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Welding of Sheet Steel

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By Teoman Pekoz[1] and William McGuire[2]

INTRODUCTION

Light, cold formed steel sections have been arc welded without the benefit of a general guiding specification for many years. By the late nineteen sixties the structural use of this fastening method was sufficient to create a demand for a more systematic approach. Rational use of light steel panels as horizontal diaphragms and vertical shear walls, as well as other applications of light steel framing, panels, and decks, requires one. Accordingly, the American Iron and Steel Institute initiated a project to develop welding procedures and to verify them through tests of welded connections. In a series of such tests at Cornell University, the behavior of the most common types of arc welds in sheet steel has been studied. This paper is a summary of the Cornell tests and an interpretation of the results.

The Cornell research has provided the basis for a forthcoming revision of the welding provisions in the AISI Specification for the Design of Cold-Formed Steel Structural Members (Ref. 1) and for a new specification, Welding Sheet Steel in Structures, AWS D1.3-80 (Ref. 2).

Sufficient data are available to support the ultimate load prediction equations proposed in this paper and the design equations contained in the specifications referred to above. Since they represent the first attempt to codify this type of structural fastening process, it is anticipated that desirable modifications will become apparent as research and practice advance.

Sheet steel may be as thick as 0.230 inches. The thicknesses commonly used in cold-formed steel in building construction are generally not as large as this, however. The largest total sheet thickness used in the Cornell tests was approximately 0.150 inches.

Although sheet steel welds may be made with conventional equipment and electrodes, the fact that they are made on thin steel results in a special situation. Stress resisting areas are not as regular or as easy to define as they are in the welding of structural steel and plate. Some welds, such as arc spot and arc seam welds (Ref. 3), are made through the welded sheet without any advance preparation. Galvanizing and paint are normally not removed prior to welding. Failure modes are complex and difficult to categorize. A relatively large amount of scatter in test results can be

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expected. Qualification of welders and welding procedures, and the inspection of work, are of particular importance. The fact that a welder may have satisfactorily passed a test for structural steel welding does not necessarily mean that he can produce sound welds on sheet steel. Welders may require considerable instruction and practice before mastering the technique.

Weld Types

The types of arc welds used to connect a light steel sheet to another plate, either light or heavy,are shown in Fig. 1. Most of the terms used follow standard nomenclature. Arc spot welds (commonly called puddle welds) are welds in which coalescence proceeds from the surface of one member into the other. As mentioned above, the weld is made without preparing a hole in either member. Arc seam welds (oblong puddle welds) are the same in that neither member is slotted. Arc spot and seam welds are commonly used to attach cold formed steel decks and panels to their supporting frames. Arc seam welds find particular application in the narrow troughs of such elements. Flare bevel and flare vee welds are used on the outside of the curved edges that are typical of cold formed members. Square groove welds are rarely used in thin steel.

As in conventional structural welding, it is general practice to require that the deposited filler metal have a tensile strength at least equal to that of the members being joined. For members of unequal strength, the weld materials should be matched at least to the strength level of the weaker member.

Failure Modes

Failures in welded sheet steel connections are generally quite complicated. They often occur as a combination of basic modes, accompanied by a large amount of out-of-plane inelastic deformation. The primary features of the basic modes encountered in the Cornell tests are illustrated in Fig. 2. While these are simplified pictures of true failures, they have been found, nevertheless, to provide reasonable categories for the assessment of strength and the development of design formulas. Photographs of some of the typical failed specimens are given in Fig. 3. For simplicity, groove weld failures are not shown. Properly matched groove welds can be expected to develop the full strength of the sheet.

For fillet welds on the sheet sizes tested, the dimension of the leg on the sheet edge is generally equal to the sheet thickness and the other leg is often two or three times longer. The throat is commonly larger than the throat of a conventional fillet weld of the same size (see Section A-A, Fig. 2a). Ultimate failure of fillet welded joints is usually found to occur by tearing of the plate adjacent to the weld. Tearing is the result of applied shearing or tensile forces, depending upon whether the weld is longitudinal or transverse. These conditions are illustrated in Fig. 2a and 2b. Also, in a number of the longitudinally welded specimens tested at Cornell, the welds were long enough to result in tensile failure of the narrow connected sheets. Some conventional weld shear was also observed in a few of the longitudinally welded specimens. These and other failure conditions will be described further in later sections of this report.

The chief mode of failure in cold-formed channels welded by flare bevel welds, and loaded transversely, was also sheet tearing along the contour of the weld. Fig. 2c shows these conditions.

Only in a few cases was weld shear a primary factor in the failure of either fillet or flare bevel welds. Most failures were accompanied by inelastic out-of-plane deformation of the connected plates.

Three modes of ultimate failure of arc spot welds were observed in the Cornell tests (see Fig. 2d). The first is simple shear failure of the weld metal in the plane of the faying surface. The second is plate tearing on the loaded side of the sheet. Failure of this sort starts by tearing along the contour of the weld; it then progresses across the sheet. In the third mode, tearing along the contour of the weld on the tension side is followed by plowing of the weld into the end material as that material buckles and shears, as shown in the third sketch of Fig. 2d. This type of failure may occur when the end distance is small. Many failures, particularly those of the plate tearing type, may be preceded or accompanied by considerable inelastic out-of-plane deformation of the type indicated in Fig. 4 . This is a form of instability similar to that observed in wide, pin-connected plates.

The general behavior of arc seam welds is similar to that of arc spot welds. No simple shear failures of arc seam welds were observed in the Cornell tests however.

In most cases the onset of yielding was either poorly defined or followed closely by ultimate failure. As in most connections, rupture rather than yielding is a more reliable criterion of failure.

TESTING PROGRAM

Tests were conducted at Cornell for the American Iron and Steel Institute on 342 symmetric fillet, flare bevel, arc spot and arc seam welded connections subjected to monotonically increasing static loading. A breakdown of the program is as follows:

Type	Number of Specimens
Transverse fillet welds	55
Longitudinal fillet welds	64
Transverse flare bevel welds	42
Longitudinal flare bevel welds	32
Arc spot welds	126
Arc seam welds	23
Total	342

130 connections were made in steel fabricating shops, 122 were made under field conditions, and 90 were fabricated in the Cornell laboratory under simulated field conditions.

All specimens had the same basic configuration. Two plates were butted together ahd having one, or in the case of double sheet arc spot and arc seam welds, two cover plate sheets welded to each side. All specimens were welded with E6010 electrodes. In most cases the connected plates were 7/16 inch thick hot rolled A36 steel plates. In some cases the connected plates were sheets having a thickness equal to or greater than the cover plate sheets. Seven different cover plate gages were investigated: 10 ga (0.138 in.), 12 ga (0.108 in.), 14 ga (0.079 in.), 18 ga (0.052 in.), 22 ga (0.034 in.), 24 ga (0.028 in.), 28 ga (0.019 in.). All of the 10, 12 and 22 gage steel, most of the 18 gage material, and some of the 14 gage cover plate sheets, were made from A446, Grade A steel (minimum σ_y = 33 ksi and σ_u = 45 ksi). The remainder of the cover plate sheets were A446, Grade E steel (minimum σ_y = 80 ksi and σ_u = 82 ksi). Tension coupon tests were made of all cover plate steel used. The measured ultimate strengths are used in the strength prediction formulas cited below.

Arc spot and arc seam welded specimens with single and double sheet cover plate were tested. The double sheet condition is encountered in practice when overlapping sheets are fastened to the supporting frame by welds that penetrate both plies of material.

Complete details of the test program and the results are contained in Refs. 4 through 8. A summary of the specimen and test data needed to interpret the results is given in Ref. 10.

TEST RESULTS AND STRENGTH PREDICTIONS

In the following sections, the performance of each of the types of weld investigated is summarized. Formulas for predicting the ultimate resistance of each type of connection are presented and compared with the test results. All the formulas are given for units of kips and inches. The predicted ultimate loads, P_u, are given for a single weld. The predicted ultimate loads for each specimen, P_{up}, is to be found by multiplying P_u by the number of welds that must fail in order to cause the failure of the specimen.

Transverse Fillet Welds

A total of 55 transverse weld specimens were tested. The cover plate material was either 12 or 18 gage A446, Grade A steel. Complete details are contained in Refs. 4 and 5. In all but eight of the tests, primary failure was by tearing of the connected sheets along, or close to, the contour of two of the welds. In the remainder, there was secondary weld shear. In seven of the tests, ultimate failure was preceded by substantial out-of-plane plastic deformation.

Fig. 5 presents a comparison of the experimental ultimate load P_{ou} with the failure load P_{up} predicted from the formula

$$P_u = tL\sigma_u \tag{1}$$

where t is the cover plate thickness, L is the length of the weld, and σ_u is the measured ultimate strength of the cover plate material. P_u is the ultimate load per weld. The ultimate load for the specimen P_{up} is twice P_u since the specimen failure involves the failure of two welds. For the twenty-four shop welded connections the average ratio of observed to predicted ultimate strength is 1.04, with a standard deviation of 0.09. For the thirty-one field welded specimens the average and standard deviation are 0.97 and 0.11 respectively and, for all specimens, these values are 1.00 and 0.11. It is believed that Eq. 1 is an excellent predictor of the failure strength of transverse fillet welds.

The basic reason for the ability of transverse fillet welds to develop the tensile strength of the adjacent sheet appears to be the one referred to earlier in the discussion of Fig. 2a. For welds on thin sheets, the dimension of the weld leg on the sheet edge is generally equal to the sheet thickness and the weld throat is commonly larger than the throat of a conventional fillet weld of the same size. Under these circumstances, if the deposited filler metal has a tensile strength greater than that of the sheets being joined, as should be the case with conventionally matched materials and properly made welds, it can be expected that the sheet is the critical element.

Fig. 5 is a graphical comparison of the actual and predicted strengths.

Longitudinal Fillet Welds

A total of 64 longitudinal fillet weld specimens were tested. Again, all of these tests were on 12 or 18 gage A446, Grade A material. Complete details are contained in Refs. 4 and 5. In 33 of the tests, tensile tearing across the connected sheets was either the sole cause of failure or a major contributing factor. In the remainder of the tests, failure was the result of weld shear, weld peeling, tearing of the sheet along, or roughly parallel to, the contour of the weld, or a combination of these effects. In many of the longitudinally welded specimens there was also a substantial amount of out-of-plane deformation.

The following equation was found satisfactory to predict failure by tensile tearing across the cover plate:

$$P_u = .4tS\sigma_u \tag{2}$$

where S and t are the width and thickness of each cover plate, respectively. However, failures involving tearing along the weld contour, weld shear and combinations of the two were predicted satisfactorily by the smaller value obtained from:

$$P_u = t L \left(1 - 0.011 \frac{L}{t}\right) \sigma_u \tag{3a}$$

or

$$P_u = .75 t L \sigma_u \tag{3b}$$

where L is the average weld length. The ultimate load for the specimens, P_{up} is four times P_u.

In the 33 tests in which tensile tearing across the sheet was a primary factor, it was observed that it occurred at an average stress on the cross section of the connected plates equal to about 80% of the ultimate strength of the sheet material. That is the reason for Eq. (2).

It was also observed that, for all other failures there appeared to be some correlation between the ultimate resistance of the connection and the length of the welds. Indeed, for very short welds, average stresses obtained by dividing the actual ultimate load by the product of the sheet thickness and total weld length were close to the ultimate strength of the sheet material. Eq. 3a was developed through a linear regression analysis of the results of the 31 tests not influenced by transverse plate tearing. It it believed that, for long L/t ratios, Eq. 3a would become overly conservative and that the limiting resistance for such specimens would become the ultimate shearing resistance of the sheet material. Assuming this to be 75% of the ultimate tensile strength (a value for shear strength which has reasonable empirical support in similar applications) Eq. 3b results. By equating the right hand sides of Eqs. 3a and 3b, it is readily seen that Eq. 3b controls for welds having an L/t ratio greater than 22.7.

Applying Eqs. 2 and 3 to the test specimens it is found that Eq. 2 controls in 38 cases, Eq. 3a in 18 cases, and Eq. 3b in 8 cases. The average ratio of observed to predicted ultimate strength and the corresponding standard deviation are, for each equation in the regime in which it controls: Eq. 2, 1.00 and 0.10; Eq. 3a, 1.05 and 0.08; Eq. 3b, 0.89 and 0.09.

The basic reasons why failures tended to initiate in the sheet rather than in the welds are believed to be the same as those cited for transverse welds; mainly the relative strengths of the weld and sheet materials, and the relatively large weld cross section dimension.

Fig. 6 is a graphical comparison of the actual and predicted strengths.

Transverse Flare Bevel Welds

The basic data and results of the 42 transverse flare bevel weld tests are summarized in Refs. 4 and 5. The channels were cold formed from 12 or 18 gage Grade A material. By far the most common mode of failure was plate tearing. In only five tests was weld shear a factor. Significant out-of-plane distortion was experienced in twelve tests.

The experimental failure loads predicted satisfactorily by the formula

$$P_u = .4tL\sigma_u \tag{4}$$

where each quantity is as previously defined. The ultimate load of the specimens P_{up} is four times P_u. For the twenty-six shop welded connections the average rate of observed to predicted ultimate strength is 0.97, with a standard deviation of 0.15. For the sixteen field welded specimens the average and standard deviation are 1.16 and 0.14 respectively and, for all specimens, these values are 1.04 and 0.17.

The basic reasons why failures tended to originate in the connected sheet rather than the weld appear to be the relative strength of the two materials and the weld dimensions. With one exception the effective weld throat dimension was greater than the sheet thickness. It is believed that this will also be the case in practice for welds made according to Ref. .

Fig. 7 is a graphical comparison of the actual and predicted strengths.

Longitudinal Flare Bevel Welds

The basic data and results of the 32 longitudinal flare bevel weld tests are contained in Refs. 4 and 5. In 22 of the tests, tensile tearing across the connected channel sections was either the sole cause of failure or a major contributing factor. In the remainder of the tests, failure was the result of weld shear or a combination of weld shear and plate tearing parallel to the weld contour, generally accompanied by out-of-plane deformation.

The failure predicted from the formula

$$P_u = .4A\sigma_u \tag{5}$$

where A is the area of the channel cover plate and two times the result obtained from Eq. 3b. The result obtained from Eq. 3b was multiplied by two in order to account for the fact that the shear force is resisted by the upstanding flange as well as the web of the channel. The ultimate load, P_{up}, for the specimens is four times P_u.

Applying Eqs. 5 and 3b to the test specimens it is found that Eq. 5 controls in 19 cases and Eq. 3b in 13 cases. The average ratio of observed to predicted ultimate strength and the corresponding standard deviation are, for each equation in the regime in which it controls: Eq. 5, 1.03 and 0.10; Eq. 3b, 1.01 and 0.14.

Fig. 8 is a graphical comparison of the actual and predicted strengths.

Arc Spot Welds

The basic data and results of the 126 arc spot weld tests are contained in Refs. 4, 5, 6, and 8.

In evaluating these tests, clarity requires that a distinction be made between those which failed in pure shear and those which failed in one of the other modes. In 31 shear failures, measurements were made of the net areas of the sheared welds, which contained substantial pitting and porosity. These irregular surfaces were converted to circles of the same area, and the equivalent diameter, d_{en}, recorded. The linear equation found to provide the best fit to these diameters is

$$d_{en} = 0.70d - 1.5t \tag{6}$$

where d is the visible diameter and t is the net sheet thickness. The measured equivalent diameter in the specimens tested ranged from 0.39 in. to 0.70 in. This equation is plotted in Fig. 9 for illustration. Weld shear failure loads were predicted satisfactorily by the formula

$$P_u = \frac{3\pi}{16} d_{en}^2 \sigma_{uw} \tag{7}$$

where σ_{uw} = 60 ksi, the nominal tensile strength of E60 filler metal. The ultimate load for the specimen, P_{up}, is two times P_u.

Based on an analysis of conditions in the cover sheets in the immediate region of the arc spot welds, Mr. Omer Blodgett of the Lincoln Electric Company proposed, in unpublished correspondence, two formulas for the prediction of the strength of arc spot welded connections that fail by plate tearing. The Blodgett formulas incorporate the observation that, for cases in which weld shear failure did not control, failure was generally by transverse tearing when d/t was less than $240/\sqrt{\sigma_y}$, and by longitudinal tearing and end zone buckling where d/t was greater than $240/\sqrt{\sigma_y}$, where σ_y is the yield stress of the sheet material. The best fit formulas were found to be, for $d/t < 140/\sqrt{\sigma_u}$

$$P_u = 2.2td\sigma_u \tag{8}$$

and, for $d/t \geq 240/\sqrt{\sigma_u}$

$$P_u = 1.4td\sigma_u \tag{9a}$$

For the range $\frac{140}{\sqrt{\sigma_u}} \leq \frac{d}{t} \leq \frac{240}{\sqrt{\sigma_u}}$ the following transition equation seems reasonable:

$$P_u = .28\left[1 + \frac{960t}{d\sqrt{\sigma_u}}\right]td\sigma_u \tag{9b}$$

In the above equations $d = d_v - t$, where d_v is the visible diameter and t is the net thickness of the single-ply or double-ply welded sheet. The limits of applicability of these equations are related to the ultimate strength rather than the yield strength of the steel. In each case, the ultimate load, P_{up}, is twice P_u.

The average ratio of the observed to the predicted strengths for the 78 tests in which Eq. 8, 9 or 10 controlled the predicted failure load is 1.07. The standard deviation is 0.26. The average ratio of the observed to the predicted strength for the 45 tests in which Eq. 7 governed is 1.22 and the corresponding standard deviation is 0.37. The conservative nature of Eq. 7 can be justified on the basis of the variability of the weld quality and particularly on the amount of porosity encountered in practice. All of the field welded arc spot welds reported in Ref. 5 were poorly made.

Fig. 10 provides a graphical comparison of the weld shear and plate failure formulas with the observed results.

Arc Seam Welds

The basic data and results of the 23 arc seam welds are contained in Ref. 8.

Based on an analysis of conditions in the cover plates in the immediate region of the arc seam welds, Mr. Omer Blodgett proposed, in unpublished correspondence, a formula for the prediction of the strength of arc seam welds that fail by a combination of tensile tearing of the sheets along the forward edge of the weld contour plus shearing of the sheets along the sides of the welds. Linear regression analysis performed by the authors on the results of the tabulated tests has resulted in the following, modified version of the Blodgett formula:

$$P_u = t\sigma_u(.63L + 2.4B) \tag{10}$$

where L is the overall length and B is the width of seam welds. The ultimate load of the specimens, P_{up}, is twice P_u.

The average ratio of the observed to the predicted strengths for all of the arc seam weld tests is 1.01. The standard deviation is 0.10.

Fig. 11 provides graphical comparison of the failure prediction formulas with the observed results.

SAFETY FACTORS AND ALLOWABLE STRESSES

The Cornell research program has been concerned with the investigation of the ultimate strength of various forms of arc welded connections in sheet steel. The following are some comments on the conversion of the strength prediction equations advanced here into design formulas.

The currently prevailing American view on the selection of safety factors for connections is indicated in a passage from Ref. 9: "If past practice is studied for riveted or bolted structural carbon steel joints, the factor of safety against sheet failure is found to vary from approximately 3.3 for compact joints to approximately 2.0 for joints with a length in excess of 50 inches. . . . Experience has shown that this factor of safety has provided a safe design condition. This indicates that a minimum factor of safety of 2.0 has been satisfactory; the same margin is also used for fasteners in tension." Similarly, in the American Institute of Steel Construction Specification, the basic allowable tensile stress is 0.60 F_y, but not more than one-half of the maximum tensile stress of the steel.

American practice in the design of statically loaded welded connections implies a basic nominal factor of safety of 2.5 with respect to failure. Thus, if, as in Eqs. 3b and 7 it is assumed that the ultimate strength in pure shear is 75% of the ultimate tnesile strength, it follows that the allowable shear stress obtained using a safety factor of 2.5 is $0.30\sigma_u$ or $0.30\sigma_{uw}$. The latter is the value prescribed for weld shear in buildings in Ref. 2. If one considers the uncertainties which are inevitable in the strength of connections, a nominal safety factor of 2.5 is consonant with the intention of having a minimum margin of safety of approximately two. The authors believe that this is a reasonable minimum margin of safety for conventional applications of sheet steel in buildings. It follows that they believe that working stress formulas obtained by applying a factor of safety of 2.5 to the ultimate resistance formulas proposed above will be reasonable design formulas.

WELDING PROCEDURES

Although this is primarily a report on the results of experimental research on the strength of welded connections, it is appropriate to include brief summaries of some of the practical requirements for obtaining sound welds in sheet steel. Detailed criteria for proper workmanship, technique, qualification, and inspection are contained in Ref. 2. Unless these criteria are satisfied, welds of the quality presumed in the above prediction equations may not be obtained.

Details, Workmanship, Technique

It is intended that arc spot welds have a fused nugget of at least 1/2 inch diameter into the supporting structural piece. The capability for making such welds is assessed during qualification tests. Generally, a flat or horizontal weld position is preferred. It is also necessary that parts to be joined be brought into close contact to facilitate complete fusion.

Effective control of current is absolutely essential for obtaining consistently sound welds. The current required for arc spot or arc seam welding is considerably higher than for most conventional welds. In preparing specimens for the Cornell tests, E6010 electrodes were used, as noted earlier. In one weld qualification test using 5/32 inch electrodes to make 1 inch (visible diameter) arc spot welds in 0.108 inch galvanized sheet, the current was 275 amps and the welding time approximately 6 seconds. The burn-off rate (called the melting rate by the AWS) of the electrode was about 22 inches/min. Using 1/8 inch electrodes to make 3/4 inch arc spot welds in 0.052 inch galvanized sheet, 210 amps and 10 seconds were required. The burn-off rate was 18 inches/min.

There is a considerable body of opinion among welding experts that the best practical way to maintain uniformity in sheet steel welding is through regulation of the electrode burn-off rate.

In making arc spot welds in sheet of 24 gage (0.028 in.) and lighter, weld washers may be required. These are small tabs of 16 gage (0.064 in.) or similar material with punched holes somewhat smaller in diameter than the visible weld diameter (see Fig. 12). They permit the weld to be made without burning the thin sheet.

Because of the relatively high currents used in arc spot and arc seam welding, the coating on some electrodes may break down and produce shallower penetration than that required. This may necessitate limiting the number of welds which may be made in rapid succession with one electrode.

Qualification, Inspection

Both the procedure and the welder must be carefully qualified following rules prescribed in an appropriate specification such as Ref. 2. Such rules include simple but severe mechanical tests on sample welds.

CONCLUSIONS

The results of an extensive test program have been evaluated and strength prediction equations have been derived. The strength prediction equations can be converted into design equations through the use of appropriate safety factors as discussed in this report.

Except for the case of the arc spot welds, the correlation between the test results and the computed results is quite satisfactory. In the case of the arc spot welds the variability of the quality of welds has led to a rather large scatter in the test results.

The application of the proposed equations presupposes welds made according to the quality standards of the Welding Sheet Steel in Structures, AWS D1.3-80 (Ref. 2).

ACKNOWLEDGMENTS

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APPENDIX — NOTATION

A = area of channel cover plate, in.^{2};

B = average width of arc seam welds, in.;

d = $d_v - t$;

d_v = visible diameter of an arc spot weld, in.;

L = average length of the welds of the specimen, in.;

P_u = ultimate strength of each weld, k;

P_{uo} = observed ultimate strength of the connection, k;

P_{up} = predicted ultimate strength of the connection, k;

S = average cover plate width, in.;

t = average cover plate thickness, in.;

σ_u = ultimate stress of the cover plate material, ksi;

σ_{uw} = nominal tensile strength of E60 filler material, ksi;

σ_y = yield stress of the cover plate material, ksi.

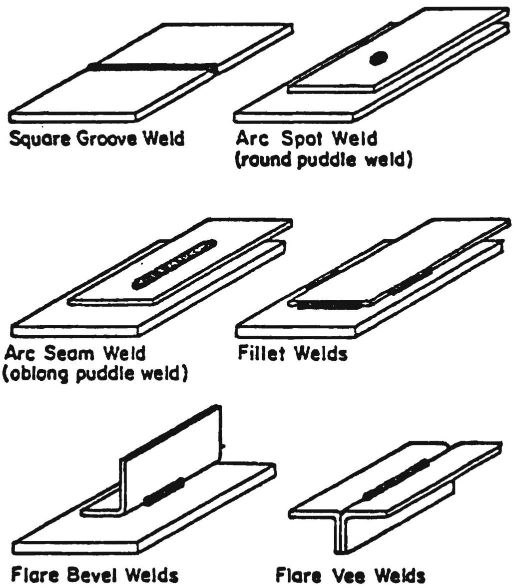


Fig. 1 Sheet Steel Weld Types

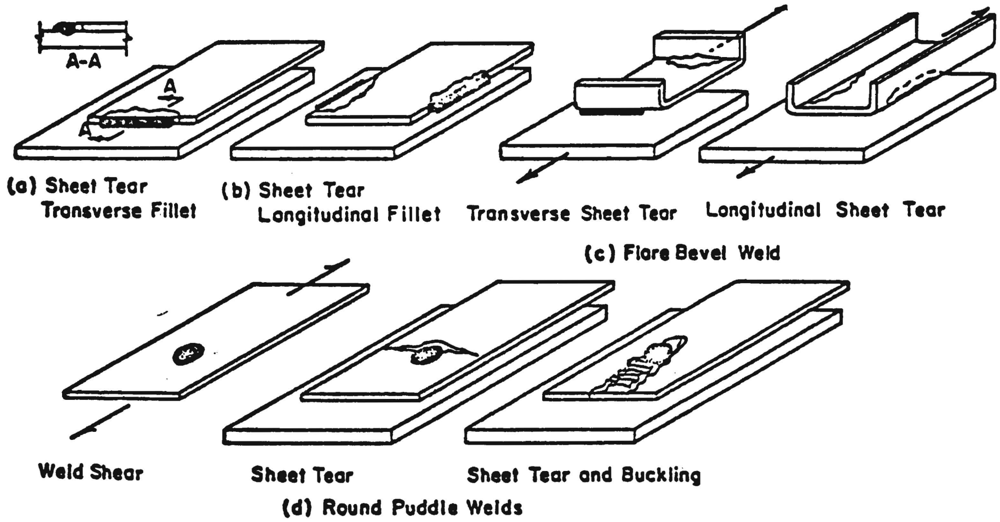
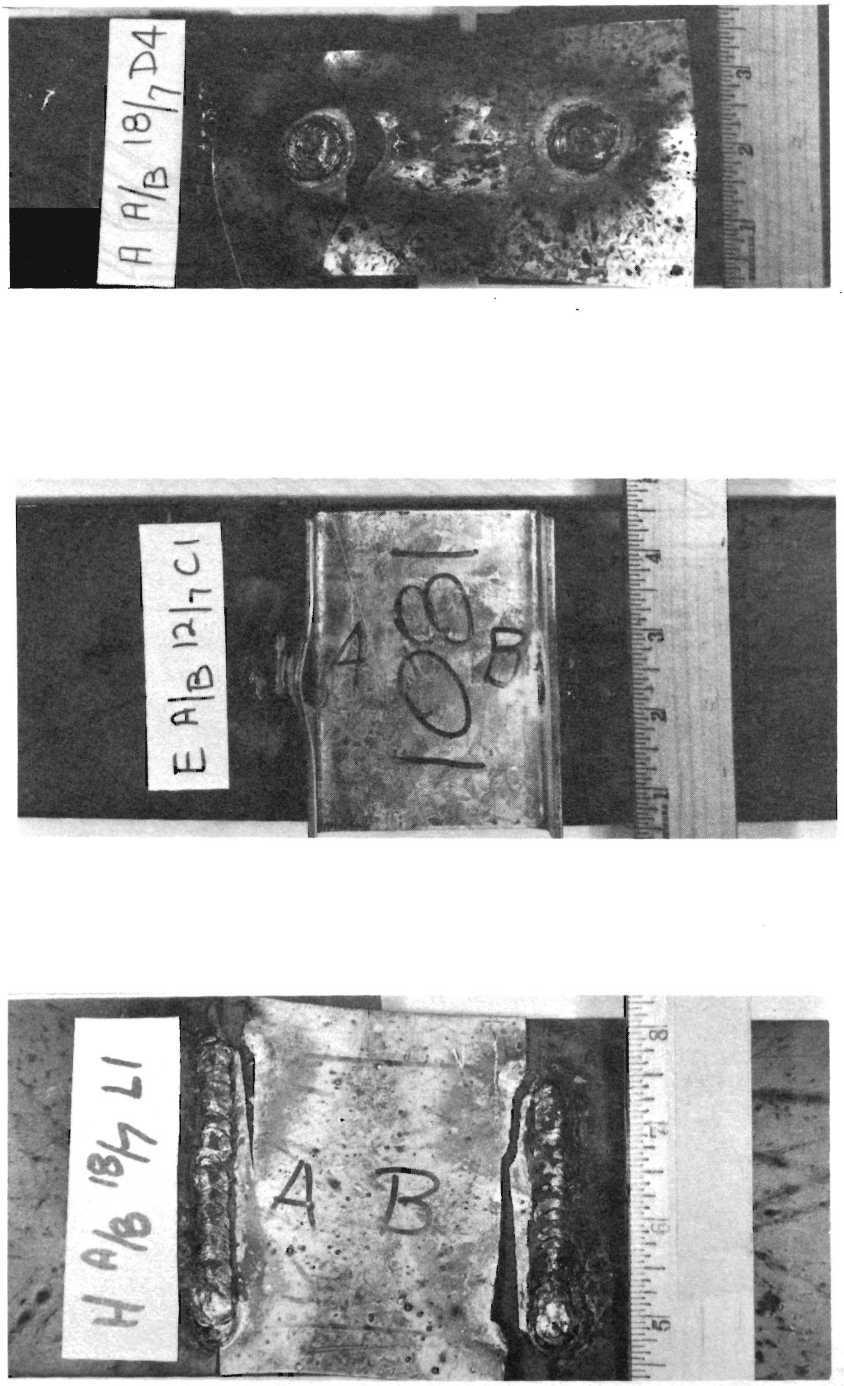


Fig. 2 Typical Failure Modes

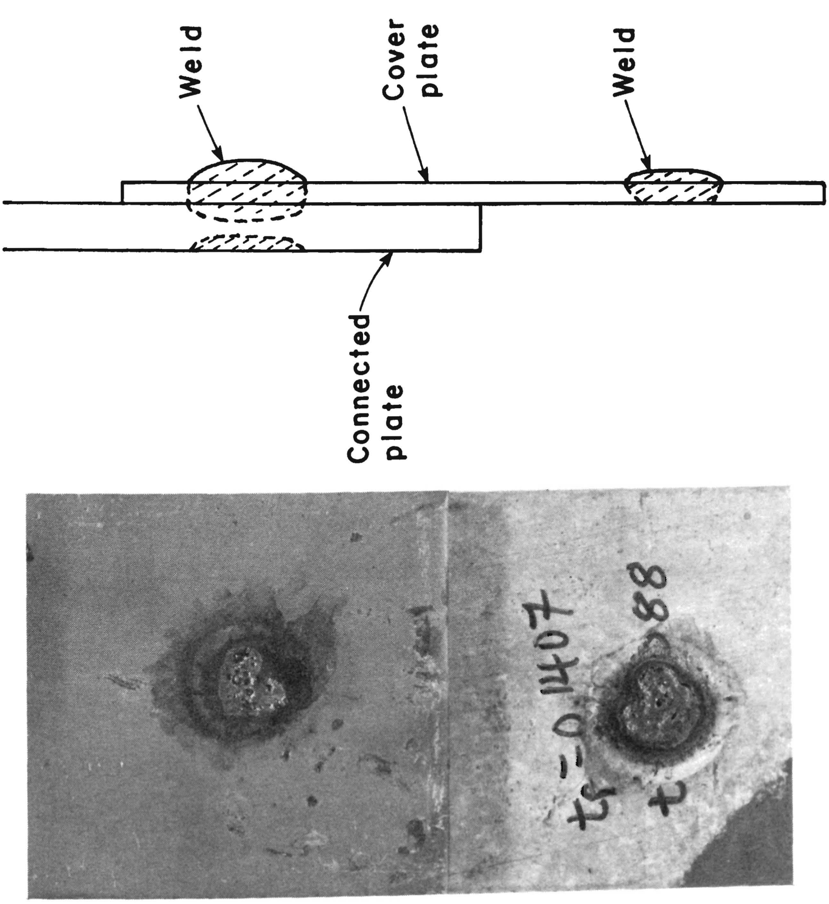


Sheet Tear
Transverse Fillet Weld

Sheet Tear
Transverse Flare
Bevel Weld

Sheet Tear
Round Puddle Weld

Fig. 3a Typical Failure Modes
(Specimens after failure)



Front View

Side View

Weld Shear - Round Puddle Weld

Fig. 3b Typical Failure Modes (Specimen after failure)

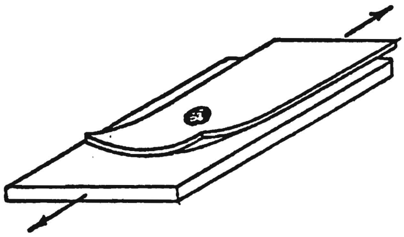


Fig. 4 Out of Plane Distortion

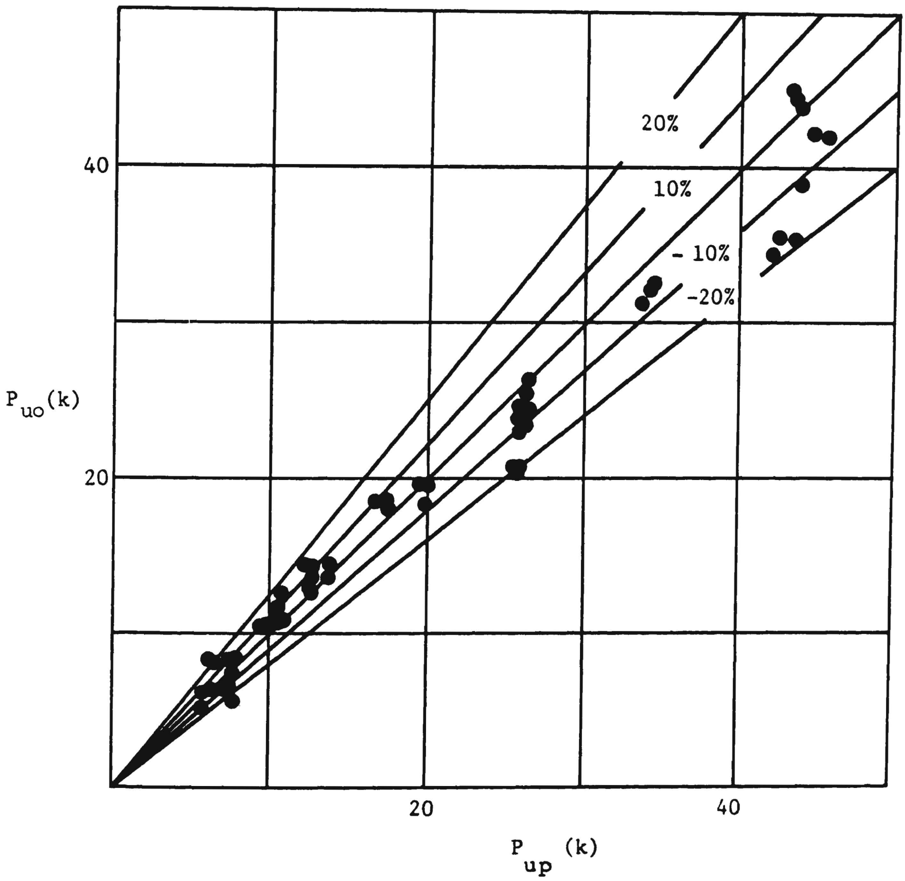


Fig. 5 Transverse Fillet Welds
P_{up} is according to Eq. 1.

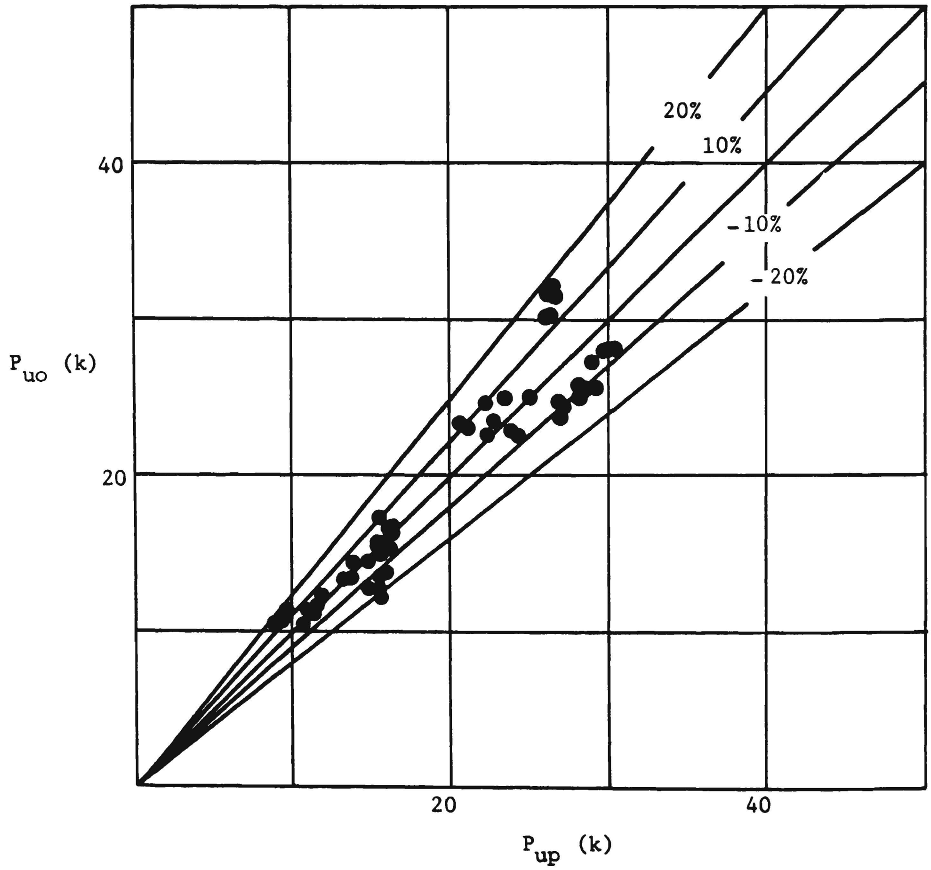


Fig. 6 Longitudinal Fillet Welds
P_{up} is according to Eq. 2, 3a and 3b.

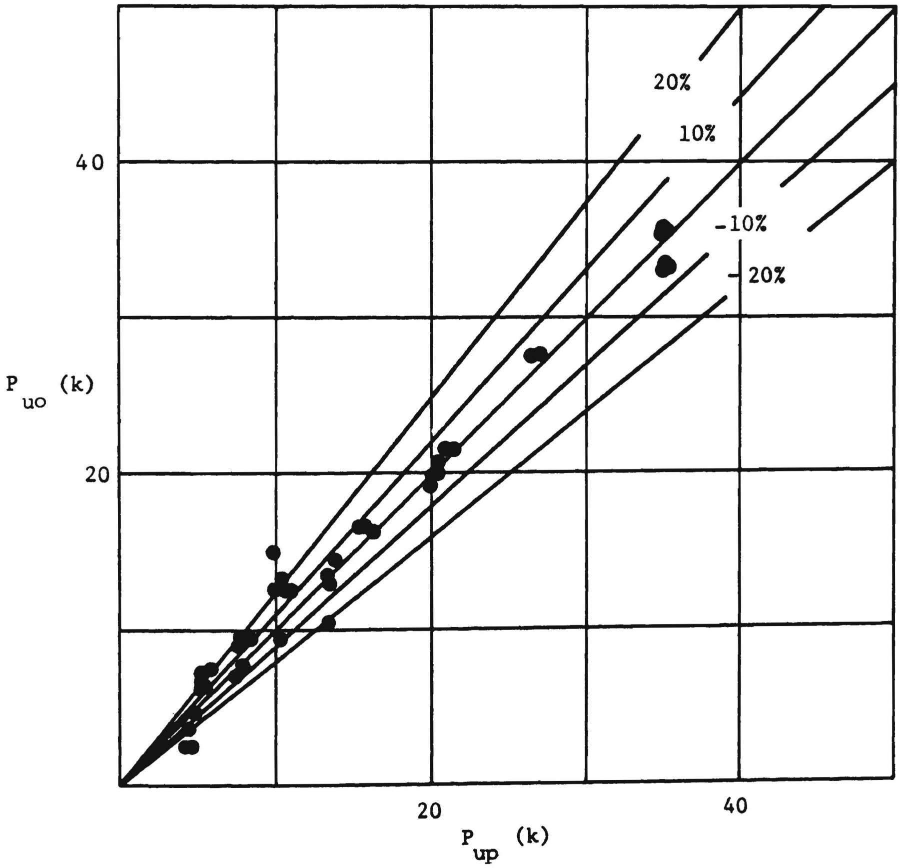


Fig. 7 Transverse Flare Bevel Welds
P_{up} is according to Eq. 4.

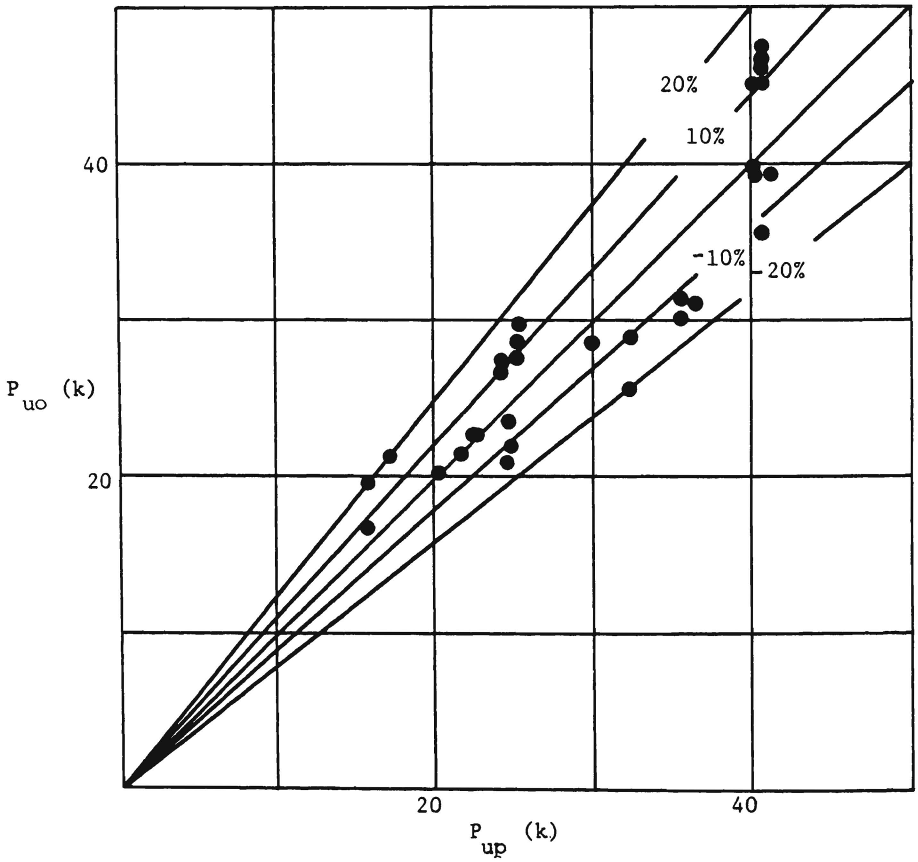


Fig. 8 Longitudinal Flare Bevel Welds
P_{up} is according to Eq. 5 or two times Eq. 3b.

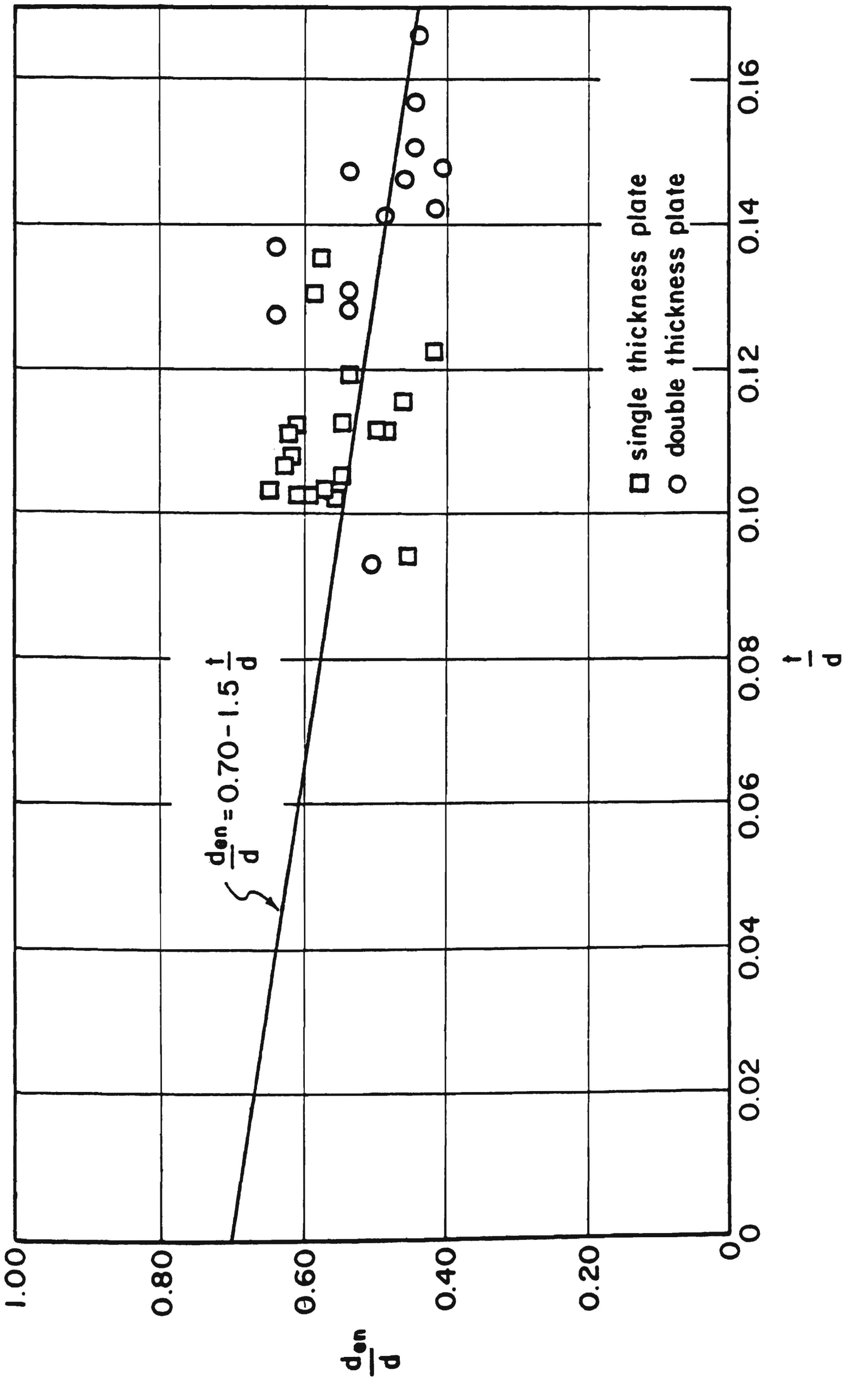


Fig. 9 Effective Net Diameter Plotted Against Cover Plate Thickness

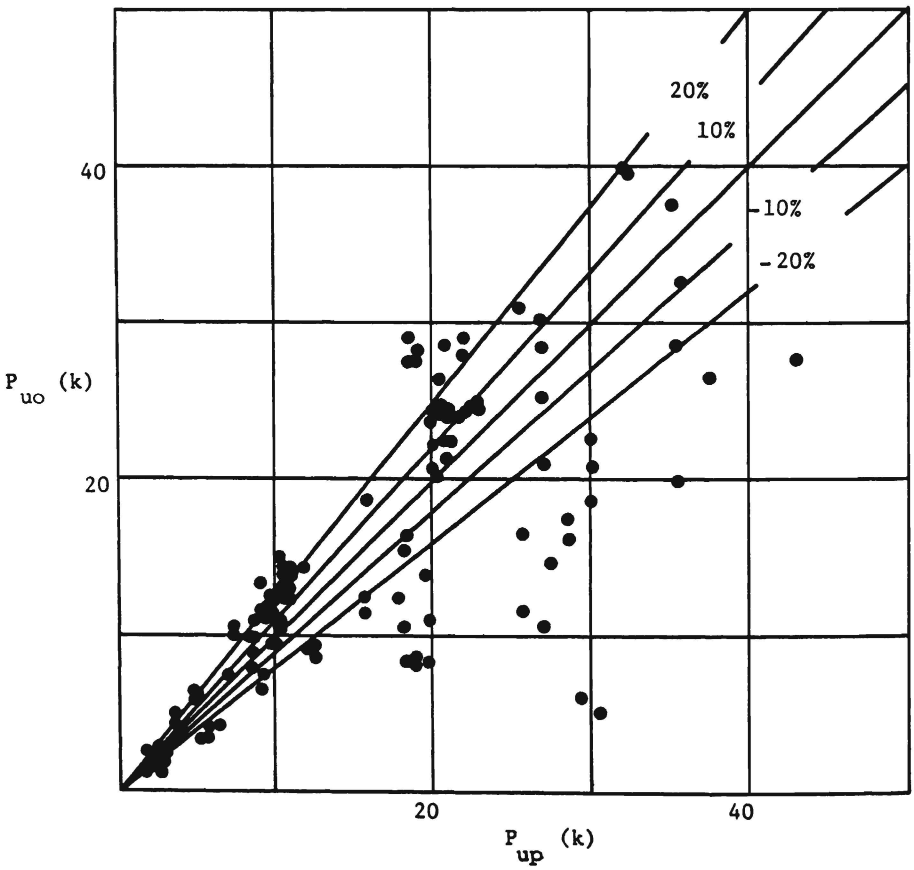


Fig. 10 Arc Spot (Puddle) Welds
P_{up} according to Eq. 8, 9, or 7.

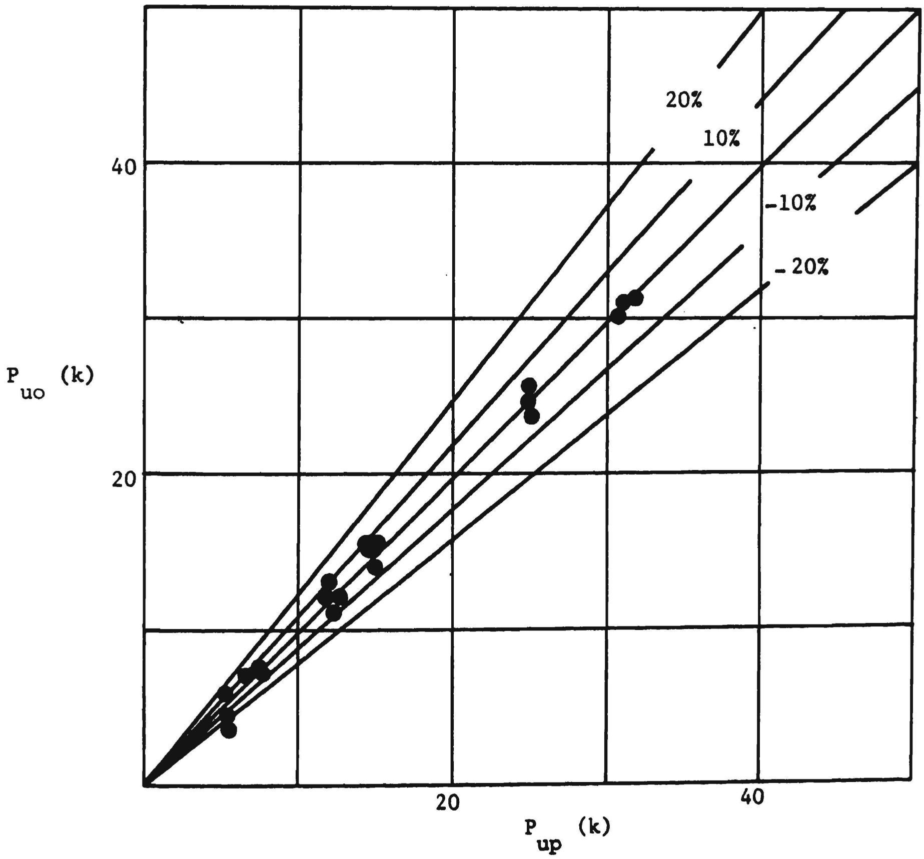


Fig. 11 Arc Seam Welds
P_{up} according to Eq. 10.

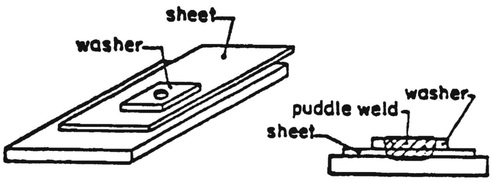


Fig. 12 Weld Washer