Design of Tee Framing Shear Connections

ABOLHASSAN ASTANEH AND MARWAN N. NADER

INTRODUCTION

Tee framing connections are listed as one of the shear connections in the AISC Manual[11] and are used in steel structures as simple connections. However, the published information on actual behavior and design of this type of connection is very limited. Figure 1 shows typical applications of tee connections in steel structures. The fasteners in tee connections can be either weld lines or bolts. Figure 2 shows four variations of tee connections with various combinations of bolts and welds. Based on interviews with steel fabricators, it appears that the degree of popularity of four connections in Fig. 2 is in the order of types "a", "c", "d" and "b" where type "a" is the most popular and efficient one. This paper is concerned with the behavior and design of tee connections shown as "a" in Fig. 2 where the tee section is bolted to the beam web and welded to the support.

Shear-rotation Relationship

Like any shear connection, tee framing shear connections should satisfy dual criteria of shear strength and rotational ductility as stated in the AISC Manual.[11] The connection should have enough shear strength to transfer the reaction of the beam. In addition, the connection should be flexible enough to rotate and release any significant moment that develops in the connection. Since relatively large rotations in the order of 0.04 radians are expected to develop at the ends of simply supported beams when a plastic hinge forms at midspan,[1,2] the connections should have sufficient rotational ductility to accommodate the large rotations without fracture.

To establish the shear-rotation relationship that will exist in the connections of simply supported beams, an extensive analytical study was undertaken.[1,2] In the study, a computer program was developed and used to simulate increased monotonic uniform loading of a beam supported by simple connections until the beam collapsed. The computer program was used to perform inelastic analyses of beams having all the cross sections from W16 to W33 that are listed in the AISC Manual.[11] Spans of 10, 30 and 50 feet were considered for all beams. The material of the beams were A36 and grade 50 steel.

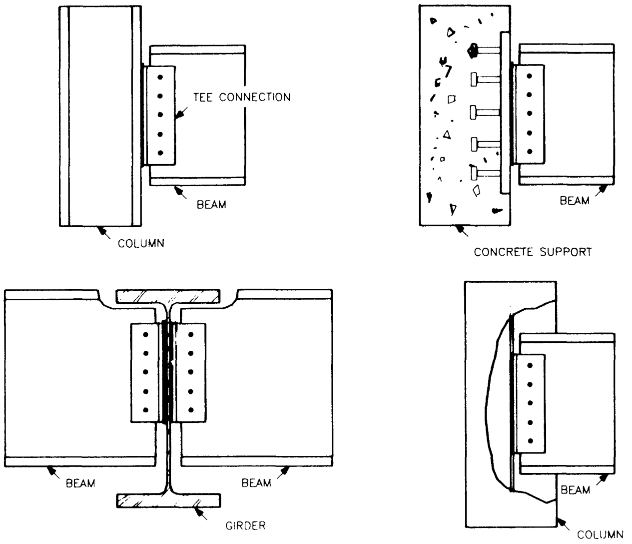


Fig. 1. Typical Applications of Tee Framing Shear Connections

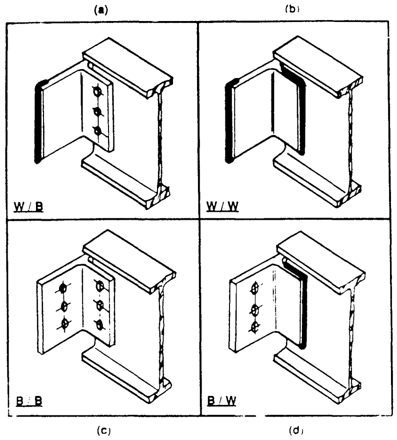


Fig. 2. Types of Tee Framing Connections

Abolhassan Astaneh is assistant professor, University of California at Berkeley.
Marwan N. Nader is graduate research assistant, University of California at Berkeley.

The studies indicated that shear-rotation relationship for the ends of simply supported beams could be represented by the line A in Fig. 3. Line A was used as the load path in experimental studies of tee framing connections reported in Reference 5 and summarized in this paper. The load path A in Fig. 3 is established by assuming that the end rotation of the beam will reach 0.03 radians when the connection reaches its capacity. The inelastic analyses of the beams[1,2] indicated that in all cases, when the midspan moment was more than $0.999M_p$, the end rotation was close but less than 0.03 radians. Therefore, the rotational ductility demand on connection was assumed to be about 0.03 radian when the beam plastic collapse is eminent. Consequently, the rotational ductility demand was set at 0.03 radians when the connection reaches its shear capacity and beam midspan moment reaches M_p. It should be noticed that load path A in Figure 3 is a conservative load path in most cases. A more realistic load path shown as curve C was used in a parallel research project to study single plate shear connections.[3,4]

The load path shown as line A in Fig. 3 was used in tests conducted to measure the realistic shear strength of tee connections whereas the load path B was used in tests conducted to measure moment-rotation response of tee connections.

Experimental Research Program

The research project summarized here consisted of testing nine tee framing connections with various geometries. Figure 4 shows a typical test specimen. Properties of test specimens are given in Table 1. The connections were tested under two different conditions. First, they were subjected to moments to measure their rotational flexibility and ductility. Second, the connections were subjected to realistic combinations of shear and rotation to measure their shear strength.

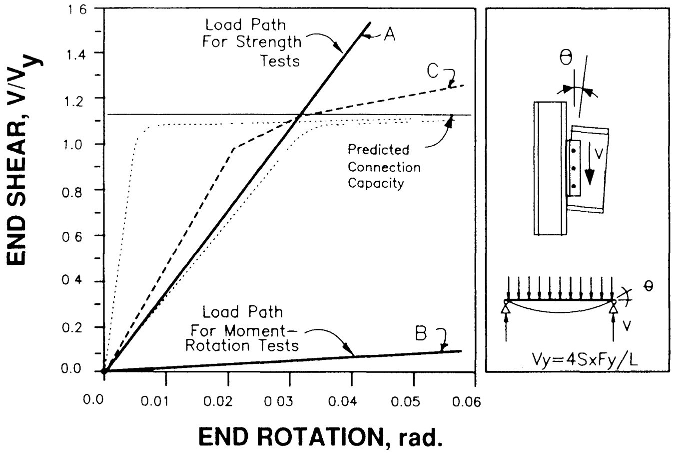


Fig. 3. *Shear-Rotation Relationship for Ends of Simple Beams*

Test Set-up

To apply shear and rotations to the connections, a special test set-up was designed and fabricated. The set-up is shown in Fig. 5 and its details are given in References 1, 2, 3 and 5.

EXPERIMENTAL RESULTS

Moment-Rotation Tests

During this phase of research the beams in the test specimens were rotated to reach a rotation of 0.07 radians. The objective was to measure flexibility and ductility of connections. Figure 6 shows plots of moment and rota-

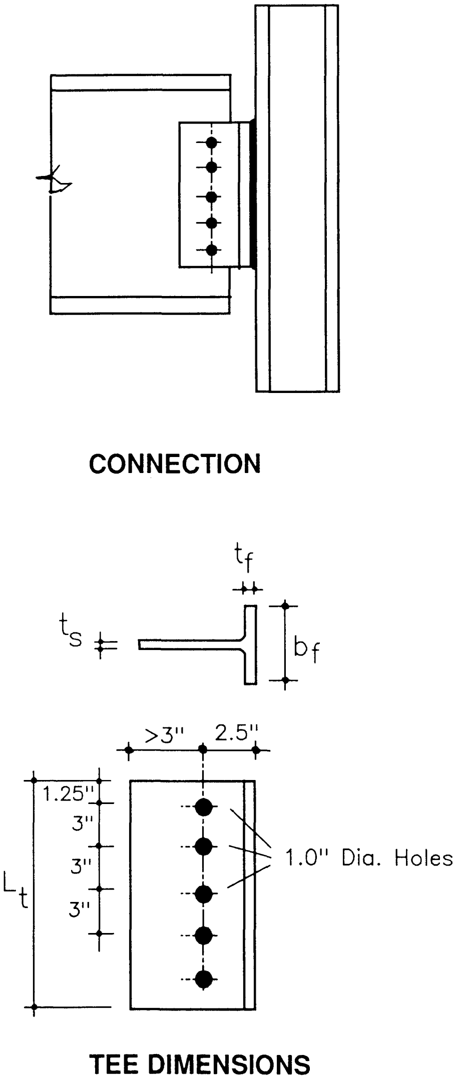


Fig. 4. *A Typical Test Specimen*

tions along the bolt line of each specimen. All specimens with the exception of specimen 4 could easily reach 0.07 radians rotation. Only minor yielding was observed in specimens during this phase. However, welds in specimen 4 suddenly fractured when rotation of connection reached 0.06 radians. Based on analysis of this failure and to avoid such failures it is recommended that L_t/b_f of the tee should not be greater than 3.50. For definition of parameters see Notation section. All connections that were tested could easily be considered simple (Type II) connections. Table 2 provides information on connection shear, rotation and moment when connection rotation reached its maximum.

Figure 7 shows specimen 8 at the end of the moment-rotation test when end rotation was equal to 0.07 radians. Other specimens had similar appearance at the end of moment-rotation tests.

After completion of moment-rotation tests, the beams were rotated back to zero rotation in preparation for the shear strength tests that followed.

Shear Strength Tests

During this phase each specimen used in moment-rotation tests was subjected to the shear-rotation history

Table 1.
Properties of Test Specimens

Test Number	WT Section	No. of Bolts	Dia. of Bolts	Type of Bolts**	Tee Dimensions				Weld Size	$\frac{t_s/b_d}{t_f/t_s}$
					L_t	t_s	b_f	t_f		
(1)	(2)	(3)	(4)	(5)	in. (6)	in. (7)	in. (8)	in. (9)	(10)	(11)
1	WT4×7.5	3	⅞	A325N	8.5	.245	4.00	.315	3/16	0.22
2	WT7×19	5	⅞	A325N	14.5	.310	6.77	.515	¼	0.21
3	WT7×19	3	⅞	A325N	8.5	.310	6.77	.515	¼	0.21
4	WT4×7.5	5	⅞	A325N	14.5	.245	4.00	.315	3/16	0.22
5	WT4×20	5	⅞	A325N	14.5	.360	8.07	.56	¼	0.26
6	WT4×20	3	⅞	A325N	8.5	.360	8.07	.56	¼	0.26
*7	WT7×19 + 0.5-in. stem	5	⅞	A325X	14.5	.500	6.77	.515	¼	0.55
*8	WT4×20 + 0.5-in. stam	3	⅞	A490X	8.5	.500	8.07	.56	¼	0.52
*9	WT4×20 + 0.5-in. stem	5	⅞	A490X	14.5	.500	8.07	.56	¼	0.52

**In these specimens, stem of* WT *was cut and replaced by ½-in. thick A36 plate.*
***Bolts were snug tight.*

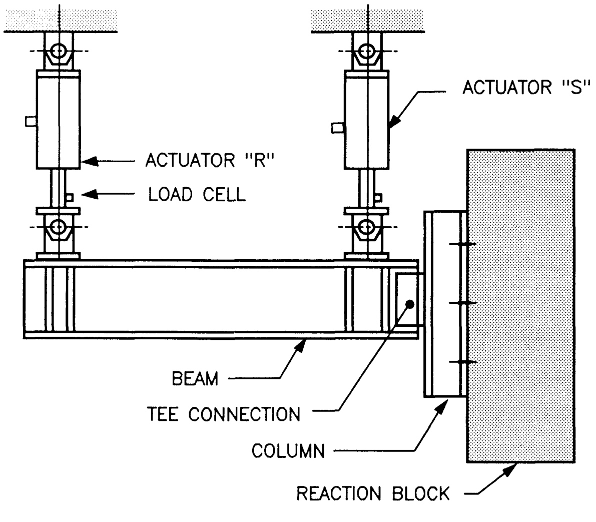


Fig. 5. Test Set-up

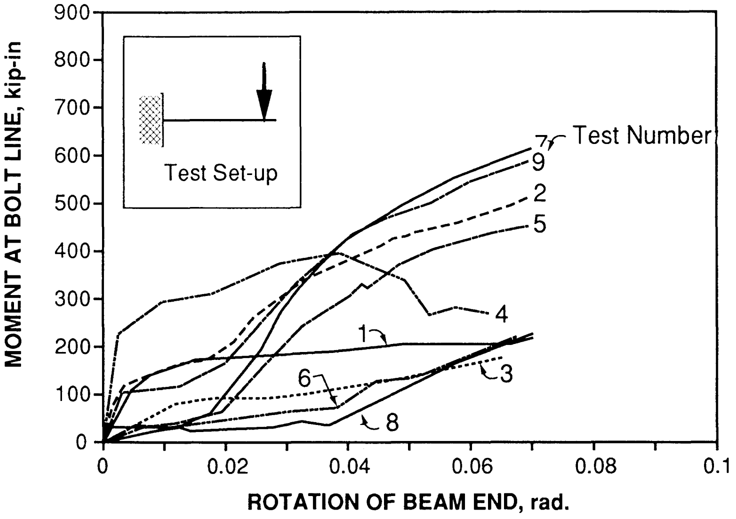


Fig. 6. Moment-Rotation Curves Obtained during Moment-Rotation Tests

of curve A in Fig. 3 until failure occurred. Figure 8 shows typical failures that occurred in test specimens. Table 3 summarizes experimental results of the shear strength tests.

DISCUSSION OF THE RESULTS OF SHEAR STRENGTH TESTS

Failure Modes

Significant shear yielding occurred in specimens 1,2,3,5, and 6 in the stem in areas located between the bolt line and weld lines (see Fig. 8a). Continued loading of specimens 2,3,5 and 6 resulted in shear fracture of net area along the edges of bolt holes as shown in Fig. 8b and 9.

Specimens 1 through 6 were cut from hot-rolled wide flange sections. As a result, the thickness of the stem in these specimens was less than the flange, causing significant yielding to occur in the stem. However, the stems in specimens 7, 8 and 9, which also were cut from wide flanges, were replaced by a 0.5-in. A36 plate. Specimens 7 and 9 failed due to bolt shear failure and the failure mode of specimen 8 was weld fracture (Fig. 8d). Also, these latter three specimens experienced significant yielding in their flanges (Fig. 8e).

Specimens 1 through 6 experienced considerable bearing yielding in their bolt holes. The bearing yielding, if it

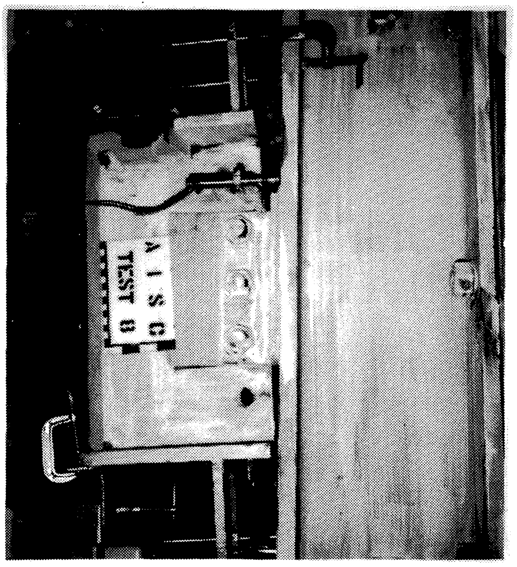
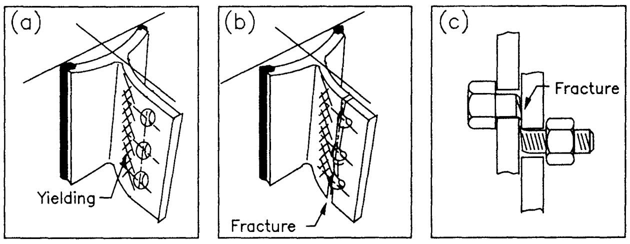
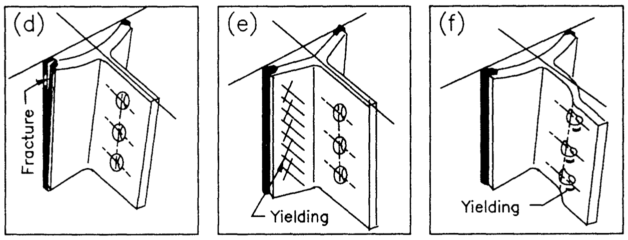


Fig. 7. A Typical Specimen at the End of Moment-Rotation Test



Yielding of Stem   Fracture of Stem   Bolt Failure



Fracture of Weld   Yielding of Flange   Bearing Failure

Fig. 8. Typical Failure Modes of Tee Connections

Table 2.
Results of Moment-Rotation Tests

Test No.	No. of Bolts	Shear Displ.	Shear Force	Rotation	Moment at Bolt Line	Moment at Weld Line
		in.	kips	rad.	kip-in.	kip-in.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	3	0.05	3.4	0.069	214.7	223.3
2	5	0.04	8.1	0.069	512.4	532.6
3	3	0.01	3.2	0.069	209.6	217.6
4	5	0.02	5.9	0.060	401.3	412.6
5	5	0.02	8.2	0.070	46.2	47.9
6	3	0.02	4.1	0.068	216.5	226.7
7	5	0.09	9.6	0.070	642.7	682.8
8	3	0.33	3.4	0.070	219.5	748.0
9	5	0.07	9.6	0.070	724.0	748.0

is not excessive, can be beneficial in releasing the rotational stiffness of the connection thus reducing connection moment. Bearing yielding and shear yielding of the stem contribute significantly to release rotational stiffness of connection. To combine these two effects a single parameter equal to $(t_s/d_b) / (t_f/t_s)$ was studied. As listed in column 11 of Table 1, $(t_s/d_b) / (t_f/t_s)$ is about ¼ for specimens that showed very desirable yield behavior (specimens 1 through 6) whereas the parameter was about ½ for specimens with less yielding and brittle fracture of welds or bolts (specimens 7,8 and 9). Therefore, relying on the behavior of nine tests, and until more research data is available, it is suggested that in tee connections, $(t_s/d_b) / (t_f/t_s)$ be about ¼. this provision resulted in sufficient ductility in test specimens.

Shear-Rotation Response

Figure 10 shows the actual shear-rotation relationship that was recorded during shear strength tests. It should be noted that in strength tests the recorded shear-rotation response is similar to the load path A shown in Fig. 3. This is due to the fact that during the tests load path A was intentionally applied to specimens. The rotation of the beam ends in all specimens reached a value of at least 0.03 radians before failure occurred. This indicates that based on the discussion of end rotation of simply supported beams, the connections that were tested could resist the shear force until the beam reached plastic collapse condition.

Movement of Point of Inflection of Beam

As the load increases in beams supported by flexible connections, due to inelastic deformations in the connections, the rotational stiffness of the connection decreases continuously. Only strain hardening or kinematic hardening due to large deformations can slow down the rate of decrease of stiffness in the connections. The decrease in rotational stiffness causes continuous decrease of moment-to-shear ratios in the connections. The decrease

Table 3.
Results of Shear Strength Tests

Test No.	No. of Bolts	Shear Displ.	Shear Force	Rotation at Bolt Line	Moment at Bolt Line	Moment at Weld Line	Failure Mode
		in.	kips	rad.	kip-in	kip-in.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	3	0.90	77	0.034	−143**	49	Stem shear yielding
2	5	0.90	174	0.032	−281	155	Stem shear yielding
3	3	0.90	107	0.037	−14	8	Stem net section and welds fractured
4***	5	*	*	*	*	*	Welds failed in Phase I
5	5	0.82	183	0.031	−300	110	Stem net section fractured
6	3	1.20	111	0.034	−128	480	Stem net section fractured
7	5	0.85	238	0.038	−37	480	Bolt shear failure
8	3	1.10	141	0.039	−393	110	Weld fracture
9	5	0.50	209	0.033	−270	110	Bolt shear fracture

**Not recorded.*
***Negative sign of moment means bottom fiber was in tension.*
****Specimen failed during moment-rotation test.*

of moment-to-shear ratio is due to the fact that in a symmetric one span beam with a uniformly distributed load, the shear reaction is a determinate parameter and is always equal to $qL/2$. Whereas the moment developed in the connections is indeterminate and depends on the rotational stiffness of the connection. For completely flexible connections the moment is zero and for completely fixed connections it is $qL^2/12$.

In beams with flexible connections, the moment-to-shear ratio is approximately equal to the distance of the point of inflection of the beam to the support. Therefore, if the location of the point of inflection is known, one can calculate the moment that will exist in the connection along any desired section. Figure 11 shows curves indicating the location of the point of inflection in test specimens as the shear force increased. The figure indicates that in all specimens the point of inflection moved rapidly toward the support and remained in the area between the bolt and weld lines.

PROPOSED DESIGN PROCEDURES

Limit States

The failure modes (limit states) that were observed during the tests are listed below in the order of their desirability in design. The limit states (a) and (b) are the most desirable limit states since they correspond to yielding of steel which is ductile and the most reliable. The limit states (e) and (f) are the least desirable since they are generally brittle and are associated with fracture which is less reliable than yielding.

The limit states are shown in Fig. 8 and are:

a. Shear yielding of gross area of stem
b. Yielding of tee flange
c. Bearing failure of beam web as well as tee stem
d. Shear fracture of net area of stem
e. Fracture of bolts connecting tee stem to beam web
f. Fracture of welds connecting tee flange to support

a. Shear Yielding of Gross Area of Tee Stem

The stem of the tee in tee framing connections is subjected to shear and a relatively small bending moment. In formulation of the proposed design equations only the shear force is considered and the effects of the bending moment is neglected. The equation defining this limit state in allowable stress design (ASD) format is:

$$f_{vy} \le F_{vy} \qquad (1)$$

where,

$$f_{vy} = R \,/\, A_{vg} \qquad (2)$$

$$F_{vy} = 0.40\ F_y \qquad (3)$$

$$A_{vg} = L_t\ t_s \qquad (4)$$

b. Yielding of Tee Flange

If thickness of the tee flange is less than the thickness of the stem, the flange will experience considerable yielding. Particularly, in built-up tees if the thickness of flange is less than ½ of the thickness of stem, this limit state can be reached before limit state (a) is reached. However, in tees produced by splitting hot rolled wide flanges, since the flange is thicker than the stem, this limit state probably will not govern.

The equation defining this limit state in ASD format is:

$$f_{vy} \le F_{vy} \qquad (5)$$

where,

$$f_{vy} = R \,/\, 2A_{vgf} \qquad (6)$$

$$F_{vy} = 0.40\ F_y \qquad (7)$$

$$A_{vgf} = L_t t_f \qquad (8)$$

Fig. 9. Failure of Net Section of Tee Stems

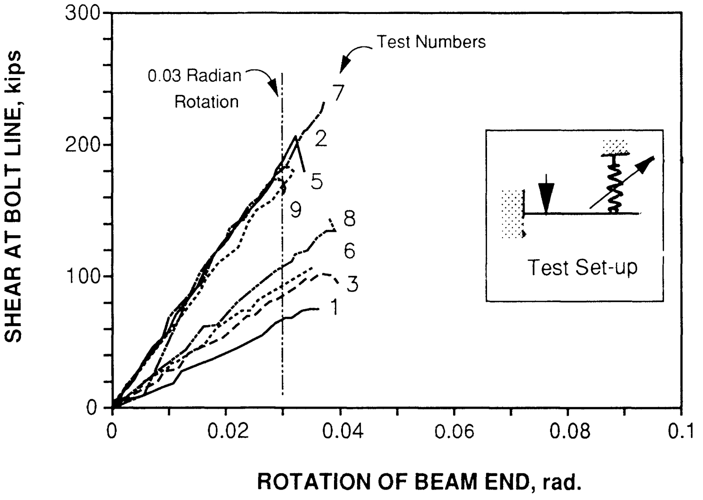


Fig. 10. Shear-Rotation Curves for Test Specimens

c. Bearing Failure of Tee Stem or Beam Web

The research[5] indicated that a limited amount of yielding in the bolt holes due to bolt bearing can be beneficial in reducing rotational stiffness of the connection. However, large bearing deformations can result in failure of connection due to fracture.

To avoid reaching this limit state, it is recommended that the established rule for horizontal and vertical edge distances of at least 1.5 times the bolt diameter be followed. The bolt spacings should satisfy requirements of the AISC-ASD Specification.[12] In all test specimens, the bolt spacing was equal to 3 in. It is recommended that in the absence of sufficient research data for other spacings, the bolt spacing be kept equal to 3 in.

d. Shear Fracture of Net Area of Tee Stem

This limit state is reached when the critical net section of tee stem fractures in shear. The research[5] clearly indicated that the critical net section in shear is located close to the edge of bolt holes and not along the centerline of the bolt holes. A more detailed explanation of the phenomenon is provided in References 3 and 5. It was shown[5] that the effective net area in shear was more realistically calculated by averaging the net area along the bolt center line and the gross area of the tee stem. The following equations define this limit state where effective net area in shear is used.

$$f_{vu} \le F_{vu} \tag{9}$$

where,

$$f_{vu} = R_o / A_{nse} \tag{10}$$

$$F_{vu} = 0.30\, F_u \tag{11}$$

$$A_{nse} = [L_t - n(\tfrac{1}{2})\,(d_b + \tfrac{1}{16})]t_s \tag{12}$$

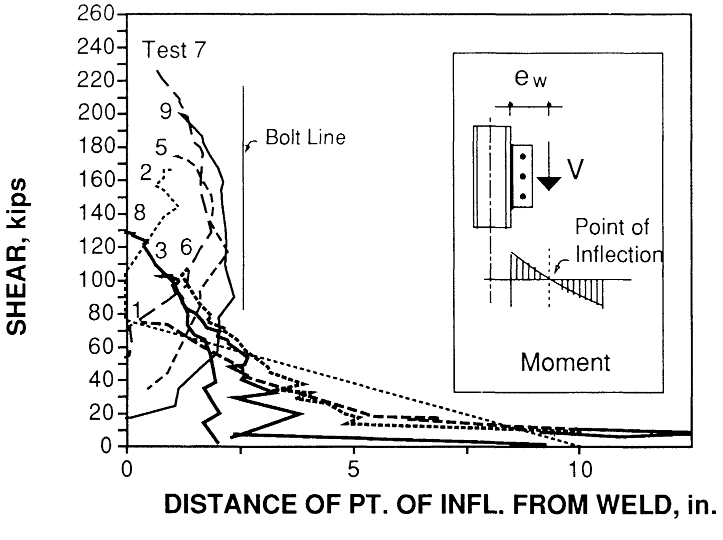


Fig. 11. Movement of Point of Inflection

The factor ½ in Eq. (12) reflects the averaging of net area and gross area of stem to obtain the effective net area in shear as defined earlier. Currently, the AISC Specification[12] recommends use of the following equation in calculation of net area in shear. This results in a more conservative design with fracture of net area normally governing the design of connection.

$$A_{nse} = [L_t - n(d_b + \tfrac{1}{16})]t_s \tag{12a}$$

If beam is coped, the block shear failure of beam web should also be considered as discussed in the AISC Manual.[11]

e. Shear Failure of Bolts

Bolts are proposed to be designed for a direct shear equal to the reaction of the beam. The eccentricity e_b for tee connections that were studied is negligible, as shown in Fig. 11. The statement is more realistic when the supporting member is a rigid element such as a non-rotating column flange or an embedded plate. For rotationally flexible supports, in the absence of experimental data, it can be assumed that point of inflection is at the weld line and the bolts are subjected to direct shear and a bending moment equal to the shear force multiplied by the distance between bolt and weld lines. The eccentricity e_b can be obtained from:

$$e_b = 0.0 \text{ (for rotationally rigid supports)} \tag{13}$$

and

$$e_b = a \text{ (for rotationally flexible supports)} \tag{14}$$

By using methods outlined in Reference 7 including using Tables X of the AISC Manual[11] the bolts are designed for the combined effects of shear R and moment equal to Re_b.

f. Weld Failure

The welds connecting the tee to the support are proposed to be designed for the combined effects of direct shear and a moment due to the eccentricity of the reaction from the weld line, e_w. The eccentricity e_w is conservatively equal to the distance between bolt and weld lines.

$$e_w = a \tag{15}$$

By using methods outlined in Reference 7 including using Tables XIX of the AISC Manual,[11] fillet welds are designed for the combined effects of shear equal to R and moment equal to Re_w.

SUMMARY OF THE PROPOSED DESIGN PROCEDURES

Based on findings of the research program a design procedure was developed and proposed in a step-by-step format in the following section. The procedure is given in the Allowable Stress Design (ASD) format. The tee framing connections covered by these procedures consist of a tee bolted to a beam web and welded to a support. The support can be a column flange, a beam or other steel elements such as embedded plates. The tee can be cut from a wide flange section or built by welding two plates.

General Requirements

In design of tee framing connections, the following requirements should be satisfied:

1. Material of the tee should be A36 steel.
2. The ratio of L_t/a of the tee stem should be more than 2.
3. ASTM A325 or A490 bolts may be used. Fully-tightened as well as snug tight bolts are permitted but snug tight bolts are preferred. If for erection purposes some bolts need to be tightened, one or two bolts at the bottom of connection can be tightened and the rest left snug tight. The bolts should be used in only one vertical row. Standard or short-slotted punched or drilled holes are permitted. The number of bolts should not be more than seven or less than two.
   The procedure is not applicable to oversized and long slotted bolt holes.
4. Welds are fillet welds with E70XXor E60XX electrodes.
5. Vertical spacing between the bolts is equal to 3 in.
6. To ensure connection flexibility, $b_f/2t_f$ ratio of tee flange should be more than 6.5.
7. The ratio of L_t/b_f of the tee should not exceed 3.5.
8. Return top of the fillet welds a distance equal to 2 times weld size.
9. If tee is welded to a column flange, it is recommended that t_{fc} of the column be greater than t_f of the tee.
10. The ratio (t_s/d_b) / (t_f/t_s) is preferred to be about ¼ to facilitate ductile behavior of tee stem.

The following steps are recommended to be taken in design of tee framing connections:

1. Select type of bolts and calculate number of bolts assuming no eccentricity:

$$n = R / (A_b F_{vb}) \tag{16}$$

If support is rotationally flexible, check the bolt group for combined effects of shear R, and moment Re_b using Table X of the AISC Manual,[11] where e_b is the distance from bolt line to the weld line.

2. Calculate required gross area of the tee stem:

$$A_{vg} = R / 0.40F_y \tag{17}$$

3. Use A36 steel and select a tee to satisfy the following requirements:

a. $b_f/2t_f$ of the tee ≥ 6.5. (18)
b. $d_b/t_s \geq 2.0$ (19)
c. l_h and $l_v \geq 1.50d_b$. (20)
d. Bolt spacings = 3 in.
e. $L_t t_s \geq A_{vg}$ (21)
f. $L_t/b_f \leq 3.5$ (22)
g. $t_{fc} > t_f$ (23)
h. $(t_s/d_b)/(t_f/t_s) \approx 1/4$ (24)

4. Calculate actual allowable shear yield capacity of gross area of stem:

$$R_o = (L_t t_s)\ (0.4F_y) \tag{25}$$

5. Check allowable capacity of effective net area of stem:

$$[L_t - (n/2)\ (\mathrm{d_b} + 1/16)]t_s\ 0.3F_u \geq R_o \tag{26}$$

or by using net area in shear as defined by AISC:[12]

$$[L_t - (n)\ (d_b + 1/16)]t_s\ 0.3F_u \geq R \tag{26a}$$

6. Check shear in the flange:

$$2L_t t_f\ (0.4F_y) \geq R \tag{27}$$

7. Design fillet welds for the combined effects of shear and moment using Table XIX.[11] Shear is equal to R_o and moment is equal to $R_o a$ where a is the distance from bolt line to weld line.

8. Check bearing capacity and satisfy the following equations for beam web and tee stem:

$$(n)\ (t_s)\ (d_b)\ (1.2F_u) \geq R_o \tag{28}$$

and for the beam web:

$$(n)\ (t_w)\ (d_b)\ (1.2F_u) \geq R_o \tag{29}$$

If the bolts are expected to resist a moment (as they normally would if the support is torsionally flexible), this calculation should reflect the reduced strength as determined by Table X of the AISC Manual.[11]

9. If beam is coped, check block shear failure of beam web.

Application to Design Problems

The following examples show how the design procedure can be implemented in design of steel tee connections.

Design Example 1

Given:

Beam:	W27 × 114, t_w=0.570 in.
Material:	A36 steel
Support:	Flange of a W10 × 77 column
Reaction:	88 kips (Service Load)

Bolts: ⅞ in. dia. A325-N (snug tight)
Bolt Spacing: 3 in.
Welds: E70XX fillet welds

Design a tee framing connection to transfer the beam reaction to supporting column.

Solution:

1. Calculate number of bolts:
   Shear = R = 88 kips
   Let us assume $M = 0$, (will be checked later)
   Since the support is relatively rigid, the moment acting on the bolts can be neglected. Therefore;
   $n = R/r_v = 88/12.6 = 6.98$

   Use: Seven ⅞ in. dia. A325-N bolts.

2. Calculate required gross ares of the tee stem:

   $A_{vg} = R\ /\ (0.4F_y)$
   $A_{vg} = 88/(0.4 \times 36) = 6.11\ \text{in.}^2$

3. Try a WT9 × 25 tee section to satisfy the following requirements:
   a. $b_f/2t_f$ of the tee ≥ 6.5.
      $7.495/(2 \times 0.57) = 6.57 > 6.5$ **O.K.**
   b. $d_b/t_s \geq 2.0$
      $(7/8)/0.355 = 2.46 > 2.0$ **O.K.**
   c. l_h and $l_v \geq 1.50d_b$.
      $l_h = l_v = 1.50d_b = 1.50 \times (7/8) = 1.32$ in.

   Use: $l_h = l_v = 1.5$ in.

   d. Bolt spacings satisfy the AISC Specification.[12]
   e. $L_t t_s \geq A_{vg}$
      $L_t = 6 \times 3 + 2 \times 1.50 = 21.0$ in.
      $L_t t_s = 21 \times 0.355 = 7.46 > 6.1\ \text{in.}^2$ **O.K.**
   f. $L_t/b_f \leq 3.5$.
      $21/7.495 = 2.8 < 3.5$ **O.K.**
   g. $t_{fc} > t_f$
      0.87 in. > 0.57 in. **O.K.**
   h. $(t_s/d_b)/(t_f/t_s) = (0.355/0.785)/(0.57/0.35) = 0.25$ **O.K.**

4. Calculate actual shear yield capacity of the gross area of stem:

   $R_o = (L_t t_s)\ (0.4F_y)$
   $R_o = (21 \times 0.355)\ (0.4 \times 36) = 107$ kips.

5. Check capacity of effective net area of stem:
   $R_{nse} = [L_t - (n/2)\ (d_b + 1/16)](t_s)\ (0.3F_u) \geq R_o$
   $R_{nse} = [21 - (7/2)\ (7/8 + 1/16)]\ (0.355)$
   $= 109.4 > 107$ kips. **O.K.**
   or by using net area in shear as defined by the AISC:[12]
   $R_{ns} = [L_t - (n)\ (d_b + 1/16)]t_s\ 0.3F_u \geq R$
   $R_{ns} = [21 - 7(7/8 + 1/16)]\ (0.355)\ (0.3 \times 58)$
   $= 89 > 88$
   kips. **O.K.**

6. Check shear in the flange:
   $2L_t t_f(0.4F_y) \geq R$
   $2 \times 21 \times 0.57 \times 0.4 \times 36 = 344.7 > 88$ **O.K.**

7. Design fillet welds for the combined effects of shear and moment using Table XIX[11]. Shear is equal to R_o and the moment is equal to $R_o a$ where a is the distance from bolt line to weld line:
   $R_o = 107$ kips
   $M = 107 \times 3 = 321$ kip-in.
   Using Table XIX AISC Manual[11] with $k = 0$;
   $a = 3/21 = 0.142$
   $C_1 = 1.0$
   $C = 1.53$
   $D_{16} = R_o/CC_IL_p = 107/(1.0 \times 1.53 \times 21) = 3.33$

   Use: ¼ in. E70 Fillet Welds.

8. Check bearing capacity:
   For tee stem:
   $r_v = d_b t_s(1.2F_u) = .875 \times .355 \times 1.2 \times 58 = 21.62$
   $R_{brg} = 7(21.62) = 151.3$ kips > 88 kips. **O.K.**
   Since the beam web is thicker than the tee stem, bearing failure of web will not govern.

9. Beam is not coped, therefore, there is no need for consideration of block shear failure.

Design Example 2

Given:

Beam: W16 × 31, $t_w = 0.275$
Beam Material: A572 Gr. 50 steel
Support: Web of a W8 × 58 column
Reaction: 30 kips (Service Load)
Bolts: ¾ in. dia. A325-N (Tight Bolts)
Bolt Spacing: 3 in.
Welds: E70XX fillet welds

Design a tee framing connection to transfer the beam reaction to the supporting column.

Solution:

1. Calculate number of bolts:
   Shear = 33 kips
   Since support is relatively rigid, the moment acting on the bolts can be neglected. Therefore;
   $n = R/r_v = 33/9.2 = 3.60$

   Use: Four ¾ in. dia. A325-N bolts.

2. Calculate required gross area of tee stem:
   $A_{vg} = R/(0.40F_y)$
   $A_{vg} = 33/(0.4 \times 36) = 2.29\ \text{in.}^2$

3. Try a WT6 × 8 tee section to satisfy the following requirements:
   a. $b_f/2t_f$ of the tee ≥ 6.5.
      $3.99/(2 \times 0.265) = 7.52 > 6.5$ **O.K.**
   b. $d_b/t_s \geq 2.0$

$(\frac{3}{4})/0.22 = 3.41 > 2.0$ **O.K.**

c. l_h and $l_v \geq 1.50 d_b$.
$l_h = l_v = 1.50 d_b = 1.50 \times (\frac{3}{4}) = 1.125$ in.

Use: $l_h = l_v =$ 1.5 in.

d. Bolt spacings satisfy the AISC Specification.[12]

e. $L_t t_s \geq A_{vg}$
$L_t = 3 \times 3 + 2 \times 1.50 = 12.0$ in.
$L_t t_s = 12 \times 0.22 = 2.64 > 2.29$ in.^{2} **O.K.**

f. $L_t / b_f \leq 3.5$.
$12/3.99 = 3.0 \leq 3.5$ **O.K.**

d. $t_{fc} > t_f$
This condition is not applicable to connections to column webs.

h. $(t_s / d_b)/(t_f / t_s) = 0.24 \approx 0.25$ **O.K.**

4. Calculate actual shear yield capacity of gross area of stem:
$R_o = (L_t t_s)\ (0.4 F_y)$
$R_o = (12 \times 0.22)\ (0.4 \times 36) = 38$ kips.

5. Check capacity of effective net area of stem:
$[L_t - (n/2)(d_b + \frac{1}{16})](t_s)(0.3 F_u) \geq R_o$
$[12 - (4/2)(\frac{3}{4} + \frac{1}{16})](0.22)(0.3 \times 58)$
$39.7 > 38$ kips **O.K.**
or using net area as defined by the AISC:[12]
$[L_t - (n)(d_b + \frac{1}{16})] t_s \geq R$
$[12 - 4(\frac{3}{4} + \frac{1}{16})](0.22)(0.3 \times 58)$
$33.5 > 33$ kips **O.K.**

6. Check shear in the flange:
$2 L_t t_f (0.4 F_y) \geq R$
$2 \times 12 \times 0.265 \times 0.4 \times 36 = 91.2 > 33$ **O.K.**

7. Design the fillet welds for the combined effects of shear and moment using Table XIX.[11] Shear is equal to R_o and moment is equal to $R_o a$ where a is the distance from bolt line to weld line.
$R = 33$ kips
$M = 38 \times 2.5 = 95$ kip-in.
Using Table XIX AISC Manual[11]
$a = 2.5/12 = 0.208$
$C_1 = 1.0$
$C = 1.19$
$D_{16} = R_o / C C_1 L = 38/(1.0 \times 1.19 \times 12) = 2.66$

Use: $\frac{3}{16}$ in. E70 Fillet Welds.

8. Check bearing capacity:

For tee stem:
$r_v = d_b t_s (1.2 F_u) = .75 \times .22 \times 1.2 \times 58 = 11.48$
$R_{brg} = 4(11.48) = 45.92$ kips > 33 kips. **O.K.**
Since beam web is thicker than the tee stem, bearing failure of the web will not govern.

9. Beam is not coped, therefore, there is no need for consideration of block shear failure.

Design Example 3

Given:

Beam:	W14 × 22, $t_w = 0.23$ in.
Beam Material:	A36 steel
Support:	A girder (only one beam from one side is attached to the girder)
Reaction:	11 kips (Service Load)
Bolts:	⅝ in. dia. A325-N (snug-tight bolts)
Bolt Spacing:	3 in.
Welds:	E70XX fillet welds

Design a tee framing connection to transfer the beam reaction to supporting girder.

Solution:

1. Calculate number of bolts:
Since support is torsionally flexible, moment acting on the bolts should be considered;
$R = 11$ kips
$M = Re = 11 \times 2.5 = 27.5$ k-in.
The number of bolts needed for direct shear:
$n = R / r_v = 11/6.4 = 1.71$
Referring to first line in Table X of the AISC Manual,[11] three bolts will be needed to carry shear combined with a moment due to 3 in. eccentricity.

Use: Three ⅝ in. dia. A325-N bolts.

2. Calculate required gross area of tee stem:
$A_{vg} = R\ /\ (0.40 F_y)$
$A_{vg} = 11/(0.4 \times 36) = 0.76$ in.^{2}

3. Try a WT4 × 5 tee section to satisfy the following requirements.

a. $b_f / 2 t_f$ of the tee ≥ 6.5
$3.94/(2 \times 0.205) = 9.6 > 6.5$ **O.K.**

b. $d_b / t_s \geq 2.0$
$(\frac{5}{8})/0.17 = 3.67 > 2.0$ **O.K.**

c. l_h and $l_v \geq 1.50 d_b$.
$l_h = l_v = 1.50 d_b = 1.50 \times (\frac{5}{8}) = 0.94$ in.

Use: $l_h = l_v =$ 1.0 in.

d. Bolt spacings satisfy the AISC Specification.[12]

e. $L_t t_s \geq A_{vgw}$
$L_t = 2 \times 3 + 2 \times 1.0 = 8$ in.
$L_t t_s = 8 \times 0.17 = 1.36 > 0.76$ in.^{2} **O.K.**

f. $L_t / b_f \leq 3.5$
$8/3.94 = 2.03 \leq 3.5$ **O.K.**

g. $t_{fc} > t_f$
This condition is not applicable to connections to girders.

h. $(t_s / d_b)/(t_f / t_s) = 0.23 \approx 0.25$ **O.K.**

4. Calculate actual shear yield capacity of gross area of stem:
$R_o = (L_t t_s)\ (0.4 F_y)$.
$R_o = (8 \times 0.17)(0.4 \times 36) = 19.6$ kips.

5. Check capacity of effective net area of stem:

   $[L_t - (n/2)(d_b + 1/16)](t_s)(0.3F_u) \geq R_o$

   $[8 - (3/2)(5/8 + 1/16)](0.17)(0.3 \times 58)$

   $20.6 > 19.6$ kips. **O.K.**

   or using net area as defined by the AISC:[12]

   $[L_t - (b)(d_b + 1/16)]t_s \geq R$

   $[8 - 3(5/8 + 1/16)](0.17)(0.3 \times 58)$

   $17.5 > 11$ kips. **O.K.**

6. Check shear in the flange:

   $2L_t t_f(0.4F_y) \geq R$

   $2 \times 8 \times 0.205 \times 0.4 \times 36 = 47.2 > 11$ **O.K.**

7. Design fillet welds for direct shear.

   $V = 11$ kips

   $D_{16} = R_o/[L_t(70 \times .3)(1/1.41)]$

   $= 19.6/[8(21)(1/1.41)] = 0.16$

   Use: 3/16 in. E70 Fillet Welds.

8. Check bearing capacity:

   For tee stem:

   $r_v = d_b t_s(1.2F_u) = 5/8 \times .17 \times 1.2 \times 58 = 7.4$

   $R_{brg} = 1.75(7.4) = 22.2$ kips > 11 kips **O.K.**

   Since the beam web is thicker than the tee stem, the web will not fail.

9. Beam is not coped, therefore, there is no need for consideration of block shear failure.

CONCLUSIONS

Based on test results and their analysis, the following conclusions were reached.

1. Realistic testing of tee connections indicates that considerable shear yielding occurs in the stem and flange of the tee prior to failure.
2. Tee connections that were studied were flexible enough to be considered type II (simple) connections.
3. The limit states that were observed during the testing were: bolt shear fracture, tee stem yielding, tee flange yielding tee stem fracture, bolt hole bearing failure and weld fracture.
4. A limit state that was not observed during the tests but can occur if the flange of the tee is too thin, is excessive shear yielding of tee flange.
5. The horizontal and vertical edge distances of bolt holes are recommended to be at least 1.5 times diameter of the bolt.
6. Tee connections studied were very ductile and accommodate 0.07 radian rotation.
7. The out of plane bending of the tee-flange is one of the major factors that contribute to high ductility of the tee-connection. The out of plane bending increases with increasing $b_f/2t_f$ ratio of tee flange. It is recommended that $b_f/2t_f$ ratio of tee flange be greater or equal to 6.5.
8. To ensure flexibility and ductility, the use of A36 steel is recommended.
9. Point of inflection of beam moves toward the connection and for connections with snug tight bolts it was approximately located at the bolt line.

ACKNOWLEDGMENTS

The research project was supported by the American Institute of Steel Construction, Inc. and the Department of Civil Engineering, the University of California at Berkeley. The support and constructive comments provided by N. Iwankiw, Director of Research and Codes of AISC, K. B. Wiesner of LeMessurier Consultants and Ted Winneberger of W & W Steel Company are sincerely appreciated. Material for the test specimens was supplied by the members of the Steel Committee of California. The efforts of R. Hofer as well as other committee members in this respect are appreciated. The opinions expressed in this paper are those of the authors.

REFERENCES

1. Astaneh, A. "Experimental Investigation of Tee-Framing Connection," *Progress Report* submitted to American Institute of Steel Construction, April 1987.
2. Astaneh, A. "Demand and Supply of Ductility in Steel Shear Connections", In *Press* by *Journal of Constructional Steel Research*, 1989.
3. Astaneh, A., K. M. McMullin and S. M. Call, "Design of Single Plate Framing Connections," *Report No. UCB/SEMM-88/12*, Department of Civil Engineering, University of California, Berkeley, July, 1988.
4. Astaneh, A., K. M. McMullin and S. M. Call, "Design of Single Plate Shear Connections," *Engineering Journal* First Quarter, 1989 American Institute of Steel Construction 1989.
5. Astaneh, A. and M. Nader, "Behavior and Design of Steel Tee Framing Connections," *Report No. UCB/SEMM-88/11*, Department of Civil Engineering, University of California, Berkeley, July, 1988.
6. Chen, F.-W. "Connection Flexibility and Steel Frames," Proceedings of a Session Sponsored by the Structural Division of the American Society of Civil Engineers in conjunction with the ASCE Convention in Michigan, October, 1985.
7. Iwankiw, N. R. "Design for Eccentric and Inclined Loads on Bolts and Weld Groups," *Engineering Journal*, AISC, 4th Quarter, 1987.

8. McMullin, K.M. and A. Astaneh, "Analytical and Experimental Investigations of Double-Angle Connections", *Report No. UCB/SEMM-88/14*, Department of Civil Engineering, University of California, Berkeley, August, 1988.
9. Veillette, J. R. and J. T. DeWolf, "Eccentrically Loaded High Strength Bolted Connections," *Journal of Structural Engineering*, Vol. 111, No. 5, May, 1985, American Society of Civil Engineers.
10. White, R. N. "Framing Connections for Square and Rectangular Structural Tubing," *Engineering Journal* July, 1965, American Institute of Steel Construction.
11. AISC. *Manual of Steel Construction,* 8th. Edition, American Institute of Steel Construction, Chicago, 1980.
12. AISC. *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* American Institute of Steel Construction, Inc. Chicago, 1978.

NOMENCLATURE

a	Coefficient in the AISC Manual,[11] Table XIX
a	Distance between bolt line and weld line, in.
A_b	Nominal cross sectional area of one bolt, in.^{2}
A_{ns}	Net area in shear, in.^{2}
A_{nse}	Effective net area in shear, in.^{2}
A_{vg}	Gross area of tee stem in shear, in.^{2}
A_{vgf}	Gross area of tee flange in shear, in.^{2}
b_f	Width of tee flange, in.
C	Coefficient in the AISC Manual,[11] Tables X and XIX
C_1	Coefficient in the AISC Manual,[11] Table XIX
d	Depth of beam, in.
d_b	Diameter of bolt, in.
D_{16}	Number of sixteenth of an inch in fillet weld size
e	Eccentricity of reaction, in.
e_b	Eccentricity of beam reaction from bolt line, in.
e_w	Eccentricity of beam reaction from weld line, in.
f_{vu}	Shear stress applied to net area, ksi
f_{vy}	Shear stress applied to gross area, ksi
F_u	Specified minimum tensile strength of steel, ksi
F_{vu}	Allowable ultimate shear strength = $0.30F_u$, ksi
F_{vy}	Allowable shear stress for plate in yielding = $0.40F_y$, ksi
F_y	Specified yield stress of steel, ksi
k	Coefficient in the AISC Manual,[11] Table XIX
l_h	Horizontal edge distance of bolts, in.
l_v	Vertical edge distance of bolts, in.
L	Length of span, in.
L_t	Length of tee, in.
M	Applied moment, k-in.
M_p	Plastic moment capacity of cross section, k-in.
n	Number of bolts
r_v	Allowable shear strength of one bolt, kips
R	Service load reaction of the beam, kips
R_{brg}	Allowable strength of connection in bearing, kips
R_{ns}	Allowable shear strength of net area, kips
R_{nse}	Allowable shear strength of effective net area, kips
R_o	Allowable shear yield strength of plate, kips
t_f	Thickness of tee flange, in.
t_{fc}	Thickness of column flange, in.
t_s	Thickness of tee stem, in.
t_w	Thickness of beam web, in.
θ	End rotations of beam, rad.