Stress Analysis and Design of Double Fillet-Welded T-Joints

Analysis showed new design procedures may be required that address the interaction of weld strength, joint design and load

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ABSTRACT. The stress distribution in double fillet-welded T-joints was investigated with a computer modeling technique. The finite element method was used for the analysis of T-joints in the plane-stress condition, under static load. Photoelastic stress analysis was employed to check the validity of the computer calculations.

To address the design aspect of the stress analysis, the American Welding Society (AWS) welding design procedure was followed, and the AWS permissible design loads were used as a reference for the stress analysis. The ultimate strength of the T-joints was determined when the plastic hinge first occurred in the weld joint as the applied load progressively increased. Design curves for the double fillet-welded T-joints with various degrees of flange flexibility were obtained. These design curves were compared with the AWS D1.1 design code and the American Institute of Steel Construction Specifications (AISCS). As a result of this comparison, a design concept and procedure for the double fillet-welded T-joints was conceived.

To determine the weld performance with respect to the joint strength, the full-strength weld size was evaluated with different web lengths and flange flexibility in accordance with the AWS and AISC design procedures. It is suggested that the industry may have reached a point where improved weld design procedures may be incorporated for a more efficient joint design.

Introduction

Fillet-welded joints are commonly used in the fabrication of steel structures. Welds are deposited along a joint without any prior joint preparation, which is advantageous to the machined groove joints. However, the greater weld metal requirement for full-strength, fillet-welded joints may be a drawback in comparison with their groove joint counterparts. Cost savings in joint preparation are usually traded off by the excessive weld requirement, especially for thick-section joints. In addition, fillet-welded joints are more sensitive to out-of-plane distortion than groove joints due to greater weld eccentricity from the neutral axis of the base plates. Therefore, sizing fillet welds according to their minimum structural requirements is an important step in the design of fillet-welded structures.

It is a common practice in the welding community to use undermatched welding electrodes for fillet-welded joints to prevent weld metal cracking in highly restrained joints. Undermatched filler metal usually possesses better ductility to withstand the shrinkage stresses resulting from joint restraints. If this is the case, weld sizing is also required in design to assure the joint strength with the electrode selected.

In many structural designs, full-strength welds may not be required. Welds may be used to maintain the joint integrity and structural rigidity instead of transferring the major structural loads. This is particularly true in machine designs for high natural frequency and for vibration control of structures subject to dynamic loads. For those partial strength joints weld sizing becomes a necessity regardless of joint types, either fillet welds or groove welds. Because the weld requirement is often small in the case of design for rigidity, metallurgical reasons for preventing weld embrittlement due to insufficient heat input from small welds result in a minimum requirement for weld sizes depending upon the thickness of thicker members of the joint.

To size welds for structural purposes, the basic idea is to make welds either at least equal to the joint strength (*i.e.*, full-strength welds) or adequately carry the transferring loads while maintaining the structural rigidity (*i.e.*, partial-strength welds). The full-strength welds are usually governed by the respective design codes or specifications depending upon structural type and load transfer conditions. In many design codes or specifications, the full-strength requirements are usually given in reference to the thickness of thinner joint members. For partial-strength welds, weld size is primarily governed by the anticipated load transfer across the joint and the minimum size requirement due to metallurgical reasons.

This paper studied the weld sizing requirements for double fillet-welded T-joints using the finite element analysis (FEA) method. The theoretical analysis was then checked with the AWS D1.1, *Structural Welding Code — Steel* (Ref. 1), and the AISC steel structural design specifications (Ref. 2). A baseline of information was developed to understand the current weld sizing procedure for a given joint detail and its interpretation for efficient joint designs. The structural performance characteristics of the T-joints was assessed based on fitness-for-service design concepts.

Fillet Weld Design Procedure

In 1927, H. Dustin in Amsterdam first

KEY WORDS

Stress Analysis
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Computer Modeling
Finite Element Mesh
Weld Design
Static Load Conditions
Ductile Tearing
Design Load
Weld Size
Design Requirements

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presented a method for calculating fillet weld stresses by dividing the throat weld area (*i.e.*, effective weld area) of the fillet by the load transmitted by the fillet (Ref. 3). As defined in the AWS D1.1 *Structural Welding Code* (Ref. 1), effective weld area equals effective weld length times effective throat. The concept of using the effective weld area, ostensibly effective throat, for calculating fillet weld stresses, which was employed by many researchers during the 1930s, was consolidated and generalized by C. H. Jennings. His paper, published in 1936 (Ref. 4), presented most of the methods and equations used today for weld stresses. Jennings assumed a uniform stress distribution at the weld to simplify design calculations in the weld. The lack of stress uniformity in real structures was taken into account by the values of the recommended permissible (allowable) stresses.

The Jennings design method is accepted by the majority of today's technical societies in the United States, although its basic assumption is often not adequate for the analysis of complex joints and/or joint loading. The design equations for double fillet-welded T-joints presented in this paper use the AWS D1.1-86 design code allowable stress (Ref. 1) for statically loaded structures, which is 30% of the ultimate tensile strength of the weld metal.

In his master's degree thesis (Ref. 5), Fairchild commented on the current status of fillet weld design, "Either Mr. H. Dustin, in 1927, was ahead of his time and extremely accurate with the first estimate of fillet weld stresses or our welding codes are relying on an unrevised, 55-(*i.e., today, 70*)-year-old method of design." From this viewpoint, welding design has fallen victim to stereotyping and, when compared to other disciplines of engineering design, it can only be considered as a collection of rules of thumb. Therefore, a more rational, mathematically proven, procedure for determining the behavior of welds in different joints under various loading conditions should be formulated.

Scope and Objectives

The primary responsibility of a designer is to ensure that his designs can function as desired for the prescribed service life. A successful design that averts premature failure is achieved by recognizing and evaluating all potential modes of failure that might occur in the service environment. The principal mechanical failure modes that have commonly been

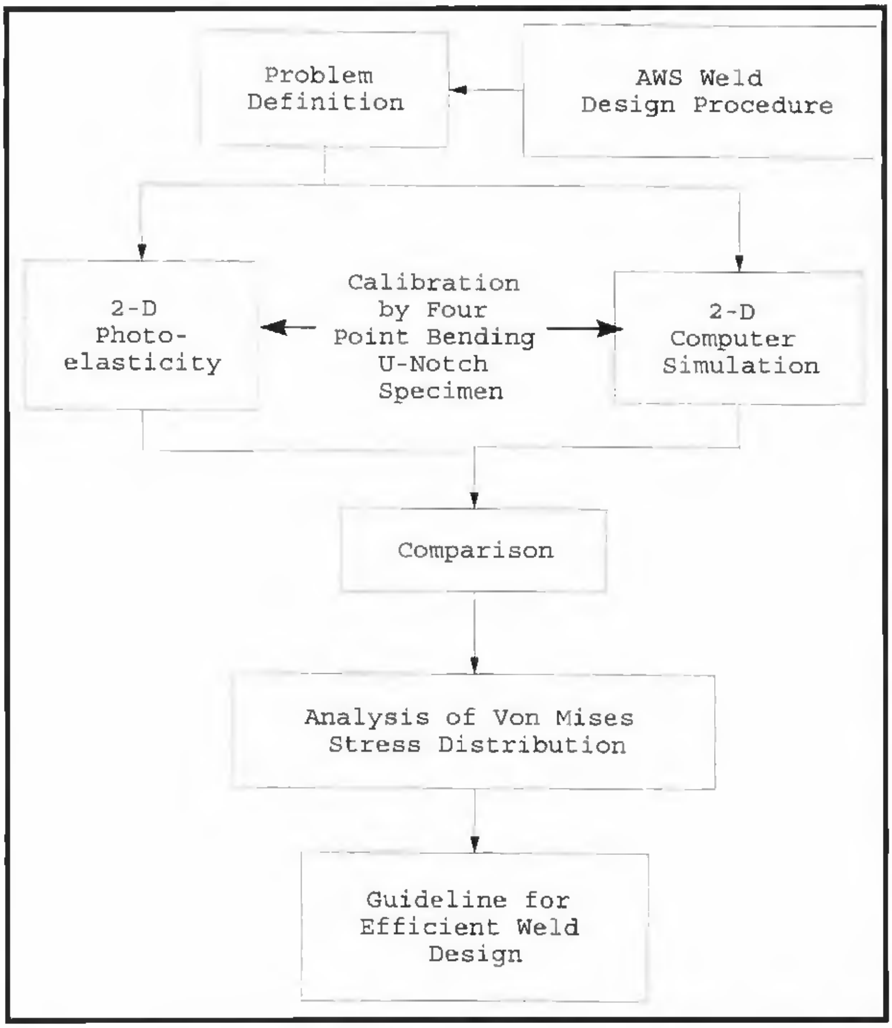

Fig. 1 — Flow chart of the overall approach

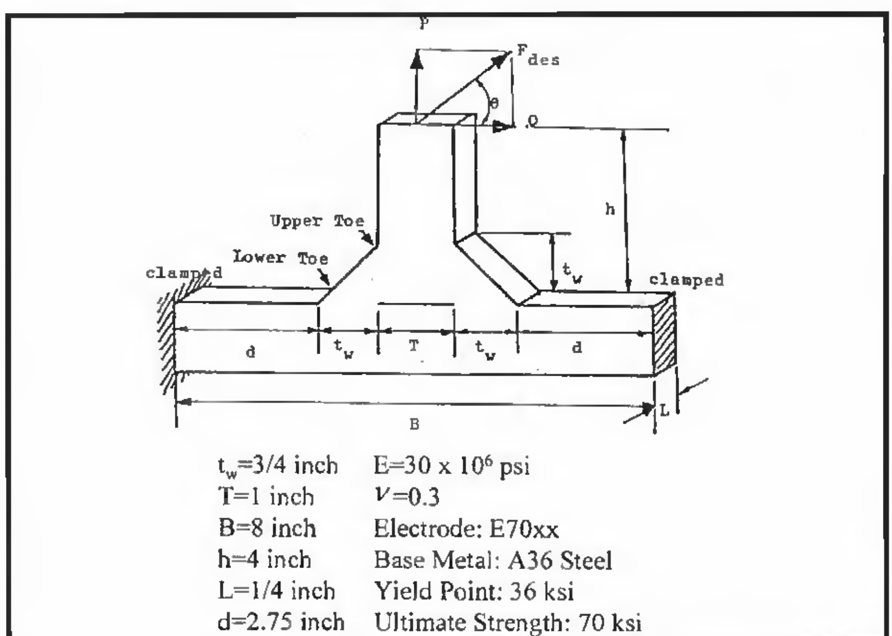

Fig. 2 — Specimen under study.

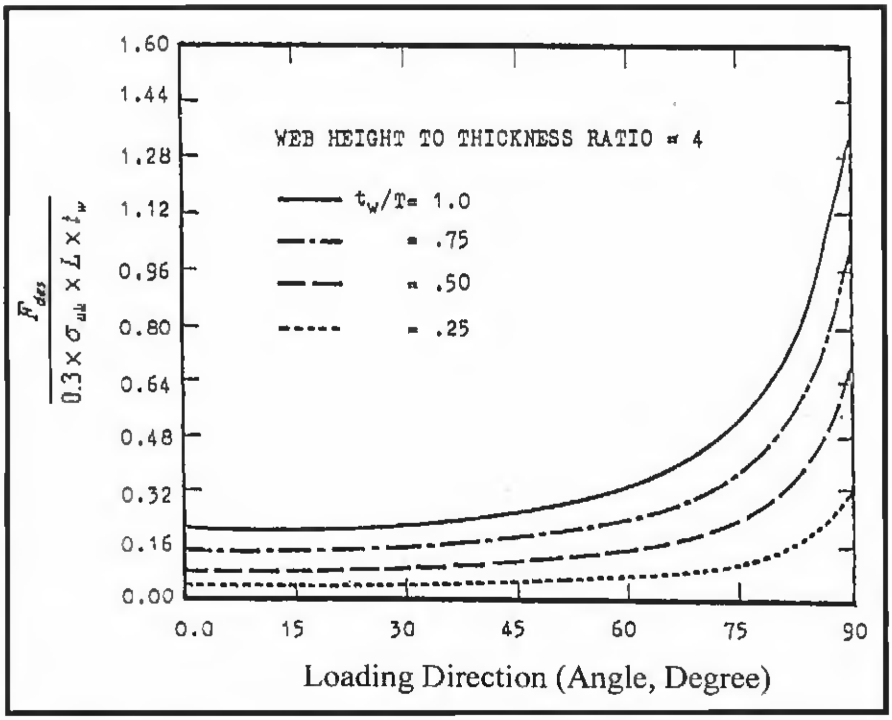

Fig. 3 — Typical welding design curves for T-joints.

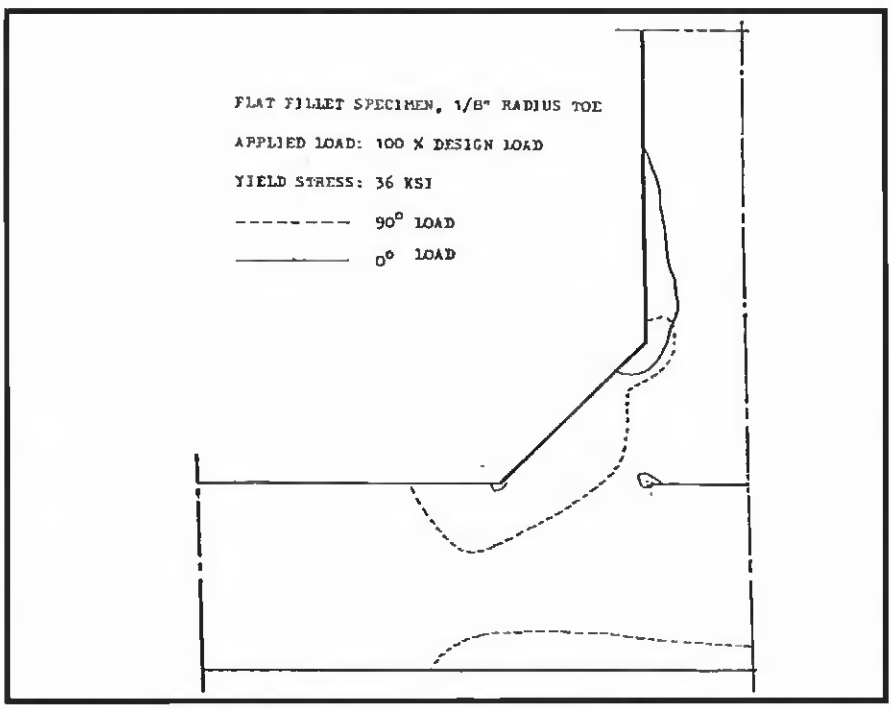

Fig. 4 — The calculated yielded area in a double fillet-welded T-joint at 0-deg and 90-deg load angles.

acknowledged in welded structures are 1) ductile tearing due to overload or high stress concentration, 2) brittle fracture and 3) fatigue failure. The scope of this paper is to study the strength of double fillet-welded T-joints under static loading conditions. Ductile tearing is the only failure mode considered, and the Von Mises equivalent stress is used for the failure criterion. The objectives of this study are twofold:

1) To provide a rational method for analyzing the stress distribution of double fillet-welded T-joints.

2) To provide some guidance for efficient design of fillet welds.

Method of Analysis

Figure 1 shows the logic flow of the analysis used in this study. A thin section of a double fillet-welded T-joint specimen was removed from a long joint sample and the analysis was conducted on this specimen. The analysis was completely defined in terms of the joint dimensions and the supporting and loading conditions. The design load was determined according to the AWS D1.1 code procedure (Ref. 1) and the AISC specifications (Ref. 2), which were used as references for the stress analysis. The finite element method was used for the analysis and checked with photoelastic stress measurements. Both were calibrated on a four-point bending, U-notch specimen.

Problem Definition

The weldment consists of two 1-in.-thick plates in a T-joint with the applied load transmitted from the vertical (web) plate to the horizontal (flange) plate through the double fillet welds. The load at an angle to the horizontal plate is uniformly distributed along the top edge of the vertical plate. The horizontal plate is clamped along both ends parallel to the weld. Any displacements in the direction transverse to the weld and angular rotations at the supports are prevented. The specimen is free from stress in the direction parallel to the weld, thus the joint can be assumed to be in a plane stress condition. This is not a realistic situation for practical T-joints, however, the simplified 2-D model and analysis would result in a more conservative design. Figure 2 is a schematic presentation of the T-joint specimen upon which the finite element stress analysis was conducted.

In this study, the full-strength weld size for the T-joints was assumed to be 3/4 in. for 1-in.-thick base plates. This is an old design concept that indicates a full-strength, double fillet-welded T-joint having a leg size three-fourths of the base plate thickness (Ref. 6). It is worth noting that the three-fourths factor was based upon the old design permissible values of 20 and 15.2 ksi for A7 steel and old E70 welds, respectively. Both permissible values have been increased for higher strength steel and weld metal, such as A36 steel (*e.g.*, 22 ksi) and newer E70 welds (*e.g.*, 21 ksi). The required full-strength double-fillet weld size would be 9/16 of the thinner thickness for this situation. However, in this paper, the 9/16-

in. weld size was used for the 1-in. joint thickness to demonstrate the analysis and design procedure of double fillet-welded T-joints using the FEM method.

Design Load and Weld Size

Various sections of the AWS D1.1 *Structural Welding Code* (Ref. 1) govern the design procedure for fillet welds in a statically loaded structure. The calculation of the design load is based on the assumption that the effective throat of the weld is the critical section. The criterion for weld size determination is

1) To let the combined stress in the effective weld area be less than the allowable (permissible) weld stress, which equals 0.3 times the ultimate tensile strength of weld metal.

2) The shear stress on base metal adjacent to the weld metal shall not exceed 0.4 times the yield strength of the base metal.

Figure 2 shows a double fillet-welded T-joint loaded by a force, F, at some arbitrary angle measured from the horizontal direction. According to the AWS D1.1 design procedure, the design formula for this welded joint based on weld strength alone was given as follows (Ref. 7):

$$\frac{F_{des}}{0.3 \bullet \sigma_{ult} \bullet L \bullet T} = \left(\frac{t_w}{T}\right) \bullet \left[\frac{1}{2} + x\sin 2\theta + 2x^2\cos^2\theta\right]^{-\frac{1}{2}} \quad (1)$$

$$x = \frac{\left(\frac{h}{T}\right) \bullet \left(1 + \frac{t_w}{T}\right)}{\frac{5}{12} \bullet \left(\frac{t_w}{T}\right)^2 + \left(\frac{t_w}{T}\right) + 1} \quad (2)$$

where F_{des} is design load, σ_{ult} is the ultimate weld metal strength, L is specimen thickness (*i.e.*, joint length), T is plate thickness, h is distance between top surface of the flange plate to the load point, t_w is fillet weld size, χ is weld size parameter, and θ is the loading angle from the horizontal direction.

Since AWS D1.1 does not allow the shear stress in the base metal to exceed 40% of its yield stress, the design load should also be governed by the following equation:

$$\frac{F_{des}}{0.3 \bullet \sigma_{ult} \bullet L \bullet T} < \frac{8}{3} \bullet \frac{F_y}{\sigma_{ult}} \bullet \frac{t_w}{T} \bullet \frac{1}{\beta} \quad (3)$$

where F_y is the yield strength of the base metal and β is Sin θ or Cos θ, whichever is larger.

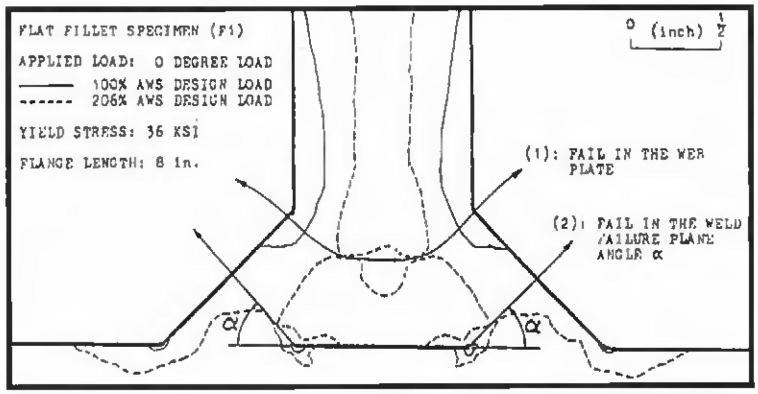

Fig. 5 — The yielded zone under AWS D1.1 design load and predicted failure plane.

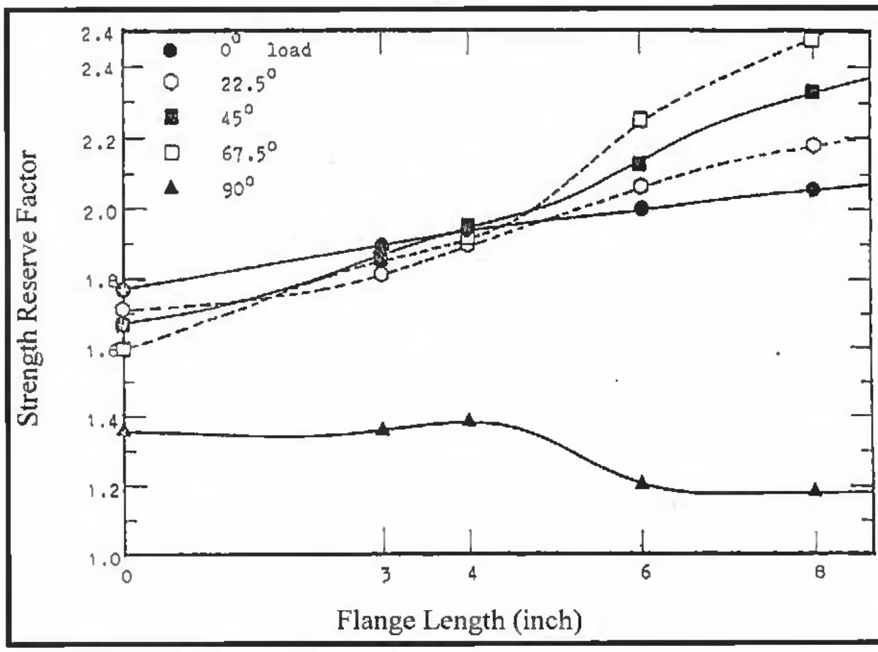

Fig. 6 — Strength reserve factor vs. flange flexibility.

For a given value of χ (weld size parameter), the minimum design load occurs when the load is applied in a direction of

$$\theta_{cr} = \frac{1}{2} \bullet \tan^{-1}\left(\frac{1}{X}\right) \quad (4)$$

The angle of minimum design load decreases as h/T increases or t_w/T decreases. The maximum design load always occurs when the load applied is normal to weld (θ = 90 deg). For a joint with given dimension and weld size, the weld stress is the highest when the load is applied in a direction corresponding to the minimum design load.

The left-hand term of Equations 1 and 3 is a nondimensional parameter that can be graphed as a function of the loading direction if all other variables are held constant. The typical design curves concerning this study are shown in Fig. 3. The curves are plotted with the h/T ratio equal to 4 and the t_w/T values from 1/8 to 1.

The design load is also governed by the size of the joint members. The governing criteria, according to the AISC specifications (Ref. 2) are as follows:

$$\left(\frac{f_t}{F_t}\right)^2 + \left(\frac{f_v}{F_v}\right)^2 < 1 \quad (5)$$

$$\tau_{max} < 0.4 \bullet F_y \quad (6)$$

where f_t = calculated maximum normal stress, f_v = calculated average shear stress, τ_{max} = calculated maximum shear stress, F_t = allowable normal stress = 0.6 F_y, F_v = allowable shear stress = 0.4 F_y, and F_y= yield stress of plate material.

By expanding and substituting the corresponding terms into Equations 5 and 6, the first criterion (Equation 5) due to flange requirements is

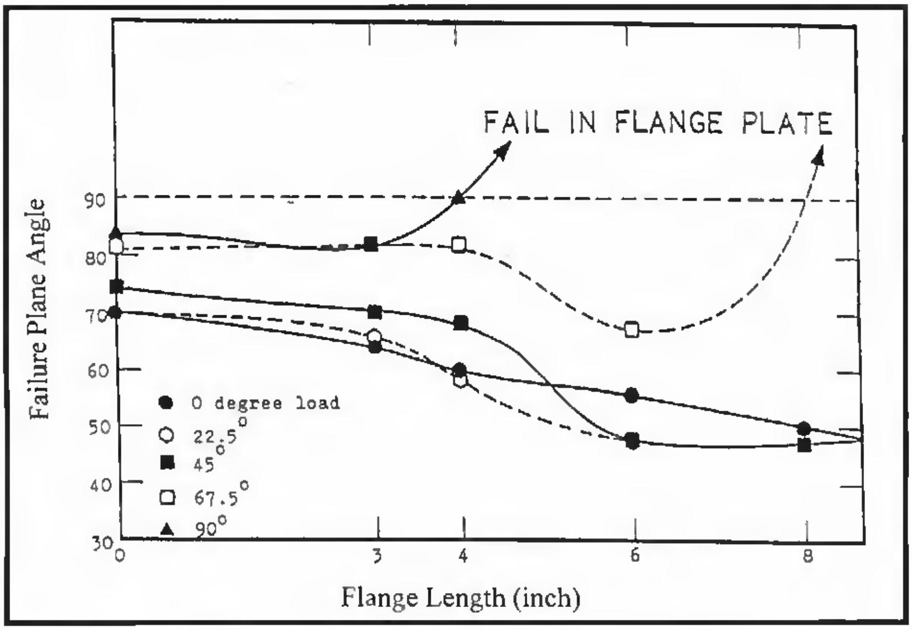

Fig. 7 — Predicted failure plane vs. flange flexibility.

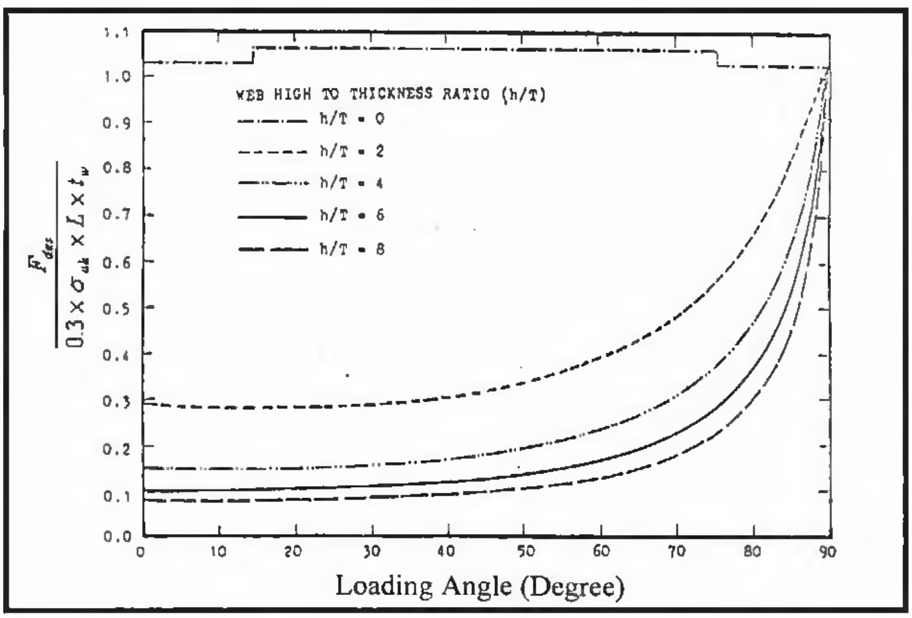

Fig. 8 — Design curves according to AWS D1.1 Code, weld size (tW/T) = 3/4.

$$\frac{F_{des}}{L \bullet T} < \frac{F_y}{\sqrt{\left[\frac{\left(\frac{3h}{T}+1\right)\bullet \text{Cos}\theta + \frac{3}{4}\bullet\frac{8}{T}\bullet \text{Sin}\theta}{0.6}\right]^2 + \left[\frac{\text{Sin}\theta}{0.8}\right]^2}} \quad (7)$$

(for nomenclature, see Fig. 2). The second criterion (Equation 5) is

$$\frac{F_{des}}{L \bullet T} < 0.53 \bullet \frac{F_y}{\sin\theta} \quad (8)$$

The same criteria (Equations 5 and 6) are required for web design. For the web requirements, the first criterion is

$$\frac{F_{des}}{L \bullet T} < \frac{F_y}{\sqrt{\left[\frac{\frac{6h}{T}\bullet \cos\theta + \sin\theta}{0.6}\right]^2 + \left[\frac{\cos\theta}{0.4}\right]^2}} \quad (9)$$

The second criterion is

$$\frac{F_{des}}{L \bullet T} < 0.27 \bullet \frac{F_y}{\cos\theta} \quad (10)$$

$F_{des}/L \bullet T$ should not be greater than any of the four equations (Equations 7–10).

Finite Element Meshes

To accurately determine the stress field in welded joints, finite element analysis procedures were carried out to create optimum mesh patterns (Ref. 8). Photoelastic measurements were conducted to calibrate the mesh refinement. The calculated fringe lines were compared with the photograph taken from the photoelastic analysis (Ref. 7). Some discrepancies were initially found between the calculated and experimental results. The mesh refinement was then continued until good agreement between these two results was obtained. The calculated general stress field of the T-joint with the refined meshes was in good agreement with the photoelastic results

Strength Reserve Factor

Local yielding usually occurs at fillet toes when a fillet-welded T-joint is loaded to its design load. This phenomenon is due to high stress concentrations at fillet toes (Ref. 9). Local yielding often may not be detrimental to the overall integrity of the joint. However, it is useful to study the size of yielded zones in the specimens under their design load. The unyielded area between two yielded zones represents the strength reserve of a fillet joint under its design load. Furthermore, the progressive expansion of the yielded zone in a joint as the load increases characterizes the ductile mode of joint failure. The Von Mises equivalent stress distributions under various load levels predict such characterizations.

If we keep increasing the applied load until the two yielded zones merge together to form a "plastic hinge," failure may be predicted to occur at this section under a certain load (*i.e.*, failure load). Therefore, a strength reserve factor based on the Von Mises failure criterion can be calculated simply by dividing the "failure load" by its permissible design load. The strength reserve factor is an indication of the conservative nature of the permissible load.

Prediction of Failure Plane

The anticipated failure plane can also be determined by connecting a certain high stress concentration point (*i.e.*, usually on a free surface) to another free-surface point in the "plastically hinged" section, either in the weld or in the base plate. In a welded T-joint, the locations that will cause a high stress concentration are the toes and roots of fillet welds. Except for the failure that may occur in the base metal, the angle of the failure plane is measured from the angle between the horizontal line and the line connecting the fillet root to the shortest distanced weld surface in the "plastically hinged" section.

It is clear that the failure plane may change with the change of loading directions as well as the flexibility of the flange plate. The potential failure plane in a welded joint will provide important information for evaluating the significance of a noncompliance in the weld. The acceptable tolerance of the weld noncompliance will be more stringent in areas near the critical failure plane. Any weld noncompliance is relatively insignificant when it is located far from the critical plane.

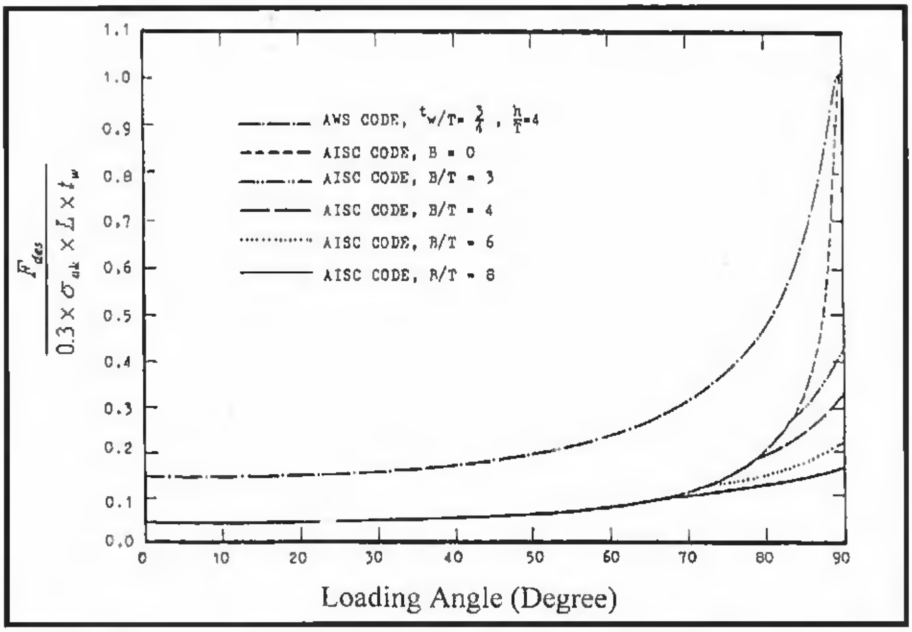

Fig. 9 — Comparison of the design load between AWS D1.1 Code and AISC specification.

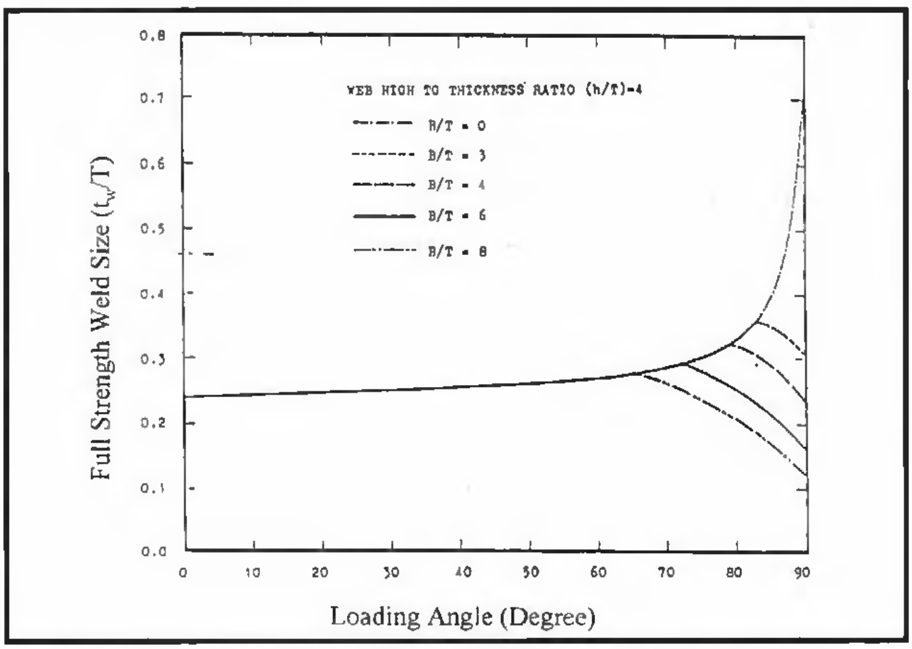

Fig. 10 — Full-strength weld size of double fillet T-joints with web height to the thickness ratio (h/T) = 4.

Results and Discussion

Limitations of this Study

In summary, several constraints on the application of the information presented in this paper should be reviewed.

1) The results from this study should refer to the fillet model described in the Definition section. The minimum thickness of base plates is 1 in. The fillet size is set to be three-fourths of the member thickness. The applied load is static and always lies in the plane of the cross section (*i.e.*, the longitudinal shear force is not considered).

2) The calculated plastic zone size is conservative because the lateral constraints due to the joint length are not considered in the analysis. Stress redistribution due to plasticity is not incorporated in the analysis. This may contribute additional conservatism to the analysis.

3) The joint material is assumed to be ductile in the service environmental conditions and brittle fracture instability is not a concern.

4) The analysis of local yielding by the finite element method in this study was limited to the linear elastic range, and the postyield and strain hardening behaviors are not considered.

Yielded Area of Fillet Joint under Design Load

Figure 4 shows the yielded area in a flat fillet specimen (1/8-in. toe radius) under 100% design load at 0- and 90-deg loading angles. The majority of the yielded area appears in the vertical web plate around the upper toes when the load is at 0-deg load angle. When the load is applied by vertical pulling of the web plate (90-deg load angle), a rather large yielded area appears in the joint under 100% design load. The standard design procedure, as discussed in the Design Load and Weld Size section of this paper, considers only the normal stress in welds and shear stress in base metal without any justification on bending of the flange plate. A much higher load is permitted at the 90-deg load angle — Fig. 3. At 100% design load, the plastic zones surround the lower toes and cover a large portion of the fillet area and the flange plate.

Since any practical joint could be in a triaxial state of stress, the actual plastic zone could be smaller than that calculated in this two-dimensional analysis. As defined previously, this paper studies T-joints with 3/4-in. double fillet welds.

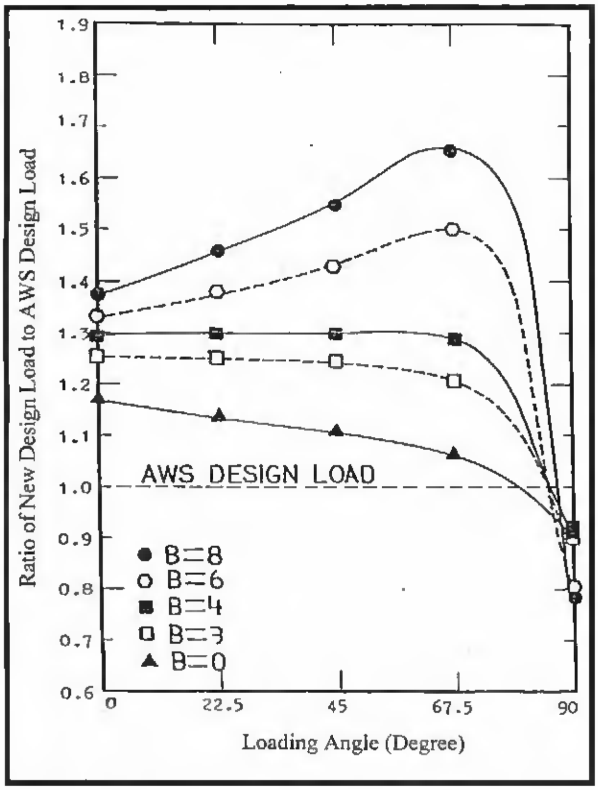

Fig. 11 — New design curves for double fillet-welded T-joints (based on the AWS D1.1 Code design load and a load factor of 1.5).

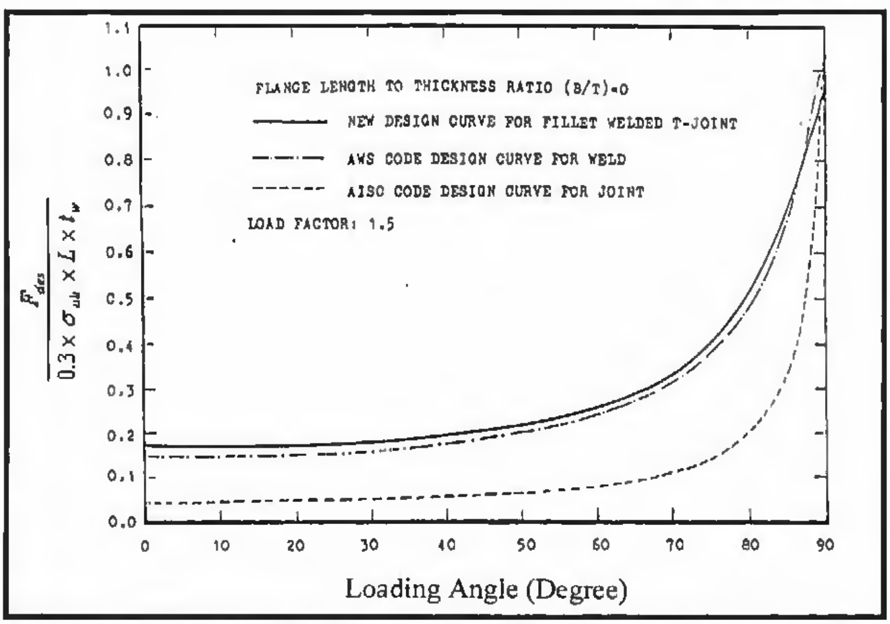

Fig. 12 — Comparison of a new design curves with AWS D1.1 and AISC Codes' design curves, rigid flange.

Strength Reserve Factor and Critical Failure Plane

Figure 5 shows the Von Mises equivalent stresses for a flat, double fillet-welded T-joint loaded at 0 deg. The solid lines of this figure are the boundaries of yielded zones under the 100% design load, whereas the dashed lines show the plastic hinge formed in the specimen when the critical load is applied. The strength reserve factor is obtained by taking the ratio of the critical load to the permissible design load. Figure 6 shows the strength reserve factors of a flat fillet specimen under various load angles with respect to various flange lengths. Specimens always have the smallest factors when loaded at 90 deg. This implies that the allowable design stress for welds according to the AWS D1.1 Code are significantly less conservative when the load is applied at high angles (around 90 deg), and precaution should be taken when fillet-welded joints are to be vertically loaded.

It has been well known for many years that fillet welds pulled transversely (90-deg load angle) are at least 4/3 as strong as A36 base steel. However, the results from this study show the significant effect of joint flexibility on reduction in joint strength at this high load angle. To assess the fitness-for-service condition of welded joints, the interaction between weld strength and base plate strength, as well as the load distribution in the joint, must be considered. A flexible support will introduce bending stresses at the lower toe of the fillet weld. This is the reason for reduction in joint strength at high load angles.

Except for the vertical loading direction, the strength reserve factor decreases as the flange length decreases. This trend is more evident at higher load angles (near 67.5 deg) and is less pronounced at lower angles (horizontal shearing). The reasons are as follows:

1) The welds are restricted from any rotation if the flange plate is very rigid. The stress in welds becomes very large. Therefore, the calculated design load is low and the strength reserve factor is small.

2) When the flange plate is flexible, part of the load is distributed to the flange. This alleviates the stress in the welds so that a higher strength reserve factor can be obtained.

3) Vertical loading may create an additional bending moment in a flexible flange plate. Therefore, the strength reserve factor is further reduced when the fillet joint is loaded at the 90-deg angle.

Figure 5 also shows the predicted failure plane of the weld joint. Although the so-called "full strength weld" is assumed in this study, some specimens were predicted to fail in the weld. As shown in Fig. 7, no failure planes below 45 deg were observed. Only a few specimens, having longer flange plates, failed in the flange plate when the load was applied at higher angles (near 90 deg), and it is possible for these specimens to fail in the web plate when they are horizontally loaded at their design load — Fig. 5. The angle of the predicted failure plane increases with an increase of flange rigidity (*i.e.*, flange length), and it reaches its highest angle (near 90 deg) when pulling an infinite rigid flange plate (B = 0) specimen. The reason for the occurrence of the near 90-deg failure plane is due to the shear stress between the weld metal and the web plate under vertical pulling. When this specimen is horizontally loaded, it will fail at approximately a 70-deg angle. For this particular case without any flange bending, the failure plane has been proven to be at a 67.5-deg angle in the fillet (Ref. 10). The FEM result agrees well with that mathematically determined.

Design Precautions for Fillet Welds

Since geometrical discontinuities such as the toes of fillet welds are unavoidable, the existence of local yielded areas in the structure should be evaluated. Therefore, plastic behavior has always been assumed in steel structures, even for those developed by the elastic design. As a matter of fact, our modern steel structures would be impossible if steel were not ductile.

However, plastic strains in a fillet joint can cause significant and permanent deformation of a welded structure. As shown in the Von Mises equivalent stress analysis in this section, large yielded area appears in the joint under 100% design load, especially at the 90-deg load angle. Even then there still exists some strength reserve because of the elastic constraints in the remaining elastic areas. Also, dimensional tolerance of a fillet welded joint could be a problem and must be studied. Precautions regarding this aspect will be prevalent if the dimensional accuracy is a major concern. Loading direction is primarily responsible for such concerns.

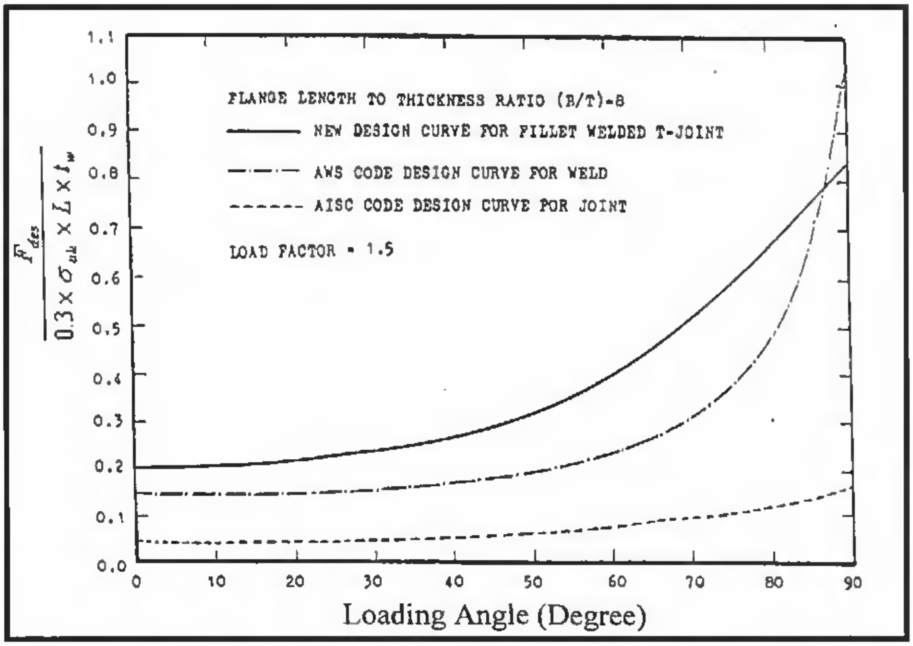

Fig. 13 — Comparison of a new design curve with AWS D1.1 and AISC Codes' design curves, flange length to the thickness ratio = 8.

Discussion of Design Requirements for a Fillet-Welded T-Joint

Allowable Design Load for a Fillet-WeldedT-Joint

In the general design procedures for welded joints, the AWS D1.1 Code only takes the allowable weld stress into account without considering the joint dimensions and its support conditions. Figure 3 shows the typical design curves for given joint dimensions with clamped supports. The design curves are also presented in such a way that the effect of the web height to thickness ratio (h/T) is shown — Fig. 8. Except for the joints with extremely small web length (zero), the shearing stress limitation in Equation 3 governs only when the applied load is in vertical tension mode. For other loading directions, having a horizontal shear force component, any increase of web length produces higher bending stress at the weld and, hence, decreases the design load rapidly.

The trend of design load varying with the web length is similar to that of AWS D1.1 design curves (without considering joint effect) since both design loads are all governed by the web length. However, the trends of the two design curves varying with the flange length are different since the AWS D1.1 Code does not take the flange flexibility into account. When the height of the web plate is held constant, the AISC design load becomes lower when the double fillet-welded T-joint is loaded at higher angles. This implies that the flange flexibility causes an extra bending moment in the joint and the flange criteria (Equations 7, 8) become important.

Full Strength Weld Size

As mentioned in the Problem Definition section, any double fillet welds with the size of 3/4 of the web thickness is considered to be at least 4/3 as strong as the base plate. To study this 3/4t concept, AISC base plate requirements are used as the reference for the determination of the full-strength weld size in this section. The full-strength weld is defined in such a way that the AWS D1.1 design load has the same value as that of the AISC specification. From Fig. 9, there is a large difference in design load between the AWS and AISC permissible values. By comparison of the AISC design curves with those in Fig. 3, smaller weld sizes than the 3/4t would be considered as full strength. In other words, much smaller weld sizes can meet the AISC requirements. To find the full-strength weld sizes mathematically, Equation 1 is rewritten as follows:

$$\frac{F_{des}}{L \bullet T} < \frac{0.3 \bullet \sigma_{ult} \bullet t_w}{T} \bullet \frac{1}{f(t_w, \theta, h, T)} \quad (11)$$

Let the smallest value of Equations 7–10 be

$$\frac{F_{des}}{L \bullet T} = f(\theta, h, B, T, F_y) \quad (12)$$

The full-strength weld size is obtained from Equations 11 and 12. The implicit equation can be expressed as follows:

$$\frac{t_w}{T} = f\left(\theta, \frac{h}{t}, \frac{B}{T}, F_y\right) \bullet f\left(\frac{t_w}{T}, \theta, \frac{h}{T}, \sigma_{ult}\right) \quad (13)$$

where the nomenclature is given in Fig. 2.

When h, B, T, F and θ are given, the weld size t_w can be obtained by solving the nonlinear Equation 13 at various load angles by iteration.

Figure 10 shows the full-strength weld size at various loading angles for the joints with different flange flexibility and a given web height to thickness ratio (h/T = 4). Failure is predicted to occur in the weld of any joint with its weld size in the area below the curves in Fig. 10. Any weld size greater than the curve values implies that failure may initiate in the base plate. Therefore, the curves indicate the weld size for full strength.

The full-strength weld size is also affected by the flange flexibility as well as the web height. For a given web height to thickness ratio (e.g., h/T = 4), the full-strength weld size remains unchanged regardless of the flange length at low load angles. The weld size for vertical tension load is the largest when the flange plate is rigid. It drops rapidly as the length of the flange plate becomes larger. The highest value of the full-strength weld size to web thickness among all cases studied is 0.727 (approximately 3/4 in.). This value is the same as that defined as the old rule of thumb as described in Ref. 6. Therefore, the weld size of three-fourths of the base plate thickness for a full-strength weld is valid only for double fillet-welded T-joints having a rigid flange plate and subjected to a vertical tension load.

The full-strength weld size is much smaller than the rule of thumb when the loading angles are low or the flange plate is long. It is very interesting to note that, as shown in Fig. 10, a 1/4-in. weld size may be considered as a full-strength weld for most of the double fillet-welded T-joints under a horizontal load (zero loading angle). The reason for such an incredibly small weld size is due to the very low permissible load by the AISC specification at the zero degree load angle. However, such a small weld size is sometimes impractical, because of the minimum size requirement by the codes to prevent cracking.

New Design Curves for Fillet Welded T-Joint

Since the AISC design load of a fillet-welded joint is much lower than the AWS D1.1 permissible values, and the weld size cannot be smaller than AWS D1.1 minimum requirements, the use of larger weld sizes than that required is inevitable if the AISC joint design procedure is to be followed. In this situation, the following question is being asked by many industries: "Are these codes too conservative?" To answer this question, let us examine the strength reserve factors that were presented in the previous section. If we divide the strength reserve factors by a load factor, a new family of design curves relative to the AWS D1.1 design load can be developed. Figure 11 shows the new design curves using a load factor of 1.5. These curves indicate that the AWS D1.1 code is conservative except at load angles near 90 deg. The problem now existing is "to what extent are the AISC requirements on the conservative or the liberal side?"

Figure 12 shows the comparison of a new design curve with those of the AWS D1.1 and the AISC codes. When the flange plate is very rigid (B = 0), the AWS D1.1 design curve is close to the calculated new design curve. Since the AWS D 1.1 code does not consider the effect of flange bending, the AWS D1.1 design curves therefore match only the calculations for the case of the rigid flange. However, some care must be taken if this joint is vertically loaded (90-deg load angle). Figure 12 also shows that the AISC design load is actually too conservative. The reason is that it permits only a 40% yield stress (14.4 ksi) as the allowable shear in the joints, which is smaller than the AWS D1.1 weld stress allowable (0.3 of ultimate strength of weld metal, 21 ksi for E70 electrodes).

As the flange length becomes relatively large (e.g., B/T = 8), Figure 13 shows that the AWS D1.1 code is very conservative at low load angles and it becomes liberal at the 90-deg load angle. The fillet joint is relatively stronger at low load angles (below 70 deg) and relatively weak at 90-deg load angles when the flange is more flexible. Figure 11 shows the same conclusions. The AISC code design curves again show high conservatism.

Although the AWS D1.1 code stands on a somewhat conservative side (except for 90-deg load), it is appropriate to be used for designing welded fillet joints if the flange plate is rigid. The AISC joint design code is very conservative in most of the cases. The new design curves developed in this paper are useful when the flange plate is relatively flexible. As mentioned previously, many assumptions on the applicability of these new design curves should be noticed. The design curves are constructed on the basis of the Von Mises yield criterion and a 1.5 load factor. In addition, the finite element analysis is limited in the linear elastic range, the actual failure load of double fillet-welded T-joints may deviate from the prediction due to stress redistribution after yielding and the strain hardening effect. Therefore, fracture tests and/or elastic-plastic finite element analysis on double fillet-welded T-joints would be necessary for an improved answer.

Concluding Remarks

The design load for a welded joint should be determined upon the interaction of joint condition and weld strength. The AWS design code provides a procedure for sizing the welds without considering the joint conditions. The AISC specifications define the structural requirements of the joint members, as well as the weld requirements. All provisions of the AWS D1.1 Code, with a few exceptions, apply to work performed under the AISC specifications. However, the interactions among weld, joint and load are not addressed in the conventional design provisions by either AWS D1.1 or AISC specifications. The new design curves resulting from this study suggest that all three factors could have significant influence on the weld requirement. It is important to analyze the joint with an appropriate engineering method, such as the finite element method, to size welds for the joint conditions and the loads imposed. Of course, the metallurgical factors must also be considered in determining the minimum weld size requirement.

Although the design conservatism was addressed by a joint AWS-AISC committee in 1968 and resulted in the 1969 code with a one-third increase in all weld allowables, the need for another look at the code for effect of joint flexibility and load condition can be conceived.

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