

CONNECTIONS FOR SKEWED BEAMS



Skewed connections result when members frame to each other at an angle other than 90 degrees



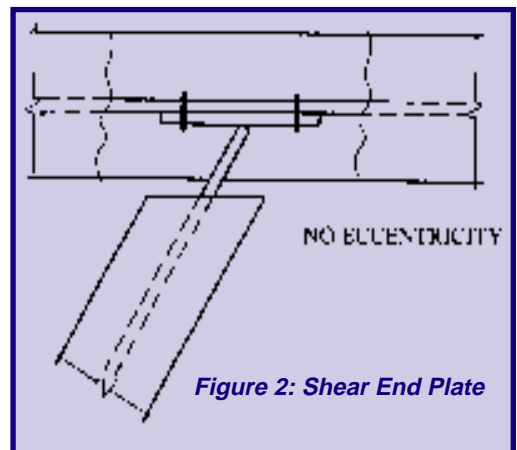
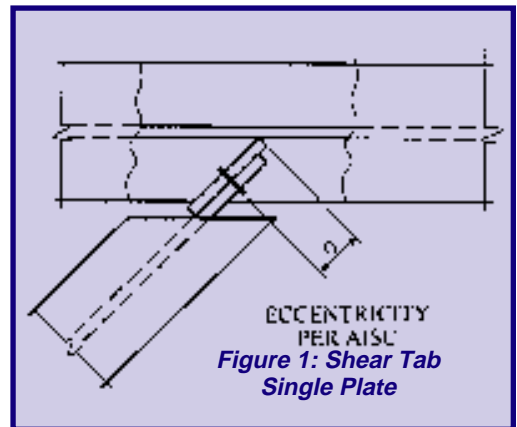
THE STANDARD CONNECTION DETAILS ASSUME THAT THE MEMBERS BEING CONNECTED FRAME AT RIGHT ANGLES TO EACH OTHER. In most structures there will be some members which do not meet at right angles. These are referred to as skewed connections. They require special design considerations to provide for safety while providing an economic structure.



Configurations For Skewed Connections To Beams

The preferred skewed connections for economy and safety are single plates (Fig. 1) and end plates (Fig. 2). Single bent plates (Fig. 3) and eccentric end plates also work well at very acute angles. The old traditional double bent plate connections are difficult to accurately fit and are expensive to fabricate. There are also quality (safety) problems with plate cracking at the bend line as the angle becomes more acute.

Single plates (Fig. 1) are the most versatile and economical skewed connection with excellent dimensional control when using short slotted holes. While capacity is limited, this is usually not a problem because skewed members generally carry less tributary area. Using the standard 3" hole gauge from the Table in *AISC Manual of Steel Construction, Vols. I and II*, single plates can be utilized for intersection angles of 90° to 30°. Snug tight bolts are preferred because they are more economical and greatly simplify installation when there are adjacent beams. They also eliminate the "banging bolt" problem which occurs in single plate connections when pretensioned bolts



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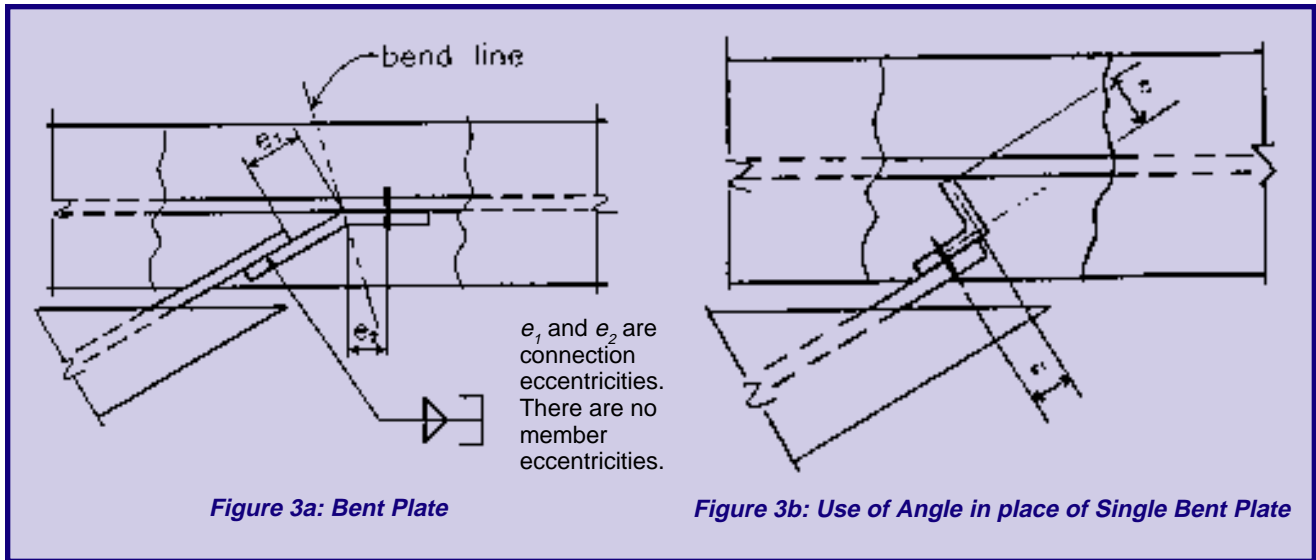


Figure 3a: Bent Plate

Figure 3b: Use of Angle in place of Single Bent Plate

slip into bearing. The AISC tables can be used to select the required plate size and bolts along with the weld capacity for the required load. This connection has an eccentricity related to physical distance a between the bolts and the weld as shown in Fig. 1. The actual eccentricity depends on support rigidity, hole type, and bolt installation.

For a flexible support and standard holes, the eccentricity for the bolts is

$$e_b = |(n-1) - a| \geq a$$

where n is the number of bolts.

For a flexible support with short slotted holes,

$$e_b = \left| \frac{2n}{3} - a \right| \geq a$$

For a rigid support and standard holes,

$$e_b = |(n-1) - a|$$

For a rigid support and short slotted holes,

$$e_b = \left| \frac{2n}{3} - a \right|$$

When it cannot be determined whether the support is rigid or flexible, the larger value of e_b from the above equations can be used.

The eccentricity for the bolts e_b is measured from the face of the support. Therefore, the eccentricity for the weld, e_w , is $e_w = e_b + a$. However, rather than using this value, AISC recommends that the weld size be such

that the plate yields before the welds yield. For A36 plate and E70 electrodes, this requires that the fillet weld size is a minimum of $\frac{3}{4}$ of the plate thickness.

The actual weld detail does, however, have to be developed for the skewed joint geometry, as will be shown later.

End plates (Fig. 2) designed for shear only are able to provide more capacity than single plates and if horizontal slots are utilized with snug tight bolts in bearing some dimensional adjustment is possible. Hole gages can be adjusted to provide bolt access for more acute skews. The only real constructability problem arises when there are opposing beams that limit access to the back side of the connection. These end plate connections can be sized using the

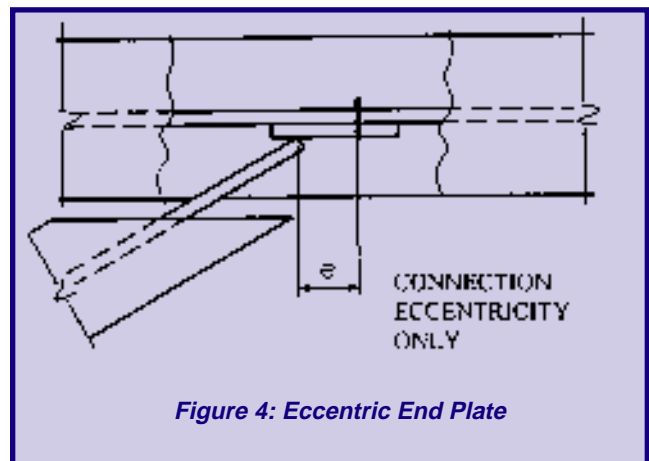


Figure 4: Eccentric End Plate

AISC Tables to select plate size, bolts, and weld capacity. Note that there is no eccentricity with this joint. The weld detail, however, has to be adjusted for the actual geometry of the joint in a manner similar to the shear plate.

Single bent plates (Fig. 3a) can be sized for either bolted or welded connections using procedures similar to those in the AISC Manual for single angle connections. These involve two eccentricities, e_1 , and e_2 from the bend line.

The eccentricities are measured from the bend line because the plate at this point has effectively zero bending strength because of the bend, i.e., the plate is "kinked" at this point.

A variation on the single bent plate of Fig. 3a is shown in Fig.

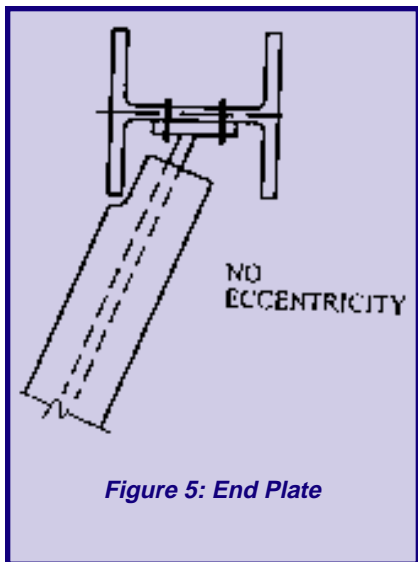


Figure 5: End Plate

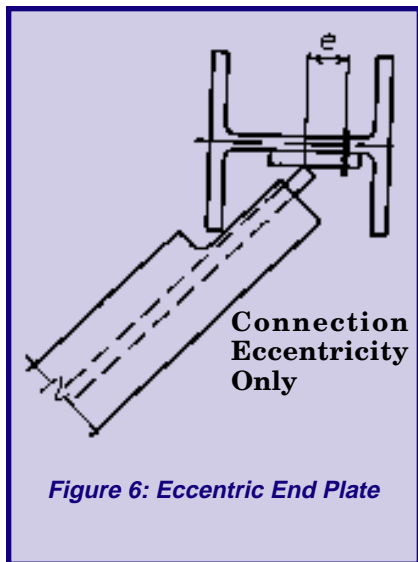


Figure 6: Eccentric End Plate

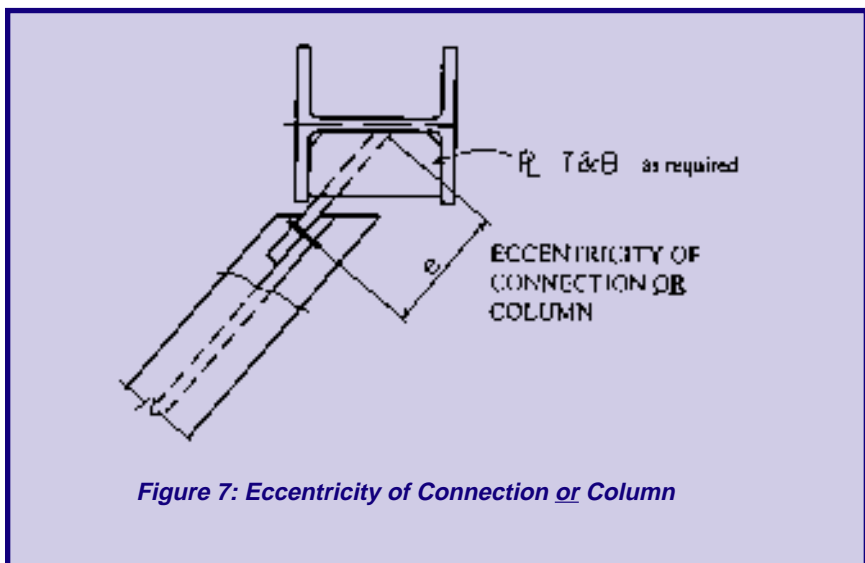


Figure 7: Eccentricity of Connection or Column

3b, where an angle is used providing a 90° bend. The eccentricities e_1 , and e_2 are again measured from the “bend line,” which is at the intersection of the two legs of the angle. Note that this connection places an eccentric load on the carrying beam which may need to be considered in the design of this beam.

Eccentric end plates (Fig. 4) can be easily sized for the eccentricity e using the tables in the AISC Manual for eccentrically loaded bolt groups.

Configurations For Skewed Connections To Columns

Skewed connections to wide flange columns present special problems. Connections to webs have very limited access and except for columns where the flange width is less than the depth, or for skews less than 30°, connections to flanges are preferred.

When connecting to column webs, it may be possible to use either a standard end plate or eccentric end plate as shown in Figs. 5 and 6. Single plate con-

nections should not be used unless the bolts are positioned outside the column flanges. This will make the connection so eccentric that top and bottom plates, as shown in Fig. 7, may be needed. Extending the single plate increases the connection cost and, unless the connection is designed for the increased eccentricity (e of Fig. 7), the column design may need to account for it. Except for Fig. 7, the eccentricities for these connections are the same as similar connections to beam webs.

Skewed connections to the column flange will also be eccentric when the beam is aligned to the column centerline. However, if the beam alignment is centered on the flange, as shown in Fig. 8, the minor axis eccentricity is eliminated and the major axis eccentricity will not generally govern the column design. The connection eccentricity is related to the parameter a here in the same way as was discussed for Fig. 1. It is reasonable to assume that the column provides a rigid support in Fig. 8.

When the beam is aligned to the column centerline either single plates (Fig. 9), eccentric end plates (Figs. 10 and 11), or single bent plates (Fig. 12) can be used. The eccentricity for each of these connections is again similar to that for the same connection to a beam web. An additional eccentricity e_2 , which causes a moment about the column weak axis, is present in these connections as shown in Figs. 9 - 12. The column may need to be designed for this moment.

A special skewed connection is often required when there is another beam framing to the column flange at 90°. If the column flange is not wide enough to accommodate a side by side connection, a bent plate can be shop welded to the column with matching holes for the second beam as shown in Fig. 13. The plate weld is sized for the eccentricity e_2 , plus any requirement for development as a fill plate in the orthogonal connection, and

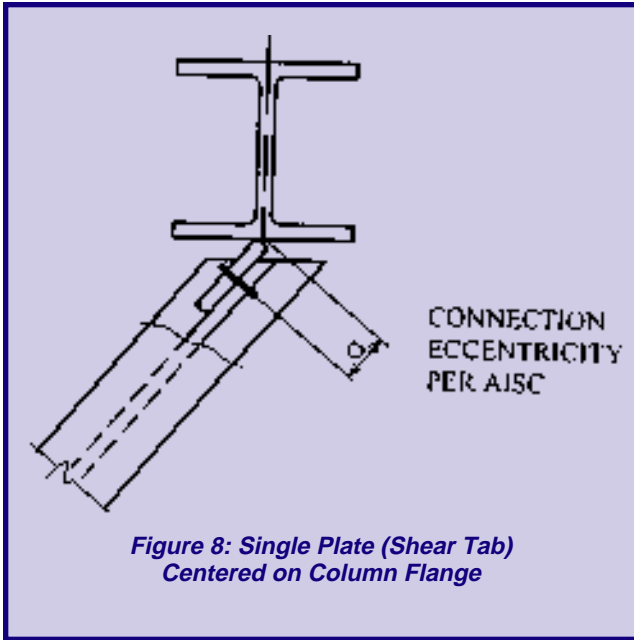


Figure 8: Single Plate (Shear Tab)
Centered on Column Flange

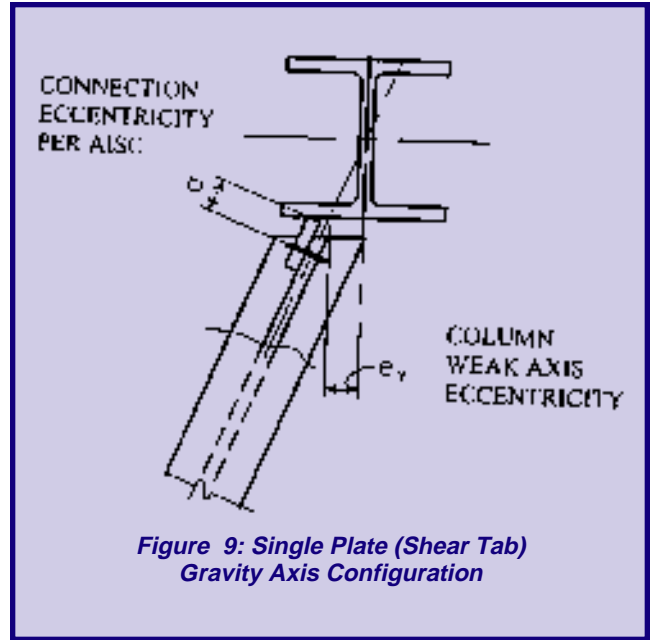


Figure 9: Single Plate (Shear Tab)
Gravity Axis Configuration

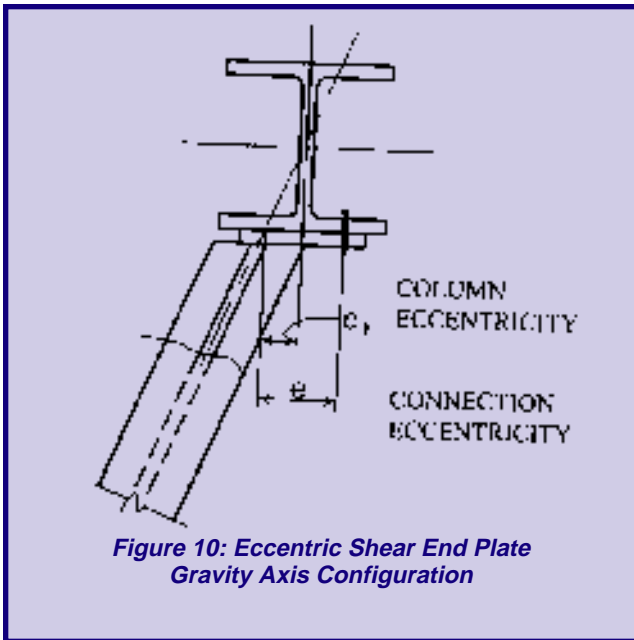


Figure 10: Eccentric Shear End Plate
Gravity Axis Configuration

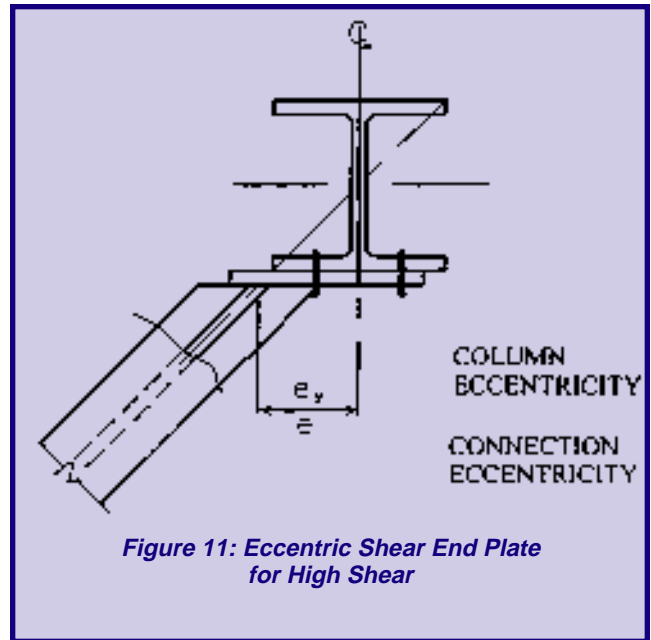


Figure 11: Eccentric Shear End Plate
for High Shear

the column sees an eccentric moment due to e_y which equals e_2 in this case.

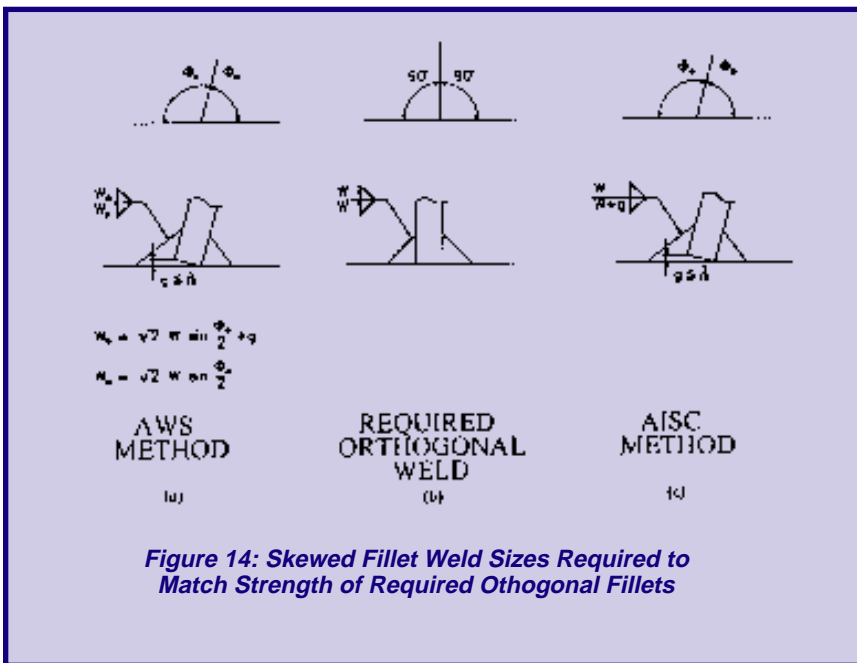
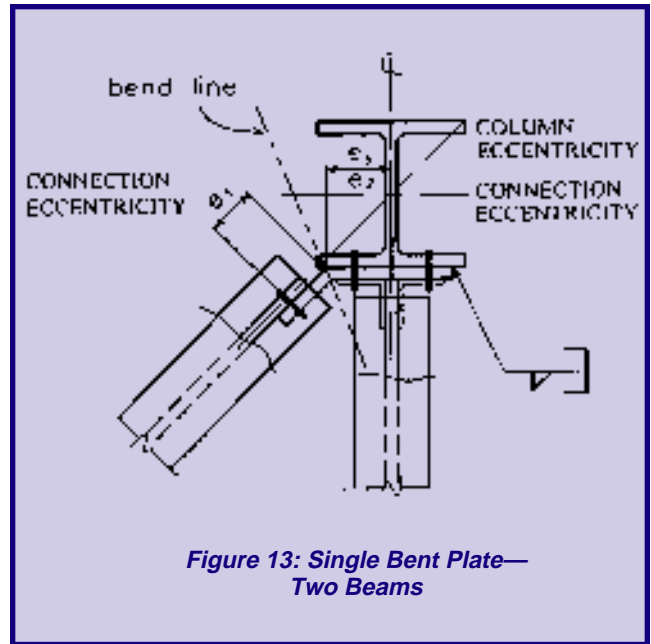
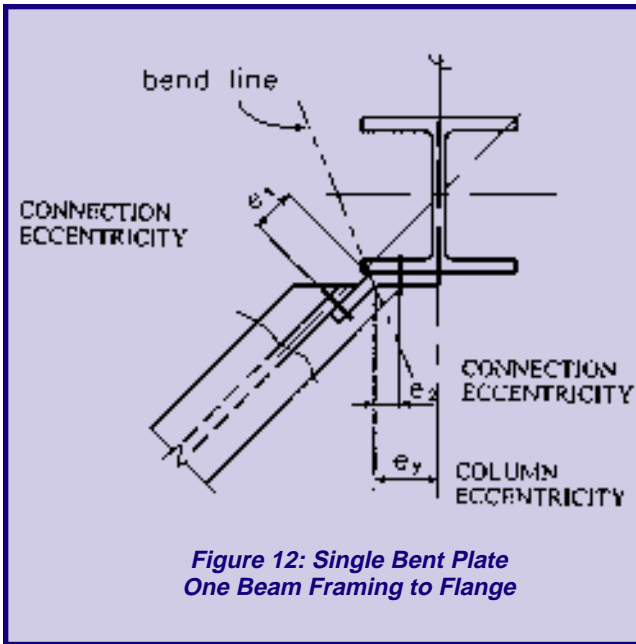
Methods For Determining Strength Of Skewed Fillet Welds

The AISC tables in the Manual of Steel Construction for single plates and end plates are

based on using standard AWS equal leg fillet welds. The single plate weld is sized to equal or exceed the strength of the plate which results in a fillet weld size of $0.75 \times t$ for orthogonal connections. The end plate weld is sized to carry the applied load. These standard orthogonal fillet welds of leg size W (Fig. 14) need to be modified as the skew becomes more acute in order to maintain the required capacity. There are two ways to do this. The AWS D1.1 Structural

Welding Code provides a method to calculate the effective throat for skewed T-joints with varying dihedral angles which is based on providing equal strength in the obtuse and acute welds. This is shown in Fig. 14a. The AISC Method is simpler, and simply increases the weld size on the obtuse side by the amount of the gap as is shown in Fig. 14b.

Both methods can be shown to provide a strength equal to or greater than the required



orthogonal weld size of W . The main difference with regard to strength is that the AWS method maintains equal strength in both fillets, whereas, the AISC method increases the strength on the acute side by maintaining a constant fillet size W while the increased size $W + g$ on the obtuse side actually loses strength because of the gap g . Nevertheless, it can be shown that the sum of the strengths of these two fillet welds W and $W +$

g is always greater than the $2W$ of the required orthogonal fillets.

It should be noted that the gap g is limited by AWS to a maximum value of 3/16 inch for both methods.

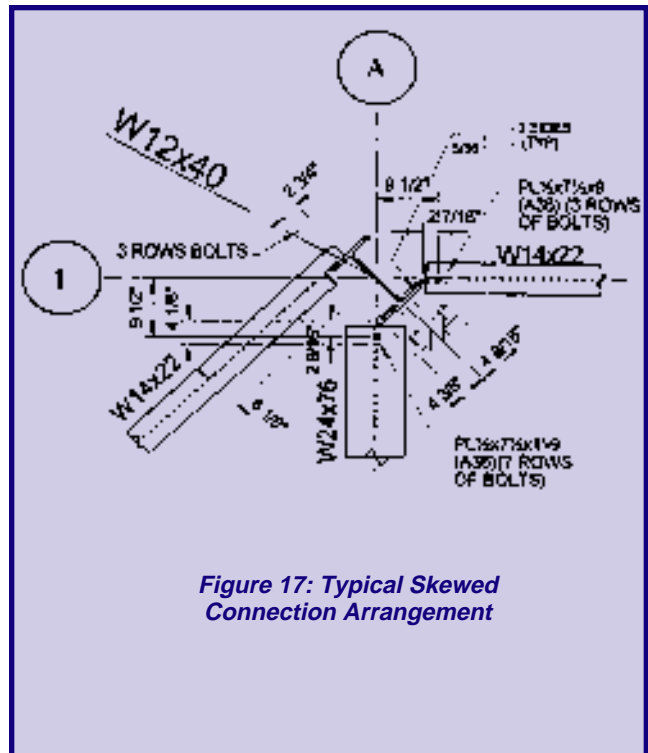
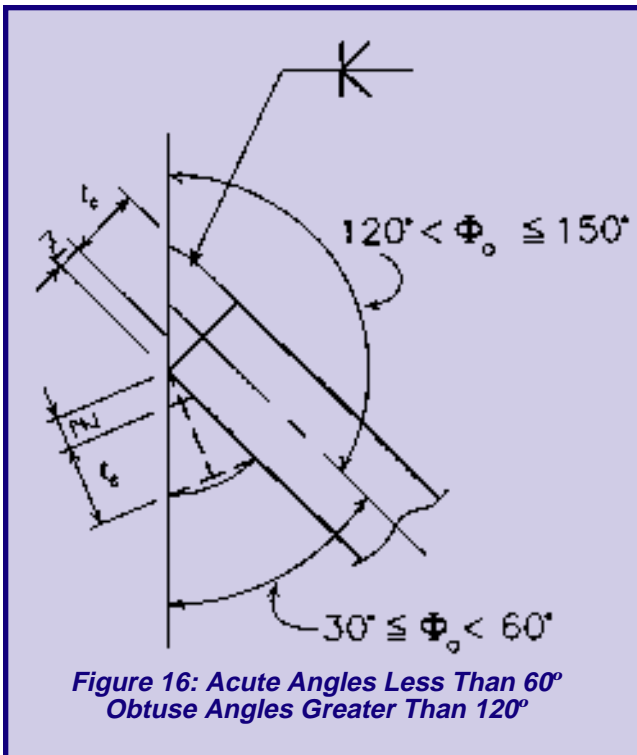
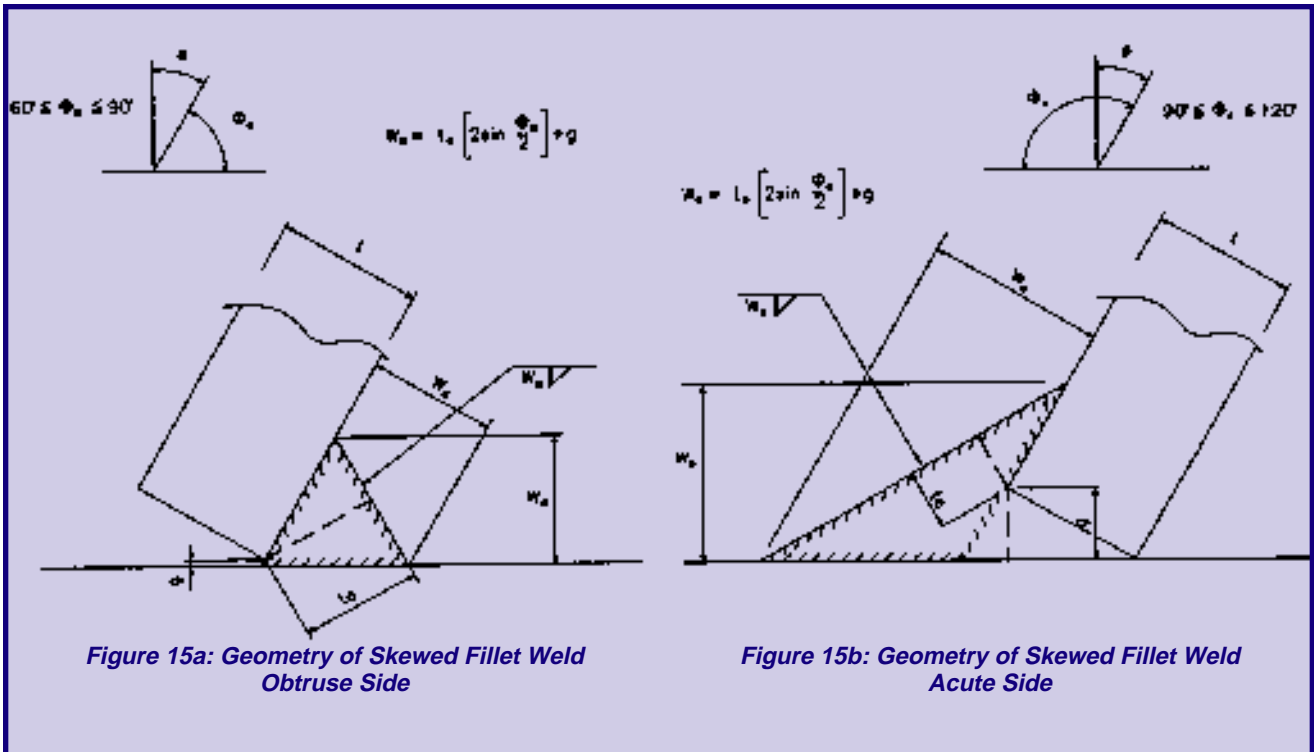
The effects of the skew on the effect throat of fillet weld can be very significant as shown in Fig. 15. Fig. 15 also shows how fillet legs WO and WA are measured in the skewed configuration. Note that in non-orthogonal fillets, the "leg size" is not the con-

tact length of the leg as would be the case for orthogonal fillets, but rather it is the projection of the contact length of one leg on a line perpendicular to the other leg. This is done to enable these fillet welds to be measured. On the acute side of the connection the effective throat for a given fillet weld size gradually increases as the connection intersection angle changes from 90° to 60°. From 60° to 30°, the weld changes from a fillet weld to a partial penetration groove weld (Fig. 16) and the effective throat decreases due to the allowance Z for the un-welded portion at the root. While this allowance varies based on the welding process and position, it can conservatively be taken as the throat less 1/8 inch for 60° to 45° and less 1/4" for 45° to 30°. Joints less than

An Example

30° are not prequalified and generally should not be used.

Fig. 17 shows an arrangement of beams framing to a column, two of which are skewed and one which is off center. This often occurs to accommodate architectural features and curtain wall



requirements. Consider the design of the connection of the W24x76 on Line A to the column. The connection is similar to that shown in Fig. 12 where the eccentricities for the connection, e_1 and e_2 of Fig. 12, are taken from the bend line. The bolts are A325N, $\frac{7}{8}$ " diameter, in standard 15/16" diameter holes. The connection material is A36, and the members are A572 Grade 50. For the bolts, $e_1 = 4.125 - 1.5 = 2.625$ and for the weld, $e_2 = 3.375 - xl$ where $l = 21$ and x will be determined from AISC Manual Table 8-42. The capacity of this connection will now be determined by checking the following limit states.

1. Bolt Shear. The design strength of one bolt is $\phi r_v = 0.75 \times 48 \times 0.6013 = 21.6$ kips (AISC Spec., Sect. J3.1). With the eccentricity $e_1 = 2.625$, AISC Table 8-18 gives $C_b = 6.22$ by interpolation. The design strength of the seven bolts is thus $\phi R_n = 21.6 \times 6.22 = 134$ kips.

2. Weld Design Strength (AISC Manual Table 8-42). A $\frac{5}{16}$ " fillet weld of "C" shape is indicated in Fig. 17. With $l = 21$, $kl = 3$, and $al + xl = 3.375$, $k = 0.143$, $x = 0.017$, $al = 3.375 - 0.017 \times 21 = 3.018$, $a = 0.144$, and $C_w = 1.85$. The weld design strength is thus $\phi R_n = 1.85 \times 5 \times 21 = 194$ kips.

3. Bearing on the W24x76 Web (AISC Spec., Sect. J3.10). $\phi R_n = 0.75 \times 2.4 \times t_w \times d \times C_b = 0.75 \times 2.4 \times 65 \times 0.440 \times 0.875 \times 6.22 = 280$ kips. This calculation assumes that the edge distances equal or exceed $1.5d$ and the spacing equals or exceeds $3d$, which is the case here.

4. Bearing on the Bent Plate (AISC Spec., Sect. J3.10). $\phi R_n = 0.75 \times 2.4 \times 58 \times 0.5 \times 0.875 \times 6.22 = 284$ kips. Again, this calculation assumes that edge distances and spacing in the plate equal or exceed $1.5d$ and $3d$, respectively, which is the case here.

5. Gross Shear - Bent Plate (AISC Spec., Sect. F2.2). $\phi R_n = 0.9 \times 0.6 \times F_y \times t_w \times l = 0.9 \times 0.6 \times 36 \times 0.5 \times 21 = 204$ kips.

6. Net Shear - Bent Plate (AISC Spec, Sect. J4.1). $\phi R_n = 0.75 \times 0.6 \times F_u \times A_{net}$. $A_{net} = 0.5(21 - 7 \times (0.9375 + 0.0625)) = 7.0$ in². $\phi R_n = 0.75 \times 0.6 \times 58 \times 7.0 = 183$ kips.

7. Net Bending Strength of Bent Plate. From AISC Manual Table 12.1, the net section modulus is $S_n = 24.8$ in³.

$$\phi R_n = \frac{0.75 F_y S_n}{e_1} = \frac{0.75 \times 58 \times 24.8}{2.625} = 411 \text{ kips}$$

8. Gross Bending Strength of Bent Plate. The gross section modulus near the bolts is $S = 0.25 \times 0.5 \times 212 = 55.1$.

$$\phi R_n = \frac{0.90 F_y S}{e_1} = \frac{0.90 \times 36 \times 55.1}{2.625} = 680 \text{ kips}$$

The design strength of the connection is the least of the Limit State values given above, or $\phi R_n = 134$ kips, and the bolts control. The last two limit states, Numbers 7 and 8, may not seem very important because they yield such large design strengths, 411 kips and 680 kips, respectively. For shallow connections, ie, small, and larger eccentricities, they can become the controlling limit states.

This paper is part of the 1999 North American Steel Construction Conference session: "Research II (Topic: Design of Skewed Connections).