INTRODUCTION

A weldment is an assembly of component parts joined by welding. It may be a bridge, a building frame, an automobile, a truck body, a trailer hitch, a piece of machinery, or an offshore oil drilling structure. In the field of weldment design, the primary objectives are to produce an assembly that (1) performs its intended functions, (2) has the required reliability and safety, and (3) can be fabricated, inspected, transported, and placed in service at a minimum total cost. The total cost includes the cost of design, materials, fabrication, erection, inspection, operation, repair, and product maintenance.

The designers of weldments must have an understanding of basic design principles and concepts. They must have some knowledge of and experience in cutting and shaping metals; assembling components; preparing and fabricating welded joints; evaluating welds in compliance with established acceptance criteria; and performing nondestructive examination and mechanical testing. Designers routinely apply knowledge of the following areas when evaluating the effects these may have on the design of weldments:

1. Mechanical and physical properties of metals and weldments;
2. Weldability of metals;
3. Welding processes, costs, and variations in welding procedures;
4. Filler metals and properties of weld metals;
5. Thermal effects of welding;
6. Effects of restraint and stress concentrations;
7. Control of distortion;
8. Efficient use of steel, aluminum, and other metals in weldments;
9. Design for appropriate stiffness or flexibility in welded beams and other structural members;
10. Design for torsional resistance;
11. Effects of thermal strains induced by welding in the presence of restraints;
12. Effects of stress induced by welding in combination with design stresses;
13. Practical considerations of welding and the selection of proper joint designs for the application;
14. Communication of weldment design to the shop, including the use of welding symbols; and
15. Applicable welding codes and safety standards.

As several of these topics involve highly specialized areas of science and technology, designers should refrain from relying entirely upon their own knowledge and experience, which may be may generalized. They are encouraged to consult with welding experts whenever appropriate.

PROPERTIES OF METALS

The properties of metals can be divided into five general groups: (1) mechanical, (2) physical, (3) corrosion, (4) optical, and (5) nuclear. The typical characteristics of each group are presented in Table 1. These are further categorized as structure-insensitive or structure-sensitive, as this distinction is made in most textbooks.
The structure-insensitive properties of metals do not vary from one piece of a metal to another of the same composition, regardless of differences in microstructure. This has been verified by data obtained from standard engineering tests and is true for most engineering purposes. These properties can often be calculated or rationalized by examining the chemical composition. Structure-insensitive properties are commonly considered constants for metals.

The structure-sensitive properties are dependent not only upon the chemical composition and crystallographic structure but also upon microstructural details that may be affected in subtle ways by the manufacturing and processing history of the metal. Even the size of the sample can influence the test results obtained for a structure-sensitive property. Structure-sensitive properties are likely to vary somewhat if differences exist in the treatment and preparation of the samples.

In the field of weldment design, the most important mechanical properties of metals, with the exception of the moduli of elasticity (see the section “Modulus of Elasticity” below), are those that are structure-sensitive. Consequently, the published single values of these properties should be considered with reservation. It is common for the mechanical properties of metal plates or bars that are unusual in size or treatment condition to deviate significantly from the values published for the particular metals. In addition, as determined by standard quality acceptance tests in an American Society for Testing and Materials (ASTM) specification, the mechanical properties of a metal do not guarantee identical properties throughout the material represented by the test sample. For example, the direction in which wrought metal is tested (longitudinal, transverse, or through-thickness) may result in significantly different values for strength and ductility.

The physical and corrosion properties of metals are considered structure-insensitive for the most part. Some of the values established for these properties apply only to common polycrystalline metals, however.


### Table 1

<table>
<thead>
<tr>
<th>General Groups of Properties</th>
<th>Structure-Insensitive Properties</th>
<th>Structure-Sensitive Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mechanical</td>
<td>Elastic moduli</td>
<td>Ultimate strength</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Yield strength</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fatigue strength</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hardness</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ductility</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Elastic limit</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Damping capacity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Creep strength</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rupture strength</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Toughness</td>
</tr>
<tr>
<td>Physical</td>
<td>Thermal expansion</td>
<td>Thermal stresses</td>
</tr>
<tr>
<td></td>
<td>Thermal conductivity</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Melting point</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Specific heat</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Emissivity</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Thermal evaporation rate</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Density</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Vapor pressure</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Electrical conductivity</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Thermoelectric properties</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Magnetic properties</td>
<td>Ferromagnetic properties</td>
</tr>
<tr>
<td></td>
<td>Thermionic emission</td>
<td></td>
</tr>
<tr>
<td>Corrosion</td>
<td>Electrochemical potential</td>
<td>Oxidation resistance</td>
</tr>
<tr>
<td>Optical</td>
<td>Color</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Reflectivity</td>
<td></td>
</tr>
<tr>
<td>Nuclear</td>
<td>Radiation absorptivity</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Nuclear cross section</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Wavelength of characteristic X-rays</td>
<td></td>
</tr>
</tbody>
</table>

MECHANICAL PROPERTIES

As metals are generally strong, tough, and ductile, they are advantageous construction materials. This combination of properties is rarely found in nonmetallic materials, so most nonmetallic construction materials depend upon composite action with metals for their usefulness. The strength, toughness, and ductility of metals can be modified by means of alloy or heat treatment. Metals offer not only many useful individual mechanical properties and characteristics but also a large number of combinations of these properties. This versatility allows designers to select the best combination of properties to ensure the intended performance level.

Among the factors that affect the mechanical properties of metals are applied heat, the cooling rate, the addition of filler metal, and the metallurgical structure of the joint. The joining of metals by welding or brazing also affects their mechanical properties.

Another factor that must be considered during the materials selection process is ease of fabrication. Base metals and welding consumables should be selected to facilitate fabrication. When making a decision regarding the selection of materials, it is necessary to examine the governing properties of the metals and consider their combined effect upon the design and service behavior of the weldment.

Modulus of Elasticity

A convenient way to assess a metal’s ability to resist stretching (strain) under stress in the elastic range is Young’s Modulus, also known as the modulus of elasticity. This is the ratio between the applied strain and the resulting stress. In the elastic range, the modulus of elasticity, a constant, is expressed by the following equation:

\[ E = \frac{\sigma}{\varepsilon} \]  

where

- \( E \) = Modulus of elasticity;
- \( \sigma \) = Stress, pounds per square inch, psi (megapascal [MPa]), and
- \( \varepsilon \) = Strain, inch per inch (in./in.) (millimeter per millimeter [mm/mm]).

Young’s Modulus, shown in Figure 1, can be calculated from the measured strain and calculated stress generated during a standard tension test. As the modulus of elasticity is a structure-sensitive property, it is not influenced by grain size, cleanliness, or heat treatment; in fact, the modulus of elasticity often remains unchanged even after considerable alloy additions have been made.11

Table 2 presents the modulus of elasticity for a number of metals.

For practical purposes, the modulus of elasticity can be used to determine the level of stress created in a piece of metal when it is forced to stretch elastically a specified amount. The stress can be determined by multiplying the strain by the modulus of elasticity. It is important to point out that the modulus of elasticity decreases with increasing temperature and that these temperature-influenced changes vary with different metals.

Elastic Limit

The elastic behavior of a metal reaches a limit at a level of stress termed the elastic limit. This is the highest stress a member can bear and still return to its original dimensions when the load is released. When the elastic limit is exceeded, the member permanently deforms. The elastic limit of a metal is structure-sensitive and dependent on the strain rate.

The design of many components is limited by the elastic limit. Therefore, several properties related to this limit have been defined. These properties can be determined; in fact, the modulus of elasticity often remains unchanged even after considerable alloy additions have been made.11

Table 2 presents the modulus of elasticity for a number of metals.


mined from the stress-strain diagram that is commonly plotted for a tension test. A typical diagram is presented in Figure 2.

The stress-strain curve is initially a straight line, which is indicated in Figure 2 as line A-A’. The slope of this line is the metal’s modulus of elasticity. As the line proceeds upward, a point is reached at which the strain exceeds the amount predicted by the earlier straight-line relationship. It is difficult to determine the exact point at which the proportionality between stress and strain ends because the clarity and interpretation of the curve may vary. The elastic limit on the stress-strain curve shown in Figure 2 is approximately 28 kips per square inch (ksi) (190 MPa). This is the maximum point at which the strain remains directly proportional to stress.

When a metal is strained below the elastic limit, it recovers upon removal of the load. On the other hand, when a metal is stressed beyond its elastic limit, the additional strain is plastic in nature and results in permanent deformation. As an example, if the tensile specimen depicted in Figure 2 were loaded to 32 ksi (220 MPa), as shown in S₁, the specimen would elongate 0.00125 in./in. (.00125 mm/mm). Upon removal of the load, the specimen would not return to its original length but would display a permanent stretch of approximately 0.00015 in./in. (0.00015 mm/mm), represented by line B-B’.

**Yield Strength**

The yield strength of a metal is the stress level at which the metal exhibits a specified deviation from the proportionality of stress and strain. A practical method utilized to determine the yield strength of a metal is illustrated in Figure 2. Line C-C’ is drawn parallel to the elastic Line A-A’ from a point on the abscissa, representing 0.2% (0.0020 in./in. [0.0020 mm/mm]) elongation. Line C-C’ intersects the stress-strain curve at S₂, where the stress level is approximately 38 ksi (260 MPa). This stress is the yield strength of the tested metal. While an offset yield strength of 0.2% is commonly used in engineering design, offsets of 0.1% and 0.5% are sometimes employed in the same manner for various metals.

**Tensile Strength**

The ratio of the maximum load sustained by a tension test specimen to the original cross-sectional area is referred to as the ultimate tensile strength (UTS). The UTS is the most common value calculated from the standard tension test. However, the true tensile strength of a metal, which is the ratio of the breaking load to the final cross-sectional area, may be substantially higher than the reported tensile strength.
The tensile strength values obtained for metals are influenced by many factors. Tensile strength is a structure-sensitive property. It is dependent upon chemical composition, microstructure, the direction of rolling, grain size, and strain history. The size and shape of the specimen and the rate of loading can also affect the result. For these reasons, the UTS of the heat-affected zone may be different from that of the unaffected base metal.

Fatigue Strength

Behavior under cyclic loading is an important aspect of the strength of metals and welded joints. Fatigue fractures develop because the applied forces, even at nominal tensile stresses lower than yield-point stress, cause the tip of a crack to advance a minute amount. This phenomenon is termed stable crack growth. The rate of crack growth increases as the area ahead of the crack decreases until the crack reaches a critical size. At this point, unstable crack growth initiates, and sudden, complete failure follows.

Crack growth does not occur when the net stress at the crack tip is compressive, however. A crack may initiate due to high residual tensile stresses, but the formation of the crack will relieve the local stress condition. Thus, if the applied load is compressive, the crack will not grow to a critical size.

The stress that a metal can endure without sustaining fracture decreases as the number of repeated stress applications increases. Fatigue strength is generally defined as the maximum stress that can be sustained for a stated number of cycles without failure. As the number of cycles is increased, the corresponding fatigue strength becomes lower. The term fatigue life, accordingly, refers to the number of cycles of stress that can be sustained by a metal under stipulated conditions.

For a given stress level, the fatigue strength of steel is constant beyond approximately two million cycles. Several million additional cycles are required to cause a significant reduction in fatigue strength. Thus, for practical purposes, the fatigue limit is the maximum stress or stress range that a metal can bear for an infinite number of cycles without sustaining fracture. Such a limiting stress level is often called the endurance limit.

The endurance limits reported for metals in engineering handbooks are usually determined using polished round specimens tested in air. These data are valid and useful for design in applications such as shafts in rotating machinery and other uniform members. However, they may have little relevance in the design of weldments, as these are characterized by abrupt changes in cross section, geometrical and metallurgical discontinuities, and residual stresses, all of which adversely affect fatigue life.

Weldments in rotating equipment are particularly prone to fatigue failure. Pressure vessels can also fail by fatigue when the pressurization is cyclic and stress above the fatigue strength is concentrated at some point. When designing welded built-up members and welded connections for structures subject to fatigue loading, the applicable standard governing the subject structure must be followed. In the absence of a specific standard, the designs of existing welded components should be used as a guide.

Localized stresses within a structure may result entirely from external loading, or they may be caused by a combination of applied and residual stresses. Residual stresses are not cyclic, but they may augment or detract from applied stresses, depending upon their respective signs. For this reason, it may be advantageous to induce, if possible, compressive residual stress in critical areas of a weldment where cyclic applied tensile stresses are expected. This can be accomplished by a welding sequence that controls the residual stresses produced during welding or by a localized treatment that acts to place the surface in compression.

Thermal stresses must be considered in the same light as applied stress. Thermal stresses result from the expansion and contraction of a material during heating and cooling. As a metal is heated, it expands. If the material is restrained and not free to expand, thermal compressive stresses are formed. Conversely, if a restrained part is cooled, the result is a tensile stress. Thermal cycling can lead to fatigue failure if the thermal gradients are steep or if the thermal stresses are concentrated by a stress raiser, such as a change in cross section or a discontinuity.

In summary, the designers of weldments require a thorough understanding of the fatigue characteristics of metals as used in weldments. For weldments subject to loading in the tension range, the most common cause of fracture is fatigue. One of the reasons for this is the frequent presence of stress raisers, which concentrate imposed cyclic stresses to levels above the fatigue limit of the metal for the existing conditions.

Ductility

The amount of plastic deformation that an unwelded or a welded specimen undergoes in a mechanical test carried to fracture is considered a measure of the ductility of the metal or the weld. Values expressing ductility in various mechanical tests are meaningful only for the relative geometry and size of the test specimen. Thus, they do not measure any fundamental characteristic, but merely provide relative values for the comparison of the ductilities of metals subjected to identical test conditions. The plasticity exhibited by a specimen is simply the deformation accomplished during the yielding process. Regardless of the method of measurement, ductil-
Fracture Toughness

A metal that is judged ductile by a standard tension or slow-bend test may perform in a brittle manner in another type of test or when exposed to service conditions. Thus, the only prediction that can be made with reasonable certainty based on tension or bend test results is that a metal with very little ductility is not likely to perform in a ductile manner in any other type of mechanical test carried to fracture. However, a metal that displays good ductility in a tensile or bend test may or may not behave in a ductile manner in other types of mechanical tests. In fact, ductile (as determined by tensile and bend tests) metals have been known to fracture in service with little or no plastic deformation. This lack of deformation and other aspects of such failures usually indicate that little energy was required to produce the fracture. This general phenomenon prompts metallurgists to speak of the toughness of metal as a property distinct from ductility.  

The term fracture toughness is defined as the ability of a metal to resist fracture under conditions that are unfavorable to energy absorption in the presence of a notch and to accommodate loads by plastic deformation. Four conditions markedly influence the behavior of a metal. These are the rate of loading, the nature of the load (i.e., whether the imposed stresses are uniaxial or multiaxial), the temperature of the metal, and the presence of a notch.

Many metals can absorb energy and deform plastically under the simple circumstances represented in tension or bend tests (and therefore would be judged ductile); however, a lesser number of these metals exhibit good toughness when tested under conditions of high stress concentration. The toughness displayed by a metal tends to decrease as (1) the rate of loading increases, (2) the applied stresses become multiaxial, and (3) the temperature of the metal is lowered. Weldments in service may easily be exposed to one or more of these conditions. Consequently, concern about the toughness of the weld metal and weld heat-affected zones is warranted.

When ductile metals are employed in the design of engineering structures, including welds, design strength is normally based on an analysis to ensure that the applied stresses are below the design strength. Failures that occur at load levels below the design strength are broadly classified as brittle failures. These failures can result from the effects of discontinuities or crack-like defects of critical size in the weld or the base metal that do not greatly alter the nominal stress distribution and are customarily neglected in the design.

When structural grade steel is tested in uniaxial tension, it typically deforms in a ductile manner prior to rupture at the ultimate load. Because the volume of metal must remain constant, any elongation in one direction must be accompanied by contraction in one or both of the other directions. The uniaxial tension test specimen is free to contract in the other direction, resulting in ductile behavior. If the necessary lateral contraction is severely constrained or prevented and the longitudinal stresses are sufficiently large, the same material that exhibits ductile behavior in a simple tension test may fail in a brittle manner.

In structures in which separate elements are joined by welding, the conditions of constraint and stress concentrations are usually very different from those produced by simple uniaxial tension. Substantial material thickness alone may provide sufficient constraint to prevent lateral contractions. Thus, structural details that have proved satisfactory in long usage and service may not necessarily have adequate ductile characteristics if material dimensions are proportionately increased to a large degree.

A complete fracture-safe analysis requires proper attention to the role of discontinuities. For many classes of structures, including ships, bridges, and pressure vessels, experience with specific designs, materials, and fabrication procedures has established a satisfactory correlation between Charpy V-notch test standards...
for base and weld metals and acceptable service. The challenge is to ensure the soundness and integrity of a new design. Fracture mechanics analyses are employed to guard against the effects of common weld discontinuities. As it is widely recognized that welded joints usually contain some discontinuities, designers using welded joints in their plans are faced with a dilemma. Although they aim to design joints that are entirely free of discontinuities, this goal is not realistic. The practical approach is to place a reasonable limit on those discontinuities that are bound to be present. Consequently, the challenge consists of determining the types and the extent of discontinuities that are acceptable.

While conventional toughness testing procedures cannot directly address this challenge, fracture mechanics tests, where applicable, specifically define the relationship between flaw size transverse to the stress field and fracture stress for a given base metal or weld joint. Thus, these tests permit a direct estimate of acceptable flaw sizes for different geometrical configurations and operating conditions. Fracture mechanics can establish the minimum or critical crack-like flaw size that will initiate unstable crack propagation under tensile stress.

It should be noted, however, that in members subject to cyclic loading or corrosion, or both, cracks might initiate at stress raisers that are considered acceptable discontinuities. These small cracks could grow by stable crack extension with each application of tensile stress until the crack reaches the critical size. For such conditions, information relative to crack growth rate is essential to the establishment of acceptance criteria and continuing inspection frequencies.

**Low-Temperature Behavior**

As pressure vessels and other welded products are sometimes expected to operate at low (below 32°F [0°C]) temperatures, weldment designers must consider the properties exhibited by metals at these low temperatures. Very low temperatures are involved in cryogenic service, which entails the storage and use of liquefied industrial gases such as oxygen and nitrogen. Lowering the temperature of a metal profoundly affects its fracture characteristics, particularly if the metal possesses a body-centered-cubic crystalline structure (carbon steel, for example).\(^{15}\)

Strength, ductility, and other properties change in all metals and alloys as the temperature decreases. The modulus of elasticity rises, for instance. As a rule, the tensile and yield strengths of all metals and alloys increase as the temperature is lowered.

Though the ductility of most metals and alloys tends to decrease as the temperature is lowered, some metals and alloys retain considerable ductility at very low temperatures. As discussed in the previous section, fracture toughness is typically a function of temperature. Figure 3 presents a transition curve that shows this relationship. At higher and lower temperatures, the fracture toughness is fairly constant. However, between these two plateaus, the fracture toughness decreases as the temperature decreases. This is known as the ductile-to-brittle transition. The suitability of metals for low temperature service is judged by testing.

The principal factors that determine the low-temperature behavior of a metal during mechanical testing are (1) crystal structure, (2) chemical composition, (3) the size and shape of the test specimen, (4) the conditions of manufacture and heat treatment, and (5) the rate of loading.

The most common specimen used for low-temperature testing is the Charpy V-notch impact specimen. The Charpy V-notch impact strengths for five common metals are listed in Table 3.

Iron and steel suffer a considerable reduction in impact strength at low temperatures. The addition of alloying elements to steel, especially nickel and manganese, can substantially improve fracture toughness at low temperatures, while increased carbon and phosphorus can greatly decrease low-temperature fracture toughness.

---

Elevated-Temperature Behavior

The performance of a metal in service at an elevated temperature (i.e., 75°F [25°C] to 500°F [260°C]) is governed by other factors in addition to strength and ductility. Time becomes a factor because at high temperatures metals undergo the phenomenon known as creep, defined as “deformation with time at service temperature.” In other words, the section under stress continues to deform even if the load is maintained constant. The rate at which a metal creeps increases rapidly with increasing temperature and increasing load. Thus, the time over which a metal under load deforms too much to be usable can vary from many years at a slightly elevated temperature to a few minutes at a temperature near the melting point.

The creep rates of metals and alloys differ considerably. If the temperature and stress are sufficiently high, the metal will creep until rupture occurs. The term creep rupture is used to identify the mechanics of deformation and the failure of metals under stress at elevated temperatures.

PHYSICAL PROPERTIES

The physical properties of metals constitute an important aspect of the weldability of metals. Welding engineers must be cognizant of the fact that the success of a particular joining operation may depend on one or more physical properties. The constants provided for metals and alloys are satisfactory for most engineering purposes.

Only those physical properties that require consideration in the fields of weldment design and fabrication of a weldment are discussed here. These include thermal conductivity, melting temperature, thermal expansion and contraction, and electrical conductivity.

Thermal Conductivity

The rate at which heat is transmitted through a material by conduction is referred to as thermal conductivity or thermal transmittance. Metals are better heat conductors than nonmetals, and metals with high electrical conductivity generally have high thermal conductivity.

Metals differ considerably in their thermal conductivities. Copper and aluminum are excellent conductors, which makes these metals difficult to weld using a relatively low-temperature heat source such as an oxyacetylene flame. Conversely, the high conductivity of copper makes it a good heat sink when employed as a hold-down or backing bar.

Melting Temperature

As a rule, the higher the melting point is, the greater the amount of heat needed to effect melting. Thus, the temperature of the heat source in welding must be well

above the melting point of the metal. In addition, the welding of two metals of dissimilar compositions becomes more challenging as the difference in their melting points become greater.

**Thermal Expansion and Contraction**

The distortion that results from welding must be considered during weldment design so that the final dimensions of the weldment are acceptable. Most metals increase in volume when heated and decrease in volume when cooled. The term coefficient of thermal expansion refers to the unit change in the linear dimensions of a body when its temperature is changed by one degree. Thus, this coefficient serves as an indicator of expansion and contraction in a metal subjected to increasing or decreasing temperature, respectively.

Although engineers are generally concerned with changes in length in metal components, a linear coefficient of thermal expansion is not the only design consideration. A coefficient for volume change is also important. Metals also change in volume when they are heated and cooled during welding. Thus, the greater the increase in volume and localized upsetting during heating, the more pronounced the distortion from welding.

**Electrical Conductivity**

Metals are relatively good conductors of electricity. However, increasing the temperature of a metal interferes with the electron flow, resulting in a decrease in electrical conductivity. Adding alloying elements to a metal and cold working also decrease conductivity. Electrical conductivity is an important variable, particularly in the resistance welding processes, inasmuch as the electrical flow is imperative to weld soundness.

**CORROSION PROPERTIES**

The corrosion properties of a metal determine its mode and rate of deterioration by means of a chemical or electrochemical reaction with the surrounding environment. As metals and alloys differ greatly in their corrosion resistance, this behavior merits consideration in planning and fabricating a weldment for a particular service. Designers must therefore be familiar with the behavior of welded joints under corrosive conditions.20

Weld joints often display corrosion properties that differ from the remainder of the weldment. Differences may be observed between the weld metal and the base metal and sometimes between the heat-affected zone and the unaffected base metal. The surface effects produced by welding—heat tint formation and oxidation, for example—are also important factors in the corrosion behavior of the weld metal.

Welds made between dissimilar metals or with dissimilar filler metals may be subject to electrochemical corrosion. Therefore, appropriate protective coatings are required to avoid corrosion in sensitive environments.

**WELDMENT DESIGN PROGRAM**

A weldment design program begins with the recognition of a need. This need may be for an improvement in an existing machine or for the building of an entirely new product or structure using advanced design and fabrication techniques. In any case, many factors must be taken into account before a design is finalized. These considerations involve numerous questions and considerable research into the various areas of engineering, production, and marketing.

**ANALYSIS OF EXISTING DESIGNS**

When an entirely new machine or structure is to be designed, information should be obtained about similar products, including those marketed by other manufacturers or builders. If a new design is to replace an existing design, the strengths and weaknesses of the existing design should be determined. The following factors should be considered in identifying the strengths and weaknesses of existing designs:

1. Performance history of the existing products;
2. Features that should be retained, discarded, or added;
3. Any suggestions for improvements that have been made; and
4. Opinions of customers and the sales force about the existing products.

**DETERMINATION OF LOAD CONDITIONS**

The service conditions and requirements of a weldment that might cause the overloading of a weldment should be ascertained. As a starting point for the calculation of loads, the following guidelines may be useful:

1. Determine the torque on a shaft or revolving part from the motor horsepower and speed;

---

2. Calculate the forces on members caused by the dead weight of the parts;
3. Determine the maximum load on members of a crane hoist, shovel, lift truck, or similar material handling equipment from the load required to tilt the machine;
4. Consider the force required to shear a critical pin as an indication of maximum loading on a member; and
5. Determine the desired service life and the frequency of loading (for example, designing for fatigue at lesser loads may be more critical than designing for maximum strength to resist infrequently applied maximum loads).

In the absence of a satisfactory starting point, the designer should plan for an assumed load and adjust the design based on experience and testing.

MAJOR DESIGN FACTORS

In developing a design, designers should consider the manner in which decisions affect production operations, manufacturing costs, product performance, appearance, and customer acceptance. Many factors that are seemingly far removed from engineering considerations can become major design factors. Some of these, along with other relevant rules, are highlighted below:

1. The design should satisfy strength and stiffness requirements, but overdesigning not only constitutes a poor engineering practice but also wastes materials and labor and increases production and shipping costs;
2. Safety factors should be realistic;
3. As an attractive appearance may be necessary in areas exposed to view, the drawing or specifications should indicate those welds that must be ground or otherwise conditioned to enhance appearance;
4. Deep, symmetrical sections should be used to minimize distortion;
5. Welding the ends of beams rigidly to supports may increase strength and stiffness;
6. Rigidity may be provided by welded stiffeners, precluding the need to increase material thickness and weight;
7. Tubular sections or diagonal bracing should be used for torsion loading, as a closed tubular section is significantly more effective in resisting torsion than an open section of similar weight and proportions;
8. Standard rolled sections, rather than special built-up sections, should be used for economy and availability when they satisfy the need;
9. Accessibility for maintenance must be considered during the design phase; and
10. Standard, commercially available components such as index tables, way units, heads, and columns should be specified when they would serve the purpose.

DESIGNING THE WELDMENT

Flexibility is one of the advantages of welded design, resulting in many opportunities for savings. The following are general suggestions for effective design:

1. Design for ease of material handling, inexpensive tooling, and accessibility of the joints for reliable welding;
2. Check with the shop for ideas that can contribute to cost savings;
3. Establish realistic tolerances based on end use and suitability for service, as excessively close tolerances serve no useful purpose and increase costs; and
4. Minimize the number of pieces to reduce assembly time and the amount of welding.

Workpiece Preparation

Thermal cutting, shearing, sawing, blanking, nibbling, and machining are methods used to cut blanks from stock material. The selection of the appropriate method depends on the available material and equipment and the relative costs. The quality of edges needed for good fitup and the type of edge preparation for groove welds must be kept in mind. The following points should also be considered when preparing material for welding:

1. Dimensioning of a blank may require a stock allowance for subsequent edge preparation,
2. The detail of the welded joints must be considered when laying out a blank with the intent to cut and prepare the edge for welding simultaneously,
3. Weld metal costs can be reduced for thick plate by specifying J- or U-groove preparations, and
4. Air carbon arc gouging, oxygen gouging, or chipping should be contemplated for back weld preparation.

Forming

The forming of parts can sometimes reduce the cost of a weldment by avoiding joints and machining
operations. The selection of a forming method is based on the composition of the base metal, thickness, overall dimensions, production volume, tolerances, and cost. The following suggestions may facilitate the decision-making process:

1. Corners can be created by bending or forming rather than by welding two pieces together,
2. Flanges can be bent on the plate rather than welding flanges to it,
3. A casting or forging can be used in place of a complex weldment to simplify the design and reduce manufacturing costs, and
4. A surfacing weld instead of an expensive alloy component can be used on an inexpensive component to provide wear resistance or other properties.

Cold forming reduces the ductility and increases the yield strength of metals. Heat treating the metal to restore ductility may increase or decrease the strength, depending on the alloy type. The heat produced by arc welding on a cold-formed material may also affect the mechanical properties of the base metal. The mechanical properties of the heat-affected zone of cold-formed materials may be reduced by the heat of welding. Generally, the relevant standard provides the maximum cold-forming allowances and minimum strength properties of cold-formed weldments. For instance, Section VIII of the 1998 Boiler and Pressure Vessel Code requires that under certain circumstances cold forming that results in extreme fiber elongation (over 5%) in carbon, low-alloy, and heat-treated steel plates must be stress relieved. It should be noted that welding in close proximity to material that has been strained 5% or more results in material with negligible notch toughness.

**Weld Joint Design**

Weld joint design should be selected primarily on the basis of load requirements. However, variables in design and layout can substantially affect costs. The following guidelines generally apply:

1. The joint design that requires the least amount of weld metal should generally be selected;
2. Otherwise, square-groove and partial joint penetration groove welds should be used whenever they satisfy strength and serviceability requirements;
3. Lap joint and fillet welds should be used instead of groove welds unless the lower fatigue resistance of these joints is inadequate to satisfy the service requirements;
4. Double-V- or U-groove welds should be used instead of single-V- or U-groove welds on thick plates to minimize the amount of deposited weld metal and resulting distortion;
5. For corner joints in thick plates in which fillet welds are not adequate, the beveling of the members subject to through-thickness weld shrinkage strains should be considered to reduce the tendency for lamellar tearing; and
6. The assembly and joints should be designed to provide ready accessibility for welding.

**Size and Amount of Weld**

Overdesign is a common error, as is overwelding in production. The control of weld size begins with design, but it must be maintained during the assembly and welding operations. The following are basic guidelines regarding the control of weld size:

1. Adequate but minimum size and length should be specified for the forces to be transferred. Oversized welds may cause excessive distortion and higher residual stress without improving suitability for service; they also contribute to increased costs. The size of a fillet weld is especially important because as the fillet weld size increases, the weld metal cross-sectional area increases in proportion relative to the weld size squared;
2. For equivalent strength, a continuous fillet weld of a given size is usually less costly than a larger-sized intermittent fillet weld. Continuous fillet welds also have fewer weld terminations, which are potential sites of discontinuities;
3. An intermittent fillet weld can be used in place of a continuous fillet weld of minimum size when static load conditions do not require a continuous weld. However, it should be recognized that intermittent fillet welds should have a low allowable stress range when cyclic loading is a design consideration;
4. To derive maximum advantage from automatic welding, it may be better to use one continuous weld rather than intermittent welds;
5. Weld size should not be larger than that required for the strength of the thinner workpiece based on the load;
6. Welding of stiffeners or diaphragms should be limited to that required to prevent out-of-plane distortion of the supported components under maximum loads as well as during shipment and handling; and

---

7. The amount of welding should be kept to a minimum to limit distortion and internal stresses, thus minimizing the need for and the cost of stress relieving and straightening.

**Subassemblies**

In visualizing assembly procedures, designers should break the weldment into subassemblies to determine the arrangement that offers the greatest cost savings. Subassemblies offer the following advantages:

1. Two or more subassemblies can be worked on simultaneously;
2. Subassemblies usually provide better access for welding and may permit automatic welding;
3. Distortion in the finished weldment may be easier to control;
4. Large-sized welds may be deposited under lesser restraint in subassemblies, which aids in minimizing residual stresses in the completed weldment;
5. Machining of subassemblies to close tolerances and stress relieving of certain sections can be performed before final assembly, if necessary;
6. Chamber compartments can be tested for leaks and painted before final assembly;
7. In-process inspection and repair is facilitated; and
8. Handling costs tend to be much lower.

**WELDING PROCEDURES**

Although designers have little control of welding procedures, they can influence which procedures are used in production. The following guidelines can help to ensure the ultimate success of weldment design:

1. Backing bars increase the speed of welding when making the first pass in groove welds;
2. The use of low-hydrogen electrodes and welding processes may eliminate or reduce preheat requirements;
3. If the plates are not too thick, a joint design requiring welding from only one side should be considered to avoid repositioning or overhead welding;
4. A built-up or crown weld is generally unnecessary to obtain a full-strength joint;
5. Joints in thick sections should be welded under conditions of the least restraint, for example, prior to the installation of stiffeners; and
6. Sequencing of fitup, fixtureing, and welding is particularly important for box members made of plates because after the completion of welding the correction of distortion is virtually impossible.

**LAMINATIONS AND LAMELLAR TEARING**

Weldment designers must understand the true significance of the residual stresses that result from the contraction of the cooling weld metal. These contractions result in strains that are in excess of yield level strains if sufficient restraint exists. The shrinkage strains provide the potential for lamellar tearing—the formation of cracks parallel to the direction of rolling in material stressed in the through-thickness direction. Lamellar tears generally initiate from flattened, low-melting-point constituents in steel (e.g., manganese sulfides).

Lamellar tears occur most often during fabrication shortly after the solidification of the weld metal. Because the strains associated with design stresses are so small (limited to less than yield point stress by code) in comparison to the weld shrinkage strains that are responsible for lamellar tearing, service loads do not initiate lamellar tearing. The detail geometry, material thickness, and weld size selected may lead to difficult or virtually impossible fabrication conditions.

In connections in which a member is welded to the outside surface of a main member, the capacity to transmit through-thickness tensile stresses is essential to the proper functioning of the joint. Laminations, which are pre-existing planes of weakness, and lamellar tears may impair this capacity.

Consideration of the phenomenon of lamellar tearing must include design aspects and welding procedures that are consistent with the properties of the base material. In connections in which lamellar tearing might be a concern, the design should provide for maximum component flexibility and minimum weld shrinkage strain. The following precautions should help to minimize lamellar tearing in highly restrained welded connections:23

1. On corner joints, the edge preparation should be on the through-thickness member, when feasible;
2. The size of the weld groove should be kept to a minimum, consistent with the design requirements;
3. Overwelding should be avoided because rather than providing conservative design it may cause the material to weaken;
4. Welds in corner and T-joints should be completed early in the fabrication process, if possible, to minimize restraint in such joints;
5. A predetermined welding sequence should be selected to minimize overall shrinkage in the most highly restrained elements;
6. The lowest-strength weld metal available, consistent with design requirements, should be used.

---


23. It is assumed that procedures producing low-hydrogen weld metal would be used.
to promote straining in the weld metal rather than in the more sensitive through-thickness direction of the base metal;

7. Buttering with weld metal approximately 3/16 in. (5 mm) thick and extending beyond the limits of the joint prior to fitup has been shown by research testing to provide an area of material less susceptible to lamellar tearing at the location where the most severe strains occur. This technique has proven very effective in reducing the hazard of lamellar tearing; and

8. Specification of a material with improved through-thickness ductility should be considered for critical connections.24

In critical joint areas subject to through-thickness direction loading, ultrasonic examination should be conducted to avoid the use of a material containing preexisting laminations and large nonmetallic inclusions. In addition, no sooner than 48 hours after the completion of welding, designers should specify postweld ultrasonic inspection of those specific highly restrained connections that could be subject to lamellar tearing and are critical to the structural integrity. They must also consider whether minor weld flaws or base metal imperfections can be left unrepaired without jeopardizing structural integrity.

Gouging and repair welding add additional cycles of weld shrinkage to the connection and may result in the extension of existing flaws or the generation of new flaws by lamellar tearing.

CLEANING AND INSPECTION

Design specifications can affect cleaning and inspection costs. The safety requirements of the weldment determine the type and amount of inspection required. The timeliness of inspection is of paramount importance in order to maintain the orderly flow of work and correct deficiencies without interfering with subsequent operations. For example, as-welded joints that have uniform appearance are acceptable for many applications. Therefore, the surface of a weld need not be ground smooth or flush unless required for another reason inasmuch as the smoothing of a weld is an additional operation.

Undesirable overwelding should be noted during inspection because it can be costly and it contributes to distortion. Corrective action should be directed at work in progress rather than at completed weldments. The type of nondestructive inspection to be used on weldments must be capable of detecting the types and sizes of weld discontinuities that are to be evaluated for acceptability.25

WELDED DESIGN CONSIDERATIONS

The performance of any member of a structure depends on the properties of the material and the characteristics of the section. If a design is based on the efficient use of these properties, the weldment should function well and conserve materials. Engineers assigned to design welded members must possess the knowledge to select the most efficient structural section and determine the required dimensions. They must also know when to use stiffeners and how to size and place them when used.

The mathematical equations utilized to calculate forces and their effects on sections and to determine the sections needed to resist such forces appear quite forbidding to the novice. With the proper approach, however, it is possible to simplify the design analysis and the application of these equations. In fact, as is explained below, it is often possible to make correct design decisions merely by examining one or two factors in an equation, precluding tedious calculations. As a whole, the mathematics used in design for welding is no more complex than that used in other engineering fields.

APPROACH TO WELD REDESIGN

From the standpoint of performance and ultimate production economics, the redesign of the machine or structure as a whole is preferable. The designer is then unrestricted by the previous design and may be able to reduce the number of pieces, the amount of material used, and the labor for assembly. A better, lower-cost product is realized immediately. When the adjustment to changes in production procedures is complete, the company is in a position to benefit more fully from welded design technology.

However, considerations other than the engineer’s wishes may prevail. Available capital and personnel considerations often limit a company’s capabilities when changing to welded design. For example, when a machine is to be converted from a casting to a welded design, management may favor the redesign of one or more components over the use of weldments and the conversion of the design over a period of years to an all-
welded product. Gradual conversion avoids the obsolescence of facilities and skills and limits the requirement for new equipment. Supplementing these considerations is the need to maintain a smooth production flow and to test the production and market value of the conversion as it is made step by step.

**REDESIGN VERSUS NEW DESIGN**

A redesign of a product may be based on the previous design or solely on loading considerations. Following a previous design has the advantages of offering a “safe” starting point if the old design is known to have performed satisfactorily. This approach has disadvantages, however, in that it stifles creative thinking with respect to the development of an entirely new concept to solve the basic problem. Little demand is made on the ingenuity of the designer when the welded design is modeled on the previous product.

A design based on the loading requires designers to analyze what is needed and propose configurations and materials that best satisfy the identified needs. They must know or ascertain the type and amount of load, the values for stress allowables in a strength design, or the deflection allowables in a stiffness design.

**STRUCTURAL SAFETY CONSIDERATIONS**

The production of a safe welded structure depends upon a combination of proficient design practices, skilled fabrication, and sound construction methods. In design, the selection of a safety factor and the appropriate analytical procedures requires experience and sound engineering judgement. Deterioration as a result of corrosion or other service conditions during the life of the structure, variations in material properties, potential imperfections in materials and welded joints, and many other factors also need consideration.

A rational approach to structural safety is a statistical evaluation of the random nature of all the variables that determine the strength of a structure as well as the variables that may cause it to fail. From these data, the probability of failure can be evaluated and the probability of occurrence kept at a safe level for the application, taking into considering the risk of injury, death, property damage, and unsatisfactory service performance. The choice of materials and safe stress levels in members may not produce the most economical structure. However, safety must take precedence over cost savings whenever a question of which should govern arises.

When designers attempt a new design or structural concept, great skill and care as well as detailed stress analyses are required. Laboratory tests of models or sections of prototype structures should be used to verify new designs.

**DESIGN EQUATIONS**

The design equations for strength and stiffness always contain terms representing the load, the member, the stress, and the strain or deformation. If any two of the first three terms are known, the others can be calculated. All problems of design thus reduce into one of the following:

1. Determination of the internal stress or the deformation caused by an external load on a given member,
2. Determination of the external load that may be placed on a given member for any given strength or deformation, or
3. Selection of a member to carry a given load without exceeding a specified strength or deformation.

A force causes a reaction such as tension, compression, bending, torsion, or shear stress in the member. The result is a strain measured by means of the relative displacements in the member. These include elongation, contraction, deflection, or angular twist. A useful member must be designed to carry a certain type of load within an allowable stress or deformation. In designing within the allowable limits, designers should select the most efficient material section size and section shape.

**DESIGNING FOR STRENGTH AND STIFFNESS**

A design may require strength only or strength and stiffness to support the load. All designs must have sufficient strength to prevent the members from failing by fracture or yielding when subjected to normal operating loads or reasonable overloads. Strength designs are common in road machinery, farm implements, motor brackets, and various types of structures. If a weldment design is based on calculated loading, design equations for strength are used to dimension the members. Tables of equivalent sections or nomographs can be used to determine required dimensions for strength and stiffness.

In weldments such as machine tools, stiffness as well as strength is important because excessive deflection under load results in a lack of precision in the product. A design based on stiffness also requires the use of design equations for sizing members.

Some parts of a weldment serve their design function without being subjected to loads much greater than their own weight, or dead loads. Typical examples of such members are fenders, dust shields, safety guards, cover plates for access holes, and enclosures included for aesthetic purposes. Only casual attention to strength and stiffness is required in sizing such members.
The properties of the material and those of the section determine the ability of a member to carry a given load.

The common design equations that have been developed for various conditions and member types are too numerous for inclusion here. However, several of these equations are presented below to illustrate specific design problems.\(^\text{26}\)

The application of design equations can be illustrated by the problem of obtaining adequate stiffness in a cantilever beam. The amount of vertical deflection at the end of the beam under a concentrated load at the end, illustrated in Figure 4, can be determined using the following equation for deflection:

\[
\Delta = \frac{FL^3}{3EI}
\]  

where

- \(\Delta\) = Deflection, in. (mm);
- \(F\) = Concentrated load, lbf (N);
- \(L\) = Length, in. (mm);
- \(E\) = Modulus of elasticity, psi (MPa); and
- \(I\) = Moment of inertia, in.\(^4\) (mm\(^4\)).

It is normally desirable to have the least amount of deflection. Therefore, the modulus of elasticity and moment of inertia values should be as large as possible. The commonly used structural metal that has the highest modulus of elasticity is steel, with a value of approximately \(30 \times 10^6\) psi (200,000 MPa).

The other factor is the moment of inertia of a cross section. The moment of inertia of a member is a function of the geometry of the member. The beam must have a cross section with a moment of inertia about the horizontal axis large enough to limit the deflection to a permissible value. A section with an adequate in-plane moment of inertia will satisfy the vertical deflection requirement, whatever the shape of the section may be. However, the out-of-plane stability of the beam may also need consideration, especially if the forces are transverse to the principal axis or if torsion is involved. A decision must then be made as to which shape should be used for the best design at the lowest cost.

Types of Loading

The five basic types of loading are tension, compression, bending, shear, and torsion. When one or more types of loading are applied to a member, they induce stress in addition to any residual stresses that are present.

The applied load results in stress within the member, and the magnitude of the strain is governed by the modulus of elasticity of the metal. Some deformation always takes place in a member when the load is applied because the associated stress inevitably causes strain.

**Tension.** Pure tensile loading is generally the simplest type of loading from a design and analysis perspective. Axial tensile loads cause axial strains and elongation. In the case of tensile loading, the principal design requirement is an adequate gross and net cross-sectional area to carry the load.

**Compression.** A compressive load may require appropriate design provisions to prevent buckling. Few compression members fail by crushing or by exceeding the ultimate compressive strength of the material. If a straight compression member such as the column represented in Figure 5(A) is loaded through its center of gravity, the resulting stresses are simple axial compressive stresses. If the member is a slender column, it will start to bow laterally as a result of small imperfections and accidental eccentricity of loading. This movement is shown in Figure 5(B). As a result of bowing, the central portion of the column becomes increasingly eccentric to the axis of the force and causes a bending moment on the column, as shown in Figure 5(C).

Under a steady load, the column will remain stable under the combined effect of the axial stress and the bending moment. However, with increasing load and the associated curvature, depicted in Figure 5(D), a
critical point will be reached at which the column will buckle and fail. The presence of residual stresses developed during manufacturing may also reduce the failure load. As a column deflects under load, a bending moment can develop in restrained end connections.

Two properties of a column are important for the calculation of compressive strength—the cross-sectional area, \( A \), and the radius of gyration, \( r \), which is the distance from the neutral axis of the section to an imaginary line in the cross section about which the entire area of the section could be concentrated and still have the same moment of inertia about the neutral axis of the section. The area is multiplied by the critical buckling stress, \( \sigma_{cr} \), to arrive at the compressive load that the column can support in the absence of buckling. The radius of gyration indicates, to a certain extent, the column’s ability to resist buckling. The equation for the radius of gyration is as follows:

\[
r = \sqrt{\frac{I}{A}} \tag{3}
\]

where

- \( r \) = Radius of gyration, in. (mm);
- \( I \) = Moment of inertia about the neutral axis, in.\(^4\) (mm\(^4\)); and
- \( A \) = Cross-sectional area of the member, in.\(^2\) (mm\(^2\)).

Inasmuch as the worst condition is always of concern in design work, it is necessary to use the smallest radius of gyration relative to the unbraced length. Thus, an overloaded unbraced wide-flange column section will always buckle, as shown in Figure 5(D). However, if the column is braced at the mid-length to prevent buckling, the column could buckle toward a flange. The critical slenderness ratio of a column is the larger of ratios \( L_x/r_x \) or \( L_y/r_y \), where \( L_x \) and \( L_y \) are the distances between braced points, and \( r_x \) and \( r_y \) are the radii of gyration about the x and y axis, respectively.

The design of a long compression member is carried out by trial and error. A trial section is selected, and the cross-sectional area and the least radius of gyration are determined. A suitable column table\(^{27}\) is used to provide the critical buckling stress, \( \sigma_{cr} \), for the column slenderness ratio, \( KL/r \). This stress is then multiplied by the cross-sectional area, \( A \), to find the buckling strength of the column and, when a suitable safety or resistance factor is chosen, the design load the column can carry. If this value is less than the load to be applied, the design must be changed to incorporate a larger section and calculated again. Table 4 presents the AISC load and resistance factor design (LRFD) column equations.

\[
\sigma_{cr} = \left( \frac{0.658}{\lambda_c^2} \right) \sigma_y \tag{1}
\]

For \( \lambda_c \leq 1.5 \):

\[
\sigma_{cr} = \left( \frac{0.877}{\lambda_c^2} \right) \sigma_y \tag{2}
\]

For \( \lambda_c > 1.5 \):

\[
\sigma_{cr} = \left( \frac{1}{\pi \sqrt{E}} \right) KL \left( r \right)^{-1}
\]

where

\( \lambda_c = \) Column slenderness parameter, which equals

\[
\frac{KL}{r}.
\]

\( \sigma_{cr} = \) Critical compressive stress, ksi (MPa);

\( K = \) Effective length factor (ratio of the length of an equivalent pinned ended member to the length of the actual member);

\( L = \) Unbraced length of the member, in. (mm);

\( r = \) Radius of gyration, in. (mm);

\( E = \) Modulus of elasticity, ksi (MPa); and

\( \sigma_y = \) Yield strength, ksi (MPa).


---

Bending. Figure 6 illustrates the bending of a member under uniform loading. Loads may also be nonuniform or concentrated at specific locations on the beam. When a member is loaded in bending within the elastic range, the bending stresses total zero along the neutral axis and increase linearly to a maximum value at the outer fibers.

The bending stress at any distance from the neutral axis in the cross section of a straight beam is shown in Figure 7.

The bending stress can be found with the following expression:

\[ \sigma = \frac{My}{I} \]  

(4)

where

- \( \sigma \) = Bending stress (tension or compression), ksi (MPa);
- \( M \) = Bending moment at the point of interest, kips inches (kips in.) (kilonewton millimeters [kN mm]);
- \( y \) = Distance from the neutral axis of bending to a specific distance, “\( y \)”, in. (mm); and
- \( I \) = Moment of inertia about the neutral axis of bending, in.\(^4\) (mm\(^4\)).

In most cases, the maximum bending stress is of greatest interest. In this case, the equation becomes:

\[ \sigma = \frac{Mc}{I} = \frac{M}{S} \]  

(5)

where

- \( \sigma \) = Bending stress (tension or compression), ksi (MPa);
- \( M \) = Bending moment at the point of interest, kips in. (kN mm);
- \( c \) = Distance from neutral axis to outermost fibers, in. (mm); and
- \( S \) = Section modulus (\( I/c \)), in.\(^3\) (mm\(^3\)).

As the bending moment decreases along the length of a simply supported beam toward the ends, the bending stresses (tension and compression) in the beam also decrease. If a beam has the shape of an I-section, the bending stress in the flange decreases as the end of the beam is approached. If a short length of the tension flange within the beam is considered, a difference exists between tensile forces \( F_1 \) and \( F_2 \) at the two locations in the flange, as shown in Figure 8.

The decrease in the tensile force in the flange results in a corresponding shearing force between the flange and the web. This shearing force must be transmitted by the fillet welds that join the two together. The same reaction takes place in the upper flange, which is in compression. The change in tensile force in the lower flange transfers as shear through the web to the upper flange and is equal to the change in compression in that flange.

28. The tensile force, \( F \), is the product of the tensile stress, \( \sigma \), and the flange cross-sectional area, \( A \).
A common bending problem in machinery design involves the deflection of beams. The beam equations found in many engineering handbooks are useful for quick approximations of deflections of common types of beams in which the span is large compared to the beam depth. An example of a typical beam and applicable equations are presented in Figure 9.

Beams that are supported or loaded in different ways have other applicable design equations. To meet stiffness requirements, the beam should have as large a moment of inertia as practical.

Information concerning the design of a compression member or column may also apply to the compression flange of a beam. The lateral buckling resistance of the compression flange must also be considered. It should have adequate width and thickness to resist local buckling. Moreover, it should be properly supported to prevent twisting or lateral movement and subjected to compressive stresses within allowable limits.

**Shear.** Shear forces in the web of a beam under load are illustrated in Figure 10. The forces are both horizontal and vertical. They create diagonal tensile and diagonal compressive stresses.

The shear capacity of a member of either an I- or a box-beam cross section is dependent upon the slenderness proportion of the web or webs. For virtually all hot-rolled beams and welded beams of similar proportions in which the web slenderness ratio \((b/t)\) is less than 260, the vertical shear load is resisted by pure beam shear, and lateral buckling does not occur. This is true for a level of loading well above that at which unacceptable deflections will develop. Thus, the design is configured to maintain the shear stress on the gross area of

---

**Figure 8—Approximate Tensile Forces (\(F_1\) and \(F_2\)) on a Section of the Lower Flange of a Loaded Beam**

Key:
- \(F_1\) = Tensile force due to bending moment \(M_1\), lbf (N)
- \(M_1\) = Bending moment at location \(M_1\), kips in. (kN mm)
- \(d\) = One half of the beam depth, in. (mm)
- \(F_2\) = Tensile force due to bending moment \(M_2\), lbf (N)
- \(M_2\) = Bending moment at location \(M_2\), kips in. (kN mm)

Source: Adapted from The Lincoln Electric Company, 1995, Procedure Handbook of Arc Welding, 13th ed., Cleveland: The Lincoln Electric Company, Figure 2-10.
DESIGN FOR WELDING

the web below the nominal shear strength of 0.6 \( \sigma_y \) by a factor of safety or a resistance factor to prevent yielding in shear.

In plate girders with slender webs, shear is resisted by plane beam shear up to a level of stress that will cause shear buckling. However, webs subject to shear stress that are stiffened by transverse stiffeners have considerable postbuckling strength. Current design specifications take this strength into account. After buckling occurs, the web resists larger shear loads with a combination of beam shear and diagonal tension in the panels between stiffeners. If the length-to-depth ratio of the panel is approximately 3 or more, the direction of the diagonal tension becomes too near to the horizontal for it to be effective in providing significant postbuckling strength, and the shear strength is limited to beam shear strength.

The onset of diagonal compression buckling in a web panel has negligible structural significance. With properly proportioned transverse stiffeners, the web continues to resist higher levels of shear loading up to the point at which diagonal tension yielding occurs.

Several practical considerations independent of maximum strength may govern a design. For architectural reasons, especially in exposed fascia girders, the waviness of the web caused by controlled compression buckles may be deemed unsightly. In plate girders subject to cyclic loading to a level that would initiate web shear buckling, each application of the critical load causes an “oil canning” or “breathing” action of the web panels. This action causes out-of-plane bending stresses at the toes of web-to-flange fillet welds and the stiffener welds. These cyclic stresses eventually initiate fatigue cracking. For these cases, web stresses should be limited.

---

Equations:

- **Reaction force \((R)\):** \( R = \frac{V_{\text{max}}}{2} \) (1)
- **Shear force \((V_x)\):** \( V_x = wL - x \) (2)
- **Maximum moment, \(M\), at center \((M_{\text{max}})\):** \( M_{\text{max}} = \frac{wL^2}{8} \) (3)
- **Moment at any point \((M_x)\):** \( M_x = \frac{wx}{2}(L - x) \) (4)
- **Deflection, \(\Delta\), at center \((\Delta_{\text{max}})\):** \( \Delta_{\text{max}} = \frac{5wl^4}{384EI} \) (5)
- **Deflection at any point \((\Delta_x)\):** \( \Delta_x = \frac{wx}{24EI}(L^2 - 2Lx^2 + x^3) \) (6)
- **Slope at the ends of the beam \((\theta)\):** \( \theta = \frac{wl^3}{24EI} \) (7)

**Key:**
- \( w \) = Equally distributed force on beam, kips (kN)
- \( R \) = End reaction, kips (kN)
- \( x \) = Distance from the reaction to a specific location, in. (mm)
- \( L \) = Span length, in. (mm)
- \( V \) = Shear force, kips (kN)
- \( M_{\text{max}} \) = Maximum moment, kips in. (kN mm)

---


**Figure 9—Equations for a Simply Supported Beam with a Uniformly Distributed Load on the Beam Span**

---

DESIGN FOR WELDING

In the case of a beam fabricated by welding, the shear load per unit of length on the welds joining the flanges of the beam to the web can be calculated by means of the following equation:

\[ \nu_s = \frac{V a y}{ln} \]  

(6)

where

- \( \nu_s \) = Shear load per unit length of weld, kip/in. (kN/mm);
- \( V \) = External shear force on the member at this location, kip (kN);
- \( a \) = Cross-sectional area of the flange, in.\(^2\) (mm\(^2\));
- \( y \) = Distance between the center of gravity of the flange and the neutral axis of bending of the whole section, in. (mm);
- \( I \) = Moment of inertia of the whole section about the neutral axis of bending, in.\(^4\) (mm\(^4\)); and
- \( n \) = Number of welds used to attach the web to the flange.

The required weld size can then be determined from the value of \( \nu_s \).

**Torsion.** Torsion creates greater design challenges for bases and frames than for other machine members. A machine with a rotating unit may subject the base to torsional loading. This is evidenced by the lifting of one corner of the base if the base is not anchored to the floor.

If torsion is a problem, closed tubular sections or diagonal bracing should be used, as shown in Figure 11. Closed tubular sections are as much as 1000 times better at resisting torsion than comparable open sections. Closed members can easily be made from channel or I-sections by intermittently welding flat plates to the toes of the rolled sections. The torsion effect on the perimeter of an existing frame may be eliminated or the frame may be stiffened for torsion by adding cross bracing. Torsion problems in structures can be avoided by the judicious arrangement of members to transmit loads by direct stresses or bending moments.

**Torsional Resistance.** The torsional resistance of a flat strip or open section (I-beam or channel) is very low. The torsional resistance of a solid rectangular section having a width of several times the thickness may be approximated by the following expression:

\[ R = \frac{b t^4}{3} \]  

(7)

where

- \( R \) = Torsional resistance, in.\(^4\) (mm\(^4\));
- \( b \) = Width of the section, in. (mm); and
- \( t \) = Thickness of the section, in. (mm).

The unit angular twist, \( \theta \), is equal to the total angular twist divided by the length, \( L \), of the member. The total angular twist (rotation) of a member can be estimated by the following equation:

\[ \theta = \frac{TL}{GR} \]  

(8)

DESIGN FOR WELDING

where

\[ \theta = \text{Angle of twist, radians}; \]
\[ T = \text{Torque, kips in. (kN mm)}; \]
\[ L = \text{Length of the member, in. (mm)}; \]
\[ G = \text{Modulus of elasticity in shear, ksi (MPa)}; \]
\[ R = \text{Torsional resistance, in.}^4 (\text{mm}^4). \]

The torsional resistance of an open structural member such as an I-beam or a channel is approximately equal to the sum of the torsional resistances of the individual flat sections into which the member can be divided. This is illustrated in Table 5, which lists the actual and calculated angle of twist of a flat strip and an I-shape made up of three of the flat strips. The applied torque is the same for both sections.

Torsional resistance increases markedly with closed cross sections, such as circular or rectangular tubing. Consequently, the angular twist is greatly reduced because it varies inversely with torsional resistance. The torsional resistance, R, of any closed box shape enclosing only one cell can be estimated by the following procedure using the equation presented in Figure 12. A dotted line is drawn through the mid-thickness around the section, as shown. The area, A, is enclosed by the dot-dash lines. The cross section of the member is divided into convenient lengths, \( L_n \), having thicknesses \( t_n \). The ratios of these individual lengths to their corresponding thicknesses are then determined.

### Table 5

<table>
<thead>
<tr>
<th>Angle of Twist, Degrees</th>
<th>Strip *</th>
<th>I-Section †</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculated using torsional resistance (R)</td>
<td>21.8</td>
<td>7.3</td>
</tr>
<tr>
<td>Actual twist</td>
<td>22</td>
<td>9.5</td>
</tr>
</tbody>
</table>

* 0.055 in. by 2 in. (1.4 mm by 50.8 mm).
† Made of three of the strips.


**Figure 11**—Application of (A) Closed Tubular Sections or (B) Open Structures with Diagonal Bracing to Resist Torsion

\[
R = \frac{4A^2}{\sum (L_n/t_n)} = \frac{4L_1^2L_2^2}{2(L_1/t_1 + (L_2/t_2 + (L_2/t_2))}
\]

**Figure 12**—Torsional Resistance, \( R \), of a Closed Box Section

**Key:**
- \( R \) = Torsional resistance, in.\(^4\) (mm\(^4\))
- \( A \) = Enclosed area, in.\(^2\) (mm\(^2\))
- \( L_n \) = Distance of each side, in. (mm)
- \( t_n \) = Thickness of corresponding side, in. (mm)
- \( L_1 \) = Distance of Side 1, in. (mm)
- \( t_1 \) = Thickness of Side 1, in. (mm)
- \( L_2 \) = Distance of Side 2, in. (mm)
- \( t_2 \) = Thickness of Side 2, in. (mm)
The torsional resistance can then be obtained using the following equation:

\[
R = \frac{4A^2}{\sum \left(\frac{L_n}{t_n}\right)}
\]

where

\[
R = \text{Torsional resistance, in.}^4 (\text{mm}^4);
A = \text{Enclosed area, in.}^2 (\text{mm}^2);
L_n = \text{Distance of each side, in. (mm); and}
t_n = \text{Thickness of corresponding side, in. (mm)}.
\]

The maximum shear stress in a rectangular section under torsion occurs on the surface at the center of the long side. When the unit angular twist is known, the following expression is used to find the maximum shear stress at the surface of a rectangular part:

\[
\tau = \theta' G = \frac{Tt}{R}
\]

where

\[
\tau = \text{Maximum shear stress, ksi (MPa)};
\theta' = \text{Unit angular twist, radians/in. (radians/mm)};
G = \text{Modulus of elasticity in shear, ksi (MPa)};
T = \text{Applied torque, kips in. (kN mm)};
t = \text{Thickness of the section, in. (mm); and}
R = \text{Torsional resistance, in.}^4 (\text{mm}^4).
\]

This equation can be applied to a flat plate or a rectangular area of an open structural shape (channel, angle, I-beam). In the latter case, \(R\) denotes the torsional resistance of the whole structural shape.

**Diagonal Bracing.** Diagonal bracing is very effective in preventing the twisting of frames. A simple explanation of the effectiveness of diagonal bracing involves an understanding of the directions of the forces involved.

A flat bar of steel has little resistance to twisting, but has exceptional resistance of bending (stiffness) about its major axis. Transverse bars or open sections at 90° to the main members are not effective for increasing the torsional resistance of a frame because, as shown in Figure 13(A), they contribute only relatively low torsional resistance. However, if the bars are oriented diagonally at 45° across the frame, as in Figure 13(B), the twisting of the frame is resisted by the stiffness of the bars. To be effective, the diagonal braces must have good bending stiffness perpendicular to the plane of the frame.

**TRANSFER OF FORCES**

Loads create forces that must be transmitted through the structure to suitable places for counteraction. Designers must therefore know how to provide efficient pathways. One of the basic rules of welding design is that a force applied transversely to a member ultimately enters the portion of the section that lies parallel to the applied force. An example is a lug welded parallel to the length of a beam, as depicted in Figure 14. The portion of the beam that is parallel to the applied force, \(F\), is the web. The force in the lug is easily transferred through the connecting welds into the web. No additional stiffeners or attaching plates are required.

**Figure 13—Frames Subjected to Torsion with (A) Transverse Rib Bracing and (B) Diagonal Bracing**

**Figure 14—Lug Welded Parallel to the Length of a Beam; No Additional Stiffeners Are Required**
Suppose, however, that the lug were welded to the beam flange at right angles to the length of the beam, as shown in Figure 15(A). The outer edges of the flange tend to deflect rather than support much load. This forces a small portion of the weld in line with the web to carry a disproportionate share of the load, as can be observed in Figure 15(B).

To distribute the load on the attachment weld uniformly, two stiffeners can be aligned with the lug and then welded to the web and to the adjacent flange of the beam. This procedure is illustrated in Figure 16. The stiffeners reinforce the bottom flange and transmit part of the load to the web of the beam. The welds labeled “B” and “C” in Figure 16 must be designed to carry the portion of applied force, \( F \), that is not directly transferred to the web by Weld “A.”

If a force is to be applied to a beam parallel to the flanges by a plate welded to the web, as indicated in Figure 17, unacceptable distortion may result. The required weld (or web) strength must be estimated by means of a yield-line analysis of the web.

On the other hand, orienting the attachment plate transverse to the web and welding it to the web only result in a low-strength connection because of the high concentrations of stress at the ends of the weld. This is depicted in Figure 18.

**Figure 15**—Transfer of Force from a Welded Lug to a Beam: (A) Lug Welded to a Flange Transverse to the Beam Length; (B) Resulting Loading and Deflection of the Flange

**Figure 16**—Additional Stiffeners Required to Transmit a Load to the Web of the Beam

**Figure 17**—Distortion of a Beam Caused by a Loaded Attachment to the Web
Similarly, if the attachment is welded to both flanges and the web without a stiffener on the opposite side of the web, the weld must not be sized on the assumption of uniform stress along the total length of weld. Only negligible loads are transferred across most of the width of the web, as shown in Figure 19. Most of the load is transmitted to the two flanges through the attachment welds by shear.

If an attachment is made by welding to the flanges only, the problem is not the distribution of stress in the weld but rather the effects of shear lag in the attachment. This would cause high stress concentrations in the plate near the flange edges, as illustrated in Figure 20. The plate should be designed on the basis of reduced allowable stresses or reduced effective area. An exception would be if the length of each weld along the edges of the plate were more than the width of the plate, as required by the AISC specifications.\textsuperscript{31}

For large forces, it might be necessary to place a stiffener on the opposite side of the web, as illustrated in Figure 21. In this case, both the plate and the stiffener must be welded to the web as well as to the

flanges. With this detail, the length of welds parallel to the direction of applied stress is effectively doubled, and the shear lag effect in the attachment is greatly reduced.

In all cases, the fillet welds joining the plate to the beam flanges must not extend around the plate along the edges of the flanges. Abrupt changes in weld direction on two planes can intensify stress concentrations.

When a force in a structure changes direction, a force component is involved. This is illustrated in Figure 22, which depicts the knee of a rigid frame subjected to a bending moment. The compressive force in the interior flanges must change direction at the knee. To transfer this component, diagonal stiffeners are placed on both sides of the web at the intersections of the two flanges, as shown in Figure 22(A). An alternate detail using vertical and horizontal stiffeners to accomplish the same effect is shown in Figure 22(B). The compressive force component in the web and stiffeners balances the change in the direction of the tensile force in the outer flanges.

DESIGN OF WELDED JOINTS

The loads in a welded structure are transferred from one member to another through the welds placed in the joints. The various types of joints used in welded construction and the applicable welds are shown in Figure 23.

The configurations of the various welds are illustrated in Figures 24, 25, and 26. Combinations of welds may be used to connect a joint, depending upon the strength requirements and loading conditions. For example, fillet and groove welds are frequently combined in corner and T-joints.
Figure 23—Types of Joints

Figure 24—Single-Groove Welds

(A) Single-Square-Groove Weld

(B) Single-Bevel-Groove Weld

(C) Single-V-Groove Weld

(D) Single-V-Groove Weld with Backing

(E) Single-V-Groove Weld on a Surface

(F) Single-J-Groove Weld

(G) Single-U-Groove Weld


Figure 24 (Continued)—Single-Groove Welds
Welded joints are designed primarily to meet the strength and performance requirements for the service conditions under which they must perform. The manner in which the stress will be applied in service—whether in tension, compression, shear, bending, torsion, or a combination of these—must be considered. When designing for fatigue, different joint details may be required. Joints should be designed to avoid stress raisers and minimize residual stresses. Conditions of corrosion or erosion require joints that are free of irregularities, crevices, and other areas that make them susceptible to such forms of attack.

Certain welding processes, in conjunction with certain related types of joints, have repeatedly provided satisfactory performance. Therefore, these processes and joints are given prequalified status, provided the weld procedures meet other specific requirements of Structural Welding Code—Steel, AWS D1.1.32 However, the use of a prequalified welding process or procedure and joint geometry on a particular design or

Figure 25—Double-Groove Welds

(A) Double-Square-Groove Weld

(B) Double-Bevel-Groove Weld

(C) Double-V-Groove Weld

(D) Double-J-Groove Weld with Backing

(E) Double-U-Groove Weld

(F) Double-Flare Bevel-Groove Weld

(G) Double-Flare-V-Groove Weld


Figure 25 (Continued)—Double-Groove Welds
application does not guarantee satisfactory results. The designer must also consider the following questions:

1. Is the joint accessible to welders and inspectors;
2. Does the design consider the economic factors of welding;
3. Will shrinkage stresses lead to excessive joint or member distortion; and
4. Will shrinkage stresses cause lamellar tearing, cracking of the heat-affected zone, or other material problems?

**GROOVE WELDS**

The selection of groove weld type and configuration is influenced by accessibility, economy, the particular design of the structure being fabricated, distortion control, and the type of welding process to be used.

Square-groove welds, shown in Figures 24(A) and 25(A), are economical, provided satisfactory soundness and strength can be obtained. However, their use is limited to relatively thin material. For thick joints, the edge of one or more members must be prepared to a particular geometry to provide accessibility for welding and ensure the desired soundness and strength.

In the interest of economy, joint designs should be selected with root openings and groove angles that require the smallest amount of weld metal while providing sufficient accessibility to achieve sound welds. The selection of a root opening and groove angle is also greatly influenced by the metals to be joined, the location of the joint in the weldment, distortion and shrinkage control, and the performance required.

Welds in J- and U-groove joints may be used to minimize the amount of weld metal required when the savings are sufficient to justify the more costly preparation of the edges, particularly in the welding of thick sections. The narrower groove angle used in J- and U-groove welds, which is possible because of the wider, rounded root, also reduces angular distortion. Bevel- and J-groove welds are more difficult to weld than V- and U-groove welds because one edge of the groove is perpendicular to the surface of the workpiece, requiring the electrode to be angled obliquely into the groove toward the vertical face. For some processes, single-bevel groove welds are prequalified only in the flat position.

The amount of joint penetration, or weld size, and the strength of the filler metal determine the strength of the welded joint. Welded joints must be designed to provide sufficient strength to transfer the design forces as well as ensure proper performance under cyclic or other severe loads, when applicable. This frequently requires that the welded joint provide strength equal to that of the base metal. To accomplish this, designs that require complete penetration through the members being joined are commonly used.

The selection of the details of welding grooves—the groove angle, root face, root opening, and so forth—depends upon the welding process and procedure to be used and the physical properties of the base metals being joined. Some welding processes characteristically provide deeper joint penetration than others. Some metals, such as copper and aluminum, have relatively high thermal conductivities. These metals require greater heat input for welding than other metals with lower thermal properties.

The various types of groove welds have certain advantages and limitations with respect to their applications. In the following discussion, comments on design, joint penetration, and effective throat apply to the joining of carbon steel by shielded metal arc (SMAW), gas metal arc (GMAW), flux cored arc (FCAW), and submerged arc welding (SAW). Joint
penetration with base metals other than carbon steel may vary due to the properties of the base metals.

**Complete Joint Penetration Groove Welds**

Groove welds with complete joint penetration are suitable for all types of loading, provided they meet the acceptance criteria for the application. In most cases, to ensure complete joint penetration with double-groove and single-groove welds without a backing bar, the root of the first weld must be backgouged to sound metal before making a weld pass on the other side.

When properly made using filler metals that match the strength of the base metal, complete penetration groove welds develop the strength of the base metal. The allowable stress range for complete joint penetration groove welds in cyclic applications depends upon the joint detail, the direction of stress, the finishing of the weld, and the testing performed.

**Partial Joint Penetration Groove Welds**

Partial joint penetration groove welds have an unwelded portion at the root of the weld. This unwelded portion constitutes a stress raiser, having significance when cyclic loads are applied transversely to the joint. This is reflected in the low allowable fatigue stress range that characterizes these welds. However, when the load is applied parallel to the weld axis, a higher stress range is permitted.

With single-sided partial penetration groove welds, the eccentricity of shrinkage forces in relation to the center of gravity of the section can result in angular distortion upon cooling after welding. This same eccentricity also tends to cause the rotation of a transverse axial load across the joint. This type of rotation must be minimized both during fabrication and in service.

For static loading, the allowable stresses in partial joint penetration groove welds depend upon the type and direction of loading and the applicable code requirements. Under *Structural Welding Code—Steel*, AWS D1.1:2000, the allowable tensile stress in the weld when loaded in tension transverse or normal to the axis of the weld is 0.30 times the nominal tensile strength of the filler metal. However, the allowable tensile stress in the base metal may not exceed 0.60 times the yield strength of the base metal.\(^{33}\)

The allowable shear stress of the weld metal may not exceed 0.30 times the nominal tensile strength of the filler metal. However, the allowable shear stress on the throat of the weld metal may not exceed 0.40 times the yield strength of the base metal.

Joints welded from one side should not be used in bending with the root in tension, nor should they be subjected to transverse fatigue or impact loading. When loaded transverse to the weld axis, partial joint penetration groove welds are recommended only for static loads. When loaded parallel to the weld axis, they may be used in both static and cyclic applications. Single-sided partial penetration joints should not be exposed to corrosive conditions at the root.

For design purposes, the effective throat is never greater than that of the depth of joint preparation. It may be less when the groove angle is small and the process and weld position used have insufficient penetration to consistently extend to the root of the joint.

**V-GROOVE WELDS**

The strength of a V-groove weld, as with all groove welds, depends upon the extent of joint penetration. For all types of loading, full-strength joints can be obtained with complete joint penetration. V-groove welds are generally considered economical when the depth of preparation of the groove does not exceed approximately 3/4 in. (19 mm). When the depth of joint preparation exceeds this amount, J- or U-groove weld details may be more economical.

**BEVEL-GROOVE WELDS**

Single-bevel-groove welds have characteristics similar to V-groove welds with respect to properties and applications. The bevel type requires less joint preparation and weld metal; therefore, it is generally more economical. However, the disadvantage of this type of weld is that the technique required to obtain complete fusion with the perpendicular face of the joint is more challenging. In addition, satisfactory backgouging of the root of the first weld pass may be harder to accomplish. In the horizontal position, the unbeveled face should be placed on the lower side of the joint to obtain good fusion.

The design for a double-bevel joint is economical when the depth of the groove does not exceed about 3/4 in. (19 mm) and the joint thickness is 1-1/2 in. (38 mm) or less. For thicker sections or deeper grooves, a double-J joint design may be more economical.

**U- AND J-GROOVE WELDS**

U-groove welds and J-groove welds are used for similar applications. However, with U-grooves, complete fusion is easier to obtain. J-groove welds have the same characteristics as similar bevel-groove welds. However, they may be more economical for thicker sections, pro-

vided the savings in deposited weld metal exceeds the cost of machining or gouging the edge preparation. Their use may be best suited to the horizontal position in some applications, with the unprepared edge on the lower surface.

**FILLET WELDS**

Design permitting, fillet welds may be used in preference to groove welds for economy. Fillet-welded joints are very simple to prepare from the standpoint of edge preparation and fitup, although groove-welded joints sometimes require less welding. If the load requires a fillet weld of approximately 5/8 in. (16 mm) or larger, a groove weld should be considered alone or in combination with a fillet weld to provide the required effective throat. In this case, the reduction in welding costs may be sufficient to offset the cost of joint preparation. When the smallest practicable continuous fillet weld results in a joint strength greater than that required, intermittent fillet welding may be used to avoid overwelding unless continuous welding is required by the service conditions.

As shown in Figure 27, the size of a fillet weld is measured by the length of the legs of the largest right triangle that may be inscribed within the cross section of the weld. The effective throat, which is a better indication of weld shear strength, is the shortest distance between the root of the weld and the weld face. The effective area of a fillet weld is based upon the effective throat and the length of the weld. The strength is determined by the effective area and the nominal tensile strength of the filler metal. The actual throat may be larger than the theoretical throat by virtue of joint penetration beyond the root of the weld. Submerged arc and flux cored arc welding are particularly deep penetrating processes. Under certain conditions, several standards allow consideration of the extra penetration that these processes provide as part of the effective throat in fillet welds.

**Applications**

As fillet welds are economical, they are used to join corner, T-, and lap joints. Edge preparation is not required, though surface cleaning may be needed. Fillet welds are generally applicable for the transfer of shear forces parallel to the axis of the weld as well as the transfer of static forces transverse to the axis of the weld. Fillet welds may be used in skewed T- or corner joints having a dihedral angle between 60° and 135°. Below 60°, these welds may be used but are considered partial-joint penetration groove welds. In this case, a Z-loss factor should be used.

Fillet welds are always designed on the basis of shear stress on the throat, regardless of the direction of applied force relative to the axis of the weld. The maximum shear stress is calculated based on the effective area of the weld. In the case of steel, the maximum shear stress is normally limited to 30% of the nominal (classification) tensile strength of the filler metal.

**Weld Size**

Fillet welds must be large enough to carry the applied load and avoid cracking by accommodating the shrinkage of the weld metal during cooling, particularly with highly restrained thick sections. To minimize distortion and welding costs, however, the specified size of the fillet weld should not be excessive. Welds in lap joints must not exceed the thickness of the exposed edge, which should be visible after welding.

Fillet welds may be designed with unequal leg sizes to provide the required effective throat or the needed heat balance for complete fusion with unequal base metal thicknesses.

**Single and Double Fillet Welds**

Fillet welds used on one side of the joint only are limited in application. Figure 28 presents examples of prohibited applications of the one-sided fillet weld. As shown in this figure, bending moments that result in tension stresses in the root of a fillet weld should be avoided because of the notch condition. For this reason, one-sided fillet welds should not be used with lap joints that can rotate about the longitudinal axis under load, nor should they be subjected to impact loads.

When access permits, smaller fillet welds on each side of the joint are preferable to one large single fillet weld. Full-plate strength can often be obtained economically with single fillet welds under static loading. The double fillet welding of corner and T-joints limits the rotation of the members about the longitudinal axis of the joint and minimizes tension stresses at the root of the welds. These types of joints can be cyclically loaded parallel to the weld axes.

Lap joints should have a minimum overlap of about five times the thickness of the base metal to limit joint rotation under load.


Figure 27—Fillet Weld Sizes: (A) Convex Fillet and (B) Concave Fillet

Weldment designers must frequently decide whether to use fillet or groove welds in a design. Cost and performance are the major considerations. Fillet welds, like those shown in Figure 29(A), are easy to apply and require no special edge preparation. They can be made using large-diameter electrodes with high welding currents for high deposition rates. In comparison, double-bevel-groove welds, like those shown in Figure 29(B), have less cross-sectional area than that of fillet welds. However, double-bevel-groove welds require edge preparation and the use of small-diameter electrodes to make the root pass.

**Figure 28—Prohibited Applications of the One-Sided Fillet Weld**

**Figure 29—Comparison of Weld Quantities for Three Conditions of T-Joints**

![Diagram of welds](image)

- **A** Double-Fillet Weld
  - AREA = 0.56 \( t^2 \)
  - Assume \( S = 0.75t \)
  - Where \( S \) = Fillet Weld Size
  - And \( t \) = Plate Thickness

- **B** Double-Bevel-Groove Weld
  - AREA = 0.25 \( t^2 \)

- **C** Single-Bevel-Groove Weld
  - AREA = 0.50 \( t^2 \)
Single-bevel-groove welds, such as that depicted in Figure 29(C), require approximately the same amount of weld metal as single fillet welds like those shown in Figure 29(A). Thus, the single-bevel-groove offers no apparent economic advantage. Disadvantages include the required edge preparation and a low-deposition root pass. In addition, groove welds usually require nondestructive examination (NDE), which increases the total cost compared to the costs associated with visual inspection of the welds. From a performance standpoint, however, groove welds provide for the direct transfer of force through the joint.

For thick plates, the cost of a double fillet welded joint may exceed that of a single-bevel-groove weld. In addition, if the joint can be positioned so that the groove weld can be made in the flat position, a single fillet weld would require more weld metal than a single-bevel-groove weld.

The use of estimating curves, which is based upon the actual cost of joint preparation, positioning, and welding, is a technique that is used to determine the plate thickness for which the various types of joint details are most economical. A sample set of curves is illustrated in Figure 30. The intersection of the fillet weld curve with the groove weld curves is a point of interest. The validity of the information is dependent on the accuracy of the cost data at a particular fabricating plant.

The combined double-bevel-groove and fillet weld joint, illustrated in Figure 31, is theoretically a full-strength weld prepared for a submerged arc weld. The plate edge is beveled to 60° on both sides to a depth of 30% of the thickness of the plate. After the groove on each side is welded, it is reinforced with a fillet weld of equal area and shape. The total effective throat of the weld is equal to the plate thickness. This partial joint penetration groove weld has only about 60% of the weld metal of a full-strength single fillet weld. It requires joint preparation, but the wide root face and groove angle permit the use of large electrodes and high welding currents.

Full-strength welds are not always required in a weldment, and increased economic benefit can often be achieved by using smaller welds where applicable and permissible. Figure 32 presents examples of the manner in which cost savings can be obtained by modifying the joint details. With equal effective throats (shear area), fillet welds such as that shown in Figure 32(A) require twice the weld metal needed for the 45° partial joint penetration single-bevel-groove weld, such as that depicted in Figure 32(B). The latter weld may not be as economical as the fillet weld, however, because of the cost of edge preparation. In addition, some welding standards limit the effective throat of this type of weld to less than the depth of the bevel with certain welding processes and positions because of the possibility of incomplete root penetration.
If a single-bevel-groove weld is combined with a 45° fillet weld, as shown in Figure 32(C), the cross-sectional area for the same effective throat is also approximately 50% of the area of the fillet weld in Figure 32(A). Here, the bevel depth is shallower than that in the single-bevel-groove weld shown in Figure 30(B). A similar weld with a 60° groove angle and an unequal leg fillet but with the same effective throat is depicted in Figure 32(D). This weld also requires less weld metal than a fillet weld alone. This joint (with a 60° bevel angle) allows the use of higher welding current and larger electrodes to obtain deep root penetration.

The desired effective throat of a combined groove and fillet weld can be obtained by adjusting the groove dimensions and the leg lengths of the fillet weld. However, consideration must be given to the accessibility of the root of the joint for welding and the potential stress concentration at the fillet weld toe. When a partial joint penetration groove weld is reinforced with a fillet weld, the minimum effective throat is used for design purposes. The effective throat of the combined welds is not the sum of the effective throat of each weld. The combination is treated as a single weld when determining the effective throat.

**CORNER JOINTS**

Corner joints are widely used in machine design and box members. Typical corner designs are illustrated in Figure 33. The corner-to-corner joint, which is illustrated in Figure 33(A), is difficult to position. Thus, it usually requires fixturing. Small electrodes with low welding currents must be used for the first weld pass to avoid excessive melt-through. This joint also requires a large amount of weld metal.

On the other hand, the corner joint shown in Figure 33(B) is easy to assemble, needs no backing, and utilizes only about one-half the weld metal required to make the joint shown in Figure 33(A). However, this joint has lower strength because the effective throat of the weld is smaller. As illustrated in Figure 33(C), two fillet welds, one outside and the other inside, can provide the same total effective throat as with the first design but with one-half the weld metal. However, joint accessibility may be a problem.

For thick sections, a partial joint penetration, the single-V groove weld, which is shown Figure 33(D), is often used. This weld requires joint preparation. For deeper joint penetration, a J-groove, depicted in Figure 33(E), or a U-groove may be used in preference to a bevel groove. A fillet on the inside corner, such as that shown in Figure 33(F), makes a neat and economical corner. This inside fillet weld can be used alone or in combination with any of the outside corner joint configurations shown in Figure 33. Joint accessibility may also be an issue for the inside weld.

The size of the weld should always be designed taking into consideration the thickness of the thinner member inasmuch as the joint is only as strong as the thinner member. In this way, the weld metal requirements are minimized, resulting in lower costs.

Lamellar tearing at the exposed edges of corner joints in thick steel plates must always be considered during the design phase. Weld joint designs that significantly reduce the through-thickness shrinkage stresses are shown in Figures 33(D), (F), and (G). These joint designs exhibit a lower tendency to lamellar tearing than that shown in Figure 33(E), in which edge preparation is limited to the base metal not stressed in the through-thickness direction.
SIZING OF STEEL WELDS

Welds are sized for their ability to withstand static or cyclic loading in accordance with Structural Welding Code—Steel, AWS D1.1, to ensure that a soundly welded joint is able to support the applied load for the expected service life. The design strengths of welds for various types of static loading are normally specified in the applicable standard for the job. These are usually based upon a percentage of the tensile or yield strength of the filler or base metal. Similarly, the allowable stress range for cyclic loading is normally specified in the applicable standard for the job.

The following material covers both load and resistance factor design (LRFD) and allowable stress design (ASD). When using LRFD, the loads and corresponding stresses must be calculated using the load factors and combinations presented in Section 2.3 in Minimum Design Loads for Buildings and Other Structures, ASCE 7-98. When using ASD, the loads and corresponding stresses must be calculated using the load factors and combinations in Section 2.4 of ASCE 7-98. When the job is one for which this standard is not applicable, the loads and load combinations used should be approximate and consistent with the intent of the standard.

Complete joint penetration groove welds, illustrated in Figure 34(A), (B), (C), and (D), are considered full-strength welds because they are capable of transferring the full strength of the connected elements. The design strengths in such welds are the same as those in the base metal, provided matching strength weld metal is used. In complete joint penetration groove welds, the mechanical properties of the selected filler metal must at least match those of the base metal. If two base metals of different strengths are welded together, the selected filler metal strength must match or exceed the strength of the weaker base metal.

Partial joint penetration groove welds, illustrated in Figure 34(B), (C), (E), and (F), are widely used for the economical welding of thick sections. These welds not only lead to savings in weld metal and welding time, but they also can provide the required joint strength. The minimum weld sizes for prequalified partial joint penetration groove welds are shown in Table 8. To avoid cracking in the weld or the heat-affected zone, the minimum weld size should provide adequate process heat input to counteract the quenching effect of the base metal.

Figure 33—Typical Corner Joint Designs

Source: Adapted from The Lincoln Electric Company, 1995, Procedure Handbook of Arc Welding, 13th ed., Cleveland: The Lincoln Electric Company, Figure 2-94.

36. See Reference 32.
38. See Reference 37.

STATIC LOADING

Examples of design strengths for static loading conditions for steel welds using the AISC load and resistance factor design (LRFD) procedures are presented in Table 6. Table 7 presents the allowable stresses for static loading which are used when designing with the conventional applied stress design (ASD) procedure. The various types of loading for the welds listed in Table 6 and Table 7 are illustrated in Figure 34.

Complete joint penetration groove welds, illustrated in Figure 34(A), (B), (C), and (D), are considered full-strength welds because they are capable of transferring the full strength of the connected elements.

The design strengths in such welds are the same as those in the base metal, provided matching strength weld metal is used. In complete joint penetration groove welds, the mechanical properties of the selected filler metal must at least match those of the base metal. If two base metals of different strengths are welded together, the selected filler metal strength must match or exceed the strength of the weaker base metal.

Partial joint penetration groove welds, illustrated in Figure 34(B), (C), (E), and (F), are widely used for the economical welding of thick sections. These welds not only lead to savings in weld metal and welding time, but they also can provide the required joint strength. The minimum weld sizes for prequalified partial joint penetration groove welds are shown in Table 8. To avoid cracking in the weld or the heat-affected zone, the minimum weld size should provide adequate process heat input to counteract the quenching effect of the base metal.
### Table 6
**Design Strength for Steel Welds Using AISC Load and Resistance Factor Design (LRFD)**

<table>
<thead>
<tr>
<th>Type of Weld</th>
<th>Stress in the Weld *</th>
<th>Design Strength</th>
<th>Required Filler Metal Strength Level</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Complete joint penetration groove welds</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension normal to the effective area</td>
<td>Same as the base metal</td>
<td></td>
<td>Matching filler metal must be used</td>
</tr>
<tr>
<td>Compression normal to the effective area</td>
<td>Same as the base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension or compression parallel to the axis of the weld</td>
<td>Same as the base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear on the effective area</td>
<td>0.48 times the nominal tensile, except shear stress on the base metal shall not exceed 0.54 times the yield strength of the base metal</td>
<td></td>
<td>Filler metal with a strength level equal to or one classification (10 ksi [69 MPa]) less than matching filler metal may be used</td>
</tr>
<tr>
<td><strong>Partial joint penetration groove welds</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compression normal to the effective area</td>
<td>0.75 times the nominal strength † of the filler metal; stress on the base metal shall not exceed 0.90 times the yield strength of the base metal</td>
<td>Joint not designed † to bear in compression</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Joint designed † to bear in compression</td>
<td>Same as base metal</td>
<td></td>
</tr>
<tr>
<td>Tension or compression parallel to the axis of the weld</td>
<td>Same as the base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear parallel to the axis of the weld</td>
<td>0.45 times the nominal strength of the filler metal, except the tensile stress on the base metal shall not exceed 0.54 times the yield strength of the base metal</td>
<td></td>
<td>Filler metal with a strength level equal to or less than a matching filler metal may be used</td>
</tr>
<tr>
<td>Tension normal to the effective area</td>
<td>0.48 times the nominal tensile strength of the filler metal, except the tensile strength shall not exceed 0.90 times the yield strength of the base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Fillet welds</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear on the effective area</td>
<td>0.45 times nominal tensile strength of the filler metal, except the shear stress on the base metal shall not exceed 0.54 times the yield strength of the base metal</td>
<td></td>
<td>Filler metal with a strength level equal to or less than a matching filler metal may be used</td>
</tr>
<tr>
<td>Tension or compression parallel to the axis of the weld</td>
<td>Same as the base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Plug and slot welds</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear parallel to the faying surfaces (on the effective area)</td>
<td>0.45 nominal tensile strength of filler metal, except shear stress shall not exceed 0.54 times the yield strength of the base metal</td>
<td></td>
<td>Filler metal with a strength level equal to or less than a matching filler metal may be used</td>
</tr>
</tbody>
</table>

---

* The effective weld area is the effective weld length multiplied by the effective throat.

† Specifications in American Institute of Steel Construction (AISC), 1994, *Manual of Steel Construction: Load and Resistance Factor Design*, Vols. 1 and 2, Chicago: American Institute of Steel Construction stipulate design strength to be the same as the base metal without distinction as to whether the joint is milled to bear or not based upon the results of full-size column tests.

### Table 7
Allowable Stresses in Nontubular Connection Welds (ASD)

<table>
<thead>
<tr>
<th>Type of Weld</th>
<th>Stress in the Weld*</th>
<th>Allowable Connection Stress†</th>
<th>Required Filler Metal Strength Level‡</th>
</tr>
</thead>
<tbody>
<tr>
<td>Complete joint penetration groove welds</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension normal to the effective area</td>
<td>Same as the base metal</td>
<td>Matching filler metal must be used</td>
<td></td>
</tr>
<tr>
<td>Compression normal to the effective area</td>
<td>Same as the base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension or compression parallel to the axis of the weld</td>
<td>Same as the base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear on the effective area</td>
<td>0.30 times the nominal tensile, except shear stress on the base metal must not exceed 0.40 times the yield strength of the base metal</td>
<td>Filler metal with a strength level equal to or less than matching filler metal may be used</td>
<td></td>
</tr>
<tr>
<td>Partial joint penetration groove welds</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compression normal to the effective area</td>
<td>Joint not designed to bear in compression</td>
<td>0.50 times the nominal strength of the filler metal; stress on the base metal must not exceed 0.60 times the yield strength of the base metal</td>
<td>Filler metal with a strength level equal to or less than matching filler metal may be used</td>
</tr>
<tr>
<td>Joint designed to bear in compression</td>
<td>Same as base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tension or compression parallel to the axis of the weld</td>
<td>Same as base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear parallel to the axis of the weld</td>
<td>0.30 times the nominal tensile strength of the filler metal, except the tensile stress on the base metal must not exceed 0.40 times the yield strength of the base metal</td>
<td>Filler metal with a strength level equal to or less than matching filler metal may be used</td>
<td></td>
</tr>
<tr>
<td>Tension normal to the effective area</td>
<td>0.30 times the nominal tensile strength of the filler metal, except the tensile strength must not exceed 0.60 times the yield strength of the base metal</td>
<td>Filler metal with a strength level equal to or less than matching filler metal may be used</td>
<td></td>
</tr>
<tr>
<td>Fillet welds</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear on the effective area</td>
<td>0.30 times the nominal tensile strength of the filler metal</td>
<td>Filler metal with a strength level equal to or less than matching filler metal may be used</td>
<td></td>
</tr>
<tr>
<td>Tension or compression parallel to the axis of the weld</td>
<td>Same as base metal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plug and slot welds</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear parallel to the faying surfaces (on the effective area)</td>
<td>0.30 nominal tensile strength of filler metal, except shear stress must not exceed 0.40 times the yield strength of the base metal</td>
<td>Filler metal with a strength level equal to or less than a matching filler metal may be used</td>
<td></td>
</tr>
</tbody>
</table>

* The effective area for groove welds is the effective weld length multiplied by the weld size. For fillet welds, the effective area is the effective throat multiplied by the effective length. For plug and spot welds, it is the nominal area of the hole or slot in the plane of the faying surface.
† For cyclic loading, see Table 1.
§ Fillet weld and partial joint penetration groove welds joining the component elements of built-up members, such as web-to-flange connections, may be designed without regard to the tensile or compressive stress in these elements parallel to the axis of the welds.

Figure 34—Examples of Welds with Various Types of Loading

(A) Complete Joint Penetration Groove Weld in Tension

(B) Compression Normal to Axis of Weld

(C) Tension or Compression Parallel to Weld Axis
Figure 34 (Continued)—Examples of Welds with Various Types of Loading

(D) Complete Joint Penetration Groove Weld in Shear

(E) Partial Joint Penetration in Groove Welds

(F) Shear Parallel to Weld Axis

(G) Fillet Welds Loaded in Shear along Weld Throat
Various factors should be considered in determining the design strength of the throat of partial joint penetration groove welds. Joint configuration is one factor. The effective throat of a prequalified partial joint penetration groove weld is the depth of the groove when the groove angle is 60° or greater at the root of the weld. For groove angles of less than 60°, the effective throat depends upon the welding process, the welding position, and the groove angle at the root. The provisions of Structural Welding Code—Steel, AWS D1.1 should be consulted to determine if an allowance for uncertain penetration is required for the conditions of a particular weld.

The LRFD design shear strength for steel weld metal in groove and fillet welds is approximately 45% of the nominal tensile strength of the weld metal \(0.75 \times 0.6\). The minimum specified ultimate tensile strength of the weld metal in ksi (MPa). Table 9 (LRFD) and Table 10 (ASD) present the design strength per inch \(^2\) of weld length for various sizes of steel fillet welds of several strength levels. These values are for equal-leg fillet welds in which the effective throat thickness is 70.7% of the weld size.

For example, the design strength, \(\phi R_n\), of a 1/4 in. (6 mm) fillet weld made with an electrode that deposits weld metal of nominal 70,000 psi (480 MPa) minimum tensile strength is determined using LRFD in the following manner:

\[
\phi R_n = \phi \left(0.6 F_{EXX}\right) A_w
\]

\[
= 0.75 \left[0.6 \times 70,000 \text{ psi (480 MPa)}\right]
\]

\[
[0.707 \times 1/4 \text{ in. (6 mm)}]
\]

\[
= 5570 \text{ lb/in. (975.4 N/mm) of weld (11)}
\]

where

\(\phi\) = LRFD resistance factor;
\(R_n\) = Nominal strength of resistance, lbf (N);
\(F_{EXX}\) = Minimum specified weld metal tensile strength, ksi (MPa); and
\(A_w\) = Area of shear plane in weld (throat dimension \(\times\) length), in.\(^2\) (mm\(^2\)).

Use of the minimum fillet weld size is intended to ensure sufficient heat input to reduce the possibility of cracking in either the heat-affected zone or the weld metal, especially in a restrained joint. The minimum size applies if it is greater than the size required for strength.

The minimum fillet weld sizes for structural welds are shown in Table 11. Where sections of different thickness are being joined, the minimum fillet weld size is governed by the thicker section if a nonlow-hydrogen electrode is used without preheat. However, if a nonlow-hydrogen electrode is used with the preheat provisions of Structural Welding Code—Steel, AWS D1.1 or

---

**Table 8**

<table>
<thead>
<tr>
<th>Thickness of Base Metal*</th>
<th>Maximum Effective Throat†</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in.</td>
</tr>
<tr>
<td>1/8 to 3/16</td>
<td>2 to 5</td>
</tr>
<tr>
<td>Over 3/16 to 1/4</td>
<td>5 to 6</td>
</tr>
<tr>
<td>Over 1/4 to 1/2</td>
<td>6 to 13</td>
</tr>
<tr>
<td>Over 1/2 to 3/4</td>
<td>13 to 19</td>
</tr>
<tr>
<td>Over 3/4 to 1-1/2</td>
<td>19 to 38</td>
</tr>
<tr>
<td>Over 1-1/2 to 2-1/4</td>
<td>38 to 57</td>
</tr>
<tr>
<td>Over 2-1/4 to 6</td>
<td>57 to 150</td>
</tr>
<tr>
<td>Over 6</td>
<td>150</td>
</tr>
</tbody>
</table>

* Thickness of the thicker section with unequal thickness.
† The effective throat should not exceed the thickness of the thinner part.


---

39. See Reference 32.
40. A footnote in these tables provides information for conversion to SI units.

41. See Reference 32.
### Table 9
**LRFD Design Sheer Strength Per Inch of Length of Steel Equal-Leg Fillet Welds**

<table>
<thead>
<tr>
<th>Weld Size</th>
<th>Classification Strength Level of the Filler Metal, ksi</th>
<th>Design Unit Strength, 1000 lb/in.†</th>
</tr>
</thead>
<tbody>
<tr>
<td>in.</td>
<td>mm/1000 lb/in.</td>
<td></td>
</tr>
<tr>
<td>1/16</td>
<td>2</td>
<td>1.19</td>
</tr>
<tr>
<td>1/8</td>
<td>3</td>
<td>2.39</td>
</tr>
<tr>
<td>3/16</td>
<td>5</td>
<td>3.58</td>
</tr>
<tr>
<td>1/4</td>
<td>6</td>
<td>4.77</td>
</tr>
<tr>
<td>5/16</td>
<td>7</td>
<td>5.97</td>
</tr>
<tr>
<td>3/8</td>
<td>8</td>
<td>7.16</td>
</tr>
<tr>
<td>7/16</td>
<td>10</td>
<td>8.35</td>
</tr>
<tr>
<td>1/2</td>
<td>10</td>
<td>8.35</td>
</tr>
<tr>
<td>5/8</td>
<td>11</td>
<td>9.54</td>
</tr>
<tr>
<td>3/4</td>
<td>12</td>
<td>11.9</td>
</tr>
<tr>
<td>7/8</td>
<td>13</td>
<td>14.3</td>
</tr>
<tr>
<td>1</td>
<td>14</td>
<td>19.1</td>
</tr>
</tbody>
</table>

† To convert 1000 lb/in. to meganewton per meter (MN/m), multiply the quantity in the table by 0.175.

### Table 10
**ASD Design Strength Per Inch of Length of Steel Fillet Welds**

<table>
<thead>
<tr>
<th>Weld Size</th>
<th>Classification Strength Level of the Filler Metal, ksi</th>
<th>Design Unit Strength, 1000 lb/in.†</th>
</tr>
</thead>
<tbody>
<tr>
<td>in.</td>
<td>mm/1000 lb/in.</td>
<td></td>
</tr>
<tr>
<td>1/16</td>
<td>2</td>
<td>0.795</td>
</tr>
<tr>
<td>1/8</td>
<td>3</td>
<td>1.59</td>
</tr>
<tr>
<td>3/16</td>
<td>5</td>
<td>2.39</td>
</tr>
<tr>
<td>1/4</td>
<td>6</td>
<td>3.18</td>
</tr>
<tr>
<td>5/16</td>
<td>8</td>
<td>3.98</td>
</tr>
<tr>
<td>3/8</td>
<td>9</td>
<td>4.77</td>
</tr>
<tr>
<td>7/16</td>
<td>10</td>
<td>5.57</td>
</tr>
<tr>
<td>1/2</td>
<td>10</td>
<td>6.36</td>
</tr>
<tr>
<td>5/8</td>
<td>12</td>
<td>7.95</td>
</tr>
<tr>
<td>3/4</td>
<td>13</td>
<td>9.55</td>
</tr>
<tr>
<td>7/8</td>
<td>14</td>
<td>11.1</td>
</tr>
<tr>
<td>1</td>
<td>15</td>
<td>12.7</td>
</tr>
</tbody>
</table>

† To convert 1000 lb/in. to MN/m, multiply the quantity in the table by 0.175.
When metals are subjected to cyclic tensile or alternating tensile-compressive stress, they may fail by fatigue. The performance of a weld under a cyclic load is an important consideration in structures and machinery. Specifications relating to fatigue in steel structures include those developed by the American Institute of Steel Construction (AISC), the American Association of State Highway and Transportation Officials (AASHTO), and the American Railway Engineering and Maintenance-of-Way Association (AREMA). The applicable standards published by these organizations are essentially the same as those presented below. Nonetheless, the latest edition of the appropriate standard should be consulted for specific information.

Although sound weld metal may have about the same fatigue strength as the base metal, any change in cross section at a weld lowers the fatigue strength of the member. In the case of a complete joint penetration groove weld, any reinforcement, undercut, incomplete joint penetration, or cracking acts as a notch or stress raiser. Each of these conditions is detrimental to fatigue life. In addition, the very nature of a fillet weld transverse to the stress field provides an abrupt change in section that may limit fatigue life. The weld heat-affected zone can also act as a stress raiser due to the metallurgical structure.

The fatigue stress provisions in *Structural Welding Code—Steel*, AWS D1.1:2000, are presented in Table 12 and illustrated in Figure 35. Curves for the allowable stress ranges for each stress category are plotted in Figure 36 for redundant structures.

The allowable stress ranges presented in Figure 36 are independent of yield strength; therefore, they apply equally to all structural steels. When fatigue conditions exist, the anticipated cyclically applied loads, the number of cycles, and the desired service life must be given. The designer then selects the materials and details to accommodate the design conditions for each member and situation (see Table 12 and Figure 35). The designer then calculates the maximum stress in each member to ensure that it does not exceed the allowable stress for the static condition. If the calculated stresses under cyclic conditions exceed the allowable stress under static conditions, the member sections must be increased to bring the stresses within the allowable stress.

Partial joint penetration groove welds are not normally used in fatigue applications. However, their response to fatigue stresses is similar to that of fillet welds.

### Table 11

<table>
<thead>
<tr>
<th>Base Metal Thickness (T) *</th>
<th>Minimum Size of Fillet Weld †</th>
</tr>
</thead>
<tbody>
<tr>
<td>in. mm</td>
<td>in. mm</td>
</tr>
<tr>
<td>T ≤ 1/4 T ≤ 6</td>
<td>1/8 ‡</td>
</tr>
<tr>
<td>1/4 &lt; T ≤ 1/2 6 &lt; T ≤ 12</td>
<td>3/16 5</td>
</tr>
<tr>
<td>1/2 &lt; T ≤ 3/4 12 &lt; T ≤ 20</td>
<td>1/4 6</td>
</tr>
<tr>
<td>3/4 &lt; T 20 &lt; T</td>
<td>5/16 8</td>
</tr>
</tbody>
</table>

*For non-low-hydrogen electrodes without preheat calculated in accordance with Section 3.5.2 of American Welding Society (AWS) Committee on Structural Welding, 2000, *Structural Welding Code—Steel*, AWS D1.1:2000, Miami, American Welding Society, T equals the thickness of the thicker part joined; single-pass welds shall be used. For non-low-hydrogen electrodes using procedures established to prevent cracking in accordance with Section 3.5.2 of American Welding Society (AWS) Committee on Structural Welding, 2000, *Structural Welding Code—Steel*, AWS D1.1:2000, Miami, American Welding Society, and for low-hydrogen electrodes, T equals the thickness of the thinner part joined; the single-pass requirement does not apply.

† Except that the weld size need not exceed the thickness of the thinner part joined.

‡ Minimum size for cyclically loaded structures is 3/16 in. (5 mm).


42. See Reference 33.

43. A stress range is the magnitude of the change in stress that occurs with the application or removal of the cyclic load that causes tensile stress or a reversal of stress. Loads that cause only changes in the magnitude of compressive stress do not cause fatigue.

44. A redundant structure provides an alternate load path in the event of failure of a member or members.
### Table 12
Fatigue Stress Provisions—Tension or Reversal Stresses* (Nontubular Members)

<table>
<thead>
<tr>
<th>General Condition</th>
<th>Situation</th>
<th>Stress Category (see Figure 35)</th>
<th>Example (see Figure 35)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain material</td>
<td>Base metal with rolled or cleaned surfaces. Oxygen-cut edges with ANSI smoothness of 1000 or less.</td>
<td>A</td>
<td>1, 2</td>
</tr>
<tr>
<td></td>
<td>Base metal and weld metal in members without attachments, built up of plates or shapes connected by continuous, complete, or partial joint penetration groove welds or by continuous fillet welds parallel to the direction of applied stress.</td>
<td>B</td>
<td>3, 4, 5, 7</td>
</tr>
<tr>
<td>Built-up members</td>
<td>Calculated flexural stress at toe of transverse stiffener welds on girder webs or flanges.</td>
<td>C</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends.</td>
<td>E</td>
<td>7</td>
</tr>
<tr>
<td>Groove welds</td>
<td>Base metal and weld metal at complete joint penetration, groove-welded splices of milled and welded sections having similar profiles when welds are ground(^1) and weld soundness established by nondestructive testing.(^2)</td>
<td>B</td>
<td>8, 9</td>
</tr>
<tr>
<td></td>
<td>Base metal and weld metal in or adjacent to complete joint penetration, groove-welded splices at transitions in width or thickness, with welds ground(^1) to provide slopes no steeper than 1 in. to 2-1/2 in. (25 mm to 64 mm)(^3) for yield strength less than 90 ksi (620 MPa) and a radius(^5) of (R \geq 2) ft (0.6 m) for yield strength (\geq 90) ksi (620 MPa), and weld soundness established by nondestructive examination.(^2)</td>
<td>B</td>
<td>10, 11a, 11b</td>
</tr>
<tr>
<td>Groove-welded</td>
<td>Base metal at details of any length attached by groove welds subjected to transverse or longitudinal loading, or both, when weld soundness transverse to the direction of stress is established by nondestructive testing(^2) and the detail embodies a transition radius, (R), with the weld termination ground(^3) when:</td>
<td>Transverse loading(^4)</td>
<td></td>
</tr>
<tr>
<td>connections</td>
<td>Longitudinal loading</td>
<td>Materials having equal or unequal thickness, sloped(^6) web connections excluded</td>
<td>Materials having equal thickness, not ground; web connections excluded</td>
</tr>
<tr>
<td></td>
<td>(a) (R \geq 24) in. (600 mm)</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>(b) 24 in. (600 mm) &gt; (R &gt; 6) in. (150 mm)</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>(c) 6 in. (150 mm) &gt; (R &gt; 2) in. (50 mm)</td>
<td>D</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>(d) 2 in. (50 mm) &gt; (R &gt; 0)^7</td>
<td>E</td>
<td>E</td>
</tr>
</tbody>
</table>

*Except as noted for fillet and stud welds.

(Continued)
<table>
<thead>
<tr>
<th>General Condition</th>
<th>Situation</th>
<th>Stress Category (see Figure 35)</th>
<th>Example (see Figure 35)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Groove welds</td>
<td>Base metal and weld metal in or adjacent to complete joint penetration groove-welded splices either not requiring transition or when required with transitions having slopes no greater than 1 in. to 2-1/2 in. (25 mm to 64 mm) (^3) for yield strength less than 90 ksi (620 MPa) and a radius (^4) of (R \geq 2) ft (0.6 m) for yield strength (\geq 90) ksi (620 MPa), and when in either case reinforcement is not removed and weld soundness is established by nondestructive testing. (^2)</td>
<td>C</td>
<td>8, 9, 10, 11a, 11b</td>
</tr>
<tr>
<td>Groove- or fillet-welded connections</td>
<td>Base metal at details attached by groove or fillet welds subject to longitudinal loading where the details embody a transition radius, (R), less than 2 in. (^7) (50 mm) and when the detail length, (L), parallel to the line of stress is:</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>((a) &lt; 2) in. (50 mm)</td>
<td>C</td>
<td>12, 14, 15, 16</td>
</tr>
<tr>
<td></td>
<td>((b) 2) in. (50 mm) (\leq L &lt; 4) in. (100 mm)</td>
<td>D</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>((c) L \geq 4) in. (100 mm)</td>
<td>E</td>
<td>12</td>
</tr>
<tr>
<td>Fillet-welded connections</td>
<td>Base metal at details attached by fillet welds parallel to the direction of stress regardless of length when the detail embodies a transition radius, (R), 2 in. (50 mm) or greater and with the weld termination ground. (^1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>((a) \text{When } R \geq 24) in. (600 mm)</td>
<td>B (^5)</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>((b) \text{When } 24) in. (600 mm) (&gt; R \geq 6) in. (150 mm)</td>
<td>C (^5)</td>
<td>13</td>
</tr>
<tr>
<td></td>
<td>((c) \text{When } 6) in. (150 mm) (&gt; R \geq 2) in. (50 mm)</td>
<td>D (^5)</td>
<td>13</td>
</tr>
<tr>
<td>Fillet welds</td>
<td>Shear stress on throat of fillet welds</td>
<td>F</td>
<td>8a</td>
</tr>
<tr>
<td></td>
<td>Base metal at intermittent welds attaching transverse stiffeners and stud-type shear connectors.</td>
<td>C</td>
<td>7, 14</td>
</tr>
<tr>
<td></td>
<td>Base metal at intermittent welds attaching longitudinal stiffeners.</td>
<td>E</td>
<td>—</td>
</tr>
<tr>
<td>Stud welds</td>
<td>Shear stress on nominal shear area of Type-B shear connectors.</td>
<td>F</td>
<td>14</td>
</tr>
<tr>
<td>Plug and slot welds</td>
<td>Base metal adjacent to or connected by plug or slot welds.</td>
<td>E</td>
<td>—</td>
</tr>
</tbody>
</table>

Notes:
4. Applicable only to complete joint penetration groove welds.
5. Shear stress on throat of weld (loading through the weld in any direction) is governed by Category F.
6. Slopes similar to those required by Note 3 are mandatory for categories listed. If slopes are not obtainable, Category E must be used.
7. Radii less than 2 in. (50 mm) need not be ground.
9. Except as noted for fillet and stud welds.

Figure 35—Examples of Various Fatigue Categories

Source: Adapted from American Welding Society (AWS) Committee on Structural Welding, 2000, Structural Welding Code—Steel, AWS D1.1:2000, Miami: American Welding Society, Figure 2.8.

Note: The numbers below each example are referenced in Table 5.11.

Reprinted with the permission of the American Association of State Highway and Transportation Officials.
DESIGN FOR RIGIDITY OR STIFFNESS

In machine design work, the primary design requirement for many members is rigidity. The members having these requirements are often thick sections that provide for the movement under load to be within close tolerances. The resulting stresses in the members are very low. Often, the actual stress in a welded machine base or frame may be on the order of one-tenth or less of the design strength. In these cases, the weld sizes need to be designed for rigidity rather than load conditions.

A very practical method involves designing the weld size to carry one-third to one-half of the strength of the thinner member being joined. In this way, if the base metal is stressed to one-third to one-half of the design strength, the weld will be strong enough to carry the load. Most rigid designs are stressed below these values. It should be noted, however, that a reduction in weld size below one-third of the normal full-strength size might produce a weld that is too small in appearance for general acceptance.

MATCHING WELD METAL

When a full-strength primary weld is required, a filler metal with mechanical properties that match those of the base metal must be selected. Generally, it is unnecessary for the compositions of the weld metal and base metal to be exactly alike. In fact, in many instances, they are dissimilar. For low-alloy chromium-molybdenum and stainless steels as well as for most nonferrous alloys, the compositions of the weld metal are similar to those of the base metals. For materials strengthened by heat treatment, the manufacturer’s recommendations should be followed to avoid the degradation of the mechanical properties by the heat or welding.
When working with high-strength steels, full-strength welds should not be used unless they are required. High-strength steel requires preheat and special welding procedures because of its propensity for weld cracking, especially if the joint is restrained.

Some welds are nonstructural; that is, rather than providing for the direct transfer of forces, they serve an ancillary purpose, such as to hold the parts together to form a built-up member. In most cases, the stresses on these secondary welds are low. Thus, they can be made with weld metal that is lower in strength than base metal. Weld metal with a minimum tensile strength of 70,000 psi to 90,000 psi (480 MPa to 620 MPa) is preferred because the likelihood of cracking is lower than when using matching weld metal. In any case, the weld must be sized to provide a joint of sufficient strength.

A comparison of the behaviors of full-strength and partial-strength welds made in quenched-and-tempered ASTM A514 steel is shown in Figure 37. The full-strength weld is transverse to the tensile load, whereas the partial-strength weld is parallel to the tensile load. As shown in Figure 37(A), the plate, which has a tensile strength of 110,000 psi (760 MPa), is welded with an E11018 covered electrode to produce a full-strength weld. When the stress is parallel to the weld axis, as shown in Figure 37(B), a weld made with an E7018 covered electrode (70,000 psi [480 MPa] minimum tensile strength) is adequate as long as it successfully transmits any shear load from one member to the other.

In full-strength welded joints, both the plate and the weld metal have equivalent strengths. Their behavior under load is shown by the stress-strain curve in Figure 37.
37(A). If a transversely loaded test weld were pulled in tension, it is likely that the failure would take place in the plate because of its slightly lower strength.

In lower-strength weld joints loaded axially, such as that illustrated in Figure 37(B), both the plate and the weld would be strained together. As the member is loaded, the strain increases from 1 to 2 on the stress-strain plot with a corresponding increase in the stress in both the plate and weld from 1 to 3. At this point, the E7018 weld metal has reached its yield strength. Upon further loading, the strain is increased to 4. The weld metal is stressed beyond its yield strength at 5, at which point it flows plastically. However, the plastic deformation is controlled and limited by the base material, which is still elastic. On the other hand, the stress in the plate is still below its yield strength at 6. With further loading, the strain will reach 7, at which point the ductility of the plate will be exhausted. The plate will fail first because the weld metal has greater ductility. The weld will not fail until its unit strain reaches 8.

Figure 37 illustrates the fact that the 70,000 psi (480 MPa) weld has sufficient strength to carry an axial load because it carries only a small portion of the total axial load on the weldment. If a weld is to transmit the total load, it has to be as strong as the base metal.

**SKewed Fillet Welds**

A special condition exists when members come together at an angle other than 90° and fillet welds are to be used to make the connection. Ordinary specifications for the weld leg at some joint angles could result in an excessive waste of weld metal along with difficulty in depositing the weld on the acute side of the joint.

Figure 38 examines the relationships between the dihedral angle, \( \Psi \), the weld size, \( b \), and the effective throat, \( t \), of skewed fillet welds. The accompanying equations are used to determine the proper effective throat for each weld to allow for the deposit of a minimum area, \( A_t \), of weld metal in the joint. Weld sizes \( b_1 \) and \( b_2 \) can be determined for the respective effective throats. The reader is advised to refer to *Structural Welding Code—Steel*, AWS D1.1, for information concerning welds in angles less than 60° for Z-loss factors on the effective throat.

**TREATING A WELD AS A LINE**

For the sake of convenience, when the total length of weld in a connection is large compared to its effective throat, the weld can be assumed to be a line having a definite length and configuration rather than an area. The proper size of weld required for adequate strength can be determined using this concept. The welded connection is considered as a single line having the same outline as the connection area. This is shown in Figure 39, where \( b \) denotes width and \( d \) represents depth. Thus, the welded connection has length, not effective area. In this way, the problem becomes one of determining the force per unit length on the weld instead of the stress on a weld, which cannot be established until the weld size is known.

When the weld is treated as a line, the property of a welded connection can be substituted in the standard design equation used for the particular type of load, as shown in Table 13. The force per unit length on the weld can then be calculated with the appropriate modified equation.

Problems involving bending or twisting loads may be satisfactorily and conservatively handled by treating the unit loads as vectors and adding the vector. The actual strength of welded connections in which the external load does not pass through the shear center of the weld requires the use of a more complex approach. This method recognizes that when an eccentric load is applied, both relative rotation and translation between the welded parts occur. The actual center of rotation is not about the center of gravity of the weld group. Instead, it is about a center that is dependent upon the relative magnitude of the shear and moment reactions, weld geometry, and deformations of obliquely loaded incremental lengths of weld. The geometrical properties of common joint configurations can be determined using the equations shown in Table 14.

For a given connection, two dimensions are needed—the length of the horizontal weld, \( b \), and the length of the vertical weld, \( d \). The section modulus, \( S_w \), is used for welds subjected to bending loads, while the polar moment of inertia of a line weld, \( J_w \), and distance, \( c \), are used for torsional loads. Section moduli are given for the maximum force at the top and bottom or right and left portions of the welded connections. For the unsymmetrical connections shown in Table 13, the maximum bending force is at the bottom of the connection.

If more than one force is applied to the weld, the unit forces are combined vectorially. All unit forces that are combined must be vectored at a common location on the welded joint. Weld size is found by dividing the resulting unit force on the weld by the design strength of the type of weld used. The steps used in applying this method to any welded construction are presented below:

1. Find the position on the welded connection where the combined unit forces are at the maximum. More than one combination deserving consideration may be present;
2. Find the value of each of the unit forces on the welded connection at this position;
3. Compare the calculated forces to the design strength of the type of weld used.
For Each Weld:

\[ t = \frac{\omega}{2 \sin \left( \frac{\psi}{2} \right)} \text{ or } \omega = 2 \ t \tan \left( \frac{\psi}{2} \right) \]

\[ f = \frac{\omega}{\cos \left( \frac{\psi}{2} \right)} = 2 \ t \tan \left( \frac{\psi}{2} \right) \]

\[ b = \frac{t}{\cos \left( \frac{\psi}{2} \right)} \]

\[ A = \frac{\omega^2}{4 \sin \left( \frac{\psi}{2} \right) \cos \left( \frac{\psi}{2} \right)} = t^2 \tan \left( \frac{\psi}{2} \right) \]

If \( b_1 = b_2 \), then for \( t = t_1 + t_2 \):

\[ t_1 = t \frac{\cos \left( \frac{\psi_1}{2} \right)}{\cos \left( \frac{\psi_1}{2} \right) + \cos \left( \frac{\psi_2}{2} \right)} \]

\[ t_2 = t \frac{\cos \left( \frac{\psi_2}{2} \right)}{\cos \left( \frac{\psi_1}{2} \right) + \cos \left( \frac{\psi_2}{2} \right)} \]

For Minimum Total Weld Metal:

\[ t_1 = \frac{t}{1 + \tan^2 \left( \frac{\psi_1}{2} \right)} \]

\[ t_2 = \frac{t}{1 + \tan^2 \left( \frac{\psi_2}{2} \right)} \]

\[ A_t = t^2 \frac{\tan \left( \frac{\psi_1}{2} \right)}{1 + \tan^2 \left( \frac{\psi_1}{2} \right)} \]

Key:
- \( t \) = Theoretical weld throat dimension, in. (mm)
- \( \omega \) = Distance from member to a parallel line extended from the bottom weld toe, in. (mm)
- \( \psi \) = Dihedral angle, degrees
- \( f \) = Weld face size, in. (mm)
- \( b \) = Weld leg size, in. (mm)
- \( A_t \) = Weld cross-sectional area, in.\(^2\) (mm\(^2\))
- \( b_1 \) = Leg size of Weld 1, in. (mm)
- \( b_2 \) = Leg size of Weld 2, in. (mm)
- \( t_1 \) = Throat dimension of Weld 1, in. (mm)
- \( t_2 \) = Throat dimension of Weld 2, in. (mm)
- \( A_t \) = Total weld cross-sectional area, in.\(^2\) (mm\(^2\))

**Figure 38—Equations for the Analysis of Skewed T-Joints**
Step 3. Select the appropriate equation from Table 13 to find the unit force on the weld;
Step 4. Use Table 13 to find the appropriate properties of the welded connection treated as a line;
Step 5. Combine vectorially all of the unit forces acting on the weld; and
Step 6. Determine the required effective throat size by dividing the total unit force by the allowable stress in the weld.

The following example illustrates the application of these steps in calculating the size of a weld considered as a line. Assume that a bracket supporting an eccentric load of 18,000 lb (80,000 N) is to be fillet welded to the flange of a vertical column, as shown in Figure 40. The procedures for determining the design strength of various eccentrically loaded welded connections used in structural steel construction are provided in the Manual of Steel Construction: Load and Resistance Factor Design.46

In Step 1, the point of maximum combined unit forces is determined to be at the right ends of the top and bottom horizontal welds.

In Step 2, the torsional force caused by the eccentric loading is divided into horizontal ($f_{h}$) and vertical ($f_{v}$) components. The distance from the center of gravity to the point of combined stress, $C$, is calculated from the equation for $C$ in Table 13 for this general shape of connection (the fourth configuration), as follows:

$$C = [C_{YR} + (\frac{d}{2})^2]^{1/2}$$

where

$C_{YR} = \text{Horizontal distance from the center of gravity to the point of combined stress, which equals}$

$$\frac{b(b + d)}{2b + d} = \frac{5(5 + 10)}{2(5 + 10)} = \frac{75}{20} = 3.75 \text{ in. (95 mm)};$$

$d = \text{Length of the vertical weld, in. (mm); and}$

$b = \text{Length of horizontal weld, in. (mm).}$

The polar moment of inertia is then determined by the following:

$$J_{w} = \frac{b^3}{3} (b + 2d) + \frac{d^2}{12} (6b + d)$$

$$= \frac{5^3}{3} (5 + 20) + \frac{10^2}{12} (30 + 10)$$

$$= 385 \text{ in.}^3 (6.3 \times 10^6 \text{ mm}^3)$$

where

$J_{w} = \text{Polar moment of inertia of a line weld, in.}^4 (\text{mm}^4);$

$b = \text{Length of horizontal weld, in. (mm); and}$

$d = \text{Length of vertical weld, in. (mm).}$

The horizontal component of twisting, $f_{h}$, is determined from the Equation (4) in Table 12, as follows:

$$f_{h} = \frac{T(d/2)}{J_{w}} = \frac{(180,000 \times 10)}{385}$$

$$= 2340 \text{ lb/in. (410 N/mm)}$$

where

$f_{h} = \text{Horizontal force component due to twisting, lb (N);}$

$T = \text{Torque (= 18,000} \times 10 = 180,000 \text{ in. lb (31.5 kN mm);}$

46. See Reference 27.
**Table 13**

Equations for the Calculation of Force per Unit Length

<table>
<thead>
<tr>
<th>Type of Loading</th>
<th>Standard Equations for Unit Stress</th>
<th>Equations for Force per Unit Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension or compression</td>
<td>$\sigma = \frac{P}{A}$ ksi (MPa)</td>
<td>$f = \frac{P}{I_w}$ lb/in. (N/mm) (1)</td>
</tr>
<tr>
<td>Vertical shear</td>
<td>$\tau = \frac{V}{A}$ ksi (MPa)</td>
<td>$f = \frac{V}{J_w}$ in./in. (N/mm) (2)</td>
</tr>
<tr>
<td>Bending</td>
<td>$\sigma = \frac{M}{S} = \frac{Mc}{I}$ ksi (MPa)</td>
<td>$f = \frac{M}{S_w} = \frac{Mc}{I_w}$ lb/in. (N/mm) (3)</td>
</tr>
<tr>
<td>Torsion</td>
<td>$\tau = \frac{Tc}{J}$ ksi (MPa)</td>
<td>$f = \frac{Tc}{J_w}$ lb/in. (N/mm) (4)</td>
</tr>
</tbody>
</table>

Key:
- $\sigma$ = Normal stress, ksi (MPa)
- $P$ = Applied force, kips (kN)
- $A$ = Total area of the cross section, in.$^2$ (mm$^2$)
- $f$ = Force per unit, kips (kN)
- $L_w$ = Total length of the line weld, in. (mm)
- $\tau$ = Shear stress, ksi (MPa)
- $V$ = Vertical shear load, kips (kN)
- $M$ = Bending moment, kips in. (kN mm)
- $S$ = Section modulus of a line weld, in.$^4$ (mm$^4$)
- $I_w$ = Moment of inertia of a line weld, in.$^4$ (mm$^4$)
- $S_w$ = Section modulus of a line weld, in.$^4$ (mm$^4$)
- $C_{yr}$ = Distance from the neutral axis to the extreme fibers of a line weld, in. (mm)
- $J$ = Polar moment of inertia of an area, in.$^4$ (mm$^4$)
- $J_w$ = Polar moment of inertia of a line weld, in.$^4$/in. (mm$^4$/mm)

\[ d = \text{Length of vertical weld, in. (mm); and} \]
\[ J_w = \text{Polar moment of inertia of a line weld, in.}^4 \text{ (mm$^4$).} \]

The vertical twisting component is determined from Equation (1) in Table 12, as follows:

\[ f_v = \frac{TC_{yr}}{J_w} = \frac{(180,000)(3.75)}{385} = 1750 \text{ lb/in. (306 N/mm)} \]  
(15)

where
- $f_v = $ Vertical twisting force component, lbf (N);
- $T = $ Torque, in. lb (kN mm);
- $C_{yr} = $ Horizontal distance from the center of gravity to the point of combined stress, which equals
  \[ \frac{b(b+d)}{2b+d} = \frac{5(5+10)}{2(5)+10} = \frac{75}{20} = 3.75 \text{ in. (95 mm)}; \]
- $J_w = $ Polar moment of inertia of a line weld, in.$^4$ (mm$^4$).

The vertical shear force is determined from Equation (1) in Table 12, as follows:

\[ f_s = \frac{P}{L_w} = \frac{180,000}{20} = 90 \text{ lb/in. (158 N/mm)} \]  
(16)

where
- $f_s = $ Vertical shear force, lb (N);
- $F = $ Applied load, lbf (N); and
- $L_w = $ Total length of the weld, in. (mm).

In Step 3, the resultant force is determined, as follows:

\[ f_r = \left[ f_h^2 + (f_v + f_s)^2 \right]^{1/2} \]
\[ = \left[ (2340)^2 + (1750 + 900)^2 \right]^{1/2} \]
\[ = 3540 \text{ lb/in. (620 N/mm)} \]  
(17)

where
- $f_r = $ Resultant force, lbf (N);
- $f_h = $ Horizontal force component due to twisting, lbf (N);
- $f_v = $ Vertical twisting force component, lbf (N); and
- $f_s = $ Vertical shear force, lb (N).
Table 14

Properties of Welded Construction Treated as a Line

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_x = \frac{d^3}{12}$</td>
<td>Moment of inertia about the x-axis</td>
</tr>
<tr>
<td>$S_x = \frac{d^2}{6}$</td>
<td>Area moment of inertia about the x-axis</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_y = \frac{b^2d}{2}$</td>
<td>Moment of inertia about the y-axis</td>
</tr>
<tr>
<td>$S_y = bd$</td>
<td>Area moment of inertia about the y-axis</td>
</tr>
<tr>
<td>$C = \frac{(b^2 + d^2)^{1/2}}{2}$</td>
<td>Radius of gyration</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_x = \frac{d^3}{12} \left( \frac{4b + d}{b + d} \right)$</td>
<td>Moment of inertia about the x-axis</td>
</tr>
<tr>
<td>$S_{xt} = \frac{d}{6} \left( \frac{4b + d}{2b + d} \right)$</td>
<td>Area moment of inertia about the x-axis</td>
</tr>
<tr>
<td>$S_{xb} = \frac{d^2}{6} \left( \frac{4b + d}{2b + d} \right)$</td>
<td>Area moment of inertia about the x-axis</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_y = \frac{b^2}{12} \left( \frac{12 + 4d}{b + d} \right)$</td>
<td>Moment of inertia about the y-axis</td>
</tr>
<tr>
<td>$S_{yl} = \frac{b^2}{6} \left( \frac{b + 4d}{b + d} \right)$</td>
<td>Area moment of inertia about the y-axis</td>
</tr>
<tr>
<td>$S_{yr} = \frac{b^2}{6} \left( \frac{b + 4d}{2b + d} \right)$</td>
<td>Area moment of inertia about the y-axis</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$J_W = \frac{b^3 + d^3}{12} + \frac{bd(b^2 + d^2)}{4(b + d)}$</td>
<td>Polar moment of inertia</td>
</tr>
<tr>
<td>$C_t = \frac{d^2}{2(b + d)}$</td>
<td>Radius of gyration</td>
</tr>
<tr>
<td>$C_B = \frac{d}{2} \left( \frac{2b + d}{b + d} \right)$</td>
<td>Radius of gyration</td>
</tr>
<tr>
<td>$C_1 = (C_t^2 + C_{yrl}^2)^{1/2}$</td>
<td>Radius of gyration</td>
</tr>
<tr>
<td>$C_{yl} = \frac{b^2}{2(b + d)}$</td>
<td>Radius of gyration</td>
</tr>
<tr>
<td>$C_{yr} = \frac{b}{2} \left( \frac{b + 2d}{b + d} \right)$</td>
<td>Radius of gyration</td>
</tr>
<tr>
<td>$C_2 = (C_B^2 + C_{yl}^2)^{1/2}$</td>
<td>Radius of gyration</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_x = \frac{d^2}{12} \left( 6b + d \right)$</td>
<td>Moment of inertia about the x-axis</td>
</tr>
<tr>
<td>$S_x = \frac{d}{6} \left( 6b + d \right)$</td>
<td>Area moment of inertia about the x-axis</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_y = \frac{b^2}{3} \left( \frac{b + 2d}{2b + d} \right)$</td>
<td>Moment of inertia about the y-axis</td>
</tr>
<tr>
<td>$S_{yl} = \frac{b}{3} \left( b = 2d \right)$</td>
<td>Area moment of inertia about the y-axis</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{yl} = \frac{b^2}{2b + d}$</td>
<td>Radius of gyration</td>
</tr>
<tr>
<td>$C_{yr} = \frac{b(b + d)}{2b + d}$</td>
<td>Radius of gyration</td>
</tr>
<tr>
<td>$S_{yr} = \frac{b^2}{3} \left( \frac{b + 2d}{b + d} \right)$</td>
<td>Area moment of inertia about the y-axis</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Equation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C = \left[ C_{yr}^2 + \left( \frac{d}{2} \right)^2 \right]^{1/2}$</td>
<td>Radius of gyration</td>
</tr>
<tr>
<td>$J_W = \frac{b^3}{3} \left( \frac{b + 2d}{2b + d} \right) + \frac{d^2}{12} \left( 6b + d \right)$</td>
<td>Polar moment of inertia</td>
</tr>
</tbody>
</table>

(Continued)
### Table 14 (Continued)
**Properties of Welded Construction Treated as a Line**

<table>
<thead>
<tr>
<th>Diagram</th>
<th>Formula</th>
<th>Formula</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.png" alt="Diagram 1" /></td>
<td>$I_X = \frac{d^2}{6} (3b + d)$</td>
<td>$S_X = \frac{d}{3} (3b + d)$</td>
<td>$J_W = \frac{(b + d)^3}{6}$</td>
</tr>
<tr>
<td><img src="image2.png" alt="Diagram 2" /></td>
<td>$I_X = \frac{d^2}{6} (b + 3d)$</td>
<td>$S_Y = \frac{b}{3} (b + 3d)$</td>
<td>$C = (b^2 + d^2)^{1/2}$</td>
</tr>
<tr>
<td><img src="image3.png" alt="Diagram 3" /></td>
<td>$I_X = \frac{d^3}{3} \left( \frac{2b + d}{b + 2d} \right)$</td>
<td>$S_{XT} = \frac{d}{3} (2b + d)$</td>
<td>$S_{XB} = \frac{d^2}{3} \left( \frac{2b + d}{b + d} \right)$</td>
</tr>
<tr>
<td><img src="image4.png" alt="Diagram 4" /></td>
<td>$I_Y = \frac{b^3}{12}$</td>
<td>$S_Y = \frac{b^2}{6}$</td>
<td>$C_T = \frac{d^2}{b + 2d}$</td>
</tr>
<tr>
<td><img src="image5.png" alt="Diagram 5" /></td>
<td>$J_W = \frac{d^3}{3} \left( \frac{2b + d}{b + 2d} \right) + \frac{b^3}{12}$</td>
<td>$C_B = d \left( \frac{b + d}{b + 2d} \right)$</td>
<td>$C = \left[ C_T^2 + \left( \frac{b}{2} \right)^2 \right]^{1/2}$</td>
</tr>
<tr>
<td><img src="image6.png" alt="Diagram 6" /></td>
<td>$I_X = \frac{d^3}{6} \left( \frac{4b + d}{b + d} \right)$</td>
<td>$S_{XT} = \frac{d}{3} (4b + d)$</td>
<td>$S_{XB} = \frac{d^2}{3} \left( \frac{4b + d}{2b + d} \right)$</td>
</tr>
<tr>
<td><img src="image7.png" alt="Diagram 7" /></td>
<td>$I_Y = \frac{b^3}{6}$</td>
<td>$S_Y = \frac{b^2}{3}$</td>
<td>$C_T = \frac{d^2}{2(b + d)}$</td>
</tr>
<tr>
<td><img src="image8.png" alt="Diagram 8" /></td>
<td>$J_W = \frac{d^3}{6} \left( \frac{4b + d}{b + d} \right) + \frac{b^2}{6}$</td>
<td>$C_B = \frac{d}{2} \left( \frac{2b + d}{b + d} \right)$</td>
<td>$C = \left[ C_T^2 + \left( \frac{b}{2} \right)^2 \right]^{1/2}$</td>
</tr>
<tr>
<td><img src="image9.png" alt="Diagram 9" /></td>
<td>$I_X = \frac{d^2}{6} (3b + d)$</td>
<td>$S_X = \frac{d}{3} (3b + d)$</td>
<td>$J_W = \frac{d^2}{6} \left( \frac{3b + d}{b + d} \right) + \frac{b^3}{6}$</td>
</tr>
<tr>
<td><img src="image10.png" alt="Diagram 10" /></td>
<td>$I_Y = \frac{b^3}{6}$</td>
<td>$S_Y = \frac{b^2}{3}$</td>
<td>$C = \frac{(b^2 + d^2)^{1/2}}{2}$</td>
</tr>
</tbody>
</table>

(Continued)
In Step 4, the design shear strength on the effective area of weld metal having an ultimate tensile strength of 60,000 psi (413 MPa) is determined using LRFD procedures, as follows (see Table 8):

\[
\Phi F_n = 0.75 (0.6 F_{EXX}) \\
= 0.75 (0.6 \times 60,000) \\
= 27,000 \text{ psi (186 MPa)}
\]

where

- \( \Phi \) = LRFD resistance factor;
- \( F_n \) = Nominal strength, kips (kN); and
- \( F_{EXX} \) = Minimum specified tensile strength of the weld metal, ksi (MPa).

The effective throat is then determined, as follows:

\[
E = \frac{f_t}{\Phi F_n} = \frac{3540}{27,000} = 0.128 \text{ in. (3 mm)} \tag{19}
\]

where

- \( E \) = Effective throat, in. (mm);
- \( f_t \) = Resultant force, lb (N);
- \( \Phi \) = LRFD resistance factor; and
- \( F_n \) = Nominal strength, kips (kN).
DESIGN FOR WELDING

Assuming an equal leg fillet weld size, the minimum leg size is equal to the following:

\[ S = \frac{E}{0.707} \]

where

\[ S = \text{Leg size, in. (mm)} \]
\[ E = \text{Effective throat, in. (mm)} \]

By using the method that involves treating the weld as a line, the appropriate weld size can be determined. In this example, a 3/16 in. (5 mm) fillet weld is adequate to transfer the applied load to the column flange. Thus, this weld size should be specified in the welding symbol.

TUBULAR CONNECTIONS

Tubular members, also called hollow structural sections, are used in structures such as drill rigs, space frames, trusses, booms, and earthmoving and mining equipment. They have the advantage of minimizing deflection under load because of their greater rigidity when compared to standard structural shapes. Various types of welded tubular connections, the component designations, and nomenclature are shown in Figure 41.

With structural tubing, holes need not be cut at intersections. Therefore, the connections are characterized by high strength and stiffness. However, connections made with complete joint penetration groove welds must be given special consideration, and appropriate care must be taken to ensure weld quality and that adequate fusion exists at the root. A complete joint penetration weld must be made from one side only and without backing as the small tube size and configuration prevent access to the root side of the weld. Special skill is required to make tubular connections using complete joint penetration groove welds from one side.

With relatively small thin-walled tubes, the end of the brace tube may be partially or fully flattened. The end of the flattened section is trimmed at the appropriate angle to abut against the main member where it is to be welded. This design should only be used with relatively low-load conditions because the load is concentrated on a narrow area of the main tube member. The flattened section of the brace member must be free of cracks.

WELD JOINT DESIGN

When tubular members are fit together for welding, the end of the branch member or brace is normally contoured to the shape of the main member. In the case of T-connections [see Figure 41(C)], the members may be joined with their axes at 80° to 100°. For Y- and K-connections [see Figure 41(D) and 41(E)], an angle less than 80° would be used. The tubes may have a circular or rectangular shape. In addition, the branch member may be equal in size or smaller than the main member.

The angle between the adjacent outside tube surfaces in a plane perpendicular to the joint (the local dihedral angle, \( \Psi \)), can vary around the joint from about 150° to 30°. To accommodate this, the weld joint design and welding procedures used must vary around the joint to obtain a weld with an adequate throat dimension.

Source: Adapted from Blodgett, O. W., 1966, Design of Welded Structures, Cleveland: The James F. Lincoln Arc Welding Foundation, Figure 15.
Figure 41—Welded Tubular Connections, Components, and Nomenclature

(A) Circular Sections

(B) Box Sections

(C) T-Connection

(D) Y-Connection

(E) K-Connection

(F) K-Combination Connections

Note: Relevant gap is between braces whose loads are essentially balanced. Type (2) is also referred to as an N-connection.
(G) Cross Connections

(H) Deviations from Concentric Connections

(I) Simple Tubular Connection

(J) Examples of Complex Reinforced Connections

(K) Flared Connections and Transitions

Figure 41 (Continued)—Welded Tubular Connections, Components, and Nomenclature
Figure 41 (Continued)—Welded Tubular Connections, Components, and Nomenclature

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>CIRCULAR SECTIONS</th>
<th>BOX SECTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \beta )</td>
<td>( r_b/R ) OR ( d_b/D )</td>
<td>( b/D )</td>
</tr>
<tr>
<td>( \eta )</td>
<td>( - )</td>
<td>( a_s/D )</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>( R/t_c )</td>
<td>( D/2t_c )</td>
</tr>
<tr>
<td>( \tau )</td>
<td>( t_p/t_c )</td>
<td>( t_p/t_c )</td>
</tr>
<tr>
<td>( \theta )</td>
<td>ANGLE BETWEEN MEMBER CENTERLINES</td>
<td></td>
</tr>
<tr>
<td>( \psi )</td>
<td>LOCAL DIHEDRAL ANGLE AT A GIVEN POINT ON WELDED JOINT</td>
<td></td>
</tr>
<tr>
<td>( C )</td>
<td>CORNER DIMENSION AS MEASURED TO THE POINT OF TANGENCY OR CONTACT WITH A 90° SQUARE PLACED ON THE CORNER</td>
<td></td>
</tr>
</tbody>
</table>

Source: Adapted from American Welding Society (AWS) Committee on Structural Welding, 2000, Structural Welding Code—Steel, AWS D1.1:2000, Figure 2.14.
Tubular joints are normally accessible for welding only from outside the tubes. Therefore, the joints are generally made with single groove or fillet welds. Groove welds may be designed for complete or partial joint penetration, depending upon the load conditions. To obtain adequate joint penetration, shielded metal arc, gas metal arc, and flux cored arc welding are generally used to make tubular joints in structures.

Suggested groove designs for complete joint penetration with four dihedral angle ranges are presented in Figure 42. The areas of the circular and box connections to which the groove designs of Figure 42 apply are shown in Figure 43(A), 43(B), and 43(C), respectively. In Figure 42, the specified root opening, R, or the width of a backing weld, W, depends upon the welding process and the groove angle. The backing welds, which are not considered part of the throat of the joint design, provide a sound root condition for the deposition of the production weld.

Suggested groove designs for partial joint penetration groove welds for circular and box connections are shown in Figure 44. The sections of circular and box connections to which they apply are shown in Figure 45.

With more conventional prequalified partial joint penetration welds, the variation of the dihedral angles (θ) around the joint, the inaccessibility for welding from the inside, and the differences in penetration of the various processes motivate a separate consideration of the questionable root area of the weld as contrasted to direct tabulation of the effective throat. An allowance should be made for incomplete fusion at the throat of partial joint penetration groove welds. This allowance, which is termed the loss factor, assures that the actual throat of the weld is not smaller than that specified by the design requirement. The loss factor, Z, is shown in Table 15 for various local dihedral angles and welding processes.

Suggested fillet weld details for T-, K-, and Y-connections in circular tubes are shown in Figure 46. These are limited to β ≤ 0.33 for circular sections and β ≤ 0.8 for box sections. The recommended allowable stress on the effective throat of partial joint penetration groove welds and fillet welds in steel T-, K-, and Y-connections is 30% of the tensile strength of the classification of the weld metal. For example, an E7018 electrode has the tensile strength of 70,000 psi (480 MPa). The stress on the adjoining base metal should not exceed that permitted by the applicable code.

A welded tubular connection is limited in strength by four factors. These are:

1. Local or punching shear failure,
2. Uneven distribution of the load on the welded connection,
3. General collapse, and
4. Lamellar tearing.

These limitations are discussed below.

LOCAL FAILURE

When a circular or stepped T-, K-, or Y-connection (see Figure 46) is made by simply welding the branch member to the main member, the local stresses at a potential failure surface through the main member wall may limit the useable strength of the main member. The actual localized stress situation is more complex than simple shear. The term punching shear describes a local failure condition in which the main member fails adjacent to the weld by shear.

Whichever the mode of failure of the main member, the allowable punching shear stress is a conservative representation of the average shear stress at failure in static tests of simple welded tubular connections. The method used to determine the punching shear stress in the main member is presented in Structural Welding Code—Steel, AWS D1.1. The actual punching shear in the main member caused by the axial force and any bending moment in the branch member must be determined and compared with the allowable punching shear stress. The effective area and length of the weld, as well as its section modulus, must be determined to treat the axial force and bending moment on the joint. These joint properties are factored into the stress and force calculations, as described in Structural Welding Code—Steel, AWS D1.1.

UNEVEN DISTRIBUTION OF LOAD

Another condition that can limit the strength of a welded connection is the uneven distribution of a load on a weld. Under load, some bending of the main member could take place, which might cause an uneven distribution of the force applied to the weld. As a result, some yielding and redistribution of stresses may have to take place for the connection to reach its design load. To provide for this, welds at their ultimate breaking strength in T-, K-, and Y-connections [see Figure 41(C), (D), and (E)] must be capable of developing the lesser of (1) the yield strength of the branch member or (2) the


49. Parameter β is defined in Figure 41.

50. See Reference 32.

51. See Reference 32.
### Figure 42—Joint Designs for Complete Joint Penetration in Simple T-, K-, and Y-Tubular Connections

<table>
<thead>
<tr>
<th>Transition from (C) to (D)</th>
<th>A (3)</th>
<th>B (3)</th>
<th>C (3)</th>
<th>D (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ψ = 180°–135°</td>
<td>Ψ = 150°–50°</td>
<td>Ψ = 75°–30°</td>
<td>Ψ = 37-1/2°–15°</td>
<td></td>
</tr>
</tbody>
</table>

#### End preparation
- max: 90°
- min: 45° or 10° for Ψ > 105°

#### Fitup or root opening (R)
- max:
  - FCAW/SMAW: 3/16 in. (5 mm)
  - GMAW/FCAW: 1/4 in. (6 mm)
  - GMAW/FCAW: 1/16 in. (1.6 mm)
  - GMAW/FCAW: 1/32 in. (0.8 mm)
- min:
  - FCAW/SMAW: 1/16 in. (1.6 mm)
  - GMAW/FCAW: 1/16 in. (1.6 mm)

#### Joint included angle φ
- max: 60° for Ψ ≤ 105°
- min: 37-1/2° if more use (B)
- 1/2 Ψ if less use (C)

#### Completed weld
- \( \frac{T}{L} \) ≥ \( t_b \)
- \( \frac{t}{\sin \Psi} \) but need not exceed 1.75\( t \)
- \( \geq t \) for Ψ > 90°
- \( \geq t \sin \Psi \) for Ψ ≤ 90°
- \( \geq 2t_b \)

Notes:
1. These root details apply to SMAW and FCAW (self-shielded).
2. These root details apply to GMAW (short-circuiting transfer and FCAW [gas shielded]).
3. See Figure 43 for locations on the tubular connection.

---

(a) Otherwise as needed to obtain required \( \phi \).
(b) Initial passes of back-up weld are discounted until width of groove (W) is sufficient to assure sound welding; the necessary width of weld groove (W) is provided by back-up weld.

Notes:
1. These root details apply to SMAW and FCAW (self-shielded).
2. These root details apply to GMAW (short-circuiting transfer and FCAW [gas shielded]).
3. See Figure 43 for locations on the tubular connection.
Figure 43—Locations of Complete Joint Penetration Groove Weld Designs on Tubular Connections: (A) Circular Sections; (B) Box Sections; and (C) Matched Box Connections.

Source: American Welding Society (AWS) Committee on Structural Welding, 2000, Structural Welding Code—Steel, D1.1:2000, Miami: American Welding Society, Figure 3.5.
Notes:
1. $t =$ Thickness of the thinner section, in. (mm).
2. Bevel to feather edge except in transition and heel zones.
3. Root opening: 0 in. to 3/16 in. (0 mm to 5 mm).
5. Weld size (effective throat) $t_w \geq t$; see Table 14 for the loss factor, $Z$.
6. Calculations per AWS D1.1:2000, Section 2.40.1.3, shall be done for leg length less than 1.5 $t$, as shown.
7. For box sections, joint preparation for corner transitions shall provide a smooth transition from one detail to another. Welding shall be carried continuously around corners, with corners fully built up and all weld starts and stops within flat faces.
8. The local dihedral angle, $\Psi$, is defined as the angle, measured in a plane perpendicular to the line of the weld, between tangents to the outside surfaces of the tubes being joined at the weld.

Source: Adapted from American Welding Society (AWS) Committee on Structural Welding, 2000, Structural Welding Code—Steel, D1.1:2000, Miami: American Welding Society, Figure 3.5.

Figure 44—Joint Designs for Partial Joint Penetration Groove Welds in Simple T-, K- and Y-Tubular Connections
ultimate punching shear strength of the shear area of the main member. These conditions are illustrated in Figure 47. This particular part of the design is best handled by working in terms of unit force (lb/linear in. [N/linear mm]).

As shown in Figure 47(A), the ultimate breaking strength of fillet welds and partial joint penetration groove welds is computed at 2.67 times the basic allowable stress for 60 ksi (413 MPa) and 70 ksi (480 MPa) tensile strength weld metal, and at 2.2 times for higher strength weld metals.

The unit force on the weld from the branch member at its yield strength, Figure 47(B), is as follows:

\[ f_1 = \sigma_y t_b \]  \hspace{1cm} (21)

where

- \( f_1 \) = Unit force, lb/in. (N/mm);
- \( \sigma_y \) = Yield strength of branch member, psi (MPa); and
- \( t_b \) = Thickness of branch member, in. (mm).

The ultimate shear force on the main-member shear area at failure, shown in Figure 47(C), is as follows:

\[ f_2 = 1.8\tau_a t \]  \hspace{1cm} (22)

where

- \( f_2 \) = Ultimate unit shear normal to the weld, lb/in. (N/mm);
- \( \tau_a \) = Allowable shear stress, psi (MPa); and
- \( t \) = Thickness of the main member, in. (mm).

The unit shear force per inch (mm) on the weld is as follows:

\[ f_3 = \frac{f_2}{\sin\theta} = \frac{1.8\tau_a t}{\sin\theta} \]  \hspace{1cm} (23)

where

- \( f_3 \) = Unit shear force per inch (mm);
- \( f_2 \) = Ultimate unit shear normal to the weld, lb/in. (N/mm);
- \( \theta \) = Angle between the two members, degrees (radians);
- \( \tau_a \) = Allowable shear stress, psi (MPa); and
- \( t \) = Thickness of the main member, in. (mm).

**GENERAL COLLAPSE**

As previously noted, the strength of the connection also depends on what is termed general collapse. The strength and stability of the main member in a tubular connection should be investigated using the proper technology and in accordance with the applicable design code. General collapse should not be a limiting factor if (1) the main member has sufficient thickness to resist punching shear and (2) this thickness extends beyond the branch members for a distance of at least one-fourth of the diameter of the main member.
THROUGH-THICKNESS FAILURES

In tubular connections such as those shown in Figures 41 through 47, the force must be transmitted through the thickness of the main member when the axial force on the branch member is tension. The ductility and notch toughness of rolled metals is significantly lower in the through-thickness (short-transverse) direction than in the longitudinal or transverse directions. Thus, a tubular member could delaminate because of tensile stresses transmitted through the thickness.

To avoid this condition, an interior diaphragm or continuity plates in combination with gusset plates or stiffening rings, as shown in Figure 41(J), can be employed at highly stressed connections. To reduce the through-thickness tensile stresses further, the diaphragm plate can penetrate the shell of the main member as shown in Figure 41(J) (left). These continuity plates are also used to prevent the main member from buckling.

The resulting single-bevel-groove weld, in which the main member is grooved, transfers the delaminating forces from the primary structural member to the secondary structural member. This follows the principles suggested earlier in the chapter regarding the beveling of the through-thickness member to avoid lamellar tears from weld shrinkage.

FATIGUE

The design of welded tubular structures subject to cyclic loading is handled in the same manner as discussed previously. The specific treatment may vary with the applicable code for the structure. Stress categories

Table 15
Loss Factors for Incomplete Fusion at the Root of Partial Joint Penetration Groove Welds

<table>
<thead>
<tr>
<th>Groove Angle, ( \Phi )</th>
<th>Welding Process * (V or OH) †</th>
<th>Loss Factor, ( Z ) ‡</th>
<th>Welding Process * (H or F) †</th>
<th>Loss Factor, ( Z ) ‡</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Phi \geq 60° )</td>
<td>SMAW</td>
<td>0</td>
<td>0</td>
<td>SMAW</td>
</tr>
<tr>
<td>FCAW-S</td>
<td>0</td>
<td>0</td>
<td>FCAW-S</td>
<td>0</td>
</tr>
<tr>
<td>FCAW-G</td>
<td>0</td>
<td>0</td>
<td>FCAW-G</td>
<td>0</td>
</tr>
<tr>
<td>GMAW</td>
<td>N/A</td>
<td>N/A</td>
<td>GMAW</td>
<td>0</td>
</tr>
<tr>
<td>GMAW-S</td>
<td>0</td>
<td>0</td>
<td>GMAW-S</td>
<td>0</td>
</tr>
<tr>
<td>( 60° &gt; \Phi \geq 45° )</td>
<td>SMAW</td>
<td>1/8</td>
<td>3</td>
<td>SMAW</td>
</tr>
<tr>
<td>FCAW-S</td>
<td>1/8</td>
<td>3</td>
<td>FCAW-S</td>
<td>1/8</td>
</tr>
<tr>
<td>FCAW-G</td>
<td>1/8</td>
<td>3</td>
<td>FCAW-G</td>
<td>1/8</td>
</tr>
<tr>
<td>GMAW</td>
<td>N/A</td>
<td>N/A</td>
<td>GMAW</td>
<td>0</td>
</tr>
<tr>
<td>GMAW-S</td>
<td>1/8</td>
<td>3</td>
<td>GMAW-S</td>
<td>1/8</td>
</tr>
<tr>
<td>( 45° &gt; \Phi \geq 30° )</td>
<td>SMAW</td>
<td>1/4</td>
<td>6</td>
<td>SMAW</td>
</tr>
<tr>
<td>FCAW-S</td>
<td>1/4</td>
<td>6</td>
<td>FCAW-S</td>
<td>1/4</td>
</tr>
<tr>
<td>FCAW-G</td>
<td>3/8</td>
<td>10</td>
<td>FCAW-G</td>
<td>3/8</td>
</tr>
<tr>
<td>GMAW</td>
<td>N/A</td>
<td>N/A</td>
<td>GMAW</td>
<td>1/4</td>
</tr>
<tr>
<td>GMAW-S</td>
<td>3/8</td>
<td>10</td>
<td>GMAW-S</td>
<td>3/8</td>
</tr>
</tbody>
</table>

*FCAW-S = Self-shielded flux cored arc welding; GMAW = Spray or globular transfer gas metal arc welding; FCAW-G = Gas shielded flux cored arc welding; GMAW-S = Short circuiting transfer gas metal arc welding.
† V = Vertical position; OH = Overhead position; H = Horizontal position; F = Flat position.
‡ Refer to Figure 46.
§ N/A = Not applicable.
are assigned to various types of tube(s), attachments to tube(s), joint designs, and loading conditions. The total cyclic fatigue stress range for the desired service life of a particular situation can be determined.

Fatigue behavior can be improved by taking one or more of the following actions:

1. Adding a capping layer to provide a smooth contour with the base metal,
2. Grinding the weld face transverse to the weld axis, and
3. Peening the toe of the weld with a blunt instrument to cause local plastic deformation and to smooth the transition between the weld and base metals.

### ALUMINUM STRUCTURES

The concepts and methods employed to design structures in aluminum are generally the same as those used with steel or other metals. The methods and stress values recommended for structural aluminum design are set forth in the Aluminum Association’s *Aluminum*...

Cast and wrought aluminum products are available in many structural forms and shapes. The designer can take advantage of the light weight of aluminum by utilizing available aluminum structural forms. Proper engineering design minimizes the number of joints and amount of welding without affecting product requirements. This, in turn, results in a good appearance and the proper functioning of the product by limiting distortion caused by heating. To eliminate joints, the designer may use castings, extrusions, forgings, or bent or roll-formed shapes to replace complex assemblies. Special extrusions that incorporate edge preparations for welding may provide savings in manufacturing costs. Typical designs are shown in Figure 48. An integral lip can be provided on the extrusion to facilitate alignment and serve as a weld backing.

For cost-effective fabrication, designers should employ the least expensive metal-forming and metal-working processes, minimize the amount of welding required, and place welds at locations of low stress. A simple example is the fabrication of an aluminum tray, as shown in Figure 49. Instead of using five pieces of sheet and eight welds located at the corners, as illustrated in Figure 49(A), this unit could be fabricated from three pieces of sheet, one of which is formed into the bottom and two sides, as shown in 49(B), thereby reducing the amount of welding.

Further reduction in welding could be achieved by additional forming, as depicted in Figure 49(C). This forming also improves performance as butt or lap joints can be welded instead of corners. However, some distortion would likely take place in the two welded sides because all the welds are located in these two planes. The refinement of a design to limit only the amount of welding could lead to problems in fabrication, end use, or appearance. Therefore, the extent of welding should not be the single consideration in weldment design.

**WELD JOINTS**

Butt, lap, edge, corner, and T-joints may be used in aluminum design. For structural applications, edge and corner joints should be avoided because they are harder to fit, weaker, and more prone to fatigue failure than other joints. These two joints are commonly used in sheet metal fabrication, however.

---

Butt Joints

Butt joints are characterized by simplicity of design, good appearance, and better performance under cyclic loading than other types of joints. However, these joints require accurate alignment and joint edge preparation on thicknesses above 1/4 in. (6 mm) to permit satisfactory root penetration. In addition, backgouging and a backing weld are recommended to ensure complete fusion on thicker sections.

Sections of different thicknesses may be butted together and welded. However, it is better to bevel the thicker section before welding to reduce the concentration of stress, particularly when the joint will be exposed to cyclic loading in service.

When thin aluminum sheets are to be welded to thicker sections, it is difficult to obtain adequate depth of fusion in the thicker section without melting away the thin section. This difficulty can be avoided by extruding or machining a lip on the thicker section equal in thickness to that of the thin part, as shown in Figure 50(B). This design will also provide a better heat balance and further reduce the heat-related distortion. If the thicker section is an extrusion, a welding lip can be incorporated in the design as described previously.

Lap Joints

Lap joints are used more frequently with aluminum alloys than with most other metals. In thicknesses up to 1/2 in. (13 mm), it may be more economical to use single-lap joints with fillet welds on both sides rather than butt joints welded with complete joint penetration. Lap joints require no edge preparation, are easy to fit, and require less fixturing than butt joints. The efficiency of lap joints ranges from 70% to 100%, depending on the composition and temper of the base metal. Preferred types of lap joints are presented in Figure 51.

Lap joints create an offset in the plane of the structure unless the members are in the same plane and strips are used on both sides of the joint. Those with an offset tend to rotate under load. Moreover, lap joints may be impractical if the joint is not accessible for welding on both sides.
T-Joints

T-joints seldom require edge preparation because they are usually connected by fillet welds. The welds should have complete fusion to or beyond the root (corner) of the joint. A single- or double-bevel-groove weld in combination with fillet welds may be used with thicknesses above 3/4 in. (19 mm) to reduce the amount of weld metal. T-joints are generally easily fitted and normally require no backgouging. Necessary fixturing is usually quite simple.

A single fillet weld is not recommended for use in a T-joint. Although the joint may have adequate shear and tensile strength, it is very weak when loaded with the root of the fillet weld in tension. Small continuous fillet welds should be used on both sides of the joint rather than large intermittent fillet welds on both sides or a large continuous fillet weld on one side. Continuous fillet welding is recommended to improve fatigue life and prevent crevice corrosion and crater cracks. The suggested allowable shear stresses in fillet welds for building and bridge structures are presented in the Aluminum Design Manual: Specifications and Guidelines for Aluminum Structure.55

JOINT DESIGN

In general, the designs for joints welded in aluminum are similar to those for steel joints.56 However, alumi-
num joints normally have smaller root openings and larger groove angles. To provide adequate shielding of the molten aluminum weld metal, larger gas nozzles are usually employed on welding guns and torches. The excellent machinability of aluminum makes J- and U-groove preparations economical due to a reduction in the volume of weld metal used, especially on thick sections.

EFFECTS OF WELDING ON STRENGTH

Aluminum alloys are normally used in the strain-hardened or heat-treated conditions, or a combination of both, to take advantage of their high strength-to-weight ratios. The effects of strain hardening or heat treatment are wholly or partially negated when aluminum is exposed to the elevated temperatures encountered in welding. The heat of welding softens the base metal in the heat-affected zone. The extent of softening is related to the section thickness, original temper, heat input, and the rate of cooling. The lower strength heat-affected zone must be considered in design. The orientation of the heat-affected zone with respect to the direction of stress and its proportion of the total cross section determines the allowable load on the joint.

The variation in tensile or yield strength across a welded joint in an aluminum structure is illustrated in Figure 52. With plate, the extent of decreased properties is considered to be a 2 in. (50 mm) wide band with the weld in the center. This band will be narrower when joining sheet gauges with an automatic welding process. The minimum mechanical properties for most commonly used welded aluminum alloys are given in the Aluminum Design Manual: Specifications and Guidelines for Aluminum Structures. The minimum tensile properties for those alloys approved for work covered by Structural Welding Code—Aluminum, ANSI/AWS D1.2-97 are shown in Table 16.

Transverse welds in columns and beams should be located at points of lateral support to reinforce the weld and the heat-affected zone to prevent buckling. The weaker heat-affected zone effects of longitudinal welds in structural members can be neglected if the softened zone is less than 15% of the total cross-sectional area. Circumferential welds in piping or tubing may reduce bending strength; longitudinal welds usually have little effect on buckling strength when the heat-affected zone is a small percentage of the total area of the cross section.

57. See Reference 55.
### Table 16
Strength of Welded Aluminum Alloys (GTAW or GMAW with No Postweld Heat Treatment)

<table>
<thead>
<tr>
<th>Material Group No.</th>
<th>Alloy</th>
<th>Temper</th>
<th>Product</th>
<th>Thickness Range, in. (mm)</th>
<th>Minimum Tensile Strength, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Extrusions</td>
<td>All</td>
</tr>
<tr>
<td>24</td>
<td>2219</td>
<td>-T62, -T81, -T851</td>
<td>All</td>
<td>Up through 2.999 (75)</td>
<td>35 (240)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-T8510, -T8511, -T87</td>
<td>Plate</td>
<td>3.000–6.000 (75–150)</td>
<td>35 (240)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-T6, -T852</td>
<td>Forgings</td>
<td>3.000–4.000 (75–100)</td>
<td>35 (240)</td>
</tr>
<tr>
<td>21</td>
<td>3003</td>
<td>-O, -H12, -H14, -H16, -H18, -H22, -H24, -H26, -H28, -H112, -H113, -F</td>
<td>All</td>
<td>Up through 3.000 (75)</td>
<td>14 (95)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Tube</td>
<td>All</td>
<td>13 (90)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Sheet and plate</td>
<td>Up through 0.499 (13)</td>
<td>13 (90)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plate</td>
<td>Up through 0.500–3.000 (13–75)</td>
<td>14 (95)</td>
</tr>
<tr>
<td>22</td>
<td>3004</td>
<td>-O, -H22, -H24, -H26, -H28, -H32, -H34, -H36, -H38, -H112, -F</td>
<td>All</td>
<td>Up through 3.000 (75)</td>
<td>22 (150)</td>
</tr>
<tr>
<td>22 Al clad</td>
<td>3004</td>
<td>-O, -H22, -H24, -H26, -H32, -H34, -H36, -H38, -H112, -F</td>
<td>Sheet and plate</td>
<td>Up through 0.499 (13)</td>
<td>21 (145)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plate</td>
<td>0.500–3.000 (13–75)</td>
<td>22 (150)</td>
</tr>
<tr>
<td>21</td>
<td>5050</td>
<td>-O, -H22, -H24, -H26, -H32, -H34, -H36, -H38, -H112, -F</td>
<td>All</td>
<td>Up through 3.000 (75)</td>
<td>18 (125)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-O, -H111, -H112</td>
<td>Forgings</td>
<td>Up through 4.000 (100)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-O, -H111, -H112, -F</td>
<td>Extrusions</td>
<td>Up through 5.000 (125)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>-O, -H112, -H116</td>
<td>Sheet and plate</td>
<td>0.051–1.500 (1–38)</td>
</tr>
<tr>
<td>25</td>
<td>5083</td>
<td>-H321, -F</td>
<td>Plate</td>
<td>1.501–3.000 (38–75)</td>
<td>39 (270)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.001–5.000 (75–125)</td>
<td>38 (262)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.001–7.000 (125–175)</td>
<td>37 (255)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.001–8.000 (175–200)</td>
<td>36 (248)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Extrusions</td>
<td>2.001–5.000 (50–125)</td>
<td>35 (240)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plate</td>
<td>2.001–3.000 (50–75)</td>
<td>34 (235)</td>
</tr>
<tr>
<td>22</td>
<td>5154</td>
<td>-O, -H22, -H24, -H26, -H28, -H32, -H111, -H112, -F</td>
<td>All</td>
<td>Up through 3.000 (75)</td>
<td>30 (205)</td>
</tr>
<tr>
<td>22</td>
<td>5254</td>
<td>-O, -H32, -H34, -H36, -H38, -H112</td>
<td>All</td>
<td>0.051–3.000 (1–75)</td>
<td>30 (205)</td>
</tr>
<tr>
<td>22</td>
<td>5454</td>
<td>-O, -H32, -H34, -H111, -H112, -F</td>
<td>All</td>
<td>Up through 3.000 (75)</td>
<td>31 (215)</td>
</tr>
</tbody>
</table>

(Continued)
### Table 16 (Continued)
Strength of Welded Aluminum Alloys (GTAW or GMAW with No Postweld Heat Treatment)

<table>
<thead>
<tr>
<th>Material Group No.</th>
<th>Alloy</th>
<th>Temper</th>
<th>Product</th>
<th>Thickness Range, in. (mm)</th>
<th>Minimum Tensile Strength, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>5456</td>
<td>-O, -H111, -H112, -F</td>
<td>Extrusions</td>
<td>Up through 5.000 (125)</td>
<td>41 (285)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-O, -H112, -H116, -H321, -F</td>
<td>Sheet and plate</td>
<td>0.051–1.500 (1–38)</td>
<td>42 (285)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-O, -H116, -F</td>
<td>Plate</td>
<td>1.501–3.000 (38–75)</td>
<td>41 (285)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-O, -F</td>
<td></td>
<td>3.001–5.000 (75–125)</td>
<td>40 (270)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.001–7.000 (125–175)</td>
<td>39 (270)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7.001–8.000 (175–200)</td>
<td>38 (262)</td>
</tr>
<tr>
<td>22</td>
<td>5652</td>
<td>-O, -H22, -H24, -H32, -H34, -H112, -F</td>
<td>All</td>
<td>Up through 3.000 (75)</td>
<td>25 (170)</td>
</tr>
<tr>
<td>23</td>
<td>6005</td>
<td>-T5</td>
<td>Extrusions</td>
<td>Up through 1.000 (25)</td>
<td>24 (165)</td>
</tr>
<tr>
<td></td>
<td>6061</td>
<td>-T4, -T42, -T451, -T51, -T6, -T62, -T651</td>
<td>All</td>
<td>Up through 3.000 (75)</td>
<td>24 (165)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-T6, -T62, -T651</td>
<td>Plate and forgings</td>
<td>3.001–4.000 (75–100)</td>
<td>24 (165)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-T62, -T651</td>
<td>Plate</td>
<td>4.001–6.000 (100–150)</td>
<td>24 (165)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-T6</td>
<td>Forgings</td>
<td>4.001–8.000 (100–200)</td>
<td>24 (165)</td>
</tr>
<tr>
<td>23</td>
<td>Alclad 6061</td>
<td>-T4, -T42, -T451, -T51, -T6, -T62, -T651</td>
<td>Sheet and plate</td>
<td>Up through 3.000 (75)</td>
<td>24 (165)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-T62, -T651</td>
<td>Plate</td>
<td>3.001–5.000 (75–125)</td>
<td>24 (165)</td>
</tr>
<tr>
<td>23</td>
<td>6063</td>
<td>-T4, -T42, -T5, -T52, -T6, -T62, -T83, -T831, -T832</td>
<td>Extrusions</td>
<td>Up through 1.000 (25)</td>
<td>17 (115)</td>
</tr>
<tr>
<td>23</td>
<td>6351</td>
<td>-T4, -T5, -T51, -T53, -T54, -T6</td>
<td>Extrusions</td>
<td>Up through 1.000 (25)</td>
<td>24 (165)</td>
</tr>
<tr>
<td>27</td>
<td>7005</td>
<td>-T53</td>
<td>Extrusions</td>
<td>0.125–1.000 (3–25)</td>
<td>40 (270)</td>
</tr>
<tr>
<td>26</td>
<td>A201.0</td>
<td>-T7</td>
<td>Castings</td>
<td>All</td>
<td>See note</td>
</tr>
<tr>
<td>26</td>
<td>354.0</td>
<td>-T61, -T62</td>
<td>Castings</td>
<td>All</td>
<td>See note</td>
</tr>
<tr>
<td>26</td>
<td>C355.0</td>
<td>-T6, -T61</td>
<td>Castings</td>
<td>All</td>
<td>See note</td>
</tr>
<tr>
<td>26</td>
<td>356.0</td>
<td>-T6, -T7, -T71</td>
<td>Castings</td>
<td>All</td>
<td>23 (159)</td>
</tr>
<tr>
<td>26</td>
<td>A356.0</td>
<td>-T6, -T61</td>
<td>Castings</td>
<td>All</td>
<td>See note</td>
</tr>
<tr>
<td>26</td>
<td>357.0</td>
<td>-T6, -T7</td>
<td>Castings</td>
<td>All</td>
<td>See note</td>
</tr>
<tr>
<td>26</td>
<td>A357.0</td>
<td>-T6, -T61</td>
<td>Castings</td>
<td>All</td>
<td>See note</td>
</tr>
<tr>
<td>26</td>
<td>359.0</td>
<td>-T61, -T62</td>
<td>Castings</td>
<td>All</td>
<td>See note</td>
</tr>
<tr>
<td>26</td>
<td>443.0</td>
<td>-F</td>
<td>Castings</td>
<td>All</td>
<td>17 (115)</td>
</tr>
<tr>
<td>26</td>
<td>A444.0</td>
<td>-T4</td>
<td>Castings</td>
<td>All</td>
<td>17 (115)</td>
</tr>
<tr>
<td>26</td>
<td>514.0</td>
<td>-F</td>
<td>Castings</td>
<td>All</td>
<td>22 (150)</td>
</tr>
<tr>
<td>26</td>
<td>535.0</td>
<td>-F</td>
<td>Castings</td>
<td>All</td>
<td>35 (240)</td>
</tr>
</tbody>
</table>

Note: Minimum as-welded tensile strength has not been established for this alloy. The tensile properties must be established by procedure qualification and approved by the engineer.

With the proper choice of filler metal, a weldment fabricated from a heat-treatable aluminum alloy can be solution-heat treated and aged after welding. The welded assembly will regain full strength with some loss in ductility. Although this is the best method of providing maximum weld strength, it is usually uneconomical or impractical. The cost of heat treatment can be high, especially if a large furnace is required, and the quenching operation may result in unacceptable distortion of the product.

At times, it may be practical to weld a heat-treatable alloy in the solution-treated condition and then age it after welding. This can increase the strength of the weld and heat-affected zone as compared to that achieved in the as-welded condition. This procedure also prevents the distortion problem associated with solution heat treating. No method exists to overcome softening in non-heat-treatable alloys other than further cold working of the parts after welding, which is seldom practical. The weakest location in an as-welded assembly is the annealed (or partially annealed) heat-affected zone.

**STRESS DISTRIBUTION**

When welds located in critical areas do not cover the entire cross section, the strength of the section depends on the percentage of the cross-sectional area affected by the heat of welding. When members must be joined at locations of high stress, the welds should be parallel to the principal member and to the main stress in that member. Transverse welds in tension members should be avoided.

Welds are frequently more highly stressed at the ends than in the central portions. To avoid using thicker sections, areas of high stress in welds can be minimized by sniping. This consists of beveling the end of a member to limit the concentration of stress in the weld at that end. The weld should wrap around the end of the member, however. This type of member termination is illustrated in Figure 53.

In many weldments, it is possible to locate the welds where they will not be subjected to high stresses. It is frequently possible to make connections between a main member and accessories such as braces by welding at the neutral axis or another point of low stress.

**SHEAR STRENGTH OF FILLET WELDS**

The shear strength of fillet welds is controlled by the composition of the filler metal. The use of a high-strength filler metal permits smaller welds. The highest strength filler metal is alloy 5556. Typical shear strengths of longitudinal and transverse fillet welds made with several aluminum filler metals are shown in Figures 54 and 55, respectively.

Assume, for example, that a longitudinal fillet weld having a strength of 4000 lb/in. (700 kN/m) is desired. If 5356 filler metal is used, a 1/4 in. (6 mm) fillet weld can be applied in a single pass. However, 4043 filler metal would require a 3/8 in. (9 mm) fillet weld that would probably require three passes to deposit. The use of the stronger filler metal has an obvious economic advantage as it results in lower labor and material costs.

The minimum practical size of a fillet weld depends on the thickness of the base metal and the welding process and procedure used. Minimum recommended fillet weld sizes are shown in Table 17. When minimum weld sizes must be used, a filler metal with the lowest suitable strength for the applied load should be selected to take advantage of the ductility of the weld metal.

By applying the appropriate safety factor to the shear strength of weld metal, the designer can determine the allowable shear stress in a fillet weld. Appropriate factors of safety and allowable shear stresses in fillet welds for aluminum structures are given in the *Aluminum Design Manual: Specifications and Guidelines for Aluminum Structures*.\(^{59}\)

---

59. See Reference 55.
The fatigue strength of welded aluminum structures follows the same general rules that apply to fabricated assemblies made of other metals. However, aluminum does not have an endurance limit like steel does. Therefore, when fatigue governs the design, the common solution is to reduce the stress by increasing the natural cross section. Fatigue strength is governed by the peak stresses at points of stress concentration rather than by nominal stresses. Eliminating stress raisers to reduce the peak stresses tends to increase the fatigue life of the assembly.

The average fatigue strengths of as-welded joints in small-scale specimens of four aluminum alloys are shown in Figure 56. These results are for butt joints in 3/8 in. (9 mm) plate welded by the gas metal arc welding process. The specimens were welded on one side, backgouged, and then backwelded. The stress ratio of zero signifies that the tensile stress increased from zero to the plotted value and returned to zero during each cycle.

---

The effect of the stress ratio on fatigue life is illustrated in Figure 57. This behavior is typical for all metals.61

The fatigue strengths of the various aluminum alloys are markedly different. Below $10^4$ cycles, designers may prefer to use one alloy rather than another for a particular application. However, beyond $10^6$ cycles, the differences among various alloys are minimal. The solution to fatigue problems beyond the range of $10^6$ cycle involves a change of design rather than a change of alloy.62, 63

Designers should utilize symmetry in the assembly for balanced loading. Sharp changes in direction, notches, and other stress raisers should be avoided. The fatigue strength of a groove weld may be increased significantly by removing weld build-up or crown or by peening the weldment. If these procedures are not practical, the weld build-up or crown should blend smoothly into the base metal to avoid abrupt changes in thickness. With welding processes that produce relatively smooth weld beads, little or no increase in fatigue strength is gained by smoothing the weld faces. It should be noted that the benefit of smooth weld beads can be nullified by excessive spatter during welding. Spatter deposits sometimes create severe stress raisers in the base metal adjacent to the weld.

While the residual stresses produced by welding are not considered to affect the static strength of aluminum, they can be detrimental in regard to fatigue strength.

Several methods can be employed to reduce residual welding stresses. These include shot peening, multiple-pin gun peening, thermal treatments, and the hydrostatic pressurizing of pressure vessels beyond the yield strength. Shot peening or hammer peening is beneficial when it changes the residual stresses at the weld face from tension to compression for a depth of 0.005 in. to 0.030 in. (0.1 mm to 0.8 mm).

However, current design codes for fatigue in civil structures have focused on utilizing only joint details.

### Table 17

<table>
<thead>
<tr>
<th>Base Metal Thickness of Thicker Part Joined (T)</th>
<th>Minimum Size of Fillet Weld*</th>
</tr>
</thead>
<tbody>
<tr>
<td>in.</td>
<td>mm</td>
</tr>
<tr>
<td>T ≤ 1/4</td>
<td>T ≤ 6</td>
</tr>
<tr>
<td>1/4 &lt; T ≤ 1/2</td>
<td>6 &lt; T ≤ 13</td>
</tr>
<tr>
<td>1/2 &lt; T</td>
<td>13 &lt; T</td>
</tr>
</tbody>
</table>

*Except that the weld size need not exceed the thickness of the thinner part joined. For this exception, particular care should be taken to provide sufficient preheat to ensure weld soundness.

† Minimum size for dynamically loaded structures is 3/16 in. (5 mm).

DESIGN FOR WELDING

and stress ranges, rather than stress ratio, as essential criteria. An overview and a comparison of current codes are presented in Sharp, Nordmark, and Menzem.

Thermal treatments to relieve residual stresses are beneficial. They increase fatigue resistance and provide dimensional stability during subsequent machining. Thermal treatments for nonheat-treatable alloys such as the 5000 series can relieve up to 80% of the residual welding stresses with little decrease in the static strength of the base metal. Heat-treatable alloys are not as well suited to thermal treatments for the relief of residual stresses because temperatures that are high enough to cause a significant reduction in residual stress may also substantially diminish strength properties. However, a reduction in residual welding stresses of about 50% is possible if a decrease in strength of approximately 20% can be tolerated.

**EFFECT OF SERVICE TEMPERATURE**

The minimum tensile strengths of aluminum arc welds at various temperatures are presented in Table 18. The performance of welds in nonheat-treatable alloys closely follows that of annealed base metals.

Most aluminum alloys lose a substantial portion of their strength at temperatures above 300°F (150°C). Certain alloys, such as 2219, have better elevated-temperature properties, but their applications have definite limitations. With a magnesium content of 3.5% or higher, the 5000 series alloys are not recommended for use at sustained operating temperatures of over 150°F (65°C). Alloy 5454, with its comparable filler metal ER5554, is the strongest of the 5000 series alloys recommended for applications such as hot chemical storage containers and tank trailers.

In summary, aluminum is an ideal material for low-temperature applications. Most aluminum alloys have high ultimate and yield strengths at temperatures below room temperature. The 5000 series alloys possess good strength and ductility at very low temperatures along with good notch toughness. Alloys 5083 and 5456 are used extensively in pipelines, storage tanks, and marine vessel tankage for the handling of cryogenic liquids and gases.

**CONCLUSION**

Many issues are related to the process of designing for welding. In addition to component performance, service, intended life, and safety, the designer should have a good understanding of the fundamentals of welding, metallurgy, fabrication technology, and inspection techniques.

Although this chapter is quite lengthy, it is not intended to be comprehensive. Many industry specific

---

Table 18

<table>
<thead>
<tr>
<th>Base Metal-Alloy Designation</th>
<th>Filler Metal</th>
<th>-300°F</th>
<th>-200°F</th>
<th>-100°F</th>
<th>100°F</th>
<th>300°F†</th>
<th>500°F†</th>
</tr>
</thead>
<tbody>
<tr>
<td>2219-T37‡</td>
<td>2319</td>
<td>48.5</td>
<td>40.0</td>
<td>36.0</td>
<td>35.0</td>
<td>31.0</td>
<td>19.0</td>
</tr>
<tr>
<td>2219-T62§</td>
<td>2319</td>
<td>64.5</td>
<td>59.5</td>
<td>55.0</td>
<td>50.0</td>
<td>38.0</td>
<td>22.0</td>
</tr>
<tr>
<td>3003</td>
<td>ER1100</td>
<td>27.5</td>
<td>21.5</td>
<td>17.5</td>
<td>14.0</td>
<td>9.5</td>
<td>5.0</td>
</tr>
<tr>
<td>5052</td>
<td>ER5356</td>
<td>38.0</td>
<td>31.0</td>
<td>26.5</td>
<td>25.0</td>
<td>21.0</td>
<td>10.5</td>
</tr>
<tr>
<td>5083</td>
<td>ER5183</td>
<td>54.5</td>
<td>46.0</td>
<td>40.5</td>
<td>40.0</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>5083</td>
<td>ER5356</td>
<td>48.0</td>
<td>40.5</td>
<td>35.5</td>
<td>35.0</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>5454</td>
<td>ER5554</td>
<td>44.0</td>
<td>37.0</td>
<td>32.0</td>
<td>31.0</td>
<td>26.0</td>
<td>15.0</td>
</tr>
<tr>
<td>5456</td>
<td>ER5556</td>
<td>56.0</td>
<td>47.5</td>
<td>42.5</td>
<td>42.0</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>6061-T6‡</td>
<td>ER4043</td>
<td>34.5</td>
<td>30.0</td>
<td>26.5</td>
<td>24.0</td>
<td>20.0</td>
<td>6.0</td>
</tr>
</tbody>
</table>

* To convert to MPa, multiply ksi by 6.895 (ksi value).
† Alloys not listed at 300°F (150°C) and 500°F (260°C) are not recommended for use at sustained operating temperatures of over 150°F (65°C).
‡ As welded.
§ Heat treated and aged after welding.
issues—codes, specifications, contract documents, inspection standards, and associated acceptance criteria—must be addressed when designing weldments. The reader is therefore advised to seek additional technical knowledge in the particular field of interest.


**BIBLIOGRAPHY**


65. The dates of publication given for the codes and other standards listed here were current at the time this chapter was prepared. The reader is advised to consult the latest edition.

**SUPPLEMENTARY READING LIST**


