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# Welded Connections

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#### 22.1 Introduction

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Arc welding has become a popular, widely used method for making steel structures more economical. Although not a new process, welding is still often misunderstood. Perhaps some of the confusion results from the complexity of the technology. To effectively and economically design a building that is to be welded, the engineer should have a knowledge of metallurgy, fatigue, fracture control, weld design, welding processes, welding procedure variables, nondestructive testing, and welding economics. Fortunately, excellent references are readily available, and industry codes specify the minimum standards that are required to be met. Finally, the industry is relatively mature. Although new developments are made every year, the fundamentals of welding are well understood, and many experienced engineers may be consulted for assistance.

Welding is the only joining method that creates a truly one-piece member. All the components of a welded steel structure act in unison, efficiently and effectively transferring loads from one piece to another. Only a minimum amount of material is required when welding is used for joining. Alternative joining methods, such as bolting, are generally more expensive and require the use of lapped plates and angles, increasing the number of pieces required for construction. With welded construction, various materials with different tensile strengths may be mixed, and otherwise unattainable shapes can be achieved. Along with these advantages, however, comes one significant drawback: any problems experienced in one element of a member may be transferred to another. For example, a crack that exists in the flange of a beam may propagate through welds into a column flange. This means

that, particularly in a dynamically loaded structure that is to be joined by welding, all details must be carefully controlled. Interrupted, non-continuous backing bars, tack welds, and even seemingly minor arc strikes have resulted in cracks propagating through primary members.

In order to best utilize the unique capabilities of welding, it is imperative to consider the entire design–fabrication–erection sequence. A properly designed welded connection not only transfers stresses safely, but also is economical to fabricate. Successful integration of design, welding processes, metallurgical considerations, inspection criteria, and in-service inspection depends upon mutual trust and free communication between the engineer and the fabricator.

### 22.2 Joint and Weld Terminology

A welded connection consists of two or more pieces of base metal joined by weld metal. Engineers determine joint type and generally specify weld type and the required throat dimension. Fabricators select the joint details to be used.

### 22.2.1 Joint Types

When pieces of steel are brought together to form a joint, they will assume one of the five configurations presented in Figure 22.1. Of the five, butt, tee, corner, and lap joints are common in construction. Coverplates on rolled beams, and angles to gusset plates would be examples of lap joints. Edge joints are more common for sheet metal applications. Joint types are merely descriptions of the relative positioning of the materials; the joint type does not imply a specific type of weld.



FIGURE 22.1: Joint types. (Courtesy of The Lincoln Electric Company. With permission.)

### 22.2.2 Weld Types

Welds may be placed into three major categories: groove welds, fillet welds, and plug or slot welds (see Figure 22.2). For groove welds, there are two subcategories: complete joint penetration (CJP) groove welds and partial joint penetration (PJP) groove welds (see Figure 22.3). Plug welds are commonly used to weld decking to structural supports. Groove and fillet welds are of prime interest for major structural connections.

In Figure 22.4, terminology associated with groove welds and fillet welds is illustrated. Of great interest to the designer is the dimension noted as the "throat." The throat is theoretically the weakest plane in the weld. This generally governs the strength of the welded connection.



FIGURE 22.2: Major weld types. (Courtesy of The Lincoln Electric Company. With permission.)



FIGURE 22.3: Types of groove welds. (Courtesy of The Lincoln Electric Company. With permission.)

### 22.2.3 Fillet Welds

Fillet welds have a triangular cross-section and are applied to the surface of the materials they join. Fillet welds by themselves do not fully fuse the cross-sectional areas of parts they join, although it is still possible to develop full-strength connections with fillet welds.

The size of a fillet weld is usually determined by measuring the leg size, even though the weld is designed by determining the required throat size. For equal-legged, flat-faced fillet welds applied to plates that are oriented  $90^{\circ}$  apart, the throat dimension is found by multiplying the leg size by 0.707 (i.e., sine  $45^{\circ}$ ).

### 22.2.4 Complete Joint Penetration (CJP) Groove Welds

By definition, CJP groove welds have a throat dimension equal to the thickness of the plate they join (see Figure 22.3). For prequalified welding procedure specifications, the American Welding Society (AWS) D1.1-96 [9] *Structural Welding Code* requires backing (see Weld Backing) if a CJP weld is made from one side, and back gouging if a CJP weld is made from both sides. This ensures complete fusion throughout the thickness of the material being joined. Otherwise, procedure qualification testing is required to prove that the full throat is developed. A special exception to this is applied to tubular connections whose CJP groove welds may be made from one side without backing.



FIGURE 22.4: Weld terminology. (Courtesy of The Lincoln Electric Company. With permission.)

### 22.2.5 Partial Joint Penetration (PJP) Groove Welds

A PJP groove weld is one that, by definition, has a throat dimension less than the thickness of the materials it joins (see Figure 22.3). An "effective throat" is associated with a PJP groove weld (see Figure 22.5). This term is used to delineate the difference between the depth of groove preparation



FIGURE 22.5: PJP groove welds: "E" vs. "S". (Courtesy of The Lincoln Electric Company. With permission.)

and the probable depth of fusion that will be achieved. When submerged arc welding (which has inherently deep penetration) is used, and the weld groove included angle is  $60^{\circ}$ , the D1.1-96 [9] code allows the designer to rely on the full depth of joint preparation to be used for delivering the required throat dimension. When other processes with less penetration are used, such as shielded metal arc welding, and when the groove angle is restricted to  $45^{\circ}$ , it is doubtful that fusion to the root of the joint will be obtained. Because of this, the D1.1-96 code assumes that 1/8 in. of the PJP joint may not be fused. Therefore, the effective throat is assumed to be 1/8 in. less than the depth of preparation. This means that for a given included angle, the depth of joint preparation must be increased to offset the loss of penetration.

The effective throat on a PJP groove weld is abbreviated utilizing a capital "E". The required depth of groove preparation is designated by a capital "S". Since the engineer does not normally know which welding process a fabricator will select, it is necessary for the engineer to specify only the dimension for E. The fabricator then selects the welding process, determines the position of welding, and thus specifies the appropriate S dimension, which will be shown on the shop drawings. In most cases, both the S and E dimensions will be contained on the welding symbols of shop drawings, the effective throat dimension showing up in parentheses.

### 22.2.6 Double-Sided Welds

Welds may be single or double. Double welds are made from both sides of the member (see Figure 22.6). Double-sided welds may require less weld metal to complete the joint. This, of course, has advantages with respect to cost and is of particular importance when joining thick members. However, double-sided joints necessitate access to both sides. If the double joint necessitates overhead welding, the economies of less weld metal may be lost because overhead welding deposition rates are inherently slower. For joints that can be repositioned, this is of little consequence. There are also distortion considerations, where the double-sided joints have some advantages in balancing weld shrinkage strains.



FIGURE 22.6: Single- vs. double-sided joints. (Courtesy of The Lincoln Electric Company. With permission.)

### 22.2.7 Groove Weld Preparations

Within the groove weld category, there are several types of preparations (see Figure 22.7). If the joint contains no preparation, it is known as a square groove. Except for thin sections, the square groove is rarely used. The bevel groove is characterized by one plate cut at a  $90^{\circ}$  angle and a second plate with



FIGURE 22.7: Groove weld preparation. (Courtesy of The Lincoln Electric Company. With permission.)

a bevel cut. A vee groove is similar to a bevel, except both plates are bevel cut. A J-groove resembles a bevel, except the root has a radius, as opposed to a straight cut. A U-groove is similar to two J-grooves put together. For butt joints, vee and U-groove details are typically used when welding in the flat position since it is easier to achieve uniform fusion when welds are placed upon the inclined surfaces of these details versus the vertical edge of one side of the bevel or J-groove counterparts.

Properly made, any CJP groove preparation will yield a connection equal in strength to the connected material. The factors that separate the advantages of each type of preparation are largely fabrication related. Preparation costs of the various grooves differ. The flat surfaces of vee and bevel groove weld preparations are generally more economical to produce than the U and J counterparts, although less weld metal is usually required in the later examples. For a given plate thickness, the volume of weld metal required for the different types of grooves will vary, directly affecting fabrication costs. As the volume of weld metal cools, it generates residual stresses in the connection that have a direct effect on the extent of distortion and the probability of cracking or lamellar tearing. Reducing weld volume is generally advantageous in limiting these problems. The decision as to which groove type will be used is usually left to the fabricator who, based on knowledge, experience, and available equipment, selects the type of groove that will generate the required quality at a reasonable cost. In fact, design engineers should not specify the type of groove detail to be used, but rather determine whether a weld should be a CJP or a PJP.

### 22.2.8 Interaction of Joint Type and Weld Type

Not every weld type can be applied to every type of joint. For example, butt joints can be joined only with groove welds. A fillet weld cannot be applied to a butt joint. Tee joints may be joined with fillet welds or groove welds. Similarly, corner joints may be joined with either groove welds or fillet welds. Lap joints would typically be joined with fillet welds or plug/slot welds. Table 22.1 illustrates possible combinations.

	Fillet Groove		Plug/Slot
Butt	N.A.		N.A.
Тее			N.A.
Corner	5		N.A.
Lap		N.A.	

Courtesy of Lincoln Electric Company. With permission.

### 22.3 Determining Weld Size

#### 22.3.1 Strength of Welded Connections

A welded connection can be designed and fabricated to have a strength that matches or exceeds that of the steel it joins. This is known as a full-strength connection and can be considered 100% efficient; that is, it has strength equivalent to that of the base metal it joins. Welded connections can be designed so that if loaded to destruction, failure would occur in the base material. Poor weld quality, however, may adversely affect weld strength.

A connection that duplicates the base metal capacity is not always necessary and when unwarranted, its specification unnecessarily increases fabrication costs. In the absence of design information, it is possible to specify welds that have strengths equivalent to the base metal capacity. Assuming the base metal thickness has been properly selected, a weld that duplicates the strength of the base metal will be adequate as well. This, however, is a very costly approach. Economical connections cannot be designed on this basis. Unfortunately, the overuse of the CJP detail and the requirement of "matching filler metal" (i.e., weld metal of a strength that is equal to that of the base metal) serves as evidence that this is often the case.

#### 22.3.2 Variables Affecting Welded Connection Strength

The strength of a welded connection is dependent on the weld metal strength and the area of weld that resists the load. Weld metal strength is a measure of the capacity of the deposited weld metal itself, measured in units such as ksi (kips per square inch). The connection strength reflects the combination of weld metal strength and cross-sectional area, and would be expressed as a unit of force, such as kips. If the product of area times the weld metal strength exceeds the loads applied, the weld should not fail in static service. For cyclic dynamic service, fatigue must be considered as well.

The area of weld metal that resists fracture is the product of the theoretical throat multiplied by the length. The theoretical weld throat is defined as the minimum distance from the root of the weld to its theoretical face. For a CJP groove weld, the theoretical throat is assumed to be equal to the

thickness of the plate it joins. Theoretical throat dimensions of several types of welds are shown in Figure 22.8.



FIGURE 22.8: Theoretical throats. (Courtesy of The Lincoln Electric Company. With permission.)

For fillet welds or partial joint penetration groove welds, using filler metal with strength levels equal to or less than the base metal, the theoretical failure plane is through the weld throat. When the same weld is made using filler metal with a strength level greater than that of the base metal, the failure plane may shift into the fusion boundary or heat-affected zone. Most designers will calculate the load capacity of the base metal, as well as the capacity of the weld throat. The fusion zone and its capacity is not generally checked, as this is unnecessary when matching or undermatching weld metal is used. When overmatching weld metal is specifically selected, and the required weld size is deliberately reduced to take advantage of the overmatched weld metal, the designer must check the capacity of the fusion zone (controlled by the base metal) to ensure adequate capacity in the connection.

Complete joint penetration groove welds that utilize weld metal with strength levels exactly equal to the base metal will theoretically fail in either the weld or the base metal. Even with matching weld metal, the weld metal is generally slightly higher in strength than the base metal, so the theoretical failure plane for transversely loaded connections is assumed to be in the base metal.

### 22.3.3 Determining Throat Size for Tension or Shear Loads

Connection strength is governed by three variables: weld metal strength, weld length, and weld throat. The weld length is often fixed, due to the geometry of the parts being joined, leaving one variable to be determined, namely, the throat dimension.

For tension or shear loads, the required capacity the weld must deliver is simply the force divided by the length of the weld. The result, in units of force per length (such as kips per inch) can be divided by the weld metal strength, in units of force per area (such as kips per square inch). The final result would be the required throat, in inches. Weld metal allowables that incorporate factors of safety can be used instead of the actual weld metal capacity. This directly generates the required throat size.

To determine the weld size, it is necessary to consider what type of weld is to be used. Assume the preceding calculation determined the need for a 1-in. throat size. If a single fillet weld is to be used, a throat of 1 in. would necessitate a leg size of 1.4 in., shown in Figure 22.9. For double-sided fillets,



FIGURE 22.9: Weld combinations with equal throat dimensions. (Courtesy of The Lincoln Electric Company. With permission.)

two 0.7-in. leg size fillets could be used. If a single PJP groove weld is used, the effective throat would have to be 1 in. The actual depth of preparation of the production joint would be 1 in. or greater, depending on the welding procedure and included angle used. A double PJP groove weld would require two effective throats of 0.5 in. each. A final option would be a combination of partial joint penetration groove welds and external fillet welds. As shown in Figure 22.9, a 60° included angle was utilized for the PJP groove weld and an unequal leg fillet weld was applied externally. This acts to shift the effective throat from the normal  $45^\circ$  angle location to a  $30^\circ$  throat.

If the plates being joined are 1 in. thick, a CJP groove weld is the only type of groove weld that will effectively transfer the stress, since the throat on a CJP weld is equal to the plate thickness. PJP groove welds would be incapable of developing adequate throat dimensions for this application, although the use of a combination PJP-fillet weld would be a possibility.

#### 22.3.4 Determining Throat Size for Compressive Loads

When joints are subject only to compression, the unwelded portion of the joint may be milled-tobear, reducing the required weld throat. Typical of these types of connections are column splices where PJP groove welds frequently are used for static structures.

### 22.3.5 Determining Throat Size for Bending or Torsional Loads

When a weld, or group of welds, is subject to bending or torsional loads, the weld(s) will not be uniformly loaded. In order to determine the stress on the weld(s), a weld size must be assumed and the resulting stress distribution calculated. An iterative approach may be used to optimize the weld size.

A simpler approach is to treat the weld as a line with no throat. Standard design formulas may be used to determine bending, vertical shear, torsion, etc. These formulas normally result in unit stresses. When applied to welds treated as a line, the formulas result in a force on the welds, measured in pounds per linear inch, from which the capacity of the weld metal, or applicable allowable values, may be used to determine the required throat size.

The following is a simple method used to determine the correct amount of welding required to provide adequate strength for either a bending or a torsional load. In this method, the weld is treated as a line, having no area but having a definite length and cross-section. This method offers the following advantages:

- 1. It is not necessary to consider throat areas.
- 2. Properties of the weld are easily found from a table without knowledge of weld leg size.
- 3. Forces are considered per unit length of weld, rather than converted to stresses. This facilitates dealing with combined-stress problems.
- 4. Actual values of welds are given as force per unit length of weld instead of unit stress on throat of weld.

Visualize the welded connection as a line (or lines), following the same outline as the connection but having no cross-sectional area. In Figure 22.10, the desired area of the welded connection,  $A_w$ ,



FIGURE 22.10: Treating the weld as a line for a twisting or bending load:  $A_w =$  length of weld (in.),  $Z_w =$  section modulus of weld (in.<sup>2</sup>),  $J_w =$  polar moment of inertia of weld (in.<sup>3</sup>). (Courtesy of The Lincoln Electric Company. With permission.)

can be presented by just the length of the weld. The stress on the weld cannot be determined unless the weld size is assumed; but by following the proposed procedure, which treats the weld as a line, the solution is more direct, is much simpler, and becomes basically one of determining the force on the weld(s).

### 22.3.6 Treating the Weld as a Line to Find Weld Size

By inserting this property of the welded connection into the standard design formula used for a particular type of load (Table 22.2), the unit force on the weld is found in terms of pounds per linear inch of weld.

	Type of Loading		Standard Design Formula	Treating the Weld as a Line	
	Type of Hoading		Stress lbs/in <sup>2</sup>	Force lbs/in	
		Transm	Primary Welds it Entire Load at th	iis Point	
	P P	Tension or compression	$\sigma = \frac{P}{A}$	$f = \frac{P}{A_w}$	
		Vertical Shear	$\tau = \frac{V}{A}$	$f = \frac{V}{A_w}$	
	M	Bending	$\sigma = \frac{M}{Z}$	$f = \frac{M}{Z_w}$	
	C T	Twisting	$\tau = \frac{TC}{J}$	$f = \frac{TC}{J_w}$	
		Hold Sec	Secondary Welds tion Together - Lo	ow Stress	
		Horizontal Shear	$\tau = \frac{VAy}{It}$	$f = \frac{VAy}{In}$	
		Torsional Horizontal Shear	$\tau = \frac{TC}{J}$	$f = \frac{TCt}{J}$	
Courtesy of The Linc	oln Electric Company. With permission.				

TABLE 22.2 Standard Design Formulas Used for Determining Force on Weld

Normally, use of these standard design formulas results in a unit stress, in pounds per square inch, but with the weld treated as a line, these formulas result in a unit force on the weld, in units of pounds per linear inch.

For problems involving bending or twisting loads, Table 22.3 is used. It contains the section modulus,  $S_w$ , and polar moment of inertia,  $J_w$ , of 13 typical welded connections with the weld treated as a line. For any given connection, two dimensions are needed: width, *b*, and depth, *d*. Section modulus,  $S_w$ , is used for welds subjected to bending; polar moment of inertia,  $J_w$ , for welds

subjected to twisting. Section modulus,  $S_w$ , in Table 22.3 is shown for symmetric and asymmetric connections. For asymmetric connections,  $S_w$  values listed differentiate between top and bottom, and the forces derived therefrom are specific to location, depending on the value of  $S_w$  used.

When more than one load is applied to a welded connection, they are combined vectorially, but must occur at the same location on the welded joint.

### 22.3.7 Use Allowable Strength of Weld to Find Weld Size

Weld size is obtained by dividing the resulting unit force on the weld by the allowable strength of the particular type of weld used, obtained from Table 22.4 or 22.5. For a joint that has only a transverse load applied to the weld (either fillet or butt weld), the allowable transverse load may be used from the applicable table. If part of the load is applied parallel (even if there are transverse loads in addition), the allowable parallel load must be used.

### 22.3.8 Applying the System to Any Welded Connection

- 1. Find the position on the welded connection where the combination of forces will be maximum. There may be more than one that must be considered.
- 2. Find the value of each of the forces on the welded connection at this point. Use Table 22.2 for the standard design formula to find the force on the weld. Use Table 22.3 to find the property of the weld treated as a line.
- 3. Combine (vectorially) all the forces on the weld at this point.
- 4. Determine the required weld size by dividing this value (step 3) by the allowable force in Table 22.4 or 22.5.

### 22.3.9 Sample Calculations Using This System

The example in Figure 22.11 illustrates the application of this procedure.

#### 22.3.10 Weld Size for Longitudinal Welds

Longitudinal welds include the web-to-flange welds on I-shaped girders and the welds on the corners of box girders. These welds primarily transmit horizontal shear forces resulting from the change in moment along the member. To determine the force between the members being joined, the following equation may be used:

$$f = \frac{Vay}{In}$$

where

- f = force on weld per unit length
- V = total shear on section at a given position along the beam
- a = area of flange connected by the weld
- y = distance from the neutral axis of the whole section to the center of gravity of the flange
- I = moment of inertia of the whole section
- n = number of welds joining the flange to webs per joint

The resulting force per unit length is then divided by the allowable stress in the weld metal and the weld throat is attained. This particular procedure is emphasized because the resultant value for the weld throat is nearly always less than the minimum allowable weld size. The minimum size then becomes the controlling factor.

Properties Of Weld Treated As A Line							
Ouline of Welded Joint b = width d=depth	Bending (about horizontal axis x - x)	Twisting					
d xx	$S_w = \frac{d^2}{6}$ In <sup>2</sup>	$J_{w} = \frac{d^{3}}{12} \qquad In^{3}$					
x x c	$S_w = \frac{d^2}{3}$	$J_{w} = \frac{d(3b^{2} + d^{2})}{6}$					
	$S_w = bd$	$J_{w} = \frac{b^3 + 3bd^2}{6}$					
$\begin{array}{c c} & & & \downarrow \\ \hline & & & \downarrow \\ d \\ & & & \downarrow \\ & & & \downarrow \\ \end{array} \begin{array}{c} & & \downarrow \\ & & & \downarrow \\ & & & \downarrow \\ & & & \\ & & $	$S_{w} = \frac{4bd + d^{2}}{6} = \frac{d^{2}(2b + d)}{3(b + d)}$ Top Bottom	$J_{w} = \frac{(b+d)^{4} - 6b^{2}d^{2}}{12(b+d)}$					
$N_{y} = \frac{b^{2}}{2(b+d)} x - \begin{bmatrix} y \\ y \\ y \\ y \end{bmatrix} $	$S_w = bd + \frac{d^2}{6}$	$\mathbf{J}_{w} = \frac{(2b+d)^{3}}{12} - \frac{b^{2}(b+d)^{2}}{(2b+d)}$					
$N_{x} = \frac{d^{3}}{2(b+d)} x - $	$S_{w} = \frac{2bd + d^{2}}{3} = \frac{d^{2}(2b + d)}{3(b + d)}$ Top Bottom	$J_{w} = \frac{(b+2d)^{3}}{12} - \frac{b^{2}(b+d)^{2}}{(b+2d)}$					
	$S_w = bd + \frac{d^2}{3}$	$J_{w} = \frac{(b+d)^3}{6}$					
$N_{y} = \frac{d^{2}}{b+2d} \xrightarrow{xx} \frac{d}{d}$	$S_{w} = \frac{2bd + d^{2}}{3} = \frac{d^{2}(2b + d)}{3(b + d)}$ Top Bottom	$J_{w} = \frac{(b+2d)^{3}}{12} - \frac{d^{2}(b+d)^{2}}{(b+2d)}$					
$N_{y} = \frac{d^{3}}{2(b+d)} \qquad \qquad$	$S_{w} = \frac{4bd + d^{2}}{3} = \frac{4bd^{2} + d^{3}}{6b + 3d}$ Top Bottom	$J_{w} = \frac{d^{3}(4b+d)}{6(b+d)} + \frac{b^{3}}{6}$					
	$S_{w} = bd + \frac{d^2}{3}$	$J_{w} = \frac{b^{3} + 3bd^{2} + d^{3}}{6}$					
$\begin{array}{c c} \hline \bullet & \bullet \\ \hline \hline & \bullet \\ \hline & \bullet \\ \hline & \bullet \\ \hline & \bullet \\ \hline \hline \hline & \bullet \\ \hline \hline \hline & \bullet \\ \hline \hline \hline \hline \hline \hline \\ \hline \hline$	$S_w = 2bd + \frac{d^2}{3}$	$J_{w} = \frac{2b^3 + 6bd^2 + d^3}{6}$					
xx	$S_{w} = \frac{\pi d^2}{4}$	$J_{\rm w} = \frac{\pi d^3}{4}$					
	$S_{\rm w} = \frac{\pi d^2}{2} + \pi D^2$						

**TABLE 22.3** Properties of Welded Connection; Treating Weld as a Line

Courtesy of The Lincoln Electric Company. With permission.

Type of weld	Stress in weld		Allowable connection stress	Required filler metal strength level	
	Tension normal to t	he effective area	Same as base metal	Matching filler metal shall be used	
Complete joint penetration groove welds	Compression norma area	ll to the effective	Same as base metal	Filler metal with a strength level equal to or one classification (10 ksi [69 MPa]) less than matching filler metal may be used	
	Tension or compresthe axis of the weld	sion parallel to	Same as base metal		
	Shear on the effectiv	e areas	$0.30 \times$ nominal tensile strength of filler metal, except shear stress on base metal shall not exceed 0.40 $\times$ yield strength of base metal	Filler metal with a strength level equal to or less than matching filler metal may be used	
	Compression nor- mal to effective area	Joint not designed to bear	$0.50 \times$ nominal tensile strength of filler metal, except stress on base metal shall not exceed 0.60 $\times$ yield strength of base metal		
		Joint designed to bear	Same as base metal	_	
Partial joint penetration groove welds	Tension or compression parallel to the axis of the weld $\!\!\!\!^a$		Same as base metal	Filler metal with strength level equal to or less than matching filler metal may be used	
	Shear parallel to axis of weld		$0.30 \times$ nominal tensile strength of filler metal, except shear stress on base metal shall not exceed 0.40 $\times$ yield strength of base metal		
	Tension normal to effective area		0.30 × nominal tensile strength of filler metal, except tensile stress on base metal shall not exceed 0.60 × yield strength of base metal		
	Shear on effective ar	ea	$\begin{array}{llllllllllllllllllllllllllllllllllll$		
Fillet weld	Tension or compres axis of weld <sup>a</sup>	ssion parallel to	Same as base metal	Filler metal with a strength level equal to or less than matching filler metal may be used	
Plug and slot welds	Shear parallel to fay effective area)	ing surfaces (on	$0.30 \times$ nominal tensile strength of filler metal, except shear stress on base metal shall not exceed 0.40 $\times$ yield strength of base metal	Filler metal with a strength level equal to or less than matching filler metal may be used	

#### **TABLE 22.4** Stress Allowables for Weld Metal

<sup>44</sup> Fillet weld and partial joint penetration groove welds joining the component elements of built-up members, such as flange-to-web connections, may be designed without regard to the tensile or compressive stress in these elements parallel to the axis of the welds. From American Welding Society. *Structural Welding Code: Steel: ANSI/AWS D1.1-96.* Miami, Florida, 1996. With permission.



**TABLE 22.5** AISC Fatigue Allowables



**TABLE 22.5** AISC Fatigue Allowables (continued)

From American Institute of Steel Construction, Chicago, IL, 1996.



Use table 1 <u>Twisting (Horizontal Component)</u>  $f_{h} = \frac{T C_{H}}{J_{W}} = \frac{(180,000) (5)}{(385.9)} = 2340 \text{ lbs/in.}$ <u>Vertical Shear</u>  $f_{s} = \frac{P}{A_{W}} = \frac{18,000}{20} = 900 \text{ lbs/in.}$ <u>Step 3.</u> <u>Twisting (Vertical Component)</u>  $f_{v} = \frac{T C_{v}}{J_{W}} = \frac{(180,000) (3.75)}{(385.9)} = 1750 \text{ lbs/in.}$ 

$$f_R = \sqrt{f_v^2 + f_h^2} = \sqrt{2340^2 + 2650^2} = 3540 \text{ lbs/in.}$$

$$f_{\rm R} = 3540$$
  $f_{\rm v} = 1750$   
 $f_{\rm r} = 900$   $f_{\rm v} = 900$ 

Step 4.

w =  $\frac{3540}{0.707 (18,000)}$  = 0.278 in. or  $\frac{5}{16}$  in. leg fillet weld

FIGURE 22.11: Sample problem using steps outlined in this approach to determine weld size. (Courtesy of The Lincoln Electric Company. With permission.)

### 22.3.11 Minimum Weld Size

Many codes specify minimum weld sizes that are a function of plate thickness. These are not designrelated requirements, but rather reflect the inherent interaction of heat input and weld size.

### 22.3.12 Heat Input and Weld Size

Heat input and weld bead size (or cross-sectional area) are directly related. Heat input is typically calculated with the following equation:

$$H = \frac{60EI}{1000S}$$

where H = heat input (kJ/in.)

- $E = \operatorname{arc} \operatorname{volts}$
- I = amperage
- S = travel speed (in./min)

In order to create a larger weld in one pass, two approaches may be used: higher amperages (I) or slower travel speeds (S) must be employed. Notice that either procedure modification results in a higher heat input. Welding codes have specified minimum acceptable weld sizes with the primary purpose of dictating minimum heat input levels. For example, almost independent of the welding process used, a 1/4-in. fillet weld will require a heat input of approximately 20–30 kJ/in. By prescribing a minimum fillet weld size, these specifications have, in essence, specified a minimum heat input.

Understanding that the minimum fillet weld size is related to heat input, we must also note that there is an inherent interaction of preheat and heat input. The prescribed minimum fillet weld sizes assume the required preheats are also applied. If a situation arises where it is impossible to construct the minimum fillet weld size, it may be appropriate to increase the required preheat to compensate for the reduced energy of welding.

The minimum fillet weld size need never exceed the thickness of the thinner part. It is important to recognize the implications of this requirement. In some extreme circumstances, the connection might involve a very thin plate being joined to an extremely thick plate. The code requirements would dictate that the weld need not exceed the size of the thinner part. However, under these circumstances, additional preheat based upon the thicker material may be justified.

### 22.3.13 Required Weld vs. Minimum Weld Sizes

When welds are properly sized based upon the forces they are required to transfer, the appropriate weld size frequently is found to be surprisingly small. Even on bridge plate girders that may be 18 to 20 ft deep, with flange thicknesses exceeding 2 in., the required fillet weld size to transmit the horizontal shear forces may be in the range of a 3/32-in. continuous fillet. Intuition indicates that something would be wrong when trying to apply this small weld to join a flange that may be 2 in. thick to a web that is 3/4 in. thick. This is not to indicate a fault with the method used to determine weld size, but rather reveals the small shear forces involved. However, when attempts are made to fabricate this plate girder with these small weld sizes, extremely high travel speeds or very low currents would be experienced by the weld metal and the base material, specifically the heat-affected zone, would be exceedingly high. A brittle microstructure could be formed. To avoid this condition, the minimum weld size would dictate that a larger weld is required. This is frequently the case for longitudinal welds that resist shear. Any further increase in specified weld size is unnecessary and directly increases fabrication costs.

### 22.3.14 Single-Pass Minimum Sized Welds

Controlling the heat input by specifying the minimum fillet weld size necessitates that this minimum fillet weld be made in a single pass. If multiple passes are used to construct the minimum sized fillet weld, the intent of the requirement is circumvented. In the past, some recommendations included minimum fillet weld sizes of 3/8 in. and larger. A single-pass 3/8-in. fillet weld can be made only in the flat or vertical position. In the horizontal position, multiple passes are required, and the spirit of the requirement is invalidated. For this reason, the largest minimum fillet weld in Table 5.8 of the AWS D1.1-96 code [9] is 5/16-in. However, even this weld may necessitate multiple passes, depending on the particular welding process used. For example, a quality 5/16-in. fillet weld cannot be made in a single pass with the shielded metal arc welding process utilizing l/8-in.-diameter electrodes, except perhaps in the vertical plane.

It may not be possible to make the required minimum sized fillet weld in a single weld pass under

all conditions. For example, it is impossible to make a 5/16-in. fillet weld in a single pass in the overhead position. Under these conditions, it is important to remember the principles that underlie the code requirements. For the preceding example, the overhead fillet weld would necessitate three weld passes. Each weld pass would be made with approximately one-third of the heat input normally associated with the 5/16-in. fillet weld. In order to ensure satisfactory results, it would be desirable to utilize additional preheat to offset the naturally resulting lower heat input that would result from each of these weld passes.

### 22.3.15 Minimum Sized Groove Welds

When CJP groove welds are made, there is no need to specify the minimum weld size, because the weld size will be the thickness of the base material being joined. This is not the case, however, for PJP, groove welds, so the various codes typically specify minimum PJP groove weld sizes as well. When making CJP groove welds, it is a good practice to make certain that the individual passes applied to the groove meet or exceed the minimum weld size for PJP groove welds.

### 22.4 Principles of Design

Many welding-related problems have at their root a violation of basic design principles. For dynamically loaded structures, attention to detail is particularly critical. This applies equally to high-cycle fatigue loading, short duration abrupt-impact loading, and seismic loading. The following constitutes a review of basic welding engineering principles that apply to all construction.

### 22.4.1 Transfer of Forces

Not all welds are evenly loaded. This applies to weld groups that are subject to bending as well as those subject to variable loads along their length. The situation is less obvious when steels of different geometries are joined by welding. A rule of thumb is to assume the transfer of force takes place from one member, through the weld, to the member that lies parallel to the force that is applied. Some examples are illustrated in Figure 22.12. For most simple static loading applications, redistribution of stress throughout the member accommodates the variable loading levels. For dynamically loaded members, however, this is an issue that must be carefully addressed in the design. The addition of stiffeners or continuity plates to column webs helps to unify the distribution of stress across the groove weld.

### 22.4.2 Minimize Weld Volumes

A good principle of welded design is to always use the smallest amount of weld metal possible for a given application. This not only has sound economic implications, but it reduces the level of residual stress in the connection due to the welding process. All heat-expanded metal will shrink as it cools, inducing residual stresses in the connection. These tendencies can be minimized by reducing the volume of weld metal. Details that will minimize weld volumes for groove welds generally involve minimum root openings, minimum included angles, and the use of double-sided joints.

### 22.4.3 Recognize Steel Properties

Steel is not a perfectly isotropic material. The best mechanical properties usually are obtained in the same orientation in which the steel was originally rolled, called the X axis. Perpendicular to the X axis is the width of the steel, or the Y axis. Through the thickness, or the Z axis, the steel will exhibit the



FIGURE 22.12: Examples of transfer of force. (a) The leg welded under the beam has direct force transfer when oriented parallel to, and directly under, the beam web. (b) The same leg rotated  $90^{\circ}$  will result in an uneven distribution of stress along the weld length, unless stiffeners are added. The stiffeners could be triangular in shape, since the purpose is to provide a path for force transfer into the weld. (c) For hollow box sections, a lug attached perpendicular to the beam's longitudinal axis results in an unevenly loaded weld until an internal diaphragm is added. (d) Wrapping the lug around the outside of the box section permits it to be directly welded to the section that is parallel to the load, i.e., the vertical sides. (e) Side plates are added to this lug in order to provide a path for force transfer to the vertical sides of the box section. (Courtesy of The Lincoln Electric Company. With permission.)

least amount of ductility, lowest strength, and lowest toughness properties. It is always desirable, if possible, to allow the residual stresses of welding to elongate the steel in the X direction. Of particular concern are large welds placed on either side of the thickness of the steel where the weld shrinkage stress will act in the Z axis. This can result in lamellar tearing during fabrication, or under extreme loading conditions, can result in subsurface fracture.

### 22.4.4 Provide Ample Access for Welding

It is essential that the design provide adequate access for both welder and welding equipment, as well as good visibility for the welder. As a general rule, if the welder cannot see the joint, neither can the inspector; weld quality will naturally suffer. It is important that adequate access be provided for the proper placement of the welding electrode with respect to the joint. This is a function of the welding gas nozzle into the weld joint. Overall access to the joint is a function of the configuration of the surrounding material. The prequalified groove weld details listed in AWS D1.1-96 [9] take these issues into consideration.

### 22.4.5 No Secondary Members in Welded Design

A fundamental premise of welding design is that there are no secondary members. Anything that is joined by welding can, and will, transfer stress between joined materials. For instance, segmented pieces of steel used for weld backing can result in a stress concentration at the interface of the backing. Attachments that are simply tack welded in place may become major load-carrying members, resulting in the initiation of fracture and propagation throughout the structure. These details must be considered in the design phase of every project, and also controlled during fabrication and erection.

### 22.4.6 Residual Stresses in Welding

As heat-expanded weld metal and the surrounding base metal cool to room temperature, they shrink volumetrically. Under most conditions, this contraction is restrained or restricted by the surrounding material, which is relatively rigid and resists the shrinkage. This causes the weld to induce a residual stress pattern, where the weld metal is in residual tension and the surrounding base metal is in residual compression. The residual stress pattern is three dimensional since the metal shrinks volumetrically. The residual stress distribution becomes more complex when multiple-pass welding is performed. The final weld pass is always in residual tension, but subsequent passes will induce compression in previous weld beads that were formerly in tension.

For relatively flexible assemblages, these residual stresses induce distortion. As assemblages become more rigid, the same residual stresses can cause weld cracking, typically occurring shortly after fabrication. If distortion does not occur, or when cracking does not occur, the residual stresses do not relieve themselves, but are "locked in". Residual stresses are considered to be at the yield point of the material involved. Because any area that is subject to residual tensile stress is surrounded by a region of residual compressive stress, there is no loss in overall capacity of as-welded structures. However, this reduces the fatigue life for low-stress-range, high-cycle applications.

Small welded assemblies can be thermally stress relieved by heating the steel to 1150°F, holding it for a predetermined length of time (typically 1 h/in. of thickness), and allowing it to return to room temperature. Residual stresses can be reduced by this method, but they are never totally eliminated. This approach is not practical for large assemblies, and care must be exercised to ensure that the components being stress relieved have adequate support when at the elevated temperature, where the yield strength and the modulus of elasticity are greatly reduced, as opposed to room temperature properties. For most structural applications, residual stresses cause no particular problem to the

performance of the system, and due to the complexity of stress relief activities, welded structures commonly are used in the as-welded condition.

When loads are applied to as-welded structures, there is some redistribution or gradual decrease in the residual stress patterns. Usually called "shake down", the thermal expansion and contraction experienced by a typical structure as it goes through a climatic season, as well as initial service loads applied to the building, result in a gradual reduction in the residual stresses from welding.

These residual stresses should be considered in any structural application. On a macro level, they will affect the erector's overall sequence of assembling of a building. On a micro level, they will dictate the most appropriate weld bead sequencing in a particular groove-welded joint. For welding applications involving repair, control of residual stresses is particularly important, since the degree of restraint associated with weld repair conditions is inevitably very high. Under these conditions, as well as applications involving heavy, highly restrained, very thick steel for new construction, the experience of a competent welding engineer can be helpful in avoiding the creation of unnecessarily high residual stresses.

### 22.4.7 Triaxial Stresses and Ductility

The commonly reported values for ductility of steel generally are obtained from uniaxial tensile coupons. The same degree of ductility cannot be achieved under biaxial or triaxial loading conditions. This is particularly significant since residual stresses are always present in any as-welded structure. A more detailed discussion on this subject is found in Section 22.7.

### 22.4.8 Flat Position Welding

Whenever possible, weld details should be oriented so that the welding can be performed in the flat position, taking advantage of gravity, which helps hold the molten weld metal in place. Flat position welds are made with a lower requirement for operator skill, and at the higher deposition rates that correspond to economical fabrication. This is not to say, however, that overhead welding should be avoided at all costs. An overhead weld may be advantageous if it allows for double-sided welding, with a corresponding reduction in the weld volume. High-quality welds can be made in the vertical plane, and, with the welding consumables available today, can be made at an economical rate.

### 22.5 Welded Joint Details

### 22.5.1 Selection of Fillet vs. PJP Groove Welds

For applications where either fillet welds or PJP groove welds are acceptable, the selection is usually based on cost. A variety of factors must be considered in order to determine the most economical weld type.

For welds with equal throat dimensions, the PJP configuration requires one-half the volume of weld metal required by the fillet weld. Alternatively, for equal weld metal volumes, the PJP option is approximately 40% stronger than the fillet weld. Additional factors must be considered, however.

For PJP welds, the bevel surface must be prepared prior to welding, increasing joint preparation cost. Typically achieved by flame cutting, this additional operation requires fuel gas, oxygen, and, most costly of all, labor.

In general, fillet welds are the easiest welds to produce. Access into the more narrow included angles of groove welds usually requires more careful control of welding parameters, commonly resulting in slower welding speeds. The root pass of a PJP groove weld, made into a joint with no root opening, necessitates sufficient included groove angles to avoid centerline cracking tendencies due to poor cross-sectional bead shape. Slag removal may be difficult in root passes as well. These problems do not exist in fillet welds when applied to  $90^{\circ}$  intersections of T joint members. Such issues can be of concern for skewed T joints, particularly when the acute angle side is less than  $60^{\circ}$ .

Typical shop practices have generated a general rule of thumb suggesting that fillet welds are the most cost-effective details for connections requiring throats of 1/2 in. or less, which equates to a leg size of 3/4 in. PJP groove welds are generally the best choice for throat sizes of 3/4 in. or greater. This would roughly equate to a 1-in. fillet weld. In general, fillet welds should not exceed 1-in., nor should PJP groove welds be specified for throat dimensions less than 1/2 in. Between these boundaries, specific shop practices will determine the most economical approach.

#### 22.5.2 Weld Backing

When there is a gap between two members to be joined, it is difficult to bridge the space with weld metal. On the other hand, when two members are tightly abutted to each other, it is difficult to obtain complete fusion. To overcome these problems, weld backing is added behind the members to act as a support for the weld metal (Figure 22.13). Weld backing fits into one of two categories: fusible-permanent steel backing or removable backing.



FIGURE 22.13: Weld backing. (Courtesy of The Lincoln Electric Company. With permission.)

### 22.5.3 Fusible Backing

Fusible steel backing, commonly known as backing bars, becomes part of the final structure when left in place, so steel that would meet quality requirements for primary members should be used for backing. In general, however, notch toughness properties are not specified for backing. The backing must be continuous for the length of the joint. If multiple pieces of steel backing are to be used in a single joint, they must be joined with CJP groove welds before being applied to the joint they are to back. Welds joining segments of backing bars should be inspected with radiography or ultrasonography to ensure soundness. Interrupted backing bars have been the source of fracture, as well as fatigue crack initiation, and are unacceptable.

For building construction, steel backing is frequently used to compensate for dimensional variations that inevitably occur under field conditions. To maintain plumb columns, there will be slight variations in the dimensions between the columns in a bay. Since the beams are cut to length before the exact dimensions are known, an oversized gap will often result between the beam and the column. Steel backing is inserted underneath this gap, and weld metal is used to bridge this space. It is important to remember, however, that the steel backing becomes part of the final structure if it is left in place.

### 22.5.4 Removable Backing

Removable backing includes fiberglass tapes, ceramic tiles, and fluxes attached to flexible tape. Removable backing generally is applied when the joint is to be welded with an open arc process such as flux core or shielded metal arc welding. Such backing is applied to the joint with some type of adhesive before the joint is welded. Upon completion of welding, the temporary backing is removed.

Removable backing may be less costly for the fabricator than using the alternatives of doublesided joints or fusible backing. A major obstacle in the use of many of these types of backing is the adhesive that holds the material in place. This is particularly a concern when preheat is required. In some situations, mechanical means have been used to assist in holding the backing in place. When supports are attached by tack welds, care should be exercised to ensure that appropriate techniques are employed.

### 22.5.5 Copper Backing

Another type of removable backing would be a copper chill bar placed under the joint. Because of the high thermal conductivity of copper, the large difference in melting points of copper versus steel, and physical and chemical differences between the metals, molten weld metal can be supported by copper and the two materials rarely fuse together. This makes copper an attractive material to use for weld backing.

However, this practice is discouraged or prohibited by many codes, because of the possibility of the arc impinging itself on the copper and drawing some of the melted copper into the weld metal. Copper promotes centerline cracking. This would, of course, be unacceptable. As a practical matter, fabricators avoid this practice simply because the copper backing is extremely expensive, and is rapidly ruined when the arc melts a portion of the copper. Copper backing can be used successfully under controlled conditions, which generally involve mechanized welding and joints that do not utilize root openings.

In some situations, the fabricator will mill a groove in a copper chill bar, and fill the groove with clean, dry submerged arc flux. The flux then acts as the backing, and ensures the arc does not melt any of the copper. This is an efficient method and does not have the same ramifications as welding directly against copper. To ensure tight fit of the copper to the back of the joint, pneumatic, mechanical, or hydraulic pressure may be applied to achieve close alignment. Any temporary welds made to attach the backing system to the structural member must employ appropriate welding techniques.

### 22.5.6 Weld Tabs

Weld tabs, commonly known as starting and run off tabs, are added to the ends of joints in order to facilitate quality welding for the full length of the joint. The start and finish ends of weld beads are known to be more defect prone than the continuous weld between these points. Under starting conditions, the weld pool must be established, adequate shielding developed, and thermal equilibrium established. At the termination of a weld, the crater experiences rapid cooling with the extinguishing arc. Shielding is reduced. Cracks and porosity are more likely to occur in craters than at other points of the weld. Starts and stops can be placed on these extension tabs and subsequently removed upon the completion of the weld (see Figure 22.14).

It is preferable to attach the weld tabs by tack welding within the joint (in Figure 22.14, notice the



FIGURE 22.14: Examples of weld tabs. (Courtesy of The Lincoln Electric Company. With permission.)

tack welds in the third example). Preheat requirements must be met when attaching weld tabs, unless the production weld is made with the submerged arc welding process, which will remelt these zones. It is important for weld tabs to have the same geometry as the weld joint to ensure the full throat or plate thickness dimension is maintained at the ends of the weld joint.

When a weld tab containing weld metal of questionable quality is left in place, a fracture can initiate in these regions and propagate along the length of the weld. Weld tabs are removed for bridge fabrications, and since 1989, weld tab removal has been required by American Institute of Steel Construction (AISC) specifications when "jumbo" sections or heavy built-up sections are joined in tension applications by CJP groove welds.

### 22.5.7 Weld Access Holes

Weld access holes are provided in the web of beam sections to be joined to columns. The access hole in the upper flange connection permits the application of weld backing. The lower weld access hole permits access for the welder to make the bottom flange groove weld. AISC and AWS prescribe minimum weld access hole sizes for these connections ([9], para. 5.17, Figure 5.2). It must be emphasized that these minimum dimensions can be increased for specific requirements necessitated by the weld process, overall geometry, etc. However, the designer must be certain that the resultant section loss is acceptable.

In order to provide ample access for electrode placement, visibility of the joint, and effective cleaning of the weld bead, it is imperative to provide adequate access. In addition to offering access for welding operations, properly sized weld access holes provide an important secondary function: they prevent the interaction of the residual stress fields generated by the vertical weld associated with the web connection and the horizontal weld between the beam flange and column face. The weld access hole acts as a physical barrier to preclude the interaction of these residual stress fields, which

can result in cracking. It is best for the weld access hole to terminate in an area of residual compressive stress [21]. More ductile behavior can be obtained under these conditions.

Weld access holes must be properly made. Nicks, gouges, and other geometric discontinuities can act as stress raisers, increasing local stress levels and acting as points of fracture initiation. AISC requires that weld access holes be ground to a bright finish on applications where tension splices are applied to heavy sections. Although not mandated by the codes, these requirements for tension members may be needed for successful fabrication of compression members when connection details typically associated with tension members are applied to compression members (e.g., CJP groove welds) [22].

### 22.5.8 Lamellar Tearing

Lamellar tearing is a welding-related type of cracking that occurs in the base metal. It is caused by the shrinkage strains of welding acting perpendicular to planes of weakness in the steel. These planes are the result of inclusions in the base metal that have been flattened into very thin plates that are roughly parallel to the surface of the steel. When stressed perpendicular to the direction of rolling, the metallurgical bonds across these plates can separate. Since the various plates are not on the same plane, a fracture may jump between the plates, resulting in a stair-stepped pattern of fractures, illustrated in Figure 22.15. This type of fracture generally occurs near the time of fabrication, and can be confused with underbead cracking.



FIGURE 22.15: Lamellar tearing. (Courtesy of The Lincoln Electric Company. With permission.)

Several approaches can be taken to overcome lamellar tearing. The first variable is the steel itself. Lower levels of inclusions within the steel will help mitigate this tendency. This generally means lower sulfur levels, although the characteristics of the sulfide inclusion are also important. Manganese sulfide is relatively soft, and when the steel is rolled at hot working temperatures of 1600–2000°F, the sulfide inclusions flatten significantly. If steel is first treated to reduce the sulfur, and then calcium treated, for example, the resultant sulfide is harder than the surrounding steel, and during the rolling process, is more likely to remain spherical. This type of material will have much less of a tendency toward lamellar tearing.

Current developments in steel-making practice have helped to minimize lamellar tearing tendencies. With continuously cast steel, the degree of rolling after casting is diminished. The reduction in the amount of rolling has directly affected the degree to which these laminations are flattened, and has correspondingly reduced lamellar tearing tendencies.

The second variable involves the weld joint design. For a specific joint detail, it may be possible to alternate the weld joint to minimize lamellar tearing tendencies. For example, on corner joints it

is preferred to bevel the member in which lamellar tearing would be expected, that is, the plate that will be strained in the through-thickness direction. This is illustrated in Figure 22.16.



FIGURE 22.16: Lamellar tearing. (Courtesy of The Lincoln Electric Company. With permission.)

A reduction in the volume of weld metal used will help to reduce the stress that is imposed in the through-thickness direction. For example, a single bevel groove weld with a 3/8-in. root opening and  $30^{\circ}$  included angle will require approximately 22% less weld metal for a 1-1/2-in.-thick plate, compared to a 1/4-in. root opening and a  $45^{\circ}$  joint. The corresponding reduction in shrinkage stresses may be sufficient to eliminate lamellar tearing.

In extreme cases, it may be necessary to resort to special measures to minimize lamellar tearing, which may involve peening. This technique involves the mechanical deformation of the weld surface, which results in compressive residual stresses that minimize the magnitude of the residual tensile stresses that naturally occur after welding. In order for peening to be effective, it is generally performed when the weld metal is warm (above 300°F), and must cause plastic deformation of the weld surface. Peening is restricted from being applied to root passes (because the partially completed weld joint could easily crack), as well as final weld layers, because the peening can inhibit appropriate visual weld inspection and embrittle the weld metal, which will not be reheated ([9], para. 5.27).

Another specialized technique that can be used to overcome lamellar tearing tendencies is the "buttering layer" technique. With this approach, the surface of the steel where there might be a risk of lamellar tearing is milled to produce a slight cavity in which the butter layer can be applied. Individual weld beads are placed into this cavity. Since the weld beads are not constrained by being attached to a second surface, they solidify and cool, and thereby shrink, with a minimum level of applied stress to the material on which they are placed. After the butter layer is in place, it is possible to weld upon that surface with much less concern about lamellar tearing. This concept is illustrated in Figure 22.17.

Lamellar tearing tendencies are aggravated by the presence of hydrogen. When such tendencies are encountered, it is important to review the low hydrogen practice, examining the electrode selec-

tion, care of electrodes, application of preheat, and interpass temperature. Additional preheat can minimize lamellar tearing tendencies.



FIGURE 22.17: "Buttered" surface. (Courtesy of The Lincoln Electric Company. With permission.)

## 22.6 Design Examples of Specific Components

To demonstrate the design principles of welded connections, five examples are presented. The objective of each example is to determine either the weld leg size or the weld length. These are representative of several beam-to-column design concepts. For further details and examples consult [20].

### 22.6.1 Flexible Seat Angles



determine maximum unit *horizontal force on weld*  $(F_n)$ 

$$M = \frac{R}{2}e_f = \frac{2}{3}L_vP$$
 also  $P = \frac{1}{2}F_n\frac{2}{3}L_v$ 

from this

$$F_n = \frac{2.25 Re_r}{L_v^2}$$

unit vertical force on weld

$$F_v = \frac{R}{2L_v}$$

resultant unit force on weld (at top)

$$f_r = \sqrt{f_n^2 + f_v^2} = \sqrt{\left(\frac{2.25Re_r}{L_v^2}\right)^2 + \left(\frac{R}{2L_v}\right)^2} = \frac{R}{2L_v^2}\sqrt{L_v^2 + 20.25e_f^2}$$

leg size of fillet weld

$$W = \frac{R\sqrt{L_v^2 + 20.25e_f^2}}{2L_v^2(.707)(.30EXX)}$$

### 22.6.2 Stiffened Seat Brackets



In this particular connection, the shear reaction is taken as bearing through the lower flange of the beam. There is no welding directly on the web. For this reason it cannot be assumed that the web can be stressed up to its yield in bending throughout its full depth. Since full plastic moment cannot be assumed, the bending stress allowable is held to  $\sigma = .60\sigma_y$ , or 22 ksi. AISC Sect. 1.5.1.4.1.

Check the bending stress in the beam:

$$\sigma = \frac{M}{S} = \frac{1100 \text{ in.-kip}}{54.7 \text{ in.}} = 20.1 \text{ ksi} < .60\sigma_y \text{ or } 22 \text{ ksi} \text{ OK}$$

Bending force in the connection plate:

$$F = \frac{M}{d} = \frac{1100 \text{ in.-kip}}{14.12 \text{ in.}} = 78.0 \text{ kip}$$

Area of the top connection plate:

$$A_p = \frac{F}{\sigma} = \frac{78.0 \text{ kip}}{22 \text{ ksi}} = 3.54 \text{ in.}^2$$

or use a 5 × 3 in. plate which gives a value of  $A_p = 3.75$  in.<sup>2</sup> > 3.54 in.<sup>2</sup> OK If a 3/8 in. fillet weld is used to connect top plate to upper beam flange:

$$f_w = (.707)(3/8 \text{ in.})(21 \text{ ksi}) = 5.56 \text{ kips for linear inch of weld}$$

Length of fillet weld:

$$L = \frac{F}{f_w} = \frac{78.0 \text{ kip}}{5.56 \text{ kip/in.}} = 14.1 \text{ in.}$$

or use 5 in. across the end of the plate, and 5 in. along each side, a total length of 15 in. > 14.1 in. OK

### 22.6.3 Web Framing Angles

Web Framing Angles to Beam (usually shop weld)



Twisting

(horizontal) 
$$F_n = \frac{JC_v}{J_w}$$
  
(vertical)  $F_{v1} = \frac{JC_n}{J_w}$ 

Shear

(vertical) 
$$F_{v2} = \frac{R}{2(2b - L_v)}$$

Resultant force

$$F_r = \sqrt{F_n^2 - (F_{v1} + F_{v2})^2}$$

Leg size of fillet weld

$$W = \frac{F_r}{.707(.30EXX)}$$



### 22.6.4 Top Plate Connections

The welding of the flanges and nearly full depth of the web would allow the beam to develop its full plastic moment. This will allow the "compact" beam to have a 10% higher bending allowable, or  $\sigma = .66\sigma_y$ . This also allows the end of the beam, and its welded connection, to be designed for 90% of end moment due to gravity loading. AISC Sect. 1.5.1.4.1.

Check the bending stress in the beam:

$$\sigma = \frac{.9M}{S} = \frac{.9(1100 \text{ in.-kip})}{41.9 \text{ in.}^3} = 23.6 \text{ ksi} < .66\sigma_y \text{ or } 24 \text{ ksi}$$
 OK

Bending force in the top connecting plate:

$$F = \frac{.9M}{d} = \frac{.9(1100 \text{ in.-kip})}{13.86 \text{ in.}} = 71.5 \text{ kip}$$

Area of top connection plate:

$$A_p = \frac{F}{\sigma} = \frac{71.5 \text{ kip}}{24 \text{ ksi}} = 2.98 \text{ in.}^2$$

Or use a 5-1/2 in. by 5/8-in. plate,  $A_p = 3.44$  in.<sup>2</sup> > 2.98 in.<sup>2</sup> OK

If a 3/8-in. fillet weld is used to connect the top plate to the upper beam flange:

 $f_w = (.707)(3/8 \text{ in.})(21 \text{ ksi}) = 5.56 \text{ kip per linear inch of weld}$ 

Length of fillet weld

$$L = \frac{F}{f_w} = \frac{71.5 \text{ kip}}{5.56 \text{ kip/in.}} = 12.9 \text{ in.}$$

or use 5-1/2 in. across the end of the plate end 4 in. along each side, a total length of 13-1/2 in.

The lower flange of the beam is butt welded directly to the flange of the column. Since the web angle carries the shear reaction, no further work is required on this lower portion of the connection. The seat angle simply serves to provide temporary support for the beam during erection and a backing for the flange groove butt weld.

### 22.6.5 Directly Connected Beam-to-Column Connections

Design a fully welded beam-to-column connection for a W14x30 beam a W8x31 column to transfer an end moment of M = 1000 in.-kips, and a vertical shear of V = 20 kips. This example will be considered with several variations. Use A36 steel and E70 filler metal.



The welding of the flanges and full depth of the web would allow the beam to develop its full plastic moment. This will allow the "compact" beam to have a 10% higher bending moment, or  $\sigma = .66\sigma_{\gamma}$ . This also allows the end of the beam, and its welded connection, to be designed for 90% of the end moment due to gravity loading. AISC Sect. 1.5.1.4.1.

actual 
$$\frac{b}{2t_f} = \frac{6.733}{2(.383)} = 8.79$$
 AISC allowable  $\frac{65}{\sqrt{\sigma_y}} = \frac{65}{\sqrt{36}} = 10.83$  OK  
actual  $\frac{\sigma}{t} = \frac{13.86 - 2(.383)}{.290} = 48.5$  AISC allowable  $\frac{d}{t} = \frac{64D}{\sigma_y} \left(1 - 3.74\frac{\sigma_x}{\sigma_y}\right)$   
 $= 106.7$  OK

hence this beam has a "compact" section.

$$\sigma = \frac{.9M}{S} = \frac{.9(1100 \text{ in.-kip})}{41.9 \text{ in.}^3} = 23.63 \text{ ksi} < .66\sigma_y \text{ or } 24 \text{ ksi} \text{ OK}$$

The weld on the web must be able to stress the web in bending to yield  $(\sigma_{v})$  throughout its depth (see the bending stress distribution above). Unit force this weld:

$$f_w = \frac{V}{2L} = \frac{20 \text{ kip}}{2[13.86 - (2 \times .383)]} = .764 \text{ kip per linear inch$$

Leg size of fillet weld:

$$w = \frac{.764 \text{ kip/in.}}{.707(21 \text{ ksi})} = .05 \text{ in.}$$

However, this is welded to a .433-in.-thick flange of the column, so the minimum fillet weld size for this would be 3/16 in.

### 22.7 Understanding Ductile Behavior

"Ductility" can mean different things to different people. Materials such as cast iron are generally considered "brittle", while steel is called "ductile". A physical metallurgist may talk in terms of cleavage and ductile dimpling to define material behavior on a microscopic level. This is of little benefit to the structural engineer, who is more concerned with global deformation than microscopic behavior. Global deformation would include buckling, plastic hinge formation, stretching of members, and other inelastic behaviors that are visually observable. To achieve ductile behavior, the structural engineer, will select ductile materials for construction. To assume, however, that global deformation will occur simply because a ductile material has been selected can lead to unexpected brittle fracture of even ductile materials.

It is essential, therefore, that global behavior be separated from microscopic behavior. A material that fails by low-energy cleavage fracture cannot be made to function in a globally ductile manner, although it is possible for a structural element to fail with little or no deformation, and yet the fracture surface would exhibit the characteristics of ductile dimpling. Microscopic ductility and global ductility are separate issues, and the structural engineer must understand what conditions lead to global ductile behavior. This is particularly important where welding is applied, since welding introduces residual stresses and geometric influences that can affect the achievement of ductile behavior.

For global ductility to be possible, the following conditions must be achieved:

- 1. There must be a shear stress component  $(\tau)$  that results from the applied load.
- 2. The shear stress must be of sufficient magnitude so as to exceed the critical shear stress of the material.
- The shear stress must result in an inelastic shear strain that acts in a direction to relieve the particular stress that is applied.
- 4. There must be a sufficient length of unrestrained material to permit a reduction in the cross-sectional area (i.e., to allow for "necking" to occur).

These conditions are met in the specimens typically used to measure ductility of steels. As illustrated in Figure 22.18, the preceding four principles will be applied to the uniaxial tensile specimen. In Figure 22.18a, the specimen has been stretched to a point so that the resultant stresses,  $\sigma_1$ , are below the yield point,  $\sigma_y$ . The stress,  $\sigma_1$ , has caused a shear stress,  $\tau_{1-2}$ , that acts on a 45° plane to the applied stress. Rather than focusing on  $\sigma_1$  being less than  $\sigma_y$ , it is better to realize the resultant shear stress,  $\tau_{1-2}$ , is less than the critical shear stress,  $\tau_{cr}$ . A Mohr's Circle diagram assists in visualizing this behavior. Once  $\tau_{cr}$  is exceeded, slippage along shear planes can occur, resulting in elongation. In Figure 22.18a, a shear stress has resulted from the applied stress (i.e., condition 1 from above was achieved), but the shear stress is not sufficient to exceed the critical value (i.e., condition 2 has not been achieved). All behavior under these conditions would be elastic, and although brittle fracture would not occur, neither would ductile behavior be achieved.

In Figure 22.18b, the load, F, has been increased so that the resultant shear,  $\tau_{1-2}$ , exceeds  $\tau_{cr}$ , resulting in slip on the plane oriented at 45°. This slip results in elongation, or stretching of the member. Global ductility is seen. This behavior occurs because condition 2 has been achieved. Figure 22.18c illustrates the continued application of force, resulting in slip occurring on multiple planes, eventually resulting in a reduction in cross-section, or necking. This is possible because all four conditions have been achieved.

A further increase in load, illustrated in Figure 22.18d, causes the critical tensile strength,  $\sigma_t$ , to be exceeded. The sample eventually breaks, and the final fracture surface exhibits little deformation. This occurs because, due to the localized deformations occurring in the necking region, the stresses in the other two principle directions are no longer zero. This is triaxial stress, and as illustrated in the



FIGURE 22.18: Shear stresses in simple, uniaxial tensile specimens. (Courtesy of The Lincoln Electric Company. With permission.)

Mohr's Circle, there is a resultant decrease in the shear stress. No longer is condition 1 maintained, and brittle fracture (in a global sense) occurs across the necked region.

These sample principles can be applied to various connection details. Consider, for example, the taper required for tension members that have thickness or width transitions. As seen in Figure 22.19, the sharp 90° transition results in a biaxial stress state near the transition. While ductile behavior could occur in the area where uniaxial stress exists, the second stress will reduce the shear stress, reducing its ductility capacity. The tapered transition allows for essentially uniaxial stresses to be maintained through the transition range, encouraging shear stresses capable of producing ductile behavior.

### 22.7.1 Two Residual Stresses Isolated

Figure 22.20 illustrates that two important residual stresses exist in the weld access hole's termination zone. This butt joint in the flange has a residual stress,  $\sigma_3$ , longitudinal to the length of the flange, as well as a stress transverse to the flange,  $\sigma_1$ . The longitudinal stress is tensile along the center line of the flange where the weld access hole terminates. It can be compared to tightening a steel cable lengthwise in the center in tension, with compression spread out on both sides. The transverse stress,  $\sigma_1$ , is positive (tensile) in the weld zone, as well as in an adjacent portion of the plate going through zero, and then compression. Beyond the adjacent plate, it becomes zero and then negative (compression). This transverse stress,  $\sigma_1$ , is also similar to tightening a steel cable.







FIGURE 22.20: Resultant residual stress of welding. (Courtesy of The Lincoln Electric Company. With permission.)

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#### 22.7.2 Residual Stresses Applied

These residual stresses may be applied to a weld detail having a narrow weld access hole, as shown in Figure 22.21. This hole terminates at a point where  $\sigma_1$  and  $\sigma_3$  are in tension. Since the web at the edge of the weld access hole offers some restraint against movement in the through-thickness direction of the flange plate, stress in the  $\sigma_2$  direction may have an appreciable value. All of the circles will be small. Neither  $\tau_{2-3}$  nor  $\tau_{1-3}$  will probably ever reach the critical shear stress value, and plastic strain or ductility will not occur, as the lower portion of Figure 22.21 illustrates.



FIGURE 22.21: Mohr's circle of stress for element 2. (Courtesy of The Lincoln Electric Company. With permission.)

If the weld access hole can be cut with circular ends, sometimes called a pear-shaped opening, the stress,  $\sigma_2$ , in the through thickness of the flange plate will be greatly reduced, probably to zero in this critical section, as shown in Figure 22.22. This will produce a very large circle with  $\sigma_3$  and the resulting shear stress,  $\tau_{2-3}$ , will be very high — high enough to exceed the critical value well before  $\sigma_3$  reaches its critical value for failure. This would result in a more ductile behavior.

If the weld access hole can be made wider, so that it terminates in a zone where the transverse residual stress,  $\sigma_1$ , is compressive (see Figure 22.23), then a more favorable stress condition will result in greater ductility in the  $\sigma_3$  direction. In this case, shear stress,  $\tau_{1-3}$ , will be high as shown on Mohr's Circle of stress, and the critical shear value will be reached at a much lower tensile stress or load value. This will produce more ductility in the  $\sigma_3$  direction, greatly reducing the chance of a transverse crack in the flange at the termination of the weld access hole.

If a pear-shaped wide weld access hole is used, and the through-thickness stress,  $\sigma_2$ , becomes zero, it simply increases the shear stress  $\tau_{2-3}$  and would seem to improve ductility (see Figure 22.24). However, looking at the resulting stress-strain curve of the flange plate at the termination of the weld access hole, it appears that rounding the ends of the wide access hole in this case does not appreciably increase the ductility. This is probably because the wide weld access hole already has excellent ductility.



FIGURE 22.22: Mohr's circle of stress for element 3. (Courtesy of The Lincoln Electric Company. With permission.)



FIGURE 22.23: Mohr's circle of stress for element 4. (Courtesy of The Lincoln Electric Company. With permission.)

Figure 22.25 shows stress-strain curves of the four different weld access hole details just discussed. The principles outlined herein can be applied to other details, evaluating the potential of biaxial or triaxial stresses and their effect on shear stress development. Consideration of these principles can assist in avoiding brittle fracture by encouraging ductile behavior.

Special Considerations for Welded Structures Subject to Seismic Loading



FIGURE 22.24: Mohr's circle of stress for element 5. (Courtesy of The Lincoln Electric Company. With permission.)



FIGURE 22.25: Stress-strain curves of four different weld access hole details. (Courtesy of The Lincoln Electric Company. With permission.)

### 22.7.3 Unique Aspects of Seismically Loaded Structures

### **Demands on Structural Systems**

During an earthquake, even structures specifically designed for seismic resistance are subject to extreme demands. Any structure designed with a response modification factor,  $R_w$ , greater than unity will be loaded beyond the yield stress of the material. This is far more demanding than other anticipated types of loading. Due to the inherent ductility of steel, stress concentrations within a

steel structure are gradually distributed by plastic deformation. If the steel has a moderate degree of notch toughness, this redistribution eliminates localized areas of high stress, whether due to design, material, or fabrication irregularities. For statically loaded structures, the redistribution of stresses is relatively inconsequential. For cyclically loaded structures, repetition of this redistribution can lead to fatigue failure. In seismic loading, however, it is expected that portions of the structure will be loaded well beyond the elastic limit, resulting in plastic deformation. Localized areas of high stress will not simply be spread out over a larger region by plastic deformation. The resultant design, details, materials, fabrication, and erection must be carefully controlled in order to resist these extremely demanding loading conditions.

#### **Demand for Ductility**

Seismic designs have relied on ductility to protect structures during earthquakes. Unfortunately, much confusion exists regarding the measured property of ductility in steel, and ductility can be experienced in steel configured in various ways. It is essential that a fundamental understanding of ductility be achieved in order to ensure ductile behavior in the steel in general, and particularly in the welded connections.

#### **Requirements for Efficient Welded Structures**

Five elements are present in any efficient welded structure:

- Good overall design
- Good materials
- Good details
- Good workmanship
- Good inspection

Each element is important, and emphasis on one will not overcome deficiencies in others. Both the Northridge earthquake in 1994 and the Kobe earthquake in 1995 showed that deficiencies in one or more of the preceding areas may have contributed to the degredation in performance of Steel Moment-Resisting Frames (SMRFs).

### 22.8 Materials

#### 22.8.1 Base Metal

Base metal properties are particularly important in structures subject to seismic loading. Unlike most static designs, seismically resistant structures depend on acceptable material behavior beyond the elastic limit. The basic premise of seismic design is to absorb seismic energies through yielding of the material. For static design, additional yield strength capacity in the steel may be desirable, but for applications where yielding is the desired method for achieving energy absorption, higher than expected yield strengths may have a dramatic negative effect. This is especially important as it relates to connections, both bolted and welded.

Figure 22.26 illustrates five material zones that occur near the groove weld in a beam-to-column connection. If it is assumed that the web is incapable of transferring any moment, it is essential that the plastic section modulus of the flanges  $(Z_f)$  times the tensile strength be greater than the entire plastic section property (Z) times the yield strength in the beam. All five material properties must be considered in order for the connection to behave satisfactorily. Note that this was the standard connection detail used for special moment-resisting frame (SMRF) systems prior to the Northridge earthquake.



FIGURE 22.26: Five material zones that occur near the groove weld in a beam-to-column connection. (Courtesy of The Lincoln Electric Company. With permission.)

Current American Society for Testing and Materials (ASTM) specifications do not place an upper limit on the yield strength for most structural steels, but specify only a minimum acceptable value. For instance, for ASTM A36 steel, the minimum acceptable yield strength is 36 ksi. This precludes a steel that has a yield strength of 35.5 ksi as being acceptable, but does nothing to prohibit the delivery of a 60-ksi steel. The tensile strength range is specified as 58–80 ksi. Although A36 is commonly specified for beams, columns are typically specified to be of ASTM A572 grade 50. With a 50-ksi minimum yield strength and a minimum tensile strength of 65 ksi, many designers were left with the false impression that the yield strength of the beam could naturally be less than that of the column. Due to the specification requirements, it is possible to produce steel that meets the requirements of both A36 and A572 grade 50. This material has been commercially promoted as "dual-certified" material. However, no matter what the material is called, it is critical for the connection illustrated in Figure 22.26 to have controls on material properties that are more rigorous than the current ASTM standards impose.

Much of the focus of post-Northridge research has related to the beam yield-to-tensile ratio, commonly denoted as  $F_y/F_u$ . This is often compared to the ratio of  $Z_f/Z$ , with the desired relationship being

$$\frac{Z_f}{Z} > \frac{F_y}{F_u}$$

This suggests that not only is  $F_y$  (yield strength) important, but the ratio is important as well. For rolled W shapes,  $Z_f/Z$  ranges from 0.6 to 0.9. Based on ASTM *minimum* specified properties,  $F_y/F_u$  is as follows:

However, when actual properties of the steel are used, this ratio may increase. In the case of one building damaged in Northridge, mill test reports indicated the ratio to be 0.83.

ASTM steel specifications need further controls to limit the upper value of acceptable yield strengths for materials as well as the ratio of  $F_y/F_u$ . A new ASTM specification has been proposed to address these issues, although its approval will probably not be achieved before 1997.

In Figure 22.26, five zones have been identified in the area of the connection, with the sixth material property being located in the beam. Thus far, only two have been discussed: the beam yield strength and the beam ultimate strength. These are designated with the subscript X to indicate that these are

the properties in the orientation of the longitudinal axis of the beam. When the beam is produced, the longitudinal direction is considered the "direction of rolling". In general, steel exhibits its best mechanical properties in this orientation. When this axis is designated as the X axis, the width of the beam would be known as the Y direction and the Z axis is through the thickness of the flanges. For beam properties, the X axis is the one of interest.

The properties of interest with respect to the column are oriented in the column Z axis, which will exhibit the least desirable mechanical properties. Current ASTM specifications do not require measurement of properties in this orientation. While there are ASTM standards for the measurement of through-thickness properties (ASTM 770), these are not normally applied for structural applications. It is this through-thickness strength, however, that is important to the performance of the connection.

Notch toughness is defined as the ability of a material to resist propagation of a preexisting cracklike flaw while under tensile stress. Pre-Northridge specifications did not include notch toughness requirements for either base materials or weld metals. When high loads are applied, and when notch-like details or imperfections exist, notch toughness is the material property that resists crack propagation from that discontinuity. Rolled shapes routinely produced today, specifically for lighter weight shapes in the group 1, 2, and 3 categories, generally are able to deliver a minimum notch toughness of 15 ft.-lb at 40°F. This is probably adequate toughness, although additional research should be performed in this area. For heavy columns made of group 4 and 5 shapes, this level of notch toughness may not be routinely achieved in standard production.

After Northridge, many engineers began to specify the supplemental requirements for notch toughness that are invoked by AISC specifications for welded tension splices in jumbo sections (group 4 and 5 rolled shapes). This requirement for 20 ft.-lb at 70°F is obtained from a Charpy V-notch specimen extracted from the web/flange interface, an area expected to have the lowest toughness in the cross-section of the shape. Since columns are not designed as tension members under most conditions, this requirement would not automatically be applied for column applications. However, as an interim specification, it seems reasonable to ensure minimum levels of notch toughness for heavy columns also.

#### 22.8.2 Weld Metal Properties

Significant properties of weld metal are yield strength, tensile strength, toughness, and elongation. These properties usually may be obtained from data on the particular filler metal that will be employed to make the connection. The American Welding Society (AWS) filler metal classification system defines the minimum acceptable properties for the weld metal when deposited under very specific conditions. Most "70" series electrodes (e.g., E7018, E70T-1, E70T-6) have a minimum specified yield strength of 58 ksi and a minimum tensile strength of 70 ksi. As in the specifications for steel, there are no upper limits on the yield strength. However, in welded design, it is generally assumed that the weld metal properties will exceed those of the base metal, and any yielding that would occur in the connection should be concentrated in the base metal, not in the weld metal, since the base metal is assumed to be more homogeneous and more likely to be free of discontinuities than the weld. Most commercially available filler metals have a "70" classification, exceeding the minimum specified strength properties of the commonly used A36 and A572 grade 50.

These weld metal properties are obtained under very specific testing conditions prescribed by the AWS *A5 Filler Metal Specifications*. Weld metal properties are a function of many variables, including preheat and interpass temperatures, welding parameters, base metal composition, and joint design. Deviations in these conditions from those obtained for the test welds may result in differences in mechanical properties. Most of these changes will result in an increase in yield and tensile strength, along with a corresponding decrease in elongation and, in general, a decrease in toughness. When weld metal properties exceed those of the base metal, and when the connection is loaded into the

inelastic range, plastic deformations would be expected to occur in the base metal, not in the weld metal itself. The increase in the strength of the weld metal compensates for the loss in ductility. The general trend to strength levels higher than those obtained under the testing conditions is of little consequence in actual fabrication.

There are conditions that may result in lower levels of strength, and the Northridge earthquake experience revealed that this may be more commonplace and more significant than originally thought. The interpass temperature is the temperature of the steel when the arc is initiated for subsequent welding. There are two aspects to the interpass temperature: the minimum level, which should always be the minimum preheat temperature, and the maximum level, beyond which welding should not be performed. Because of the relatively short length of beam-to-column flange welds, an operator may continue welding at a pace that will allow the temperature of the steel at the connection to increase to unacceptably high levels. After one or two weld passes, this temperature may approach the 1000°F range. In such a case, the strength of the weld deposit will be rapidly decreased.

Weld metal toughness is an area of particular interest in the post-Northridge specifications. Previous specifications did not include any requirement for minimum notch toughness levels in the weld deposits, allowing for the use of filler metals that have no minimum specified requirements. For connections that are subject to inelastic loading, it now appears that minimum levels of notch toughness must be specified. The actual limits on notch toughness have not been experimentally determined. With the AWS filler metal classifications in effect in 1996, electrodes are classified as either having no minimum specified notch toughness or having notch toughness values of 20 ft.-lb at a temperature of  $0^{\circ}$ F or lower. As an interim specification, 20 ft.-lb at  $0^{\circ}$ F or lower has been recommended. It should be noted that the more demanding notch toughness requirements impose several undesirable consequences upon fabrication, including increased cost of materials, lower deposition rates, less operator appeal, and greater difficulty in obtaining sound weld deposits. Therefore, ultra-conservative requirements imposed "just to be safe" may actually be unacceptable. Research will be conducted to determine precise toughness requirements. Until then, based upon practical issues of availability, 20 ft.-lb at  $0^{\circ}$ F is a reasonable specification.

#### 22.8.3 Heat-Affected Zones

As illustrated in Figure 22.26, the base metal heat-affected zones (HAZs) represent material that may affect connection performance as well. The HAZ is that base metal that has been thermally changed due to the energy introduced into it by the welding process. In the small region immediately adjacent to the weld, the base metal has experienced a different thermal history than the rest of the base material. For most hot-rolled steels, the area of concern is a HAZ that is cooled too rapidly, resulting in a hardened HAZ. For quenched-and-tempered steels, the HAZ may be cooled too slowly, resulting in a softening of the area. In columns, the HAZ of interest is the Z direction area immediately adjacent to the groove weld. For the beam, these are oriented in the X direction.

Excessively high heat input can negatively affect HAZ properties by causing softening in these areas. Excessively low heat input can result in hardening of the HAZs. Weld metal properties may be negatively affected by extremely high heat input welding procedures, causing a decrease in both the yield strength and tensile strength, as well as the notch toughness of the weld deposit. Excessively low heat input may result in high-strength weld metal and also decrease the notch toughness of the weld deposit. Optimum mechanical properties are generally achieved if the heat input is maintained in the 30–80 kJ/in. range. Post-Northridge evaluation of fractured connections has revealed that excessively high heat input welding procedures were often used, confirmed by the presence of very large weld beads that sometimes exceeded the maximum limits prescribed by the D1.1-96 code. These may have had some corollary effects on weld metal and HAZ properties.

### 22.9 Connection Details

Since there are no secondary members in welded construction, any material connected by a weld participates in the structural system — positively or negatively. Unexpected load paths can be developed by the unintentional metallurgical path resulting from the one-component system created by welding. This phenomenon is particularly significant in detailing.

### 22.9.1 Weld Backing

Pre-Northridge specifications typically allowed steel backing to be left in place. Most of the fractures experienced in Northridge initiated immediately above the naturally occurring unfused region, between the backing and the column face. When this area experienced tensile loading due to lateral displacements, this region would result in a stress concentration and a notch that served as a crack initiation point (see Figure 22.27).



FIGURE 22.27: Weld backing and fracture initiation. (Courtesy of The Lincoln Electric Company. With permission.)

After Northridge, many specifications began to call for the removal of steel backing from the bottom beam-flange-to-column connection. This activity not only eliminates the notch-like condition, it permits gouging the weld root to sound metal, and allows for the depositing of a reinforcing fillet weld that provides a more gradual transition in the 90° interface between the beam and the column.

Not all backing is required to be removed. For welds subject to horizontal shear (such as corner joints in box columns), backing can be left in place. In butt joints, the degree of stress amplification that occurs due to backing left in place is much less severe than what occurs in T joints. Backing removal is expensive and, particularly when done in the overhead position, requires considerable welder skill. Some recommendations have not required removal of top beam-flange-to-column

connection backing because the removal operation (gouge, clean, inspect, and reweld) must be performed through the weld access hole. This difficult operation may do more harm than good.

There is increased interest in ceramic backing. Welding procedures that employ ceramic backing must be qualified by test for work done in accordance with D1.1-96, para. 5.10. Welders must be trained in the proper use of these materials. Ceramic is nonconductive, requiring that the welder establish a "bridge" between the two steel members to be welded in order to maintain the electrical arc between the two members. While this can be accomplished fairly readily with small root opening dimensions (such as 1/4 in.), it becomes increasingly difficult with larger root openings (such as 1/2 in.). Wide, thin root passes on ceramic-backed joints may crack due to high shrinkage stresses imposed on small weld throats.

One benefit of the activity of fusible backing removal is that it permits the weld joint to be back gouged to sound material. The root of the weld joint is always the most problem-prone region. The center of the length of the bottom beam-flange-to-column weld is difficult to make, since the welder must work through the weld access hole. This is also one of the most difficult areas to inspect with confidence. In a typical beam-to-column connection, the bottom beam-flange-to-column weld must be interrupted midlength due to interference with the web. This area is particularly sensitive to workmanship problems, and is also a difficult region to inspect with ultrasonic testing. The back-gouging operations provide the opportunity for visual verification that sound weld metal has been obtained, particularly in the center of the joint length. This is similar to the D1.1-96 code requirement for back gouging of double-sided joints.

#### 22.9.2 Weld Tabs

Weld tabs are auxiliary pieces of metal on which the welding arc may be started or stopped. For statically loaded structures, these are usually left in place. For seismic construction, weld tabs should be removed from critical connections that are subject to inelastic loading, because metal of questionable integrity may be deposited in the region of these weld tabs.

Weld tab removal is probably most important on beam-to-column connections where the column flange width is greater than the beam flange width. It is reasonable to expect that stress flow would take place through the left-in-place weld tab. However, for butt splices where the same width of material is joined, weld tabs extending beyond the width of the joint would not be expected to carry significant stress, making weld tab removal less critical. It is unlikely that tab removal from continuity plate welds would be justified.

For beam-to-column connections where columns are box shapes, the natural stress distribution causes the ends of the groove weld between the beam and column to be loaded to the greatest level, the same region as would contain the weld tab. Just the opposite condition exists when columns are composed of I-shaped members. The center of the weld is loaded most severely, causing the areas in which the weld tabs would be located to have the lowest stress level. For welds subject to high levels of stress, however, weld tabs should be removed.

### 22.9.3 Welds and Bolts Sharing Loads

Welding provides a continuous metallurgical path that relies upon the internal metallurgical structure of the fused metal to provide continuity and strength. Rivets and bolts rely on friction, shear of the fastening element, or bearing of the joint material to provide for transfer of loads between members. When mechanical fasteners such as bolts are combined with welds, caution must be exercised in assigning load-carrying capacity to each joining method.

Traditionally, it was thought that welds used in conjunction with bolts should be designed to carry the full load, assuming that the mechanical fasteners have no load-carrying capacity until the weld fails. The development of high-strength fasteners, however, created the assumption that loads can be shared equally between welds and fasteners. This has led to connection details that employ both joining systems. Specifically, the welded flange, bolted web detail used for many beam-to-column connections in SMRFs assumes that the bolted web is able to share loads equally with the welded flanges. Although most analyses suggest that vertical loads are transferred through the shear tab connection (bolted) and moments are transferred through the flanges (welded), the web does have some moment capacity. Depending on the particular rolled shape involved, the moment capacity of the web can be significant. Testing of specimens with the welded web detail, as compared to the bolted web detail, generally has yielded improved performance results. This has called into question the adequacy of the assumption of high-strength bolts sharing loads with welds when subject to inelastic loading. Post-Northridge research provides further evidence that the previously accepted assumptions may have been inadequate. Previous design rules regarding the capacity of bolted connections should be reexamined. This may necessitate additional fasteners, or larger sizes of shear tabs (both in thickness and in width). Stipulations regarding the addition of supplemental fillet welds on shear tabs, currently a function of the ratio of  $Z_f/Z$ , are probably also inadequate and will require revision.

Pending further research, the conservative approach is to utilize welded web details. This does not preclude the use of a bolted shear tab for erection purposes, but would rely on welds as a singular element connecting the web to the column.

### 22.9.4 Weld Access Holes

The performance of a connection during seismic loading can be limited by poorly made, or improperly sized, weld access holes. In the beam-to-column connection illustrated in Figure 22.28, a welded web connection has been assumed. As the flange groove weld shrinks volumetrically, a residual stress field will develop perpendicular to the longitudinal axis of the weld, as illustrated in direction X in the figure. Concurrently, as the groove weld shrinks longitudinally, a residual stress pattern is established along the length of the weld, designated as direction Y. When the web weld is made, the longitudinal shrinkage of this weld results in a stress pattern in the Z direction. These three residual stress patterns meet at the intersection of the web and flange of the beam with the face of the column. When steel is loaded in all three orthogonal directions simultaneously, even the most ductile steel cannot exhibit ductility. At the intersection of these three welds, cracking tendencies would be significant. A generous weld access hole, however, will physically interrupt the interaction of the Z axis stress field and the biaxial (X and Y) stress field, thereby increasing the resistance to cracking during fabrication.

The quality of weld access holes may affect both resistance to fabrication-related cracking and resistance to cracking that may result from seismic events. Access holes usually are cut into the steel by a thermal cutting process, either oxy-fuel or plasma arc. Both processes rely on heating the steel to a high temperature and removing the heated material by pressurized gases. In the case of oxy-fuel cutting, oxidation of the steel is a key ingredient in this process. In either process, the steel on either side of the cut (called the "kerf") has been heated to an elevated temperature and rapidly cooled. In the case of oxy-fuel cutting, the surface may be enriched with carbon. For plasma cut surfaces, metallic compounds of oxygen and nitrogen may be present on this surface. The resultant surface may be hard and crack sensitive, depending on the combinations of the cutting procedure, base metal chemistry, and thickness of the materials involved. Under some conditions, the surface may contain small cracks, which can be the points of stress amplification that cause further cracking during fabrication or during seismic events.

Nicks or gouges may be introduced during the cutting process, particularly when the cutting torch is manually propelled during the formation of the access hole. These nicks may act as stress amplification points, increasing the possibility of cracking. To decrease the likelihood of notches and/or microcracks on thermally cut surfaces, AISC has specific provisions for making access holes in heavy group 4 and 5 rolled shapes. These provisions include the need for a preheat before cutting, requirements



FIGURE 22.28: A generous weld access hole in this beam-to-column connection provides resistance to cracking. (Courtesy of The Lincoln Electric Company. With permission.)

for grinding of these surfaces, and inspection of these surfaces with magnetic particle (MT) or dye penetrant (PT) inspection. Whether these provisions should be required for all connections that may be subject to seismic energies is unknown at this time. However, for connection details that impose high levels of stress on the connection, and specifically those that demand inelastic performance, it is apparent that every detail in the access hole region is a critical variable. In the Northridge earthquake, some cracking initiated from weld access holes.

### 22.10 Achieving Ductile Behavior in Seismic Sections

### 22.10.1 System Options

Several systems may be employed to achieve seismic resistance, including eccentrically braced frames (EBFs), concentrically braced frames (CBFs), SMRFs, and base isolation. Of the four mentioned, only base isolation is expected to reduce demand on the structure. The other three systems assume that at some point within the structure, plastic deformations will occur in members, thus absorbing seismic energy.

In a CBF, the brace member is expected to be subject to inelastic deformations. The welded connections at the termination of a brace are subject to significant tension or compression loads, although rotation demands in the connections are fairly low. Designing these connections requires the engineer to develop the capacity of the brace member in compression and tension. Recent experiences with CBF systems have reaffirmed the importance of the brace dimensions (b/t ratio), and the importance of good details in the connection itself. Problems appear to be associated with misplaced welds, undersized welds, missing welds, or welds of insufficient throat due to construction

methods. In order to place the brace into the building frame, a gusset plate is usually welded into the corners of the frame. The brace is slit along its longitudinal axis and rotated into place. To maintain adequate dimensions for field assembly, the slot in the tube must be oversized, compared to the gusset, resulting in natural gaps between the tube and the gusset plate. When this dimension increases, as illustrated in Figure 22.29, it is important to consider the effect of the root opening on the strength of the fillet weld. For gaps exceeding 1/16 in., the D1.1-96 code requires that the weld leg size be increased by the amount of the gap, ensuring a constant actual throat dimension is maintained.



FIGURE 22.29: Effect of root openings (gaps) on fillet weld throat dimensions. (Courtesy of The Lincoln Electric Company. With permission.)

EBFs and SMRFs are significantly different structural systems, but some welding design principles apply equally to both systems. It is possible to design an EBF so that the "link" consists simply of a rolled steel member. In Figure 22.30, these examples are illustrated by the links designated as c1. In other EBF systems, however, the connection itself can be part of the link, as illustrated by c2. When this design method is used, the welded connections become critical since the expected loading on the connection is in the inelastic region. Much of the discussion under SMRF may be applied to these situations.



FIGURE 22.30: Examples of EBF systems. (From American Institute of Steel Construction. *Seismic Provisions for Steel Buildings*. 1992.)

The SMRF system is commonly applied to low-rise structures. Advantages of this type of system include desirable architectural elements that leave the structure free of interrupting diagonal members. Extremely high demands for inelastic behavior in the connections are inherent to this system.

When subject to lateral displacements, the structure assumes a shape as shown in Figure 22.31. Note that the highest moments shown in Figure 22.31 are applied at the connection. Figure 22.31 shows a plot of the section properties. Section properties are at their lowest value at the column face, because of the weld access holes that permit the deposition of the CJP beam-flange-to-column-flange welds. These section properties may be further reduced by deleting the beam web from the calculation of section properties. This is a reasonable assumption when the beam-web-to-column-shear tab is connected by the means of high-strength bolts. Greater capacity is achieved when the beam web is directly welded to the column flange with a CJP weld. The section properties at the end of the beam are least, precisely an area where the moment levels are the greatest, leading to the highest level of stresses. A plot of stress distribution is shown in Figure 22.31. The weld is therefore in the area of highest stress, making it critical to the performance of the connection. Details in either EBF or SMRF structures that place this type of demand on the weld require careful scrutiny.

### 22.10.2 Ductile Hinges in Connections

The SMRF concept is based on the premise that plastic hinges will form in the beams, absorbing seismically induced energies by inelastically stretching and deforming the steel. The connection is not expected to break. Following the Northridge earthquake, however, there was little or no evidence of hinge formation. Instead, the connections or portions of the connection experienced brittle fracture.

Most of the ductility data is obtained from smooth, slowly loaded, uniaxially loaded tensile specimens that are free to neck down. If a notch is placed in the specimen, perpendicular to the applied load, the specimen will be unable to exhibit its normal ductility, usually measured as elongation. The presence of notch-like conditions in the Northridge connections reduced the ductile behavior.

In 1994, initial research on SMRF connections attempted to eliminate the issues of notch-like conditions in the test specimens by removing weld backing and weld tabs and controlling weld soundness. Even with these changes, brittle fractures occurred when the standard details were tested. The testing program then evaluated several modified details with short cover plates, with better success. The beam-to-column connection will be examined with respect to the previously outlined conditions required for ductility (see Section 22.7).

Figure 22.32 shows two regions in question. Point A is at the weld joining the beam flange to the face of the column flange. Here there is restraint against strain (movement) across the width of the beam flange ( $\varepsilon_1$ ) as well as through the thickness of the beam flange ( $\varepsilon_2$ ). Point B is along the length of the beam flange away from the connecting weld. There is no restraint across the width of the flange or through its thickness.

The following equations can be found in most texts concerning strength of materials:

$$\varepsilon_3 = \frac{1}{E} (\sigma_3 - \mu \sigma_2 - \mu \sigma_1)$$
 (22.1a)

$$\varepsilon_2 = \frac{1}{E} \left( -\mu \sigma_3 + \sigma_2 - \mu \sigma_1 \right)$$
 (22.1b)

$$\varepsilon_1 = \frac{1}{E} \left( -\mu \sigma_3 - \mu \sigma_2 + \sigma_1 \right)$$
 (22.1c)



b. Moment diagram of SMRF subject to lateral displacements.



c. Section propeties of SMRF subject to lateral displacements.



d . Stress distribution of SMRF subject to lateral displacements.

FIGURE 22.31: Analysis of SMRF behavior. (Courtesy of The Lincoln Electric Company. With permission.)



FIGURE 22.32: Regions to be analyzed relative to potential for ductile behavior. (Courtesy of The Lincoln Electric Company. With permission.)

It can be shown that:

$$\sigma_1 = \frac{E\left[\mu\varepsilon_3 + \mu\varepsilon_2 + (1-\mu)\varepsilon_1\right]}{(1+\mu)(1-2\mu)}$$
(22.2a)

$$\sigma_2 = \frac{E\left[\mu\varepsilon_3 + (1-\mu)\varepsilon_2 + \mu\varepsilon_1\right]}{(1+\mu)(1-2\mu)}$$
(22.2b)

$$\sigma_3 = \frac{E\left[(1-\mu)\varepsilon_3 + \mu\varepsilon_2 + \mu\varepsilon_1\right]}{(1+\mu)(1-2\mu)}$$
(22.2c)

The unit cube in Figure 22.33 is an element of the beam flange from point B in Figure 22.32. The applied force due to the moment is  $\sigma_3$ . Assuming strain in direction 3 to be + 0.001 in./in., and Poisson's ratio of  $\mu = 0.3$  for steel,  $\varepsilon_2$  and  $\varepsilon_3$  can be found to be equal to -0.0003 in./in.

Using these strains, from Equations 22.2a to 22.2c, it is found that

$$\sigma_1 = 0$$
 ksi  
 $\sigma_2 = 0$  ksi  
 $\sigma_3 = 30$  ksi

These stresses are plotted as a dotted circle on Figure 22.34. These values are then extrapolated to the point where fracture would occur, that is, where the net tensile strength is 70 ksi. The larger solid line circle is for a stress of 70 ksi or ultimate tensile stress. The resulting maximum shear stresses,  $\tau_{1-3}$  and  $\tau_{2-3}$ , are the radii of these two circles, or 35 ksi. The ratio of shear to tensile stress for steel is 0.5. Figure 22.35 plots this as line B. At a yield point of 55 ksi, the critical shear value is half of this, or 27.5 ksi. When this critical shear stress is reached, plastic straining takes place and ductile behavior will result up to the ultimate tensile strength, here 70 ksi. Figure 22.38 shows a predicated stress-strain curve indicating ample ductility.

Figure 22.36 shows an element from point A of Figure 22.32 at the junction of the beam and column flange. Whether weld metal or the material in the column or beam is considered makes little



FIGURE 22.33: Unit cube showing applied stress from Figure 22.32. (Courtesy of The Lincoln Electric Company. With permission.)



FIGURE 22.34: A plot of the tensile stress and shear stress from Figure 22.32. (Courtesy of The Lincoln Electric Company. With permission.)

difference. This region is highly restrained. Suppose it is assumed:

$$\varepsilon_3 = +0.001$$
 in./in. (as before)  
 $\varepsilon_2 = 0$  (since it is highly restrained  
 $\varepsilon_1 = 0$  with little strain possible)

Then, from the given equations, the following stresses are found:

$$\sigma_1 = 17.31 \text{ ksi}$$
  
 $\sigma_2 = 17.31 \text{ ksi}$   
 $\sigma_3 = 40.38 \text{ ksi}$ 

The stresses are plotted as a dotted circle in Figure 22.37.



FIGURE 22.35: The ratio of shear to tensile stress. (Courtesy of The Lincoln Electric Company. With permission.)



FIGURE 22.36: The highly restrained region at the junction of the beam and column flange shown in Figure 22.32. (Courtesy of The Lincoln Electric Company. With permission.)

If these stresses are increased to the ultimate tensile strength, it is found that

$$\sigma_1 = 30.0$$
 ksi  
 $\sigma_2 = 30.0$  ksi  
 $\sigma_3 = 70.0$  ksi

The solid line circle in Figure 22.37 is a plot of stresses for this condition. The maximum shear stresses are  $\tau_{1-3} = \tau_{2-3} = 20$  ksi. Since these are less than the critical shear stress (27.5 ksi), no plastic movement, or ductility, would be expected.

In this case, the ratio of shear to tensile stress is 0.286. In Figure 22.35, this condition is plotted as line A. It never exceeds the value of the critical shear stress (27.5 ksi); therefore, there will be no plastic strain or movement, and it will behave as a brittle material. Figure 22.38 shows a predicated stress-strain curve going upward as a straight line (A) (elastic) until the ultimate tensile stress is



FIGURE 22.37: A plot of the tensile stress and shear stress from Figure 22.35. (Courtesy of The Lincoln Electric Company. With permission.)



FIGURE 22.38: Stress-strain curve. (Courtesy of The Lincoln Electric Company. With permission.)

reached in a brittle manner. It would therefore be expected that, at the column face or in the weld where high restraint exists, little ductility would result. This is where brittle fractures have occurred, both in the laboratory and in actual Northridge structures.

In the SMRF system, the greatest moment (due to lateral forces) will occur at the column face. This moment must be resisted by the beam's section properties, which because of weld access holes are lowest at the column face. Thus, the highest stresses occur at this point, the point where analysis shows ductility to be impossible.

In Figure 22.32, material at point B was expected to behave as shown in Figure 22.34a, and as line B in Figure 22.35, and curve B in Figure 22.38; that is, with ample ductility. Plastic hinges must be forced to occur in this region.

Several post-Northridge designs have employed details that encourage use of this potential ductility. The coverplated design illustrated in Figure 22.39 accomplishes two important things: first, the stress



FIGURE 22.39: Coverplate detail takes advantage of the region where ductility is possible. (Courtesy of The Lincoln Electric Company. With permission.)

level at point A is reduced as a result of the increased cross-section at the weld. This region, incapable of ductility, must be kept below the critical tensile stress and the increase in area accomplishes this goal. Second, and most significant, the most highly stressed region is now at point B, the region of the beam that is capable of exhibiting ductility.

The real success of this connection will depend upon getting the adjacent beam to plastically bend before this critical section cracks. The way in which a designer selects structural details under particular load conditions greatly influences whether the condition provides enough shear stress component so that the critical shear value may be exceeded first, producing sufficient plastic movement before the critical normal stress value is exceeded. This will result in a ductile detail and minimize the chances of cracking.

### 22.11 Workmanship Requirements

In welded construction, the performance of the structural system often depends on the ability of skilled welders to deposit sound weld metal. As the level of loading increases, dependence on high-quality fabrication increases. For severely loaded connections, good workmanship is a key contributor to acceptable performance.

Design and fabrication specifications such as the AISC *Manual of Steel Construction* and the AWS D1.1 *Structural Welding Code: Steel* [9] communicate minimum acceptable practices. It is impossible for any code to cover every situation that will ever be contemplated. It is the responsibility of the engineer to specify any additional requirements that supersede minimum acceptable standards.

The D1.1-96 code does not specifically address seismic issues, but does establish a minimum level of quality that must be achieved in seismic applications. Additional requirements are probably

warranted. These would include requirements for nondestructive testing, notch tough weld deposits, and additional requirements for in-process verification inspection.

### 22.11.1 Purpose of the Welding Procedure Specification

The welding procedure specification (WPS) is somewhat analogous to a cook's recipe. It outlines the steps required to make a good-quality weld under specific conditions. It is the primary method to ensure the use of welding variables essential to weld quality. In addition, it permits inspectors and supervisors to verify that the actual welding is performed in conformance with the constraints of the WPS. Examples of WPSs are shown in Figures 22.40 and 22.41.

WPSs typically are submitted to the inspector for review prior to the start of welding. For critical projects, the services of welding engineers may be needed. WPSs are intended to be communication tools for maintenance of weld quality. All parties involved with the fabrication sequence must have access to these documents to ensure conformance to their requirements.

#### 22.11.2 Effect of Welding Variables

Specific welding variables that determine the quality of the deposited weld metal are a function of the particular welding process being used, but the general trends outlined below are applicable to all welding processes.

*Amperage* is a measure of the amount of current flowing through the electrode and the work. An increase in amperage generally means higher deposition rates, deeper penetration, and more melting of base metal. The role of amperage is best understood in the context of heat input and current density, which are described below.

*Arc voltage* is directly related to arc length. As the voltage increases, the arc length increases. Excessively high voltages may result in weld metal porosity, while extremely low voltages will produce poor weld bead shapes. In an electrical circuit, the voltage is not constant, but is composed of a series of voltage drops. Therefore, it is important to monitor voltage near the arc.

*Travel speed* is the rate at which the electrode is moved relative to the joint. Travel speed, which has an inverse effect on the size of weld beads, is a key variable used in determining heat input.

*Polarity* is a definition of the direction of current flow. Positive (or reverse) polarity is achieved when the electrode lead is connected to the positive terminal of the direct current power supply. The work lead would be connected to the negative terminal. Negative (or straight) polarity occurs when the electrode is connected to the negative terminal. For most welding processes, the required electrode polarity is a function of the design of the electrode. For submerged arc welding, either polarity could be utilized.

*Current density* is determined by dividing the welding amperage by the cross-sectional area of the electrode. The current density is therefore proportional to  $I/d^2$ . As the current density increases, both deposition rates and penetration increase.

*Preheat and interpass temperatures* are used to control cracking tendencies, typically in the base material. Excessively high preheat and interpass temperatures will reduce the yield and tensile strength of the weld metal as well as the toughness. When base metals receive little or no preheat, the resultant rapid cooling can promote cracking as well as excessively high yield and tensile properties in the weld metal, and a corresponding reduction in toughness and elongation.

The WPS defines and controls all of the preceding variables. Conformance to the WPS is particularly important in the case of seismically loaded structures, because of the high demand placed on welded connections under these situations.

WELDING PROCEDURE SPE PREQUALIFIED X QUA or PROCEDURE QUALIFICATIO	CIFICATION (WPS) Yes 🖾 LIFIED BY TESTING ON RECORDS (PQR) Yes 🗌
Company Name <u>ABC Fabricating</u> Welding Process(es) <u>FCAW</u> Supporting PQR No.(s) <u>N.A.</u>	Identification #     96 - 008       Revision 1     Date 4 - 96       Authorized by 0. KAY     Date       Type-Manual 1     Semi-Automatic X       Machine 1     Automatic 1
JOINT DESIGN USED Type: T-joint, Fillet Weld S/IC" Single Double Weld B Backing: Yes No A Backing Material:	POSITION Position of Groove: Fillet: HORIZ. (2F) Vertical Progression: Up Down N.A. ELECTRICAL CHARACTERISTICS
Root Opening        Root Face Dimension          Groove Angle:        Radius (J-U)          Back Gouging:       Yes       No       Method	Transfer Mode (GMAW) Short-Circuiting Globular Spray Current: AC DCEP X DCEN Pulsed
BASE METALS Material Spec. <u>ASTM AS72 / A36</u> Type or Grade <u>Gr 50 /</u> Thickness: Groove Fillet <u>1/2*-&gt; 21/4</u> "	Other Tungsten Electrode (GTAW) Size: Type:
FILLER METALS AWS Specification <u>A 5.20</u> AWS Classification <u>E 707-1</u>	TECHNIQUE Stringer r Weave Bead: Multi-pass or Single Pass (per side) Number of Electrodes I Electrode Spacing Longitudinal
SHIELDING       Gas       X         Flux       Composition       100%       CO2         Electrode-Flux (Class)       Flow Rate       45       CFH         Gas Cup Size       #4	Contact Tube to Work Distance 18 ± 1/4 " Peening
PREHEAT Preheat Temp., Min <u>150°F</u> Interpass Temp., Min <u>150°F</u> Max ——	POSTWELD HEAT TREATMENT Temp Time

Pass or		Filler Metals		WELDING PROCEDURE Current				
Weid Layer(s)	Process	Class	Diam.	Type & Polarity	Amps or Wire Feed Speed	Volts	Travel Speed	Joint Details
1	FCAW	Е 707-1	3/32"	DC+	<b>460 A</b>	31 V	15-17 ipm	

Form E-1 (Front)

FIGURE 22.40: Example of Welding Procedure Specification (WPS). (From American Welding Society. *Structural Welding Code: Steel: ANSI/AWS D1.1-96.* Miami, Florida, 1996. With permission.)

#### WELDING PROCEDURE SPECIFICATION (WPS) Yes PREQUALIFIED \_\_\_\_\_X QUALIFIED BY TESTING \_\_\_\_\_ or PROCEDURE QUALIFICATION RECORDS (PQR) Yes Identification # 12817

Company Name XYZ Erectors Welding Process(es) FCAN Supporting PQR No.(s) N.A.	Revision     2     Date     3-89     By     I     See       Authorized by     J     Walk     Date     4-89       Type     Manual     Semi-Automatic X       Machine     Automatic	
JOINT DESIGN USED Type: Lap Joint - Fillet Weld 5/16" Single X Double Weld Backing: Yes No X	POSITION Position of Groove:Fillet:Vertical Vertical Progression: Up 🕱 Down 🗍	(3F)
Backing Material:	ELECTRICAL CHARACTERISTICS	
Root Opening       Root Face Dimension         Groove Angle:       Radius (J-U)         Back Gouging: Yes No x       Method	Transfer Mode (GMAW)     Short-Circuiting       Globular Spray       Current: AC DCEP DCEN x     Pulsed	
BASE METALS	Other	
Material Spec. ASTM AS0/AS0	Tungsten Electrode (GTAW)	
Type or Grade	Size:	
Thickness: Groove Fillet 1/2" = 5/8"	Type:	
Diameter (Pipe)		
	TECHNIQUE	
FILLER METALS	Stringer or Weave Bead:	
AWS Specification <u>S, 20</u>	Multi-pass or Single Pass (per side)	
AWS Classification <u>E71T-8</u>	Number of Electrodes1	
	Electrode Spacing Longitudinal	
	Lateral	
SHIELDING	Angle	
Flux GasNONE Composition	Contact Tube to Work Distance 1/2" - 3/4"	
Electrode-Flux (Class) Flow Rate	Peening	
Gas Cup Size	Interpass Cleaning: N.A.	
PREHEAT	POSTWELD HEAT TREATMENT	
Preheat Temp., Min_NONE (/U-F min.)	Тетр	
interpass Temp., MinNONE Max	Time	

Pass or	Filler Metals		Current					
Weld Layer(s)	Process	Class	Diam.	Type & Polarity	Amps or Wire Feed Speed	Voits	Travel Speed	Joint Details
1	FCAW	E71T-8	0.068	DC-	250A (150 ipm)	19-21	5.5 - 6.5ipm	3F

Form E-1 (Front)

FIGURE 22.41: Example of Welding Procedure Specification (WPS). (From American Welding Society. *Structural Welding Code: Steel: ANSI/AWS D1.1-96.* Miami, Florida, 1996. With permission.)

### 22.11.3 Fit-Up

The orientation of the various pieces prior to welding is known as "fit-up". The AWS D1.1-96 code [9] contains specific tolerances that are applied to the as-fit dimensions of a joint prior to welding. There must be ample access to the root of the joint to ensure good, uniform fusion between the members being joined. Excessively small root openings or included angles in groove welds do not permit uniform fusion. Excessively large root openings or included angles result in the need for greater volumes of weld metal, with corresponding increases in shrinkage stresses, which in turn increases distortion and cracking tendencies. The D1.1-96 tolerances for fit-up are generally measured in 1/16-in. increments.

### 22.11.4 Field vs. Shop Welding

Many believe that the highest quality welding is obtained under shop welding conditions. The greatest differences between field and shop welding are related to control. For shop fabrication, the work force is generally more stable. Supervision practices are well understood and communication is generally more efficient. Under field welding conditions, control of a project seems to be more difficult. While there are environmental challenges to field conditions, including temperature, wind, and moisture, these seem to pose fewer problems than do the management issues.

For field welding, the gasless welding processes such as self-shielded flux cored welding and shielded metal arc welding usually are preferred. Gas metal arc, gas tungsten arc, and gas-shielded flux cored arc welding are all limited due to their sensitivity to wind-related gas disturbances. It is imperative that field welding conditions receive an appropriate increase in the monitoring and control area to ensure consistent quality. D1.1-96 imposes the same requirements on field welding as on shop welding. This includes qualification of welders, the use of welding procedures, and the resultant quality requirements.

### 22.12 Inspection

The AWS D1.1-96 code requires that all welds be inspected, specifically by means of visual inspection. In addition, at the engineer's discretion and as identified in contract documents, nondestructive testing may be required for finished weldments. This enables the engineer with a knowledge of the complexity of the project to specify additional inspection methodologies commensurate with the degree of confidence required for a particular project. In the case of seismically loaded structures, and connections subject to high stress levels, the need for inspection increases.

### 22.12.1 In-Process Visual Inspection

D1.1-96 mandates the use of in-process visual inspection. Before welding, the inspector reviews welder qualification records, welding procedure specifications, and the contract documents to confirm that applicable requirements are met. Before welding is performed, the inspector verifies fit-up and joint cleanliness, examines the welding equipment to ensure it is in proper working order, verifies that the materials involved meet the various requirements, and confirms that the required levels of preheat have been properly applied. During welding, the inspector confirms that the WPS is being carried out and that the intermediate weld passes meet the various requirements. After welding is finished, final bead shapes and welding integrity can be visually confirmed. Effective visual inspection is a critical component for ensuring consistent weld quality.

### 22.12.2 Nondestructive Testing

Four major nondestructive testing methods may be used to verify weld integrity after welding operations are completed. Each should be used in conjunction with effective visual inspection. No process is 100% capable of detecting all discontinuities in a weld.

Dye penetrant (PT) inspection involves the application of a liquid that is drawn into a surfacebreaking discontinuity, such as a crack or porosity, by capillary action. When the excess residual dye is removed from the surface, a developer is applied that will absorb the penetrant contained within the discontinuity. The result is a stain in the developer that shows that a discontinuity is present. PT testing is limited to surface-breaking discontinuities. It cannot read subsurface discontinuities, but it is highly effective in accenting very small discontinuities.

Magnetic particle (MT) inspection utilizes the change in magnetic flux that occurs when a magnetic field is present in the vicinity of a discontinuity. The change will show up as a different pattern when magnetic dust-like particles are applied to the surface of the part. The process is highly effective in locating discontinuities that are on the surface or slightly subsurface. The magnetic field can be created in the material in one of two ways: the current is directly passed through the material or the magnetic field is induced through a coil on a yoke. Since the process is most sensitive to discontinuities that lie perpendicular to the magnetic flux path, it is necessary to energize the part in two directions in order to fully inspect the component.

Radiographic (RT) inspection uses X-rays or gamma rays that are passed through the weld to expose a photographic film on the opposite side of the joint. High-voltage generators produce X-rays, while gamma rays are created by atomic disintegration of radioisotopes. Whenever radiographic inspection is employed, workers must be protected from exposure to excessive radiation. RT relies on the ability of the material to pass some of the radiation through, while absorbing part of this energy within the material. Different materials have different absorption rates. As the different levels of radiation are passed through the material, portions of the film are exposed to a greater or lesser degree. When this film is developed, the resulting radiograph will bear the image of the cross-section of the part. The radiograph is actually a negative. The darkest regions are those that were most exposed when the material being inspected absorbed the least amount of radiation. Porosity will show up as small dark round circles. Slag is generally dark and will look similar to porosity, but will have irregular shapes. Cracks appear as dark lines. Excessive reinforcement will result in a light region.

A radiographic test is most effective for detecting volumetric discontinuities such as slag or porosity. When cracks are oriented perpendicular to the direction of a radiographic source, they may be missed with the RT method. Therefore, RT inspection is most appropriate for butt joints and is generally not appropriate for inspection of corner or T joints. Radiographic testing has the advantage of generating a permanent record for future reference.

In ultrasonic (UT) inspection, solid discontinuity-free materials will transmit high-frequency sound waves throughout the part in an uninterrupted manner. A receiver "hears" the sound reflected off of the back surface of the part being inspected. If there is a discontinuity between the transmitter and the back of the part, an intermediate signal will be sent to the receiver indicating its presence. The pulses are read on a CRT screen. The magnitude of the signal received from the discontinuity indicates its size. UT is most sensitive to planar discontinuities, i.e., cracks. UT effectiveness is dependent on the operator's skill, so UT technician training and certification is essential. With currently available technology, UT is capable of reading a variety of discontinuities that would be acceptable for many applications. Acceptance criteria must be clearly communicated to the inspection technicians so unnecessary repairs are avoided.

### 22.12.3 Applications for Nondestructive Testing Methods

Visual inspection is the most comprehensive method available to verify conformance with the wide variety of issues that can affect weld quality and should be thoroughly applied on every welding project. To augment visual inspection, nondestructive testing can be specified to verify the integrity of the deposited weld metal. The selection of the inspection method should reflect the probable discontinuities that would be encountered, and the consequences of undetected discontinuities. Consideration must be given to the conditions under which the inspection would be performed, such as field vs. shop conditions. The nature of the joint detail (butt, T, corner, etc.) and the weld type (CJP, PJP, fillet weld) will determine the choice of the inspection process in many situations. MT inspection is usually preferred over PT inspection because of its relative simplicity. Cleanup is easy, and the process is sensitive. PT is normally reserved for applications where the material is nonmagnetic, and MT would not be applicable. While MT is suitable for detection of surface or slightly subsurface discontinuities only, it is in these areas that many welding defects are located. It is very effective in crack detection, and can be utilized to ensure complete crack removal before subsequent welding is performed on damaged structures.

UT inspection has become the primary nondestructive testing method used for most building applications. It can be utilized to inspect butt, T, and corner joints, is relatively portable, and is free from the radiation concerns associated with RT inspection. UT is especially sensitive to the identification of cracks, the most significant defect in a structural system. Although it may not detect spherical or cylindrical voids such as porosity, nondetection of these types of discontinuities has fewer consequences.

### 22.13 Post-Northridge Assessment

Prior to the Northridge earthquake, the SMRF with the "pre-Northridge" beam-to-column detail was unchallenged regarding its ability to perform as expected. This confidence existed in spite of a fairly significant failure rate when these connections had been tested in previous research. The pre-Northridge detail consisted of the following:

- CJP groove welds of the beam flanges to the column face, with weld backing and weld tabs left in place.
- No specific requirement for minimum notch toughness properties in the weld deposit.
- A bolted web connection with or without supplemental fillet welds of the shear tab to the beam web.
- Standard ASTM A36 steel for the beam and ASTM 572 grade 50 for the column (i.e., no specific limits on yield strength or the  $F_{\gamma}/F_{u}$  ratio).

As a result of the Northridge earthquake, and research performed immediately afterward, confidence in this detail has been severely shaken. Whether any variation of this detail will be suitable for use in the future is currently unknown. More research must be done, but one can speculate that, with the possible exception of small-sized members, some modification of this detail will be required.

Although testing of this configuration had a fairly high failure rate in pre-Northridge tests, many successful results were obtained. Further research will determine which variables are the most significant in predicting performance success. Some changes in materials and design practice also should be considered. In recent years, recycling of steel has become a more predominant method of manufacture. This is not only environmentally responsible, it is economical. However, in recycling, residual alloys can accumulate in the scrap charge, inadvertently increasing steel strength levels. In the past 20 years, the average yield strength of ASTM A36 steel has increased approximately 15%. Testing done with lower yield strength steel would be expected to exhibit different behavior than

test specimens made of today's higher strength steels (in spite of the same ASTM designation). For practical reasons, laboratory specimens tend to be small in size. Success in small-sized specimens was extrapolated to apply to very large connection assemblies in actual structures. The design philosophy that led to fewer SMRFs throughout a structure required that each of the remaining frames be larger in size. This corresponded to heavier and deeper beams, and much heavier columns, with an increase in the size of the weld between the two rolled members. The effect of size on restraint and triaxial stresses was not researched, resulting in some new discoveries about the behavior of the large-sized assemblages during the Northridge earthquake.

The engineering community generally agrees that the pre-Northridge connection (as defined above) is no longer adequate and some modification will be required. Any deviation from the definition above will constitute a modification for the purposes of this discussion.

#### 22.13.1 Minor Modifications to the SMRF Connection

With the benefit of hindsight, several aspects of the pre-Northridge connection detail appear to be obviously deficient. Weld backing left in place in a connection subject to both positive and negative moments where the root of the flange weld can be put into tension creates high-stress concentrations that may result in cracking. Failure to specify minimum toughness levels for weld metal for heavily loaded connections is another deficiency. The superior performance of the welded web vs. the bolted web in past testing draws into question the assumption of load sharing between welds and bolts. Now it seems that tighter control of the strength properties of the beam steel and the relationship to the column is essential.

Some preliminary tests suggest that tightly controlling all of these variables may result in acceptable performance. However, the authors know of no test of unmodified beam-to-column connections where the connection zone has remained crack free when acceptable rotation limits were achieved. For smaller sized members, this approach may be technically possible, although the degree of control necessary on both the material properties and the welding operations may make it impractical.

### 22.13.2 Coverplated Designs

This concept uses short coverplates that are added to the top and bottom flanges of the beam. Fillet welds transfer the coverplate forces to the beam flanges. The bottom flange coverplate is shop welded to the column flange, and the bottom beam flange is field welded to the column flange and to the coverplate. Both the top flange and the top flange coverplate are field welded to the column flange with a common weld. The web connection may be welded or high-strength bolted. These connections have been tested to a limited extent, with generally favorable results.

Following Northridge, the coverplate approach received significant attention because it offered early promise of a viable solution. Other methods may prove to be superior as time passes. While the coverplate solution treats the beam in the same way as other approaches (i.e., it moves the plastic hinge into a region where ductility can be demonstrated), it concentrates all the loading to the column into a relatively short distance. Other alternatives may treat the column in a more gentle manner.

### 22.13.3 Flange Rib Connections

This concept utilizes one or two vertical ribs attached between the beam flanges and column face. The intent of the rib plates is to reduce the demand on the weld at the column flange and to shift the plastic hinge from the column face. In limited testing, flange rib connections have demonstrated acceptable levels of plastic rotation provided that the girder flange welding is done correctly.

Vertical ribs appear to function very similarly to the coverplated designs, but offer the additional advantage of spreading the load over a greater portion of the column. The single-rib designs appear to

be better than the twin-rib approaches because the stiffening device is in alignment with the column web (for I-shaped columns) and facilitates easy access to either side of the device for welding. It is doubtful that the rib design would be appropriate for box column applications.

### 22.13.4 Top and Bottom Haunch Connections

In this configuration, haunches are placed on both the top and bottom flanges. In two tests of the top and bottom haunch connection, it has exhibited extremely ductile behavior, achieving plastic rotations as great as 0.07 rad. Tests of single, haunched beam–column connections have not been as conclusive; further tests are planned.

Although they are costly, haunches appear to be the most straightforward approach to obtaining the desired behavior out of the connection. The treatment to the column is particularly desirable, greatly increasing the portion of the column participating in the transfer of moment.

### 22.13.5 Reduced Beam Section Connections

In this configuration, the cross-section of the beam is deliberately reduced within a segment to produce a plastic hinge within the beam span, away from the column face. A variant of this approach produces the so-called "dog bone" profile.

Reduced section details offer the prospect of a low-cost connection and increased performance out of detailing that is very similar to the pre-Northridge connection. Control of material properties of the beam will still be a major variable if this detail is used. Lateral bracing will probably be required in the area of the reduced section to prevent buckling, particularly at the bottom flange when loaded in compression.

### 22.13.6 Partially Restrained Connections

Some have suggested that partially restrained (PR) connection details will offer a performance advantage over the SMRF. The relative merits of a PR frame vs. a rigid frame are beyond the scope of this work. However, many engineers immediately think of bolted PR connections when it is possible to utilize welded connections for PR performance as well.

Illustrated in Figure 22.42 is a detail that can be employed utilizing the PR concept. Detailing rules must be developed, and tests done, before these details are employed. They are supplied to offer welded alternatives to bolted PR connections.

### 22.14 Defining Terms

As-welded: The condition of weld metal, weld joints, and weldments after welding, but prior to any subsequent thermal, mechanical, or chemical treatments.

Autogenous weld: A fusion weld made without the addition of filler metal.

- Back gouging: The removal of weld metal and base metal from the other side of a partially welded joint to facilitate complete fusion and complete joint penetration upon subsequent welding from that side.
- Backing: A material or device placed against the back side of the joint, or at both sides of a weld in electroslag and electrogas welding, to support and retain molten weld metal. The material may be partially fused or remain unfused during welding and may be either metal or nonmetal.

Base metal: The material to be welded, brazed, soldered, or cut.



FIGURE 22.42: A partially restrained welded connection detail. (Courtesy of The Lincoln Electric Company. With permission.)

Deposition rate: The weight of material deposited in a unit of time.

- Effective throat: The minimum distance minus any convexity between the weld root and the face of a fillet weld.
- Filler metal: The metal to be added in making a welded, brazed, or soldered joint.
- Heat affected zone (HAZ): That portion of the base metal that has not been melted, but whose mechanical properties or microstructure have been altered by the heat of welding, brazing, soldering, or cutting.
- Nugget: The weld metal joining the workpieces in spot, roll spot, seam, or projection welds.
- Postheating: The application of heat to an assembly after welding, brazing, soldering, thermal spraying, or thermal cutting.
- Preheating: The application of heat to the base metal immediately before welding, brazing, soldering, thermal spraying, or cutting.
- Residual stress: Stress present in a member that is free of external forces or thermal gradients.
- Theoretical weld throat: The distance from the beginning of the joint root perpendicular to the hypotenuse of the largest right triangle that can be inscribed within the cross-section of a fillet weld. This dimension is based on the assumption that the root opening is equal to zero.
- Weldability: The capacity of material to be welded under the imposed fabrication conditions into a specific, suitable designed structure and to perform satisfactorily in the intended service.
- Weldment: An assembly whose component parts are joined by welding.
- Weld metal: That portion of a weld that has been melted during welding.
- Weld pool: The localized volume of molten metal in a weld prior to its solidification as weld metal.

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