Design of Skewed Connections

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ABSTRACT

Skewed connections result when members frame to each other at an angle other than 90º. This paper provides some guidance in the choice of connection, based on safety and economy, and discusses some of the design considerations required for these connections.

INTRODUCTION

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 (Figure 1) and end plates (Figure 2). Single bent plates (Figure 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 (Figure 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-in. hole gauge from the AISC *LRFD Manual of Steel Construction* Connection Tables (AISC, 1994), single plates can be utilized for intersection angles of 90º to 30º. 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 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 Figure 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

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

where n is the number of bolts.

For a flexible support with short-slotted holes,

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

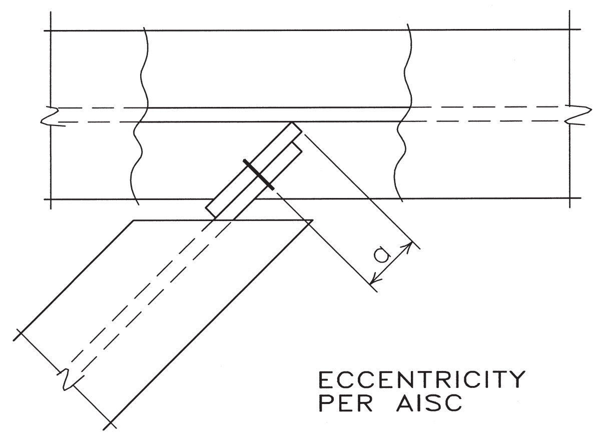

Fig. 1. Shear tab (single plate).

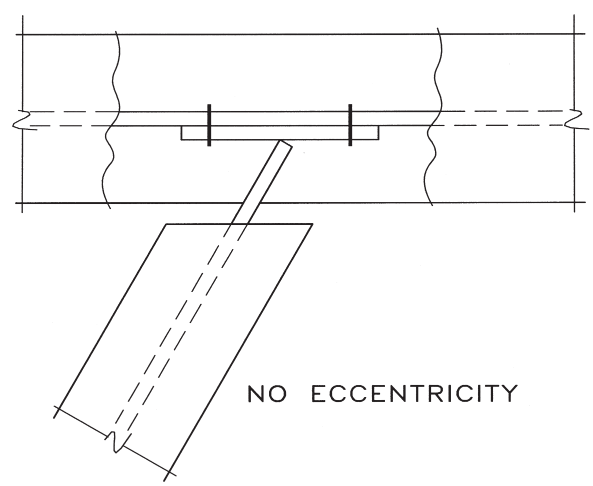

Fig. 2. Shear end plate.

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For a rigid support and standard holes,

$$e_b = \left|(n-1) - a\right|$$

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 ¾ 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 (Figure 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 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 (Figure 3A) can be sized for either bolted or welded connections using procedures similar to those in the AISC *Manual* (AISC, 1994) 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 has effectively zero bending strength out of plane.

A variation on the single bent plate of Figure 3A is shown in Figure 3B, where an angle is used providing a 90º 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 and its connections.

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

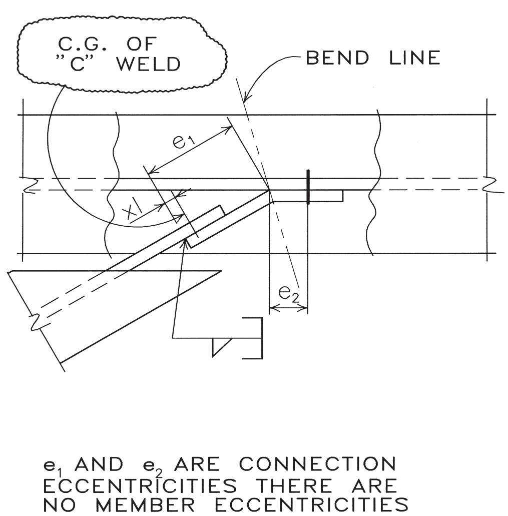

Fig. 3A. Bent plate.

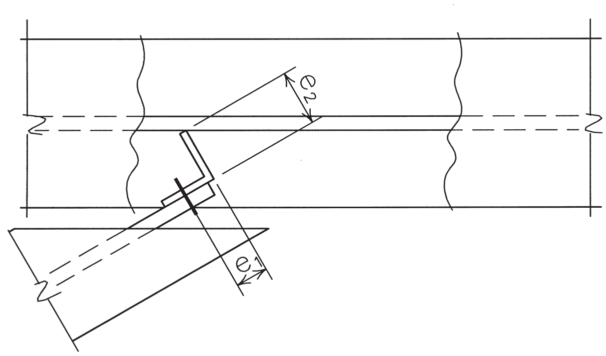

Fig. 3B. Use of angle in place of single bent plate.

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 30º, connections to flanges are preferred.

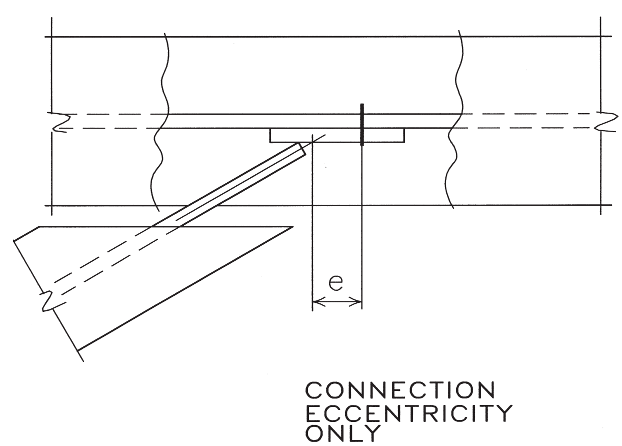

Fig. 4. Eccentric end plate.

When connecting to column webs, it may be possible to use either a standard end plate or eccentric end plate as shown in Figures 5 and 6. Single plate connections 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 Figure 7, may be needed. Extending the single plate increases the connection cost and, unless the connection is designed for the increased eccentricity (*e* of Figure 7), the column must be designed for it. Except for Figure 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 Figure 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 Figure 1. It is reasonable to assume that the column provides a rigid support in Figure 8.

When the beam is aligned to the column centroid either single plates (Figure 9), eccentric end plates (Figures 10 and 11), or single bent plates (Figure 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_y, which causes a moment about the column weak axis, is present in these connections as shown in Figures 9 through 12. The column design should be checked for this moment. In many cases, other members framing to the column may provide enough restraint to offset this eccentric moment. Note that the column eccentricities discussed here are considered about the column weak axis only. Non-skew shear connections to column flanges are not normally considered to induce any significant moment about the column strong axis, and thus the strong axis component of a skewed shear connection is ignored also.

The column eccentricities shown in Figures 9 through 12 perhaps need some further explanation. Generally, the eccentricity e_y in these figures is the distance from the point where the beam controidal axis intersects the line of the column flange face. This is clearly the case in Figures 9, 10, and 11. In Figure 12, because of the bent plate, the eccentricities are measured from the bend line as discussed earlier for the skewed connections to beams. The shear load is delivered as a shear only load at this point. As discussed subsequently in the example section of the paper, this point is usually taken at the face of the bent plate at the inside of the bend. This is what is shown in Figure 12.

A special skewed connection is often required when there is another beam framing to the column flange at 90º. 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 Figure 13. The plate weld is sized for the eccentricity e_2

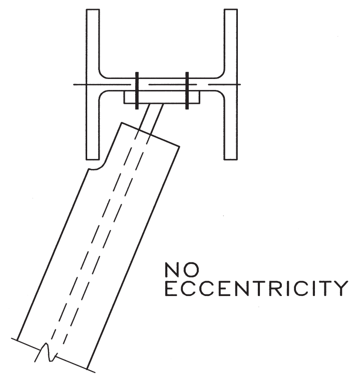

Fig. 5. End plate.

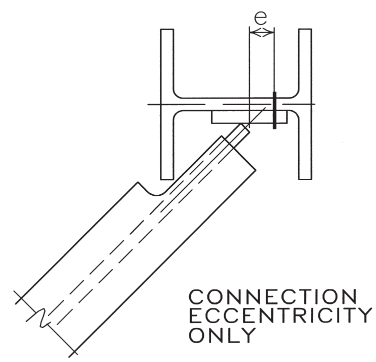

Fig. 6. Eccentric end plate.

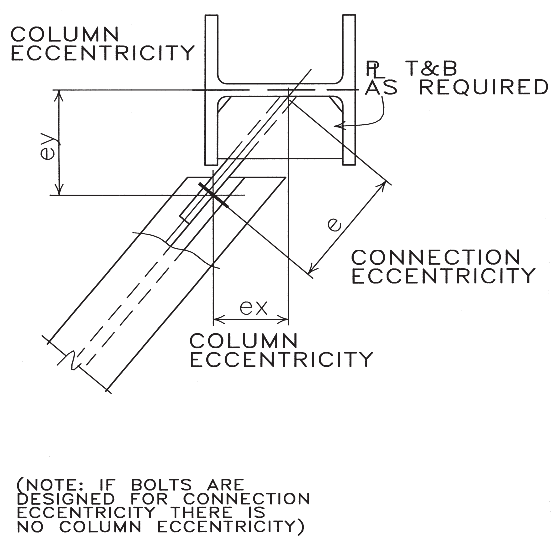

Fig. 7. Single plate (extended shear tab).

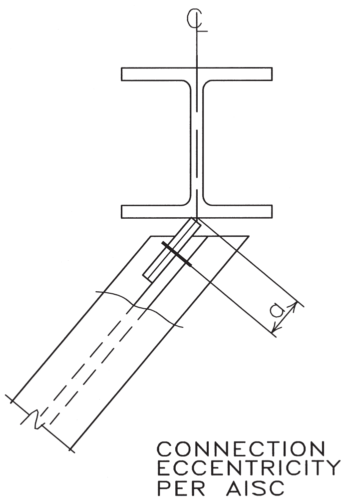

Fig. 8. Single plate (shear tab) centered on column flange.

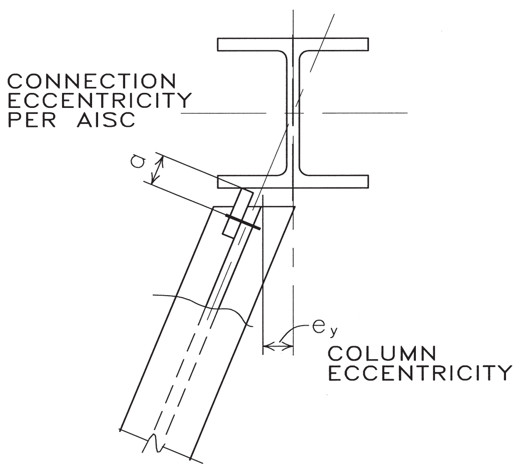

Fig. 9. Single plate (shear tab) gravity axis configuration.

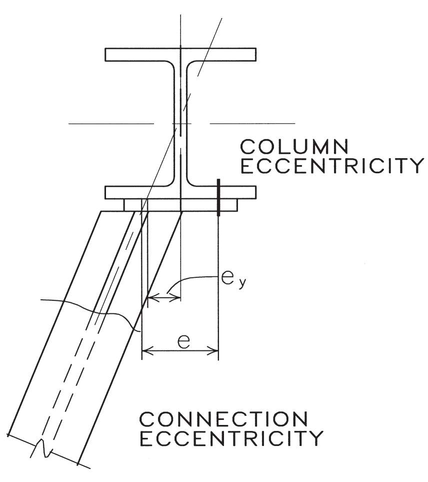

Fig. 10. Eccentric shear end plate gravity axis configuration.

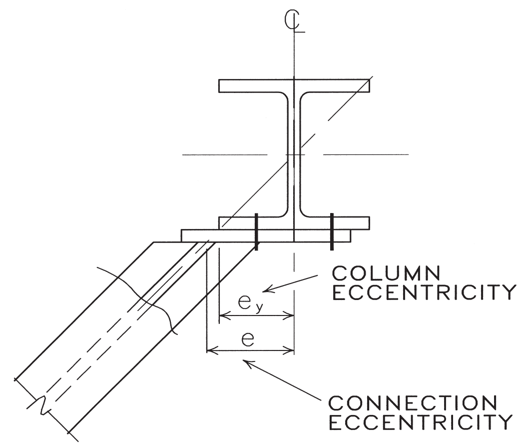

Fig. 11. Eccentric shear end plate for high skew.

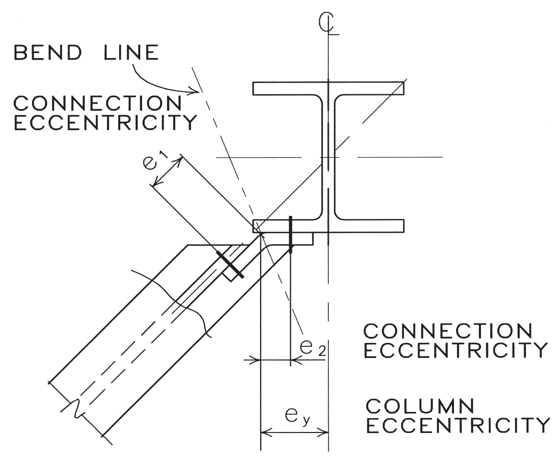

Fig. 12. Single bent plate one beam framing to flange.

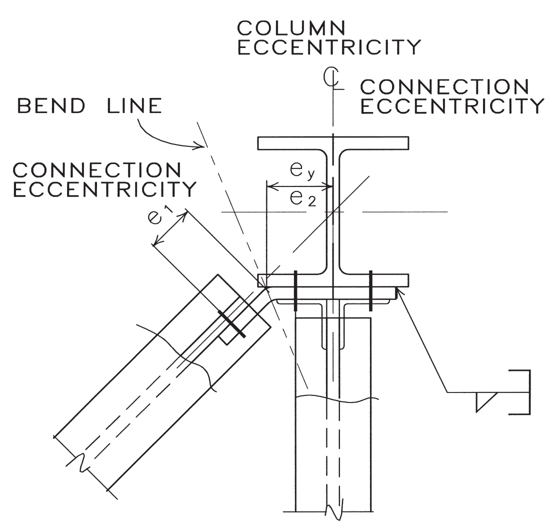

Fig. 13. Single bent plate – two beams.

plus any requirement for development as a fill plate in the orthogonal connection, and 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 *Manual* Tables 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 $0.75t$ for orthogonal connections. The end plate weld is sized to carry the applied load. These standard orthogonal fillet welds of leg size W (Figure 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 (AWS, 2000) 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 Figure 14a. Table II-1 of Annex II of AWS D1.1 provides coefficients based on the formulas of Figure 14a to size welds of equal strength for various dihedral angles. The AISC Method (AISC, 1994, pp. 9-232 and 9-233) is simpler, and simply increases the weld size on the obtuse side by the amount of the gap as is shown in Figure 14c.

Both methods can be shown to provide 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 and the geometrical reduction in effective throat, due to the dihedral angle being greater than 90°. 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 $\frac{3}{16}$ in. for both methods.

The effects of the skew on the effective throat of fillet welds can be very significant as shown in Figures 15A and 15B. These figures also show how fillet legs W_o and W_a are measured in the skewed configuration. Note that in non-orthogonal fillets, the "leg size" is not the contact 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 ϕ changes from 90º to 60º. From 60º to 30º, the weld changes from a fillet weld to a skewed T-joint (Figure 16) and the effective throat decreases due to the allowance Z (AWS D1.1, 2000, Table 2.2) for the unwelded portion at the root. While this allowance varies based on the welding process and position, it can conservatively be taken as the throat less $\frac{1}{8}$ in. for 60º to 45º and $\frac{1}{4}$ in. for 45º to 30º. Joints less than 30º are not prequalified and generally should not be used.

AWS D1.1 recommends that the contract drawings either specify the required strength (load) or the effective throat required. The fabricator then sizes the weld based on the process and position that will be used.

A $\frac{3}{16}$-in. root gap occurs for $\frac{3}{8}$-in. thick material at a connection intersection angle ϕ (Figures 15A and 15B) of 30°, i.e. $g = t \sin\phi = 0.375 \times \sin 30° = 0.1875$. For thicker material or larger angles, the contact point of the skewed material will need to be chamfered to reduce the gap to $\frac{3}{16}$ in. or less.

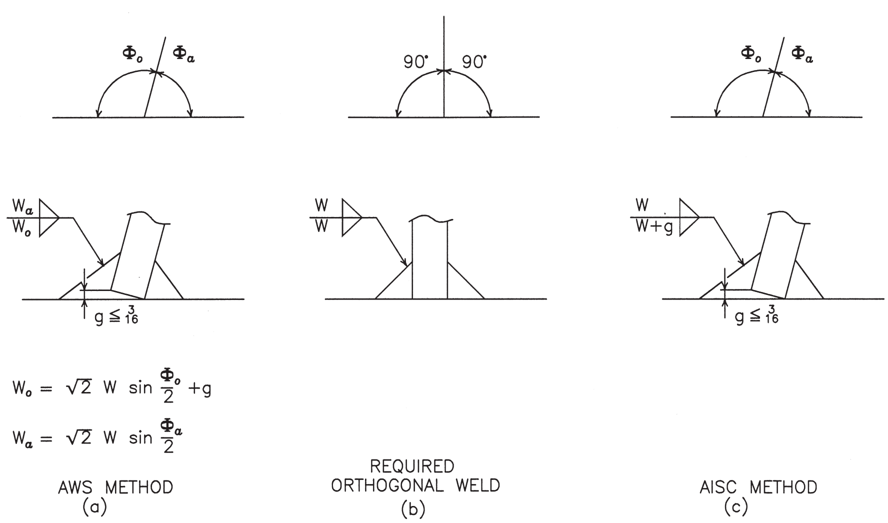

Fig. 14. Skewed fillet weld sizes required to match strength of required orthogonal fillets.

The joint of Figure 16 is shown as a partial joint penetration (PJP) weld by the weld symbol. This is done for convenience in order to allow the required effective throat to be specified. As noted above, it is a skewed T-joint and, as such, is prequalified by AWS D1.1. It should be noted, however, that the obtuse side weld which is placed in the opening between the end of the skewed material and the other material will not be able to be achieved if $Z + t_e > t$. In this case, the skewed material will need to be chamfered and the weld on the obtuse placed as shown in Figure 15B.

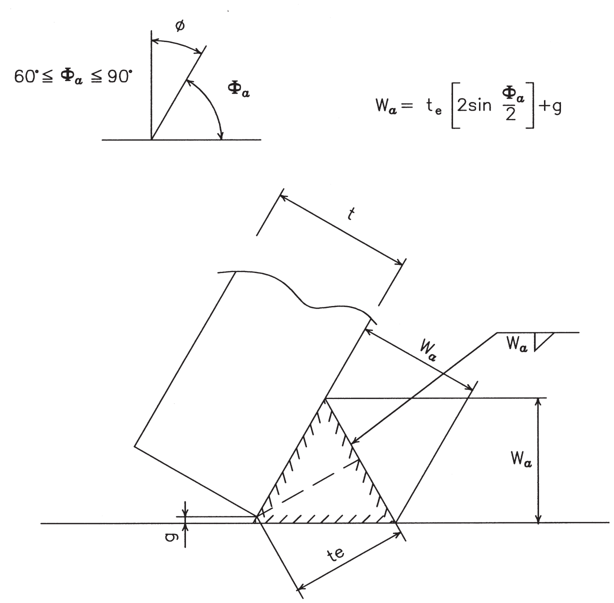

Fig. 15A. Geometry of skewed fillet weld acute side.

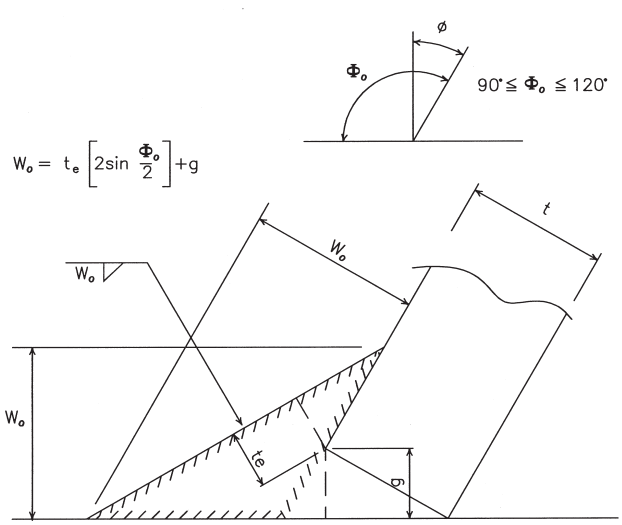

Fig. 15B. Geometry of skewed fillet weld obtuse side.

AN EXAMPLE

Figure 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 W24×76 on Line A to the column. The bolts are A325-N, 7/8-in. diameter, in standard 15/16-in. diameter holes. The connection material is A36, and the members are A572 Grade 50 or A992. The connection is similar to that shown in Figure 12 where the eccentricities for the connection, e_1 and e_2 of Figure 12, are taken from the bend line. Bent plates are usually dimensioned to the inside of the bend. Thus, in Figure 17, the bent plate dimensions for the beam on Column Line A are 4 1/8 in. and 4 3/8 – 1 = 3 3/8 in., which add up to the 7 1/2-in. dimensioned length. It is usual practice to derive the eccentricities from these dimensions. Thus, for the bolts, e_1 = 2.5625 and for the weld, e_2 = 3.375 – xl where l = 21 in. 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 ϕr_v = 0.75 × 48 × 0.6013 = 21.6 kips (AISC Spec., Sect. J3.6 (AISC, 1993)). With the eccentricity e_1 = 2.5625 in., the AISC *Manual* Table 8-18 gives C = 6.24 by interpolation. The design strength of the seven bolts is thus ϕR_n = 21.6 × 6.24 = 135 kips.

2. *Weld Design Strength* (AISC *Manual* Table 8-42): A 5/16-in. fillet weld of "C" shape is indicated in Figure

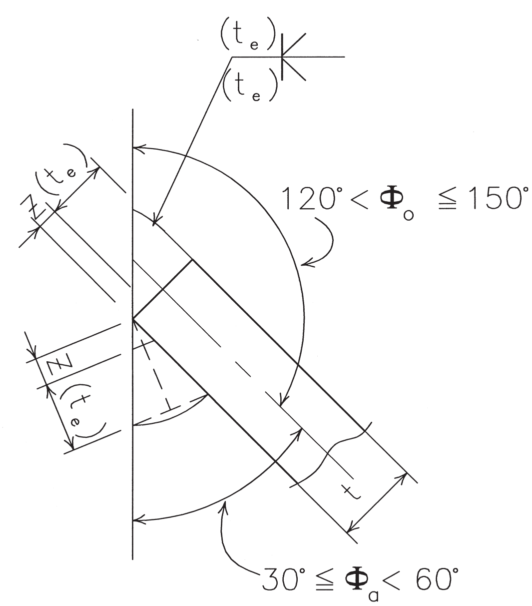

Fig. 16. Skewed T-joint acute angles less than 60°, obtuse angles Greater than 120°.

17. With l = 21 in., kl = 8/2 − 1 = 3 in., and $al + xl$ = 3.375 in., k = 0.143, x = 0.017 by interpolation in Table 8-42, p. 8-187. Therefore, al = 3.375 − 0.017 × 21 = 3.018 in., a = 0.144, and C = 1.85. The weld design strength is thus ϕR_n = 1.85 × 5 × 21 = 194 kips.

3. *Bearing on the W24×76 Web* (AISC Spec., Sect. J3.10): $\phi R_n = 0.75 \times 2.4 \times F_u \times t_w \times d \times C = 0.75 \times 2.4 \times 65 \times 0.440 \times 0.875 \times 6.24$ = 281 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): $\phi R_n = 0.75 \times 2.4 \times 58 \times 0.5 \times 0.875 \times 6.24$ = 285 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. J5.3): $\phi R_n = 0.9 \times 0.6 \times F_y \times t_w \times l = 0.9 \times 0.6 \times 36 \times 0.5 \times 21$ = 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 × (0.9375 + 0.0625)) = 7.0 in.^{2}. $\phi R_n = 0.75 \times 0.6 \times 58 \times 7.0$ = 183 kips.

7. *Net Bending Strength of Bent Plate*: From AISC *Manual* Table 12-1, the net section modulus is S_n = 24.8 in.^{3}.

$$\phi R_n = \frac{0.75 \times F_u \times S_n}{e_1} = \frac{0.75 \times 58 \times 24.8}{2.5625} = 421 \text{ kips}$$

8. *Gross Bending Strength of Bent Plate*: The gross section modulus near the bolts is $S = 1/6 \times 0.5 \times 21^2$ = 36.7 in.^{3}.

$$\phi R_n = \frac{0.9 \times F_y \times S}{e_1} = \frac{0.9 \times 36 \times 36.7}{2.5625} = 464 \text{ kips}$$

The design strength of the connection is the least of the Limit State values given above, or ϕR_n = 135 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, i.e. 421 kips and 464 kips, respectively. For shallow connections, i.e., when fewer rows of bolts are used (l smaller) or eccentricities are larger, they can become the controlling Limit States. The AISC Manual Table 9-10 could have been used to check the bolted side of this connection. With a flexible support, standard holes, and an eccentricity of 3 in. (greater than actual eccentricity of e_1 = 2.5625 in.), Table 9-10 gives ϕR_n = 131 kips.

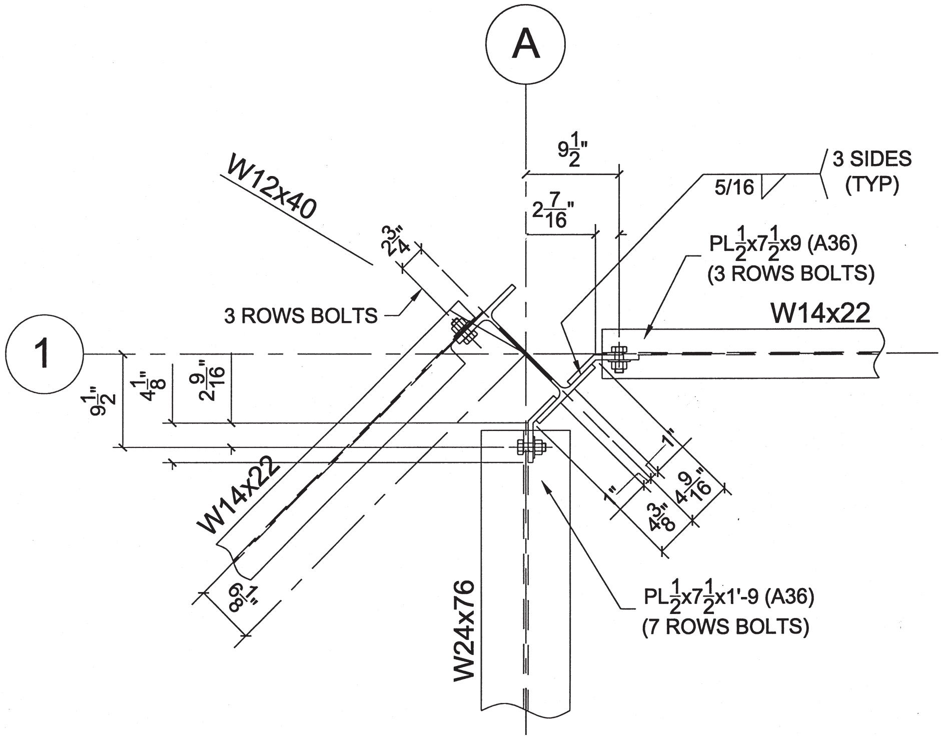

Fig. 17. Typical skewed connection arrangement.

CONCLUSIONS

Skewed connections are very common in steel construction. Very little guidance is available for their economic selection and their analysis and design. The purpose of this paper is to fill this gap with suggested connections and to provide a consistent limit state approach to their analysis and design as shown in the provided example design. As noted in the paper, the selected connection configuration in some cases may affect the design of the beam or column, which supports the skewed beam.

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