Welding on Existing Structures

1. LOAD CARRYING CAPACITY OF CONNECTION

In the modification or repair of buildings, it may be necessary to weld to the existing steel framework.

When welding and riveting are combined on the same strength joint, the riveted portion of the joint may slip or yield slightly, thus throwing the entire load eventually on the weld. Normally, on new construction where welding and riveting are combined, the joint would be figured on the basis of the weld taking the entire load. Since 1930, most of the old riveted railroad trestles have been reinforced by welding because of the newer and heavier locomotives.

Riveted connections can be reinforced with plates, with holes to fit over the rivets. The plate is welded to the existing connection with fillet welds all around its edge, and is plug welded to the plate at each rivet hole. This technique, however, requires a considerable amount of out-of-position welding with small electrodes.

2. EFFECT OF WELDING HEAT ON MEMBER'S STRENGTH

Frequently, a question arises as to the effect of welding on the strength of an existing structure already under a stress. Actually the strength of steel does not drop off upon heating, until a temperature of about 650°F is reached. This is brought out in the table of allowable strengths of materials in the ASME Unfired Pressure Vessels, Section 8. Here the same allowable is used from minus 20°F all the way up to 650°F. The ASME code body recognizes the fact that the strength of steel rises slightly upon heating and does not start to drop off until a temperature of 600°F or 700°F is reached.

In welding to an existing structure, the amount of material actually heated momentarily above 700° would be a very small spot right at the welding arc. Figure 1 shows the temperature rise in a plate while making a 5/16" fillet weld in the vertical-up position. This indicates that in using a 3/16" E6010 electrode, the temperature on the back side of the 3/4" thick plate opposite the weld was held below 600°F. Figure 2 shows the same weld using a 5/32" E6010 electrode. Here the temperature on the back side of the 1/2" thick plate was held below 650°F. Also see Figure 3.

The very tiny area of the member heated above this temperature does not represent a sizable percentage of the entire cross-section of the stress carrying member. This has been the opinion of many fabricators and erectors who have been welding on existing structures for several years.

All welds will, however, shrink. This creates a shrinkage force which, if welds are not placed symmetrically about the member, will result in some distortion of that member. This could occur in welding to an existing member if most of the welding is done on one side. For example, if all of the welding is done on the bottom flange of a beam, the unsymmetrical welding will tend to distort the beam upward in the

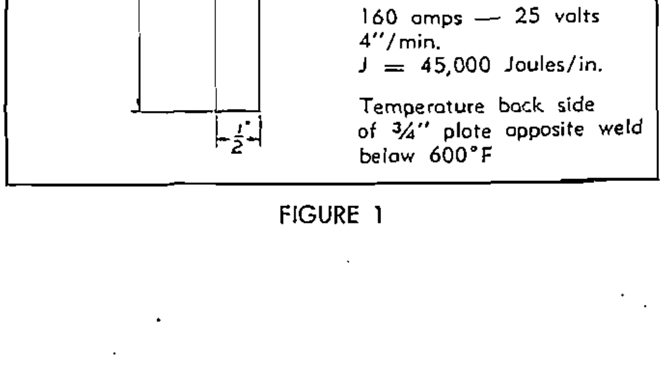


FIGURE 1

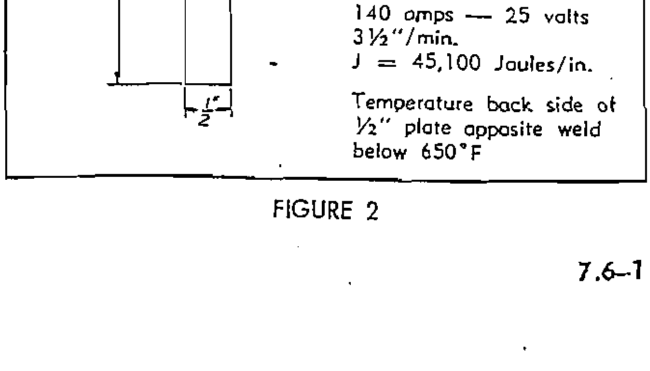


FIGURE 2

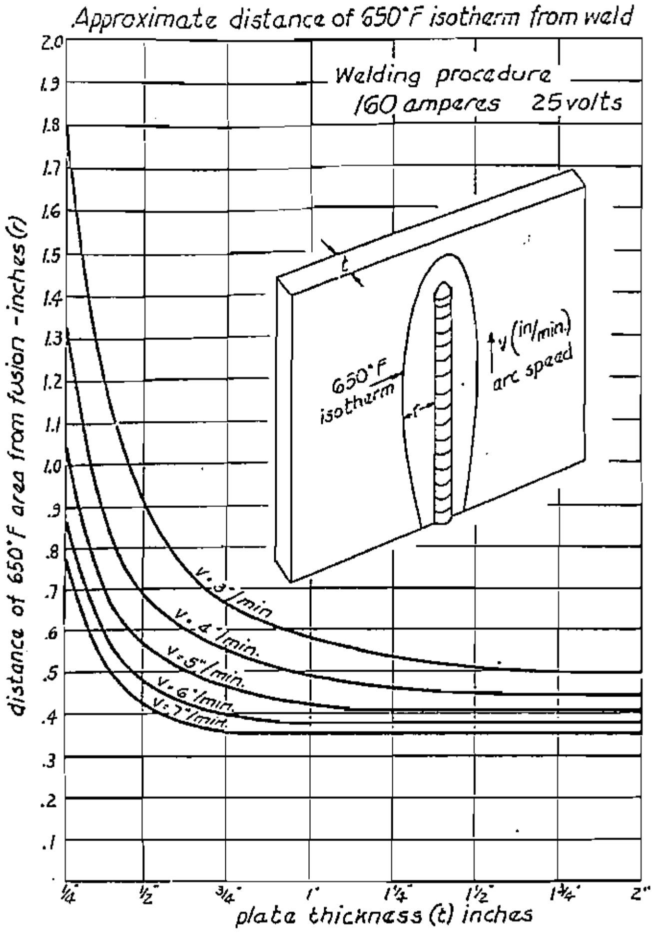


FIG. 3 A guide to establishing proper welding procedures for minimum heat input.

opposite direction as the applied load to the beam. If the welding were done along the top flange only, this would tend to distort the beam downward in the same direction as the applied load. Therefore, it might be well, in some cases, to temporarily shore up a beam in order to reduce some or all of the beam load while welding.

3. AWS, AISC AND AASHO SPECIFICATIONS

Section 7 of the present AWS Code for Welding in Building Construction, and the Specifications for Welded Highway and Railway Bridges, cover the strengthening and repairing of existing structures.

The engineer shall determine whether or not a member is permitted to carry live load stresses while welding or oxygen-cutting is being performed on it, taking into consideration the extent to which the member's cross-section is heated as a result of the operation being performed.

If material is added to a member carrying a dead load stress of 3000 psi, either for repairing corroded parts or for strengthening, it is desirable to relieve the member of dead load stresses, or to pre-stress the material to be added. If neither is practical, the new material to be added shall be proportioned for a unit stress equal to the allowable unit stress in the original member minus the dead load unit stress in the original member.

Problem 1

To reinforce an existing member to withstand an additional live load of 20,000 lbs. The existing section has a cross-sectional area of 10.0 in.², with an allowable working stress of $\sigma = 18{,}000$ psi. The original design loads—dead (DL), live (LL), and impact (I)—gave the following:

DL force	100,000 lbs ÷ 10.0 in.² =	10,000 psi
LL + I force	80,000 lbs ÷ 10.0 in.² =	8,000 psi
DL + LL + I force	180,000 lbs	18,000 psi
	and 18,000 psi ≦	18,000 psi OK

The member must now be increased in section for an additional 20,000 lbs of live load (LL):

Allowable stress in original member =	18,000 psi
Dead load stress in original member =	10,000 psi
To be used in new steel to be added =	8,000 psi

$$\frac{20{,}000 \text{ lbs}}{8{,}000 \text{ lbs}} = 2.5 \text{ in.}^2 = \text{area of new steel to be added}$$

Check this as follows:

DL force	100,000 lbs ÷ 10.0 in.² =	10,000 psi
LL + I force	100,000 lbs ÷ 12.5 in.² =	8,000 psi
DL + LL + I	200,000 lbs	18,000 psi
	and 18,000 psi ≦	18,000 psi OK

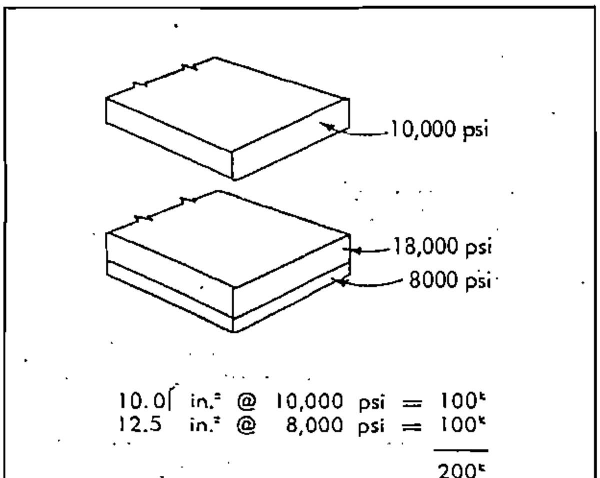


FIGURE 4

In making alterations to structures, existing rivets may be utilized for carrying stresses resulting from dead loads and welding shall be provided to carry all additional stress. However, if the framing is shored during repairs and the member to be reinforced is thus relieved of stress, the welding shall carry the entire stress.

AISC Requirements

AISC Sec 1.15.10: Rivets and Bolts in Combination with Welds. In new work, rivets, bolts or high strength bolts used in bearing type connections shall not be considered as sharing the stress in combination with welds. Welds, if used, shall be provided to carry the entire stress in the connection. High strength bolts installed in accordance with the provisions of Sec 1.16.1 as friction-type connections prior to welding may be considered as sharing the stress with the welds. In making welded alterations to structures, existing rivets and properly tightened high strength bolts may be utilized for carrying stresses resulting from existing dead loads, and the welding need be adequate only to carry all additional stress.

AASHO Requirements

AASHO 1.12.7: The unit working stresses used in determining the load-carrying capacity of each member of a structure shall take into account the type of material from which the member is made. The unit working stress assumed for the inventory rating shall not exceed 0.545% of the yield point and for the operating rating shall not exceed 0.82 of the yield point.

Where information concerning the specification under which the metal was supplied is not available, it will be assumed that the yield point does not exceed 30,000 psi for all bridges built after 1905.

Bridges built previous to 1905 shall be checked to see that the material is not of a fibrous nature. If it is fibrous or of doubtful character, the yield point will be assumed to be equal to that of wrought iron which shall be taken as 26,000 psi.

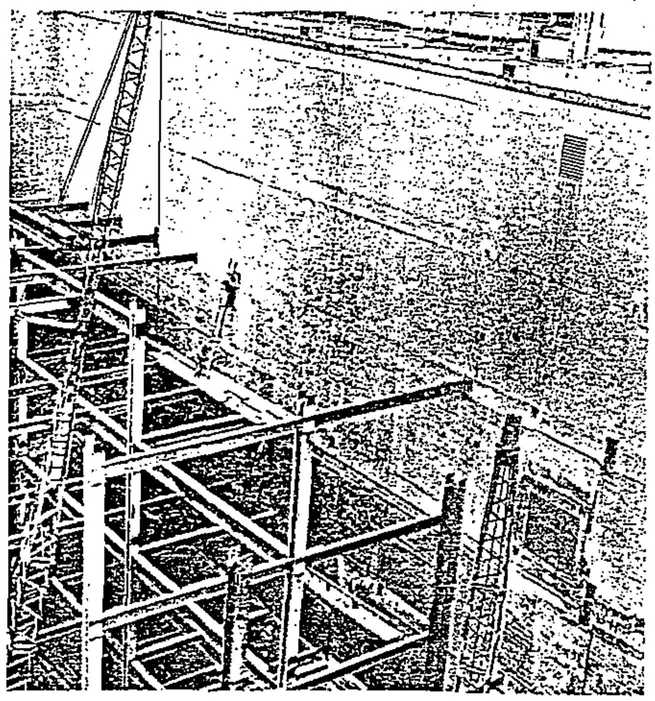
In the absence of definite information, it shall be assumed that the yield point of wrought iron is 26,000 psi, and the unit working stress shall be taken as 14,000 psi.

4. GENERAL

Proposed repairs and methods should be considered and approved by a qualified engineer. Welding on a job of this type should be of the best quality and adequately inspected. An E6010 type of electrode would normally be recommended for this welding, if it involves vertical and overhead positions or painted or dirty material. Material should be cleaned as thoroughly as possible before welding. If the material is unusually thick, a low-hydrogen electrode should be used, and it would be well to check for any preheat which might be recommended. See the following topic, Temperature for Welding.

When making a repair on a structure it is necessary to know the type of steel it is made of. It may be possible to get a mill report from the steel mill which furnished the steel. Sometimes on very old structures this information cannot be obtained. If this is an important structure, it would be a good idea to get test drillings and have them analyzed.

An experienced weldor will sometimes weld a small piece of mild steel to the structure and then knock it off with a hammer. If the weld cracks out of the base metal, taking some of it with the weld, this indicates that the steel is hardenable and the heat-affected zone adjacent to the weld has been hardened. If the weld itself cracks, this indicates higher carbon or alloy in the steel which has been picked up in the molten weld and become hard during cooling. In both cases, preheating and low hydrogen electrodes should be used. If the mild steel bar bends down without the weld breaking, this indicates good weldable ductile steel.



All structural work for a major addition to the Jordan-Marsh Department Store in Boston was completed without interruption of business. The concrete wall was penetrated and new steel welded successfully to vintage steel under load —without removal of the load.

There is little chance that the structure to be repaired is made of wrought iron, which was used in structures prior to 1900. Wrought iron contains slag rolled into it as tiny slag inclusions or laminations, and is low in carbon. The slag pockets might bother the welding operator a little, but this should be no real problem. Some engineers recommend that extra effort be made to fuse or penetrate well into the wrought iron surface, especially if the attached member is going to pull at right angles to the wrought iron member; otherwise they reason, the surface might pull out because of the laminations directly below the surface.

It is also possible for the sulphur content of wrought iron to be excessive, and it should be checked. Keep in mind that any chemical analysis for sulphur represents the average value in the drillings of steel taken for analysis. It is possible in wrought iron to have the sulphur segregated into small areas of high concentrations. The low-hydrogen electrodes (EXX15, EXX16 and EXX18) should be used where sulphur might be a problem.

The AISC published in 1953 a complete listing of steel and wrought iron beams and columns that were rolled between 1873 and 1952 in the United States.

5. TEMPERATURE FOR WELDING

The AWS Building and Bridge codes require that welding shall not be done when the ambient temperature is lower than 0°F. When the base metal temperature is below 32°F, preheat the base metal to at least 70°F, and maintain this temperature during welding.

Under both codes, no welding is to be done on metal which is wet, exposed to ice, snow, or rain, nor when the weldors are exposed to inclement conditions, including high wind, unless the work and the weldors are properly protected.

In general, the AISC and AWS specifications on minimum temperature for welding are a good guide to follow. See Table 1. The following thoughts might supplement them in producing better welds at these cold temperatures.

Welding on plates at cold temperatures results in a very fast rate of cooling for the weld metal and adjacent base metals. With thicker sections of mild steel, A7, A373, and A36, this exceptionally fast rate of cooling traps hydrogen in the weld metal. This reduces ductility and impact strength of the weld and may cause cracking, especially of the root bead or first pass. This type of weld cracking has been shown to occur almost entirely in the temperature range below 400°F.

With a preheat or interpass temperature of 200°F, this cracking does not occur, even with the organic type of mild steel electrodes. This is because the higher temperature results in a slower cooling rate, and more time for this entrapped hydrogen to escape.

Low-hydrogen electrodes greatly reduce the source of hydrogen and, therefore, the cracking problem. This weld metal has greater impact strength and a lower transition temperature. In general, the use of low-hydrogen electrodes will lower any preheat requirement by approximately 300°F.

The fastest cooling rate occurs with so-called "arc strikes", when at the start of a weld the electrode is scratched along the surface of the plate without any metal being deposited. This can be damaging and

TABLE 1—Minimum Preheat and Interpass Temperatures [1, 2]

Thickness of Thickest Part at Point of Welding, in inches	Welding Process	
	Shielded Metal-Arc Welding with Other than Low-Hydrogen Electrodes	Shielded Metal-Arc Welding with Low-Hydrogen Electrodes or Submerged Arc Welding
	ASTM A36[3], A7[3, 4], A373[3]	ASTM A36[5], A7[4, 5], A373[5], A441[6]
To 3/4, incl.	None[7]	None[7]
Over 3/4 to 1 1/2, incl.	150°F	70°F
Over 1 1/2 to 2 1/2, incl.	225°F	150°F
Over 2 1/2	300°F	225°F

[1] Welding shall not be done when the ambient temperature is lower than 0°F.
[2] When the base metal is below the temperature listed for the welding process being used and the thickness of material being welded, it shall be preheated for all welding (including tack welding) in such manner that the surfaces of the parts on which weld metal is being deposited are at or above the specified minimum temperature for a distance equal to the thickness of the part being welded, but not less than 3 in., both laterally and in advance of the welding.
Preheat temperature shall not exceed 400°F. (Interpass temperature is not subject to a maximum limit.)
[3] Using E60XX or E70XX electrodes other than the low-hydrogen types.
[4] See limitations on use of ASTM A7 steel in Par. 105(b).
[5] Using low-hydrogen electrodes (E7015, E7016, E7018, E7028) or Grade SAW-1 or SAW-2.
[6] Using only low-hydrogen electrodes (E7015, E7016, E7018, E7028) or Grade SAW-2.
[7] When the base metal temperature is below 32°F, preheat the base metal to at least 70°F.

should be avoided. Next to this in seriousness are very short tack welds.

The following will illustrate the effect which weld length has on cooling rate. The length of time to cool from 1600°F to 200°F when a single weld is placed on a ¾" plate is:

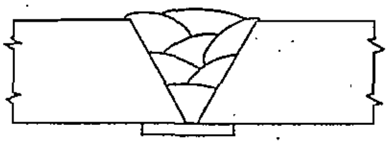
Length of Weld	2½"	4"	9"
Time (Seconds)	90.	300.	2000.

A weld 9" long made at a temperature of 70°F has about the same cooling rate as the same weld 3" long at a preheat of 300°F. Welds of larger cross-section have greater heat input per inch of weld. High welding current and slow travel speeds slow down the rate of cooling and decrease the cracking problem.

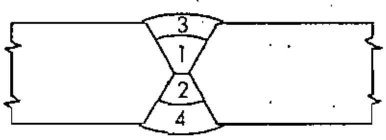
Perhaps the greatest difficulty in cold temperature welding is the discomfort of the welding operator. It becomes more awkward to move around the weld because of the extra clothing required. The welding lens continually becomes frosted or fogged from the breath of the operator. The helmet must be removed and the lens wiped.

6. WELDING OF INSERT PLATES

For thick plates, a double V or U joint would reduce the amount of weld metal and therefore transverse shrinkage. The balanced weld would preclude any angular distortion.



(a) Single Vee



(b) Double Vee

FIGURE 5

The use of round corners will tend to reduce any notch effect at the corners of the welded insert.

Sometimes the plate to be inserted is pre-dished, providing a little excess material in the plate to offset the transverse shrinkage. However, longitudinal shrinkage stresses will build up around the periphery of the plate, because the edge welded lies in a flat plane and therefore is more restrained.

The following sequence is usually used:

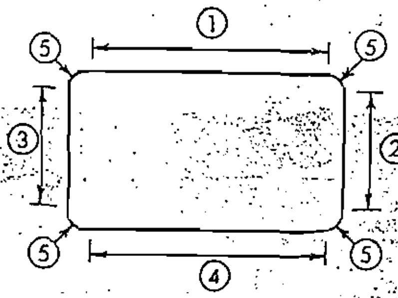


FIGURE 6

Weld side (1) complete. So far this should be rather unrestrained. A few tack welds on the opposite *side might crack; if so, they should be realigned* and rewelded. Weld side (2) complete. It might be argued that this is free to shrink because the opposite side (3) is unwelded. However there is some restraint offered by the weld along side (1). Now side (3) directly opposite side (2) is welded; this will start to lock-up now. Then weld side (4) opposite side (1). If either *weld (3) or (4) should crack, it should be gouged out* to sound metal and rewelded. Finally, the four corners (5) are completed.

Another suggestion is to estimated the amount of transverse shrinkage and to open up the joint initially by this amount, by driving in several hardened steel drift pins. The joint is then welded, full throat, up to *these pins. The pins are then removed,* and the joint completed.

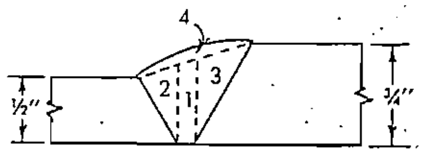


FIGURE 7

Figure 7 illustrates the geometrical method of obtaining the weld area. This value is needed to determine transverse shrinkage:

$$\text{transverse shrinkage } (\Delta) = 10\% \ \frac{\text{weld area}}{\text{thickness}}$$

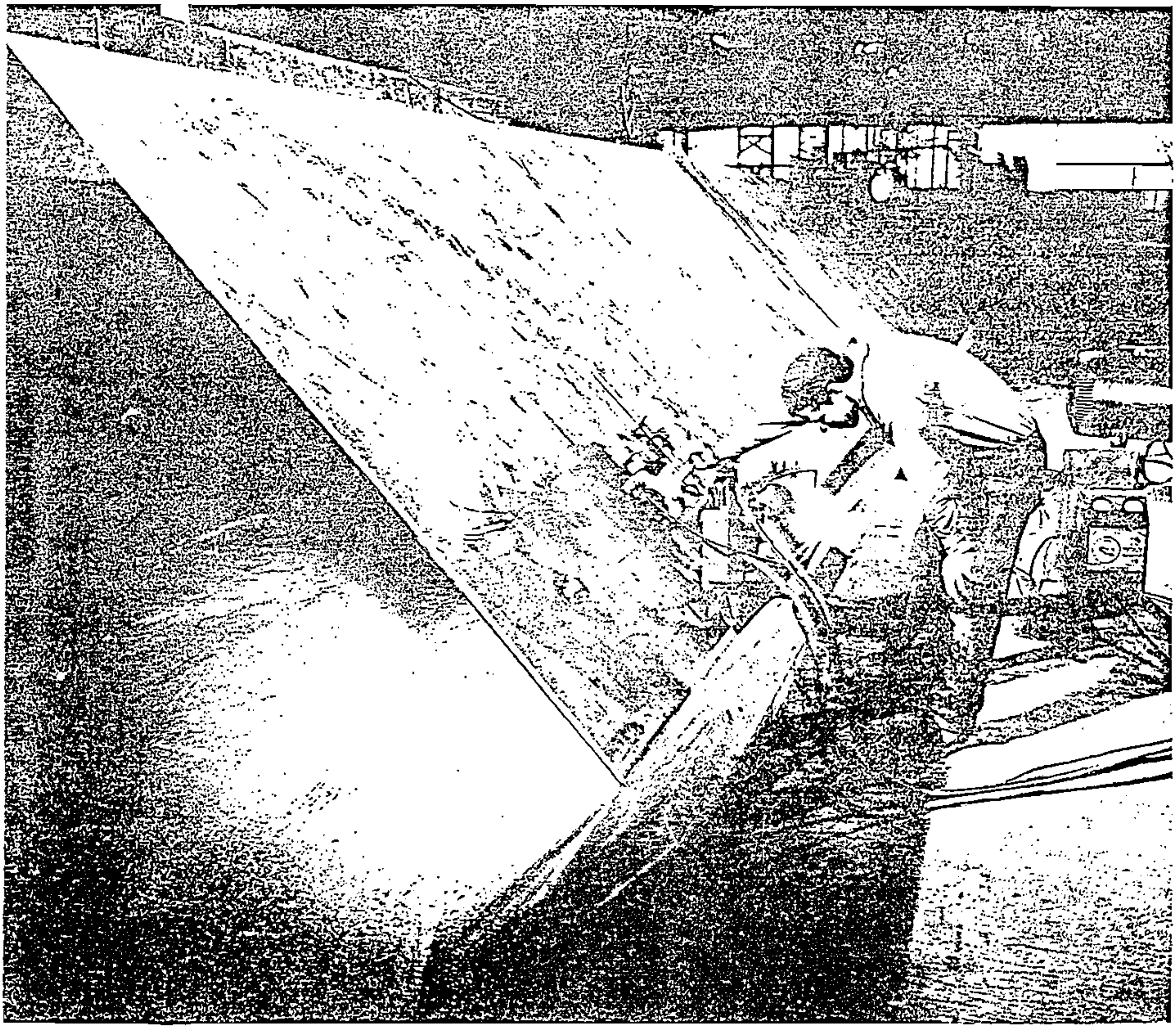
$$= 10\% \text{ average width of weld}$$

area of weld

$$\begin{aligned} (3/16'')(.62'') &= .1162 \\ \tfrac{1}{2}\ (.62'')(.30'') &= .0930 \\ \tfrac{1}{2}\ (.90'')(.30'') &= .1350 \\ \tfrac{2}{3}\ (1.0'')(.10'') &= .0667 \\ \hline & \ .4109 \text{ in.}^2 \end{aligned}$$

$$\therefore \Delta = .10 \ \frac{(.411)}{\left(\frac{\tfrac{1}{2}'' \times \tfrac{3}{4}''}{2}\right)}$$

$$= \underline{.07''}$$



In production of large plate girders, flange is commonly tack welded to the web. Then, with the girder web held at a 45° angle, the web-to-flange weld can be efficiently made using a self-propelled submerged-arc welding unit. This $\frac{1}{2}''$ fillet is here being made in two passes. Flange is 4″ thick, web $\frac{3}{4}''$. Improvements in equipment and technique are currently permitting many $\frac{1}{2}''$ fillets to be made in a single pass.