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Notes on Design of Double-Angle and Tee Shear Connections for Gravity and Seismic Loads

By

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By Abolhassan Astaneh-Asl

This document presents information on behavior and design of steel double-angle and tee shear connections. In these connections, a tee section or two angles are welded or bolted to the web of a beam and to the supporting member. The main role of a tee or of double angles in these connections is to transfer shear force from the end of the beam to its support. Chapter 1 provides a brief summary of the behavior of shear connections under gravity and seismic effects. Chapters 2 and 3 provide updated summaries of behavior and design procedures for double-angle and tee connections, respectively. The report includes Notations and a list of References.

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The opinions expressed in this report are solely those of the author and do not necessarily reflect the views of the University of California, Berkeley, where he is a professor of civil and environmental engineering, the Structural Steel Educational Council, or other agencies and individuals whose names appear in this report.

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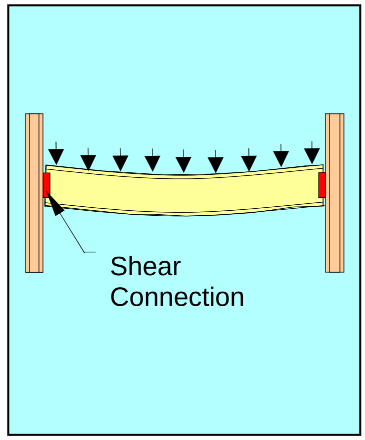
Notations

In preparing the following notations, whenever possible, the definitions are taken from the AISC *Manual of Steel Construction* (AISC-ASD 1989 and AISC-LRFD 2000) and the Seismic Provisions for Structural Steel Buildings (AISC 2002).

A_b = area of bolt
A_g = gross area; gross area of one angle in case of double-angle connections
A_n = net area of a plate in tension = [L – n (d_b + 1/8 inch)] (t)
A_{nv} = net area of a plate in shear as defined by the author = [L – 0.5n (d_b + 1/8 inch)](t)
a = distance from the bolt line to the weld line; depth of the compression zone; distance of the point of inflection of the beam from the web of the column
b = angle leg size
C_d = deflection magnification factor
D = actual size of weld
d = beam depth
d_b = diameter of bolt
$d_{b\,min}$ = minimum diameter of the bolt in tee connections for ductility check
E = modulus of elasticity
e_b = distance of the point of inflection from the bolt line
e_w = distance of the point of inflection from the weld line
F_{Exx} = specified minimum strength of weld electrode
F_t = specified minimum tensile strength of bolt
F_u = specified minimum tensile strength of steel
F_{vb} = specified shear strength of bolt
F_w = specified minimum strength of the weld electrode
F_u = specified ultimate strength of steel
F_y = specified minimum yield stress of steel
h_x = height of the x^{th} story in a building
I= moment of inertia
K_{conn} = rotational stiffness of connection
L= span of a beam; length of double-angle or tee shear connection
L_c = clear distance, in the direction of the force, between the edge of a hole and the edge of the member
L_{c1} = clear distance, in the direction of the force, between the edge of a hole and the edge of the adjacent hole or edge of the material
M = bending moment , nominal (unfactored) bending moment
M_b = plastic moment capacity of bolt group
M_{conn} = moment capacity of the connection
M_{pb} = plastic moment capacity of the beam section
M_y = yield moment of the beam section
M_u = factored moment applied to connection or bolt group
m = normalized stiffness term for rotational stiffness of beam-to-column connections
N =nominal (unfactored) applied axial force

N_n = nominal axial strength
N_s = axial force due to seismic effects
N_u = factored applied axial force
n = total number of bolts
q = uniformly distributed load per unit of length
R = required strength
R_u = required strength in LRFD
S_x = elastic section modulus of cross section
t = thickness of the connected material
t_f = thickness of flange
t_s = thickness of tee stem
$t_{s\,max}$ = maximum thickness of tee stem for ductility check
T= tension force on the bolt due to external moment
V = shear force, nominal (unfactored) shear force
V_b = shear strength of bolt group
V_{br} = nominal bearing strength
V_g = shear force in the connection due to gravity only
V_{gs} = shear force in the connection due to combined gravity and seismic effects
V_n = nominal shear strength
V_s = shear force in the connection due to seismic effects only
V_u = factored shear force acting on the shear connection
V_w = shear force causing failure of the weld= 2 (0.707 D_w) L (0.6 F_w)
V_y = shear force causing yielding of a connection; shear yield capacity of a beam
δ = horizontal displacement of a floor
δ_x = horizontal displacement of story x
δ_{xe} = horizontal displacement of story x resulting from elastic analysis
ϕ = resistance factor for the bolt in the LRFD method = 0.75
ϕ_r = resistance factor for the bolts in the LRFD method = 0.75
ϕ_{br} = resistance factor for the bearing in the LRFD method = 0.75
ϕ_n = resistance factor for fracture in the LRFD method = 0.75
ϕ_w = resistance factor for weld = 0.75
ϕ_y = resistance factor for yielding in the LRFD method = 0.90
Ω = safety factor for the bolt = 2.00 (ASD)
Ω_b = safety factor for bolts = 2.00 (ASD)
Ω_{br} = safety factor for bearing = 2.00 (ASD)
Ω_n = safety factor for fracture in the ASD method = 2.00
Ω_y = safety factor for shear yielding in the ASD method = 1.50
Ω_w = safety factor for welds = 2.00 (ASD)
θ = angle of rotation
θ_g =end rotation of a beam due to gravity load
θ_{gs} = end rotation due to combined gravity and seismic effects
θ_{hs} =end rotation of a beam due to horizontal seismic load
θ_p =end rotation of a beam when its midspan moment reaches the plastic moment
θ_s =end rotation of a beam due to seismic load
θ_{vs} =end rotation of a beam due to vertical seismic load

1. BEHAVIOR OF SHEAR CONNECTIONS UNDER GRAVITY AND SEISMIC LOADS



1.1. Introduction to Simple Shear Connections

Shear connections are used in steel structures to connect a simply supported beam to its support. These connections are primarily designed to transfer gravity shear force and to be sufficiently flexible to accommodate end rotation of the beam. During an earthquake, shear connections are subjected to additional forces and deformations. The main goals of this report are twofold:

1. To provide an updated summary of the information on the behavior and design of shear tab connections subjected to *gravity* shear; and,

2. To provide design-oriented information on behavior and design of shear tab connection under combined gravity and seismic effects.

1.2. Definition of Shear Connections

The AISC specifications (AISC 1999) states that for shear connections the following requirements apply:

> *Excerpts from the AISC Manual of Steel Construction, 2000 (Page 16.1–2):*
>
> *(1) The connections and connected members shall be adequate to resist the factored gravity loads as "simple beams."*
> *(2) The connections and connected members shall be adequate to resist the factored lateral loads.*
> *(3) The connections shall have sufficient inelastic rotation capacity to avoid overload of fasteners or welds under combined factored gravity and lateral loading.*

Figure 1.1 shows three types of beam-to-column connections. These are fully restrained (FR), partially restrained (PR), and simple or "shear" connections. As shown in Figure 1.1, a relatively small bending moment, less than 20% of the plastic moment capacity of the beam, can develop in a shear connection. This relatively small negative moment acting at the ends of the beam is usually ignored in the design of the beam itself, and the beam is designed as a simply supported beam. Doing so satisfies the AISC specification requirement (1) in the above box. However, the relatively small moment at the end of the beam can have detrimental effects on connection elements such as the plate, welds, and bolts and is, therefore, considered in design of the connection itself and the supporting member (that is, the column or girder).

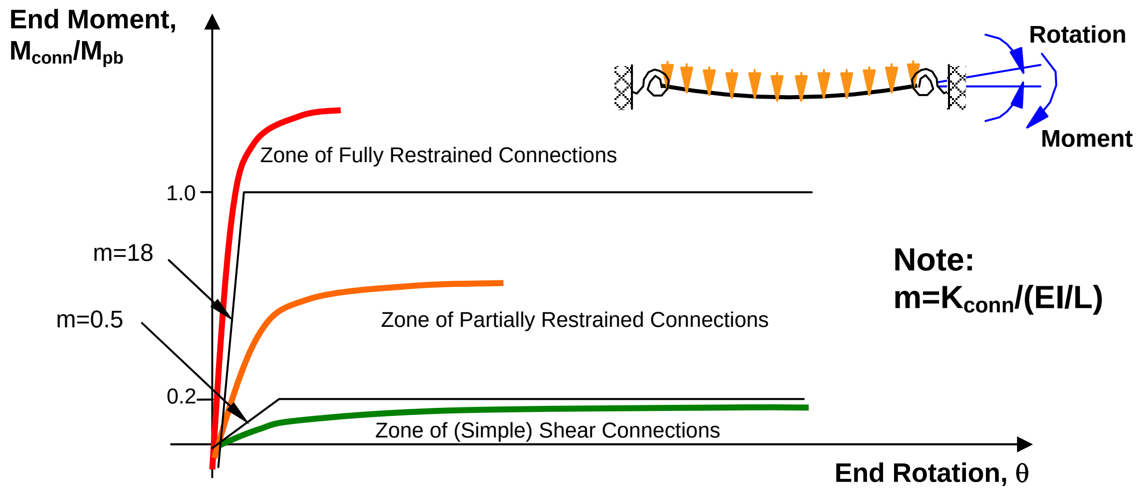


Figure 1.1. Definition of Shear, Partially Restrained, and Rigid Connections

1.3. Types of Shear Connections

Figure 1.2 shows typical shear connections used to connect a simply supported beam to a column or a girder. In connections a through e in Figure 1.2, the shear force in the web of the girder is directly transferred to the support, while in connections f and g, the shear force in the web is supported on a bottom-flange seat angle. Behavior and design of shear tab connections, shown in Figure 1.2(a) and (b) were discussed in a separate Steel Technical Information and Product Services (Steel TIPS) report (Astaneh-Asl 2005). The focus of this Steel TIPS report is on the double-angle and tee shear connections shown in Figure 1.2(c) and (d) below.

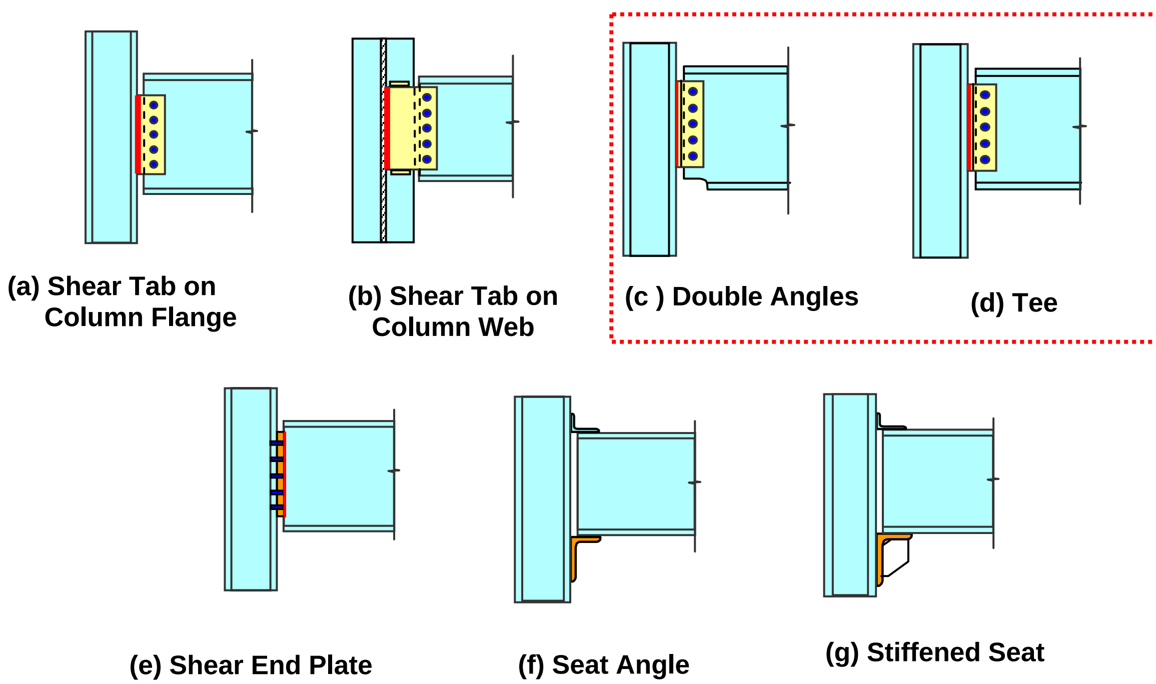


Figure 1.2. Common Types of Shear Connections

1.4. Design of Shear Connections for Gravity Effects

Design of shear connections should be done such that the following requirements are satisfied:

1. The connection should have sufficient shear strength to resist applied forces. Under gravity load alone, the main force acting on a shear connection is shear force. Under the combined effects of gravity load and earthquakes, shear connections are expected to transfer seismic axial load in addition to shear.

2. Shear connections should be sufficiently flexible in rotation such that the fixed end moments in the beam are small and less than 20% of the plastic moment capacity of the connected beam. If the end moments are larger than 20% of the plastic moment capacity of the beam, the connection would act as a semirigid connection.

3. Shear connections should have sufficient rotational ductility to tolerate rotations due to gravity load as well as due to combined effects of gravity and seismic loads.

In design of shear connections for gravity, we need to know the gravity shear force and bending moment that are acting on the connection as well as the maximum rotation demand that will be imposed on the connection due to gravity loads. The applied shear can easily be established as being the reaction of the beam. To establish the applied moment, we need to know the location of the point of inflection of the beam where the bending moment is zero. Then, the bending moment at any section across the connection conservatively can be established as the shear force times the distance of the cross section from the point of inflection.

Due to the complexity of the behavior of the shear connections and the nonlinearity of the behavior under relatively small forces, in order to establish the location of the point of inflection, the most reliable approach is to conduct actual tests of full-size specimens subjected to realistic loading conditions. By conducting such tests, not only can the location of the point of inflection be established, but failure modes, rotational stiffness, and ductility of the connections can also be established.

1.5. General Behavior of Shear Connections of Simply Supported Beams

The general behavior of shear connections is discussed more extensively in another Steel TIPS report by the author (Astaneh-Asl 2005) and will not be repeated here. Figure 1.3 shows a simply supported beam subjected to uniformly distributed loads. In general, because of a small amount of bending stiffness of the shear connections, a relatively small bending moment develops in the shear connections resulting in the point of inflection of the beam being located at a relatively small distance from the end connection; see Figure 1.3.

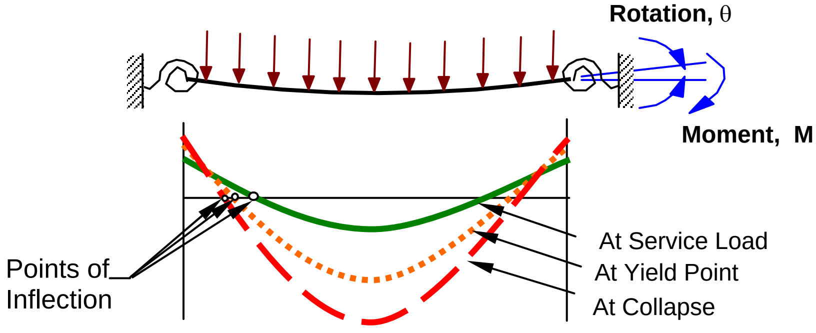


Figure 1.3. Bending Moment Diagrams for a Simply Supported Beam

As the load on the beam increases, shear force in the connection also increases. The increase in shear stresses, combined with bending stresses, facilitates yielding in the shear connection resulting in rotation of the connection and further movement of the point of inflection towards the support. Slippage of bolts in bolted shear connections has a similar effect in reducing the rotational stiffness, which in turn results in facilitating rotation of the connection. As a result, in simply supported beams, shear connections follow more or less the end rotation of the beam. As load continues to increase, eventually a plastic hinge forms at the midspan, and the beam collapses. At the time of the beam collapse, the rotation of the beam ends and the shear connections rapidly increases. This is the reason why shear connections need to have sufficient ductility to rotate and follow the large end rotations of the beam when the beam reaches its maximum load capacity.

1.5.a. Actual Tests of Shear Connections

As mentioned earlier, due to complexity of behavior of shear connections and nonlinearity of behavior, even under service loads, design procedures for shear connections should be based on data obtained from full-scale tests of specimens under realistic loading conditions. When studying behavior of shear connections under gravity loads, the following parameters are important:

1. Shear strength
2. Bending strength
3. Rotational ductility

To study the behavior of double-angle and tee shear connections under realistic conditions of shear and rotation, a special test setup, shown in Figure 1.4, developed by the author (Astaneh-Asl 1988) was used. During the tests, the shear force and corresponding rotation applied to the connections were as shown by the curve A in Figure 1.5. The curve represents a realistic shear-rotation variation that a shear connection would see if it were supporting a simply supported beam.

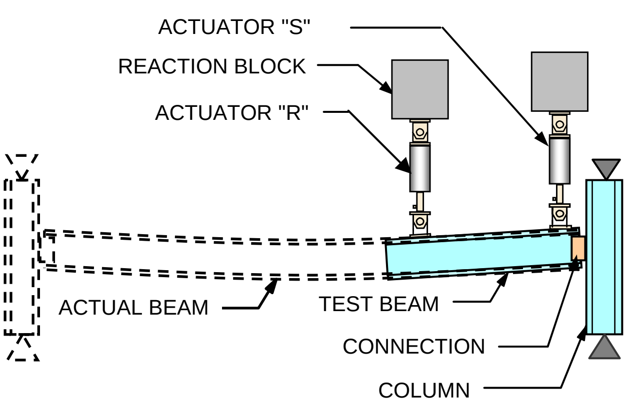


Figure 1.4. Test Setup for Realistic Testing of Shear Connections

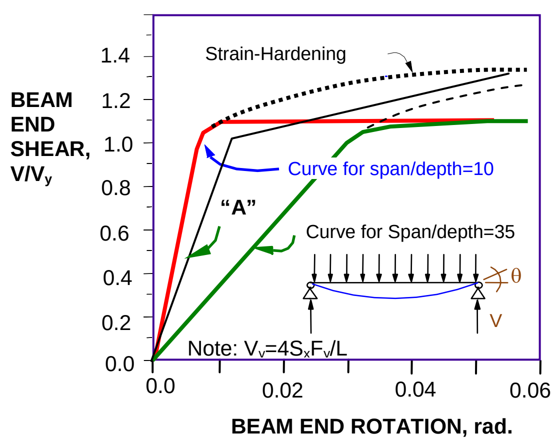


Figure 1.5. Shear-Rotation Relationship for Shear Connections

For more information on realistic testing of shear connections, the reader is referred to Astaneh-Asl (1988). Using actuator S, located near the support, and actuator R, at the tip of the other end, any desired combination of shear and rotation of the beam end could be applied to the connection. Manual control of the actuators R and S allowed testing of shear connections to follow the desired shear-rotation loading protocol of Figure 1.5.

1.5.b. What Is the Gravity Shear Force and Bending Moment in a Shear Connection?

The design shear force in a shear connection of a simply supported beam is the reaction of the beam, which in general is equal to one-half of the load on the span. The moment in the shear connection depends on the rotational stiffness of the connection relative to the rotational stiffness of the beam. Due to highly nonlinear behavior of the connection, the moment needs to be established by actual tests. Using the test setup shown in Figure 1.4, realistic tests of shear connections were conducted at the University of California at Berkeley (Astaneh-Asl, Nader, and Malik 1989; Astaneh-Asl and Nader 1989; and Astaneh-Asl and McMullin 1993). The connections that were tested were shear tabs, double angles, and tee shear connections. The most important outcome of the tests was to establish the location of the point of inflection of simply supported beams using these three types of shear connections. By knowing the location of the point of inflection, one can calculate the end moments of the beam. In addition to establishing the location of the point of inflection, the tests also resulted in establishing the failure modes and their hierarchy. The results of tests of these connections under gravity effects (shear and rotation) are discussed in the next chapter of this report for shear tabs and in Astaneh-Asl (2005) for double angles and tee connections, respectively.

1.5.c. What Should Be the Rotational Ductility of a Shear Connection?

In Astaneh-Asl (2005) this question is discussed and equations are derived that can be used to establish rotational ductility demand on shear connections. Following the equation from that reference provides a good approximation of the rotational ductility demand:

$$\theta_p = \frac{M_{pb} L}{2.5 EI} \qquad (1.1)$$

Assuming a conservative ratio of M_{pb}/M_y of 1.25 for wide flanges, the above equation can be written as:

$$\theta_g = \frac{F_y L}{Ed} \qquad (1.2)$$

Equation 1.2 is proposed to be used in design as a reasonable estimate of the maximum rotational ductility demand imposed on shear connections of simply supported beams under gravity load. Considering a value of F_y=50 ksi, E=29,000 ksi, and L/d of 17, a conservative estimate of θ_g can be made as:

$$\theta_g \cong 0.03 \text{ Radians} \qquad (1.3)$$

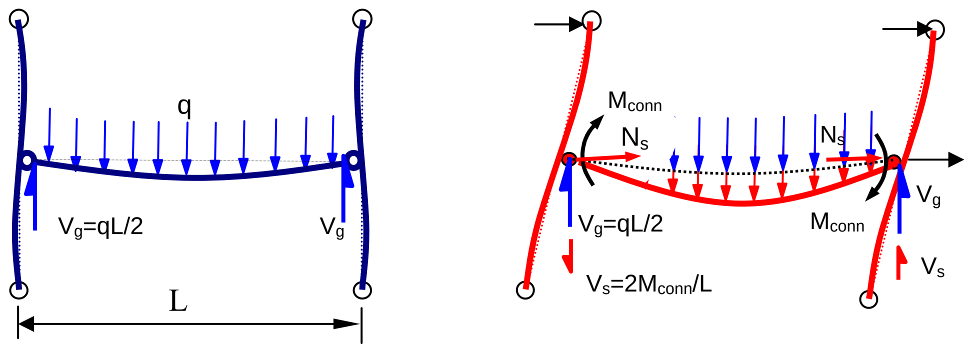
1.6. Seismic Behavior of Shear Connections of Simply Supported Beams

During an earthquake, shear connections in a building are subjected to additional bending moment, shear and axial forces and corresponding rotations, and shear and axial deformations. All of these seismic effects are cyclic in nature.

1.6.a. Seismic Forces in a Shear Connection

During the earthquake, shear connections can be subjected to additional, and sometimes quite significant, axial load and rotations. The axial load is the result of inertia forces in the floor collected in the beam and transmitted to the columns by shear connections. The additional shear force is due to development of two bending moments, with the same sign, one at each end of the beam as shown in Figure 1.6. Therefore, the total shear force in the connection under combined gravity and seismic effects is:

$$V_{gs} = V_g + V_s = \frac{qL}{2} + \frac{2M_{conn}}{L} \qquad (1.4)$$

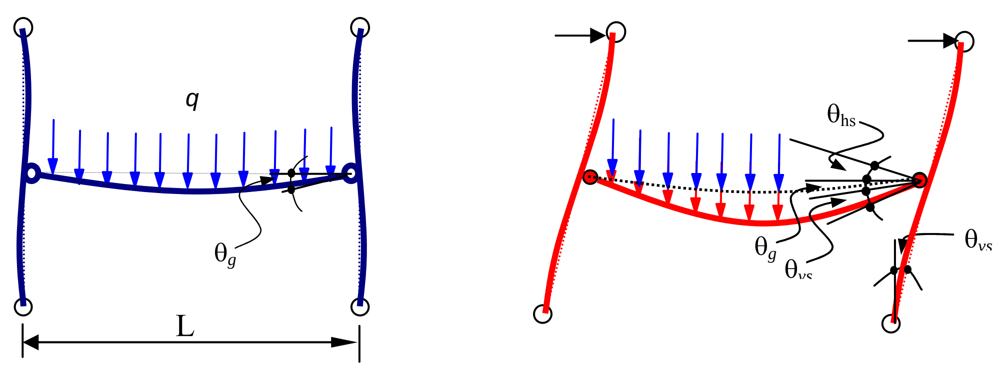


(a) Under Gravity Load Only (b) Under Combined Gravity and Seismic Loads

Figure 1.6. Shear and Axial Forces in the Shear Connections

1.6.b. Seismic Rotations in Shear Connections

Figure 1.7 shows rotations that develop in a shear connection under gravity load alone and under combined gravity and seismic loads. In Figure 1.7, Angle θ_g shows rotation of the end of the beam relative to the column due to gravity load alone. If we conservatively ignore the bending moment in the shear connection and assume it is a pin connection, then Angle θ_g is the end rotation of a simply supported beam. In reality, due to bending capacity of shear connections, however small, the actual angle of rotation is less than that for a simply supported beam. Angle θ_{hs} in Figure 1.7(b) is the rotation of the beam end due to seismic drift of the floor. Angle θ_{vs}, also a seismic rotation, is due to deflection of the beam in the vertical direction due to vertical inertia forces of earthquake.



(a) Under Gravity Load Only (b) Under Combined Gravity and Seismic Loads

Figure 1.7. Rotations in the Shear Connections

Maximum values of rotation demand on a shear connection, during earthquakes, can be established as:

$$\theta_{gs} = \theta_g + \theta_s \qquad (1.5)$$

where,

$$\theta_s = \theta_{hs} + \theta_{vs} \quad (1.6)$$

Angle θ_{hs} in the above equation is equal to the inelastic story drift angle. Using the ASCE-7 standard (ASCE 2002), story drift for story *x* is given as:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \quad (1.7)$$

Angle θ_{hs} is equal to δ_x divided by the story height, h_x, resulting in:

$$\theta_{hs} = \frac{C_d \delta_{xe}}{h_x I} \quad (1.8)$$

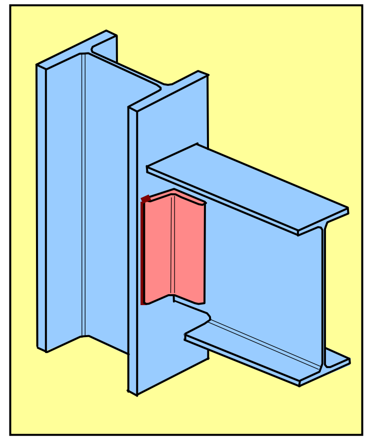
Therefore, an approximate value of the maximum rotation demand on shear connections during earthquakes can be estimated from following equation:

$$\theta_{gs} = \theta_g + \theta_s \quad (1.9)$$

Where θ_g is the maximum value of the rotation demand on the shear connection under gravity load alone and was established earlier by Equation 1.5. Angle θ_s is the maximum rotation due to seismic effects, given approximately as:

$$\theta_s = 0.04 + 0.30(0.04) = 0.052 \quad \text{Radians} \quad (1.10)$$

2. DOUBLE-ANGLE SHEAR CONNECTIONS



2.1. Introduction

Double-angle shear connections have been used frequently in the past in bridges and buildings to transfer shear from the web of the beam to their support. The angles can be bolted or welded to the beam and the supporting member. Figure 2.1 shows four types of double-angle connections. From a construction point of view, connections in which the angle is bolted to the beam web, Figure 2.1(a) and (c), provide more tolerances in fabrication due to the presence of a gap between the bolt hole and the bolt. When the angles are welded to the web of the beam and bolted to the column, Figure 2.1(b), the back to back dimension of the beam with angles connected to it should be slightly (about ¼ inch) smaller than the actual face to face dimension of the column flanges in the span to enable the erectors to place the beam easily in its location. The gap between the back of the angles and the face of the column flange can be filled with a shim plate on one end or both ends. For the welded-welded connection shown in Figure 2.1(d), the back to back dimension of the beam should be very close to the distance between the face of the flanges of the columns, since in this case the shims cannot be used between the angle legs and column faces. If double angles are connected to the column in the shop, then in order to place the beam web between the two outstanding legs of the angles, the bottom flange of the beam is usually coped, and the beam is brought down and its web placed between the legs of the angles. The AISC *Manual of Steel Construction* (AISC-LRFD 2000) has a section on "Constructability Considerations" on double-angle shear connections that needs to be followed in design as well as construction of these connections.

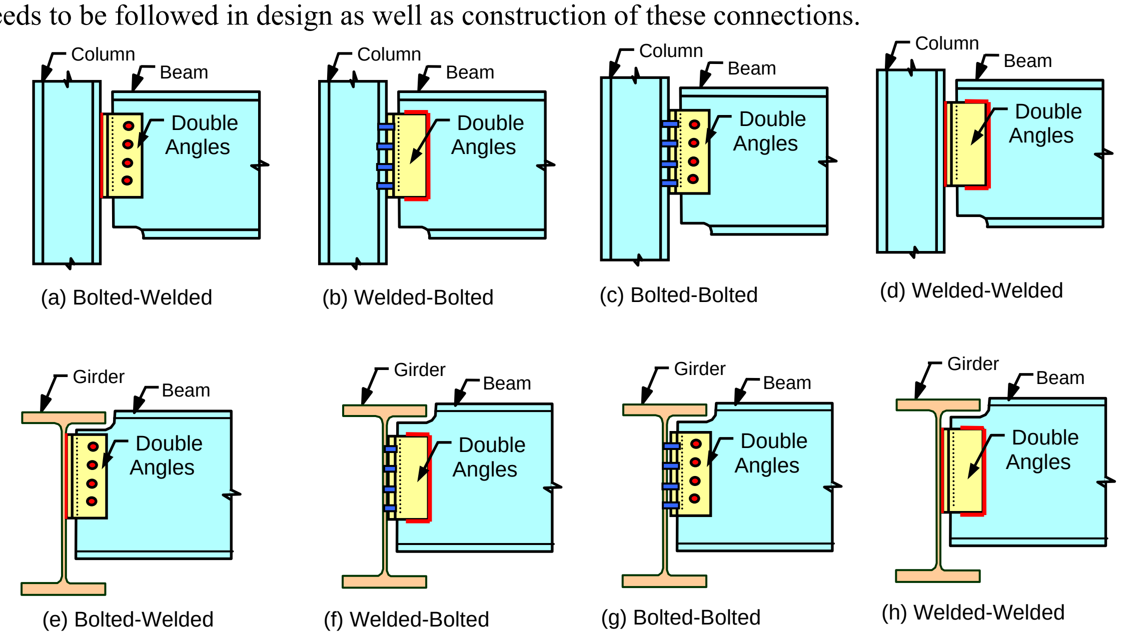


Figure 2.1. Four Types of Double-Angle Shear Connections for Beam-to-Column and Beam-to-Girder Cases

2.2. Behavior of Double-Angle Shear Connections under Gravity Load Effects

Behavior of double-angle shear connections under realistic shear and rotations was studied by Astaneh-Asl and McMullin (1993) using the test setup and test procedures discussed earlier in Section 1.5.a. The studies indicated that under gravity load, double-angle shear connections that were tested behaved in a very flexible manner and more or less as a pin connection. Compared to the shear tabs with the same depth of connection, double angles are more rotationally flexible. The flexibility of a double-angle shear connection is primarily due to the bending of outstanding legs of the angle when subjected to moment as shown in Figures 2.2 through 2.5.

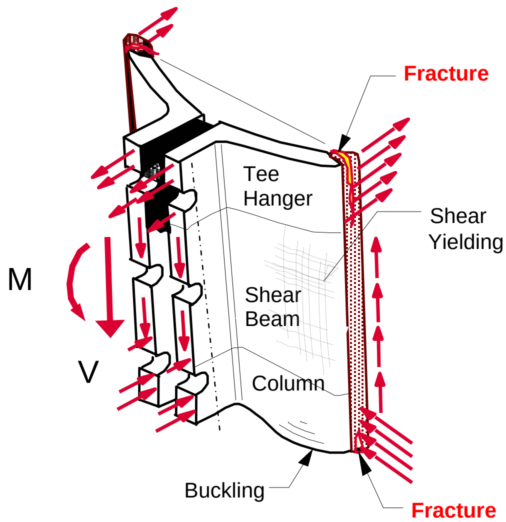


Figure 2.2. Behavior of Bolted-Welded Double-Angle Connection

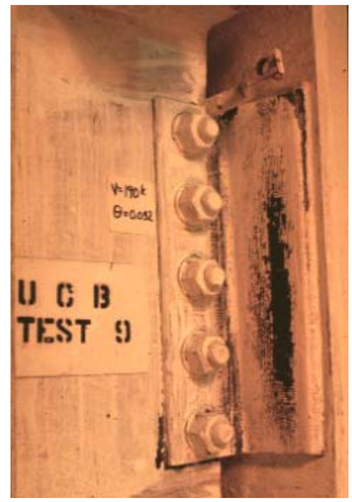


Figure 2.3. Failure of Weld

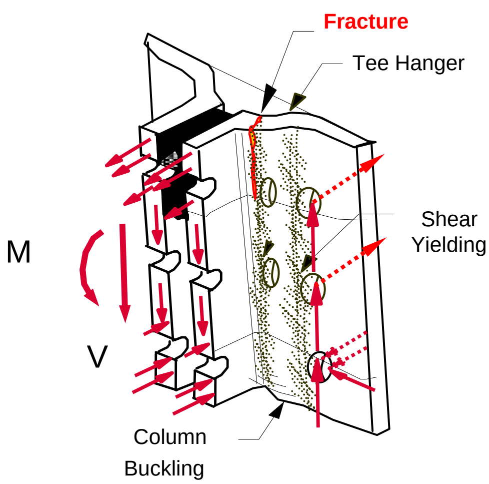


Figure 2.4. Behavior of Bolted-Bolted Double-Angle Connection

Figure 2.5. Failure of Angle

Figure 2.2 shows a bolted-welded double-angle shear connection subjected to shear force. Based on test results, three distinct zones of behavior could be observed. The top portion of the connection was primarily subjected to "pull-out" force due to bending moment in the connection. This top portion acted like a "tee hanger" being pulled out. Since angles were relatively flexible, the pull-out force resulted in angles being bent and pulled out. The bending of the course created a root-notch opening in the fillet welds connecting the angles to the column. Eventually such a root-notch opening resulted in the fracture of the top portion of the weld as shown in Figure 2.3. The bottom portion of the double angles was subjected to compression. The legs of the angles connected to the column were also subjected to compression due to the eccentricity of the applied shear from the weld line or bolt line on the column. These legs in welded connections showed a tendency to buckle and cause root-notch opening of the bottom portion of the welds. The middle portion of the double angles was actually the part that primarily was subjected to shear and was yielding in shear.

Tests of bolted-bolted double angles (Astaneh-Asl and McMullin 1993) showed a behavior similar to that of the welded connections discussed above. As shown in Figure 2.4, the top portion of the connection was acting similar to a tee hanger, the middle portion acting as a shear beam, and the bottom portion acting as a horizontal column. The final failure in this case was fracture of the top portion of the double angles near the fillets as shown in Figure 2.5.

2.2.a. Location of the Point of Inflection in Double-Angle Shear Connections

Figure 2.6 shows the moment-rotation curves for double-angle test specimens. The maximum bending capacity is related to the number of bolts, which itself is related to the depth of the connection. The relationship between shear and moment was quite nonlinear starting at relatively small moments.

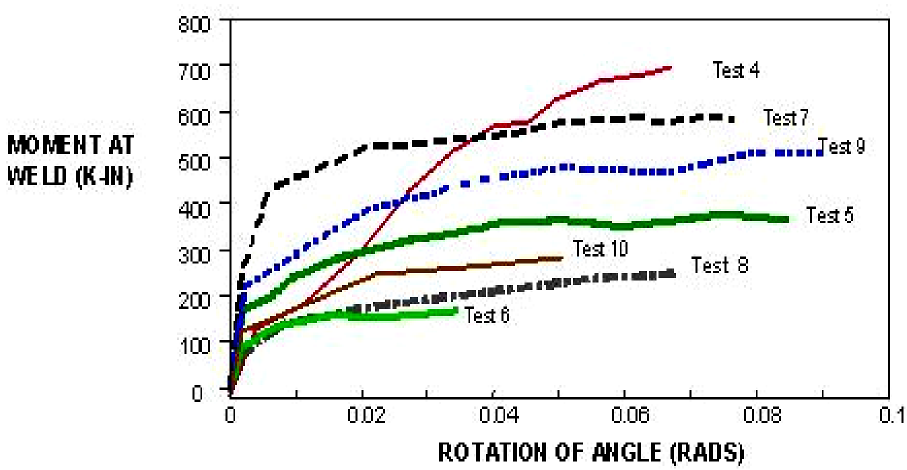


Figure 2.6. Moment-Rotation Behavior of Double-Angle Specimens

The maximum moment typically occurred when connection failed. Figure 2.7 shows the relationship between the bending moment in the connection and applied shear. As Figures 2.6 and 2.7 indicate, the connections developed relatively small moments compared to similar shear tab connections discussed in the previous chapter.

As was done for the shear tabs, it was important to measure the rotational stiffness and ductility of these connections as well as their shear strength. Also, possible failure modes had to be established and design equations to be developed. The starting point was to establish the likely location of the point of inflection of the beam. Then, with the assumption that the shear force does not change along the length of the beam between the point of inflection and the connection, the moments acting on the bolt lines and weld lines could be established and used in design of these elements. Figure 2.8 shows the eccentricity of the point of inflection from the bolt line for two specimens. Other specimens showed similar trends.

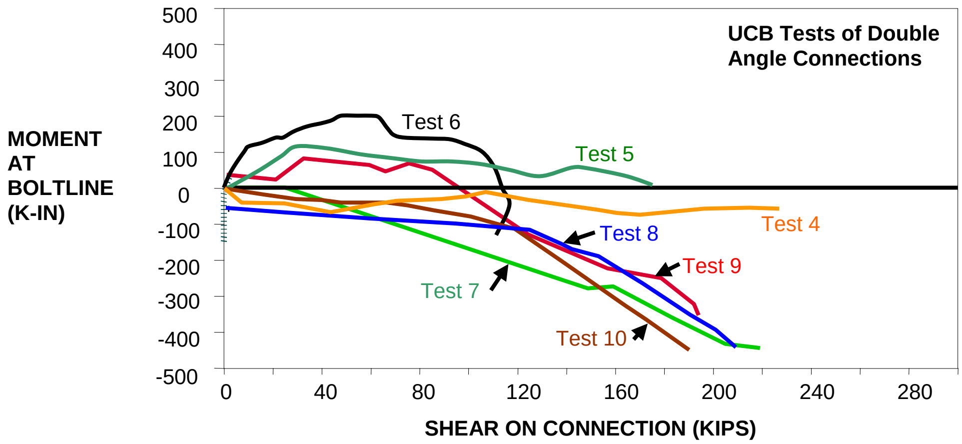


Figure 2.7. Moment versus Shear in Double-Angle Connections

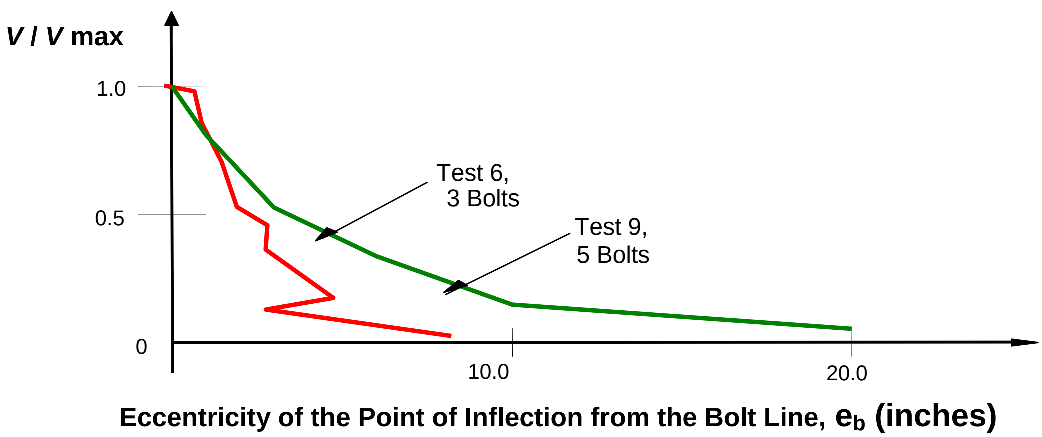


Figure 2.8. Variation of Distance of the Point of Inflection from the Bolt Line vs. Applied Shear

As Figure 2.8 indicates, the point of inflection moved towards the bolt line as the shear increased and was on or very close to the bolt line when the connections failed. Based on these results, it was concluded that the location of the point of inflection in these connections, for design purposes, can be assumed to be on the bolt line. Figure 2.9 shows the shear force acting along the bolt line in a double-angle connection. The equation that was suggested for the eccentricity of the point of inflection from the weld line was:

$$e_w = \sqrt{a^2 + b^2} \qquad (2.1)$$

Where the eccentricity of the point of inflection from the bolt line, e_b, is equal to zero:

$$e_b = 0.0 \qquad (2.2)$$

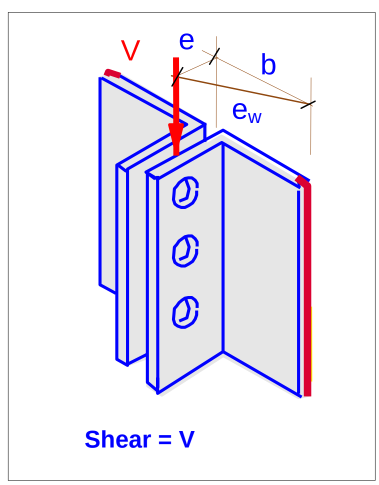


Figure 2.9. Eccentricity of Weld

where,
e_w = distance from the weld line to the point of inflection (bolt line)
a = distance from the center of the bolts to the weld line
b = width of the angle leg welded to the column.

The tests of double angles subjected to realistic shear and bending indicated that:

1. Double-angle shear connections supported the gravity load at maximum rotations varying from 0.05 to 0.09 radians. When the double support was a column, the maximum rotation achieved increased as the number of bolts in the connection decreased.
2. Shear yielding of the angles contributed significantly to the behavior of the connection, especially above expected service-level loading.
3. Moment capacity of the connections was relatively small, while flexibility and ductility were relatively high, indicating that these connections are indeed very close to the pin connection.
4. The research that has been conducted at UC-Berkeley (see Astaneh-Asl [2005] for a summary), resulted in establishing the location of the point of inflection for shear tabs having standard or short-slotted holes connected to rigid or flexible supports, which are given in the following sections.

2.2.b. Failure Modes of a Double-Angle Shear Connection

The main failure modes of a double-angle shear connection are shown in Figure 2.10 and listed in the following in the order of their desirability.

1. Yielding of the gross area of the angle legs (ductile)
2. Bearing yielding of the bolt holes in the angles and/or the beam web (ductile)
3. Fracture of the edge distance of the bolts (brittle)
4. Shear fracture of the net area of the angles (brittle)
5. Fracture of the bolts (brittle)
6. Fracture of the welds (brittle)

Failure modes 1 and 2 in Figure 2.10 are ductile, while failure modes 3, 4, 5, and 6 are brittle. The edge distance failure mode, number 3 in Figure 2.10, is unlikely in double-angle connections if the edge distances specified in the AISC specifications (AISC 1999, 2005) are satisfied. Failure modes 4, 5, and 6 are relatively brittle. An additional failure mode of the connection is block shear failure of the beam web, especially in coped beams. This failure mode is separately addressed by the AISC *Manual of Steel Construction* (AISC-LRFD 2000) and will not be further discussed in this report.

Currently, the design of double-angle shear connections is done by simply selecting a connection to carry the applied load from the tables in the AISC manuals (AISC-ASD 1989 and AISC-LRFD 2000). The tables, over the years, have proven to be easy to use, and the connections designed using these tables have performed quite satisfactorily in the field and in the laboratory as was established by tests. The AISC manual tables provide capacities that are based on the assumption of the point of inflection being along the bolt line on the web of the beam. The research summarized earlier confirmed the validity of this assumption. The tables are also based on a few other limitations, which include the maximum thickness of the angle as well as the size of the angle used.

If one desires to design a double-angle shear connection using dimensions and properties other than those tabulated in the AISC manuals (AISC-ASD 1989 and AISC-LRFD 2000) or to design the double-angle connections such that ductile failure modes govern, the design procedures given in the following section is suggested. The design procedure given below, which ensures that ductile failure modes are the governing failure modes, are well suited for seismic areas since, as indicated in Chapter 1, during a seismic event, shear connections can be subjected to relatively large cyclic rotations as well as axial forces. Therefore, it is essential that these shear connections be designed to be ductile before, during, and after the seismic event to ensure that they can resist the gravity shear after the earthquake. In addition, if these connections behave in a ductile manner during an earthquake, they can contribute to overall stiffness, strength, and damping of the structure during the earthquake, providing assistance to the main lateral load resisting system.

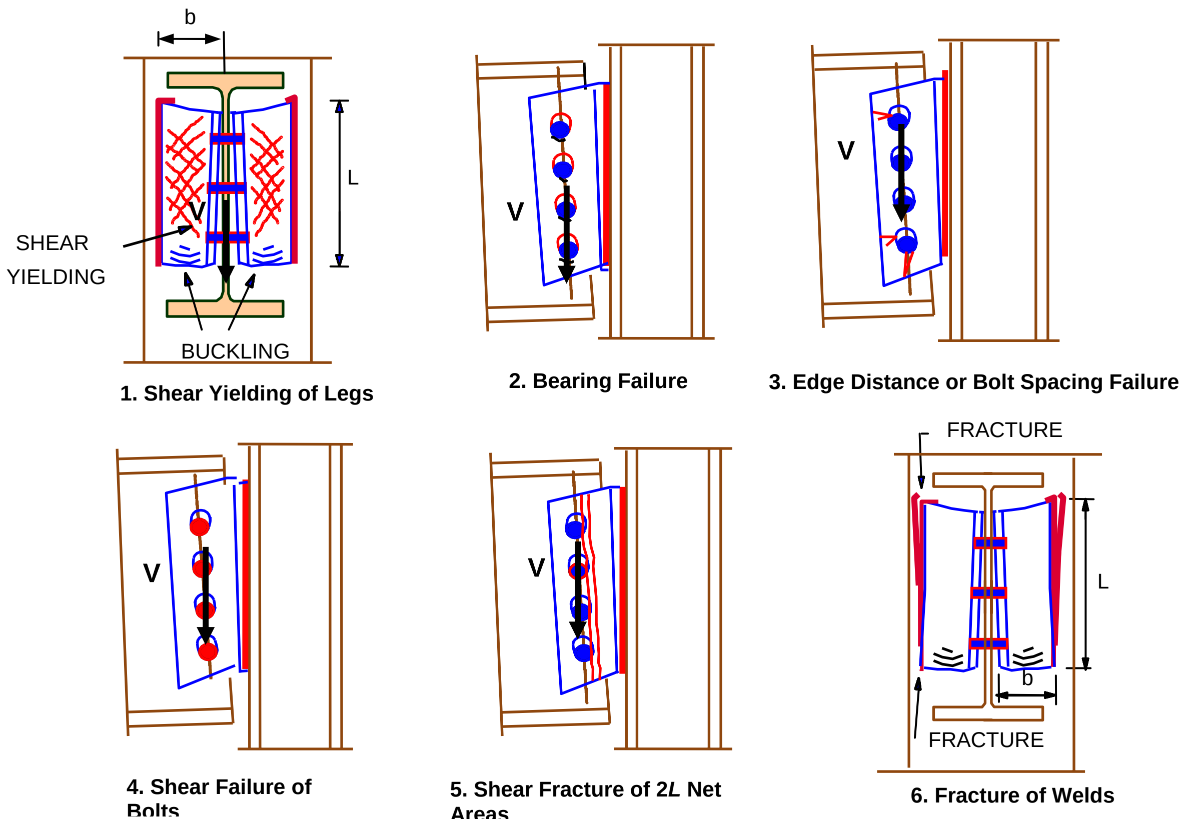


Figure 2.10. Failure Modes of Double-Angle Shear Connections

2.3. Equations for Design of *Ductile* Double-Angle Shear Connections

Failure modes of double-angle connections were discussed earlier and were shown in Figure 2.10. In the following, design equations for each failure mode are given. Following these equations in design will result in making yielding of the gross area of the angles the governing failure mode for the shear connection. As a result, the connection behavior is expected to be quite ductile not only under gravity loads but also under seismic and other extreme loads such as blast. It should be stated that these procedures are not by any means presented to replace the design procedures and tables for double-angle shear connections in the AISC manuals (AISC-ASD 1989 and AISC-LRFD 2000), and if, in any applications, the procedures given below result in a capacity more than those given in the AISC manuals, the lesser values given in the AISC manuals should be used. The design procedures given below can be considered "performance-based" design procedures, formulated to result in double-angle shear connections with a ductile failure mode of yielding of the gross area of the angle governing over other brittle modes such as fracture of the net area, bolts, or welds.

2.3.a. Yielding of Double Angles in Shear (Limit State 1)

This the most desirable failure mode to achieve a ductile behavior. Figure 2.11 shows the yielding of the angles in bolted-welded and bolted-bolted double-angle shear connections. Design of a double-angle shear connection starts with this limit state and then checks other failure modes to ensure that the shear capacity of the connection based on other failure modes is greater than the capacity due to this yield failure mode.

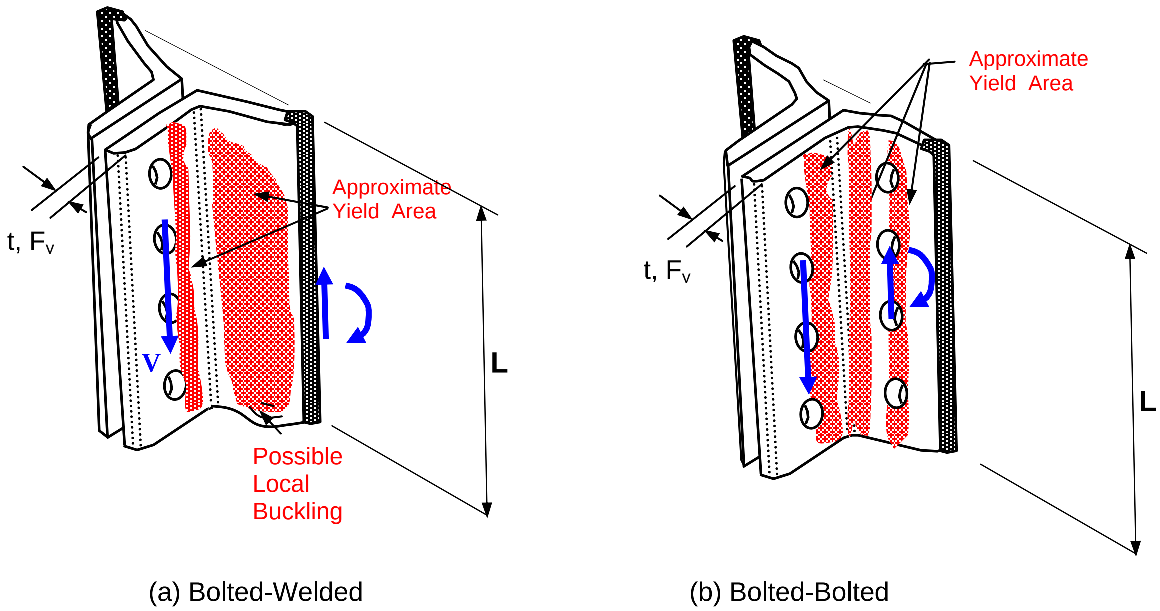


Figure 2.11. Yielding of Double Angles

The first step in design is to select double angles and design them for shear yield failure mode by using the following equations in LRFD and ASD:

$V_u \le \phi_y V_y$ (LRFD) (2.3a)

$V \le V_y / \Omega_y$ (ASD) (2.3b)

In the above equations, $\phi_y V_y$ and V_y / Ω_y are design shear strength in LRFD and ASD respectively, and

V_u = applied factored shear to the connection in LRFD

V = applied shear to the connection in ASD

$V_y = 0.60 F_y A_g$

$\phi_y = 0.90$ (LRFD) and $\Omega_y = 1.50$ (ASD)

$A_g = 2Lt$

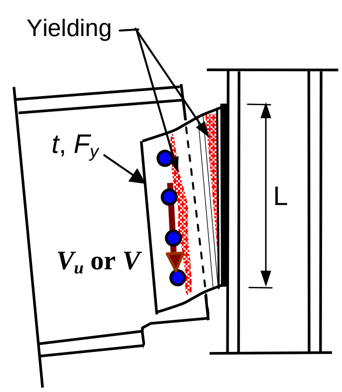


Figure 2.12. Yield Failure Mode

For definitions of the other terms in the above equations, please see the "Notations" section on page 4.

2.3.b. Bearing Failure of the Double Angle, Beam Web, or Supporting Member (Limit State 2)

For double angles, the limit state of the bearing failure should be checked against the shear yield capacity to ensure that the strength in the bearing is greater than the strength in the shear yielding:

$\phi_{br} V_{br} > \phi_y V_y$ (LRFD) (2.4a)

$V_{br} / \Omega_{br} > V_y / \Omega_y$ (ASD) (2.4b)

In the above equations, $\phi_{br} V_{br}$ and V_{br}/Ω_{br} are the design strength in LRFD and ASD, respectively, and

$V_{br}=\Sigma(1.2L_c tF_u \leq 2.4\, d_b\, tF_u)$

$\phi_{br} = 0.75$ (LRFD) and $\Omega_{br} = 2.00$ (ASD)

The term $1.2L_c tF_u$ in the above equations is the bearing capacity of each bolt using its own L_c, where L_c is the greater of the distance from the edge of the bolt hole to the edge of the plate or to the edge of the adjacent bolt hole in the direction of the applied shear. For definitions of the other terms in the above equations, please see the "Notations" section on page 4.

Equation 2.4a or 2.4b should be applied not only to angles but also to the beam web and the flange of the supporting column, and the bearing capacity of all three elements needs to be greater than the shear yield capacity of the angles.

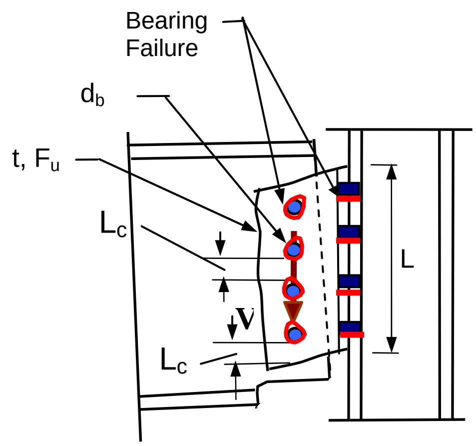


Figure 2.13. Bearing Failure Mode

2.3.c. Edge Distance Failure in the Angles or in the Beam Web (Limit State 3)

The required minimum edge distances for the shear tab as well as for the beam web should be equal to or greater than those given in the AISC specifications (AISC-ASD 1989 and AISC 1999).

2.3.d. Net-Area Fracture of the Double Angle (Limit State 4)

For the angles, the design shear strength for net area fracture in LRFD and the allowable shear force for net area fracture in ASD are $\phi_n V_n$ and V_n/Ω_n, respectively, where:

$V_n = 0.60F_u A_{nv}$

$\phi_n = 0.75$ (LRFD) and $\Omega_n = 2.00$ (ASD)

The term A_{nv} in the above equations is the "net section for shear." Currently, the AISC specifications (AISC-ASD 1989 and AISC 1999) define the net area in shear to be the area along the centerline of the bolts. However, as discussed in Astaneh-Asl (2005), the actual net section fracture occurs not through the centerline of the bolts but through the line at the edge of the bolts, Figure 2.14.

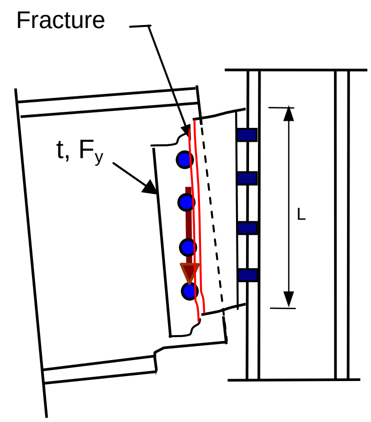


Figure 2.14. Net Section Fracture

Observing this failure mode in actual tests, Astaneh-Asl, Nader, and Malik (1989) recommended that the net section in shear be taken as the average of the net section through the center of the bolt and the gross area. The equation they recommended for net section in shear was:

$$A_{nv} = 2[A_g - 0.5n(d_b + 1/8 \text{ inch})t] \qquad (2.5)$$

For definitions of the terms, see the "Notations" section on page 4 of this report.

After establishing the net area in shear, the limit state of the fracture of net area should be checked against the shear yield capacity to ensure that the net section fracture strength is greater than the strength in shear yielding. This can be done by satisfying the following Equations 2.6a and 2.6b in ASD or LRFD, respectively:

$$\phi_n V_n > \phi_y V_y \quad \text{(LRFD)} \qquad (2.6a)$$

$$V_n/\Omega_n > V_y / \Omega_y \quad \text{(ASD)} \qquad (2.6b)$$

Where,

$\phi_n = 0.75$ and $\phi_y = 0.90$ (LRFD)

$\Omega_n = 2.0$ and $\Omega_y = 1.5$ (ASD)

In general, the number and size of the bolts and holes on both legs of the angles in bolted-bolted connections are the same. However, if the net area on the two legs of the angles differs, the above equations should be satisfied for both legs.

2.3.e. Fracture of Bolt Group (Limit State 5)

a. Design of bolts on the beam web. The bolts connecting the beam web to the double angles are subjected to pure shear. The limit state of shear fracture of these bolts should be checked against the shear yield capacity of the double angles to ensure that bolt fracture, a brittle failure mode, does not occur prior to the shear yielding of the angles, which is the desirable ductile failure mode of this connection. This can be done by satisfying the following Equations 2.7a and 2.7b in LRFD and ASD formats, respectively:

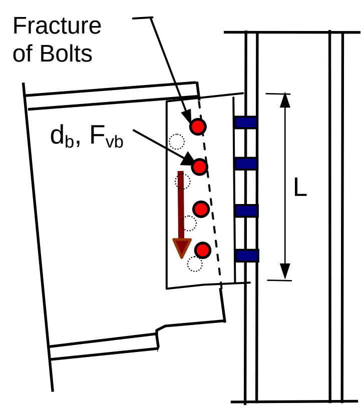


Figure 2.15. Fracture of Bolts

$$\phi_b V_b > \phi_y V_y \quad \text{(LRFD)} \qquad (2.7a)$$

$$V_b/\Omega_b > V_y / \Omega_y \quad \text{(ASD)} \qquad (2.7b)$$

Where,

$V_b = 2nA_b F_b$

$\phi_b = 0.75$ and $\phi_y = 0.90$ (LRFD)

$\Omega_b = 2.0$ and $\Omega_y = 1.5$ (ASD)

For definitions of the terms in the above equations, see the "Notations" section on page 4.

b. Design of bolts on the column flange. The bolts on the column flange, Figure 2.16, should be designed for the combined effects of direct shear and bending moment.

To design the bolt groups for the combined effects of shear and bending moment, the circular interaction Equations 2.8a and 2.8b below are suggested. Selection of a circular interaction curve for shear and moment is based on RCSC (2000), which recommends a circular interaction curve for combined shear and tension. It should be mentioned that application of the RCSC recommendation, which is for combined shear and tension acting on a bolt group, to this case of combined shear and bending may be somewhat conservative. The reason is that, for the case of combined shear and tension studied by Chesson, Faustio, and Munse (1965), all the bolts are assumed to be subjected to the same combined shear and axial load, whereas in the case of combined bending and shear, a few bolts at the bottom part of the connection and in the compression zone are subjected to shear only. In the event, due to a lack of extensive test data on bolt groups subjected to combined shear and bending, the somewhat conservative circular interaction curve, recommended for *V+N*, was adapted for *V+M* as well.

$$\left(\frac{V_u}{\phi V_b}\right)^2 + \left(\frac{M_u}{\phi M_b}\right)^2 \geq 1.0 \qquad \text{(LRFD)} \qquad (2.8a)$$

$$\left(\frac{V}{V_b/\Omega}\right)^2 + \left(\frac{M}{M_b/\Omega}\right)^2 \geq 1.0 \qquad \text{(ASD)} \qquad (2.8b)$$

Where,

$V_u = V_y$ and $M_u = V_y e_b$ (LRFD)

$V = V_y/\Omega$ and $M = (V_y/\Omega)e_b$ (ASD)

$\Omega = 2.00$ (ASD)

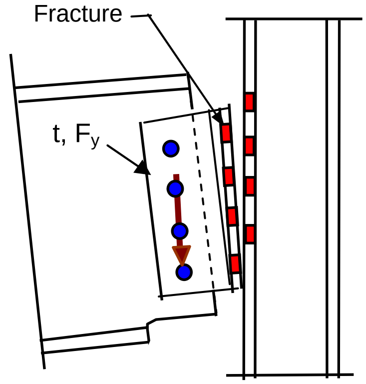
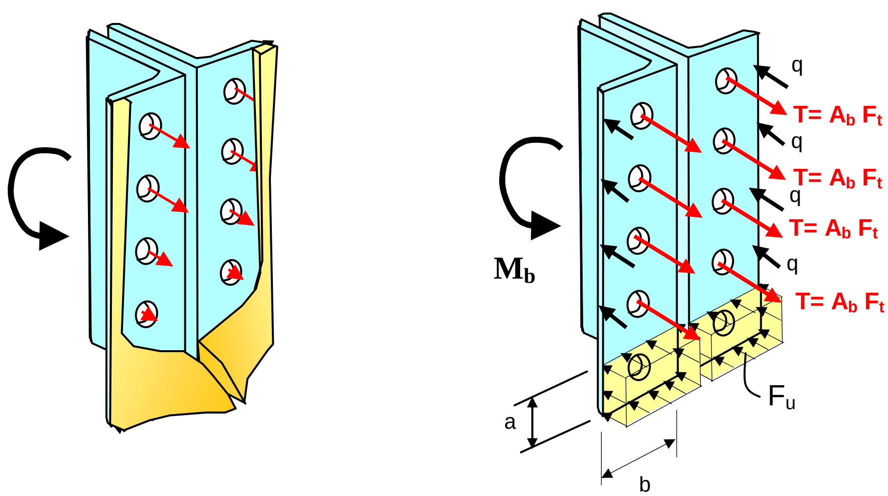


Figure 2.16. Fracture of Bolts

In the above equations, V_b and M_b are the capacity of the bolt group subjected to pure shear and pure bending. V_b is equal to $2nA_b F_b$. The bending capacity, M_b, can be established by using the stress distribution shown in Figure 2.17(b), which is a simplified version of the probable stress distribution that occurs behind the angles and is shown in Figure 2.17(a).



(a) Probable Stress Distribution (b) Assumed Stress Distribution

Figure 2.17. Probable and Assumed Stress Distribution

In Figure 2.17(b), it is assumed that a number of bolts from the top of the connection being in tension act as the tension force of the M_p while a block of compressive stresses at the bottom of the connection acts as the compression force of the moment M_p. The tension force in the top bolts resisting bending is taken as equal to $A_b F_t$, where A_b is the nominal area of the bolt and F_t is the tensile strength of the bolt. The force q, shown in Figure 2.17(b), is the prying force acting at the edge of the angle. This force is established by following the procedures in the AISC manuals (AISC-ASD 1989 and AISC-LRFD 2000) on "prying action."

2.3.f. Fracture of Welds (Limit State 6)

a. Welds connecting double angles to their support

As discussed earlier, the welds in double-angle connections are subjected to a combination of shear force and bending moment. The values of the shear force and bending moment to be combined in design according to LRFD and ASD are:

$V_u = \phi_y V_y$ and $M_u = \phi_y V_y e_w$ (LRFD) (2.9a)

$V = \phi_y V_y / \Omega_y$ and $M = (\phi_y V_y / \Omega_y) e_w$ (ASD) (2.9b)

$\phi_y = 0.9$ (LRFD) and $\Omega_y = 1.5$ (ASD)

The eccentricity, e_w, in double-angle connections is equal to the distance from the weld line to the location of the point of inflection of the beam. As discussed earlier in double-angle connections, the point of inflection can be assumed to be at the center of the bolt group or weld lines connecting the double angles to the beam web, Figure 2.18. To design the welds for combined effects of shear and bending moment, the tables for "eccentrically loaded welds" of the AISC manuals (AISC-ASD 1989 and AISC-LRFD 2000) can be used.

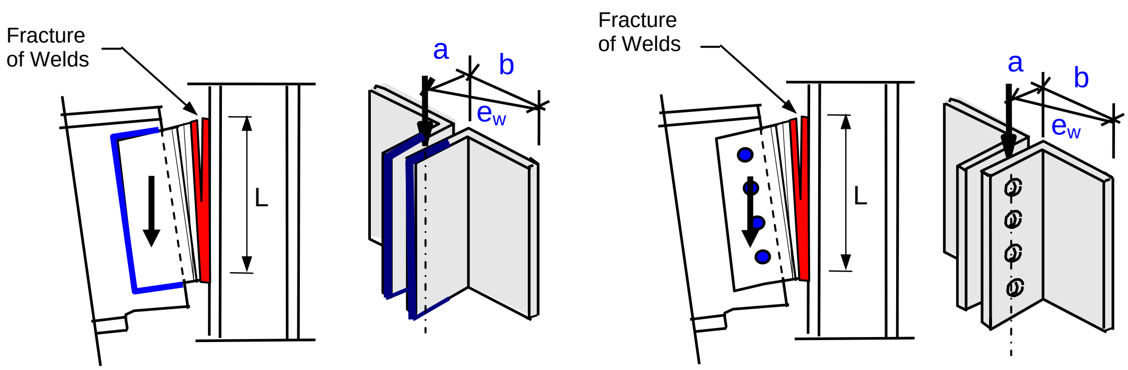


Figure 2.18. Eccentricity of Shear Force from Welds on the Support

b. Welds connecting double angles to the beam web

The welds connecting the angles to the beam web are subjected to pure shear acting at the centroid of C-shaped weld lines. Therefore, the shear strength of these welds in LRFD and ASD should be greater than the corresponding values for shear yielding of the angles:

$\phi_w V_w > \phi_y V_y$ (LRFD) (2.10a)

$V_w / \Omega_w > V_y / \Omega_y$ (ASD) (2.10b)

Where,

$V_w = 2(\Sigma L)(0.707D)(F_w)$

$\phi_w = 0.75$ and $\phi_y = 0.90$ (LRFD)

$\Omega_w = 2.0$ and $\Omega_y = 1.5$ (ASD)

For definitions of the terms in the above equations, see the "Notations" section on page 4.

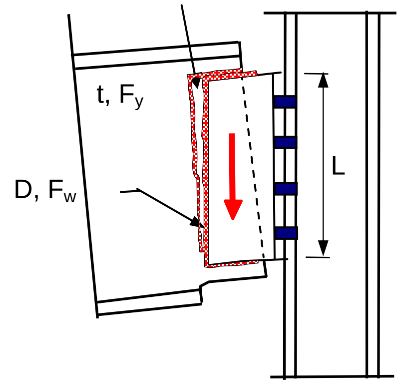


Figure 2.19. Fracture of Bolts

2.3.g. Block Shear Failure of the Double Angles or Beam (Limit State 7)

This limit state can be a governing limit state in double-angle shear connections, especially when the beam web is coped. To check the block shear failure of the coped beam, the reader is referred to the procedures in the AISC-LRFD manual (2000).

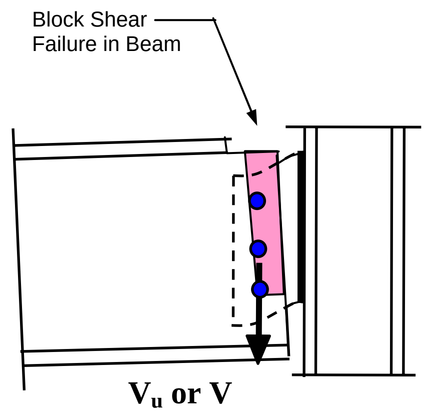


Figure 2.20

2.4. Seismic Considerations for Double-Angle Shear Connections

During earthquakes, double-angle shear connections, in addition to shear forces, are expected to develop axial force, relatively small cyclic bending moment, and relatively large cyclic rotations. Due to relatively large flexibility and ductility of typical double angles designed to behave in a ductile manner, these shear connections are expected to survive earthquakes with minimal damage to their capacity to transfer gravity shear force after the earthquake. However, in conducting time-history dynamic analyses of steel structures with double-angle connections, mathematical models of cyclic behavior of these connections are needed. To establish such mathematical models, a series of cyclic tests of double angles were conducted by Astaneh-Asl, Nader, and Malik (1989), Ho and Astaneh-Asl (1990), and Shen and Astaneh-Asl (1999). The results of these tests were then used to establish actual cyclic behavior and failure modes as well as to develop seismic design recommendations. In the following brief summaries of these projects, the results that can be used in design offices are provided.

2.4.a. Behavior of Double Angles Subjected to Combined Shear and Axial Force

Redwood and Eyre (1984) performed a series of tests by subjecting double-angle connections to axial cyclic loads. All connections tested were welded to the beam web and bolted to the column. The connections were subjected to a constant shearing force and cyclic load applied along the axis of the beam. Reserve strength after fifteen cycles was investigated. The test results indicated that the behavior was highly ductile due to flexibility of the angles, and failure was due to localized plastic bending. Redwood and Eyre concluded that considerable tensile membrane action has developed on the angle legs and that flexural action was of lesser significance near the ultimate load. It was also found that the reserve strength of the connection after cyclic loading dropped with increasing load amplitude. The effect of shear was noticeable.

During the late 1980s and 1990s a series of studies of bolted and riveted double-angle connections subjected to cyclic axial force, with or without shear force, were conducted at the University of California at Berkeley (Astaneh-Asl, Nader, and Malik 1989; Astaneh-Asl and Nader 1990; and Shen and Astaneh-Asl 1999). These studies, which were part of a greater research project on the San Francisco–Oakland Bay Bridge, resulted in information on actual cyclic behavior of riveted and bolted double-angle connections as well as models that can be used to establish the realistic stiffness, strength, and ductility of these connections. One of the projects by Ho and Astaneh-Asl (1990) studied the behavior of double-angle shear connections subjected to constant shear and cyclic axial load representing shear due to gravity combined with cyclic axial load due to seismic effects. This project is summarized below.

The main objective of the study of double angles by Ho and Astaneh-Asl (1990) was to perform experiments and to establish hysteresis behavior of double-angle shear connections under the effects of axial pull-push by the beam while gravity shear force was kept constant. The pull-push cycles were designed to simulate wind and earthquake loading, and the constant shear represented the gravity load acting on the connection. The research program consisted of testing five full-size double-angle shear connections. All connection angles were of the same length but of varying thicknesses. Two types of connections were tested: in the first type the angles were bolted to the beam web and welded to the column flange, and in the second type the angles were bolted to the beam web and bolted to the column flange. Figure 2.21 shows typical test specimens and test setup. The setup shown in Figure 2.21 was specially designed with three actuators to subject the connection to constant shear and variable cyclic axial load. Each specimen was subjected to a small monotonic pull cycle first in order to determine the axial stiffness of the connection. After completion of this pull cycle, a shear and a rotation were applied to the connection to represent service gravity load effects using actuators S and R in Figure 2.21. Shear force applied to each specimen was the design shear given in the AISC-ASD manual (1989). The specimen was then subjected to pull-push cycles of increasing displacements using actuator A while shear force was kept constant. The cyclic axial loading continued until failure occurred.

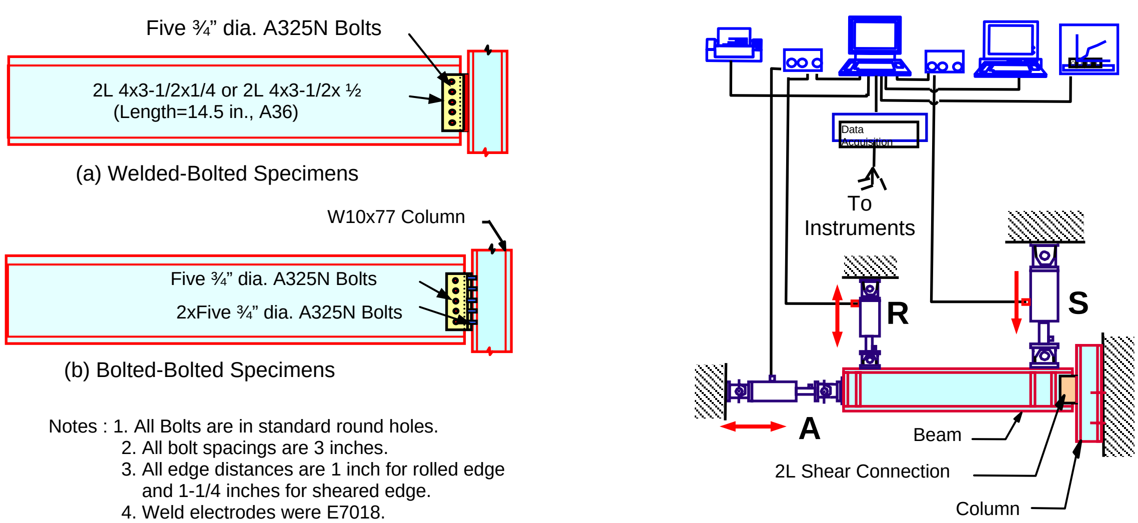


Figure 2.21. Typical Specimens and Test Setup

Figure 2.22 shows two of the test specimens at the end of the test. In the specimen on the left, the thickness of the angle is ½ inch, while for the specimen on the right, the thickness of the angle was ¼ inch. All specimens performed in a ductile manner and formed two distinct plastic hinges on each outstanding leg, as shown in Figure 2.22, one along the bolt line and one near the angle fillet. Figure 2.23 shows typical test results.

The tests indicated that:

(1) Double-angle connections behaved in a highly ductile manner under cyclic axial loading. The hysteresis response was unsymmetrical, showing very small strength and stiffness under pulling action but much higher strength and stiffness when subjected to pushing by the beam.

(2) The connections deteriorated due to cyclic axial loading. As a result, shear resistance of the connections was reduced markedly. This effect was more pronounced in bolted-bolted connections compared to bolted-welded connections in which shear deformation was insignificant.

3. The failure mode in bolted-welded connections was governed by fracture of welds. In all bolted-welded connections tested, cracks initiated in the weld return area.
4. The failure mode in bolted-bolted connections was fracture of the connection angle along the fillets with widespread yielding around bolt holes. Plastic yield lines were formed adjacent to the fillets on both legs as well as along the bolt line on the outstanding leg. Fracture was due to effects of combined shear and bending.
5. Strains developed near the fillets of the angles increased with increasing angle thickness, which resulted in a more rapid loss of shear resistance in the thicker angles.
6. Prying action was significant due to flexibility of the connection angles. The effect was an amplification of bolt forces by 25 to 41percent in the tested specimens.

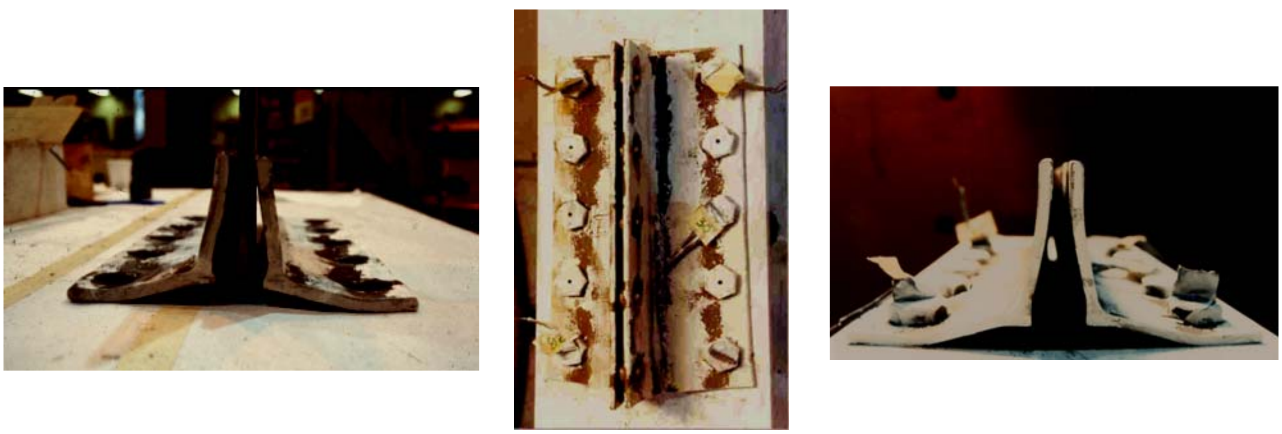
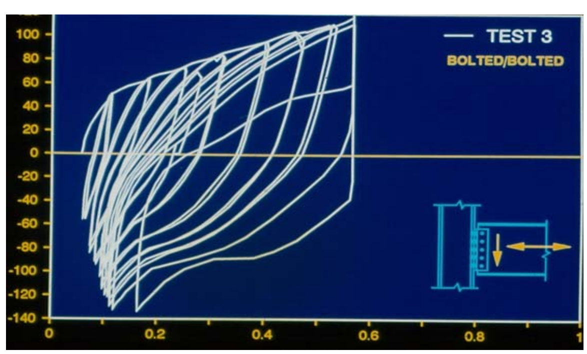


Figure 2.22 Typical Specimens at the End of the Tests

Axial Force, kips



Axial Displacement, inches

Figure 2.23. Typical Axial Force–Axial Deformation of the Connections

In addition to the above experimental studies, Thornton (1997), using available test data, developed and proposed equations for the minimum size of the welds and the diameter of the bolts in terms of the geometry and the material properties of the angles that would ensure ductile behavior of double-angle shear connections. In the same paper, similar equations were proposed for tee and end-plate shear connections to ensure their ductile behavior. These equations are now part of the AISC-LRFD *Manual of Steel Construction* (AISC-LRFD 2000), and the reader is referred to the manual or the paper (Thornton 1997) for further information.

2.5. Notes on the Design of Double-Angle Shear Connections Subjected to Shear and Axial Load

Figure 2.24 shows four types of double-angle shear connections subjected to combined shear and axial forces. The shear force is primarily due to gravity, and the axial load is assumed to be due to seismic effects. In designing the connection to resist the applied forces, three major properties of the connection, stiffness, strength, and ductility, need to be considered.

In modeling a steel structure for the analysis, usually shear connections are modeled as pin connections with zero rotational stiffness and infinite shear and axial stiffness. In reality, as shown earlier, these connections have some, although not very large, rotational stiffness. However, the assumption of zero rotational stiffness for these connections can be justified to obtain conservative values of drift. For the shear stiffness, the assumption of infinite rigidity implied by a pin connection can also be justified since, even during an inelastic range of deformations, the total shear deformation is relatively small. As for the axial stiffness, the situation is quite different. Depending on whether the axial force is a push or a pull force against the support, these connections will show very large or very small axial stiffness. The axial stiffness of a shear connection plays an important role in two situations: one is when the shear connection is acting as the bracing point for a column to reduce its buckling length, and the second situation is when a shear connection is connecting a collector beam to its support and is expected to transfer large axial forces to the support. The latter occurs in collectors of braced frames transferring lateral force to the braced bay.

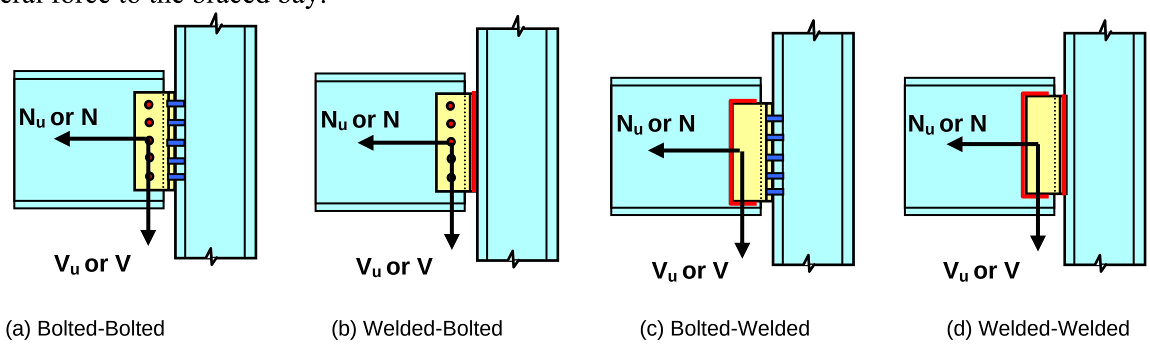


Figure 2.24. Four Types of Double-Angle Connections Subjected to Shear and Axial Forces

2.5.a. Yielding of the Double Angles under Combined Shear and Axial Load (Limit State 1)

For this failure mode, which involves yielding of the plate under combined shear and normal stresses, the Von Mises yield criterion and a circular interaction curve are used. The maximum factored axial force (in LRFD) and the maximum allowable axial force (in ASD) can be obtained from the following interaction equations:

$$\left(\frac{V_u}{\varphi_y V_n}\right)^2 + \left(\frac{N_u}{\varphi_y N_n}\right)^2 = 1.0 \qquad \text{(LRFD)} \qquad (2.11a)$$

$$\left(\frac{V}{V_n/\Omega_y}\right)^2 + \left(\frac{N}{N_n/\Omega_y}\right)^2 = 1.0 \qquad \text{(ASD)} \qquad (2.11b)$$

Where $\phi_y = 0.90$ (LRFD) and $\Omega_y = 1.50$ (ASD)

For definitions of the other terms in the above equations, see the "Notations" section on page 4.

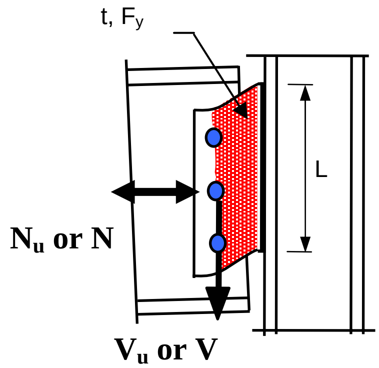


Figure 2.25

2.5.b. Bearing Failure of Double Angles under Combined Shear and Axial Load (Limit State 2)

Similar to the yielding of gross area, the Von Mises yield criterion is used for this failure mode as well. The maximum factored axial force in LRFD and the maximum allowable axial force in ASD can be obtained from the following interaction equations respectively:

$$\left(\frac{V_u}{\phi_{br} V_{br}}\right)^2 + \left(\frac{N_u}{\phi_{br} N_{br}}\right)^2 = 1.0 \qquad \text{(LRFD)} \quad (2.12a)$$

$$\left(\frac{V}{V_{br}/\Omega_{br}}\right)^2 + \left(\frac{N}{N_{br}/\Omega_{br}}\right)^2 = 1.0 \qquad \text{(ASD)} \quad (2.12b)$$

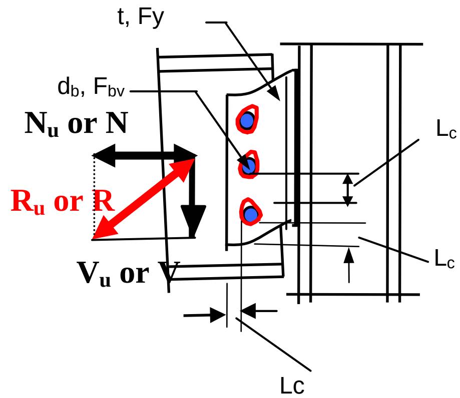


Figure 2.26

Where;
V_{br} = bearing capacity of the bolt group in the direction of shear (vertical direction)
N_{br} = bearing capacity of the bolt group in the direction of axial force (horizontal direction)
$\phi_{br} = 0.75$ (LRFD) and $\Omega_{br} = 2.00$ (ASD)

For definitions of the terms in the above equations, please see the "Notations" section on page 4.

2.5.c. Edge Distance Failure in the Angles or in the Connected Members (Limit State 3)

This failure mode is the same as the edge distance failure under pure shear discussed in Section 2.4 earlier. The required minimum edge distances for the beam web are equal to those given in the AISC-LRFD (2000) specifications or two times the bolt diameter, whichever is greater.

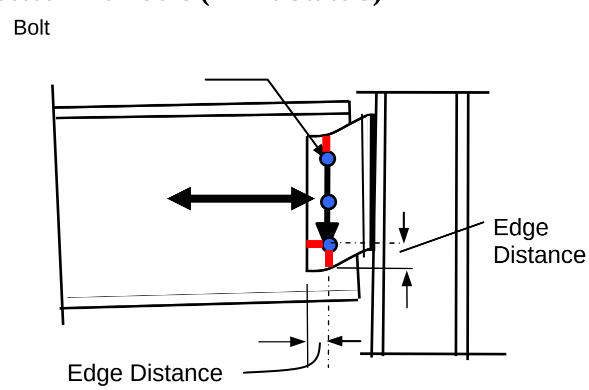


Figure 2.27

2.5.d. Net-Area Fracture of the Plate under Combined Shear and Axial Force (Limit State 4)

Similar to the yielding of gross area, the Von Mises yield criterion is used for this failure mode as well. The maximum factored axial force in LRFD and the maximum allowable axial force in ASD can be obtained from the following interaction equations, respectively:

$$\left(\frac{V_u}{\varphi_n V_n}\right)^2 + \left(\frac{N_u}{\varphi_n N_n}\right)^2 = 1.0 \quad \text{(LRFD)} \qquad (2.13a)$$

$$\left(\frac{V}{V_n/\Omega_n}\right)^2 + \left(\frac{N}{N_n/\Omega_n}\right)^2 = 1.0 \quad \text{(ASD)} \qquad (2.13b)$$

Where:

$V_n = 0.60F_u\, A_{nv}$

$N_n = F_u\, A_n$

$A_{nv} = 2[A_g - 0.5n(d_b + 1/8 \text{ inch})]$

$A_n = 2[A_g - n(d_b + 1/8 \text{ inch})]$

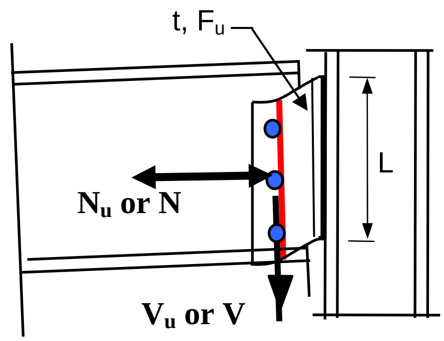


Figure 2.28

For definitions of the terms in the above equations, please see the “Notations” section on page 4.

2.5.e. Fracture of Bolts under Combined Shear and Axial Force (Limit State 5)

a. Design of bolts on the beam web. The shear and axial force applied to the connection created only shear in the bolts connecting the beam to the double angles. Therefore, the strength of the bolt group in shear, $\phi_b V_b$ in LRFD and V_b/Ω in ASD, should satisfy the following equations:

$$(\varphi_y V_y)^2 + (\varphi_y N_y)^2 \le (\varphi_b V_b)^2 \quad \text{(LRFD)} \quad (2.14a)$$

$$(V_y/\Omega_y)^2 + (N_y/\Omega_y)^2 \le (V_b/\Omega_b)^2 \quad \text{(ASD)} \quad (2.14b)$$

Where,

$V_b = 2nA_b F_b$

$\phi_b = 0.75$ and $\phi_y = 0.90$ (LRFD)

$\Omega_b = 2.0$ and $\Omega_y = 1.5$ (ASD)

For definitions of the terms in the above equation, see the “Notations” section on page 4.

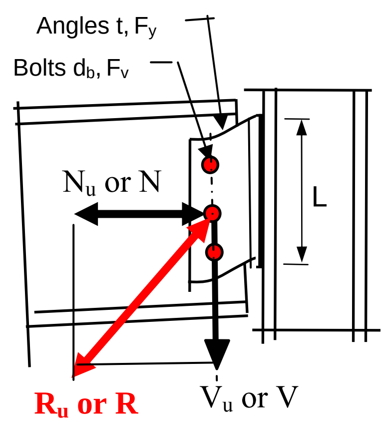


Figure 2.29

b. Design of bolts on the column flange. The bolts on the column flange, Figure 2.30, should be designed for the combined effects of direct shear, axial force, and bending moment. To design the bolt groups for the combined effects of shear, axial load and bending moment, the circular interaction Equations 2.15a and 2.15b below are suggested:

$$\left(\frac{V_u}{\varphi_b V_b}\right)^2 + \left(\frac{N_u}{\varphi_b N_b}\right)^2 + \left(\frac{M_u}{\varphi_b M_b}\right)^2 = 1.0 \quad \text{(LRFD)} \qquad (2.15a)$$

$$\left(\frac{V}{V_n/\Omega_b}\right)^2 + \left(\frac{N}{N_b/\Omega_b}\right)^2 + \left(\frac{M}{M_b/\Omega_b}\right)^2 = 1.0 \quad \text{(ASD)} \qquad (2.15b)$$

Where,

V_u = factored applied shear

N_u = factored applied axial force

M_u = factored applied bending moment

V = unfactored applied shear

N = unfactored applied axial force

M = unfactored applied bending moment

V_b = shear strength of the bolt group under pure shear

N_b = tensile strength of the bolt group under pure tension

M_b= plastic moment capacity of the bolt group in bending given in Section 2.3.e above.

ϕ_b = 2.00 (LRFD) and Ω_b = 2.0 (ASD)

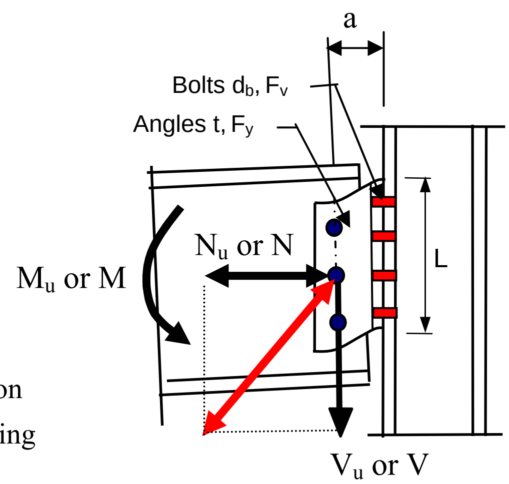


Figure 2.30

2.5.f. Fracture of Welds under Combined Shear and Axial Load (Limit State 6)

As discussed earlier, welds in these connections are designed to be stronger than the plate to force the plate to yield first and undergo inelastic deformations prior to failure of the welds. The equation that was derived in Astaneh-Asl (2005) for this purpose was in the form of:

$$D\,F_w \geq 1.45\,t\,F_y \qquad (2.16)$$

The above equation can also be used for combined shear, bending, and axial load.

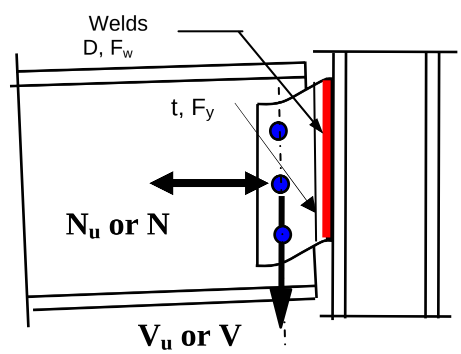


Figure 2.31

2.5.g. Block Shear Failure of the Double Angles or Beam Web under Combined Shear and Axial Force (Limit State 7)

This limit state can be a governing limit state in double-angle shear connections, especially when the beam web is coped.

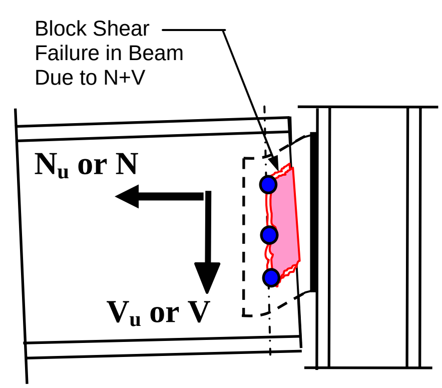


Figure 2.32

2.6. Behavior of Double Angles Subjected to Cyclic Moment Rotations

As mentioned earlier, during earthquakes, double-angle shear connections, especially those that are part of moment frames buildings, can be subjected to relatively large cyclic rotations. In order to study cyclic behavior of double-angle shear connections subjected to cyclic moments and rotations, a series of tests were conducted by Astaneh-Asl, Nader, and Malik (1989). A typical specimen and test setup are shown in Figure 2.33. Three specimens were representative of current steel construction and are chosen to be discussed here. In these specimens, 2*L*3x3x3/8, A36 double angles were welded to the beam web and bolted to the flange of the supporting column using A325 bolts. The three specimens had 4, 5, or 6 bolts on each angle with respective length of angle being 12, 15, or 18 inches. The beams were W16x40 for 4-bolted connections and S24x80 for both 5- and 6-bolted connections.

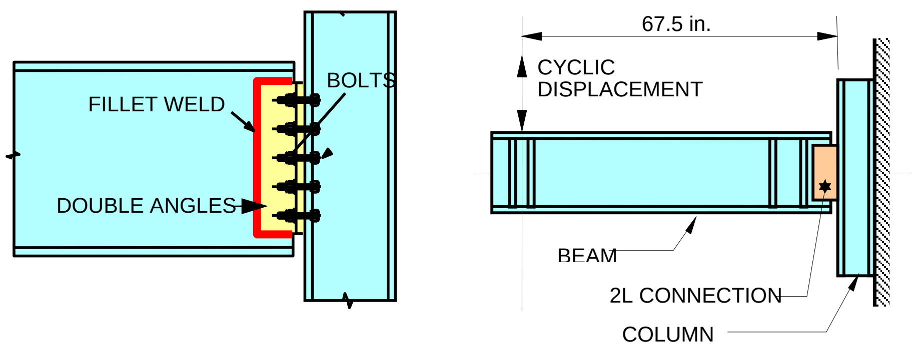


Figure 2.33. Typical Specimen and Test Setup

Figure 2.34 shows a typical specimen at the end of the test and the corresponding moment-rotation curve. Based on cyclic tests, the following conclusions were reached:

1. All three specimens behaved in a very ductile manner under cyclic loading.
2. In all three specimens two plastic hinges formed on each bolted leg of the angles. One plastic hinge was just at the end of the fillet, and the other plastic hinge was along the side of the bolt hole.
3. The failure mode of all three specimens was eventual fracture of the top or bottom of the bolted leg near the fillet.
4. Measurements of tension force in the bolts indicated that the bolts maintained about one-third of their pretensioning and did not become totally loose.
5. During cyclic testing, the neutral axis was moving between the center of the top and bottom bolts.
6. The back-to-back legs, welded to the beam web, as well as the welds themselves, remained essentially elastic throughout the cyclic loading.

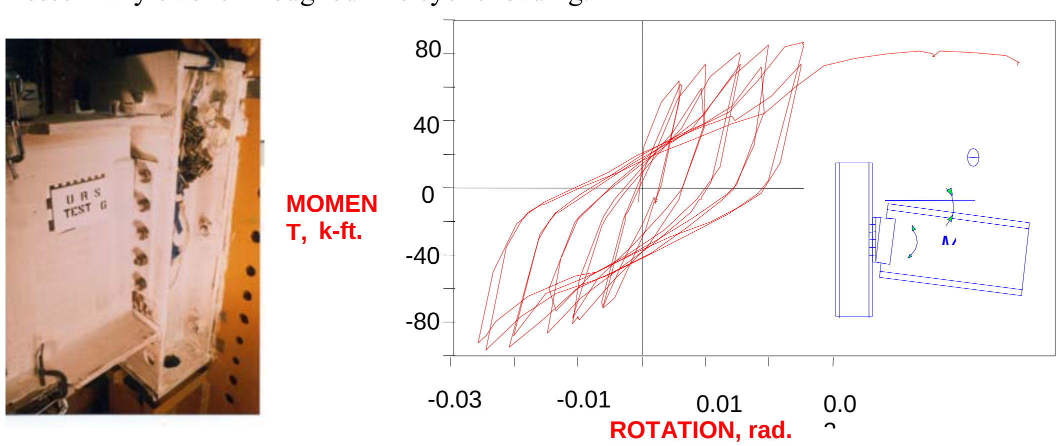
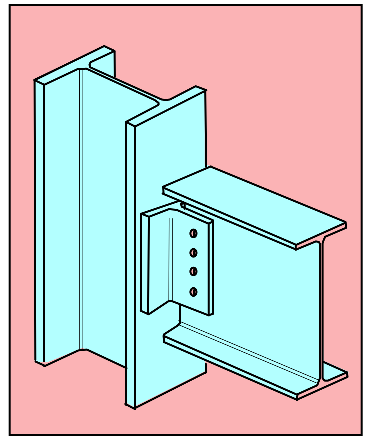


Figure 2.34. Specimen with Five Bolts at the End of Cyclic Tests and Its Moment-Rotation Curves

3. TEE SHEAR CONNECTIONS



3.1. Introduction

Tee shear connections have many advantages of both shear tabs and double angles. Similar to shear tabs, the stem of the tee is connected to one side of the beam web. This makes the erection easy and there is no need to cope a flange on the connected beam for erection as is the case for some double-angle connections. On the other hand, similar to double angles, the flange of the tee bends and permits the connection to rotate. As a result, the connection is very flexible and the moment in the connection is negligible. Another advantage of a tee shear connection is that it can be used on box columns or on the web side of the wide flange columns with great efficiency without causing out-of-plane bending of the web. Figure 3.1 shows typical applications of the tee shear connections. As for the connector types, tee shear connections can be used with welds and bolts as shown in Figure 3.2. The most common types today are (a) and (b) in the figure, where the tee is bolted or shop-welded to the column and field-bolted to the beam.

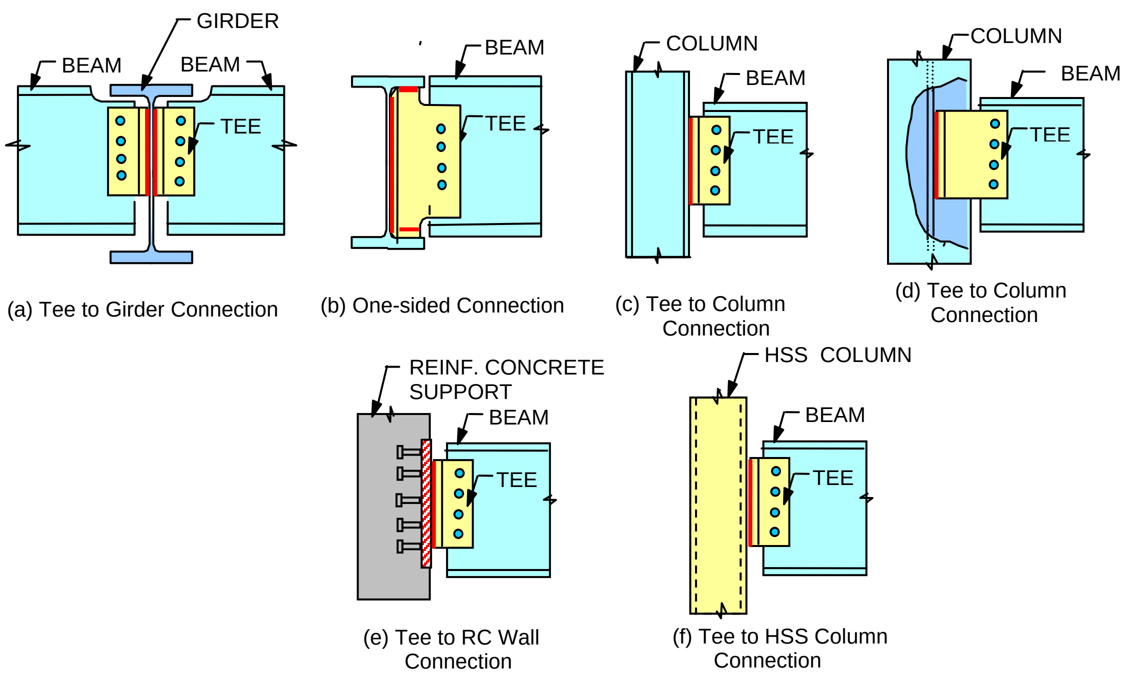


Figure 3.1. Typical Applications of Tee Shear Connections

Behavior of tee shear connections under gravity load has been studied by Astaneh-Asl and Nader (1989 and 1990) by conducting full-scale tests, and design recommendations have been developed and proposed by the researchers and others (Thornton 1996 and Thornton 1997). Currently, the AISC manuals (AISC-ASD 1989 and AISC-LRFD 2000) do not have tables that can be used in rapid selection of tee shear connections. Perhaps this is one of the reasons for tee shear connections not being used as frequently as shear tabs or double-angle connections, for which design tables are available in the manual. The information available on the behavior of tee shear connections, as summarized later in this chapter, indicates that these connections are very versatile and have the advantages of both shear tabs and double angles.

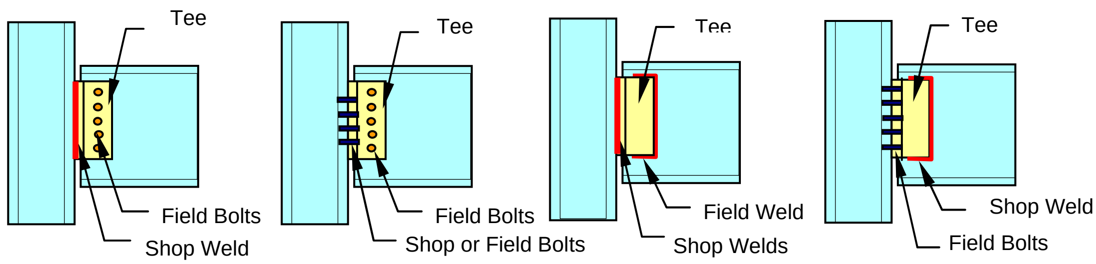


Figure 3.2. Use of Bolts and Welds in Tee Shear Connections

In the area of seismic design, a literature survey did not turn up any data on cyclic tests of tee shear connections. Similarly, no published information could be found on recommendations on seismic design of tee shear connections. In this report, an attempt is made to formulate seismic design recommendations, albeit on the conservative side, that can result in more ductile tee shear connections. It is hoped that as cyclic test results become available in the future, better and more efficient seismic design recommendations can be formulated and proposed for these seemingly versatile connections.

3.2. Behavior of Tee Connections under Gravity Load Effects

Tests of tee shear connections subjected to realistic effects of gravity load (Astaneh-Asl and Nader 1989 and Astaneh-Asl and Nader 1990) have indicated that tee connections are quite flexible and, similar to double angles, can easily accommodate the end rotation demand of simply supported beams. Tee connections are also rotationally quite ductile. The flexibility and rotational ductility of tee shear connections are primarily due to three effects: (a) out of plane bending of the flange of the tee, (b) yielding of the stem under combined shear and bending, and (c) slip of bolts and bearing deformation of bolt holes (in bolted tees). The three effects are clearly visible in Figure 3.3, where specimens of tee shear connections are shown at the end of the tests.

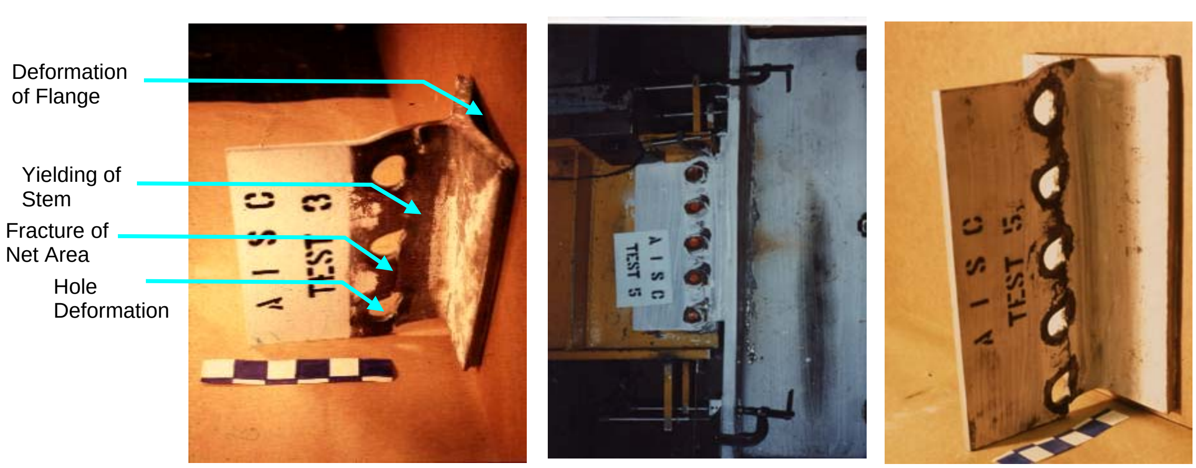


Figure 3.3. Inelastic Deformations of Tee Shear Connections
(Astaneh-Asl and Nader 1990)

3.2.a. Location of the Point of Inflection for Tee Shear Connections

Astaneh-Asl and Nader (1990) reported results of nine tests of full-size tee shear connections subjected to realistic shear and rotations. The tee connections either had 3 or 5 bolts. A325 as well as A490 bolts were used in the tests. The material of the test specimens was A36. Full details of test specimens can be found in Astaneh-Asl and Nader (1990). The tests indicated that the location of the point of inflection of a simply supported beam with tee end connections is between the bolt line and the weld line. In a conservative approach it was recommended that the location of the point of inflection be taken as along the bolt line. This is similar to double-angle shear connections, where the point of inflection of the beam is also along the bolt line, making these two shear connections develop a relatively small moment compared to the shear tab connections.

The tests of nine tee shear connections also revealed that:

1. Tee shear connections supported gravity load at maximum rotations varying from 0.06 to 0.07 radians.
2. Shear deformation and distortion contributed significantly to the behavior of the connection, especially above expected service-level loading.
3. Moments developed in the connections were relatively small.

3.2.b. Failure Modes of a Tee Shear Connection

When a tee shear connection is subjected to shear and rotation, the following failure modes are possible:

1. Yielding of the gross area of the stem of the tee (ductile)
2. Yielding of the gross area of the flange of the tee (ductile)
3. Bearing yielding of the bolt holes in the stem, tee flange, and/or beam web (ductile)
4. Fracture of the edge distance of the bolts in bolted tees (brittle)
5. Shear fracture of the net area of the stem or tee flange (brittle)
6. Fracture of the bolts (brittle)
7. Fracture of the welds (brittle)

In the above list, the failure modes are divided into two categories of "ductile" and "brittle" and have been placed in the order of their desirability. Figure 3.4 graphically shows the same failure modes and their hierarchy.

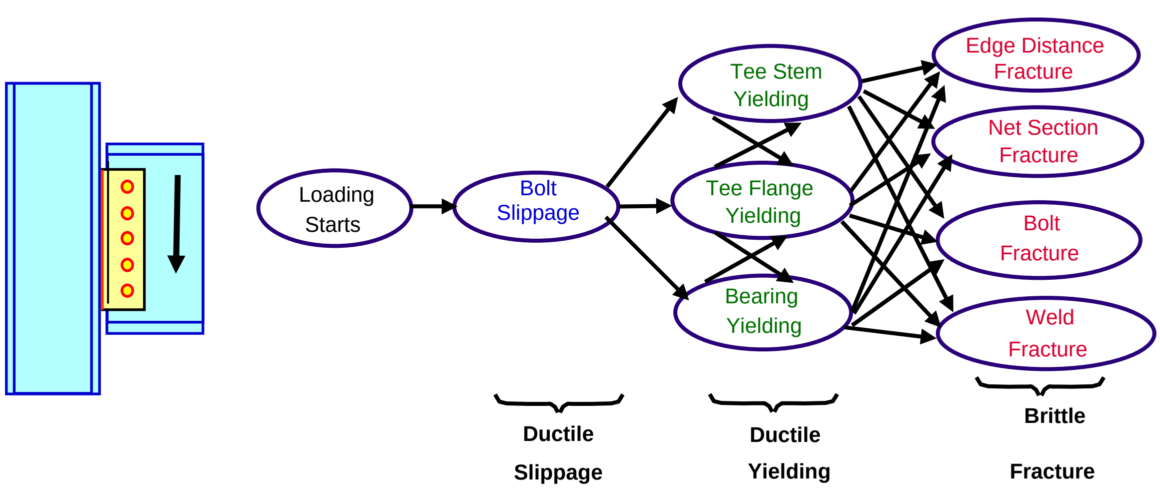


Figure 3.4. Limit States (Failure Modes) of Tee Shear Connections

Failure modes 1, 2, and 3 in the above list are associated with yielding of the steel and are considered ductile failure modes. Ductile failure modes are more desirable than the more brittle failure (that is, failure modes 4 through 7 in the above list). As mentioned in previous chapters as well, during a ductile failure mode, a relatively large volume of steel yields, plastically deforms, and yet maintains its yield strength. Brittle failure modes involve yielding of a relatively small volume of steel, bolts, or welds followed by fracture in a relatively abrupt and undesirable manner. When a brittle failure mode occurs, the fractured part loses its strength without much noticeable deformation. Slippage of bolts is also included in Figure 3.4 in the hierarchy of failure modes. However, slippage of bolts is not a failure mode as long as it does not occur under service load. With current AISC requirements on bolt tightening, bolt slippage is expected to occur under a load greater than nominal (unfactored) design loads.

3.3. Design of Tee Shear Connections for Shear Force

In the following, design equations for each of the six failure modes discussed earlier are provided.

3.3.a. Yielding of Stem and/or flanges of the Tee in Shear (Limit State 1)

The design of tee shear connections starts with this limit state. To ensure that this ductile limit state governs over other more brittle limit states (3 through 6 above), the factored shear force (in LRFD) and the applied shear force (in ASD), established by analysis, should be less than or equal to the design shear strength of the tee in LRFD and ASD, respectively:

$$V_u \leq \phi_y V_y \quad \text{(LRFD)} \qquad (3.1a)$$

$$V \leq V_y / \Omega_y \text{(ASD)} \qquad (3.1b)$$

The design shear yielding strength, $\phi_y V_y$, and the allowable shear yielding strength, V_y/Ω_y, of a tee shear connection due to yielding of the stem are given as:

$V_y = 0.60 F_y A_g$

A_g = smaller of the gross area of the stem or the flange of the tee = smaller of $(t_s L)$ or $(2 t_f L)$.

$\phi_y = 0.90$ (LRFD), $\Omega_y = 1.5$ (ASD)

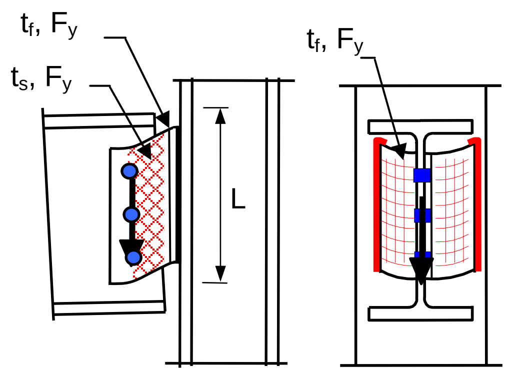


Figure 3.5. Yielding of Tee

For definitions of the terms in the above equations, please see the "Notations" section on page 4.

3.3.b. Bearing Failure of the Tee Shear Connection (Limit State 2)

The limit state of bearing failure should be checked against the shear yield capacity to ensure that the strength in bearing is greater than the strength in shear yielding.

$$\phi_{br} V_{br} > \phi_y V_y \qquad \text{(LRFD)} \qquad (3.2a)$$

$$V_{br}/\Omega_{br} > V_y / \Omega_y \qquad \text{(ASD)} \qquad (3.2b)$$

The design bearing strength, $\phi_{br} V_{br}$, and the allowable bearing strength, V_{br}/Ω_{br}, of the tee stem, tee flange, beam web, and column flange, Figure 3.6, is given as:

$V_{br} = \Sigma[1.2 L_c t F_u \leq 2.4 d t F_u]$

$\phi_{br} = 0.75$ (LRFD) and $\Omega_{br} = 2.00$ (ASD)

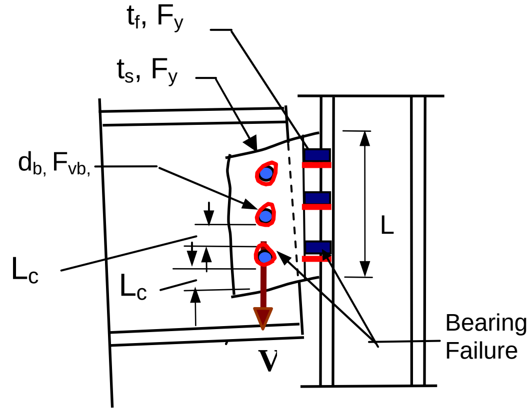


Figure 3.6. Bearing Failure Mode

3.3.c. Edge Distance Failure in the Tee or in the Beam Web (Limit State 3)

The required minimum edge distances for the tee or the beam web are equal to those given in the AISC Specifications (AISC 2005).

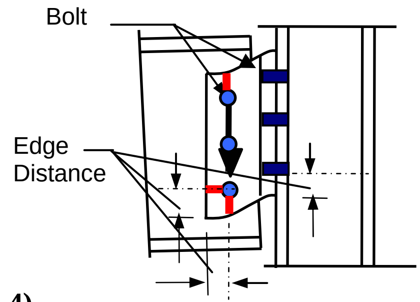


Figure 3.7

3.3.d. Net-Area Fracture of the Tee Stem or Tee Flange (Limit State 4)

The limit state of fracture of the net area should be checked against the shear yield capacity to ensure that the net section fracture strength is greater than the strength in shear yielding:

$\phi_n V_n > \phi_y V_y$ (LRFD) (3.3a)

$V_n / \Omega_n > V_y / \Omega_y$ (ASD) (3.3b)

The design shear fracture strength, $\phi_n V_n$, and the allowable shear fracture strength, V_n/Ω_n, of the tee stem or tee flange are given as:

V_y = shear yield strength of the tee established in Section 3.3.a above.

$V_n = 0.60 F_u A_{nv}$

A_{nv} = smaller of the net area in shear for the stem or flange of the tee calculated according to Section 2.3.d.

$\phi_n = 0.75$ (LRFD) $\Omega_n = 2.00$ (ASD)

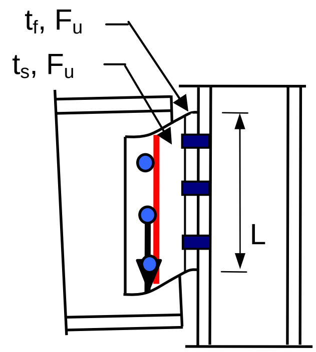


Figure 3.8

3.3.e. Fracture of Bolt Groups (Limit State 5)

a. Design of bolts on the tee flange. The bolts on the tee flange are subjected to a combination of shear and bending moment (due to eccentricity e_b.) Values of the shear and bending moment for LRFD and ASD are as follows:

$V_u = V_y$ and $M_u = V_y\, e_b$ (LRFD) (3.4a)

$V = V_y / \Omega$ and $M = (V_y / \Omega) e_b$ (ASD) (3.4b)

In order to ensure a ductile behavior, the strength of this bolt group under combined shear and bending should be greater than the strength of the tee:

$$\left(\frac{V_u}{\phi V_b}\right)^2 + \left(\frac{M_u}{\phi M_b}\right)^2 \geq 1.0 \qquad \text{(LRFD)} \qquad (3.5a)$$

$$\left(\frac{V}{V_b/\Omega}\right)^2 + \left(\frac{M}{M_b/\Omega}\right)^2 \geq 1.0 \qquad \text{(ASD)} \qquad (3.5b)$$

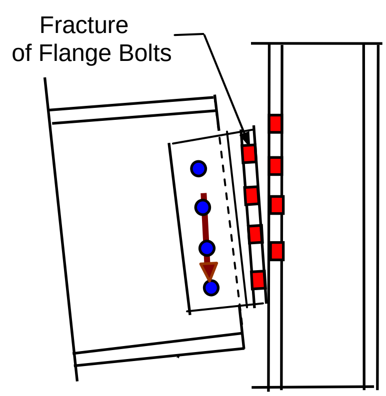


Figure 3.9. Fracture of Flange Bolts

Where,

$V_u = V_y$ and $M_u = V_y e_b$ (LRFD)

$V = V_y/\Omega$ and $M = (V_y/\Omega)e_b$ (ASD)

M_b= plastic moment capacity of the bolt group in bending given in Section 2.3.e above.

ϕ= 0.75 (LRFD) and Ω = 2.00 (ASD)

b. Design of bolts on the beam web. As mentioned earlier, the stem bolts are subjected to shear only. In order to ensure a ductile behavior, the shear strength of this bolt group should be greater than the shear yield strength of the tee:

$$\phi_b V_b > \phi_y V_y \quad \text{(LRFD)} \qquad (3.6a)$$

$$V_b/\Omega_b > V_y/\Omega_y \quad \text{(ASD)} \qquad (3.6b)$$

Where,

V_y = shear yield strength of the tee established in Section 3.3.a above.

$V_b = nA_b F_{bv}$

ϕ_b = 0.75 and ϕ_y =0.90 (LRFD)

Ω_b = 2.0 and Ω_y =1.5 (ASD)

For definitions of the terms in the above equations, see the "Notations" section on page 4.

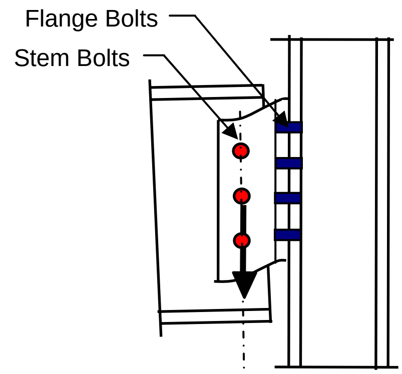


Figure 3.10. Fracture of Stem Bolts

3.3.f. Fracture of Welds (Limit State 6)

a. Design of welds on the flange of the tee. These welds are subjected to a combination of shear force and bending moment. The values of the shear force and bending moment to be combined in design according to LRFD and ASD are:

$$V_u = \phi_y V_y \quad \text{and} \quad M_u = \phi_y V_y e_w \quad \text{(LRFD)} \qquad (3.7a)$$

$$V = \phi_y V_y/\Omega_y \quad \text{and} \quad M=(\phi_y V_y/\Omega_y)e_w \quad \text{(ASD)} \qquad (3.67b)$$

Where,

V_y = shear yield strength of the tee established in Section 3.3.a above.

ϕ_y = 0.9 (LRFD) and Ω_y = 1.5 (ASD)

The eccentricity, e_w, in tee connections is equal to the distance from the weld line on the flange to the centroid of the weld line on the web. Notice that unlike double angles, in this case, due to continuity of the flange of the tee, both welds participate in bending together, resisting the bending moment, $V e_w$, as shown in Figure 3.12.

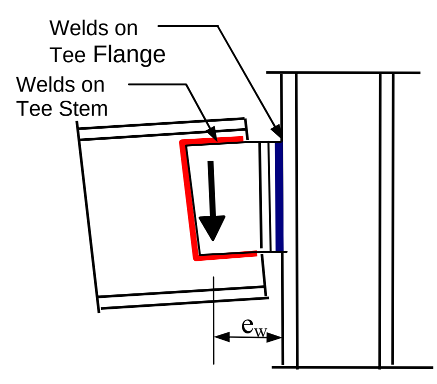


Figure 3.11

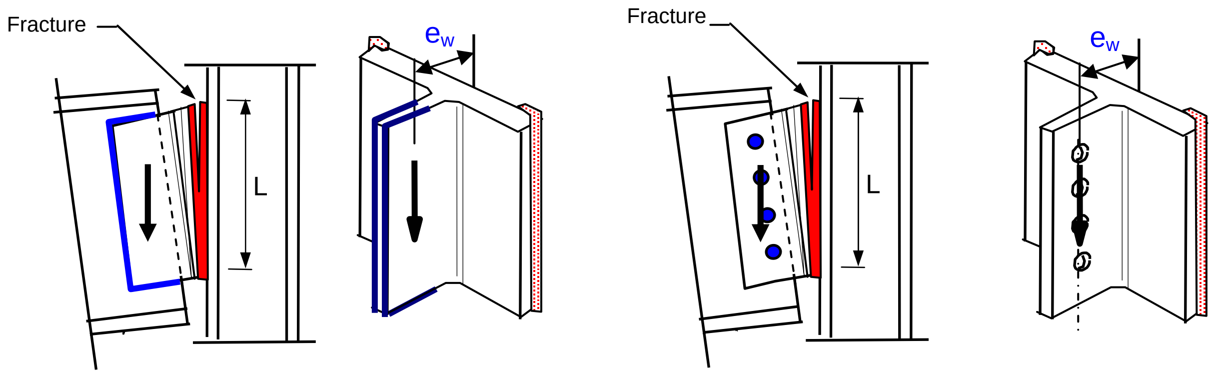


Figure 3.12. Eccentricity of Shear Force from Welds on the Support

b. Design of Welds on the Tee Stem

The welds connecting the tee stem to the beam web are subjected to pure shear acting at the centroid of C-shaped weld lines. Therefore, shear strength of these welds in LRFD and ASD should be greater than the corresponding values for shear yielding of the angles:

$\phi_w V_w > \phi_y V_y$ (LRFD) (3.8a)

$V_w / \Omega_w > V_y / \Omega_y$ (ASD) (3.8b)

Where,

$V_w = 2(\Sigma L)(0.707D)(F_{Exx})$

V_y = yield shear capacity of the tee as established in Section 3.3.a

$\phi_w = 0.75$ and $\phi_y = 0.90$ (LRFD)

$\Omega_w = 2.0$ and $\Omega_y = 1.5$ (ASD)

For definitions of the terms in the above equations, see the "Notations" section on page 4.

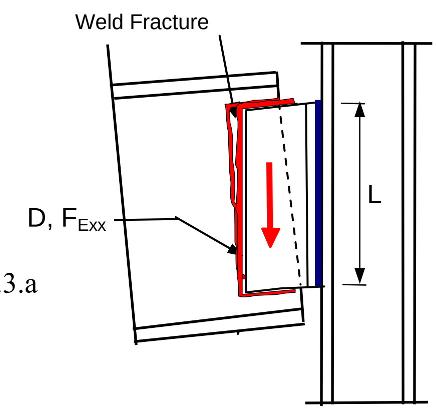


Figure 3.13. Fracture of Bolts

3.4. Seismic Issues Related to Tee Shear Connections

A literature survey did not result in information on the cyclic behavior of tee shear connections in the laboratory or during actual earthquakes. Based on behavior of this type of shear connection under mono-tonic loading, as summarized earlier in this chapter, an attempt is made here to establish expected cyclic behavior of these connections and to develop and suggest guidelines for seismic design considerations. Due to lack of actual cyclic test results, these suggestions are on the conservative side and are meant to make the connection to survive the seismic effect without major fracture so that after the earthquake the connection is able to transfer the shear due to gravity and prevent collapse of simply supported beams. It is hoped that a program of cyclic tests of these shear connections will be undertaken in the future to enable the structural engineer to conduct seismic design of these connections using more reliable information and with more confidence. Until then, the suggestions here should be treated as the opinions of the author subject to acceptance of the structural designer.

3.4.a. Expected Cyclic Behavior of Tee Shear Connections

The main role of a shear connection, before, during, and after an earthquake, is to transfer the shear force from the beam end to the supporting member. During an earthquake, in addition to their gravity shear force, shear connections primarily are subjected to cyclic moment-rotations and cyclic axial load axial deformations. Due to flexibility of shear connections when subjected to moment or axial tension, cyclic bending moments and axial tension force in the tee connection are relatively small. However, cyclic rotations and cyclic axial tension deformation of the connection can be significant. To ensure that a tee shear connection will survive an earthquake still capable of carrying its gravity shear, the connection should be strong enough to resist the combined effects of gravity shear and seismic bending and axial force as well, as it needs to have sufficient ductility to yield during earthquake and tolerate large inelastic deformations without "consequential" fracture. It should be mentioned that in major earthquakes, it is possible that minor and self-arresting cracks develop in highly restrained areas of connections. Such cracks (fractures) should not be of concern as long as: (a) the cracks have been arrested shortly after their initiation and have not propagated beyond the highly restrained areas and (b) the cracks have not resulted in reducing the shear capacity of the connections to a value smaller than the applied shear.

If shear connections are designed according to the procedures outlined in the previous section, due to inherent flexibility and ductility of these connections, it is expected that they will survive earthquakes and will be able to resist the gravity shear force after the earthquake. To achieve this desirable behavior, the role of the ductility of the connection is emphasized, and it is necessary that the ductility requirements initially proposed by Thornton (1995) and currently included in the AISC manual (AISC-LRFD 2000) be satisfied. These requirements for tee shear connections are as follows:

For tee connections where the flange is welded to the support, as shown in Figure 3.14(a), the minimum size of fillet welds done using E70 ksi electrodes is given by:

$$w_{min} = 0.0158 \frac{F_y t_f^2}{b}\left(\frac{b^2}{L^2} + 2\right) \le (3/4)t_s \qquad (3.9)$$

For tee connections where the flange is bolted to the support , as shown in Figure 3.14(b), the minimum diameter of bolts through the tee flange is given by:

$$d_{b\,min} = 0.613 t_f \sqrt{\frac{F_y}{b}\left(\frac{b^2}{L^2} + 2\right)} \le 0.69\sqrt{t_s} \qquad (3.10)$$

Additionally, for tee connections where the tee stem is bolted to the beam web, as shown in both cases of Figure 3.14, the maximum thickness of the stem is limited to:

$$t_{s\,max} = \frac{d_b}{2} + (1/16) \text{ inch} \qquad (3.11)$$

3.4.b. Design of Tee Shear Connections for Combined Shear and *Relatively Small* Axial Forces

In some applications, due to seismic or wind effects, tee shear connections are subjected to shear and axial load. The situation is very common in shear connections of "collector" beams, which collect seismic forces of the floor as axial force and transfer the axial force to the support through shear connections.

In designing tee shear connections for combined shear and axial load, the following steps are suggested.

Step 1. Establish the shear and axial force acting on the connection. Make sure that live load reductions permitted by the governing code are applied to live loads.

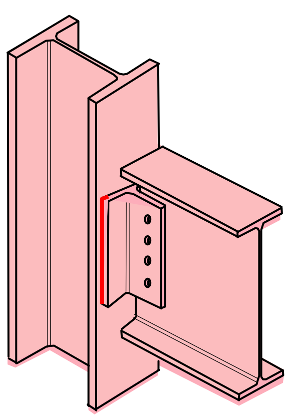
Step 2. Design the tee shear connection for applied shear only, following the procedures described in earlier sections.

Step 3. Check the tee connection for the combined effects of shear and axial load by checking failure modes of the tee stem as well as failure modes of the tee flange as discussed below.

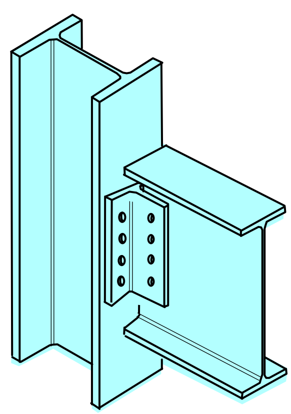
a. Checking failure modes of the tee stem subjected to combined shear and axial force. The stem of a tee acts similar to a shear tab. Therefore, it is suggested that the equation given for shear tabs in Astaneh-Asl (2005) be used for the design of the stem of a tee shear connection subjected to combined shear and axial load.

b. Checking failure modes of a tee flange subjected to combined shear and axial force. In addition to shear and axial load, a relatively small but self-limiting bending moment can also be present in tee shear connections. In the following sections, this small bending is not included in the design. There are two reasons for ignoring the bending moment in the connection. One reason is that, due to yielding of connection elements, the rotational stiffness of the connection will decrease, which in turn will result in reduction of the bending moment in the connection. The second reason for ignoring this relatively small bending moment is that the strain hardening is used to resist this relatively small and relatively short-lived moment, which occurs during the earthquakes.

The main failure modes of the tee flange subjected to combined shear and axial load are yielding of the gross area in combined shear and "tee-hanger" effects and fracture of the bolts or welds on the tee flange under the combined effects of shear and tension. These failure modes are discussed below.



(a) Welded-Bolted



(b) Bolted-Bolted

Figure 3.14. Welded-Bolted and Bolted-Bolted Tee Shear Connections

1. *Yielding of the Tee Flange under Combined Shear, Axial Load, and Bending Moment*

For this failure mode, which involves yielding of the plate under combined shear and normal stresses, the Von Mises yield criterion is used. The maximum factored axial force (in LRFD) and the maximum allowable axial force (in ASD) can be obtained from the following interaction equations:

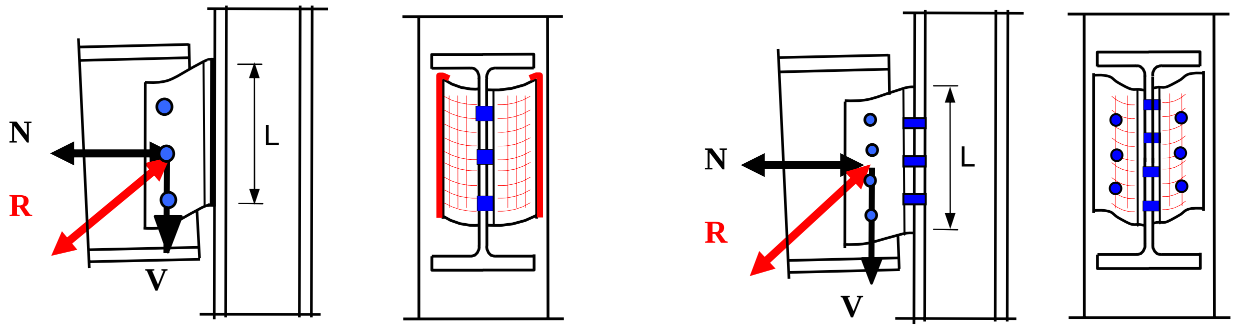


Figure 3.15. Welded-Bolted and Bolted-Bolted Tee Connection

$$\left(\frac{V_u}{\varphi_y V_n}\right)^2 + \left(\frac{N_u}{\varphi_y N_n}\right)^2 = 1.0 \qquad \text{(LRFD)} \qquad (3.12a)$$

$$\left(\frac{\Omega_y V}{V_n}\right)^2 + \left(\frac{\Omega_y N}{N_n}\right)^2 = 1.0 \qquad \text{(ASD)} \qquad (3.12b)$$

Where;
$\phi_y = 0.90$ (LRFD) and $\Omega_y = 1.67$ (ASD)
$V_n = (2L)(t_f)(0.6F_y)$ and N_n = tension capacity of tee hanger.

For definitions of the other terms in the above equations, please see the "Notations" section on page 4.

2. *Fracture of Tee Flange Bolts under Combined Shear and Axial Force*

The bolts are subjected to combined shear and axial tension. The shear force can be divided equally among the bolts. The tension force in the bolt should include the prying action effect and the additional tension that results from this effect. The equations in the AISC manuals (AISC-ASD 1989 for ASD and AISC 1999 for LRFD) for tee hangers can be used to establish the prying action.

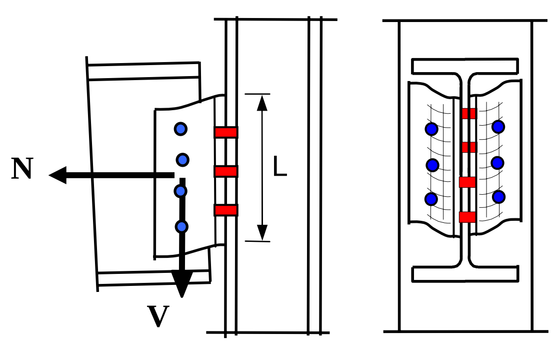


Figure 3.16

3. Fracture of Tee Flange Welds under Combined Shear and Axial Force

There is very limited information on the behavior of welded tee hangers where under applied axial tension the flange of the tee bends and results in opening the root-notch of the one-sided fillet welds. The situation does not seem to be a favorable stress condition for the flange welds. In the absence of reliable data on actual behavior of welded tees subjected to axial pull, it is suggested that if the axial force is not negligible, instead of using flange-welded tee shear connections, flange-bolted tees, Figure 3.17, be used.

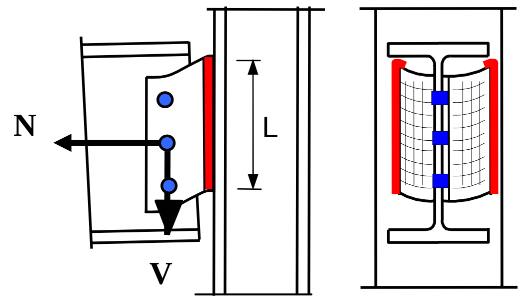


Figure 3.17

3.4.c. Design of Tee Shear Connections for Combined Shear and *Relatively Large* Axial Force

When tee shear connections are used to support a "collector beam," the axial load collected in the beam can be large. If the axial load is compression, pushing the tee against the support, the only check required will be to check the stem of the tee to ensure that the stem itself and the bolts on it are capable of resisting the combined effects of shear and axial compression. This can be done using procedures outlined in Chapter 2 for shear tabs subjected to shear and relatively large axial load. In this case, the flange of the tee will bear against the support and transfer the compression through the bearing.

However, if the axial force applied to the tee is tension force, the flange of the tee needs to be checked as a tee hanger to ensure its adequacy to transfer the axial tension force. Considering the flexibility of the flange of the tee, it is not expected that large axial tension forces can be transferred by the tee shear connection. To design a tee connection for combined effects of shear and relatively large axial load, one can first apply the procedures outlined in the previous section. If the design results in a tee connection that is not available in a standard WT section cut from a wide flange, a built-up fabricated by welding two plates can be used as shown in Figure 3.18a. Of course a reinforced standard WF section can also be used, Figure 3.18b.

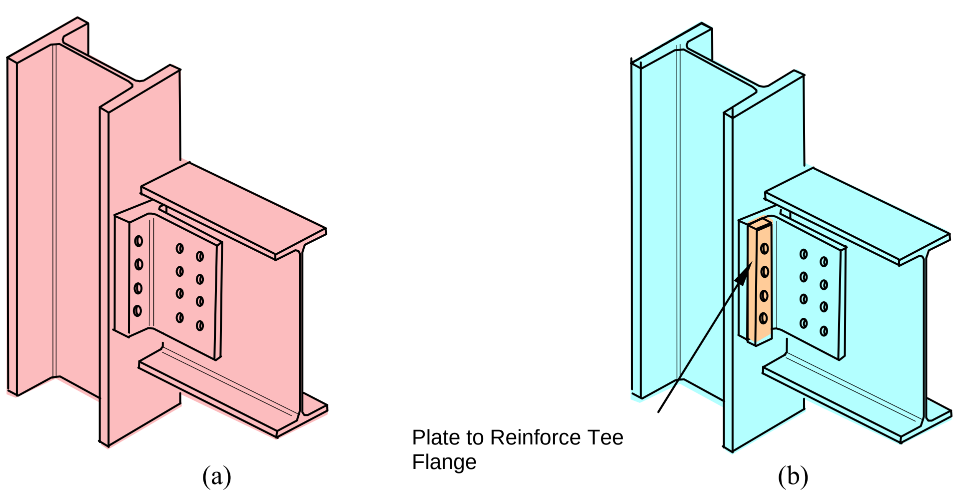
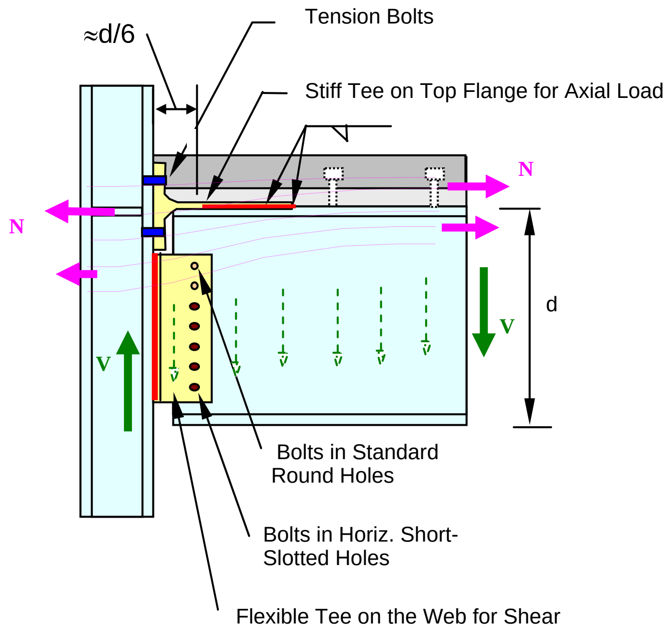


Figure 3.18. Welded-Bolted and Bolted-Bolted Tee Shear Connections

Other suggested details to transfer large axial force from a beam to a column through tee shear connections are shown in Figure 3.19. In Figure 3.19(a), a relatively stiff tee, with thick flanges, is attached to the top flange of the beam and is designed to transfer the axial force. The tee shear connection on the web of the column is designed to transfer shear and has horizontal short-slotted holes to facilitate rotation. Figure 3.19(b) shows a suggested detail where some of the axial tension in the beam is transferred by the tee on the flange of the beam and the rest by the upper bolts of the tee on the web of the beam.



Note 1: The tee can be bolted to the top flange instead of using the fillet welds shown.

(a)

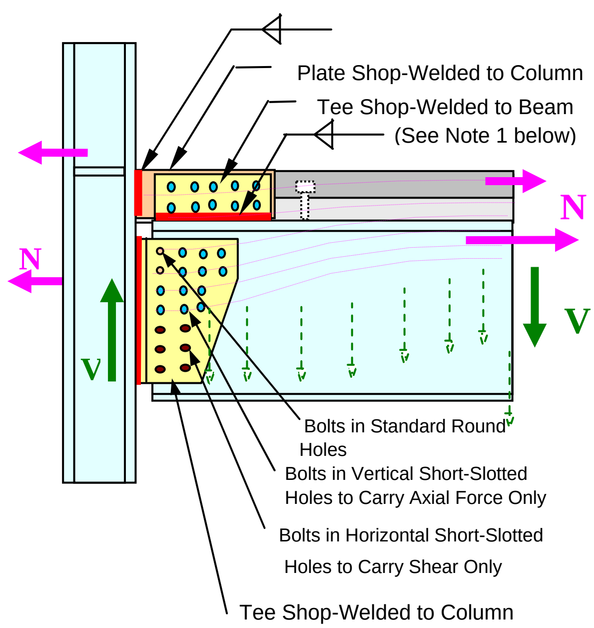


Figure 3.19. Suggested Details for Tee Shear Connections Subjected to Combined Shear and Axial Forces

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From 1968 to 1978 he was a structural engineer and construction manager in Iran designing and constructing buildings, bridges, water tanks, transmission towers, and other structures. During the period 1978–1982, he completed his M.S. and Ph.D. in structural engineering, both at the University of Michigan in Ann Arbor. Since 1982, he has been a faculty of structural engineering member involved in teaching, research, and design of both building and bridge structures, both in steel and composite, in the United States and abroad, particularly with respect to the behavior"and design of these structures under gravity combined with seismic effects. He has conducted several major projects on seismic design and retrofit of steel long-span bridges and tall buildings. Since 1995, he has also been studying behavior and design of steel structures subjected to blast and impact loads and has been involved in testing and further development of mechanisms to prevent progressive collapse of steel and composite building and bridge structures subjected to terrorist blast (car bombs) or impact (planes and rockets) attacks.

After the September 11, 2001, tragic terrorist attacks on the World Trade Center and the collapse of the towers, armed with a research grant from the National Science Foundation, he conducted a reconnaissance investigation of the collapse and collected perishable data. As an expert, he later testified before the Committee on Science of the House of Representative of the U.S. Congress on his findings regarding the collapse of the World Trade Center towers. His current research projects are on blast resistance of steel buildings and bridge structures, progressive collapse prevention in steel structures and seismic behavior and design of steel and composite buildings and bridge structures. Since 2004 he is also working on reconstruction of seismically damages areas of the Middle East.

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