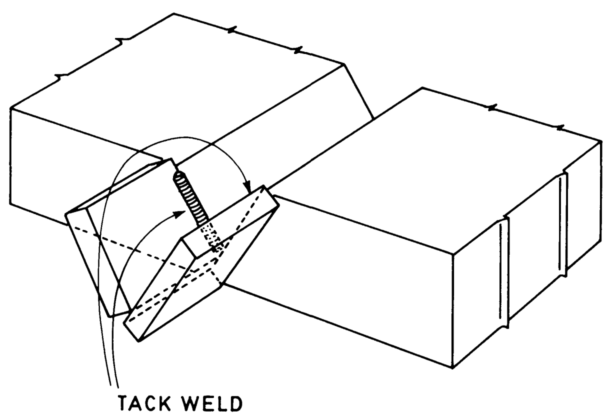


Figure 29

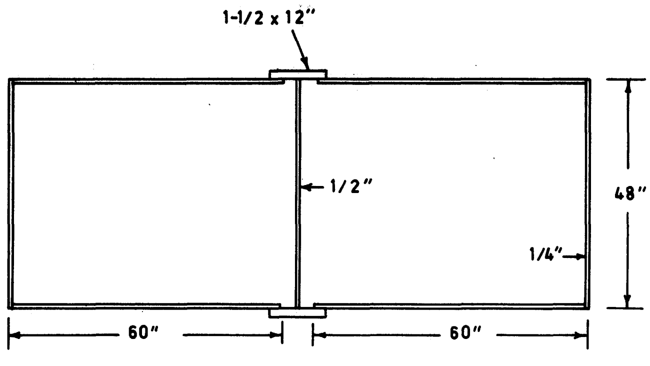


Figure 30

weld metal with a strength level equal to *or less than* matching weld metal could be used. E70 electrode would meet code.

In another instance, a building was being erected using A588 steel and welded with E70 flux-cored weld metal. When the framing was half-completed, someone suggested that procedure qualification tests should be made of the welding process. You can imagine the consternation—with the building half erected—when the specimens failed in the guided bend tests. But there was nothing wrong with the procedure and process! How could that be?

A588 steel has a minimum tensile strength of 70 ksi and a minimum yield strength of 50 ksi. The all-weld metal properties required by AWS is a tensile strength of 70 ksi and a yield strength of 60 ksi. The guided bend specimens should bend 180 degrees and look like the bent specimen in Fig. 31a. In the tests, they broke, as in Fig. 31b.

When samples of the steel were checked, however, it was found that the A588 had a yield strength far in excess of the specified minimum. Tests showed the steel being used had

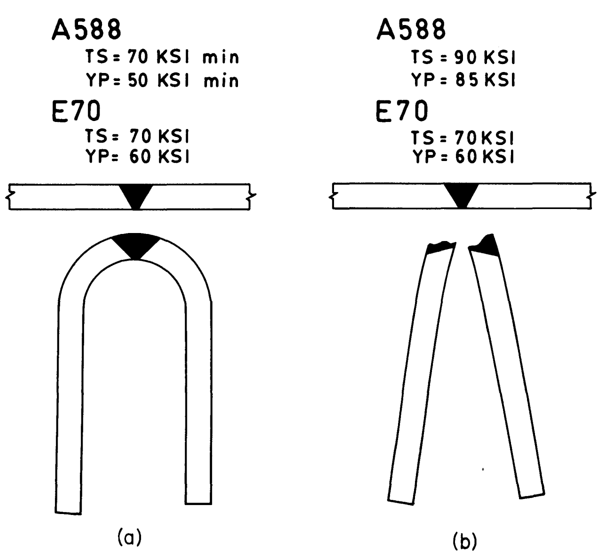


Figure 31

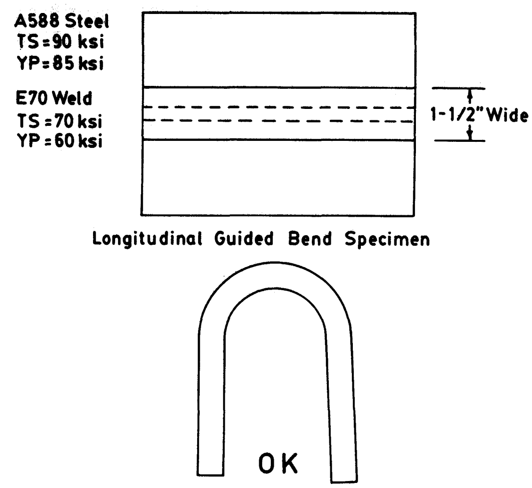


Figure 32

a yield strength of 80 ksi. Although admixture between weld metal and plate would raise the mechanical properties of the resulting weld higher than 70–60, they would still be less than the 80 strength for the steel. Thus, when a specimen was bent, all the plastic yielding occurred within the weld. After a few degrees of bend, the ductility of the weld was exhausted, and the specimen broke.

Since a large mismatch in mechanical properties had been established, it was decided to make longitudinal guided bend tests as in Fig. 32. By this method—now permitted by AWS D-1.1—the weld lies parallel with the length of

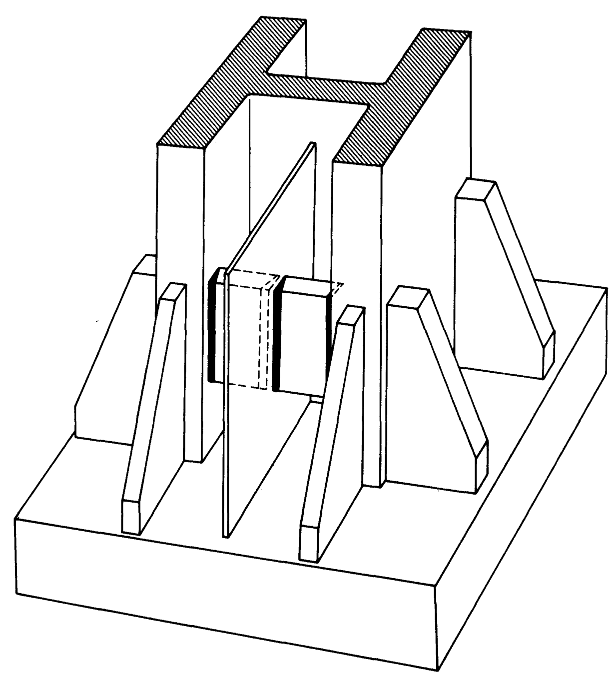


Figure 33

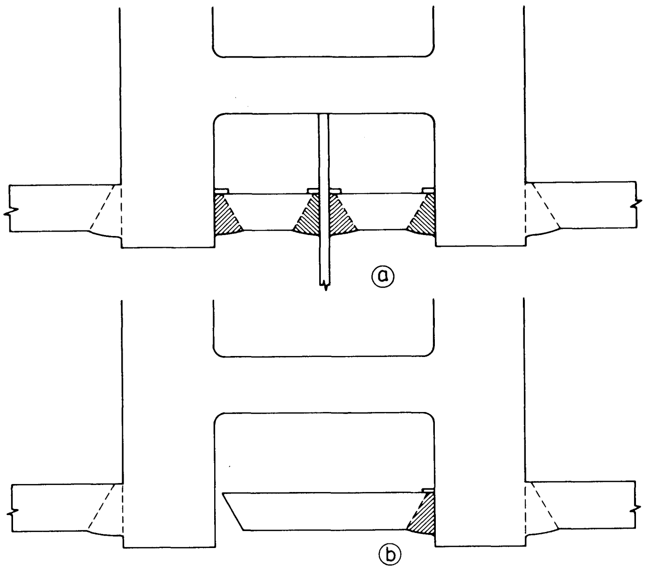


Figure 34

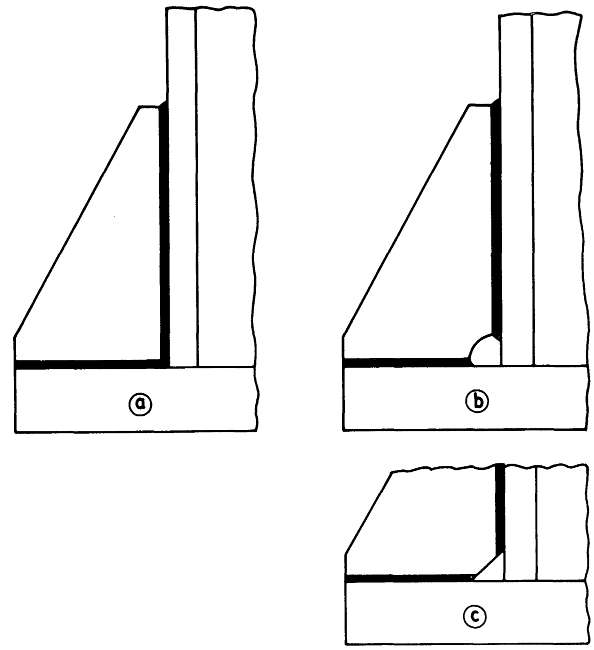


Figure 35

the specimen and direction of bending. The specimens now had uniform properties along their entire length and readily passed the bending test. Erection of the building proceeded—with a big load off the minds of engineers, erection management, and owners.

In the case of massive weldments that are highly restrained, Fig. 33, it is important not to use a welding sequence that will lock in considerable stress. If all four welds are made at the same time, the joints will be locked in the moment the four core beads have been made. From then on shrinkage will develop from the four welds (see Fig. 34a).

It would be better to tack the two plates in place and complete one weld at a time. Then the final shrinkage will be the result of one weld, the final weld, rather than four welds. If some tack welds should crack open during welding, simply stop and retack after making sure the plate is in proper alignment. After all, if a tack weld should crack, it simply proves that locked in stress is present and could be a problem.

A better idea would be to use one stiffener plate rather than two (Fig. 34b) and reduce the number of welds from four to two. For a given size weld, fewer weld passes will reduce the amount of distoration and, hopefully, the amount of residual stress.

If some weld cracking should occur in the welding of a heavy bracket (for example between the base plate and flange of a column), it might help if one weld were completed before the other weld. Also, it would be better to complete the longer weld (here against the column flange) before starting the other weld (here against the base plate), Fig. 35a. This will allow the shrinkage of the first weld to take place before being locked in by the other weld.

If a small crack should occur at the corner of the bracket where the two welds intersect, then cutting or snipping the corner as in Fig. 35c might help to reduce the stress and allow each weld to act more independently of the other. A further refinement would be to cut the corner with a radius as in Fig. 35b. This will provide sound weld metal and reduce the chance of initiating a crack at this critical section.