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Design of Shear Tab Connections for Gravity and Seismic Loads

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This document presents information on design of steel shear tab connections. Shear tab connections consist of a plate welded to the support and bolted to the web of a simply supported beam. The main role of a shear tab is to transfer shear force from the end of the beam to its support. Chapter 1 provides a summary of behavior of shear connections under gravity and seismic effects based on the results of actual tests and observations made in the aftermath of earthquakes. Chapter 2 provides an updated summary of behavior and design procedures for shear tabs subjected only to shear due to gravity. Chapter 3 provides information on seismic behavior of shear tab connections and their seismic design. In this report, design equations are given in the "AISC Unified ASD/LRFD" format, which is the new format being adapted by the American Institute of Steel Construction (AISC) in its 2005 specifications. The unified format provides consistent equations both in Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD), and designers can choose either method (ASD or LRFD) and base their design on that method. The Appendix to the report provides numerical examples in unified format (both in ASD and LRFD) to demonstrate the application of both formats of design equations. The report includes a "Notations" section and a "References" section.

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Drawings and Photos by Abolhassan Astaneh-Asl unless otherwise indicated.

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In memory of my beloved brother Ali Astaneh-Asl (1946–2004),
a civil engineer and a dedicated public servant.
A. Astaneh-Asl

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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.

DESIGN OF SHEAR TAB CONNECTIONS FOR GRAVITY AND SEISMIC LOADS

By:

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TABLE OF CONTENTS

Notations

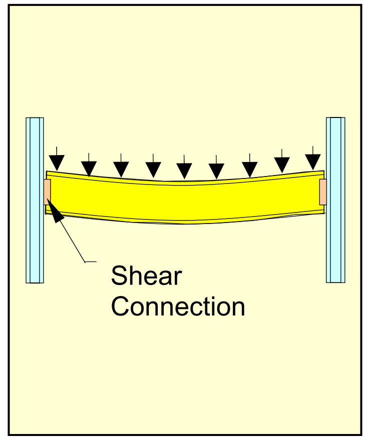
In preparing the following notations, whenever possible, the definitions are taken, with permission of the AISC, from the AISC Manual of Steel Construction (AISC 1989 and 2000) and the Seismic Provisions for Structural Steel Buildings (AISC 2002). In some cases, following the definition of the parameter, the location at which this parameter appears is given inside the parenthesis.

A_b = area of bolt
A_g = gross area
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_{tB} = area (as shown in Figure 3.20)
A_{tT} = area (as shown in Figure 3.20)
A_{vB}= area (as shown in Figure 3.20)
A_{vT} = area (as shown in Figure 3.20)
A_{vnB}= area (as shown in Figure 3.20)
A_{vnB}= area (as shown in Figure 3.20)
A_{vnT}= area (as shown in Figure 3.20)
A_{vnM} = area (as shown in Figure 3.20)
A_{vnT} = area (as shown in Figure 3.20)
a = distance from the bolt line to the weld line; depth of the compression zone
b = distance of the point of inflection of the beam from the web of the column (Figure 2.13)
b_{eff} = effective width of the floor slab
C= coefficient given in Table 8-5 of the AISC LRFD Manual (AISC, 2000)
C_1 = electrode strength coefficient from Table 8-4 of the AISC LRFD Manual (AISC, 2000),
C^- = compressive force due to moment acting on the connection
C^+ = tensile Compressive force due to moment acting on the connection
C_d = deflection magnification factor (Figure 1.10)
C_{Pr}= strain-hardening factor= $(F_y+F_u)/2F_y$
D = size of weld in 1/16 of an inch= actual size of weld in inches times 16.
D_w = actual size of weld in inches
d = nominal bolt diameter; beam depth; distance as shown in Figure 3.10
d^+ = moment arm for positive moment acting on the connection as shown in Figure 3.6
d^- = moment arm for negative moment acting on the connection as shown in Figure 3.6
d_1 = distance (Figure 3.10)
d_2 = distance (Figure 3.10)
d_b = diameter of the bolt
d_f = distance from the midheight of the shear tab to the furthest beam flange, or the largest of d_1 and d_2, (Figure 3.10)
E = modulus of elasticity.
e_b = distance of the point of inflection from the bolt line
e_p = eccentricity of the point of inflection in connections with long-slotted holes (Figure 2.4(c))
e_w = distance of the point of inflection from the weld line
e_{cw} = distance from the shear force to the column web
F = nominal shear force capacity of the bolt (kips)
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_y = specified minimum yield stress of steel

F_{ye} = expected yield stress of steel = R_y F_y
f'c = specified compressive strength of concrete
g = gap between the end of a beam and its supporting column (Figure 3.10)
H = horizontal force in extended shear tabs (Figure 2.14)
h = height of shear tab (Figure 2.13(a))
h_x = height of the x^{th} story in a building (in Equation 1.11)
I= moment of inertia
K_{Conn} = rotational stiffness of connection
L= span of a beam; length of shear tab
Lc = clear distance, in the direction of the force, between the edge of a hole and the edge of the Lc_1 = 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 (Appendix)
Lc_2 = 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 (Appendix)
M= bending moment
M_e= expected moment acting on the weld and plate = $V_e e_w$
M^+_{max} = maximum positive moment (Figure3.9)
M^-_{max} = maximum negative moment (Figure3.9)
M^+_{drop} =bending moment after failure of the slab (Figure3.9)
M^+_{slip} = positive moment causing slip in the bolts (Figure3.9)
M^-_{slip} = negative moment causing slip in the bolts (Figure3.9)
M_{Conn} = moment capacity of the connection
M_P = plastic moment
M_{Pb} = plastic moment capacity of the beam
M_{pp} = plastic moment capacity of the plate= (t)(L^2 /4)(F_y)
M_{ppe} = expected plastic moment capacity of the plate= (R_y)(C_{pr})(t)(L^2 /4)(F_y)
M_{pw} = plastic moment of the weld lines = 2 (0.707 D)(L^2 /4)(F_w)
M_u = bending moment to be considered in combination with shear in LRFD of bolts
M_y = yield moment of a beam
m= normalized stiffness term for rotational stiffness of beam-to-column connections (Figure 1.1)
N=applied axial force
N_n = nominal axial strength
N_s = axial force in the connection due to seismic effects
N_v = number of shear bolts in a shear tab being designed for seismic effects
n = total number of bolts
P = vertical load
q = uniformly distributed load per unit of length
R_{1n} = nominal strength
R_{2n} = nominal strength
R_{3n} = nominal strength
R_{br} = nominal bearing strength
R_e = a factor given in Column 4 or 7 of Table 2.1 above depending on type of bolt hole
R_n = nominal strength
R_u = required strength
R_y = ratio of the expected yield strength F_{ye} to the minimum specified yield strength F_y
r = total thickness of the floor slab
S_x = elastic section modulus of cross section (Figure 1.8)
T = height of flat portion of web in wide flanges
T^+ = tension force in the connection due to applied positive moment (Figure 3.11)
T^- = tension force in the connection due to applied negative moment (Figure 3.11)
t = thickness of the connected material
t_w = thickness of web in wide flanges
V = shear force

V_{br} = nominal bearing strength
V_e= expected shear acting on the weld and plate
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 tab
V_{up} = applied shear
V_{uw} = applied shear acting on the weld line
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 plate section= t L (0.6 F_y)
V_{yp} = shear yield capacity of plate based on Von Mises yield criterion =(t)(L) [(√3)/3]F_y ≈ (t)(L) (0.6F_y)
V_{ype} = expected shear yield capacity of plate = (R_y)(C_{pr})(t)(L)(0.6F_y)
V_{yw} = shear yield capacity of welds = (2)(0.707w)(L)(1/2)(0.6F_{Exx})
W = size of fillet weld
δ = horizontal displacement of a floor (Equation 1.10)
δ_x = horizontal displacement of story x (Equation 1.10)
δ_{xe} = horizontal displacement of story x (Equation 1.10) resulting from elastic analysis
ϕ = resistance factor for the bolt in the LRFD method = 0.75
ϕ_b = resistance factor for bolt failure 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_y = safety factor for shear yielding in the ASD method = 1.50
Ω_{br} = safety factor for bearing = 2.00 (ASD)
Ω_n = safety factor for fracture in the ASD method = 2.00
θ = angle of rotation
θ^+_{drop}= rotation when the moment has dropped to the level of the bare shear tab moment (Figure 3.9)
θ^+_{max} rotation when the maximum positive moment is reached (Figure 3.9)
θ^+_{slip} = rotation when positive moment causing a slip in the bolts is reached (Figure 3.9)
θ^+_{ult} = rotational ductility in the positive moment direction (Figure 3.9)
θ^-_{slip} = rotation when negative moment causing a slip in the bolts is reached (Figure 3.9)
θ^-_{max} = rotation when the maximum negative moment is reached (Figure 3.9)
θ^-_{ult} = rotational ductility in the negative moment direction (Figure 3.9)
θ_g =end rotation of a beam due to gravity load
θ_{gs} = end rotation due to combined gravity and seismic effects
θ_h = angle between horizontal force and the resultant
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_{total} = maximum end rotation of the beam when the end of the beam bearing against column
θ_{vs} =end rotation of a beam due to vertical seismic load
θ_y =end rotation of a beam when its midspan moment reaches the yield moment.

1. BEHAVIOR OF SHEAR CONNECTIONS UNDER GRAVITY AND SEISMIC LOADS



1.1. Introduction to Simple Shear Connections

Simple 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 goals of this report are two:

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

In steel structures, depending on rotational stiffness and flexural strength of a connection relative to those of the connected beam, the connection is categorized as FR (fully restrained) or PR (partially restrained) (AISC 1999). A third category of connections is the shear connections that are designated by the AISC Specification, Load and Resistance Factor Design (AISC 1999) as "simple framing" connections and by the AISC Manual of Steel Construction—Load and Resistance Factor Design (AISC 2000) as "simple shear" connections. In the remainder of this report, we will focus on connections in this third category and will call them shear connections. Figure 1.1 shows a more formal definition of the three categories of beam end connections. Shear connections are defined in the literature (Salmon and Johnson 1996 and Astaneh-Asl 1989) as connections with moment capacity less than 20% of the plastic moment capacity of the connected beam.

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.

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 preceding 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)..

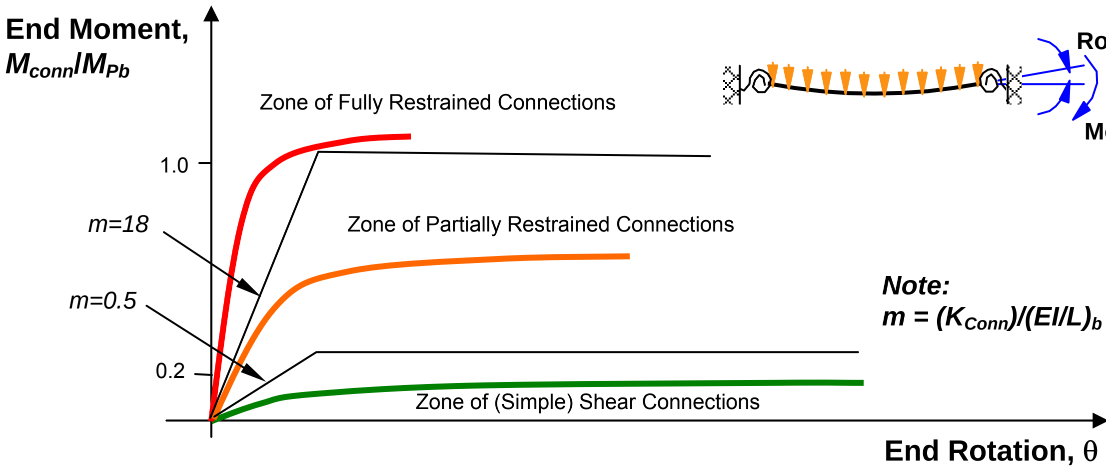


Figure 1.1. Three Types of Beam-to-Column Connections

Under the combined effects of gravity and lateral load (that is, seismic effects or wind), shear connections are subjected to shear combined with cyclic rotations and cyclic axial load. The AISC specification requirements (2) and (3) in the preceding box appear to have been formulated to ensure that the shear connections can not only resist the combined forces of gravity and lateral loads but also have adequate rotational ductility to tolerate cyclic rotations imposed on them during lateral load applications. More information on the combined effects of gravity and seismic load is provided in Chapter 3.

1.3. Types of Shear Connections

Figure 1.2 shows typical shear connections in which the beam is connected to one side of the supporting element. Quite often, shear connections are double sided, where two shear connections are connected to each side of the support. Figure 1.3 shows examples of one-sided and double sided shear tab connections.

Shear tabs, double-angle, and tee connections, shown in Figure 1.2, are three of the most frequently used shear connections in the United States. The AISC Manual of Steel Construction (AISC 2000) provides valuable information for construction and structural design of these shear connections when subjected to shear alone. Other publications, such as those listed in the “References” section of this report, also provide information on behavior and design of these three connections subjected to shear. However, in recent years, it has been recognized that in seismic regions, shear connections, in addition to shear due to gravity, can be subjected to relatively large cyclic rotations as well as cyclic axial loads. In this report, first, we consider behavior and design of shear connections subjected only to gravity shear. Then, behavior and design of shear connections under combined gravity and seismic effects are discussed. The shear connection considered in this report is a shear tab that consists of a plate welded to the support and bolted to the web of the supported beam. Behavior and design of tee and double-angle connections are discussed in Astaneh-Asl (2005).

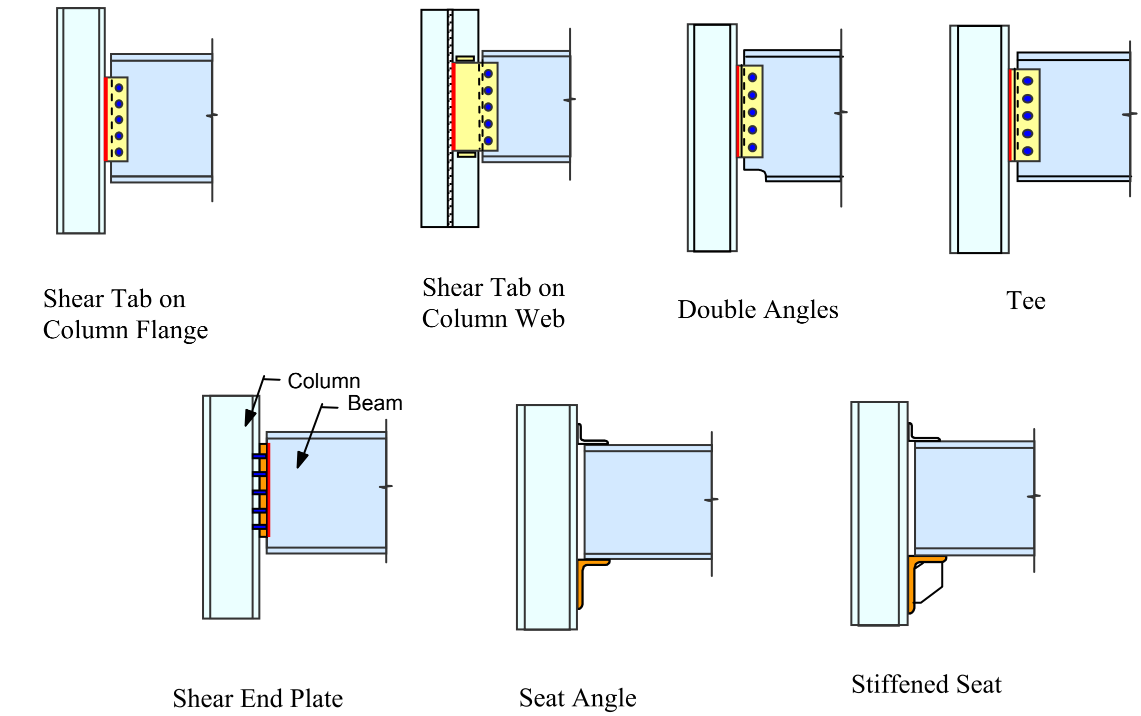
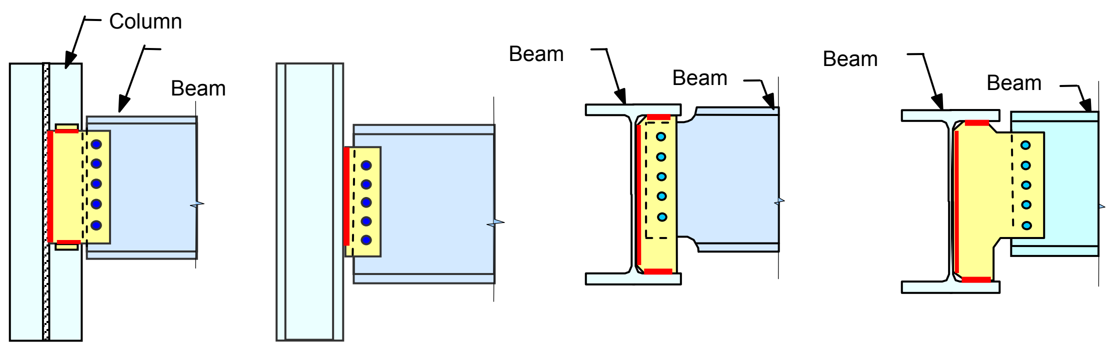
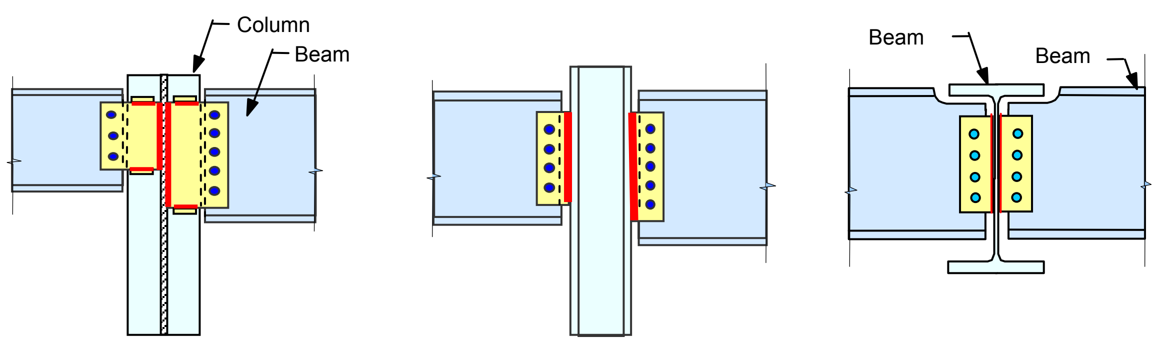


Figure 1.2. Common Types of Shear Connections



(a) One-Sided   Shear Tab Connections



(b) Double-Sided   Shear Tab Connections

Figure 1.3. Examples of one-sided and Double-Sided Shear Tab 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 end moments are larger than 20% of the plastic moment capacity of the beam, the connection would behave as a semi-rigid 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 order to satisfy the above requirements, one needs to answer the following questions:

a. What is the gravity shear force to be used in the design of a shear connection?

b. What is the bending moment to be used in the design of a shear connection?

c. What is the "sufficient" rotational ductility that a shear connection should possess?

To answer the above questions, we need to study the behavior of shear connections. Due to the material and geometric nonlinearity that are observed in almost all shear connections, such studies need to be done in laboratories and by testing of large-scale specimens subjected to realistic gravity load effects. In the next section, general behavior of shear connections, based on test results, is discussed.

1.5. General Behavior of Shear Connections of Simply Supported Beams

Depending on the location of the applied shear, connection elements, such as bolts, welds, angles and plates, will be subjected to a combination of shear and moment. As a result, the behavior and design of a shear connection strongly depend on the location of the point of inflection where the bending moment is zero. If the location of the point of inflection is known, one can calculate the moment acting on any element of the connection by simply multiplying the shear acting through the point of inflection by the eccentricity of the element under consideration from the point of inflection.

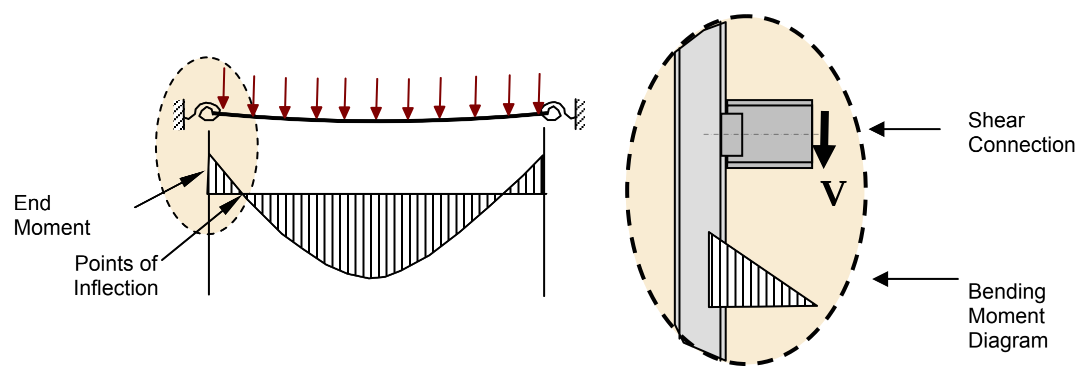


Figure 1.4. Location of the Point of Inflection and Free Body Diagram of the Connection

The location of the point of inflection in simply supported beams depends on the rotational stiffness and bending strength of its end shear connections. Consider a simply supported beam and its bending moment diagram shown together in Figure 1.5. As the applied load increases, the bending moment diagram becomes larger and the end moments increase. When the end moment exceeds the yield moment capacity of the connection, the rotational stiffness of the connection decreases due to yielding. Decrease of the rotational stiffness of the connection results in redistribution of the end moments to the midspan of the beam.

Further increase of the bending moment will not result in increasing the end moments, since the connection has reached its plastic moment capacity due to yielding of connection plates and angles. The only increase in the end moment will be due to strain hardening or kinematical hardening in the connection, which are ignored in design and in this discussion. At any time during the loading, one can find the location of the point of inflection, which is the point of zero moment, from the bending moment diagram. Notice that as the loading increases, the point of inflection moves towards the supports decreasing the fixed end moments in the beam. Another way of looking at the movement of the point of inflection towards the support is that as the load applied to a beam increases, the shear connections behave initially as fully restrained (FR), then partially restrained (PR), and finally as a flexible shear connection acting more or less as a pin connection as the connection experiences more and more yielding and loss of rotational stiffness.

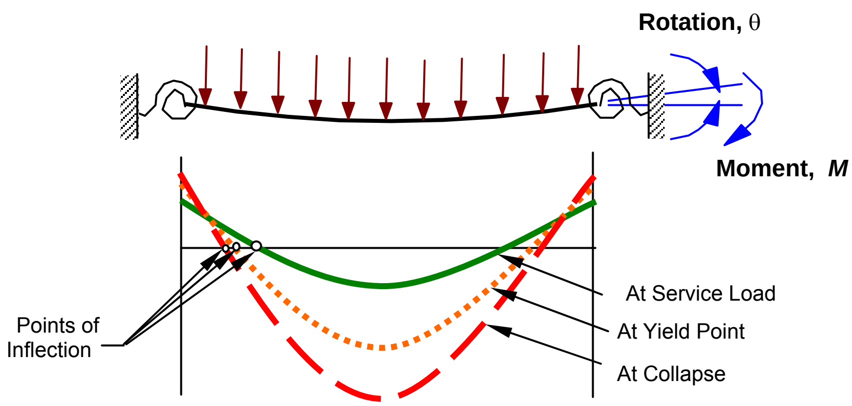


Figure 1.5. Bending Moment Diagrams for a Simply Supported Beam

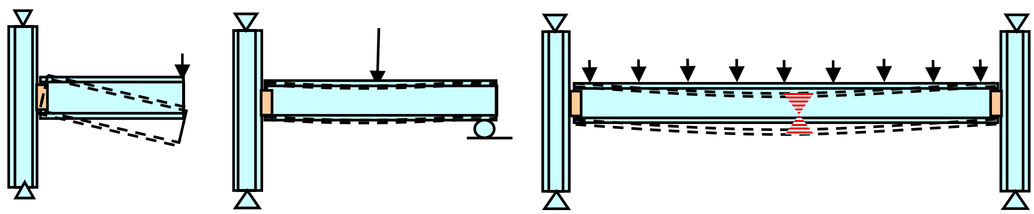
As the load on the beam increases, the shear force in the connection also increases. The increase in shear stresses, combined with bending stresses, facilitates yielding in the shear connection resulting in rotation of the connection and further movement of the point of inflection towards the support. Slippage of bolts in bolted shear connections has a similar effect in reducing 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 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. If the shear connection is not sufficiently ductile in rotation, when the plastic hinge starts forming at midspan of the beam, the end shear connections will fracture in a brittle manner due to excessive rotations imposed on them by the beam end. Such fractures, due to lack of rotational ductility, can result in loss of shear capacity of the connections, which in turn can result in failure of the connections to support the beam.

1.5.a. Actual Tests of Shear Connections

As mentioned earlier, due to the complex and nonlinear behavior of shear connections, even under service loads, design procedures for shear connections should be based on data obtained from large-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

By establishing the location of the point of inflection in a shear connection and establishing the shear-rotation behavior of the connection, we can establish all of the above three parameters. Until the1980s, tests on shear connections were done by subjecting a cantilever beam to a concentrated load at its end, Figure 1.6(a). Such tests can be considered realistic for moment connections but quite unrealistic for shear connections. The reason is that, due to the flexibility of the connection in shear connections, the cantilever beam rotates easily under a relatively small and unrealistic shear force. So, the cantilever tests provide some information on moment-rotation behavior of shear connections but not on their shear strength. A few tests in the past have been done using full span beams subjected to a concentrated load as shown in Figure 1.6(b). These tests were usually carried on until the shear connection failed under the shear force while the beam was still elastic. Although these tests shed some light on shear strength of the connection, they did not establish the realistic shear strength or the failure modes. The reason is that in these tests the connections were not subjected to realistic and relatively large rotations that a shear connection is expected to be subjected to when a plastic hinge forms at midspan of the beam.



(a) Large Rotations, Small Shear (b) Large Shear, Small Rotation (c) Realistic Condition of Shear and Rotation in the Connection

Figure 1.6. (a) Cantilever Test Specimen, (b) Elastic Beam Specimen, and (c) Realistic Specimen

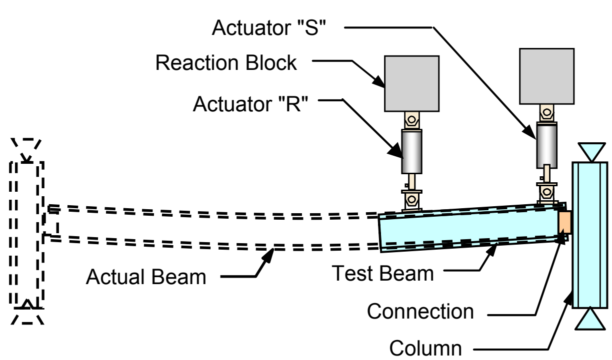


Figure 1.7. Test Setup for Realistic Testing of Shear Connections

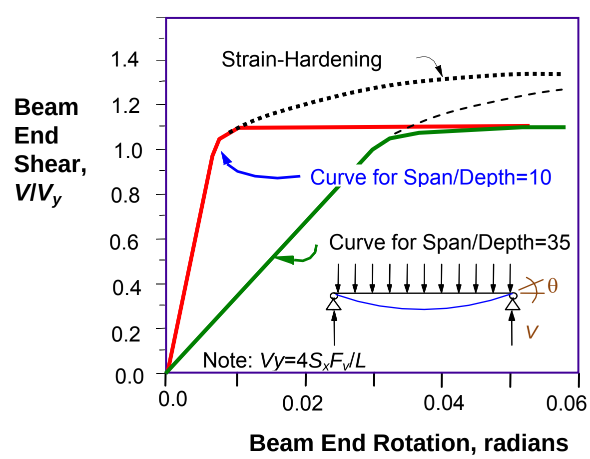


Figure 1.8. Shear-Rotation Relationship for Shear Connections

In order to establish what is the realistic relationship between shear and rotation, a series of inelastic analyses of simply supported beams subjected to uniform loading was conducted. Spans of 10, 20, 30, 40, 50, and 60 feet were used in the studies. All rolled shapes tabulated in the AISC Manual were included. In the analysis, the beams were loaded until they formed a plastic hinge in the midspan and collapsed. The established shear-rotation relationship for shear connections is shown in Figure 1.8 in terms of two extreme curves. All other curves were between these two curves. To study the behavior of shear connections under realistic conditions of shear and rotation, shown in Figure 1.6(c), a special test setup (see Figure 1.7) was developed (Astaneh-Asl 1988).

The test setup shown in Figure 1.7 was used to subject shear tabs, double angles, and tee shear connections to the realistic combination of shear and rotations shown in Figure 1.8. The test setup had two actuators, indicated as "R" and "S" in Figure 1.7, attached to a beam. 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 1.8.

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 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 the highly nonlinear behavior of the connection, the moment needs to be established by actual tests. Using the test setup shown in Figure 1.7, realistic tests of shear connections were conducted at the University of California at Berkeley (Astaneh-Asl, Call, and McMullin 1989; Astaneh-Asl, Malik, and Nader 1989; Astaneh-Asl and Nader 1989; Astaneh-Asl and McMullin 1993; and Astaneh-Asl, Liu, and McMullin 2002). 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 established failure modes and their hierarchy. The test results 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?

Earlier, in Section 1.4, three requirements for shear connections were listed. Two requirements were related to the shear force and bending moment developed in shear connections and were discussed in the previous section. The third requirement is: "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 this section, we attempt to answer the question of what is "sufficient" rotational ductility expected of a shear connection.

Let us consider a simply supported beam subjected to a uniformly distributed load. The end rotation of the beam, as long as the beam remains elastic, is given by:

$$\theta = \frac{\mathrm{q}\mathrm{L}^3}{24\mathrm{EI}} = \frac{qL^2}{8}\frac{L}{3EI} = \frac{ML}{3EI} \tag{1.1}$$

For definition of terms in the above and all other equations, please see the "Notations" section on page 4 of this report. When the midspan moment in the beam reaches the yield moment, the Equation 1.1 is still valid and can be written in terms of the yield moment capacity of the beam as:

$$\theta_y = \frac{M_y L}{3EI} \tag{1.2}$$

By substituting M_y with $F_y S_x = F_y I/(d/2)$ in Equation 1.2, one obtains:

$$\theta_y = \frac{2F_y}{3E}\left(\frac{L}{d}\right) \tag{1.3}$$

After the midspan moment reaches the yield moment, increase in the load will result in yielding of more and more fibers at midspan and more end rotations in the beam. Beyond the yield point, due to yielding of steel, Equation 1.1 no longer can predict the end rotations accurately. The inelastic analysis, mentioned earlier, the results of which are plotted in Figure 1.8, indicated that the end rotation of a simply supported beam when its midspan moment reaches plastic moment can be obtained from the following approximate but somewhat conservative equation:

$$\theta_p = \frac{M_{pb} L}{2.5EI} \tag{1.4}$$

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

$$\theta_g = \frac{F_y L}{Ed} \tag{1.5}$$

Equation 1.5 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:

$$\theta_g \cong 0.03 \text{ Radians} \tag{1.6}$$

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. Since shear connections are also responsible for transferring gravity forces during and after an earthquake, seismic design of shear connections should be done such that the damage to the connection during an earthquake does not diminish its capacity to resist the gravity forces after the earthquake.

A rational seismic design philosophy for shear connections should be based not only on providing sufficient strength in the connection to resist expected moment, shear, and axial forces due to combined gravity and seismic effects, but also on ensuring that the connection has sufficient ductility in rotational, shear, and axial deformation to survive earthquakes. An approach to ensure that this happens is to design the connection such that the yielding of the most ductile element(s) of the connection is the governing failure mode. This philosophy was proposed and used in developing design procedures for the shear tab connections subjected to the gravity load (Astaneh-Asl, Call, and McMullin 1989). The proposed philosophy and design procedures are the basis of the current procedure for the design of shear tabs in the AISC LRFD and ASD manuals (AISC 2000 and AISC 1989). Typically, in most shear connections, steel plates, angles, or tees are the most ductile elements of the connection, and yielding of these elements should be the governing failure mode to achieve a sufficiently ductile connection. This philosophy is also followed throughout this document in developing seismic design procedures for shear connections.

1.6.a. Seismic Forces in a Shear Connection

During an 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 equal sign bending moments, one at each end of the beam as shown in Figure 1.9. Therefore, the total shear force in the connection under combined gravity and seismic effects is:

$$V_{gs} = V_g + V_s = \frac{qL}{2} + \frac{2M_{Conn}}{L} \qquad (1.7)$$

1.6.b. Seismic Rotations in Shear Connections

Figure 1.10 shows the rotations that develop in a shear connection under gravity load alone and under combined gravity and seismic loads. In Figure 1.10, 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 the 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 1.10(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 the earthquake.

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

$$\theta_{gs} = \theta_g + \theta_s \qquad (1.8)$$

In the above equation, θ_g is the maximum value of the rotation demand on the shear connection under the gravity load alone, as shown in Figure1.10, and was established earlier by Equation 1.5. Rotation angle θ_s is the maximum rotation imposed on the connection by seismic forces and is equal to summation of angles θ_{hs} and θ_{vs} in Figure 1.10:

$$\theta_s = \theta_{hs} + \theta_{vs} \qquad (1.9)$$

Angle θ_{hs} in the above equation is equal to the inelastic story drift angle. Current seismic design codes provide simple equations to estimate inelastic story drift in terms of elastic drift values. For example, the ASCE-7 Standard (ASCE 2002) provides the following equation for story drift for story x:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad (1.10)$$

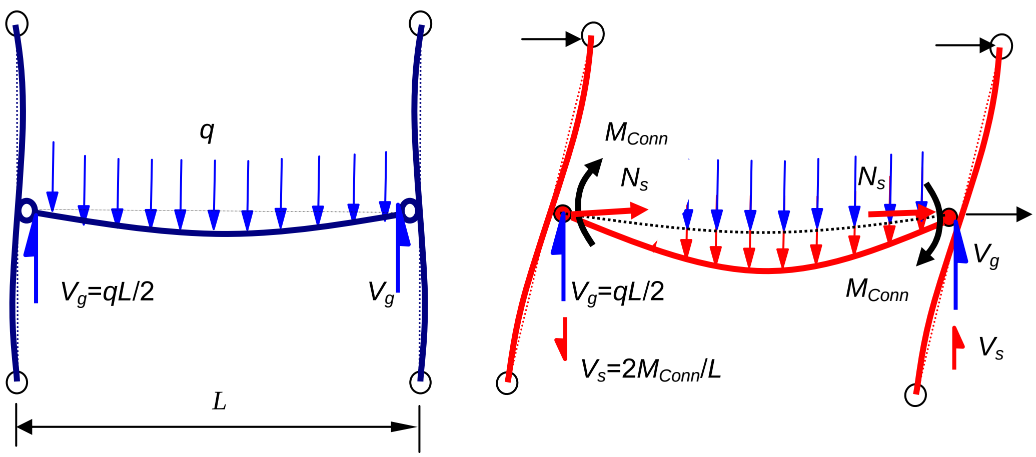
Angle θ_{hs} is equal to δ_x divided by the story height, h_x, resulting in:

$$\theta_{hs} = \frac{C_d \delta_{xe}}{h_x I} \qquad (1.11)$$

In the above equation, C_d is the deflection magnification factor with a value of 4 to 6.5 for steel structures, δ_{xe} is the elastic story drift, h_x is the story height, and I is the importance factor for the building. In the absence of values for δ_{xe}, or if one wants to use only one value of θ_{hs} for all shear connections in the building, a value of 0.04 radians seems to be a reasonable maximum drift angle in buildings with moment frames. For braced frames an average of 0.025 radians is suggested.

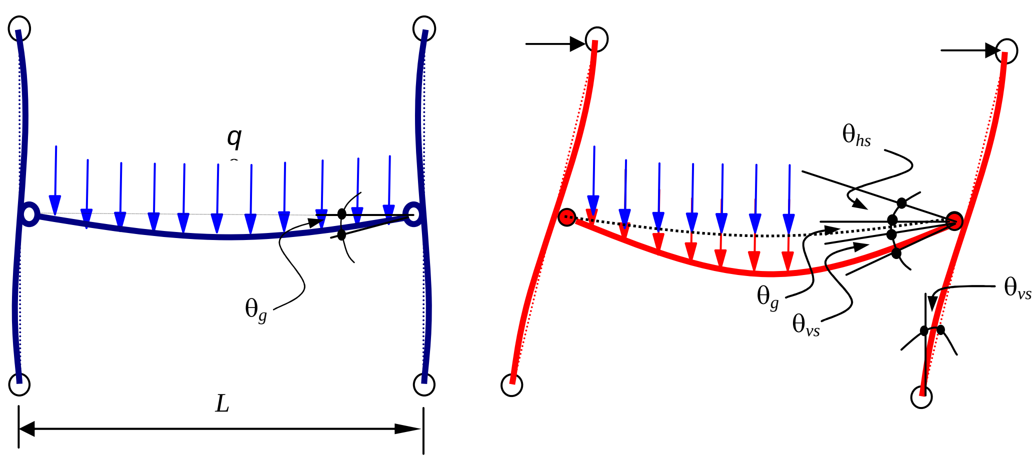
Angle θ_{vs} in Equation 1.9 is the rotation of the connection due to the vertical component of the seismic forces acting on the beam as shown in Figure 1.10(b). Current codes provide procedures to calculate the vertical component of seismic force. After establishing the vertical seismic force acting on the simply supported beam, one can calculate the end rotations θ_{vs} due to vertical seismic loads. In the absence of rigorous analysis to establish θ_{vs}, or if one wants to use a single value of γ for all shear connections, a value equal to 30% of the end rotation due to dead load only seems to be a reasonable value. This assumes that in most cases of design, the vertical seismic forces acting on the simply supported beams are less than 30% of the dead load.

Therefore, an approximate value of the maximum rotation demand on shear connections during earthquakes can be estimated from Equation 1.8 given earlier.



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

Figure 1.9. Shear and Axial Forces in the Shear Connections



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

Figure 1.10. Rotation in the Connections

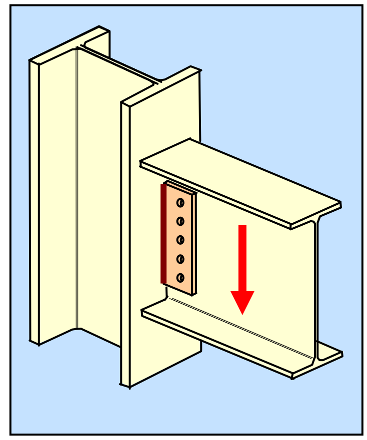
In Equation 1.8, θ_g is the maximum value of rotation demand on the shear connection under gravity load alone and was established earlier by Equation 1.5. Angle θ_s is the maximum rotation due to seismic effects given approximately as:

$$\theta_s = 0.04 + 0.3(0.04) = 0.05 \text{ Radians} \quad (1.12)$$

The total rotation due to combined gravity and seismic effects is approximately:

$$\theta_{gs} = 0.05 + 0.03 = 0.08 \text{ Radians} \quad (1.13)$$

2. DESIGN OF SHEAR TABS FOR GRAVITY LOAD



2.1. Introduction

The behavior of shear tabs subjected to gravity shear has been studied by Astaneh-Asl (1989), Porter and Astaneh-Asl (1990), and Shaw and Astaneh-Asl (1992) at the University of California, Berkeley. Based on this and previous on shear tabs (Richard et al. 1980), new and performance-based design procedures were developed and proposed by Astaneh-Asl, Call, and McMullin (1989), which were then included in the AISC ASD and LRFD Manuals of Steel Construction (AISC 1989 and AISC 2000). The details of these studies and their results can be found in the above-mentioned references. Below, a summary of the behavior of shear tab connections subjected to vertical shear force due to gravity is provided followed by a summary of design procedures for shear tab connections. Later, in Chapter 3, behavior and design of shear tabs under seismic effects are discussed.

2.2. Gravity Load Effects on Shear Tabs

Under gravity load, a shear tab is subjected to shear force (that is, reaction of the beam), a relatively small moment, which is usually less than 20% of the plastic moment capacity of the beam, and relatively large rotations on the order of 0.03–0.05 radians. As discussed in Chapter 1, to establish the moment acting on any shear connection, the first step is to establish the location of the point of inflection.

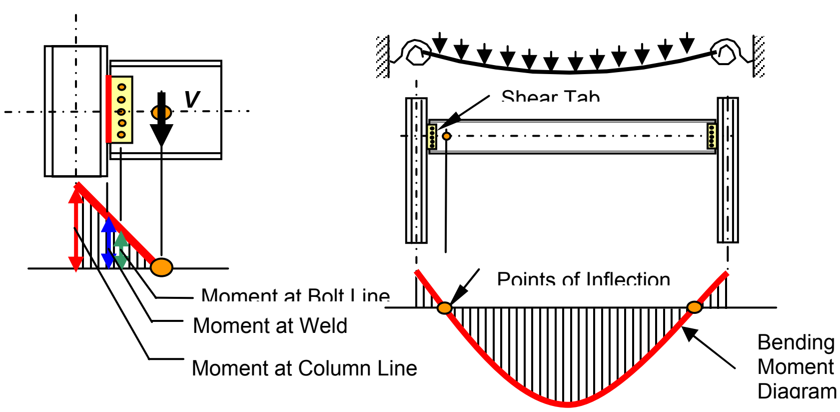


Figure 2.1. Point of Inflection and Bending Moments Acting on Connection Elements

The point of inflection is the point along the beam centerline where bending moment is zero. After establishing the location of the point of inflection, the free body diagram of the connection and a segment of the beam up to the point of inflection can be used to calculate the shear force and bending moment acting on the connection and its components such as the plate, bolts, and welds as shown in Figure 2.1.

Tests of shear tabs under gravity load effects have established the location of the point of inflection for single row shear tabs (Astaneh-Asl, Call, and McMullin 1989). The location of the point of inflection primarily depends on the depth of the shear tab, the amount of slippage in the bolts, and the rotational stiffness of the supporting member. If the supporting member is relatively rigid, such as a flange of a column or a plate embedded in a reinforced concrete wall, the distance of the point of inflection from the centerline of the support and the moments generated in the connection are larger than the corresponding values for a case in which the support is relatively flexible. Examples of flexible support cases are shear tabs connected to one side of a column web or a girder web. If equal size shear tabs are connected to both sides of a column web or girder web, the support will act as a rigid support. Examples of rigid and flexible supports for shear tabs are shown in Figure 2.2. In a conservative approach, one can ignore the extra shear capacity of shear tabs with flexible supports and design all shear tab connections using eccentricities associated with rigid supports. This approach is used in the remainder of this report.

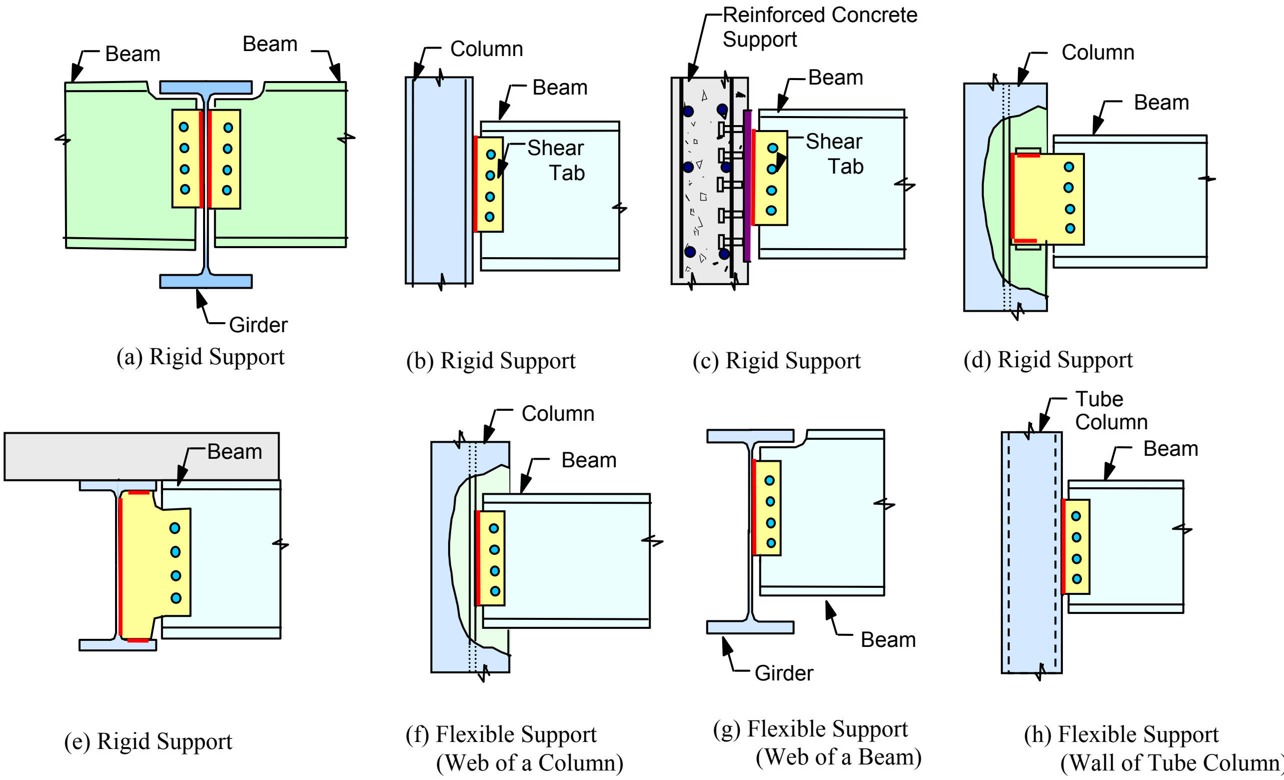


Figure 2.2. Rigid and Flexible Supports for Shear Tab Design

2.3. Location of the Point of Inflection in Shear Tab Connections

The research conducted at UC-Berkeley (Astaneh-Asl, Call, and McMullin 1989; Astaneh-Asl, Malik, and Nader 1989) 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.

2.3.a. Location of the Point of Inflection for Shear Tabs with Standard Holes

When standard holes are used in shear tabs as well as in beams, depending on the condition of support being rigid or flexible, the location of the points of inflection were established in terms of the distance (or eccentricity) from the weld line or bolt line of the connection as given below. It should be noted that the equations for the location of the point of inflection given below are for both the LRFD and ASD methods. As mentioned above, in a slightly conservative approach, one can use eccentricities given below for rigid supports for both flexible and rigid supports.

(a) The distance from the point of inflection to the weld line was established as:

e_w = n inches (for standard holes) (2.1)

(b) The distance from the point of inflection to the bolt line was established as:

e_b = (n – 1) inches – a ≥ a (for standard holes) (2.2)

Where:
e_w = distance from the weld line to the point of inflection
e_b = distance from the bolt line to the point of inflection
a = distance from the center of the bolts to the weld line
n = number of bolts

Figure 2.3 shows the eccentricities established by the tests (as dots) plotted against the eccentricities calculated by the above equations. Note that in these tests shear tabs were connected to column flanges of wide flange columns making the connection “rigid support,” shown in Figure 2.2(b).

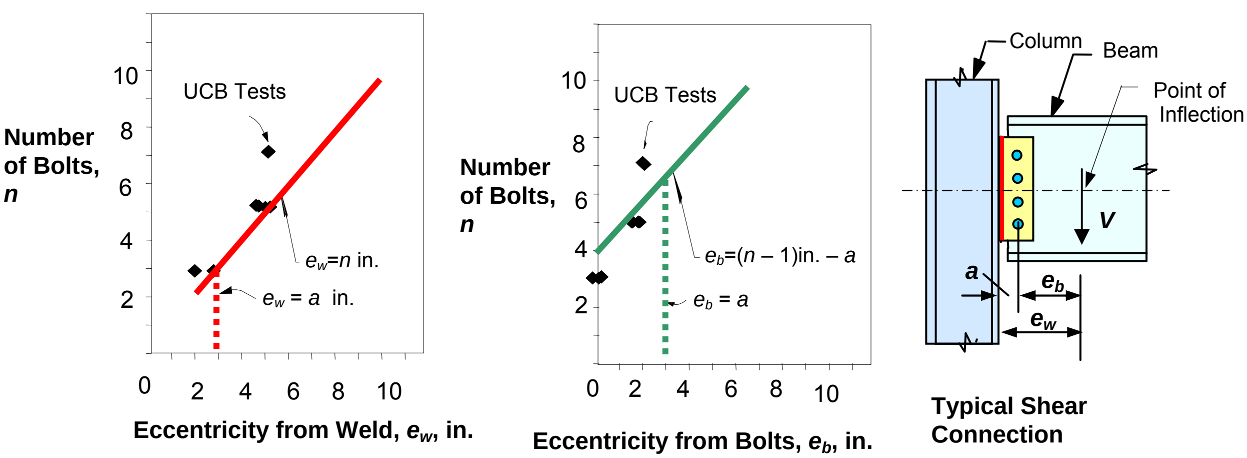


Figure 2.3. Eccentricity of the Point of Inflection from Weld and Bolt Lines

The tests of shear tabs connected to rigid supports also revealed that:

1. Single-plate shear connections (shear tabs) supported gravity load at maximum rotations varying from 0.026 to 0.103 radians. The maximum rotation achieved increased as the number of bolts in the connection decreased.

2. Shear deformation and distortion contributed significantly to the behavior of the connection, especially when the applied vertical load exceeded the service load level.
3. Moment capacity of connections varied from 35% to 50% of plastic moment capacity of the gross area of the plate.

2.3.b. Location of the Point of Inflection for Shear Tabs with Short-Slotted Holes

In many applications, to facilitate fitting of the bolts inside the holes, it is desirable to use short- slotted holes in the shear tab and standard holes in the beam web. Using slotted holes makes the shear tab connection more flexible than with standard holes since the bolt can slide more inside the slot and permit beam end rotation more easily. Because of the higher flexibility of shear tab connections with slotted holes, the point of inflection in these connections is located closer to the weld line than with connections that use standard round holes as discussed in section 2.3.a.

Porter and Astaneh-Asl (1990) conducted a series of tests of shear tabs with short-slotted bolt holes in the shear tabs and standard round holes in the beam web. The equations that were suggested by Porter and Astaneh-Asl (1990) for the eccentricity of the point of inflection were:

(a) The distance from the point of inflection to the weld line was established as:

$e_w = 2n/3$ inches (for short-slotted holes in shear tab) (2.3)

(b) The distance from the point of inflection to the bolt line was established as:

$e_b = (2n/3-1)$ inches $- a \geq a$ (for short-slotted holes in shear tab) (2.4)

2.3.c. Location of the Point of Inflection for Shear Tabs with Long-Slotted Holes

In some applications, such as expansion joints, long-slotted holes are used in shear tabs as shown in Figure 2.4(a). In these cases, the length of the slot can provide sufficient freedom for bolts to move horizontally and accommodate end rotation of the beam with almost no restraint. As a result, the point of inflection moves to the bolt line. Since there are no data and results of tests of shear tabs with long-slotted holes, to be conservative, it is suggested that the location of the point of inflection be considered as far away from the weld line as possible. This means that for the purpose of design, the bolts are to be considered at the far end of the long slot as shown in Figure 2.4(b). In addition, it is also suggested that long-slotted shear tabs be used only on rigid supports, such as column flanges, and not on flexible supports such as webs. This is to avoid cyclic deformation of flexible supports as the bolts travel in the long-slotted holes as the beam expands and contracts due to temperature changes.

Therefore, the distance from the point of inflection to the bolt line and weld line in a shear tab connection with long-slotted bolt holes is given as:

$e_b = 0.0$

e_w = Distance from the center of the bolts to the weld line when the bolts are at the end of the slot

For shear tabs with long-slotted holes, in addition to the six failure modes discussed in section 2.4, it may be necessary to check a seventh failure mode, that is, the bending failure of the narrow bands of plate between the long-slotted holes as shown in Figure 2.4(c). To check this failure mode, the applied moment is defined as shear yield capacity of the plate multiplied by the eccentricity e_p shown in Figure 2.4(c). The resisting moment is the summation of bending moment capacities of the narrow strips of plate between the holes shown in Figure 2.4(d).

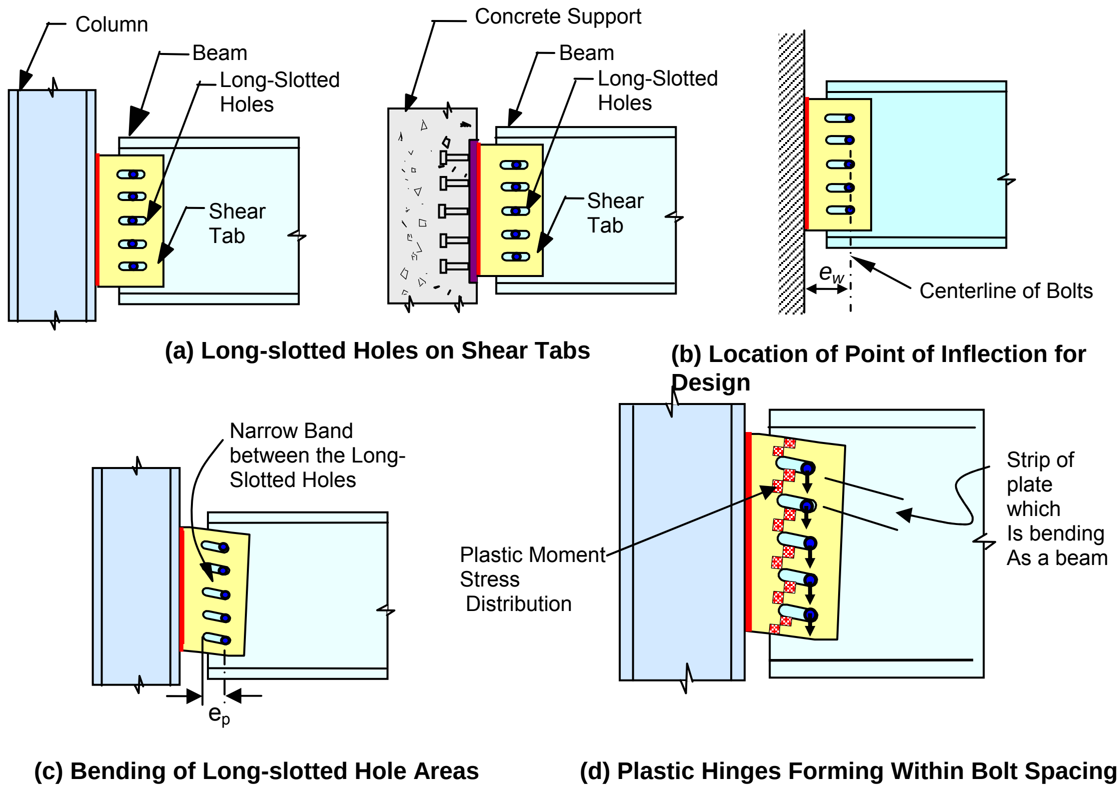


Figure 2.4. Shear Tabs with Long-Slotted Holes

2.3.d. Location of the Point of Inflection for Shear Tabs within a Moment Connection

The above discussions regarding the location of the point of inflection apply only to shear tabs in simple supports. Shear tabs are used in many bolted and welded moment connections to carry shear. For shear tabs in moment connections, one can conservatively use the above locations of the point of inflection and design the shear tab elements accordingly or simply select the shear tab from the AISC Manual tables (AISC 1989 and AISC 2000). The tables are based on using the above eccentricities. However, in reality, the bulk of the bending moment in the connection will be resisted by the flanges, and the shear tabs will be subjected only to shear and a relatively small bending moment. This may be the reason some structural engineers design the moment connection shear tabs and the bolts and welds on the shear tab for pure shear. It must be added that seismic design of shear tab connections that are part of special moment resisting frame, should be done in accordance with FEMA-350 (2000).

2.4. Failure Modes of a Shear Tab Connection

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

1. Yielding of gross area of plate (very ductile)
2. Bearing yielding of bolt holes in the plate and/or beam web (ductile)
3. Failure of edge distance of bolts (somewhat ductile until fractures)
4. Shear fracture of net area of plate (somewhat ductile until fractures)

5. Fracture of bolts (limited ductility)
6. Fracture of welds (limited ductility)

In the above list, failure modes are divided into two categories of "ductile" and "brittle" and have been placed in the order of their desirability with the first one being the most desirable and the last one being the least desirable. Figure 2.5 graphically shows the same failure modes and their hierarchy. Failure modes (1) and (2) in the above list are associated with yielding of steel and are considered ductile failure modes. Ductile failure modes are more desirable than the more brittle failure (that is, failure modes 3 through 6 in the above list). 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 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 2.5 in the hierarchy of failure modes. However, slippage of bolts is not a failure mode as long as the slippage 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 (un-factored) design loads.

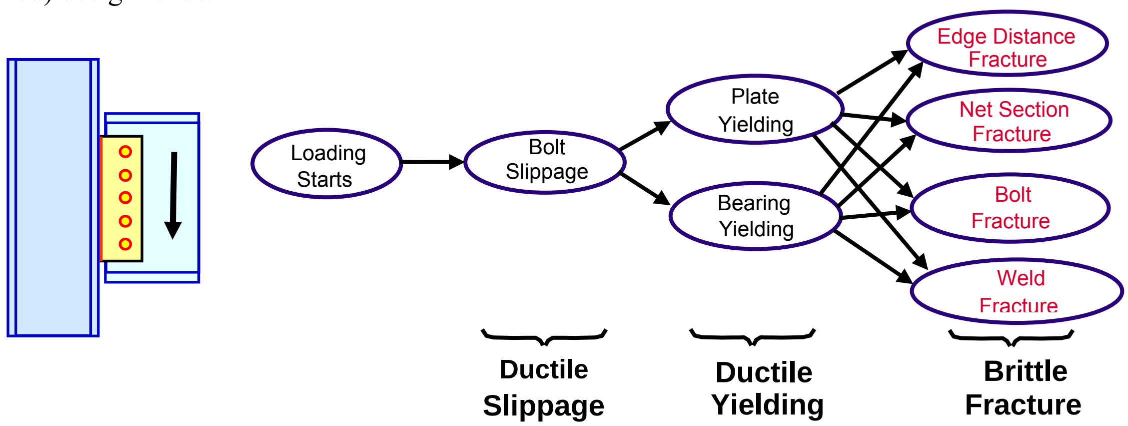


Figure 2.5. Limit States (Failure Modes) of Shear Tab Connections

2.4.a. Yielding of the Plate in Shear (Limit State 1)

As discussed earlier, for shear tab connections, it is essential that this failure mode is the governing failure mode resulting in inelastic deformation of yielded plate to contribute to the rotational ductility of the connection. To ensure that yielding of the shear tab plate governs over other failure modes, design of a shear tab starts with this limit state and to ensure that this limit state governs over other limit states, the factored shear force (in LRFD) and the applied shear force (in ASD), established by analysis, should be less than or equal to the values of design shear yield strength in LRFD and ASD:

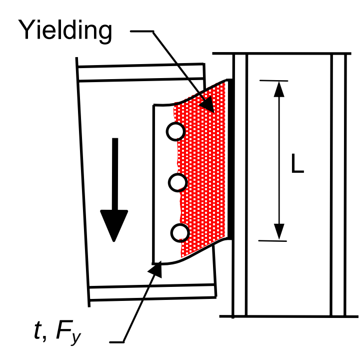


Figure 2.6

$V_u \le \phi_y V_y$ (LRFD) (2.5a)

$V \le V_y / \Omega_y$ (ASD) (2.5b)

For a shear tab, the design shear yielding strength in LRFD and the allowable shear yielding strength in ASD are $\phi_y V_y$ and V_y/Ω_y respectively where:

$V_y = 0.60F_y\, t\, L$ (2.6)

$\phi_y = 0.90$ (LRFD) (2.7a)

$\Omega_y = 1.5$ (ASD) (2.7b)

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

2.4.b. Bearing Failure of the Shear Tab or Beam Web (Limit State 2)

After designing the shear tab plate following the procedures in previous section, the remaining failure modes, such as this failure mode, are checked to ensure that they have a strength equal or greater than the shear yield strength of the plate. This “capacity design” approach will ensure that the yielding of the shear tab plate is the governing failure mechanism. Therefore, 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:

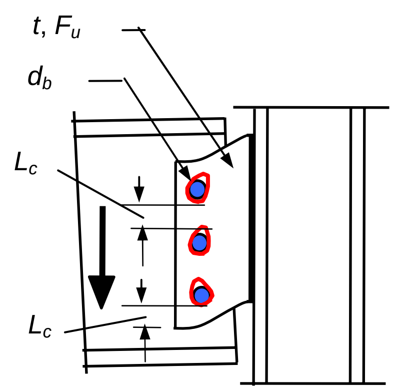


Figure 2.7

$\phi_{br}\, V_{br} > \phi_y\, V_y$ (LRFD) (2.8a)

$V_{br}/\Omega_{br} > V_y / \Omega_y$ (ASD) (2.8b)

For a shear tab, the design bearing failure strength in LRFD and allowable bearing failure strength in ASD are $\phi_{br}\, V_{br}$ and V_{br}/Ω_{br} respectively where:

$V_{br}=\Sigma(1.2L_c tF_u \leq 2.4d_b\, tF_u)$ (2.9)

$\phi_{br} = 0.75$ (LRFD) (2.10a)

$\Omega_{br} = 2.00$ (ASD) (2.10b)

The term $1.2L_c tF_u$ in Equation 2.9 is the bearing capacity of one bolt using its corresponding L_c, where L_c is the distance from the edge of the bolt hole to the edge of the plate or to the edge of the adjacent bolt hole, whichever is smaller. For definitions of other terms in the above equations, please see the “Notations” section on Page 4.

2.4.c. Edge Distance Failure in the Plate or in the Beam Web (Limit State 3)

Edge distance for a bolt is defined as the distance from center of the bolt to the edge of a connected part. The required minimum edge distances for the shear tab as well as for the beam web in a shear tab connection should be equal to or greater than those provided by the AISC-LRFD Specifications (AISC, 1999) or two times diameter of bolt whichever is greater. The minimum edge distance of two times diameter of bolt resulted from the tests of the shear tabs conducted at the University of California, Berkeley (Astaneh-Asl, Call, and McMullin 1989). The tests has since formed the basis of procedures currently in the AISC Manuals (AISC 1989 and AISC 2000).

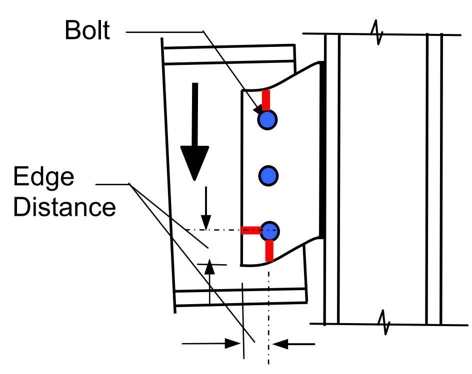


Figure 2.8

2.4.d. Net Area Fracture of the Plate (Limit State 4)

For a shear tab, the limit state of the 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) (2.11a)
$V_n / \Omega_n > V_y / \Omega_y$ (ASD) (2.11b)

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

$V_n = 0.60 F_u A_{nv}$ (2.12)
$\phi_n = 0.75$ (LRFD) (2.13a)
$\Omega_n = 2.00$ (ASD) (2.13b)

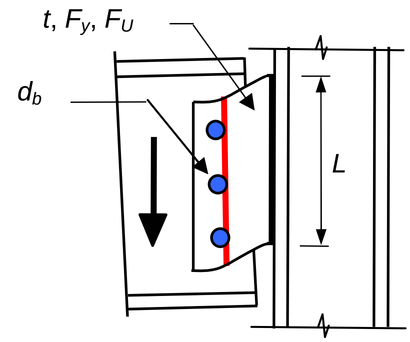
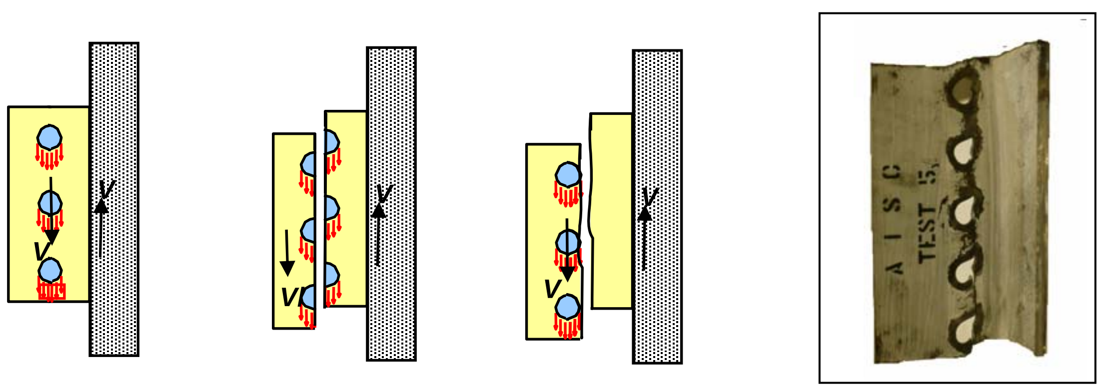


Figure 2.9

For definitions of the terms in the above equation, please see the "Notations" section on Page 4. The term A_{nv} in Equation 2.12 is the "net section for shear." Currently, the AISC specifications (AISC 1989 and AISC 1999) define the net area in shear to be the area along the centerline of the bolts as shown in Figure 2.10(b). However, considering the shear stress distribution in Figure 2.10(b), it is not possible for the shear tab to fracture in shear along the centerline of the bolts unless the bolts also fracture in half. The tests that were done by Astaneh-Asl, Call, and McMullin (1989) and Astaneh-Asl, Malik, and Nader (1989) indicated that the fracture of the net section in the shear occurs through a section at the edge of the bolt as shown in Figure 2.10(c) and not through the centerline of the bolts. Observing this failure mode, Astaneh-Asl, Call, and McMullin (1989) and Astaneh-Asl, Malik, and Nader (1989) recommended that the net section in the 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 the net section in the shear was:

$$A_{nv} = tL - 0.5n(d_b + 1/8 \text{ inch})(t) \quad (2.14)$$

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



(a) Shear Tab Subjected to Shear (b) AISC Model Net Area Fracture (c) Observed Net Area Fracture in Tests (d) Example of Net Section Fracture in a Tee Shear Connection (Nader and Astaneh-Asl, 1990)

Figure 2.10. Bolt Group Subjected to Shear and Fracture of Net Area

The above equation is similar to the equation used to calculate the net section subjected to tension but has a factor of 0.5 in front of the hole area term. In early 1980s when the American Institute of Steel Construction (AISC) was adopting the recommendations made by Astaneh-Asl, Call, and McMullin. (1989) for design of shear tabs, in a conservative move the AISC chose to use the equation of tension net area for shear net area. Although the conservative approach of the AISC does not affect the design outcome for A36 steel in most cases, it can result in preventing the use of steel with a higher yield point, such as 50 ksi, in shear tab connections. Therefore, the author suggests that this issue be revisited by the AISC and, after approval, that the net area for shear be established using the above Equation 2.13. Until then, the equation given in the AISC specification (AISC 1999) for calculation of net area, as given below, should be used in design.

$$A_{nv} = tL - n(d_b + 1/8 \text{ inch})(t) \qquad (2.15)$$

2.4.e. Fracture of the Bolt Group (Limit State 5)

The bolts in a shear tab connection should be designed for a shear force eccentrically applied to the bolt group as shown in Figure 2.12. The shear force to be used in design of bolt group on the shear tab is shear yield strength of the plate given in LRFD and ASD formats is:

$$V_u = \phi_y V_y \qquad \text{(LRFD)} \qquad (2.16a)$$
$$V = V_y / \Omega \qquad \text{(ASD)} \qquad (2.16b)$$

Where;

$$\phi_y = 0.90 \qquad \text{(LRFD)} \qquad (2.17a)$$
$$\Omega_n = 2.00 \qquad \text{(ASD)} \qquad (2.17b)$$

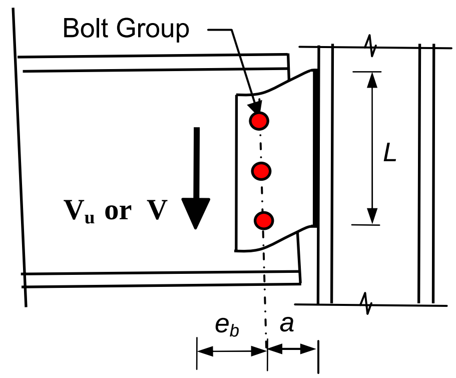


Figure 2.11

Using the value of $\phi_y V_y$ in Equation 2.16 above as the shear force applied to the bolt group will ensure that the strength of bolt group is equal or greater than the shear yield strength of the plate making the ductile and desirable shear yielding failure mode the governing failure mode of the connection

The eccentricity of shear force, e_b, is the distance from the location of point of inflection of the beam to the bolt line, Figure 2.11, and was given earlier by Equations 2.2 and 2.4 for shear tab connections with standard and short-slotted holes respectively. The equations are repeated here for convenience:

$$e_b = (n - 1) \text{ inches} - a \geq a \qquad \text{(for standard holes)} \qquad (2.2)$$
$$e_b = (2n/3 - 1) \text{ inches} - a \geq a \qquad \text{(for short-slotted holes)} \qquad (2.4)$$

To design the bolt groups for combined effects of shear and bending moment, given by Equations 2.16, the tables given in the AISC-ASD and -LRFD Manuals (AISC, 1989 and AISC, 2000) for "eccentrically loaded bolt groups". Table 2.1 shows results of applying these tables for single row bolt group with 2 to 12 bolts with an eccentricity of shear force equal to those calculated using above Equations 2.2 and 2.4. The table is for bolts with 3 inches of spacing. First column shows number of bolts on shear tab and Columns 2, 3 and 4 show eccentricity, e_b, number of bolts to carry pure shear and fraction of strength of bolts to carry pure shear for shear tabs with standard bolt holes. Values in Column 4 are obtained by dividing values in Column 3 by the corresponding values in Column 1. Values in Columns 5,6 and 7 are values similar to Columns 2,3 and 4 but for short-slotted bolt holes. Columns 4 and 7 can be used as design guide in design of bolts in shear tabs instead of directly using the AISC

Manual tables for bolts subjected to eccentric shear. Notice that in Table 2.1, value of "a" , the distance from bolt line to weld line is assumed to be equal to 3 inches. Figure 2.12. shows values of R_e given in Table 2.1. In designing the bolts, the following simple equation can be used:

$$\phi_y V_y \leq (\phi_b \, n \, A_b F_{bv})(R_e) \quad (2.18)$$

where, R_e is a factor given in Column 4 or 7 of Table 2.1 below for standard or slotted-holes respectively. Using the R_e factor, the bolts can be designed for direct pure shear force of $\phi_y V_y / R_e$ instead of designing for a shear force of $\phi_y V_y$ with eccentricity of e_b. For definition of other terms see "Notations" in Page 4.

Notice that for shear tabs with 4 to 12 bolts, an average and approximate value of R_e equal to 0.80 (for standard holes) and 0.90 (for short-slotted holes) can be used

Table 2.1. Reduced Shear Capacity of Bolt Group Due to Presence of Eccentricity

Type of Holes → (1)	Standard Holes (2)	(3)	(4)	Short-slotted Holes (5)	(6)	(7)
Number of Bolts ↓	Ecc. e_b * (inches)	Bolts for Pure Shear	R_e	Ecc. e_b * (inches)	Bolts for Pure Shear	R_e
2	3	0.88	0.44	3	0.88	0.44
3	3	1.75	0.58	3	1.75	0.58
4	3	2.81	0.70	3	2.81	0.70
5	3	3.9	0.78	3	3.9	0.78
6	3	4.98	0.83	3	4.98	0.83
7	3	6.06	0.86	3	6.06	0.86
8	4	6.64	0.83	3	6.64	0.90
9	5	7.22	0.80	3	7.22	0.91
10	6	7.79	0.78	3	7.79	0.92
11	7	8.36	0.76	3.3	8.36	0.91
12	8	8.93	0.75	4	8.93	0.91

* : Notice that in calculating eccentricity, value of a, the distance from bolt line to weld line is assumed to be 3 inches.

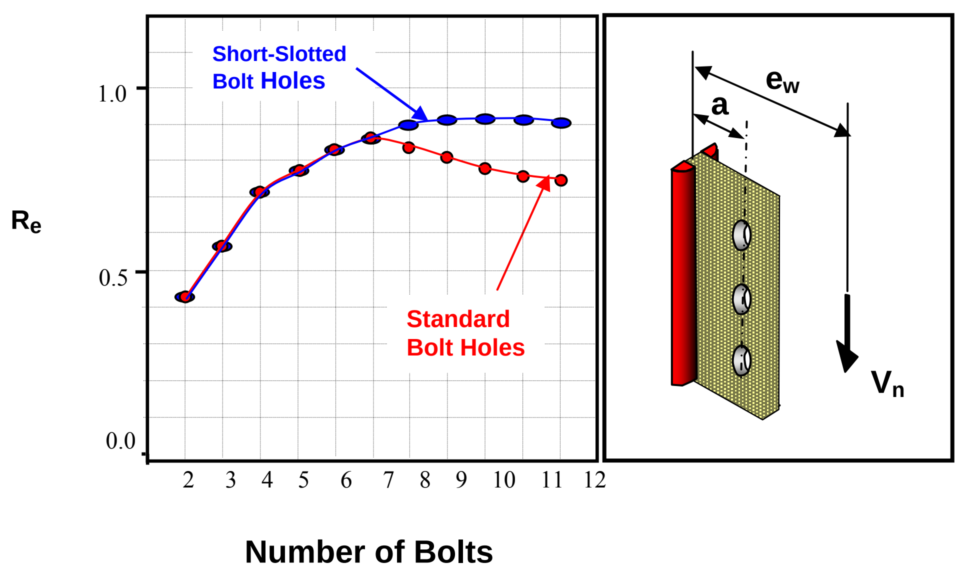


Figure 2.12. Reduction in Shear Capacity of bolt group due to presence of Eccentricity

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

As discussed earlier, the fillet welds connecting a shear tab to its support are subjected to a combination of shear force V and bending moment Ve_w due to eccentricity of shear force from the weld line as shown in Figure 2.13. To ensure that the shear tab connection has sufficient rotational ductility to accommodate the end rotation demand of the beam, it is essential that the shear tab plate yields prior to yielding of the welds as shown in Figure 2.13(a).

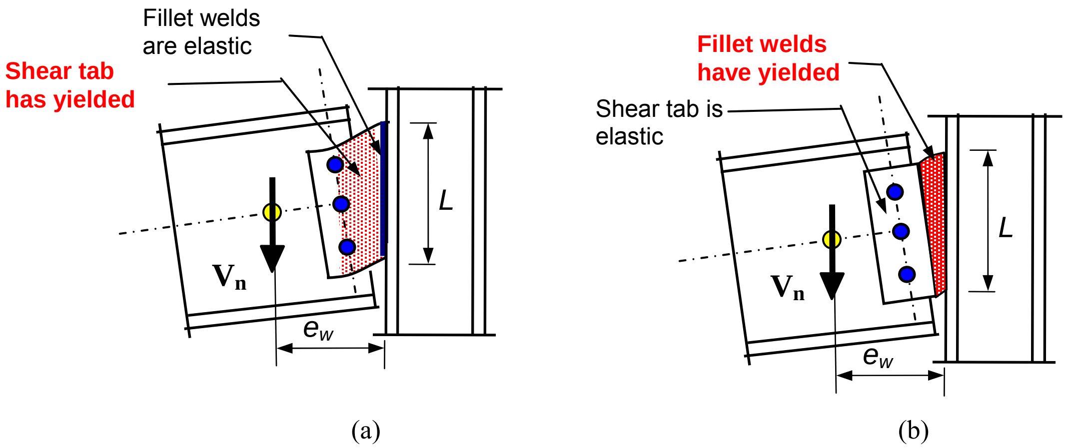


Figure 2.13. (a) Shear Tab Has Yielded Prior to the Welds; and
(b) Welds Have Yielded Prior to Shear Tab

When shear tab yields prior to the welds, as shown in Figure 2.13(a), the inelastic rotation of the connection due to yielding of the shear tab plate in the area between the bolt line and weld line is:

$$\theta_u = (\varepsilon_u)(a)/(L/2) \qquad (2.19)$$

Considering length "a" to be typically about three inches, the length "L" to be at the most 30 inches (for a 10 bolted shear tab), and ε_u , the ultimate strain for steel plate to be 0.20, the above equation indicates that by making the plate yield prior to weld, the connection plate will be able to provide a rotational ductility of about 0.04 radians. Additional rotational ductility is also provided by the bolt slippage, bolt hole elongation due to bearing, bolt shear deformation as well as the edge distance deformations. The measurements during the tests of shear tabs by Astaneh-Asl, Call and McMullin (1989) indicated that this additional rotation for A36 specimens was in the order of 0.01-0.03 radians with deeper connections having smaller value of this rotation. Therefore, the total rotational ductility of connection, *if the plate yields prior to yielding of the weld,* will be in the order of 0.05-0.07 radians. This amount of rotational ductility is sufficient to accommodate the end rotation of the beam which is in the order of 0.03-0.04 radians (see Chapter 1).

If instead of plate yielding first, *the welds yield prior to the plate*, as shown in Figure 2.13(b), the rotational ductility provided by the inelastic deformations of the welds is expected to be rather limited and not as reliable as the ductility provided by yielding of a relatively large volume of the steel plate that occurs when the steel plate yields prior to the welds. Therefore, in order to ensure that shear tabs are ductile enough to yield and accommodate end rotation demand of the beam, the design of shear tab should be based on designing welds such that *the shear tab plate yields prior to yielding of the welds.*

In the late 1980's, behavior of shear tabs were studied by the author and his research associates (Astaneh-Asl, Call and McMullin, 1989) and new design procedures for design of shear tab connections were developed (Astaneh-Asl, 1990). These design procedures, which are currently included in the AISC Manuals (AISC, 1999 and AISC, 1989), were based on ensuring that welds are designed such that the shear tab plate yields prior to yielding of the welds. Since welds as well as the shear tab plate are subjected to combined shear and bending moment, a circular yield condition (i.e. interaction curve) was considered for both in the form of:

$$(V/V_{yp})^2 + (M/M_{pp})^2 = 1.0 \qquad (2.20)$$

$$(V/V_{yw})^2 + (M/M_{pw})^2 = 1.0 \qquad (2.21)$$

Where;

V= shear acting on the weld and plate
M= moment acting on the weld and plate = Ve_w
V_{yp} = shear yield capacity of plate = $(t)(L)(0.6F_y)$
M_{pp} = plastic moment capacity of the plate= $(t)(L^2/4)(F_y)$
V_{yw} = shear yield capacity of welds = $(2)(0.707w)(L)(1/2)(0.6F_{Exx})$
M_{pw} = plastic moment capacity of the welds= $(2)(0.707w)(L^2/4)(1/2)(F_{Exx})$
L= length of shear tab = (n)(3 inches)
n = number of bolts
w= size of welds
e_w = eccentricity of point of inflection from the weld line = n inches
F_{Exx} = Strength of weld electrode
$(1/2)(0.6F_{Exx})$= shear yield stress of fillet welds based on test results reported in the literature
$(1/2)(F_{Exx})$= yield stress of transverse fillet welds based on test results reported in the literature

By eliminating V between Equations 2.20 and 2.21, the minimum weld size to ensure yielding of plate prior to yielding of weld was obtained and reported in Astaneh-Asl (1990) as:

$$w \geq 1.45\, t\, F_y / F_w \qquad (2.22)$$

The above equation for E70 electrode (F_w =70 ksi) and A36 steel (F_y =36 ksi) or A572 Grade 50 steel (F_y =50 ksi) results in:

$w \geq 0.75\, t$ (For A36 steel and E70 Electrodes) (2.23a)
$w \geq 1.03\, t$ (For Grade 50 steel and E70 Electrodes) (2.23b)

Checking failure of the base metal, following the same approach, as was done for the welds in above derivations, results in the following Equation 2.24 for minimum size of weld:

$w \geq 0.65\, t$ (for any grade of steel for failure of base metal) (2.24)

The justification for use of circular interaction curve for yielding of steel and weld is shown in Figure 2.14 by three curves. One curve shows interaction of shear and bending assuming a circular interaction in the form of Equations 2.20 and 2.21 above. A second curve shows an interaction curve where shear term is considered with power of 1.0 and the moment term is considered with power of 4.0 as given by Equation 2.25 below. A third curve is also shown in Figure 2.14 which represents the interaction of shear and bending in welds subjected to eccentric shear according to the AISC-LRFD

Manual (AISC, 2000). The AISC-LRFD curve for welds subjected to eccentric shear is almost identical to Equation 2.25 below.

$$V/V_{yp} + (M/M_{pp})^4 = 1.0 \quad (2.25)$$

Where,
V = applied shear
V_{yp} = shear yield capacity of rectangular shape based on Von Mises criterion= [Fy(√3)/3](tL)
M = applied bending moment
M_{pp} = plastic moment capacity of rectangular cross section = $Fy(tL^2)/4$

As Figure 2.14 indicates, the use of circular interaction equation, compared to the AISC-LRFD curves or Equation 2.25, will be slightly conservative for the region with high shear. For shear tabs, this is the most applicable region.

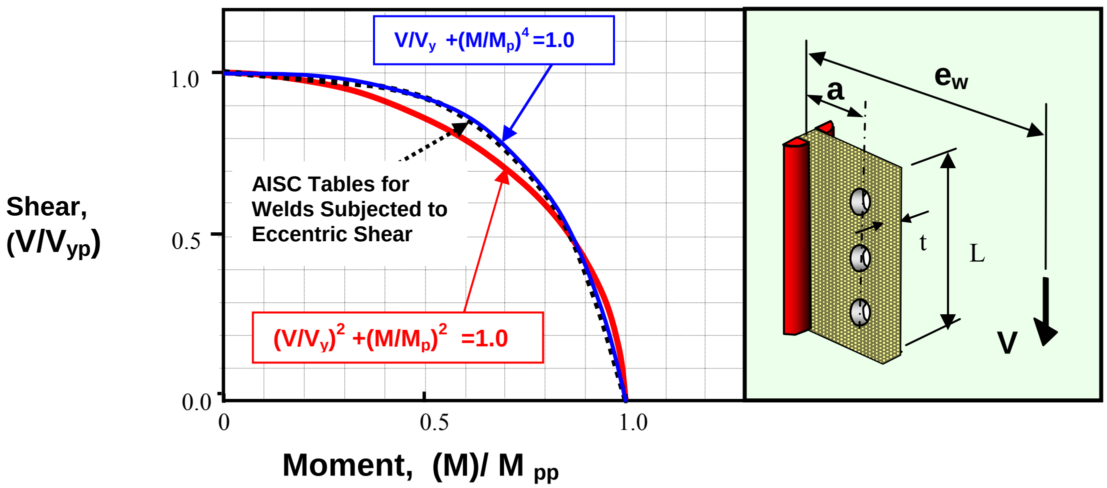


Figure 2.14. Interaction of Shear and Bending Moment Acting on Weld Lines

Since development of design procedures for shear tabs in late 1980's (Astaneh-Asl, 1990), new information on the actual properties of steel and performance of fillet welds have been generated. Since the concept of ensuring yielding of the plate prior to yielding of the welds depends heavily on the mechanical properties of the plate and welds, for this report, an attempt was made to reexamine the weld design for shear tabs and the minimum weld size requirement. One of the important developments since the release of shear tab design procedures was the 1994 Northridge earthquake and recognition of the variability of the "specified yield stress" of steel. Considering this important item, in the following, an attempt is made to establish minimum size of weld that will result in yielding of the shear tab plate prior to failure of the welds.

Earlier, we used circular interaction curve with "specified" values of yield stress for the plate and the "test" values of yield stress for the weld and obtained Equations 2.23(a) and (b) for minimum size of weld. Now, let us again use circular interaction equation but this time use "expected" yield point for the plate and "failure" strength for the welds.

$$(V_e/\phi_y V_{ype})^2 + (M_e/\phi_y M_{ppe})^2 = 1.0 \quad (2.26)$$

$$(V_e/\phi_w V_w)^2 + (M_e/\phi_w M_w)^2 = 1.0 \quad (2.27)$$

Where;
V_e= expected shear acting on the weld and plate
M_e= expected moment acting on the weld and plate = $V_e e_w$
V_{ype} = expected shear yield capacity of plate = $(R_y)(C_{pr})(t)(L)(0.6F_y)$
M_{ppe} = expected plastic moment capacity of the plate= $(R_y)(C_{pr})(t)(L^2/4)(F_y)$
V_w = shear capacity of welds = $(2)(0.707w)(L)(0.6F_w)$
M_w = moment capacity of the welds = $(2)(0.707w)(L^2/4)(F_w)$
C_{Pr}= strain-hardening factor= $(F_y+F_u)/2F_y$ (given by FEMA, 2000)
R_y = a factor to be multiplied by the specified yield stress to obtain expected yield stress
R_yF_y = expected yield stress

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

It should be mentioned that the use of parameter C_{pr} in above procedures is necessary since as shear tabs undergo relatively large yielding to accommodate rotation demand of the beam end, they experience significant strain hardening as well. The strain hardening of a shear tab specimen is shown in Figure 2.15 (Astaneh-Asl, Call and McMullin ,1989).

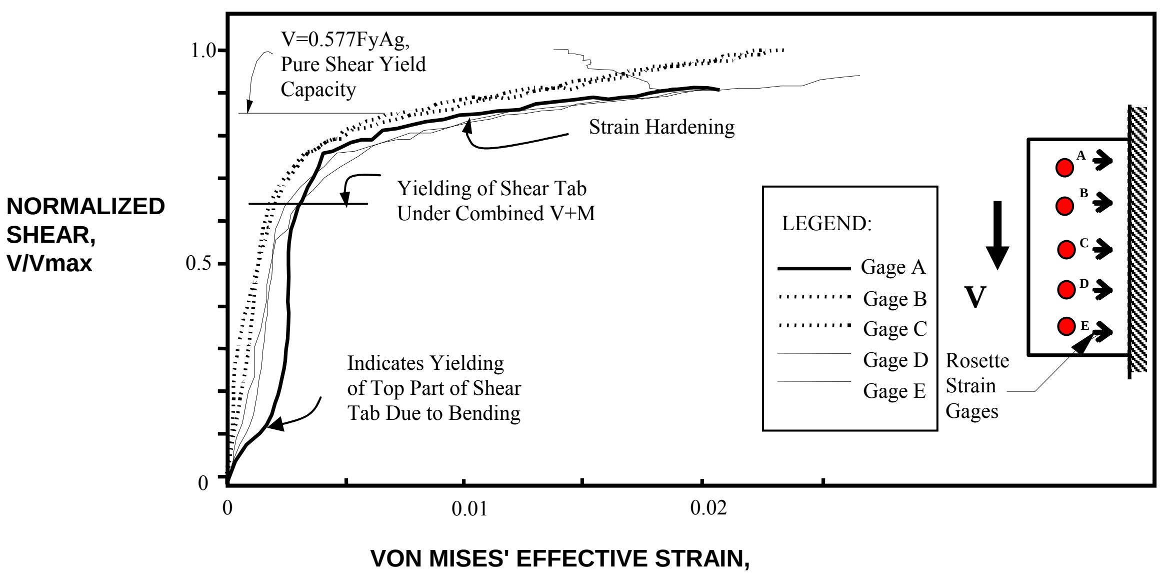


Figure 2.15. Von-Mises Strains versus Applied Shear in a Shear Tab
(Astaneh-Asl, Call and McMullin, 1989)

Therefore, considering the fact that shear tabs experience strain-hardening while undergoing rotations, the welds connecting the plate to the support should be designed to resist not just the "specified" yield stress of the plate but the strain hardened value of the yield stress. In order to establish a strain hardened value of yield stress, we have followed the recommendations of FEMA-350 (FEMA, 2000) and have multiplied the specified yield stress, Fy, by the a strain hardening factor, C_{Pr} given as $(F_y+F_u/2F_y.)$

By eliminating V in Equations 2.25 and 2.26, the minimum weld size to ensure continued yielding of the plate prior to fracture of the welds is obtained as:

$$w \geq 0.73\, t\, R_y\, C_{Pr}\, \phi_y\, F_y \,/\, \phi_w F_w \qquad (2.28)$$

The above equation for E70 electrode (F_w =70 ksi) and E36 steel (F_y =36 ksi , R_y=1.3, C_{Pr}= 1.3) or A572 Grade 50 steel (F_y =50 ksi , R_y=1.17, C_{Pr}= 1.15) results in:

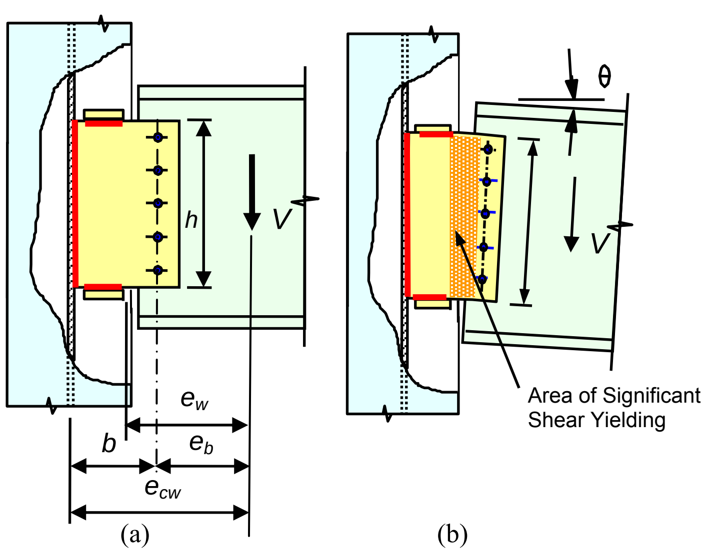
$w \geq 0.76\, t$ (For A36 steel and E70 Electrodes) (2.29a)

$w \geq 1.15\, t$ (For Gr 50 steel and E70 Electrodes) (2.29b)

The values of R_y used in above derivations for A36 steel and Grade 50 steel are 1.3 and 1.17 respectively. These values are based on the work of Brockenbrough (2001) and Liu et al (2005). The above values of minimum weld size predicted using the latest available information on material properties and latest version of the AISC-LRFD Manual (AISC, 2000) are very close to those predicted by Equations 2.23 above in 1980's (Astaneh-Asl, 1990). Therefore, the minimum size of E70 fillet welds for shear tabs are recommended to be equal to 0.75t and 1.0t for A36 and grade 50 steels respectively.

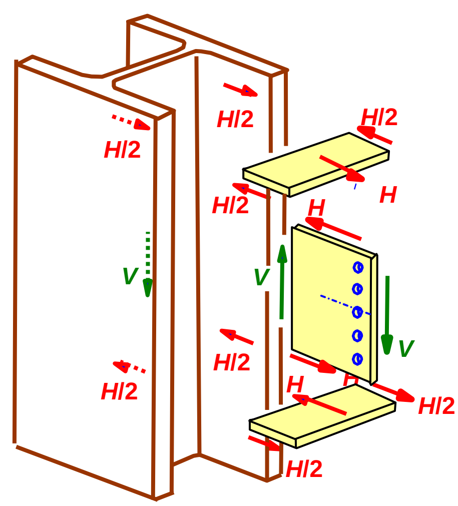
2.5. Design Considerations for Shear Tabs on the Web Side of the Column

Shear tabs quite often are used to connect the beam end to the web side of wide flange columns, as shown in Figure 2.16. In this case, the shear tab is widened to be welded to the web of the column while the bolt holes remain on the extended portion of the shear tab and outside the column. The failure modes and eccentricities for shear tabs discussed earlier in this chapter remain in effect for the extended portion of the shear tab as shown in Figure 2.16. In this case, a relatively large moment is created in the connection due to the relatively large eccentricity of the point of inflection from the column web. To resist this moment, horizontal plates are welded to the top and bottom of the shear tab, and the ends of these plates are welded to the column flange, Figure 2.16. Figure 2.17 shows a free body diagram of the connection element for this connection with design forces acting on each element. The elements should be designed for these forces.



(a) (b)

Figure 2.16. (a) Shear Tabs Connected to the Web Side of the Column and (b) Expected Deformations



***V* = Design Shear Force = (0.6)F_y A_g**

H* = (*V* e_{cw})/ *L

Figure 2.17. Free-Body Diagram of Connection Elements

2.6. Notes on Shear Tabs on the Web of the Girders

Quite often shear tabs are used to connect a beam to another beam or a girder. Figure 2.18 shows examples of such applications. Behavior of shear tabs connected to the web of a beam was studied by Shaw and Astaneh-Asl (1992). The studies included testing six full-size specimens of beams connected to one side of girders using shear tabs. Shear tabs had either 4 or 6 bolts. Figure 2.18 shows a typical specimen schematically. The tests indicated that behavior of connection and failure modes strongly depends on h/t of the web of the girder. The location of point of inflection was established at a distance of e_w from the weld line given by Equation 2.30:

$$e_w = [40/(h/t)]\ n \text{ inches} \qquad (2.30)$$

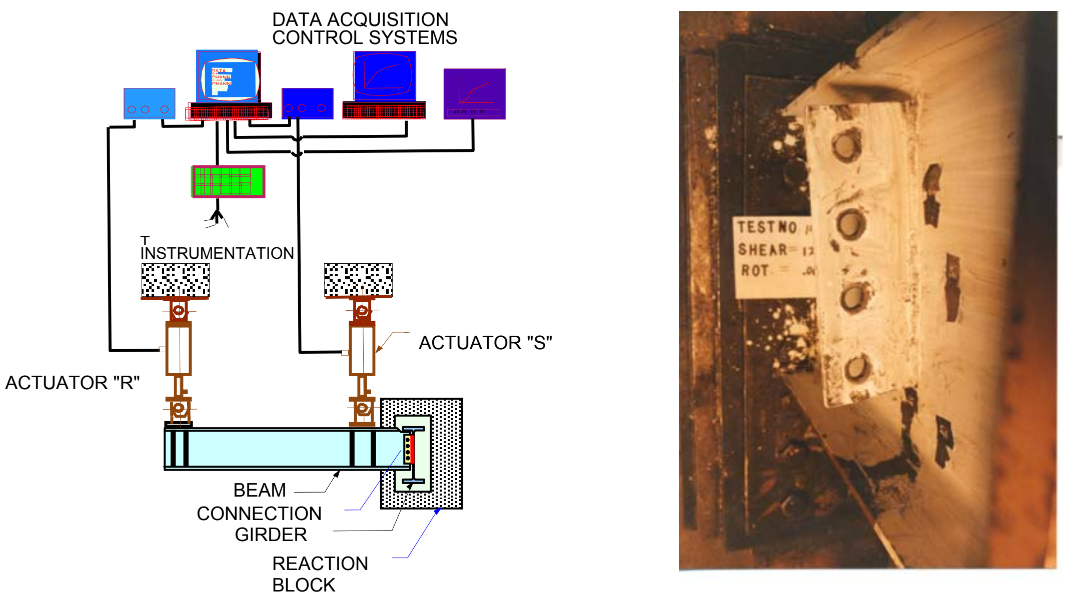


Figure 2.18. Test Set-up and Typical Specimen of Beam-to-Girder Shear Tab Connection (Shaw and Astaneh-Asl, 1992)

Failure modes that were observed included bolt shearing, weld fracture starting from the top of the weld lines, deformation of girder web and torsional deformation of shear tabs. Figure 2.19 shows sketches of these failure modes.

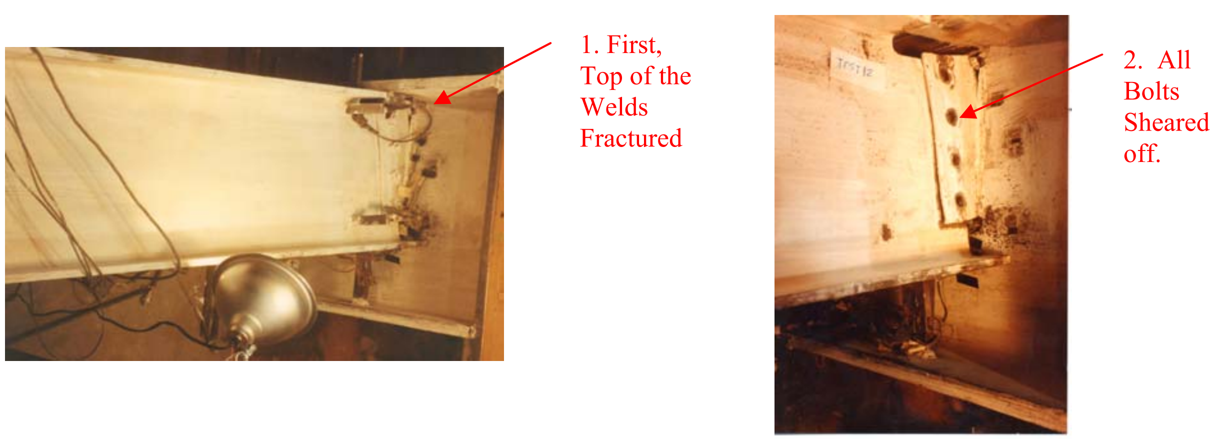


Figure 2.19. Failure of Specimen 12, a Beam-to-Girder Shear Tab Connection

2.7. Double Plate Shear Tab Connection

In some applications, due to limitations of depth and the relatively large value of the shear force, use of two plates instead of a single shear tab may be more economical. By using two plates, one on each side of the beam web, the bolts will be subjected to double shear, and their shear strength will be twice as much as the shear strength of similar bolts in a single-plate shear tab connection. Of course, when two plates are used, the bottom or top flange of the beam needs to be coped to enable the erectors to place the beam web between the two plates. Figure 2.15 shows a suggested detail for double-plate shear tabs. The welds on both sides of the first shear tab to be welded are fillet welds. For the second plate, there is only one partial penetration weld line on the exterior surface of the plate as shown in Figure 2.15.

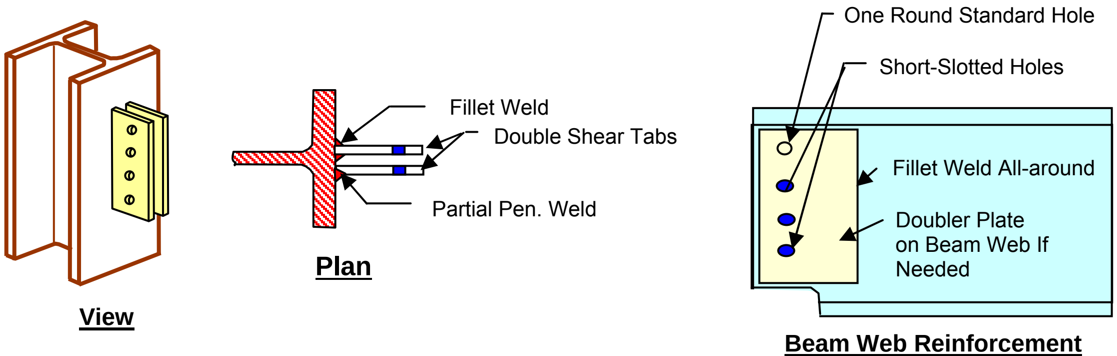


Figure 2.15. Double Shear Tab on Column Flange

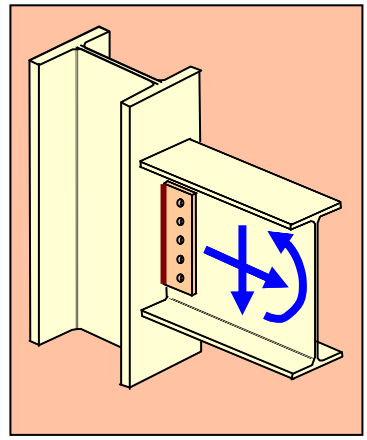
Design of double shear tabs can follow the same procedures presented in earlier sections of this chapter for the single-plate shear tab. The two shear tabs should be the same size and same material. Special attention should be paid to checking the bearing and block shear failure modes of the beam web. If necessary, the beam web can be reinforced with doubler-plate as shown in Figure 2.15. It should be added that in all cases of connections, the limit states of the beam also should be checked. In this case, since one flange of the connected beam is coped, it is necessary that, in addition to other beam failure modes in the connection area, the block shear failure of the beam web also be checked.

It should be mentioned that, in the opinion of the author, the above “double-shear-tab” detail is not a first choice for the shear tab connections and should not be used if a single-plate shear tab can be used. It is included here because of its rare use in some cases, which prompted the author to provide some information on it in this report.

2.7. Material Considerations in Design of Shear Tab Connections

Almost in all of the tests done on shear tabs, the steel for the shear tab plate has been A36 with specified yield stress of 36 ksi. Considering design procedures established for shear tabs with failure modes set in a hierarchical order, the use of a higher strength steel should not affect the behavior as long as the ductility of the steel is similar to the ductility of A36 steel. Therefore, the use of A572-grade 50 steel plate in shear tabs is expected to result in similar ductile behavior as the A36 shear tabs, as long as the hierarchical design procedures of Section 2.4 are followed. As for the bolts, both A325 and A490 bolts have been used in the tests, and shear tab specimens with these two types of bolts have performed satisfactorily. Since no data is available on the behavior of shear tabs with large diameter bolts, it is suggested that the diameter of the bolts used in shear tab connections not exceed 1-1/8 inch and preferably be less than 1 inch. These, or smaller bolt diameters, will be easier to install and tighten as well during the erection process.

3. DESIGN OF SHEAR TABS FOR SEISMIC EFFECTS



3.1. Introduction

The behavior of welded steel moment frame buildings in the Northridge earthquake suggested that the contribution of shear connections, such as shear tabs, to the lateral load-resisting system was greater than previously thought. Parts of frames designed solely for carrying the gravity loads using simple shear connections might have contributed to the overall lateral stability of these buildings. Furthermore, it was hypothesized that the presence of the floor slab must have had an impact on the moment-rotation behavior of the simple connections.

In an effort to quantify the contribution of the simple connections to the lateral load resistance of steel structures, as well as to define better the role of the floor slab, experimental studies were undertaken at the University of California at Berkeley by Liu and Astaneh-Asl (2000a–d). The studies were part of a greater multidisciplinary and coordinated research effort by the SAC Steel Joint Venture Project and were funded by the Federal Emergency Management Agency.

3.2. Cyclic Behavior and Seismic Design of Shear Tabs (Liu and Astaneh-Asl 2000a–d)

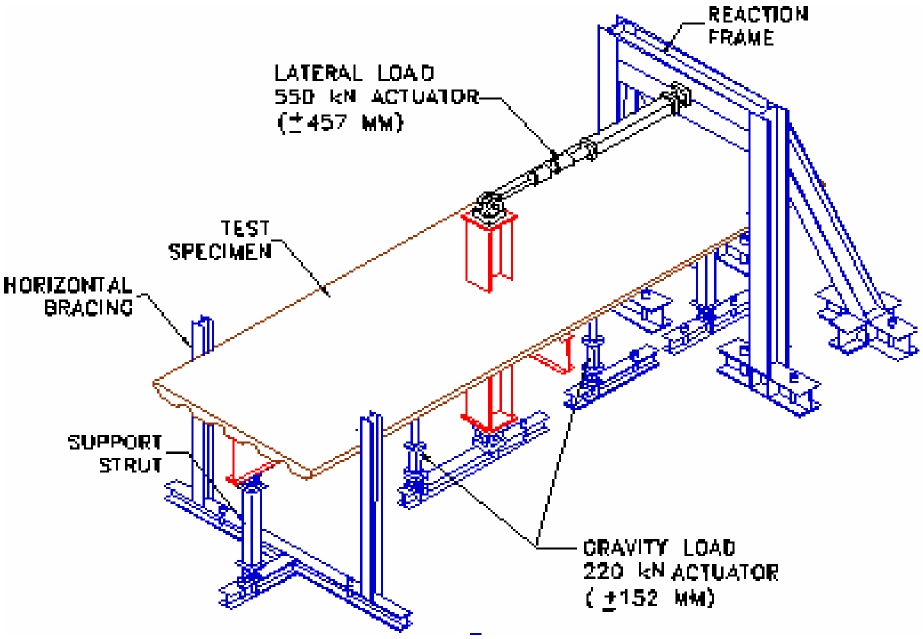
As part of the SAC Joint Venture Steel Project, Liu and Astaneh-Asl (1998, 1999, and 2000a–d) conducted a series of cyclic tests of shear connections including shear tab connections and developed data on cyclic behavior of shear tab connections with or without floor slabs. In addition, Liu and Astaneh-Asl (2004) proposed seismic design recommendations and models of behavior. In the following, a summary of this work on shear tabs is presented. For more information on this study, the reader is referred to the above references.

3.2.a. Cyclic Tests of Shear Tabs

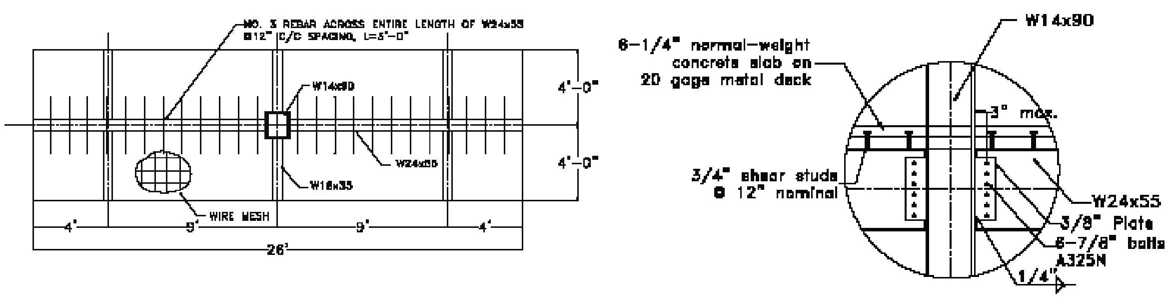
In this study, ten full-size shear tab specimens were subjected to cyclic loading representing the effects of earthquakes on shear tab connections of steel buildings. The cyclic loading that was applied to the specimens was according to a protocol developed by the SAC Joint Venture (1997). The protocol later became part of the AISC Seismic Provisions for Structural Steel Buildings as its Appendix S (AISC 2002).

In the test program, two specimens were tested without the floor slab, while the remaining eight specimens had typical steel deck and concrete slab floors.
Figure 3.1 shows the test setup used in the cyclic testing of single-plate connections with a floor slab (shown in the figure) or without a slab as well as two typical specimens. The gravity load, simulating dead and live load, was applied first and kept constant throughout the tests. After application of constant

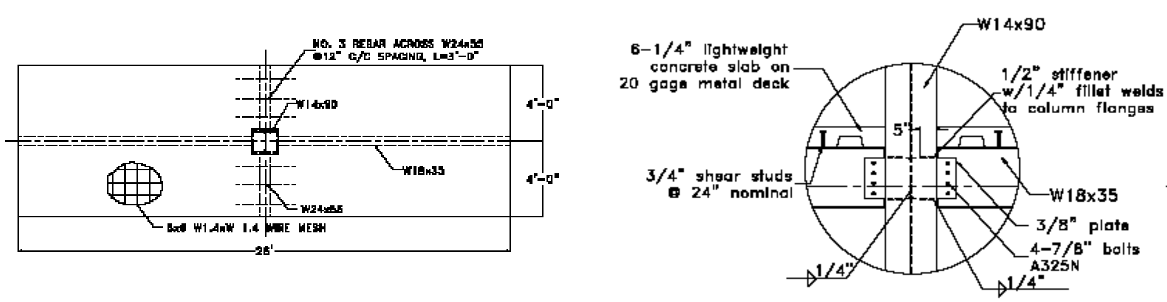
gravity load, the horizontal “550 kN ACTUATOR” shown in Figure 3.1(a) was used to apply cyclic lateral load at the midheight of the upper floor column.



(a) Test Setup



(b) Typical Specimen with the Shear Tab Connected to the Flange Side of the Column



(c) Typical Specimen with the Shear Tab Connected to the Flange Side of the Column

Figure 3.1. Test Setup and Two Typical Specimens of Shear Tab Assemblies (Liu and Astaneh-Asl 2000d)

Parameters that varied in the specimens included the number of bolts, type of concrete, and level of reinforcement in the floor slab. As is the case in design offices, these connections were designed for gravity loads only. Each specimen was constructed as if it were from a prototype building with W14x90 columns at 25-foot spacing, and W18x35 beams framing into W24x55 girders. The W-shapes were A572 Grade 50 steel with specified yield stress of 50 ksi; the connection plates were typically A36 steel with a specified yield stress of 36 ksi. The welds were flux-cored arc welds with E70T-7 electrodes with a specified strength of 70 ksi. In all shear tabs the weld design was based on making the size of weld equal to 0.75 times the thickness of the A36 plate. The bolts were 7/8-inch diameter, A325N tension-control high-strength bolts. The floor slab was a 6 ¼-inch concrete slab on steel 20-gage decking with 2.5-inch corrugation. The ribs of the deck were oriented perpendicular to the W18 beams. The concrete had a specified compressive strength of 3,000 psi. Lightweight as well as normal weight concrete was used in the floor slab to study the difference in behavior.

Reinforcement for the floor slab for most specimens was nominal, limited to welded wire mesh for temperature and shrinkage control and nominal reinforcement across the girders for crack control. This welded wire mesh was a 6-inch grid of nominal 1/8-inch wire. There was also nominal reinforcement across the W24x55 girders for crack control under gravity loads. This consisted of D10 reinforcing bars at 12 inches spacing, with a concrete cover of ¾ inch.

Welded shear studs connected the floor slab to the beams and girders. The number of shear studs was nominal. Shear stud spacings of 24 inches and 12 inches were used for the W18x35 beams and W24x55 girders, respectively. The shear studs were ¾ inch diameter and were 5-3/8 inches long, made from AISI Grade C-1015 steel with a nominal yield stress of 50 ksi. The floor beams were not designed as composite. Shear studs were used to control deflection of the beams and to transfer seismic inertia forces from the floor to the steel structure. By using the above-mentioned shear studs, the beams and girders ended up being 20–30% partially composite.

The earlier research on shear tab connections under gravity load, as summarized in Chapter 2, had indicated that the initial shear and rotation on the connection due to gravity loads would have a significant effect on the response. While some load was present in the system due to the self-weight of the specimen, it was necessary to apply additional load in order to represent the initial gravity load seen in typical structures. The choice of gravity loads followed the philosophy of ATC-33 (NEHRP 1997), which states that 25% of the unreduced live load, but not less than the real live load, can be used for the analysis of buildings under seismic loads. As a best approximation, two actuators on each beam, each located at 5 feet, 6 inches from the centerline of the column, were used to create the appropriate shear and rotation at the joint. The actuator loads were applied monotonically and held constant for the duration of the cyclic test. Later analysis of the test results showed that the initial gravity moments in the connections averaged 20–25% of the maximum moment experienced during cyclic loading.

3.2.b. Results of Cyclic Tests

Cyclic tests of shear tabs indicated that:

1. The connections were very ductile and could tolerate cyclic rotations of at least 0.09 radians, Figure 3.2, before fracture started. It should be mentioned that the connection rotations mentioned here are actually interstory drifts measured by dividing the horizontal movement of the top of the middle column (see Figure 3.1) relative to its base divided by the distance between its top and bottom hinges. The large rotational ductility is the result of the design philosophy, discussed in previous sections, that makes the yielding of the plate the governing failure mechanism. As the load increases and yielding of the shear tab plate continues, due to strain-hardening, the gross

area becomes stronger. At this time, the second failure mode, which is the bearing yielding of the plate and yielding of the gross area of the beam web, becomes the governing failure mode and yielding shifts to these areas of connection. The yielding provides further rotational ductility. The very desirable yielding of the shear tab and beam web around the shear tab is evident in the close-up photo in Figure 3.2.

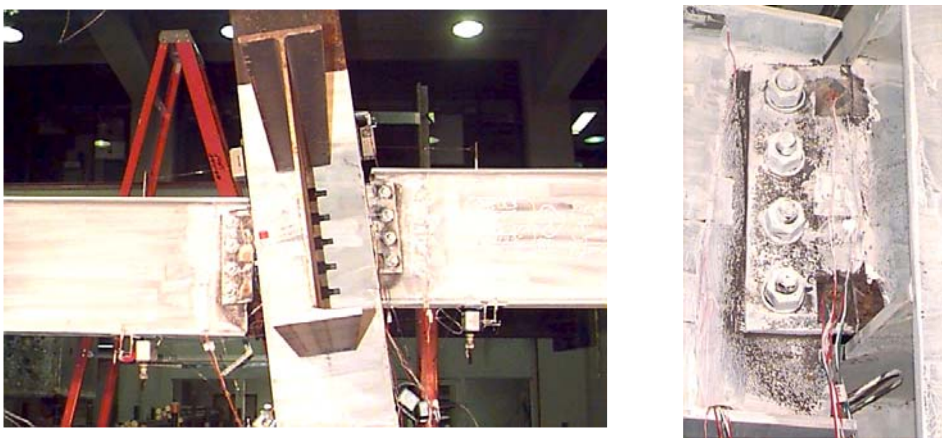


Figure 3.2. Specimen without Slab at the End of the Test

2. When the gravity load, equal to DL+0.25LL, was applied, bending moments developed in the connections were between 0.25–0.050 of the maximum moment developed in the connections during later cyclic loading.

3. Under very large rotations (beyond 0.09 radians), cracks were initiated in the connection plate. The cracks were either in the net section of the plate along the bolt lines or the gross area of the plate at the heat affected zone adjacent to the welds as shown in Figure 3.3. In most specimens, the cracks in the shear tab did not propagate throughout the depth and were stabilized after propagating about 1/3 of the depth of the plate. The remaining uncracked cross section of the plate was able to carry the gravity shear.

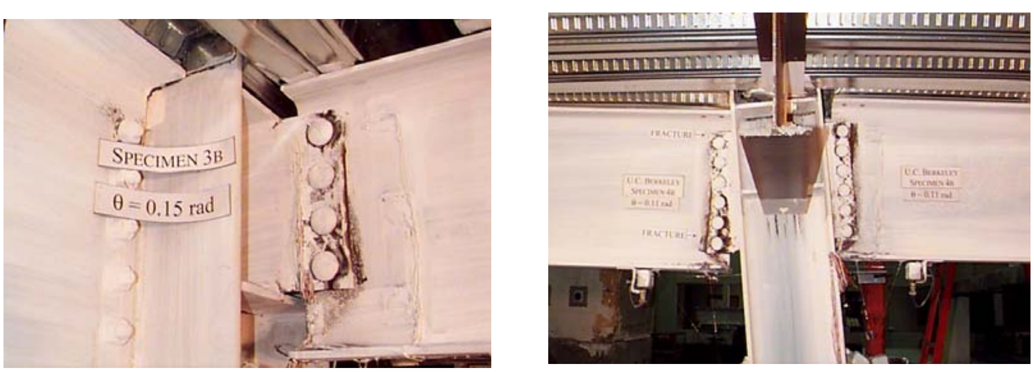


Figure 3.3. Fracture of Net Area under Very Large Rotations (Larger Than 0.09 Radians)

In one specimen, the entire depth of the shear tab cracked during the later large rotation cycles. However, strange enough, the beam in this case, which had lost one of its end supports, did not collapse. After further inspection of the specimen, we became convinced that after fracture of the shear tab, the reaction of the beam was transferred to the floor deck by shear studs and the steel deck in a catenary action as shown in Figure 3.4. Notice that in Figure 3.4, the vertical separation of the left span beam from the floor deck is significantly magnified. In reality, the separation of the beam top flange and the steel deck was hardly noticeable.

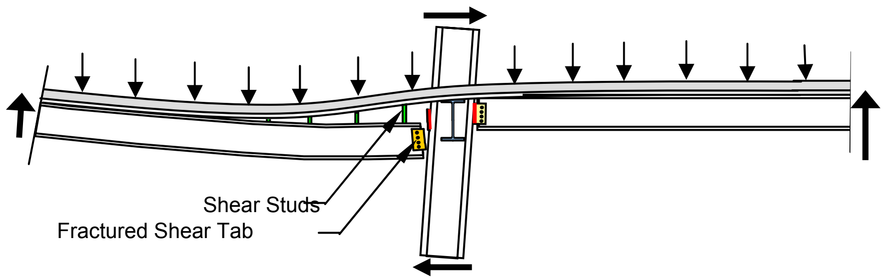


Figure 3.4. Beam Remaining in Place after Complete Fracture of Shear Tab

The beam remaining in place even after the complete fracture of the shear tab may not seem important since the shear tab fractured under very large rotations that are almost impossible to achieve under ordinary loading. However, this behavior was an indicator of the robustness of the system where complete failure of a local element (the shear tab) did not result in any significant change in gravity load carrying capacity of the system nor in progressive collapse. Such robustness and redistribution of load to other elements without causing them to fail can make all the difference between survival and catastrophic progressive collapse of the floor.

4. In specimens with a slab, the concrete around the column crushed during later cycles, as shown in Figure 3.5, after specimens underwent rotations (that is, interstory drifts) in excess of 0.15

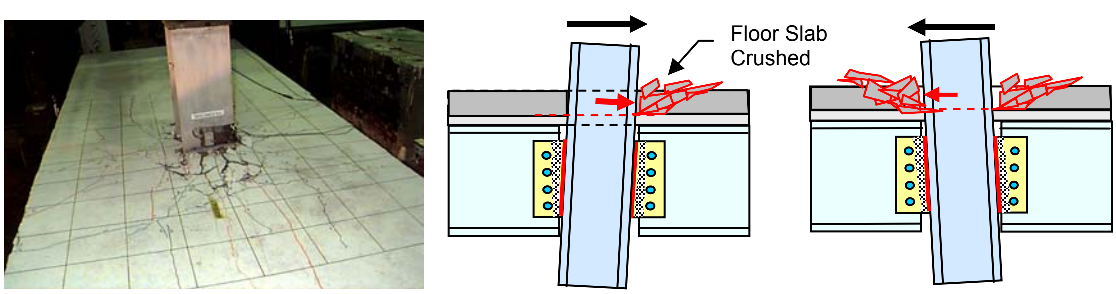


Figure 3.5. Damage to the Floor Slab during Cyclic Rotation of the Column

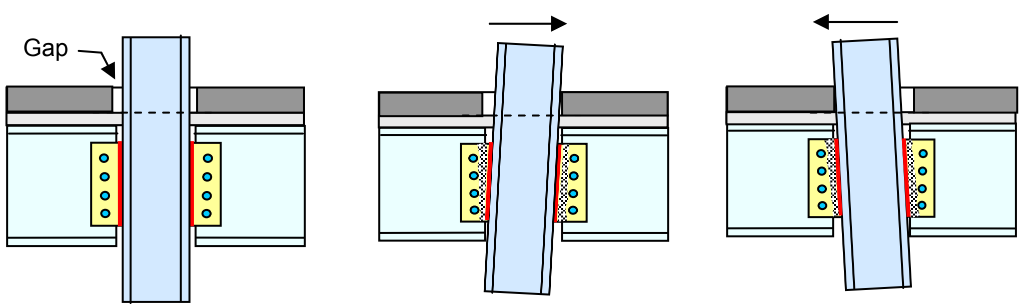


Figure 3.6. Gap around the Steel Column Prevents Damage to the Floor Slab

radians. In general, the crushing of concrete was limited to an area of the floor slab around the column with a radius of about 3 feet (Figure 3.5, left). If one desires to prevent such damage to the floor, which is expected only during very large earthquakes, one can provide a gap between the floor slab and the steel column as shown in Figure 3.6. In this case, when the column rotates, it cannot press against the slab and crush it; see Figure 3.6. A gap of ¾ to 1 inch is suggested.

The tests' results also yielded valuable information on the cyclic moment-rotation behavior of shear connections. A typical example is shown in Figure 3.7. Three distinct zones can be identified on the curves: (1) initial cycles where the moment rotation curve is almost elastic with relatively large moments developed in the connection due to compression in the concrete slab; (2) the zone of ductile rotation with relatively small moments in the connection where after crushing of the floor concrete, the only element resisting the moment is the shear tab itself; and (3) the zone of increased moment and relatively large rotational stiffness due to the bottom flange of the beam pressing against the column. These three stages are shown in Figure 3.7.

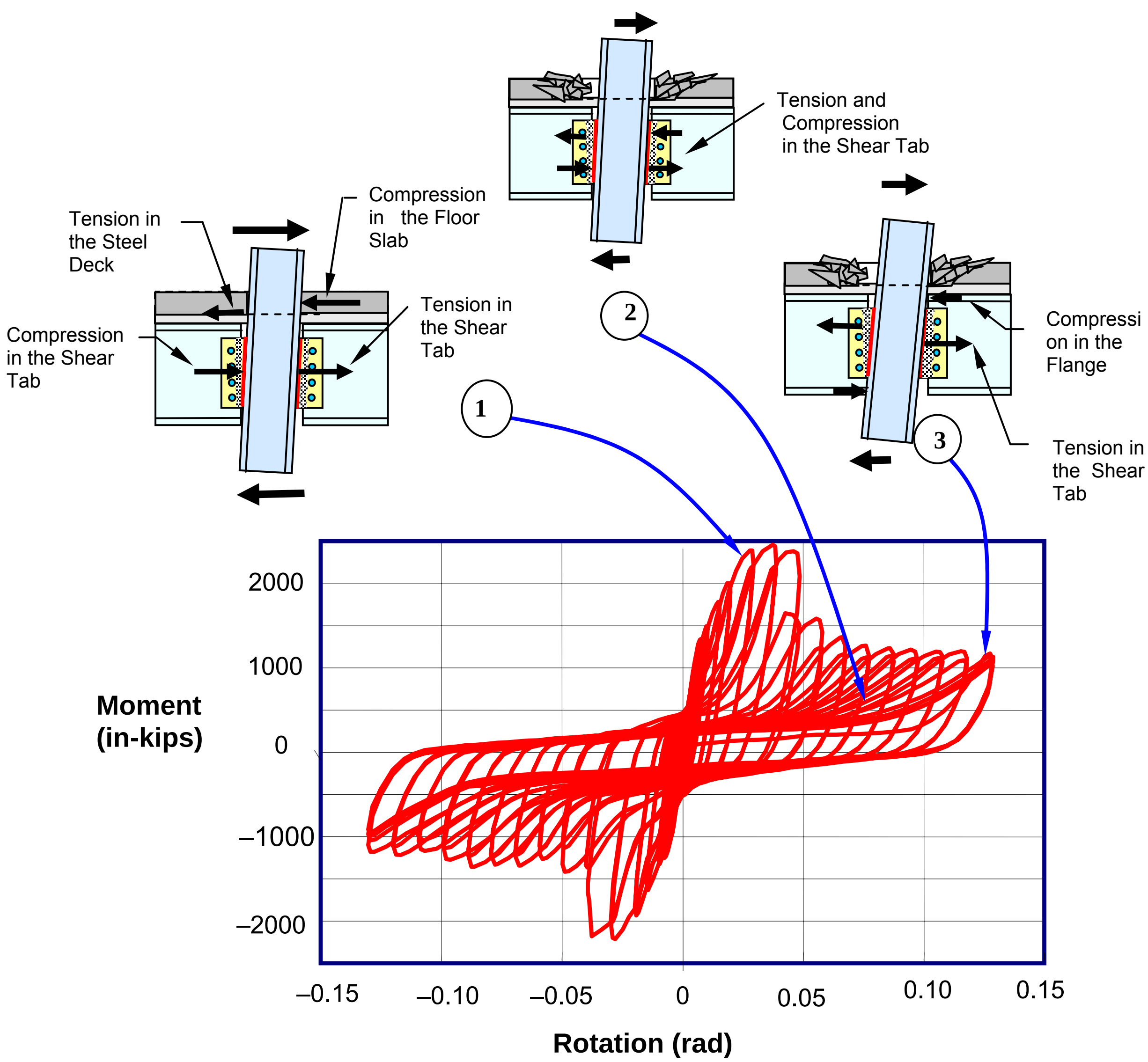


Figure 3.7. Three Distinct Stages of Moment-Rotation Behavior

During initial cycles, the connection showed large moment capacity almost equal to the plastic moment strength of the connected beam. When rotation reached about 0.04 radians, the concrete in front of the column was crushed and the moment started to drop. Development of this large moment is important since, during small earthquakes and winds, this extra moment and rotational stiffness of the shear connections can contribute to the overall lateral strength and stiffness of the structure and limit the damage to nonstructural elements. However, this strength and stiffness may not be very reliable since after exceeding rotations of about 0.04 radians and crushing of the floor slab concrete, this moment strength rapidly drops. In fact the initial peak in the moment capacity and rotational stiffness may result in development of large inertia forces in the structure and its elements such as columns supporting the shear tab connections. By providing a gap around the column, as was suggested earlier and shown in Figure 3.6, development of the large initial moment and rotational stiffness in shear connections can be avoided. The third zone of distinct behavior of connection occurs when, under large rotations, the horizontal flange of the beam is pressed against the vertical flange of the column as shown in Figure 3.7. This results in development of relatively large moment and rotational stiffness in the connection. During this stage, shear tabs can be subjected to relatively large tension, which in turn can result in fracture of the net section. This was observed in some specimens during the cyclic tests. To mitigate this undesirable behavior, one can provide larger clearance between the end of the beam and the face of the column. A clearance of at least ¾ inch and preferably 1 inch is recommended. Such clearances also help erectors during the construction phase.

3.2.c. Summary and Conclusions of Cyclic Tests of Shear Tabs

1. Slip and yielding of the bottom of the shear tab began at low levels of drift of about 0.005 for connections with floor slabs.
2. Shear tabs behaved in a very ductile manner and were able to tolerate story drifts in excess of 8%.
3. Bending moment strength of typical shear tab connections with the slab were on the order of 30–60% M_{Pb}.
4. All shear tab specimens typically lost the composite action of the slab after reaching 4% story drift, with a significant drop in the moment developed in the connection. After loss of composite action, the moment in the connection dropped to the moment capacity of specimens without a floor slab.
5. Rotational ductility of shear tabs was inversely proportional to the depth of the connection and the gap between the end of the beam and the face of the column. When the end of the beam rotated and the beam flange touched the column face, due to binding, large moments were developed in the connection resulting in fracture of the shear tab.

3.3. Modeling of Cyclic Behavior of Shear Tabs and Their Seismic Design

During earthquakes shear tabs are expected to be subjected to shear, axial load, and bending moment as shown in Figure 3.8. Currently, in seismic design of typical buildings, shear tab connections are considered to act as pin connections and have shear and axial load strength and stiffness but no moment capacity and rotational stiffness. It was shown in the previous section that deep shear tab connections with more than 5 bolts can have considerable initial rotational stiffness and thus develop significant bending moment in the connection especially when there is a floor slab present. Due to the advent of computer analysis of structures in many cases of design, especially in cases of seismic retrofit, pushover, and time history analyses, the designer is required to use more realistic models of connection behavior. This section is devoted to establishing realistic axial, shear, and rotational stiffness and strength of the shear tabs.

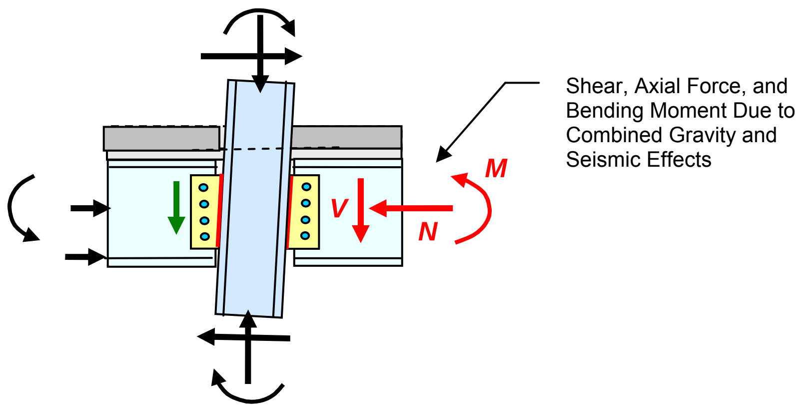


Figure 3.8. Shear, Axial Force, and Bending Moment on a Shear Tab Connection

3.3.a. Realistic Models of Rotational Stiffness and Bending Strength of Shear Tabs

Figure 3.9 shows a simplified model of the moment-rotation behavior of typical shear tab connections. The model was developed using the results of tests discussed in the preceding sections.

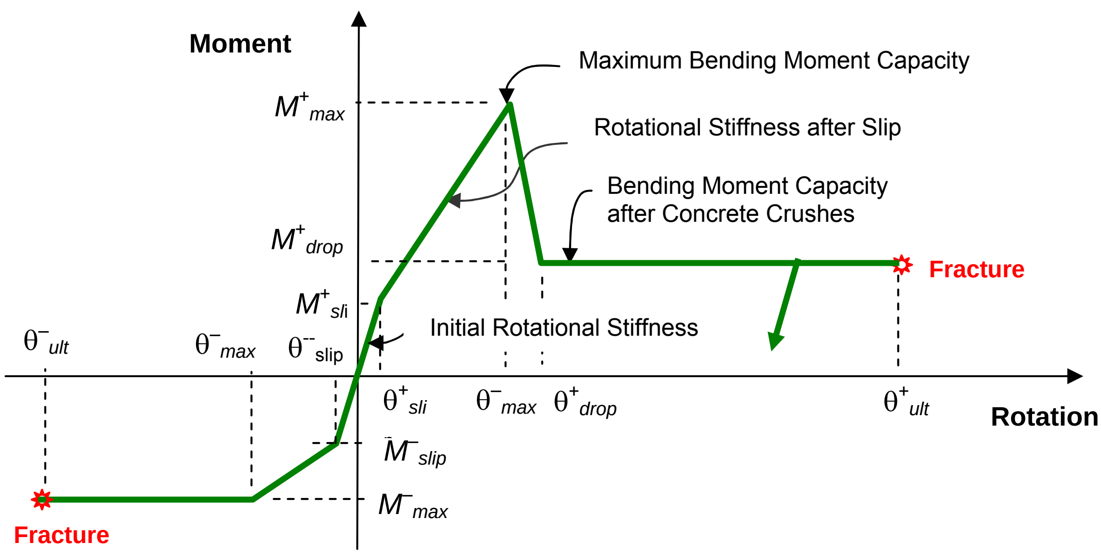


Figure 3.9. Realistic Model of Moment-Rotation Behavior of Shear Tabs
(Liu and Astaneh-Asl, 2004)

The parameters governing the behavior are shown in Figure 3.9 and are as follows:

1. **Initial rotational stiffness**—is the stiffness of the composite connection with the floor slab acting as a compression element and the shear tab acting as a tension element.
2. **Slip moment—**is the bending moment at which bolts in the connection slip.
3. **Maximum bending moment capacity**—is the bending moment reached before the concrete floor slab is crushed in front of the column.

4. **Secondary rotational stiffness**—is the rotational stiffness of the connection after the floor slab has been crushed and the connection has been reduced to just the shear tab alone.
5. **Secondary moment capacity**—is the moment capacity after the floor slab has been crushed and the bending capacity has been reduced to more or less that of the shear tab alone.
6. **Maximum rotational ductility**—is the final rotation at the fracture when the net section or gross area of the shear tab partially fractures due to relatively large rotations.

Because of rotational stiffness, considerable bending moments can develop in deep shear tab connections. Usually, the rotational stiffness of a shear tab connection and the bending moment in the connection are relatively small compared to the corresponding values for the beam itself. However, even small values of stiffness and bending moment developed in the shear connections can have beneficial as well as harmful effects on the seismic behavior of the structure as a whole. The beneficial effects of shear tab connections on seismic behavior are that the bending strength of shear tabs can add to the overall lateral load resisting strength of the frame and the rotational stiffness of shear tabs can add to lateral stiffness and help reduce interstory drift and P-δ effects. One of the harmful effects of rotational stiffness and bending moment developed in the shear connections is that the moment can result in panel zone yielding in the column or even buckling of the column under combined effects of axial load and this extra moment developed in the shear tab connection, which usually is not considered in typical seismic design. Another important parameter affecting seismic behavior of shear tabs is rotational ductility.

The research and development project done by Liu and Astaneh-Asl (2004) provides details of how to establish a realistic moment-rotation model of a shear tab connection as shown in Figure 3.9. A summary of the equations to establish various values of bending moments and rotations shown in Figure 3.9 is given in Table 3.1. The equation for rotation capacity, θ_{total}, is defined as:

$$\theta_{total} = g/d_f \qquad (3.1)$$

Where:

g = gap between the beam flange and the column

d_f = distance from the midheight of the shear tab to the furthest beam flange, or the largest of d_1 and d_2, Figure 3.10.

In the above equation, conservatively we can use the same θ_{total} for both positive and negative rotations.

Table 3.1 Values of Parameters for Moment-Rotation Curves of Shear Tabs (from Liu and Astaneh-Asl 2004)

Quantity	**Average Value of Quantity**
M^+_{slip}	$0.25M^+_{max}$
θ^+_{slip}	0.0042 radians
M^+_{drop}	$0.55\ M^+_{max}$
θ^+_{drop}	0.04 radians
M^+_{max}	Calculated using the procedure following this table
θ^+_{max}	0.03 radians
θ^+_{ult}	g/d_f
M^-_{slip}	$0.50M^-_{max}$
θ^-_{slip}	0.0042 radians
M^-_{max}	Calculated using the procedure below this table
θ^-_{max}	0.02 radians
θ^-_{ult}	g/d_f

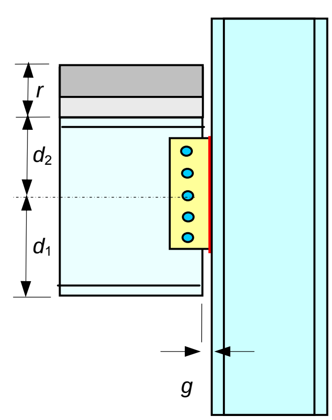


Figure 3.10

Calculating M^+_{max}—The maximum positive moment capacity of a shear tab with the slab is established by following the procedure given in Liu and Astaneh-Asl (2004). In this procedure it is assumed that:

1. Distribution of the forces acting on the cross section of the shear tab connection is plastic as shown on the right side of Figure 3.11 for positive moment and on the left side of Figure 3.11 for negative moment.
2. The necessary number of bolts near the neutral axis is responsible for carrying only shear while the remaining bolts away from the neutral axis are responsible for carrying the tension force due to the bending moment.
3. The concrete floor slab is responsible for carrying the compression force due to the bending moment.
4. The axial force acting on the shear tab connection is negligible. For large axial loads, such as those in "collector" beam connections, see later sections in this chapter.

The preceding assumptions are consistent with the observed behavior of shear tab connections tested under cyclic loading by Liu and Astaneh-Asl (2004) as part of the SAC Joint Venture Steel project.

It should be mentioned that, in design of shear tabs for gravity alone, we ignore the effects of the slab. This is a conservative assumption and may be justified since when the bulk of the dead load, the weight of the wet concrete, is placed on the steel deck, the shear tab is the only structural element until the floor concrete hardens.

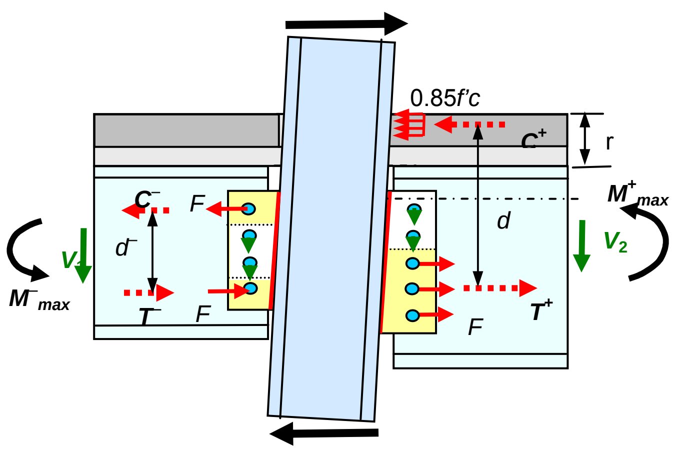


Figure 3.11. Forces Acting on Connection Elements

Following are the steps to be taken to calculate M^+_{max} and M^-_{max}.

a. Calculating M^+_{max}:

Step 1—Calculate the number of bolts, N_v, needed to resist shear force in the connection. Consider the number of bolts from the top of the shear tab to be "shear bolts," carrying only shear and not participating in resisting bending moment. These bolts are shown in Figure 3.11 (on the right side connection) with vertical shear forces acting on them. To find the number of shear bolts needed to carry shear, use the following equations:

$$N_v = V/F \qquad (3.2)$$

Where force F, which is the shear strength of one bolt, is given in ASD and LRFD formats as:

$$F = \phi_b F_b \quad \text{(LRFD)} \quad (3.3a)$$
$$F = \Omega \, A_b F_b \quad \text{(ASD)} \quad (3.3b)$$

Where ϕ_b =0.75 and Ω=2.0.

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

Step 2—Identify the remaining bolts in the connection, below the "shear bolts," as being bolts to resist the tension component of the bending moment. This is shown in Figure 3.11 (on the right side connection). Assign a force equal to F to each of these bolts, where F is shear capacity of each bolt defined by Equations 3.3(a) and (b) above. Total tension force, T^+, due to applied positive moment is then:

$$T^+ = F\,(N - N_v) \quad (3.4)$$

Where N is the total number of bolts and N_v is the number of shear bolts established by Equation 3.2 above.

Step 3—Establish the compressive force capacity of the floor slab pressing against the column face using the following equation:

$$C^+ = 0.85\, f'_c\, b_{eff}\, a \quad (3.5)$$

Where a is equal to r for the deck parallel to the beam and equal to 0.6 r for the deck perpendicular to the beam. The term r is the total thickness of the floor slab as shown in Figure 3.11 above.

Step 4—If T^+ is less than C^+, then calculate the new value of "a" by making C^+ equal to T^+ given by Equation 3.4. If T^+ is greater than C^+, then use T^+ equal to C^+.

Step 5—Calculate the distance d^+ between the tension and compression forces.
Step 6—Calculate M^+_{max} as:

$$M^+_{max} = (T^+)(d^+) \quad (3.6)$$

b. Calculating M^-_{max}:

Step 1—Calculate the number of bolts, N_v, needed to resist shear force in the connection. Consider the number of bolts located adjacent to the neutral axis to be "shear bolts," carrying only shear and not participating in resisting bending moment. These bolts are shown in Figure 3.11 (on the left side connection) with vertical shear forces acting on them. To find the number of shear bolts needed to carry shear, use the following equations:

$$N_v = V/F \quad (3.7)$$

Where force F is defined by Equations 3.3(a) and (b) for ASD and LRFD formats respectively.

Step 2—Identify the remaining bolts in the connection, below and above the shear bolts, as being bolts to resist tension and compression forces due to applied moment as shown in Figure 3.11 (left side connection). Notice that in this case of negative moment, the floor slab is in tension and its resistance to tension is ignored. Assign a force equal to F to each of these bolts where F is the shear capacity of each bolt defined by Equations 3.3(a) and (b) above. Total tension and compression force, T^- and C^-, as shown in Figure 3.11 (left side connection), is then:

$$T^- = F\,(N - N_v)/2 \qquad (3.8)$$

Where N is the total number of bolts and N_v is the number of shear bolts established by Equation 3.2 above.

Step 5—Calculate distance d^- between tension and compression forces (see Figure 3.11).

Step 6—Calculate M^-_{max} as:

$$M^-_{max} = (T^-)(d^-) \qquad (3.9)$$

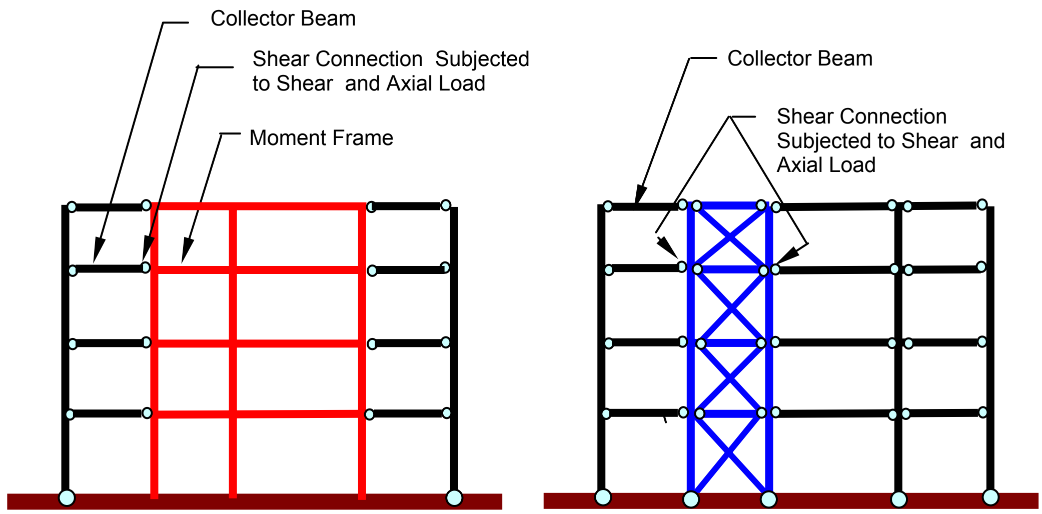


Figure 3.12. Shear Connectors of Collector Beams in Moment Frames and Braced Frames

3.4. Design of Shear Tabs for Combined Shear and Relatively Small Axial Force

In some applications shear connections are subjected to shear and axial load. The situation is very common in shear connections of "collector" beams. The collector beams collect seismic forces of the floor as axial force and transfer the axial force to the support through shear connections, Figure 3.12.

In designing shear tabs 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 shear tab for pure shear following procedures described in earlier sections or use tables for shear tab design in the AISC Manual (AISC 1989 and AISC 2000).
Step 3—Check all six failure modes of the shear tab for combined shear and axial load effects. The failure modes were discussed in Section 2.4 and shown in Figure 2.5. In the following, the equations to be used to check these six failure modes for combined axial load and shear are given for both ASD and LRFD formats consistent with the proposed new AISC specification to be released in 2005.

a. Yielding of the Plate 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 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:

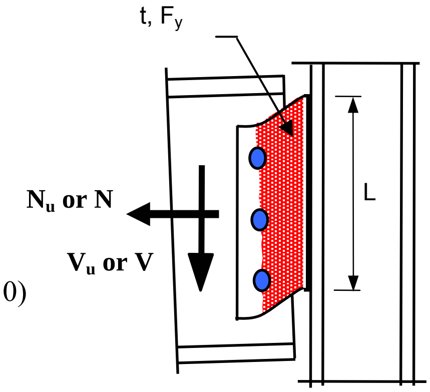


Figure 3.13

$$\left(\frac{V_u}{\varphi_y V_n}\right)^2 + \left(\frac{N_u}{\varphi_y N_n}\right)^2 = 1.0 \qquad \text{(LRFD)} \qquad (3.10)$$

$$\left(\frac{V}{V_n/\Omega_y}\right)^2 + \left(\frac{N}{N_n/\Omega_y}\right)^2 = 1.0 \qquad \text{(ASD)} \qquad (3.11)$$

Where:

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

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

b. Bearing Failure of the Shear Tab 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:

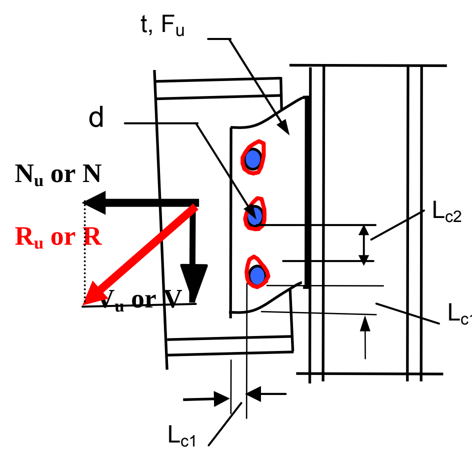


Figure 3.14

$$\left(\frac{V_u}{\phi_{br} V_{br}}\right)^2 + \left(\frac{N_u}{\phi_{br} N_{br}}\right)^2 = 1.0 \qquad \text{(LRFD)} \quad (3.12a)$$

$$\left(\frac{V}{V_{br}/\Omega_{br}}\right)^2 + \left(\frac{N}{N_{br}/\Omega_{br}}\right)^2 = 1.0 \qquad \text{(ASD)} \quad (3.12b)$$

Where:

V_{br} = bearing capacity of bolt group in direction of shear

N_{br} = bearing capacity of bolt group in direction of axial force

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

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

c. Edge Distance Failure in the Plate or in the Beam Web Due to Combined Shear and Axial Load (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 specifications (AISC 2000) or two times the bolt diameter, whichever is greater.

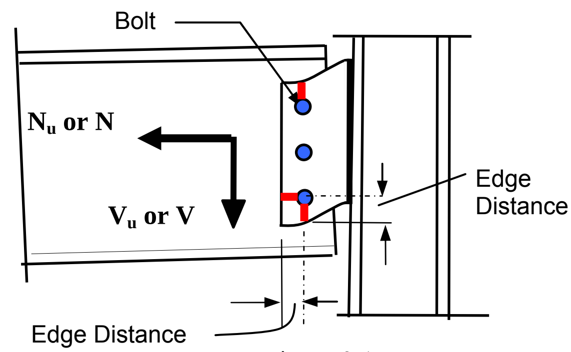


Figure 3.15

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:

$$\left(\frac{V_u}{\varphi_n V_n}\right)^2 + \left(\frac{N_u}{\varphi_n N_n}\right)^2 = 1.0 \quad \text{(LRFD)} \qquad (3.13a)$$

$$\left(\frac{V}{V_n/\Omega_n}\right)^2 + \left(\frac{N}{N_n/\Omega_n}\right)^2 = 1.0 \quad \text{(ASD)} \qquad (3.13b)$$

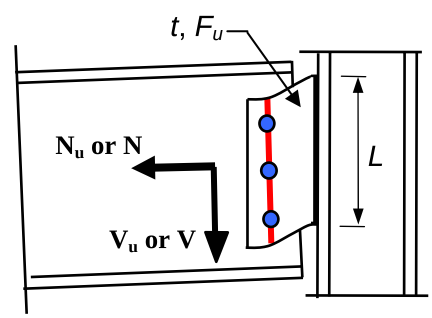


Figure 3.16

Where:
$V_n = 0.60F_u A_{nv}$
$N_n = F_u A_n$
$A_{nv} = A_g - 0.5n(d_b + 1/8 \text{ inch})$
$A_n = A_g - n(d_b + 1/8 \text{ inch})$
$\phi_n = 0.75$ (LRFD) and $\Omega_n = 2.00$ (ASD)

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

e. Fracture of the Bolt Group under Combined Shear and Axial Force (Limit State 5)

The maximum factored axial force (in LRFD) and the maximum allowable axial force (in ASD) can be obtained using the AISC Manual tables for bolt groups subjected to eccentric shear combined with axial load. In order to use the tables, one needs to know the eccentricity of the shear force from the bolt line. For shear tabs, the eccentricity of the shear force was discussed earlier in Chapter 2 and given by equations 2.2 and 2.4 for shear tabs with standard and short-slotted bolt holes respectively.

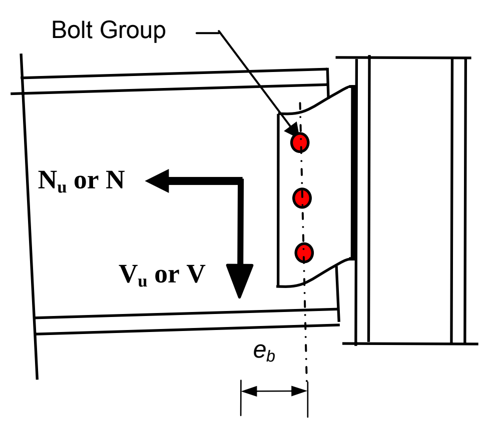


Figure 3.17

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

As discussed in Chapter 2, welds in shear tabs are designed to be stronger than the plate to force the plate to yield and undergo inelastic deformations prior to failure of the welds. The equation that was derived was in the form of:

$$D\ F_w \geq 1.45\ t\ F_y \quad (3.14)$$

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

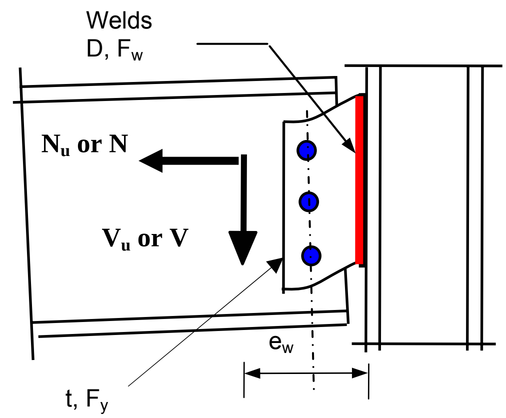


Figure 3.18

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

This limit state was not considered in earlier discussion of the design of shear tabs subjected to pure shear (Section 2.4). The reason is that under pure shear, unless the beam is coped, block shear failure does not seem to become a governing failure mode. However, when axial load is present, it is possible that due to combination of shear and axial force block shear failure occurs in the beam web or even in the shear tab.

To check this failure mode, Rafael Sabelli has proposed the following method (Sabelli 2004), which is included in the following section of this report with his kind permission.

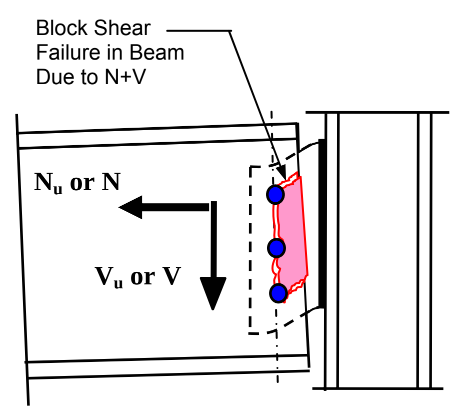


Figure 3.19

Rafael Sabelli's Method to Check Block Shear Failure Under Combined Shear and Axial Force (Sabelli 2004)

Figure 3.20 shows various shear and tension sections used in Sabelli's method to check block shear failure of plates subjected to shear and axial load. The cross sections subjected to shear and tension are either parallel to the shear force or parallel or perpendicular to the resultant R as shown in Figure 3.20. Using the Von Mises yield criterion and applying it to ultimate stress, Sabelli suggests the following equations for block shear failure strength of shear tabs (and beam webs) subjected to simultaneous shear V and axial force N.

The design strength ϕR_n in LRFD and the allowable shear strength R_n/Ω in ASD of a plate subjected to combined shear and axial load for LRFD and ASD, respectively, are as follows:

$$\phi_n R_n = \phi(R_{1n} + R_{2n} + R_{3n}) \quad \text{(LRFD)} \tag{3.15a}$$

$$R_n/\Omega_n = (R_{1n} + R_{2n} + R_{3n})/\Omega_n \quad \text{(ASD)} \tag{3.15b}$$

Where:

$$R_{1n} = \text{Lesser of: } (\gamma A_{vnT} F_u),\ (A_{tT} F_u), \text{ and } (0.6 A_{vT} F_u) \tag{3.16}$$

$$R_{2n} = \gamma A_{vnM} F_u \tag{3.17}$$

$$R_{3n} = \text{Lesser of: } (\gamma A_{vnB} F_u),\ (A_{tB} F_u), \text{ and } (0.6 A_{vB} F_u) \tag{3.18}$$

$$\phi_n = 0.75 \text{ (LRFD) and } \quad \Omega_n = 2.00 \quad \text{(ASD)} \tag{3.19}$$

Using the Von Mises criterion:

$$\sqrt{(R_u \cos\theta_h)^2 + 3(R_u \sin\theta_h)^2} = F_u A_n \tag{3.20}$$

Fracture occurs at: (3.21) (3.22) (3.23)

$$R_u \geq \frac{\phi\, F_u\, A_n}{\sqrt{\cos^2\theta_h + 3\sin^2\theta_h}} = \phi\, F_u A_n \gamma$$

$$R_u \geq \frac{\phi\, F_u\, A_n}{\sqrt{\cos^2\theta_h + 3\sin^2\theta_h}} = \phi\, F_u A_n \gamma$$

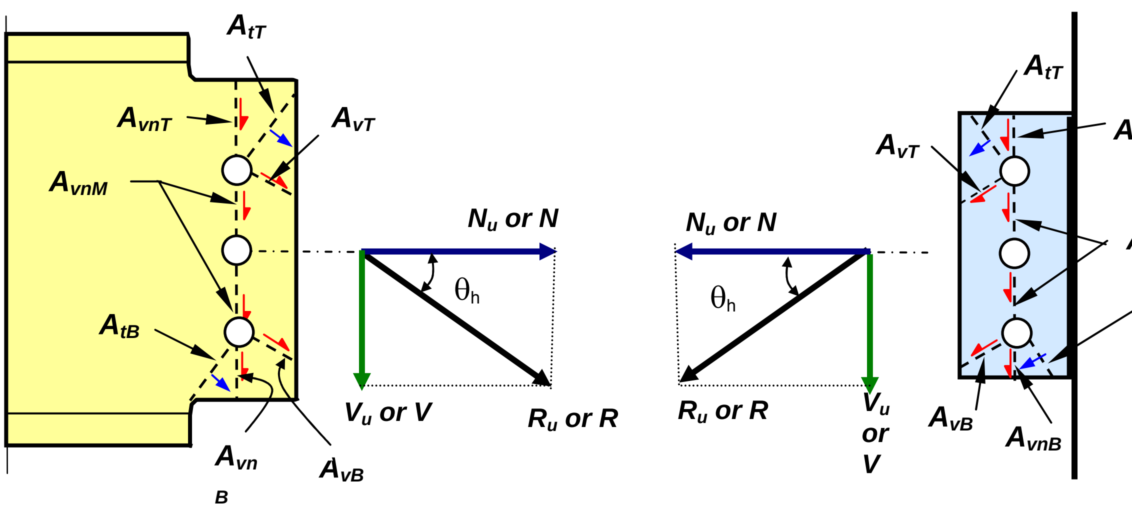


Figure 3.20. Cross Sections Identified in Sabelli's Method for Block Shear Failure under Combined Shear and Axial Force (Sabelli 2004)

3.5. Design of Shear Tabs for Combined Shear and Relatively Large Axial Force

When the axial load applied to a shear connection is large, the standard shear tab with one row of bolts may not be sufficient to transfer both shear and axial load. The situation arises when a collector beam is bringing relatively large seismic force to the braced bay. In this case, instead of typical shear tabs with one row of bolts, a special shear connection has to be designed that satisfies the following conditions as required by the AISC specifications for shear connections:

> *Excerpts from the AISC Manual of Steel Construction, 2000:*
>
> *(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.*

In other words, the shear connection subjected to combined shear and axial load should be designed to have sufficient strength to resist the combined effects of gravity and seismic loads and at the same time the connection should be rotationally flexible enough and have sufficient rotational ductility to be able to tolerate rotations that it will be subjected to. These seemingly competing requirements pose a challenge to the designer. If the axial load applied to the connection is relatively large, a typical shear tab with one row of bolts may not be sufficient to resist the combined shear and axial forces. In this case, it may be necessary to use two or more rows of bolts on the shear tab and even connect the top flanges of the beam to the support to transfer some of the axial force through the flange connection.

Figure 3.21 shows a suggested details for shear connection of collector beams with relatively large axial load. This solution was suggested to the author by Mr. Mason T. Walters, SE, of Forell/Elsesser Engineers, Inc. of San Francisco, California (2004). His kind permission to include his solution in this report for other designers' use is appreciated. The detail is a very practical and economical way of transferring axial load of a simply-supported beam to the column.

Figure 3.22 shows two more suggested detail, dveloped using the concept of detail shown in Figure 3.21. In developing these two additional suggested details, the author, in consultation with steel fabricators and structural engineers, has tried to suggest details that can be economical and easy to fabricate and erect. In addition, in these suggested details due to horizontal and vertical slots in the bolts, a clearly identifiable load path exists to ensure that the assumptions made at the design level for transfer of shear and axial force actually can materialize during a seismic event. The main reason for introducing slotted holes was to facilitate rotation of the connection as much as possible yet be able to transfer the axial force to the support.

The detail shown in Figure 3.22(a) is for a shear tab used in connecting a collector beam to its supporting column. If the axial load in the beam is large, the shear tab can be extended to cover the flat portion of the web and include as many bolts as possible. In addition, part of the axial load can be transferred to the support using the top bolts in the shear tab as shown in Figure 3.22.

The detail shown in Figure 3.22(b) is similar to the one shown in Figure 3.22(a) but has two rows of bolt lines on the shear tab to enable the connection to transfer larger axial loads to the support. In both connections shown in Figure 3.22, the detailing is done such that rotational flexibility of the connection is not compromised and yet relatively large axial loads are transferred to the support.

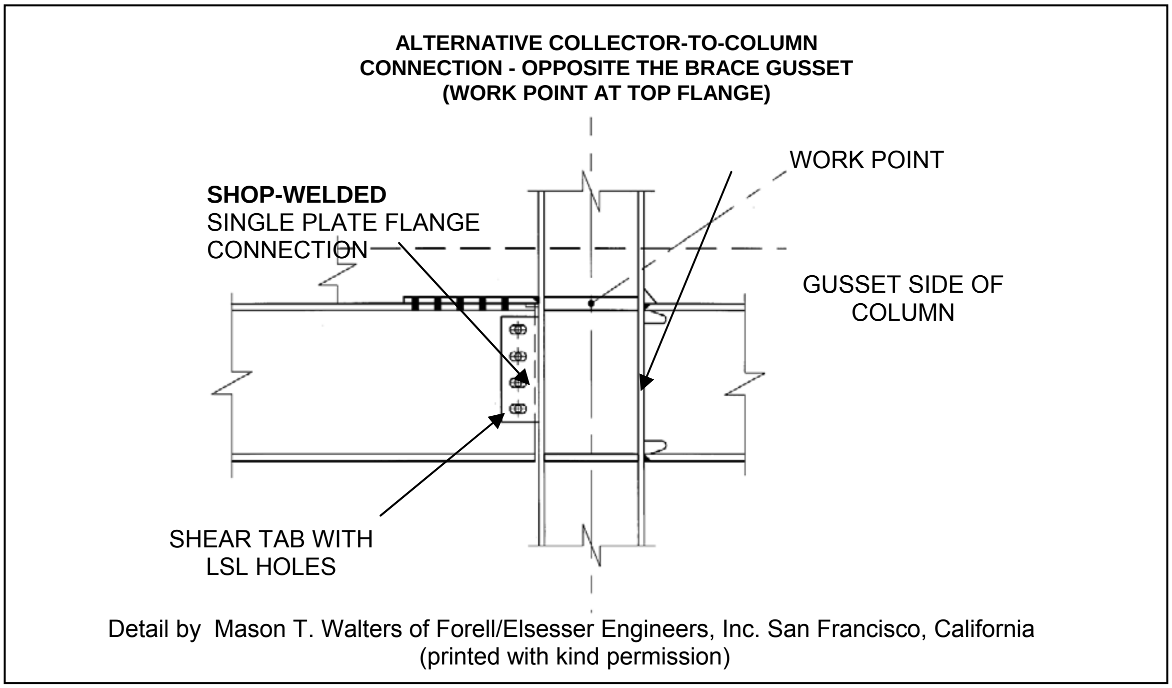


Figure 3.21. A Detail for Shear Connections Subjected to Combined Shear and Axial Load

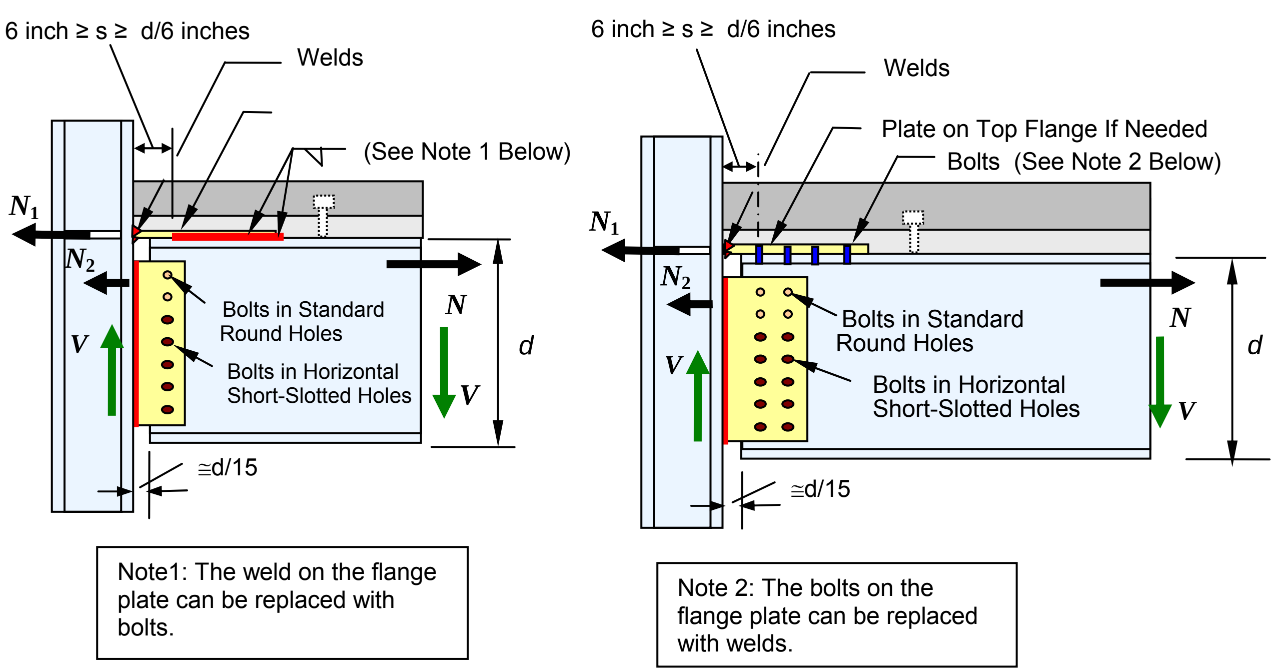


Figure 3.22. Suggested Details for Shear Connections Subjected to Combined Shear and Large Axial Load

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APPENDIX: Numerical Examples

Example 1—Design of a Shear Tab for Direct Shear Due to Gravity Loads

Design a shear tab connection for a W24x94 beam to a W14x120 column flange connection.

Given:
W24x94, A992 beam properties:
t_w = 0.515 in., d= 24.3 in., T= 20.75 in.,
F_y= 50 ksi, F_u= 65 ksi
Shear tab plate properties:
F_y= 36 ksi, F_u= 58 ksi
Bolts: A490N with F_{bv}= 60 ksi
Welds: E70xx with F_w= 70 ksi
Service (nominal) Loads:
Shear due to dead load= 75 kips
Shear due to live load= 55 kips

Solutions:

LRFD Solution	ASD Solution

LRFD Solution

Determine loads:
Establish total factored shear force, V_u:
V_u=1.2V_{DL}+1.6V_{LL}= 1.2x75+1.6x55=178 kips.

a. **Determine shear tab size:**
Cross-sectional area of shear tab is established by checking yield failure mode:
$A_p = V_u/[(\phi)(0.60F_y)]=$
$178/[(0.9)(0.6x36)]=9.15$ in^2
Try PL 5"x1/2"x1'-7" (A=9.5 in^2)
Notice that the depth of shear tab should be less than the flat portion of the web of the beam given as "T" dimension in the AISC Cross Section Tables. In this case, dimension T for a W24x94 is 20.75">19" , O.K.
Use PL 5"x1/2"x1'-7"

Establish yield strength of the shear tab:
V_y= A_p(0.6xF_y)= 19"x0.5"x(0.6x36 ksi)=205 kips.
Use $\phi_y V_y$ as the design shear force in the remainder of this LRFD solution for design of bolts and welds and for checking bearing and net section failure modes.

ASD Solution

Determine loads:
Establish total un-factored shear force, V:
V=V_{DL}+V_{LL}= 75+55=130 kips.

a. **Determine shear tab size:**
Cross-sectional area of the shear tab is established by checking yield failure mode:
$A_p = V/[(1/\Omega)(0.60F_y)]=$
$130/[(1/1.5)(0.6x36)]=9.1$ in^2
Try PL 5"x1/2"x1'-7" (A=9.5 in^2)
Notice that the depth of shear tab should be less than the flat portion of the web of the beam given as "T" dimension in the AISC Cross Section Tables. In this case, dimension T for a W24x94 is 20.75">19" , O.K.
Use PL 5"x1/2"x1'-7"

Establish yield strength of the shear tab:
V_y= A_p (0.6xF_y)= 19"x0.5"x(0.6x36 ksi)=205 kips.
Use V_y/Ω_y as the design shear force in the remainder of this ASD solution for design of bolts and welds and for checking bearing and net section failure modes.

LRFD Solution	ASD Solution

LRFD Solution

b. Design of bolts:

First, design bolts for pure shear, $\phi_y V_y$, to obtain an estimate of the required number of bolts. Then, check the bolt group for combined effects of direct shear, $\phi_y V_y$, and shear due to torque as a result of eccentricity, e_b. Let us use 1-1/8 inch diameter A490N bolts.

Number of bolts= $\phi_y V_y /[(R_e)((\phi_b)(A_b F_{bv})] =$
$(0.9)(205)/[(0.80)(0.75)(3.1425x1.125^2/4)(60)]= 5.2$
Try six 1-1/8"diameter A490N bolts.
Ecc. of shear force from bolt line = N – a – 1 ≥ a inch
= 6 in.–3 in.–1 in.= 2 in.≥ 3 inch   Use e_b=3"

The bolts need to be checked for combined effects of shear force $\phi_y V_y$ applied at a distance of e_b from the bolt line. The check can be done using Table 7-17 (p. 7-38) of the AISC–LRFD Manual (AISC 2000). For shear force applied to six bolts with an eccentricity of e_b=3 inches the table gives a value of $C_{req.}$ = 4.98. The shear capacity $\phi_b V_n$ of the bolts is:

$\phi_b V_n = \phi_b C_{req.} A_b F_{bv}$ =(0.75)(4.98)(0.994in2)(60ksi)
=223k > 0.9x205k. O.K .
Use 6 1-1/8 inch diameter A490N bolts.

c. Check bearing failure mode:

The bolts need to be checked for shear force $\phi_y V_y$:
Calculate Lc_1, the clear edge distance:
Lc_1 = 2" – (1.125+1/16)/2=1.41"
Calculate Lc_2, the clear bolt spacing:
Lc_2 = 3 – (1.125+1/16)=1.81"
Bearing strength of the bolt group in LRFD is:
$\phi_{br} V_{br} = \phi[\Sigma(1.2\ Lc_i\ t\ F_u \leq 2.4\ d\ t\ F_u)]$
Since in this case, for all bolts, $1.2 Lc_i t F_u \leq 2.4 d t F_u$, then;
$\phi_{br} V_{br}$=0.75(1.2x1.41"+5x1.2x1.81")(1/2")(58 ksi)= 273 kips. > 0.9x205 kips OK.

d. Check edge distance:

Edge distance= larger of the AISC Spec. values and 1.5 times diameter of bolt.
Edge distance for 1-1/8" bolt= larger of 2"and 1.5x1.125" = 1.7" ; Use edge distance= 2"
With the edge distance of 2 inches, the total depth of shear tab is 2"+5x3"+2"=19" which is the depth selected in Part "a" above.

ASD Solution

b. Design of bolts:

First, design bolts for pure shear, V_y/Ω_y, to obtain an estimate of the required number of bolts. Then, check the bolt group for combined effects of direct shear, V_y/Ω_y, and shear due to torque as a result of eccentricity, e_b. Let us use 1-1/8 inch diameter A490N bolts.

Number of bolts= $(V_y/\Omega_y)/[(R_e)(A_b F_{bv}/\Omega_b)]$ ==
$(205/1.5)/[(0.80)(3.1425x1.125^2/4)(60)/2]= 5.7$
Try six 1-1/8"diameter A490N bolts.
Ecc. of shear force from bolt line = N – a – 1≥ a inch
= 6 in.–3 in.–1 in.= 2 in.≥ 3 inch   Use e_b=3"

The bolts need to be checked for combined effects of shear force V_y/Ω_y applied at a distance of e_b from the bolt line. The check can be done using Table XI (p. 4-62) of the AISC–ASD Manual (AISC 1989). For shear force applied to six bolts with an eccentricity of e_b=3 inches the table gives a value of $C_{req.}$ =4.99. The allowable shear V_n/Ω_b of the bolts is:

$V_n/\Omega_b = [C_{req.} A_b F_{bv}]/\Omega_b$ =(4.99)(0.994in2)(60ksi)/2
=148k > V_y/Ω_y =205/1.5= 136.8k. O.K .
Use 6 1-1/8 inch diameter A490N bolts.

c. Check bearing failure mode:

The bolts need to be checked for shear force V_y/Ω_y
Calculate Lc_1, the clear edge distance:
Lc_1 = 2" – (1.125+1/16)/2=1.41"
Calculate Lc_2, the clear bolt spacing:
Lc_2 = 3 – (1.125+1/16)=1.81"
Allowable bearing of the bolt group in ASD is:
$V_{br}/\Omega_{br} = [\Sigma(1.2\ Lc_i\ t\ F_u \leq 2.4\ d\ t\ F_u)]/2.0$
Since in this case, for all bolts, $1.2 Lc_i t F_u \leq 2.4 d t F_u$, then;
V_{br}/Ω_{br} =[(1.2x1.41"+5x1.2x1.81")(1/2")(58 ksi)]/2= 182 kips. > V_y/Ω_y = 205/1.5=136.8 kips OK.

d. Check edge distance:

Edge distance= larger of the AISC Spec. values and 1.5 times diameter of bolt.
Edge distance for 1-1/8" bolt= larger of 2"and 1.5x1.125" = 1.7" ; Use edge distance= 2"
With the edge distance of 2 inches, the total depth of shear tab is 2"+5x3"+2"=19" which is the depth selected in Part "a" above.

LRFD Solution | ASD Solution

LRFD Solution

e. Check net areas:

The net area needs to be checked for shear force $\phi_y V_y$:

$\phi V_n = \phi A_n F_u = \phi [A_g - (n/2)(d_h + 1/16")(t)](F_u)$

$= 0.75[19" - (6/2)(1.125" + 1/16" + 1/16")](0.5")(58)$

$= 331$ kips > 0.9x205.2 kips OK.

By inspection, net section of the beam is OK.

f. Design welds:

For A36 steel and E70 electrode;

D = (0.75)t = 0.75x0.5" = 3/8"

Use D=3/8" E70 fillet welds.

g. Check rotational ductility:

$\theta_g = 0.03$ radians

s=0.03d=0.03x18"=1/2"

Use a gap greater than ½" between the end of the beam and face of column.

The designed connection is shown below.

ASD Solution

e. Check net areas:

The net area needs to be checked for shear V_y/Ω_y

$V_n/\Omega_n = A_n F_u/\Omega_n = [A_g - (n/2)(d_h + 1/16")(t)](F_u)]/2$

$= [19" - (6/2)(1.125" + 1/16" + 1/16")](0.5")(58)]/2$

$= 221$ kips > 0.9x205.2 kips OK.

By inspection, net section of the beam is OK.

f. Design welds:

For A36 steel and E70 electrode;

D = (0.75)t = 0.75x0.5" = 3/8"

Use D=3/8" E70 fillet welds.

g. Check rotational ductility:

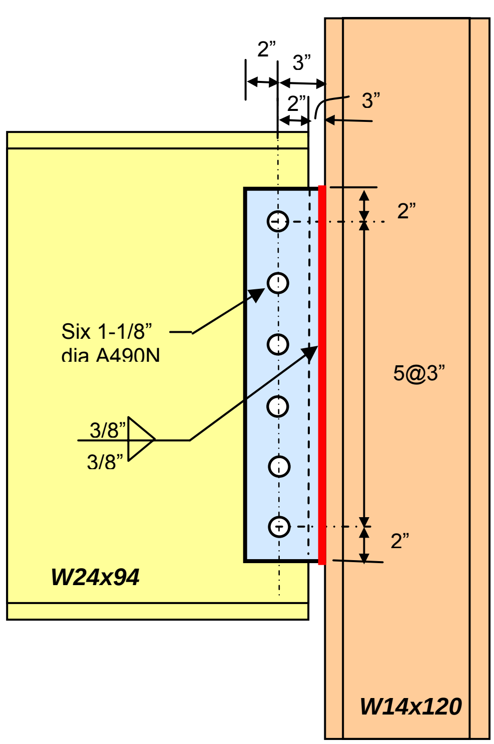
$\theta_g = 0.03$ radians

s=0.03d=0.03x18"=1/2"

Use a gap greater than ½" between the end of the beam and face of column.

The designed connection is shown below.

LRFD Solution | ASD Solution



Final Design of Shear Tab in Example 1

Example 2—Design of a Shear Tab for Combined Shear Force and Axial Load

Check the shear tab designed in Example 1 above for the shear and axial loads given below. The shear tab connects a W24x94 beam to a W14x120 column flange. Other properties are as before and given below:

Given:
W24x94, A992 beam properties:
t_w = 0.515 in., d = 24.3 in., T = 20.75 in. ,
F_y = 50 ksi, F_u = 65 ksi
Shear tab plate properties:
F_y = 36 ksi, F_u = 58 ksi
Bolts: A490N with F_{bv} = 60 ksi
Welds: E70xx with F_w = 70 ksi
Applied Service (nominal) Loads:
Shear due to dead load = 75 kips
Shear due to live load = 55 kips
Axial force in the beam due to earthquake = 160 kips

Solutions:

LRFD Solution

Determine loads:
Total factored combined shear force, V_u, and axial force N_u, using the ASCE-7 load combination is:
V_u=1.2V_{DL}+V_{LL}+V_E =1.2x75+55 =145 kips.
N_u= N_E = 160 kips.

a. Check yielding of shear tab under V_u+N_u:
$(V_u/\phi_y V_y)^2 + (N_u/\phi_y N_y)^2 \leq 1.0$
V_u=145 k , N_u=160 k
$\phi_y V_y$=0.9x0.5”x19”x0.6x36 ksi= 185 kips
$\phi_y N_y$ =0.9x0.5”x19”x36 ksi =308 kips
$(145k/185k)^2+(160k/308k)^2 = 0.78^2+0.52^2$
= 0.94 ≤ 1.0 OK.

b. Check bearing failure mode under V_u+N_u:
The equation to be used is:
$(V_u/\phi_{br} V_{br})^2 + (N_u/\phi_{br} N_{br})^2 \leq 1.0$
Where V_u = 145 and N_u = 160 k,
$\phi_{br} V_{br} = \phi[\Sigma(1.2\ Lc_i\ t\ F_u \leq 2.4\ d\ t\ F_u)\ ]$
Since in this case, for all bolts, $1.2Lc_i tF_u \leq 2.4dtF_u$, then;
$\phi_{br} V_{br}$=0.75(1.2x1.41”+5x1.2x1.81”)(1/2”)(58)=273k.
$\phi_{br} N_{br}$=0.75(6x1.2x1.41”)(1/2”)(58)=220k, therefore;
$(145/273)^2 + (160/220)^2 = 0.81 < 1.0$ O.K.

ASD Solution

Determine loads:
Total un-factored shear force, V, using ASCE-7 load combination is:
V=V_{DL}+V_{LL}+V_E =75+.75x55+.75(.7x0.0) =116 kips.
N= 0.75(0.7N_E)=(0.75x0.7x160)= 84 kips.

a. Check yielding of shear tab under V+N:
$[V/(V_y/\Omega)_y]^2 + [N)/(N_y/\Omega_y)]^2 \leq 1.0$
V=116 k, N=84 k
V_y/Ω_y =(0.5x19x0.6x36 ksi)/1.5=137 k
N_y/Ω_y = (0.5x19x36 ksi)/1.5=228 k
$(116/137)^2+(84/228)^2 = 0.84^2+0.36^2$
= 0.91 < 1.0 OK.

b. Check bearing failure mode under V+N:
The equation to be used is:
$[V/(V_{br}/\Omega_{br})]^2+[N/(N_{br}/\Omega_{br})]^2 \leq 1.0$
Where V = 116 and N = 84 k,
$V_{br}/\Omega_{br} = [\Sigma(1.2\ Lc_i\ t\ F_u \leq 2.4\ d\ t\ F_u)\ ]/\Omega_{br}$
Since in this case, for all bolts, $1.2Lc_i tF_u \leq 2.4dtF_u$, then;
V_{br}/Ω_{br}=(1.2x1.91”+5x1.2x1.81”)(1/2”)(58)/2=182k.
N_{br}/Ω_{br} =(6x1.2x1.41”)(1/2”)(58)/2=147k, therefore;
$(116/182)^2 + (84/147)^2 = 0.76 < 1.0$ O.K.

LRFD Solution | ASD Solution

LRFD Solution	ASD Solution

LRFD Solution

c. Check edge distance:

Edge distance= larger of the AISC Spec. values and 1.5 times diameter of bolt.
Edge distance for 1-1/8" bolt= larger of 2"and 1.5x1.125" = 1.7" ; Use edge distance= 2"
With the edge distance of 2 inches, the total depth of shear tab is 2"+5x3"+2"=19" which is the depth selected in Part "b" above.

d. Check net area fracture under V_u+N_u:

The equation to be used is:

$(V_u/\phi_n V_n)^2 + (N_u/\phi_n N_n)^2 \leq 1.0$

$\phi_n V_n$= 0.75[0.5x19-(6/2)(1.125+0.125)(0.5)](0.6x58)]
=199 kips

$\phi_n N_n$= 0.75[0.5x19-(6)(1.125+0.125)(0.5")](58)
=250 kips

$(154/199)^2 + (160/250)^2 = 0.77^2 + 0.64^2 = 1.001 \approx 1.0$ O.K.

By inspection, net section of beam is OK.

e. Check bolt failure under V_u+N_u:

Check the bolt group for the combined effects of direct shear V_u =145 kips located at a distance of e_b from the bolt line and axial force N_u =160 kips acting through the center of the bolt group.
Eccentricity of shear force from the bolt line as calculated in Example 1 above = N – a – 1 inch ≥ a in. = 6 in. – 3 in. – 1 in. = 2" ≥ 3 in. Use e_b =3 inches
The angle between vertical load V_u and resultant R_u is θ where:
Tan θ = N/V = 160/145 = 1.1, θ = 48 degrees.
Using Table 7-17 (pp. 7-40 and 7-41) of the AISC–LRFD Manual (2000):
For shear force applied at an eccentricity of e_b=3 inches and above θ, the tables give a value of $C_{req.}$ = 4.89. The capacity ϕP_n of the bolt group is:

$\phi P_n = \phi C_{req} A_b F_{bv}$= 0.75x4.89x0.994x60
= 219 kips

The combined resultant of shear and axial force is:

$P_u = (V_u^{\,2} + N_u^{\,2})^{0.5}$

$P_u = (145^2 + 160^2)^{0.5}$ = 216 kips. < 219 kips O.K.

Use 6 1-1/8 inch diameter A490N bolts.

ASD Solution

c. Check edge distance:

Edge distance= larger of the AISC Spec. values and 1.5 times diameter of bolt.
Edge distance for 1-1/8" bolt= larger of 2"and 1.5x1.125" = 1.7" ; Use edge distance= 2"
With the edge distance of 2 inches, the total depth of shear tab is 2"+5x3"+2"=19" which is the depth selected in Part "a" above.

d. Check net area fracture under V+N:

The equation to be used is:

$[V/(V_n/\Omega_n)]^2 + [N/(N_n/\Omega_n)]^2 \leq 1.0$

V_n/Ω_n = [0.5x19-(6/2)(1.125+0.125)(0.5)](0.6x58)]/2
=133 kips

N_n/Ω_n = [0.5x19-(6)(1.125+0.125)(0.5")](58)/2
=167 kips

$(116/133)^2 + (84/167)^2 = 0.87^2 + 0.50^2 = 1.003 \approx 1.0$ O.K.

By inspection, net section of beam is OK.

e. Check bolt failure under V+N:

Check the bolt group for the combined effects of direct shear V =116 kips located at a distance of e_b from the bolt line and axial force N =84 kips acting through the center of the bolt group.
Eccentricity of shear force from the bolt line as calculated in Example 1 above = N – a – 1 inch ≥ a in. = 6 in. – 3 in. – 1 in. = 2" ≥ 3 in. Use e_b =3 inches
The angle between vertical load V and resultant R is θ where:
Tan θ = V/N = 116/84 = 1.38, θ = 54 degrees.
Using "Alternate Method 2" of the ASD Manual (p. 4-57), you can establish the allowable axial force:
For shear force applied at an eccentricity of e_b=2 inches, Table XI (p. 4-62) of the AISC–ASD Manual gives a value of C_o as

C_o =4.99, Then, A= C_{max}/C_o=6/4.99=1.2, and;

$C_a = C_o A/(\sin\theta + A \cos\theta) \geq C_o$
= 4.99x1.2/(sin 54+1.2xcos 54)=3.95, Use C_a=4.99

Value of allowable resultant P_a on the connection is:

P_a=4.99x0.994in^{2}x60ksi/2= 149

The combined resultant of shear and axial force is:

$P_u = (V^2 + N^2)^{0.5} = (116^2 + 84^2)^{0.5}$ = 143 < 149 O.K.

Use 6 1-1/8 inch diameter A490N bolts.

LRFD Solution	ASD Solution

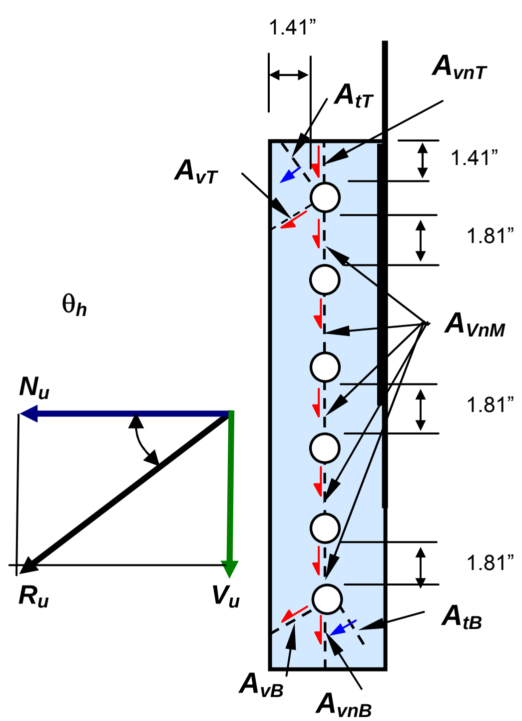
LRFD Solution | ASD Solution

LRFD Solution

f. Design welds:
For A36 steel and E70 electrode;
D = (5/8)t =5/8x0.5"= 5/16
Use D=3/8" E70 fillet weld.

g. Check block shear failure:
Since thickness of web of beam is 0.515" and greater than thick ness of shear tab which is 0.5" and beam is grade 50 compared to shear tab which is A36 steel, only shear tab will be checked for block shear failure. The method used in this solution is the Sabelli's method which was explained in the body of this report in Chapter 2.

$\phi_n R_n = \phi(R_{1n} + R_{2n} + R_{3n})$
where;
R_{1n} = Lesser of: $(\gamma A_{vnT} F_u)$, $(A_{tT} F_u)$, and $(0.6 A_{vT} F_u)$
$R_{2n} = \gamma A_{vnM} F_u$
R_{3n} = Lesser of: $(\gamma A_{vnB} F_u)$, $(A_{tB} F_u)$, and $(0.6 A_{vB} F_u)$
$\phi_n = 0.75$
$\theta_h = \tan^{-1}(V_u / N_u) = \tan^{-1}(154 / 160) = 43.9$ degrees
$\gamma = 1/(\cos^2\theta_n + 3\sin^2\theta_n)^{0.5} = 1/(0.72^2 + 3 \times 0.69^2)^{0.5}$
$= 0.72$



ASD Solution

f. Design welds:
For A36 steel and E70 electrode;
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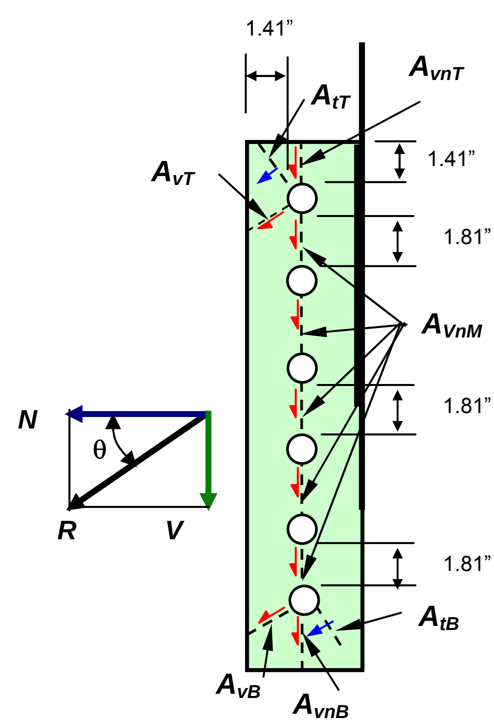
Since thickness of web of beam is 0.515" and greater than thick ness of shear tab which is 0.5" and beam is grade 50 compared to shear tab which is A36 steel, only shear tab will be checked for block shear failure. The method used in this solution is the Sabelli's method which was explained in the body of this report in Chapter 2.

$R_n/\Omega_n = (R_{1n} + R_{2n} + R_{3n}) / \Omega_n$
where;
R_{1n} = Lesser of: $(\gamma A_{vnT} F_u)$, $(A_{tT} F_u)$, and $(0.6 A_{vT} F_u)$
$R_{2n} = \gamma A_{vnM} F_u$
R_{3n} = Lesser of: $(\gamma A_{vnB} F_u)$, $(A_{tB} F_u)$, and $(0.6 A_{vB} F_u)$
$\Omega_n = 0.75$
$\theta_n = \tan^{-1}(V_u / N_u) = \tan^{-1}(84/116) = 36$ degrees
$\gamma = 1/(\cos^2\theta_n + 3\sin^2\theta_n)^{0.5} = 1/(0.81^2 + 3 \times 0.59^2)^{0.5}$
$= 0.77$



LRFD Solution | ASD Solution

LRFD Solution	ASD Solution
$\gamma A_{vnT} F_u$=0.72x(1.41"/cos θ)(58ksi)= 81 kips	$\gamma A_{vnT} F_u$=0.77x(1.41"/cos θ)(58ksi)= 87 kips
$A_{tT} F_u$= (1.41")(58ksi)= 82 kips	$A_{tT} F_u$= (1.41")(58ksi)= 82 kips
$0.6A_{vT} F_u$=0.6x(1.41"/cos θ)(58ksi)=68 kips	$0.6A_{vT} F_u$=0.6x(1.41"/cos θ)(58ksi)=68 kips
Therefore, R_{1n} = 68 kips	Therefore, R_{1n} = 68 kips
$R_{2n} = \gamma A_{vnM}$ = 0.72x4x1.81x58ksi=302 kips	$R_{2n} = \gamma A_{vnM}$ = 0.77x4x1.81x58ksi=323 kips
R_{3n} = Lesser of: $(\gamma A_{vnB} F_u)$, $(A_{tB} F_u)$, and $(0.6A_{vB} F_u)$	R_{3n} = Lesser of: $(\gamma A_{vnB} F_u)$, $(A_{tB} F_u)$, and $(0.6A_{vB} F_u)$
Due to symmetry of the connection, $R_{3n} = R_{1n}$,	Due to symmetry of the connection, $R_{3n} = R_{1n}$,
R_{3n} = 68 kips.	R_{3n} = 68 kips.
$\phi R_n = \phi(R_{1n} + R_{2n} + R_{3n})$ =0.75(68+302+68)	$R_n/\Omega_n = (R_{1n} + R_{2n} + R_{3n})/\Omega_n$=(68+323+68)
=438 k > 226 kips O.K.	=459 k > 226 kips O.K.

LRFD Solution	ASD Solution

About the author…

Abolhassan Astaneh-Asl, Ph.D., P.E., is a professor of structural engineering at the University of California, Berkeley. He is the winner of the 1998 T. R. Higgins Lectureship Award of the American Institute of Steel Construction.

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 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. Since1995, 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. Since 2004 he has been doing research on the Middle East earthquake hazard reduction.

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