Finite Element Analysis of Single Plate Shear Connections

by

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(Abstract)

There have been several design models for single plate shear connections in the past 20 years. The current design model states that the bolt shear rupture strength of a connection is a function of the number of bolts and the a-distance, which is the distance from the weld line to the bolt line. The evaluation of this design model demonstrates inconsistent predictions for the strength of the connection.

The finite element program ABAQUS was used throughout the research to study single plate shear connections. Finite element analyses included model verification and investigations of parameters, including the effect of a-distance, plate thickness, plate material, and the position of a connection with respect to a beam neutral axis. In addition, double-column bolt connections were studied.

The results show that bolt shear rupture strength of a connection is not a function of the a-distance. Plate materials and thicknesses that do not satisfy ductility criteria result in connections with significant horizontal forces at the bolts. This horizontal force reduces the shear strength of a bolt group and creates a moment that must be considered in design. The magnitude of the force depends on the location of the bolt with respect to the beam neutral axis. A new design model for single plate shear connections with bolts in a single column is proposed.

It was found that in double-column bolt connections, force redistribution among the bolt columns occurs. Force redistribution does not occur when thick plates are used, resulting in bolts in the outer column (from the support) fracturing while bolts in the inner column resist much less force. Further study is needed for double-column configurations.

The study of plate behavior shows that the shear stress distribution when a plate reaches the strain hardening stage is not constant throughout the cross section. A relationship for calculating plate shear yielding strength based on this shear distribution is proposed.

KHANTI PARAMUM TAPO

Patience is the Absolute Exertion

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Finite Element Analysis of Single Plate Shear Connections

Chapter I

Introduction

1.1 A Single Plate Shear Connection

A single plate shear connection, also known as shear tab in the United States or fin plate in other countries (Australia, Japan, and United Kingdom), is a connection that consists of a plate welded on both sides to the supporting member, and field bolted to the supported member with single shear plane, single or double column, bolts. Materials usually used in this type of connection are A36 steel for the plate, ASTM A325 or A490 bolts in either rounded or slotted holes, and E70 electrode for SMAW welding. Its simplicity in erection and the economy in both material and construction cost have gained the connection its popularity for many years. A single plate shear connection can be used to connect the supported member to the flange of a steel column as illustrated in Figure 1.1 or the web of a steel girder with the supported member coped, as shown in Figure 1.2, or intact. It is generally not used to connect the supported member to the web of a column due to difficulties in erection unless the size of the column is at least 18-24 in. deep, which is required for ease of erection.

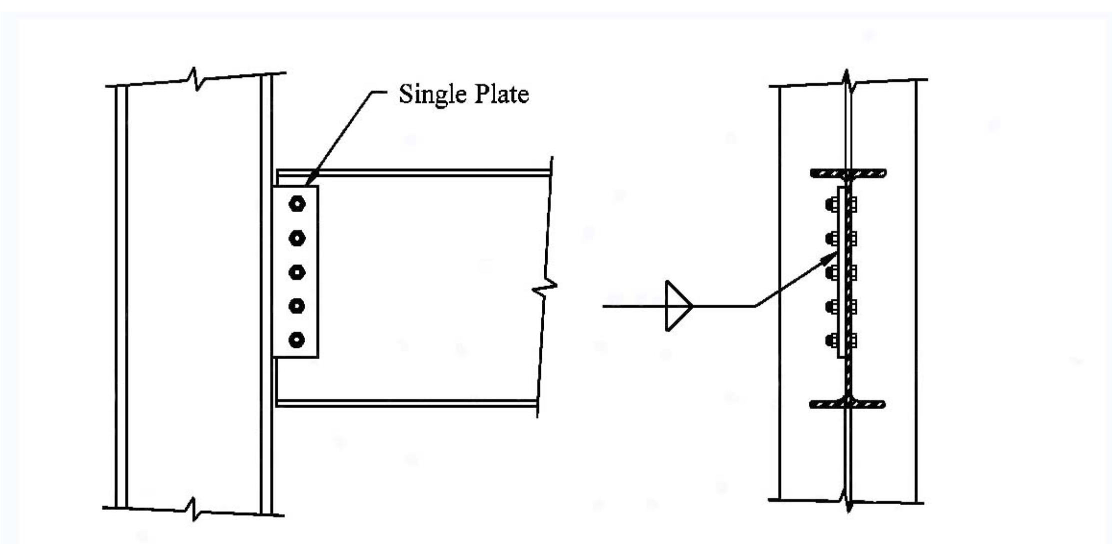


Figure 1.1 A Typical Beam-to-Column Single Plate Shear Connection

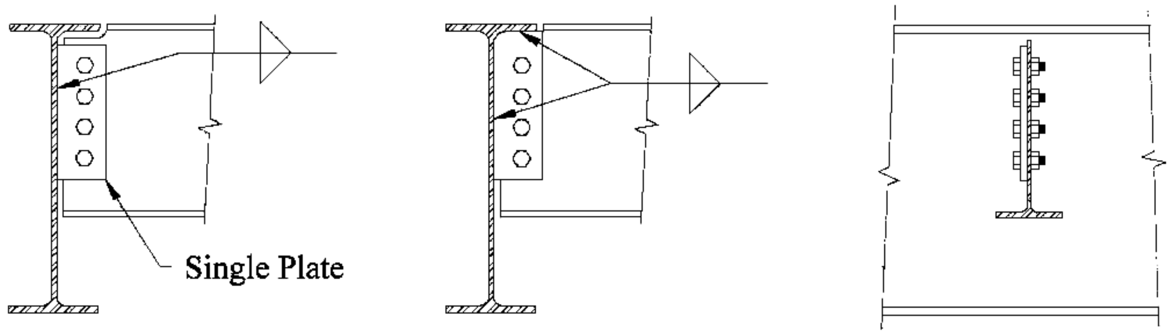


Figure 1.2 A Typical Coped Beam-to-Girder Single Plate Shear Connection

1.2 Design Considerations for A Single Plate Shear Connection

According to the AISC Load and Resistance Factor Design Specifications (AISC, 1999), any shear connection must be designed to carry the required factored gravity load, and accommodate the simple-beam end rotation with inelastic rotation permitted. It is further discussed in the Specification Commentary that such a connection can be considered as not having flexural strength and thus can be interpreted as a simple shear connection when it transmits moment less than $0.2M_{p,beam}$ and a beam end rotation of 0.02 radian, otherwise it is a partially restrained connection. A single plate shear connection, normally treated as a simple shear connection, is able to sustain a small amount of moment due to its rotational stiffness. However, the inherent connection stiffness leads to ductility considerations. The ductility required in a single plate shear connection is provided by means of plate yielding and bearing deformation of the bolt holes. To achieve the ductility, the geometry and thickness of the plate must be such that the plate will yield, the bolt group will rotate, and/or the bolt holes will elongate prior to the failure of the welds or bolts. In general, requirements for designing a connection are that the connection must be able to resist a factored gravity load, and that the connection must be able to provide sufficient rotation.

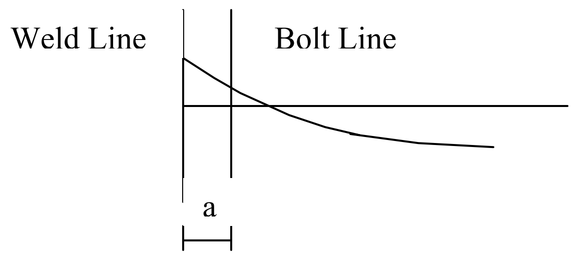


Figure 1.3 The a-distance and Eccentricity of Connection

Characteristics that make a single plate shear connection stand out from the rest of the shear connections are moments at the bolt group and the welds. Current design procedures for a single plate shear connection in both the Allowable Stress Design and Load and Resistance Factor Design approaches are based on the research developed by Astaneh and his research team (1988, 1989, 1990, 1993, 2002). The design model indicates that eccentricities are a function of the number of bolts in the connection, and the distance measured from the weld line to the bolt line, also known as the "a-distance" as demonstrated in Figure 1.3.

In general, the following aspects are considered in the design of a single plate shear connection in accordance with AISC (Manual, 2001):

1. Shear yielding of plate.
2. Shear rupture of plate.
3. Block shear of plate and coped beam if applicable.
4. Bearing/ tear-out of plate and coped beam if applicable.
5. Buckling of plate.
6. Eccentric shear on bolt group.
7. Eccentric shear on welds.

The bolt group and welds are considered to have moment imposed upon them and must be designed for such moment. The design moment strength of a bolt group, by means of using eccentricities, was developed by Crawford and Kulak (1968, 1971) and that for welds was developed by Butler et al. (1972) and Lesik and Kennedy (1990).

In addition to the required design calculations, Astaneh makes the following recommendations in the above cited references. The geometry of the connection is defined in Figure 1.4.

1. The distance between the weld line and the bolt line, or the a-distance, must be between 2.5 in. and 3.5 in.
2. Edge distances of the plate, L_{eh}, should be equal to or greater than 1.5 times the diameter of the bolt. The vertical edge distance of the lowest bolt should be at least 1.5 in.
3. The spacing between each bolt is 3 in.
4. To ensure that ductility will be achieved via bolt plowing, the thickness of the plate should be less than or equal to one-half the diameter of the bolt plus 1/16 in.
5. To ensure that the plate will yield before the welds fracture for the A36 steel plate associated with a 70 ksi electrode, the weld size should be equal to or greater than three-quarters of the plate thickness.
6. To prevent local buckling of the plate, the plate thickness should satisfy $\frac{L}{234}\sqrt{\frac{F_y}{K}} \geq \frac{1}{4}$ in., where K is a buckling coefficient.
7. A bolt configuration must be single column and the number of bolts must be between two and nine.

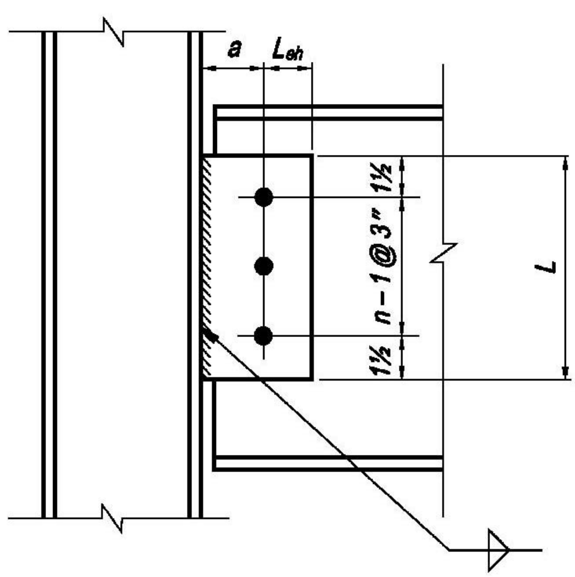


Figure 1.4 A Single Plate Shear Connection Configuration

Currently, there is no AISC design procedure for a single plate shear connection with a double column bolt configuration as shown in Figure 1.5.

1.3 Outline of the Dissertation

Chapter II contains the summary of completed research regarding single plate shear connections with standard or slotted holes, and with both rigid and flexible supports. Chapter III consists of the analysis of the current design procedure and the method employed in the research. Chapter IV presents the construction of the finite element model. Chapter V includes the results of all simulations carried out during the research and the analyses of related parameters to study the behavior of single plate shear connections. Lastly, Chapter VI summarizes the knowledge obtained from the research, including a proposed design method for single plate shear connections with single-column bolt configurations.

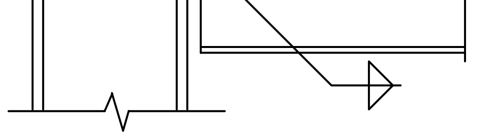


Figure 1.5 Double-Column Bolt Single Plate Shear Connection.

Chapter II

Literature Review

2.1 Development of the Design Procedure of Eccentricity of a Single Plate Shear Connection

A single plate shear connection has the capacity to resist a small amount of moment because of its relatively high stiffness, as compared to other shear connection types, as found in research conducted in the past. The moment imposed on the connection due to its stiffness is always represented in terms of eccentricity, $e = M/V$, where M is the moment at the support and V is the shear at the same location. The bolt group and welds are designed considering this eccentricity. The inherent connection stiffness, however, requires consideration of ductility or rotation capacity, that is, whether or not the connection will allow a beam to rotate sufficiently to reach its plastic moment capacity.

According to Lipson (1968), as presented in the paper by Richard et al. (1980), the moment introduced to a connection is dependent upon the following factors:

1. The number and size of the bolts and their configuration.
2. The thickness of the plate and/or beam web.
3. The beam span-to-depth ratio.
4. The loading type.
5. The relative flexibility of the supporting member.

Thus, the moment at the connection and its required rotation depend on stiffness of the supported member, the connection itself, and the supporting member.

Richard et al. (1980), following the above indication by Lipson, conducted research regarding this type of connection at the University of Arizona in the late 1970s. In Richard's first study, the beam-line method was adopted in the development of the design of the connection. To obtain the moment-rotation curves used in the beam-line theory, Richard first conducted a series of experiments on double plates connected by single-shear bolts to determine the deformation of the plates that is affected by the relationship of the plate and the bolts. Upon the results from the tests, Richard suggested that the bolt diameter to A36 plate (or beam web) ratios in Table 2.1 be used to ensure the connection ductility.

Table 2.1 Recommended A36 Plate Thicknesses by Richard et al. (1980)

Bolt Size (in.)	PL Thickness (in.)	
	A325	A490
3/4	3/8	1/2
7/8	7/16	5/8
1	9/16	11/16

Following the results obtained from Richard's double-plate tests, finite element models for connections with a cantilever beam were developed using a program written by Richard called INELAS. The moment-rotation curves obtained from the simulations illustrated that the moment on a connection depended on the shear force when the length of a modeled cantilever beam was less than the height of a bolt pattern, and became independent of the shear force when the length of the cantilever was greater. From the finite element results, an equation for predicting the moment in the connection was developed. This equation was further modified into a design equation to accommodate the design procedure. Seven experimental tests involving a cantilever beam with its length equal to the height of the connection were also conducted to verify the curves produced by the moment equation.

Five full-scale simple beam tests of 3-, 5-, and 7-bolt connections were then conducted to be compared with the results from the finite element program and the moment predicted by the proposed design method. The specimens for the tests consisted of an A36 plate with various numbers of A325 bolts forming a single plate shear connection at one end of the A36 beam, and a roller as a simple support at the other end. The beam was then loaded to 1.5 times its working stress design capacity with a concentrated load at mid-span. It is noted that the full-scale tests conducted by Richard were all non-destructive tests.

The equation for calculating a parameter e/h, which yields the eccentricity of the connection to be used in calculating the moment at the weld line, was obtained from finite element analysis. With the number of bolts required, the plate thickness chosen with a variation of $\pm$ 1/16 in. from the supported beam web as recommended by Richard, and the beam L/d ratio known, the e/h ratio of the connection is calculated from:

$$e/h = \left(e/h\right)_{ref}\left(\frac{n}{N}\right)\left(\frac{S_{ref}}{S}\right)^{0.4} \tag{2.1}$$

where

e	=	connection eccentricity
$(e/h)_{ref}$	=	0.06L/d – 0.15 when L/d ≥ 6
	=	0.035L/d when L/d < 6
n	=	number of bolts
N	=	5 for 3/4-in. and 7/8-in. bolts, and 7 for 1-in. bolts
S_{ref}	=	100 for 3/4 –in. bolts, 175 for 7/8-in bolts, and 450 for 1-in. bolts
S	=	section modulus of beam

where

h	=	(n-1)p
p	=	pitch

The next step is to calculate the moment at the weld line:

$$M = V\left(e + a\right) \tag{2.2}$$

where

V	=	beam shear
a	=	distance from the bolt line to the weld line

It should be noted that the bolt group is assumed to have no moment acting upon it. More importantly, the equation for calculating eccentricity proposed by Richard can never give a negative value, which implies that the eccentricity for calculating moment at the weld line is always greater than the a-distance. The design tables based on Richard's research were later provided by Young and Disque (1981), with further discussion about the eccentricity coefficients for concentrated loads by Griffiths (1982).

Richard et al. (1982, 1985) and Hormby et al. (1984) conducted further research regarding this connection with the A307 bolt type, A572 Grade 50 beams, and composite structures. Because the popularity of A307 bolts no longer exists, the design method regarding this bolt type will not be discussed herein. However, for the use of a Grade 50 beam, the eccentricity of a connection is slightly modified to:

$$e_{50} = e_{36}\left(\frac{36}{50}\right) \tag{2.3}$$

where

e_{36} = eccentricity of the connection calculated using Equation 2.1

For composite construction, the equation for calculating eccentricity was also modified to:

$$e/h = \left(e/h\right)_{ref}\left(\frac{n}{N}\right)\left(\frac{S_{ref}}{S_g}\right)^{0.4}\left(\frac{S_{gnp}}{S_g}\right)^{0.5}\frac{36}{F_{yg}} \tag{2.4}$$

where

$(e/h)_{ref}$ = 0.06 L/d – 0.15

d = beam depth from the top of the concrete slab including the thickness of a cover plate if applicable

S_g = governing section modulus

S_{gnp} = governing section modulus with no cover plates

F_{yg} = governing minimum steel yield stress except for sections where concrete stress governs (F_{yg} = 36)

The governing section moduli, S_g and S_{gnp}, are calculated according to the method in Chapter I of the ASD Manual (AISC, 1989).

In the late 1980s, after the implementation of the Load and Resistance Factor Design procedure, Astaneh and his colleagues (Astaneh et al. 1988, Astaneh et al. 1989, Astaneh 1989) conducted research at the University of California, Berkeley. Astaneh stated the importance of the shear-rotation relationship of the connection upon which he based his research.

Curves showing the relationship between shear and rotation of A36 simply supported beams varying from W16 to W33 with different L/d ratios were constructed using a computer program developed by Astaneh. The curves consist of three distinctive portions: elastic, inelastic, and an extra strain-hardening portion. The elastic and inelastic regions are the typical portions in the conservative elastic-perfectly-plastic curve. The elastic behavior ends when the bending moment in the beam reaches its in-span yielding capacity, M_y, whereas the inelastic region ends when the moment in the beam reaches its plastic capacity, M_p. The strain hardening stage was added to the elastic-perfectly-plastic curve to represent the extra beam capacity developed by the hardening behavior.

The curve for a beam with an L/d ratio of 25 was then selected to develop the load-rotation relationship for five full-scale tests of 3-, 5-, and 7-bolt connections. Each test consisted of a cantilever beam connected by various single-column bolt configurations of a single plate shear connection to a column. The beam was then loaded at two locations by two actuators that were controlled by computer. One actuator controlled the amount of load, while the other controlled the amount of rotation of the beam so that the beam would behave according to the selected load-rotation curve. Unlike the tests that had been previously conducted by Richard, these tests were carried on until the structure failed. The essence of a beam being able to reach its plastic moment capacity as well as the ductility of the structure was also considered in the research. However, it must be noted that short cantilever beams were used to simulate the behavior of simply-supported beams. In addition, neither the actual W18x55 A36 beam used with the 3- and 5-bolt connections nor the W24x84 A36 beam used with the 7-bolt connection with L/d ratio equal to 25 was able to reach the loadings carried by the connections. In reality, the beams would fail under bending moment long before connection failure. The properties and results of each test by Astaneh are summarized in Tables 2.2 and 2.3.

Table 2.2 Properties of Test Specimens Used by Astaneh et al. (1989)

No	No. of Bolts	Dia. of Bolts (in.)	Type of Bolts	Plate Dimensions (in. x in. x in.)	Edge Distance (in.)	Weld Size (in.)	Beam	Beam Material	Plate Material
1	3	3/4	A325-N	3/8x4-1/4x9	1-1/2	1/4	W18x55	A36	A36
2	5	3/4	A325-N	3/8x4-1/4x15	1-1/2	1/4	W18x55	A36	A36
3	7	3/4	A325-N	3/8x4-1/4x21	1-1/2	1/4	W24x84	A36	A36
4	3	3/4	A490-N	3/8x3-7/8x8-1/4	1-1/8	7/32	W18x55	Gr.50	A36
5	5	3/4	A490-N	3/8x3-7/8x14-1/4	1-1/8	7/32	W18x55	Gr.50	A36
6	9	3/4	A490-N	3/8x4-1/4x27	1-1/2	9/32	W24x84	N/A	A36

Note: Test number 6 was not presented in Astaneh et al. (1989); it was presented in Astaneh et al. (1993).

Table 2.3 Astaneh's Test Results

No.	Failure Mode	Shear Force (kips)	Beam End Rotation (rad)	Moment at Bolt Line (kip-in.)	Moment at Weld Line (kip-in.)
1	Bolts Fractured	94	0.056	20	279
2	Bolts Fractured	137	0.054	314	691
3	Bolts Fractured	160	0.026	306*	745*
4	Welds and Bolts Fractured	79	0.061	-47	170
5	Bolts Fractured	130	0.053	273	631
6	Bolts Fractured	260	0.045	591	1,153

Note: * Moments at the bolt line and the weld line shown in the table are taken from Astaneh et al. (1989). Different values are presented in Astaneh et al. (1993): 560 and 1,010 kip-in.

In his first paper, Astaneh et al. (1989) suggested that the eccentricity at the weld line, referred to by Astaneh as the connection eccentricity, be calculated as follows:

$$e = (n-1)(1.0) \tag{2.5}$$

where

n = number of bolts used in the connection

The bolt eccentricity, e_b, is then:

$$e_b = (n-1)(1.0) - a \tag{2.6}$$

where

a = distance between the bolt line and the weld line

However, the above equation is used only for the case that the connection is welded to a rigid support. For the case that the connection is welded to a flexible support, it was suggested by Astaneh et al. (1989) without any experimental results that the eccentricity of the bolt group is:

$$e_b = Max \left| \begin{array}{l} (n-1)(1.0) - a \\ a \end{array} \right. \tag{2.7}$$

For the eccentricity of the welds, he suggested that the following formula be used (Astaneh et al., 1989):

$$e_w = Max \left| \begin{array}{l} (n)(1.0) \\ a \end{array} \right. \tag{2.8}$$

To preclude the brittle failure mode where welds rupture before the plate yields, Astaneh (1989) developed a formula based on the shear-moment interaction curve with 50 percent of yield strength of the weld. The shear-moment interaction curve for any rectangular section is:

$$\left(V/V_y\right)^2 + \left(M/M_p\right)^2 = 1.0 \tag{2.9}$$

By replacing the capacity of the section V_y and M_p by the capacity of the welds V_w and M_w, the equation becomes:

$$\left(V/V_w\right)^2 + \left(M/M_w\right)^2 = 1.0 \tag{2.10}$$

The capacity of the welds V_w and M_w can be written as

$$V_w = 2\left(0.707 t_w L_w\right)\left(\frac{1}{\sqrt{3}} 0.5 F_{EXX}\right) \tag{2.11}$$

$$M_w = 2\left(0.707 t_w L_w^2/4\right)\left(0.5 F_{EXX}\right) \tag{2.12}$$

where

t_w	=	weld size (in.)
L_w	=	length of weld (always equal to plate depth)
F_{EXX}	=	yield strength of weld

With the expression of L_w equal to (3 in.)(n), and $M=Ve_w$ where the eccentricity is suggested equal to n, the approximate shear force that would cause yielding of the two welds is:

$$V = 0.95 n t_w F_{EXX} \tag{2.13}$$

By the same manner, substituting the shear and bending capacity of the plate into Equation 2.9, the approximate shear force that would cause yielding of the plate is:

$$V = 1.38 n t_p F_y \tag{2.14}$$

To achieve the goal previously set, the shear force calculated from Equation 2.13 must be equal to or greater than that of Equation 2.14. With F_y = 36 ksi and F_{EXX} = 70 ksi, the following simple expression is obtained:

$$t_w \geq 0.75 t_p \tag{2.15}$$

Aside from the formulas for calculating eccentricity and the formula for the weld size to assure that the brittle failure mode will not occur, Astaneh et al. (1989), based on his research of tee connections, also suggested that the effective net shear area used in the formula for calculating shear rupture of the plate is:

$$A_{nse} = A_{vg} - \left(n/2\right)\left(d_b + 1/16\right) t_p \tag{2.16}$$

where

A_{vg}	=	gross shear area of the plate
d_b	=	diameter of the bolts
t_p	=	thickness of the plate

Porter and Astaneh (1990) further conducted research regarding a single plate shear connection with snug-tight bolts in short slotted holes. The concept used previously with standard holes was adopted for this research. Connections with 3, 5, 7, and 9 bolts in slotted holes were tested. The resulting recommended eccentricities are:

For a rigid support with short-slotted holes

$$e_b = \left| \frac{2n}{3} - a \right| \tag{2.17}$$

For a flexible support with short-slotted holes

$$e_b = \left| \frac{2n}{3} - a \right| \geq a \tag{2.18}$$

More single plate shear connections were tested by Astaneh in 1992. The beam-to-girder single plate shear connection, which is a flexible connection, was investigated in this research.

At the University of Oklahoma, Sarkar (1992) conducted tests of 2-, 4-, and 6-bolt connections, with both standard and slotted holes using A36 steel plate and beam material. In one test, two identical connections were used to symmetrically connect the beam to the supports. Instead of using the same loading application as used by Astaneh, Sarkar used a concentrated load applied at different locations throughout the experiment. The starting location was far from the support (the connection) to impose high rotation on the structure. The loading location was then gradually moved toward the support to place high load on the connections. The details of each test and the results are presented in Tables 2.4 and 2.5.

During most of the tests, the loading location was changed from time to time by Sarkar in an attempt to gain rotation. In test no. 3, a 4-A325 bolt connection with round holes, the topmost bolt in the North connection ruptured at a shear of merely 66.5 kips while the bolts in the South connection withstood a shear of 84.6 kips when the loading was 92 in. from the support. When the loading was moved to a location 78 9/16 in. from the support, the North side connection, now having only 3 bolts remaining, was able to carry a shear of 81.6 kips while the topmost bolt in the South connection shear ruptured, notably lower than the first load of 84.6 kips. For the 6-bolt connection with round holes (test no. 5 in the table), the experiment was carried out with the connections on both ends

Table 2.4 Properties of Test Specimens Used by Sarkar (1992)

No	No. of Bolts	Dia. of Bolts (in.)	Type of Bolts	Plate Dimensions (in. x in. x in.)	Plate Holes	Weld Size (in.)	Beam	Beam Length (ft)	Beam & Plate Material
1	2	3/4	A325-X	3/8x5x6	Standard	5/16	W12x35	21	A36
2	2	3/4	A325-N	3/8x5x6	Standard	5/16	W12x35	21	A36
3	4	3/4	A325-N	3/8x5x12	Standard	5/16	W18x76	33	A36
4	4	3/4	A325-N	3/8x5x12	Short Slots	5/16	W18x76	33	A36
5	6	3/4	A325-N	3/8x5x18	Standard	5/16	W21x93	25	A36
6	6	3/4	A325-N	3/8x5x18	Short Slots	5/16	W21x93	25	A36

Table 2.5 Sarkar's Test Results

No	Shear Force (kips)	Beam End Rotation (rad)	Observation
1	64.3	0.025	Shear distortion of plate.
2	51.8	0.033	Weld tearing at South connection.
	60.8	0.028	Weld tearing at North connection.
3	66.5	0.028	Topmost bolt sheared in North connection, no failure in South.
	84.6	0.033	Load moved from 92 in. to 78 9/16 in. from support.
	81.6	0.038	Topmost bolt sheared in South connection, load moved to 50 in.
	93	0.032	Bolt line deflection, test stopped.
4	129	0.042	All bolts sheared.
5	102	0.014	Topmost bolt sheared in North connection.
	109	0.019	Topmost bolt sheared in South connection.
	119	0.027	Second topmost bolt sheared in North connection.
6	168	0.03	Topmost bolt sheared in North connection.
	194		Test stopped.

of the beam having only 5 bolts left until the second topmost bolt shear ruptured.

From the test results, Sarkar suggested that a bolt group be designed without eccentricity. The following plate thickness requirements were also recommended:

For A325 bolts

$$t_p \leq 0.42 d_b \tag{2.19}$$

For A490 bolts

$$t_p \leq 0.52 d_b \tag{2.20}$$

The relationships were developed by setting the nominal shear strength of the bolt equal to the nominal bearing strength of the plate using the formulas from the AISC LRFD Manual (AISC, 2001).

Astaneh et al. (1993) later presented more results on the design of single plate shear connections. This paper presented new experimental results of a 9-bolt connection. The previous formulas for calculating the eccentricity for the welds were also modified, as follows:

The eccentricity of a bolt group with a rigid support

$$e_b = |n-1-a| \tag{2.21}$$

The eccentricity of a bolt group with a flexible support

$$e_b = Max\left|\begin{matrix} |n-1-a| \\ a \end{matrix}\right. \tag{2.22}$$

The eccentricity of welds with a rigid support

$$e_w = (n-1) \tag{2.23}$$

The eccentricity of welds with a flexible support

$$e_w = Max\left|\begin{matrix} (n) \\ 0.0 \end{matrix}\right. \tag{2.24}$$

The absolute value operator was introduced to the formulas for calculating the eccentricity for a bolt group. This indicates that the point of inflection can be located either between the weld line and the bolt line or outside the bolt line.

Duggal and Wallace (1996) at the University of Oklahoma carried out a number of tests to study the behavior of a single plate shear connection with slotted holes. The research was primarily focused on the force that was required to move the bolt in the longitudinal direction (in the slotted hole). A design procedure that takes into account the longitudinal force was recommended. The longitudinal force is calculated as:

$$F_L = \mu T + \mu_{1s} P + KD_L \tag{2.25}$$

where

F_L	=	longitudinal force required to slide the bolt
μ	=	coefficient of friction
T	=	bolt tension
μ_{1s}	=	coefficient of lateral swelling friction, taken as 0.25
P	=	perpendicular load applied
K	=	slot stiffness
D_L	=	longitudinal displacement along the slot length

The slot stiffness is only available for 3/4- and 1-in. A325 bolts through the figures provided by Duggal and Wallace. The longitudinal displacement can be calculated based on the 0.03 radian demand on the connection with the assumption that the connection rotates about the centroid of a bolt group. It is recommended that the bolts are designed with the resultant force of the longitudinal force calculated from Equation 2.25 and the vertical shear. In the design of a connection with the flexible support condition, the term KD_L is to be excluded from the equation.

Sherman and Ghorbanpoor (2000) studied an extended single plate shear connection attached to a column web or a girder. An extended single plate connection is the connection with the plate being extended so that the connection can be used to connect a beam to a column web or connect a beam to a girder without the beam being coped. The purpose of the research was to study the extended plate and obtain a design formula to accommodate the beam-to-column web and beam-to-girder constructions. Tests of 17 extended plate connections with and without horizontal stiffening plates were carried out at the University of Wisconsin-Milwaukee. A stiffened connection is when the plate is welded to the stiffening plates which are assembled on top and/or bottom of the plate in a beam-to-column web connection or when the plate is welded to the top flange of a girder. The following formula was recommended for calculating the required plate thickness:

$$t_p \geq \left(Va^2 / 1200L\right)^{1/3} \tag{2.26}$$

where

V = applied shear, kips

L = plate length, in.

The eccentricity with respect to the bolt line is the following:

For h/t_w of the supporting member greater than 35

$$e = a \tag{2.27}$$

For stiffened or unstiffened connections to a girder with h/t_w equal to or less than 35

$$e = 0.5a \tag{2.28}$$

For stiffened connections to a column with h/t_w equal to or less than 35

$$e = 0.25a \tag{2.29}$$

Astaneh et al. (2002) later tested a single plate shear connection under cyclic loading. From the results of the test, it was concluded that the current design formulas were still applicable to a connection under cyclic loading.

2.2 Development of Design Specification of Structural Components under Eccentricity

2.2.1 Design of Bolt Group under Eccentricity

The history of the design of a bolt group subjected to eccentricity is dated back to the use of the elastic method to calculate the stress in each bolt. The elastic method assumes that the stress on a bolt group caused by direct shear is distributed equally to each bolt. The stress caused by moment is then distributed proportionally to each bolt by the ratio of the bolt distance from the center of gravity of the bolt group to the maximum bolt distance to the same reference; the farthest bolt from the center will have the highest shear stress. The calculation using the elastic method proves to be convenient; however, it is rather too conservative in many cases due to the assumption that bolts behave elastically.

To remedy the problem, as stated by Crawford and Kulak (1971), Yarimci and Slutter (1963) tested riveted connections at Lehigh University. The results of the tests were later presented by Higgins (1964). The riveted connections, single and double columns, were eccentrically loaded with eccentricities varying from 2-1/2 in. to 6-1/2 in.

The concept of using an instantaneous center along with the effective eccentricity was introduced to replace the elastic method by Yarimci. An instantaneous center is the point that defines a rotation and translation on a structural element caused by eccentricity.

The fact that the method was based on riveted connections rather than bolted connections, and the assumption that the material was perfectly elastic, caused the method to be short-lived. Crawford and Kulak (1968, 1971) carried out tests on eccentrically loaded bolted connections. The concept of using the instantaneous center was kept, but a new load-deformation relationship was introduced. To achieve the load-deformation curve, several single 3/4-in. diameter A325 bolts in double shear were tested. The following formula was proposed for calculating shear strength of a bolt at any given deformation:

$$R = R_{ult}(1 - e^{-10\Delta})^{0.55} \tag{2.30}$$

where

R_{ult}	=	ultimate shear strength of one bolt, kips
e	=	2.718
Δ	=	shearing, bending, and bearing deformation of a bolt, and local bearing deformation of the plate, in.

After the load-deformation curve was established to explain the behavior of the bolts, eight full-scale double-angle bolted-bolted connections were tested to verify the proposed relationship under various eccentricities. The eccentricity in the experiments by Kulak is the distance from the bolt line to the back of the outstanding leg of the angles. This eccentricity is then interpreted as the distance projecting from the load line to the center of the bolt group.

Each connection configuration was designed such that the 3/4-in. diameter A325 bolt group would fail under shear. This bolt group was on the 1/2-in. thick leg of the angles, while 7/8-in. A325 bolts were on the outstanding legs to connect the connection to the 3/4-in. thick support. The main reason for the use of more robust components on the outstanding legs was to prevent any slip that might occur. Two connections were set in mirror image to carry the 2-ft beam upon which the load was applied. The tests would then be loaded until the connection failed. The details and results of the tests are summarized in Table 2.6.

Equation 2.30 developed by Kulak and the instantaneous center of rotation method developed by Yarimchi are used in the AISC Manuals (1989, 2001). Design aid tables available are the expansion of these concepts with the variety of number of bolts, bolt patterns, spacing, eccentricities, and angles of inclined loading. The values of coefficient C, also known as the number of effective bolts, tabulated in the tables can be used with any bolt diameter as suggested by Kulak. However, Kulak did not strongly recommend that the load-deformation relationship described by Equation 2.30 be used with A490 bolts, even though it is allowed by the Manual with a precaution of being conservative.

Table 2.6 Details and Results of Tests by Kulak

Specimen	Number of Bolts	Eccentricity (in.)	Pitch (in.)	Gage (in.)	Predicted Load (kips)	Test (kips)	Error (%)
B1	5	8	2-1/2	-	252	225	12
B2	5	10	3	-	244	230	6.1
B3	6	12	3	-	206	190	8.4
B4	6	13	3	-	274	251	9.2
B5	6	15	3	-	239	221	8.1
B6	8	12	3	2-1/2	293	264	11
B7	8	15	3	2-1/2	239	212	13
B8	10	15	2-1/2	2-1/2	309	266	16

Notes: 1. Specimens B6 to B8 were double-column connections.
2. For specimens B6 to B8, the number of bolts shown is the total number.
3. Gage is the distance between the bolt columns.
4. The percentage of error is calculated with respect to the test load.

2.2.2 Design of Welds under Eccentricity

Current design tables for welds in the AISC LRFD Manual (2001) are based on the load-deformation relationship of a unit-length segment of weld as developed by Lesik and Kennedy (1990). As stated in the Manual, the formula is:

$$R = 0.60 F_{EXX} \left(1.0 + 0.50 \sin^{1.5}\theta\right)\left[p\left(1.9 - 0.9p\right)\right]^{0.3} \tag{2.31}$$

where

R	=	nominal shear strength of weld segment at a deformation Δ, kips
F_{EXX}	=	weld electrode strength, ksi
θ	=	load angle measured relative to the weld longitudinal axis, degrees.
p	=	ratio of element deformation to its deformation at maximum stress.

This formula is used with the method developed by Butler et al. (1972), which uses the same instantaneous center of rotation approach as previously described in Section 2.2.1.

2.3 A Single Plate Shear Connection Associated with Tubular Columns

White (1965) conducted tests to verify whether or not a single plate shear connection can be used with a structural tubing section. Eight tests featuring a series of 3/8-in thick plates welded to 4x4 and 8x8 tubular columns were carried out. It was concluded that a single plate shear connection induced excessive distortion to the column and therefore was not appropriate to be used with a tubular column.

Sherman (1996) also carried out tests of a single plate shear connection used with rectangular and square HSS sections (Hollow Structural Shapes). It was stated in his 1996 paper that a single plate shear connection was also tested with HSS sections previously (Sherman and Ales, 1991). It was concluded that when the connection is used with a thin HSS section, the eccentricity always lies between the bolt line and the weld line with the value less than provided in the AISC Manual, and became close to that value when the connection was used with a relatively thick section and a flexible beam.

Sherman (1996) further stated that a single plate shear connection could be used with an HSS section that was not defined as a thin-walled section. However, a connection used with a thin-walled section would reduce the column strength to a great extent (Sherman, 1995). Additional limit states must be considered for an HSS section when used with a single plate shear connection, such as shear strength of the column at the weld, and punching shear.

2.4 Conclusion on Literature Review

From the literature survey, several design models of single plate shear connections exist. The major difference among the design models is whether or not the strength of the connection is a function of the beam size and length. The current design model in the Manuals (AISC 1989, 2001), which states that the strength of the connection is not a function of the beam, has been changed several times without further research conducted. The most important change is the introduction of the absolute value operator to the formulas. Much research has involved single-column bolt connections.

Because of the range of predicted strength in the model, it is apparent that furthur research is justified. The performance and the accuracy of the current design model, including research conducted in the past, are investigated in Chapter III.

Chapter III

Review of Design Model and Scope of Research

3.1 Limitations and Flaws of the Current Design Model

3.1.1 Limitations and Flaws

As described in previous chapters, the single bolt column single plate shear connection design model in the AISC Manuals (1989, 2001) has considerable limitations, mostly concerning dimensions and material properties. The limitations of the design model are mainly due to the limitations in the experimental tests carried out in the past. The most recognizable limitation is the distance between the weld line and the bolt line, or the a-distance. According to the AISC Manuals (1989, 2001), the distance must be between 2.5 and 3.5 in. Any design with the a-distance out of this range has to be based on engineering judgement or fundamental analysis. For any design of a single plate shear connection with a-distance within this range, the eccentricity on a bolt group, e_b in inches, is determined from the following formulas:

The eccentricity of a bolt group with a rigid support

$$e_b = \left|(n-1) - a\right| \tag{3.1}$$

The eccentricity of a bolt group with a flexible support

$$e_b = Max \left| \begin{matrix} \left|(n-1) - a\right| \\ a \end{matrix} \right. \tag{3.2}$$

The formulas state that the eccentricity of a bolt group for either support condition is a function of the number of bolts and the a-distance. The values of the eccentricity calculated, with the a-distance from 2.5 to 3.5 in. and the number of bolts from two to nine, are illustrated in Tables 3.1 through 3.4 and Figures 3.1 through 3.4.

Table 3.1 Values of Eccentricity of a Bolt Group with Rigid Support

No. of Bolts	Eccentricity (e=|(n-1)-a|)				
	a-distance (in.)				
	2.5	2.75	3	3.25	3.5
2	1.50	1.75	2.00	2.25	2.50
3	0.50	0.75	1.00	1.25	1.50
4	0.50	0.25	0.00	0.25	0.50
5	1.50	1.25	1.00	0.75	0.50
6	2.50	2.25	2.00	1.75	1.50
7	3.50	3.25	3.00	2.75	2.50
8	4.50	4.25	4.00	3.75	3.50
9	5.50	5.25	5.00	4.75	4.50

Table 3.2 Number of Effective Bolts Tabulated Using the Eccentricity in Table 3.1 and the Instantaneous Center of Rotation Method

No. of Bolts	Number of Effective Bolts C				
	a-distance (in.)				
	2.5	2.75	3	3.25	3.5
2	1.39	1.28	1.18	1.09	1.01
3	2.88	2.81	2.71	2.60	2.48
4	3.88	3.91	4.00	3.91	3.88
5	4.60	4.69	4.77	4.83	4.87
6	5.23	5.34	5.45	5.54	5.63
7	5.81	5.94	6.06	6.17	6.28
8	6.39	6.52	6.64	6.77	6.89
9	6.96	7.09	7.22	7.35	7.47

Table 3.3 Values of Eccentricity of a Bolt Group with Flexible Support

No. of Bolts	Eccentricity (e=max {|(n-1)-a|,a}) a-distance (in.)				
	2.5	2.75	3	3.25	3.5
2	2.50	2.75	3.00	3.25	3.50
3	2.50	2.75	3.00	3.25	3.50
4	2.50	2.75	3.00	3.25	3.50
5	2.50	2.75	3.00	3.25	3.50
6	2.50	2.75	3.00	3.25	3.50
7	3.50	3.25	3.00	3.25	3.50
8	4.50	4.25	4.00	3.75	3.50
9	5.50	5.25	5.00	4.75	4.50

Table 3.4 Number of Effective Bolts Tabulated Using the Eccentricity in Table 3.3 and the Instantaneous Center of Rotation Method

No. of Bolts	Number of Effective Bolts C a-distance (in.)				
	2.50	2.75	3.00	3.25	3.50
2	1.01	0.94	0.88	0.82	0.77
3	1.98	1.86	1.75	1.65	1.56
4	3.07	2.94	2.81	2.69	2.58
5	4.15	4.03	3.90	3.77	3.64
6	5.23	5.11	4.98	4.86	4.73
7	5.81	5.94	6.06	5.94	5.81
8	6.39	6.52	6.64	6.77	6.89
9	6.96	7.09	7.22	7.35	7.47

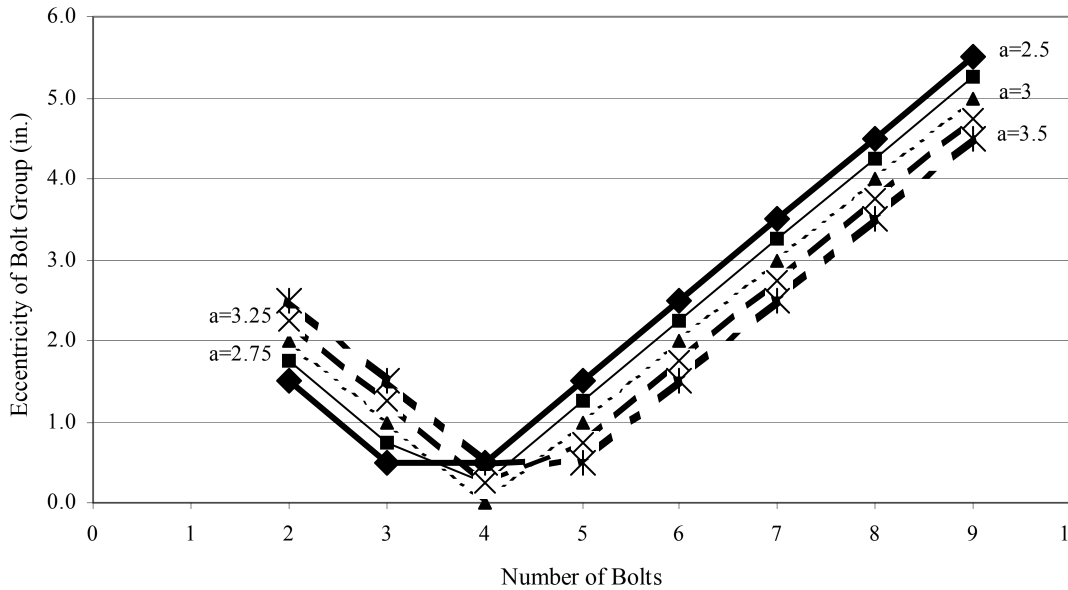


Figure 3.1 Eccentricity of Bolt Group vs. Number of Bolts for Rigid Supports

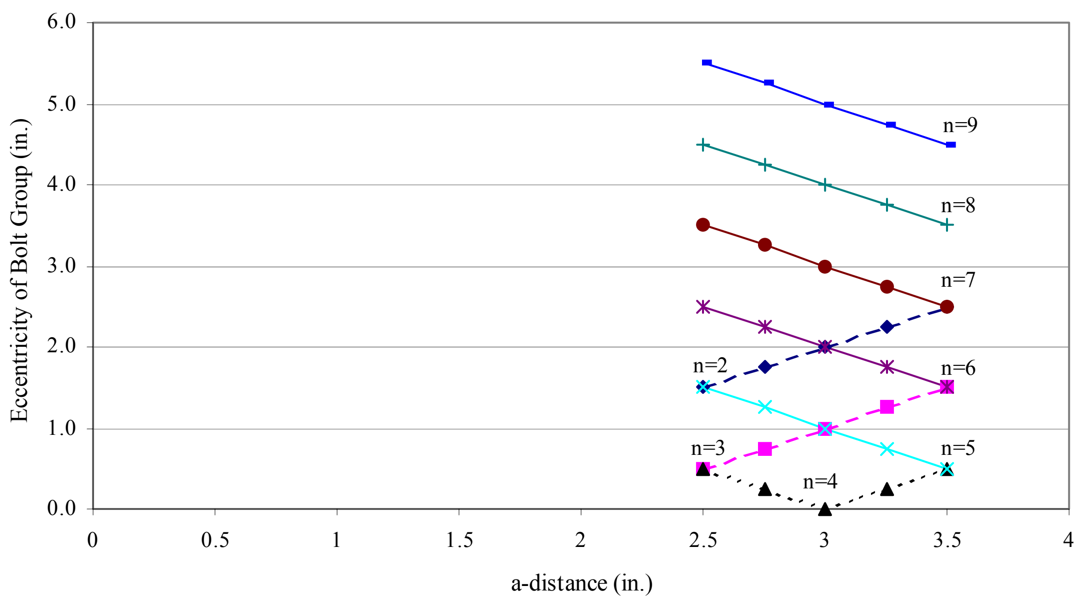


Figure 3.2 Eccentricity of Bolt Group vs. a-distance for Rigid Supports

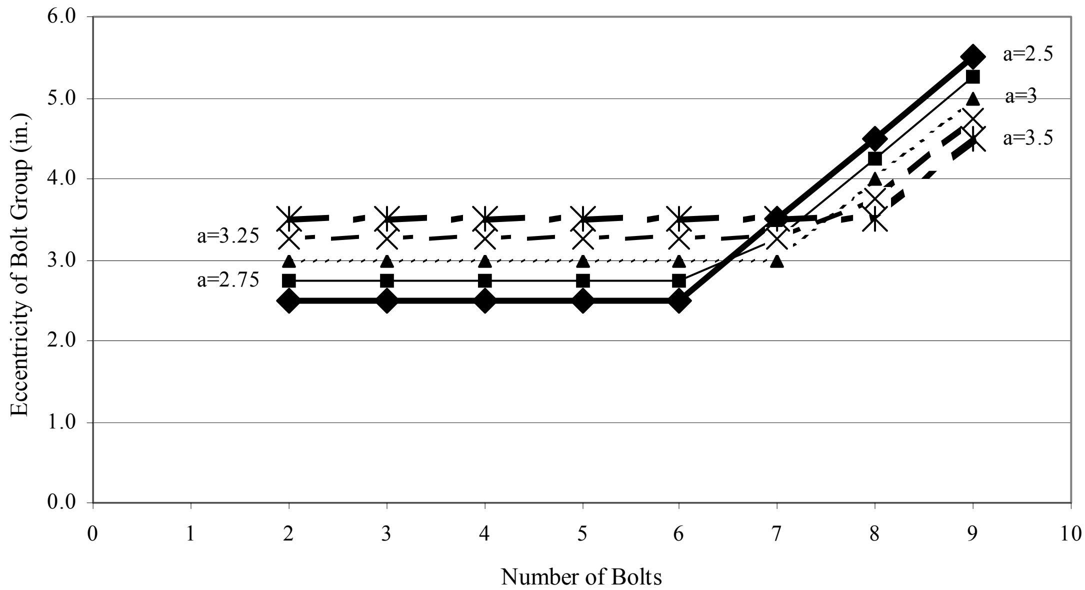


Figure 3.3 Eccentricity of Bolt Group vs. Number of Bolts for Flexible Supports

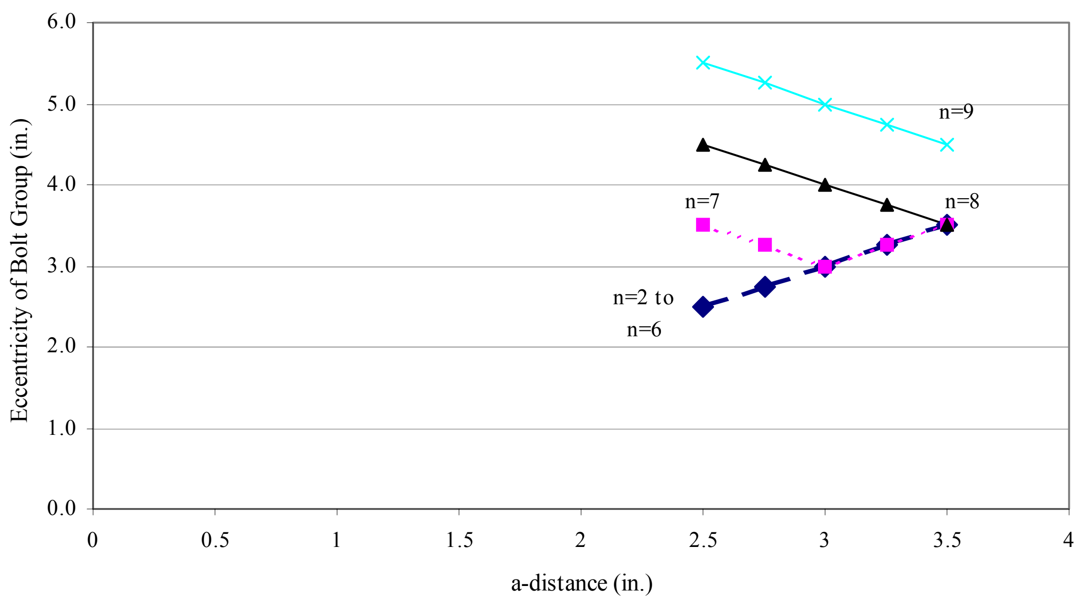


Figure 3.4 Eccentricity of Bolt Group vs. a-distance for Flexible Supports

It can be clearly seen from both the tables and the graphs how inconsistent the current formulas are. With a rigid support condition, Equation 3.1 seems to predict reasonable results when the calculated eccentricity increases with the number of bolts as shown in Figure 3.1. However, it is noticeable that a connection with two bolts has more eccentricity when the a-distance increases, whereas a connection with nine bolts behaves in the reverse fashion. This matter is made clear in Figure 3.2. When the a-distance is varied and the number of bolts is fixed, the eccentricities of 2- and 3-bolt connections increase when the a-distance of the connection increases. The value of eccentricity begins to be level for a 4-bolt connection: the eccentricity of the bolt group decreases as the a-distance increases, and eventually becomes zero at an a-distance of 3 in. The eccentricity of a 4-bolt connection rises once again when the a-distance is greater than 3 in. The value of the eccentricity then begins to decrease as the a-distance increases when the number of bolts is equal to or greater than five. The number of effective bolts in the connection shares the same pattern, since it is directly calculated from the value of eccentricity using Table XI in the ASD Manual (1989) or Table 7-17 in the LRFD Manual (2001) developed in accordance with the instantaneous center of rotation method.

Even though the shapes of the graphs look different for the rigid and flexible support cases, the nature of results obtained by Equation 3.1, as shown in Figures 3.1 and 3.2, is similar to that predicted by Equation 3.2, as illustrated in Figures 3.3 and 3.4. For the flexible support condition, a connection with two to six bolts will have the same value of eccentricity. The value of eccentricity for those connections is the a-distance since the number of bolts is small in those connections (two to six); thus, the absolute value of the (n-1-a) term will never supersede the value of the a-distance. The value of eccentricity becomes the absolute value of the (n-1-a) term once the number of bolts in the connection is seven or greater. The same behavior as previously described for a connection with the rigid support condition can be observed with the flexible support condition. The value of eccentricity of a connection with two to six bolts will increase along with the increase of the a-distance; however the value will decrease once the number of bolts in the connection is eight or greater. The observation is illustrated more clearly in Figure 3.4.

It is understandable from the derivation of the formula that the formulas themselves locate the inflection point of the connection. However, Astaneh et al. (1989) stated that, when the plate was in the elastic region, it behaved as a short cantilever beam before it started behaving as a deep beam in the inelastic region, and as a diagonal truss once the plate was in the strain-hardening region. On the contrary, the eccentricity of the bolt group predicted by Astaneh's formulas decreases when the a-distance, which can be viewed as a cantilever portion of the plate, increases once a connection has more than four bolts for a rigid support condition, and more than seven bolts for a flexible support condition. Arguably, this could indicate that a large a-distance will move the point of inflection toward the bolt group, but it is not rational to design a bolt group for the eccentricity to be less when the a-distance increases.

For a connection with a flexible support condition, the connection is under less restraint and has more freedom to move or rotate. Fundamentally, the moment acting on the bolts and the welds should be less than that for the same connection with a rigid support condition. Values of eccentricity calculated by Equations 3.1 and 3.2 for each connection, varying from 2-bolt to 7-bolt, are presented in Figures 3.5 through 3.10 to demonstrate how the two equations predict eccentricity.

Figures 3.5 through 3.10 show that eccentricity for connections with the flexible support condition are always greater than connections with the rigid support condition. The difference of values of eccentricity between the two support conditions is small with a 2-bolt connection and becomes larger when the number of bolts increases. The difference is extremely large for 4- and 5-bolt connections with the a-distance equal to 3.5 in. In addition, for a connection with five to seven bolts, the difference increases as the a-distance increases. It is stated in the LRFD Manual (2001) that the larger value of e_b may be conservatively used if the support condition is intermediate or not classified. Should the situation occur with either a 4- or 5-bolt connection under an a-distance equal to 3.5 in., the determination of the support condition can result in a connection capacity equivalent to one bolt. The same type of plot is not presented for the 8- or 9-bolt connections since both support conditions produce identical results for these connections.

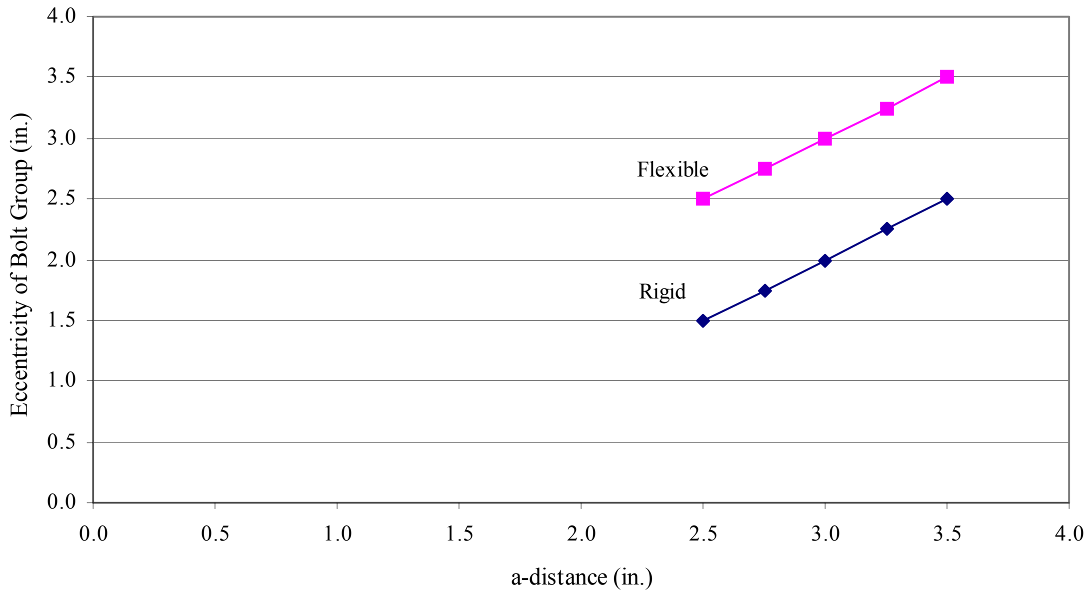


Figure 3.5 The 2-Bolt Connection with Different Support Conditions

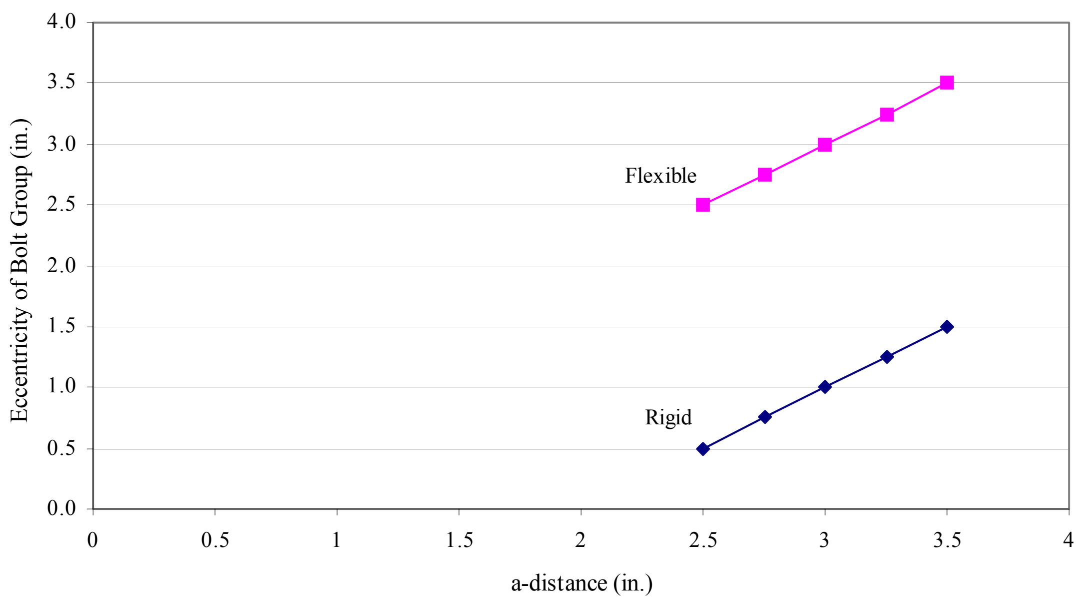


Figure 3.6 The 3-Bolt Connection with Different Support Conditions

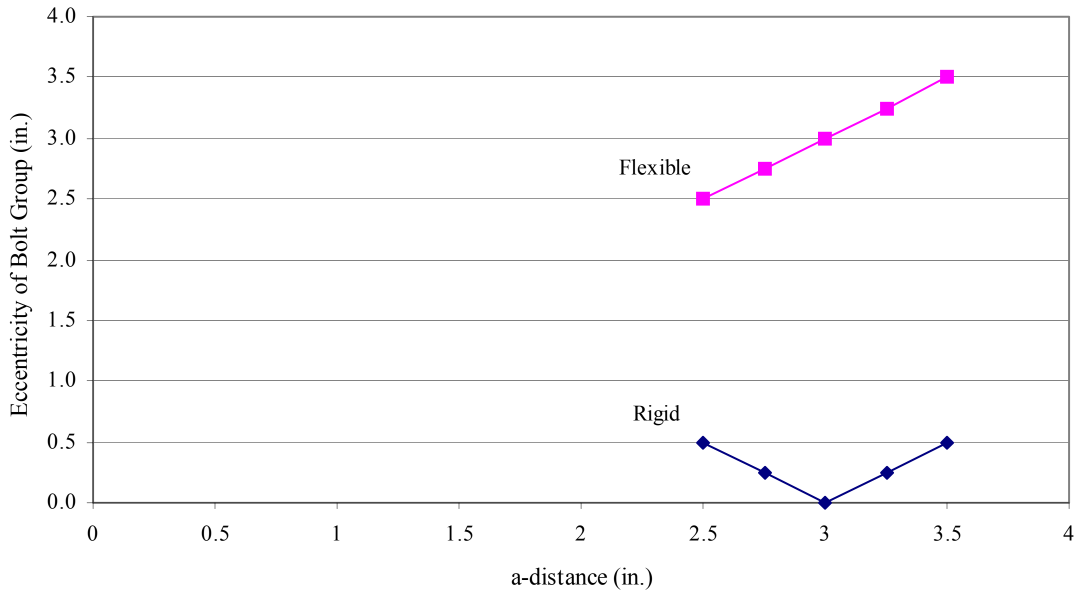


Figure 3.7 The 4-Bolt Connection with Different Support Conditions

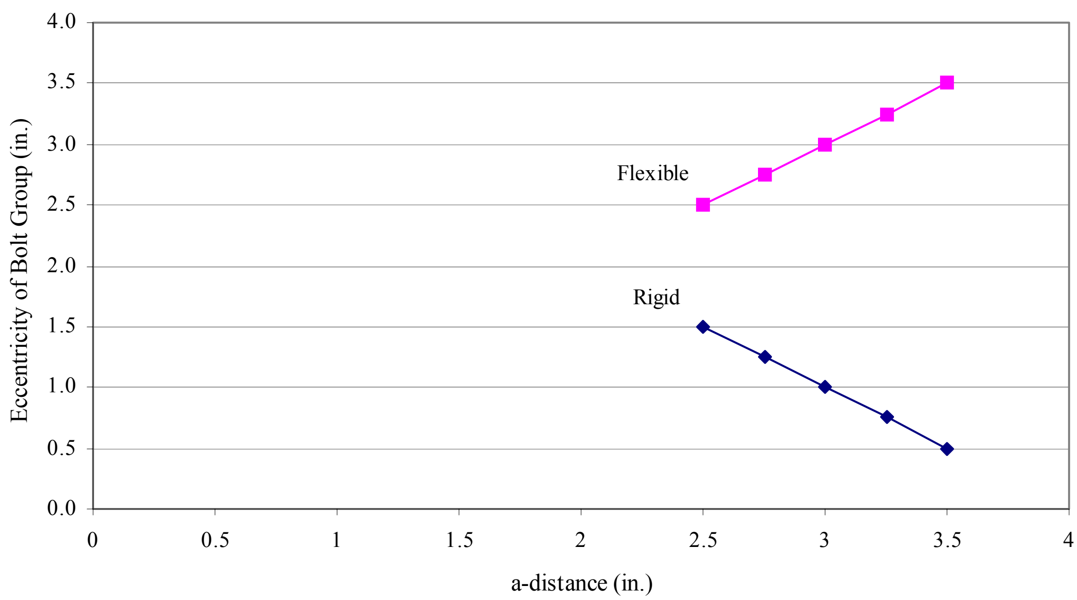


Figure 3.8 The 5-Bolt Connection with Different Support Conditions

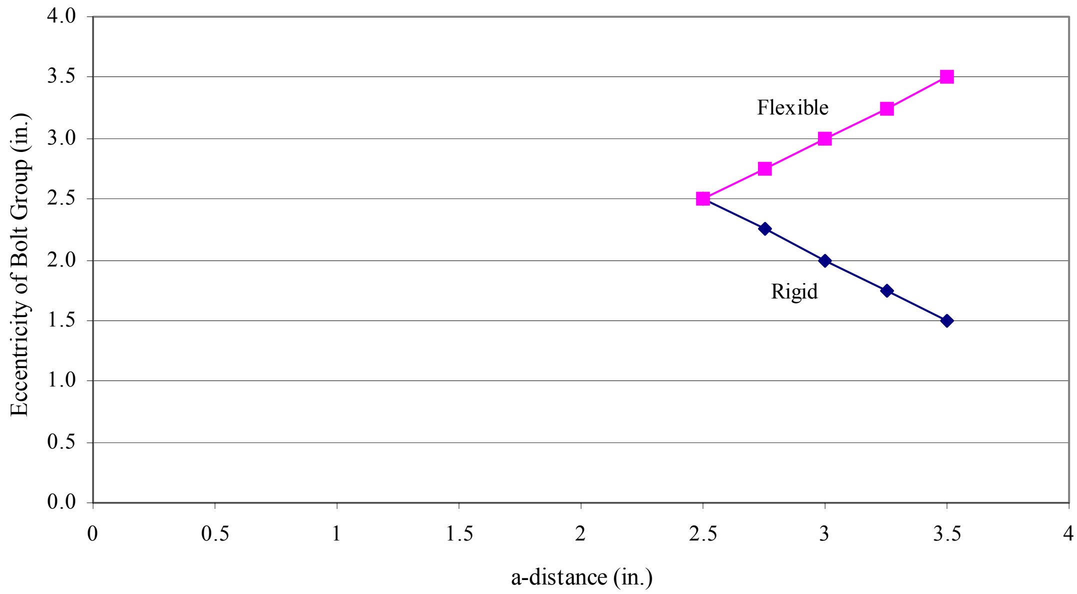


Figure 3.9 The 6-Bolt Connection with Different Support Conditions

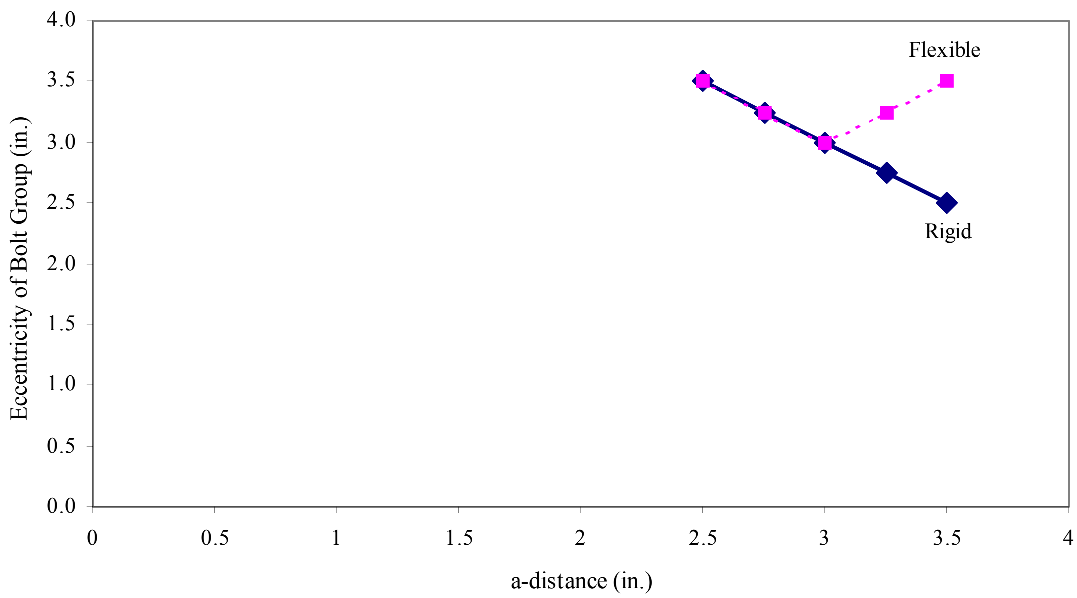


Figure 3.10 The 7-Bolt Connection with Different Support Conditions

Even though both the AISC ASD and LRFD Manuals do not state how the design of welds should be carried out, Astaneh et al. (1993) suggested that the eccentricity of welds be calculated as follows:

The eccentricity of welds with a rigid support

$$e_w = (n-1) \tag{3.3}$$

The eccentricity of welds with a flexible support

$$e_w = Max \left| \begin{matrix} (n) \\ 0.0 \end{matrix} \right. \tag{3.4}$$

With no further illustrations necessary, it is obvious that the formulas suggest that the difference in weld eccentricity between the two support conditions is a constant 1 in. While the differences of the eccentricity of a bolt group between the two support conditions vary as shown previously, the difference of eccentricity for welds remains a constant. It is clear that the formulas for calculating eccentricity for a bolt group and the welds are not based on the inflection point location for all cases.

In case of a connection attached to a flexible support condition, such as a beam girder, the flexibility of the supporting member is not directly considered in the determination of eccentricity. The stiffness of a single plate shear connection has always been a primary concern, that is, whether or not it will allow sufficient rotation so that the supported member can reach its maximum bending capacity. Support condition, or flexibility of the supporting member, is a major contribution to the rotation capacity of a connection (Lipson, 1968). A connection attached to a much different girder, in either size or span length, will not experience the same rotation due to the difference in torsional stiffness of the girder. The design model for calculating the eccentricity of a connection should be able to characterize and take into the account the difference in the torsional stiffness of the supporting member.

It is recommended by Astaneh et al. (1989) that the design procedure should not be used with an a-distance less than 2.5 in. or greater than 3.5 in. From the numbers tabulated in Tables 3.1 through 3.4, it is also not encouraged to use the formulas with connections with an a-distance out of the predefined range. In the ASD Manual (1989), tables are provided for the design of single plate shear connections. The tables are limited to an a-distance equal to 3 in. as required in the first paper published by Astaneh et al. (1989).

Along with the thickness of the plate (or beam web), the rotation of a bolt group caused by the beam might affect the deformation of the bolts. Should the bolt group be assumed to rotate with the beam, the bolt line can be viewed as having the same amount of rotation as the beam, or about 0.03 radian at maximum loading. For any rotation, the topmost and/or bottommost bolt in different bolt group geometries would experience different amounts of displacement. For example, if the assumed Y-coordinate of the instantaneous center and that of the center of gravity of the bolt group coincide, and the rotation of a bolt group as a whole is fixed as 0.03 radian, the topmost or bottommost of single row bolt groups will undergo a different amount of displacement as illustrated in Table 3.5.

Table 3.5 Displacements in Topmost (or Bottommost) Bolt Caused by Beam Rotation

Number of bolts	Displacement in Y-direction (in.)
2	0.045
3	0.090
4	0.135
5	0.180
6	0.225
7	0.270
8	0.315
9	0.360

3.1.2 Limitations on Configurations

The limitations of connection configurations in the current design model should also be emphasized. There are three limitations for the design of a single plate shear connection: maximum plate thickness to ensure ductility, minimum plate thickness to prevent buckling, and minimum size of welds to preclude a brittle failure mode.

The first limitation is with respect to the maximum plate thickness. It is recommended by Astaneh that the plate thickness satisfy:

$$t_{p\max} \leq \frac{d_b}{2} + \frac{1}{16} \tag{3.5}$$

The development of Equation 3.5 was based on early research on single plate shear connections by Richard et al. (1980) and the research on tee connections by Astaneh and Nader (1989). The maximum plate thickness recommended by Richard, as discussed in Chapter II, is that the bolt diameter-to-plate thickness ratio equal two for 3/4- and 7/8-in. diameter A325 bolts. The ratio becomes 9/16, the same as given by Equation 3.5, with 1-in diameter bolts. The maximum plate thickness recommended by Astaneh in his research regarding the tee connections is one-half of the bolt diameter. The maximum plate thickness with A490 bolts is increased by 1/4 in. instead of 1/16 as with Equation 3.5 when 3/4-in. bolts are used, and by 1/2 in. when 7/8- and 1-in. bolts are used. It is noted that all of these recommendations are with A325 and A490 bolts and 3/8-in. A36 plate.

The maximum plate thicknesses allowed for different bolt diameters according to Equation 3.5 are listed in Table 3.6.

Table 3.6 Maximum and Minimum Thickness Allowed for Plate Due to Bolt Diameter

Nominal Bolt Diameter (in.)	Max. t_p (in.)	Max. Practical t_p (in.)
5/8	0.38	3/8
3/4	0.44	3/8
7/8	0.50	1/2
1	0.56	1/2
11/8	0.62	5/8
11/4	0.69	5/8
13/8	0.75	3/4
11/2	0.81	3/4

The minimum thickness of the plate required to prevent local buckling is calculated using the following expression (AISC, 2001):

$$t_{p\min} = \frac{L}{234}\sqrt{\frac{F_y}{K}} \geq \frac{1}{4} \qquad (3.6)$$

where

K = plate buckling coefficient for local buckling of double coped beam that can be found in Part 9 in LRFD Manual of Steel Construction (AISC, 2001)

L = length of plate

To obtain the minimum plate thickness, a plate buckling coefficient must first be calculated. Table 3.7 lists the plate aspect ratios 2a/L as defined in the Manual (AISC, 2001). The plate aspect ratios are then used to calculate the plate buckling coefficients, which are shown in Table 3.8. The length of the plate used in the calculation is determined from the two edge distances of 1 1/2 in. plus (n-1) x 3 in., where n is the number of bolts. From the plate buckling coefficients, the minimum plate thicknesses are finally calculated with the results shown in Table 3.9. The blank spaces in Tables 3.8 and 3.9 are because the corresponding plate buckling coeffiicients for the plate aspect ratios calculated in Table 3.7 are not provided in the Manual. However, for A36 plate, the minimum plate thickness required never exceeds a value of 0.25 in.

Table 3.7 Plate Aspect Ratios for Calculating Plate Buckling Coefficient

No. of Bolts	Plate Aspect Ratio 2a/L				
	a-distance (in.)				
	2.5	2.75	3	3.25	3.5
2	0.833	0.917	1.000	1.083	1.167
3	0.556	0.611	0.667	0.722	0.778
4	0.417	0.458	0.500	0.542	0.583
5	0.333	0.367	0.400	0.433	0.467
6	0.278	0.306	0.333	0.361	0.389
7	0.238	0.262	0.286	0.310	0.333
8	0.208	0.229	0.250	0.271	0.292
9	0.185	0.204	0.222	0.241	0.259

Table 3.8 Plate Buckling Coefficients

No. of Bolts	Plate Buckling Coefficient K				
	a-distance (in.)				
	2.5	2.75	3	3.25	3.5
2	2.10	1.70	1.30	1.22	1.13
3	5.16	4.35	3.61	2.87	2.37
4	9.32	7.68	6.00	5.37	4.76
5	12.01	10.99	10.00	8.68	7.32
6	14.32	12.82	12.01	11.17	10.33
7		15.28	13.84	12.70	12.01
8			16.00	14.74	13.48
9					15.46

Table 3.9 Minimum Plate Thicknesses

No. of Bolts	Minimum Plate Thickness (in.)				
	a-distance (in.)				
	2.5	2.75	3	3.25	3.5
2	0.11	0.12	0.13	0.14	0.14
3	0.10	0.11	0.12	0.14	0.15
4	0.10	0.11	0.13	0.13	0.14
5	0.11	0.12	0.12	0.13	0.14
6	0.12	0.13	0.13	0.14	0.14
7		0.14	0.14	0.15	0.16
8			0.15	0.16	0.17
9					0.18

3.1.3 Limitations of Experimental Tests

Astaneh tests. A comparison between test results published by Astaneh et al. (1989) and predicted strength from the current design model are summarized in Table 3.10. Material properties used in the calculations are taken directly from test data if available; otherwise, nominal values are used. The measured yield strength of the plates was 35.5 ksi and the ultimate strength was 61 ksi. Material properties for both bolts and welds were not determined, so nominal values were used in the calculations. Limit states that are considered in the calculations are shear yielding, shear rupture, block shear, and bearing/tear-out of plate, bolt shear rupture, and weld rupture. The eccentricity and coefficient C for a bolt group were calculated as demonstrated in Section 3.1.1. Two methods were used to estimate the eccentricity on welds: Equations 2.8 and 3.3. The a-distance used in Astaneh's tests was 2.75 in. The calculation details can be found in Appendix A. The following conclusions can be drawn:

3-A325 bolt connection: The governing limit state for this three-bolt connection is shear strength of the bolt group, with or without considering eccentricity. The predicted strength of the bolt group is 59.6 kips (or 63.6 kips without eccentricity), which is more than 30 percent below the test result, 94 kips. Further, the shear yielding of the plate, shear rupture of the plate, and block shear of the plate limit state values are also below the maximum test load. The average shear stress in the bolts from the test data is 70.8 or 63.4 ksi, which is greater than the nominal shear stress of an A325 N-type bolt, 48 ksi. The reason that two numbers are included, one outside the parentheses and one inside with an asterisk, is that the original plot presented by Astaneh et al. (1988) indicated that the connection reached a load of 84 kips at a maximum rotation of 0.056 radian before the rotation started to decrease, while the loading increased until the first bolt sheared off at a load of 94 kips. The plot presented in a later paper (Astaneh et al., 1989) for the same connection, however, is different from the original one.

5-A325 bolt connection: The governing limit state for the five-bolt connection is shear strength of the bolt group, with or without eccentricity. The predicted strength of the connection is 101 kips (or 106.1 kips without eccentricity), which is more than 25 percent below the test result, 137 kips. The shear yielding strength of the plate is lower

Table 3.10 Astaneh's Experimental Results vs. Design Specifications

Type of Limit States	3-A325 bolt	5-A325 bolt	7-A325 bolt	3-A490 bolt	5-A490 bolt
1. Shear yielding of plate	71.9 kips	119.8 kips	167.7 kips	65.9 kips	113.8 kips
2. Shear rupture of plate	87.5 kips	145.8 kips	204.2 kips	77.2 kips	135.5 kips
3. Block shear of plate	92.9 kips	151.2 kips	209.6 kips	82.8 kips	141.1 kips
4. Bearing/tear-out of plate	112.4 kips	194.8 kips	277.2 kips	102.1 kips	184.5 kips
5. Flexural yielding of plate	179.7 k-in	499.2 k-in	978.5 k-in	151.0 k-in	450.5 k-in
6. Shear strength of bolts, N	59.6 kips (63.6)	101.0 kips (106.1)	125.6 kips (148.5)	74.5 kips (79.6)	124.4 kips (132.6)
eccentricity	0.75 in.	1.25 in.	3.25 in.	0.75 in.	1.25 in.
Coefficient C	2.81	4.69	5.92	2.81	4.69
7. Shear strength of weld (I)	106.6 kips	177.6 kips	248.6 kips	81.9 kips	143.6 kips
eccentricity	3 in.	5 in.	7 in.	3 in.	5 in.
Shear strength of weld (II)	123.6 kips	193.3 kips	263.4 kips	96.7 kips	158.5 kips
eccentricity	2 in.	4 in.	6 in.	2 in.	4 in.
Experimental load	94 kips (84)*	137 kips	160 kips	79 kips	130 kips
Governing limit state	Bolt shear	Bolt shear	Bolt shear	Bolt shear & weld	Bolt shear
Average shear force/bolt	31.3 kips (28)*	27.4 kips	22.9 kips	26.3 kips	26 kips
Average shear stress/bolt	70.8 ksi (63.4)*	62 ksi	51.8 ksi	59.5 ksi	58.9 ksi

Notes: 1. Numbers in shaded blocks are the governing limit state for the connection.
2. Numbers in parentheses are the shear strengths of the bolt groups without eccentricity, e.g. direct shear.
3. * indicates different results reported in the tests (Astaneh et al. 1988, Astaneh et al.1989).

than the tested strength. The average tested shear stress for the bolts is 62 ksi.

7-A325 bolt connection: The governing limit state for this seven-bolt connection is shear strength of the bolt group, with or without eccentricity. The predicted strength of the connection is 125.6 kips (or 148.5 kips without eccentricity), which is 20 percent less than the tested strength, 160 kips. Unlike the first two connections, the predicted shear yielding capacity of the plate exceeds the strength from the test. The average shear stress of the bolts is 51.8 ksi.

3-A490 bolt connection: The governing limit state for this three-bolt connection is the yield strength of the plate. The edge distance used in this connection was reduced from a typical 1-1/2 in. to 1-1/8 in., which reduced the plate shear yielding and weld rupture strength. Also, the actual size of the welds was slightly less than the specified 1/4 in. Even though it was reported that the connection failed by both bolt and weld rupture, the predictions indicate that the bolt might have ruptured after the welds ruptured. It is noted that the 3-A490 bolt connection failed before the 3-A325 bolt connection, presumably because of shorter welds.

The average shear stress of the bolts (59.5 ksi) is nearly the nominal shear stress of A490 N-type bolts (60 ksi). This average shear stress of the bolts from the test, and the shear strength of bolts without considering eccentricity, shown in the parentheses, indicates that there might not be eccentricity on the bolt group. It also should be noted that the value for shear rupture of the plate is lower than the tested capacity.

5-A490 bolt connection: The governing limit state for this five-bolt connection is the yield strength of the plate. However, the failure mode of the tested connection was bolt fracture. It is noted that the tested strength of this 5-A490 bolt connection was below the tested strength of the 5-A325 bolt connection. The average shear stress of the bolts (58.9 ksi) is again close to the nominal shear strength of A490 bolts (60 ksi). The tested strength of the connection is slightly below that of the bolt group when eccentricity is ignored.

Conclusion: From the data in Table 3.10, the nominal shear strengths of the bolts in the first three connections are much lower than the values from the test results. When eccentricity is taken into the account, the predicted strengths are further reduced. The greatest difference is with the 7-bolt connection because the current design model suggests that eccentricity increases with the number of bolts.

Sarkar tests. The test results reported by Sarkar (1992) and predicted strengths are shown in Table 3.11. Reported material properties were used in the calculations. The average yield strength of the plate was 47.4 ksi and the average ultimate strength was 65 ksi. The measured average tensile strength of the bolts was 120 ksi, which is the same as used in previous calculations. The major difference in Sarkar's connection configurations from Astaneh's is the a-distance, which was 3.5 in. Calculation details can be found in Appendix B. Observations for each test follow:

2-A325 bolt connection: The governing limit state for this two-bolt connection is shear strength of the bolt group, with or without considering the eccentricity. The predicted strength of the connection is 21.4 kips, which is over 50 percent less than the test. The tested strengths of the connection are 51.8 and 60.8 kips. Failure was caused by weld rupture; no bolts fractured during the test.

4-A325 bolt connection: The governing limit state for this four-bolt connection is shear strength of the bolt group. The shear strength of the bolt group with or without the eccentricity is nearly the same since the eccentricity prediction is only 0.5 in. The test values are from three stages of the experiment. The topmost bolt in two connections ruptured when the load reached 66.5 and 84.6 kips in the North and the South connections, respectively. The results show great inconsistency in the experimental data. However, the maximum capacity of 84.6 kips is close to the predicted value.

6-A325 bolt connection: The governing limit state for this six-bolt connection is shear strength of the bolt group, with or without eccentricity. In this case, the predicted strength of the connection is higher than the test value. The capacity of the connection from the experiment is only 102 and 107 kips, whereas Equation 3.1 predicts that the capacity of the connection should be 119.4 kips.

Table 3.11 Sarkar's Experimental Results vs. Design Specifications

Type of Limit States	2-A325 N bolt	4-A325 N bolt	6-A325 N bolt
1. Shear yielding of plate	64.0 kips	128.0 kips	192.0 kips
2. Shear rupture of plate	62.2 kips	124.3 kips	186.5 kips
3. Block shear of plate	72.5 kips	134.7 kips	196.8 kips
4. Bearing/tear-out of plate	75.9 kips	163.7 kips	251.5 kips
5. Flexural yielding of plate	106.7 k-in	426.6 k-in	959.9 k-in
6. Shear strength of bolts, N	21.4 kips (42.4)	82.3 kips (84.9)	119.4 kips (127.3)
Eccentricity	2.5 in.	0.5 in.	1.5 in.
Coefficient C	1.01	3.88	5.63
7. Shear strength of weld (I)	49.4 kips	177.6 kips	266.4 kips
Eccentricity	3.5 in.	4 in.	6 in.
Shear strength of weld (II)	86.6 kips	198.4 kips	286.1 kips
Eccentricity	1 in.	3 in.	5 in.
Experimental number	51.8 kips	66.5 kips	102 kips
Governing limit state	Weld tearing at one end	Topmost bolt in North connection sheared off	Topmost bolt in North connection sheared off
	60.8 kips	84.6 kips	107 kips
	Weld tearing at another end	No failure in South connection (loading 92 in.)	Topmost bolt in South connection sheared off
		81.6 kips	119 kips
		Topmost bolt in South connection sheared off (loading 78 in.)	Second topmost bolt in North connection sheared off
		93 kips	
		Stopped (bolt line Deflection)	
Average shear force/bolt	25.9 or 30.4 kips	23.2 kips	19.8 kips
Average shear stress/bolt	58.6 or 68.8 ksi	52.6 ksi	44.9 ksi

Notes: 1. Numbers in shaded blocks are the governing limit state of each connection.
2. Numbers in the parentheses are shear strengths of bolts without eccentricity.

3.1.4 Final Observation

In addition to the highlighted problem regarding the eccentricity of the bolt group, the predicted shear yielding capacity of the plate is lower than the experimental value in many cases. In the 3-A325 and 3-A490 bolt connections, the shear rupture of the plate is also lower than the test value. In the 3-A325 bolt connection, the block shear of the plate is lower than the tested strength as well.

The shear yielding and shear rupture of the plate limit states are calculated using the following relationships:

Shear yielding of plate

$$R_n = 0.6 A_g F_y \tag{3.7}$$

Shear rupture of plate

$$R_n = 0.6 A_n F_u \tag{3.8}$$

The $0.6F_y$ term in Equation 3.7 is from the von Mises yield criterion. The AISC Specifications use $0.6F_u$ in Equation 3.8 assuming similar behavior as for yielding. The relationships also assume that the shear stress is constant through the section. According to fundamental structural analysis, a shear stress distribution for any rectangular section is a parabolic shape with the maximum value located in the middle. This holds true for any beam section. Therefore, the behavior of the plate needs to be further investigated to clarify this aspect.

3.2 Summary of Current Problems and Scope of Research

As described throughout this chapter, the current design model does not predict the behavior of single plate shear connections accurately and effectively. The results from the experiments carried out by Astaneh also did not agree with the design method he developed. In addition, as stated in Chapter II, variables investigated in Astaneh's research were insufficient. Connections tested only had one value of a-distance, which was 2.75 in. The thickness of the plate, and the amount of displacement due to beam rotation, for example, should also be investigated.

The purpose of this research is to find a better solution to the design of single plate shear connections by using computer simulations. The finite element analysis program ABAQUS is used exclusively throughout the research. Description of model construction is in Chapter IV.

The results of the simulations, including the evaluation of the finite element model with the available test results and the investigation of the parameters involving the design of the connection, are presented in Chapter V. In addition, a number of cases with minimum a-distance under direct shear loading are carried out to verify the bolt shear strength of the connection. The plate behavior is observed to verify the validity of the current plate formulas. Furthermore, double-column bolt single plate shear connections are studied. Finally, the effect of the position of the connection with respect to the neutral axis of the beam, which is a center of rotation, is investigated.

The results of the research are summarized in Chapter VI along with the proposed design model and suggestions for further studies regarding single plate shear connections.

Chapter IV
Finite Element Model

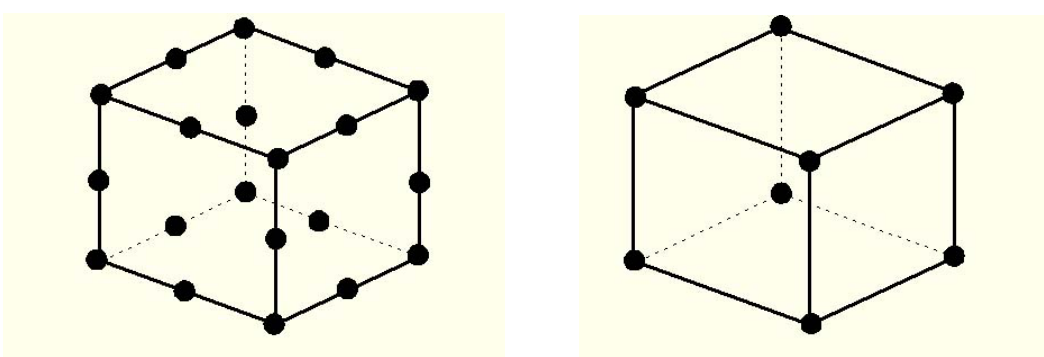
4.1 Introduction

The ABAQUS program (2000a) was used for the simulations reported in this study. The elements used, modeling detail, and analysis techniques are described in the following sections. The simulation models consist of the beam, the shear tab plate, the bolts, and the welds.

4.2 Element Selection and Related Problems

The following elements were chosen for the simulations:

The C3D20 element, a solid 20 node second-order element, is used throughout the model where the stress is significant and the geometry of the model permits. The C3D8 element, a solid 8-node first-order element, is used in regions where the stress is low or of little interest, such as the beam flanges, the beam web portion under the bottommost hole, and the beam web some distance from the bolt line. Use of the C3D8 element in these areas significantly reduces running time. The C3D8 and C3D20 elements are shown in Figure 4.1.



(a) C3D20 Element (b) C3D8 Element

Figure 4.1 Solid Continuum C3D8 and C3D20 Elements

The modeling of the connection involves contact problems between a bolt and a bolt hole. A first-order element is suggested in the ABAQUS Manual (2000a) for contact problems. However, the contact areas of concern are generally in the vicinity of high stress. Consequently, a first-order element at this location is not suitable, and it is necessary to refine the mesh so that the contact is smooth, with second-order elements used to capture the stress concentrations. Therefore, the second-order reduced-integration C3D20R elements are used throughout the body of the plate, the beam web region that contains the bolt holes, the bolts, and the welds.

The element shape can affect the accuracy and the running time of a simulation. Whenever possible, a hexahedral or brick element of Figure 4.1 is used. Nevertheless, in some regions, such as the innermost elements of the bolts, hexahedral elements cannot be used due to the geometry confinement. The second-order prism element C3D15 is used instead to fill in these regions.

The incompatible element C3D8I is used in the remaining portion of the beam to simulate beam rotation, with as much accuracy at least cost as possible.

4.3 Mesh Refinement

An optimal mesh refinement study was carried out with the shear tab plate. A model of a 3/8 in. x 4-1/4 in x 9 in. plate, which was used in Astaneh's experiments, was simulated using a relatively fine mesh with an element size of 0.25 in. x 0.25 in. x 0.1875 in. and a very refined mesh with an element size of 0.125 in. x 0.125 in. x 0.1875 in. as shown in Figures 4.2 and 4.3. Two elements through the plate thickness were used for both cases. The plate was attached to two 1/4-in. E70xx welds, which are also meshed with the same refinement as the plate, by using the ABAQUS TIE constraint option. The welds were restrained at the back as if they were attached to a rigid column flange. To avoid catenary effects, every node which forms the back of the weld was restrained in two directions instead of three except for the two extra nodes which were restrained in the out-of-plane direction to control the movement in that direction. The plate was loaded by a uniformly distributed pressure at the center of each hole to simulate the bolts in the connection without excessively complicating the model.

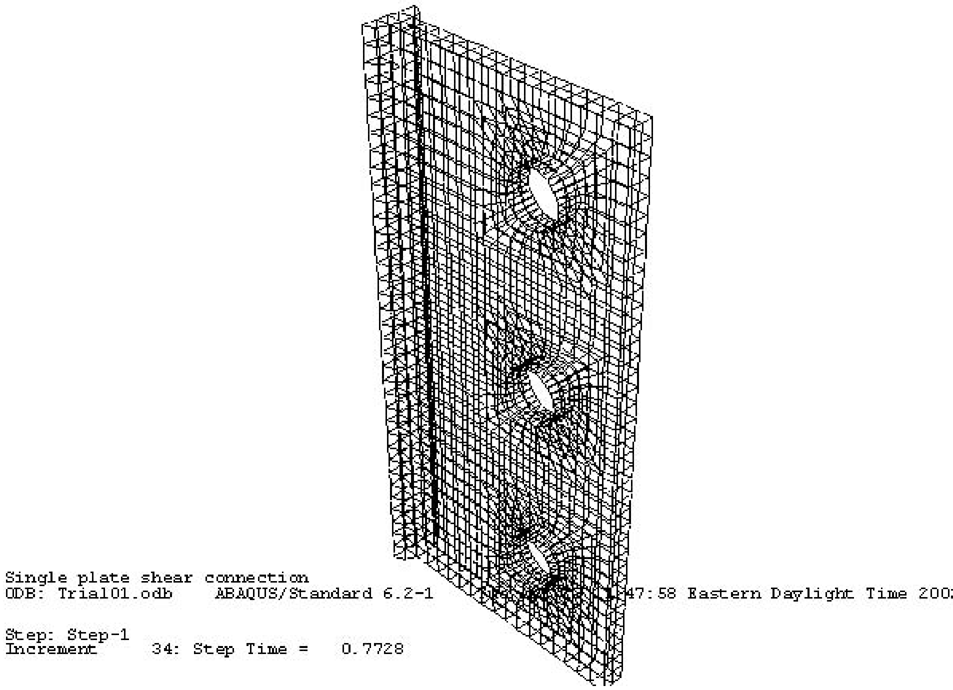


Figure 4.2 Fine Mesh of 3/8x4-1/4x9 in. Plate

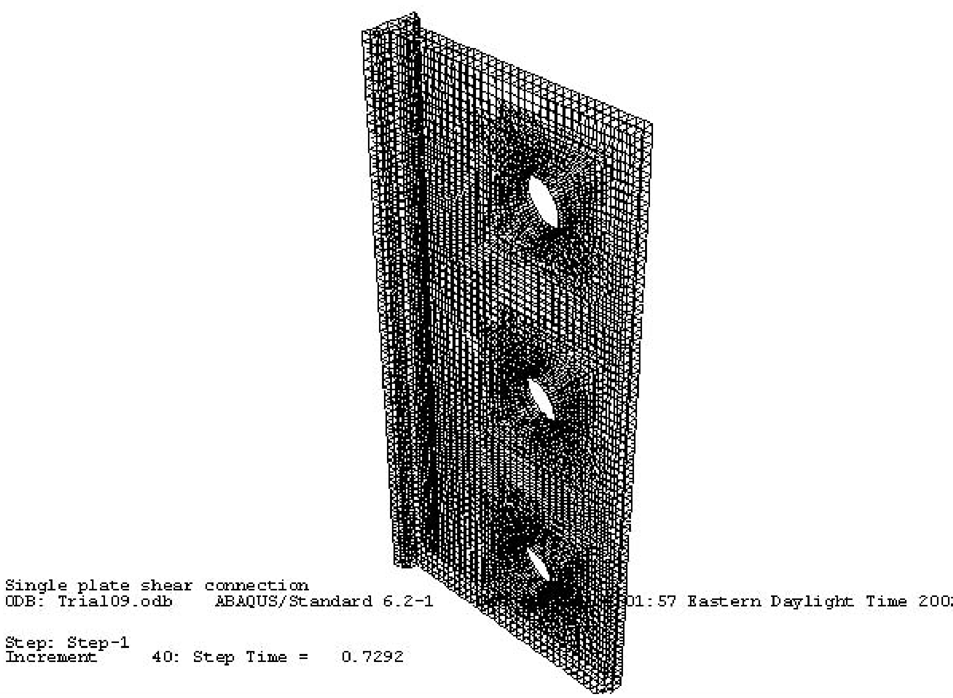


Figure 4.3 Very Fine Mesh of 3/8x4-1/4x9 in. Plate

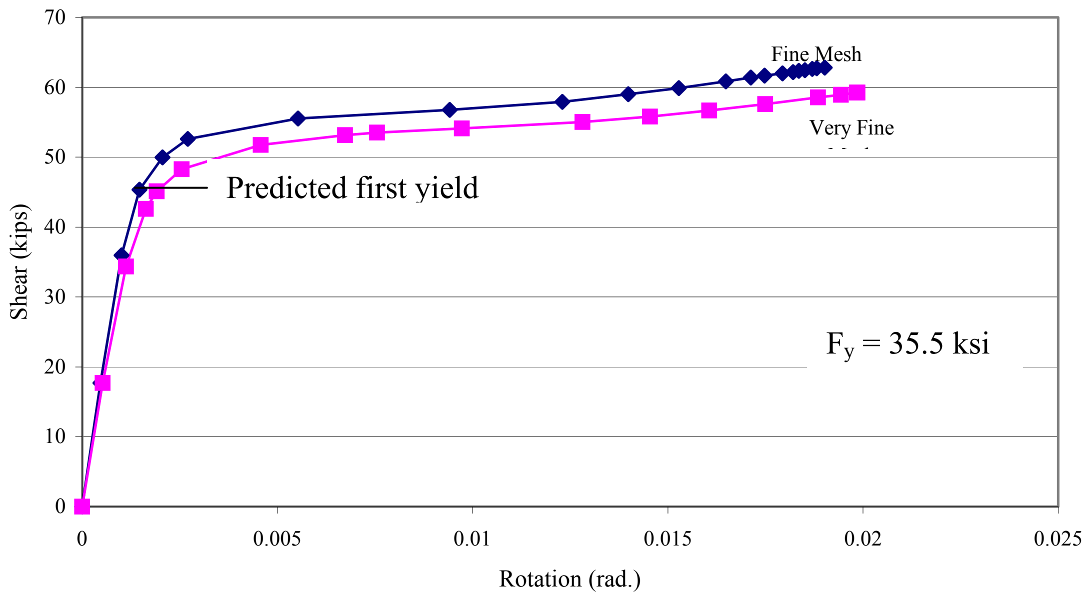


Figure 4.4 Shear vs. Rotation at Bolt Line for Fine and Very Fine Plate

Shear versus rotation results for the two models are shown in Figure 4.4. The model with the fine mesh yields a slightly stiffer result than the model with the very refined mesh, but consumes five times less CPU time.

From the figure, shear yielding occurs very near the von Mises yield criterion shear force $(35.5/\sqrt{3})(0.375 \text{ x } 9.0)/1.5 = 46.1$ kips. The 1.5 factor is to account for the parabolic stress distribution in a plate. It is noted that the nominal shear yield strength from LRFD specification is $0.6F_yA_g = 0.6\text{x}35.5\text{x}0.375\text{x}9 = 71.9$ kips.

The very fine mesh model was selected for the simulation, primarily because the refinement of the model allows the contacts to be simulated more smoothly and effectively. Moreover, the results show that even a more refined mesh is unnecessary since the result given by the very fine mesh is close to the calculated value from the shear yielding force.

To have a smooth contact established at a bolt and a corresponding hole, the bolt must have the same refinement as the plate. To correctly simulate the bearing stress in the bolt hole, the model also includes a 1/16-in. gap between the bolt and the hole since standard bolt holes are 1/16 in. larger than the bolt diameter. Because complete contact between the bolt and the plate is not possible, the ABAQUS contact element, GAP, was

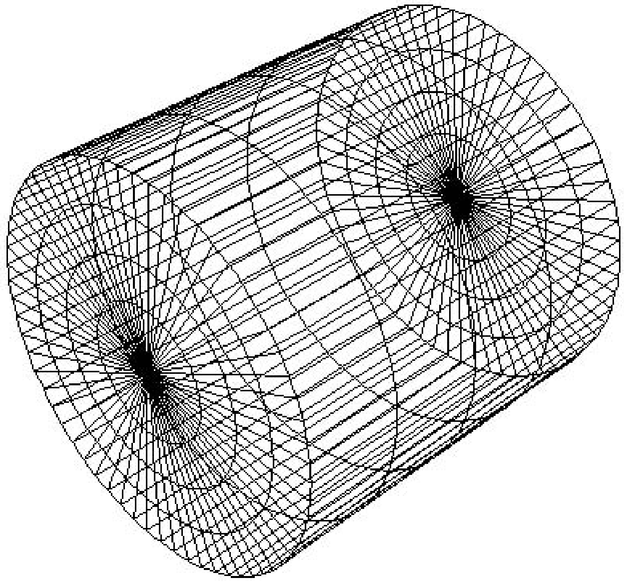


Figure 4.5 Bolt Mesh Used in Simulations

used. The GAP element is explained in Section 4.4. Figure 4.5 shows the bolt mesh used in the simulations.

To complete the mesh used for modelling a bolt, wedge-shaped elements were used to fill the innermost layer. Although these elements have a very poor aspect ratio, comparison of model predictions and calculated nominal strength for a number of models in Chapter V, which were used for calibration purposes, show excellent correlation.

An example of the beam mesh pattern used in the simulations is illustrated in Figure 4.6. In the proximity of the bolt line, where stress is high, the mesh is well refined and a second-order reduced-integration element is used. The refinement is then reduced once the elements are sufficiently away from the bolt line, with the size of the element eventually becoming 3 in. x 3 in. The type of element used was also changed to a first-order incompatible element to save CPU time without losing the beam rotation. The incompatible element is also employed in a zone starting from a distance of 1.5 in. below the center of the lowest bolt for the same reason.

A few models were also constructed to find the most suitable mesh for the beam section. It was found that the refinement of the mesh in the beam flange region does not affect the performance of the simulation. As a result, each flange was modeled with just one element.

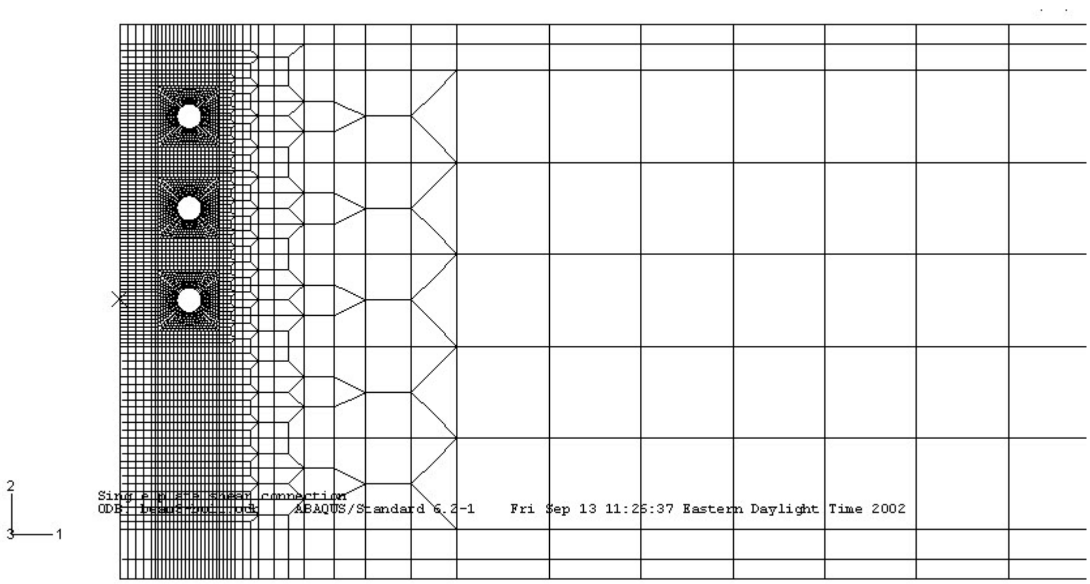


Figure 4.6 A W18x55 beam used with 3-bolt connection

4.4 Special Modelling Techniques

The TIE constraint option was used to connect the plate to the welds. The TIE constraint option eliminates degrees of freedom on the slave surface (the plate) by tying every node to the nodes on the master surface (the weld). The TIE constraint option constrains each node on the slave surface to have the same displacement as its corresponding node on the master surface.

There is one contact problem in the simulations: the contact between the bolt and the holes in the plate and the beam web. The contact requires attention because a bolt and a hole are not initially in contact. The special ABAQUS GAP element is employed to handle the gap involved in the contact. The GAP element is a special element that consists of two nodes that are presumed to come into contact. The element is defined by the initial separation distance and the contact direction. The initial separation distance must be provided, which is the gap distance between the bolt and the hole. The contact direction is then automatically calculated by ABAQUS from the initial coordinates of the two nodes forming the GAP element. The separation distance between two nodes, h, is recalculated in each step of a simulation. The two nodes are in contact when the distance becomes negative. The structure of a GAP element is shown in Figure 4.7.

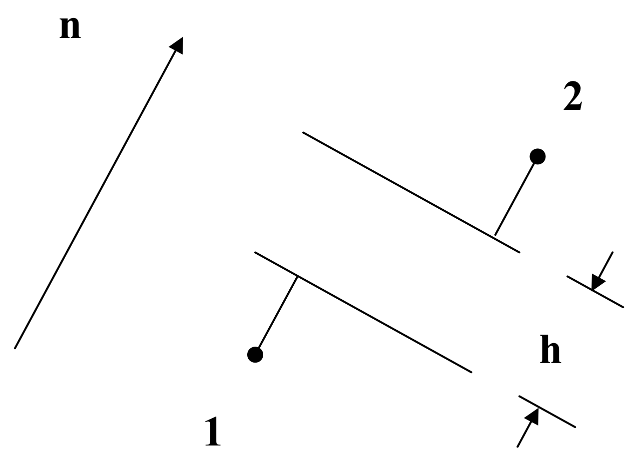


Figure 4.7 Gap Element Used in the Simulations

The GAP element is used around the top half of the holes in the beam web and bottom half of the holes in the plate. The center three peripheral elements of the bolts and the holes are tied together by a TIE constraint, which represents initial contact at this location (top on the beam side and bottom on the plate side). This setup not only simulates the fact that the bolt and the hole are in contact before the connection is loaded, but it also smooths the contact problem in the beginning of the simulation. Figure 4.8 shows a bolt aligned with holes in the beam web and the plate without GAP elements connecting it to the holes. Figure 4.9 illustrates a model after the GAP elements have been installed.

Three options are available in ABAQUS for simulating the relative tangential movement of a contact surface: FINITE, SMALL SLIDING, and INFINITESIMAL. The FINITE option allows any arbitrary movement. The SMALL SLIDING option is used when the movement between the contact surfaces is small but the movements of the two bodies that contain the surfaces might be large. The INFINITESIMAL option is used when the total motion of the structure is small. Since the contact between the bolt and the hole does not involve complex tangential movement or sliding, the FINITE option is not necessary. The plate and the beam in the simulation will undergo large amounts of

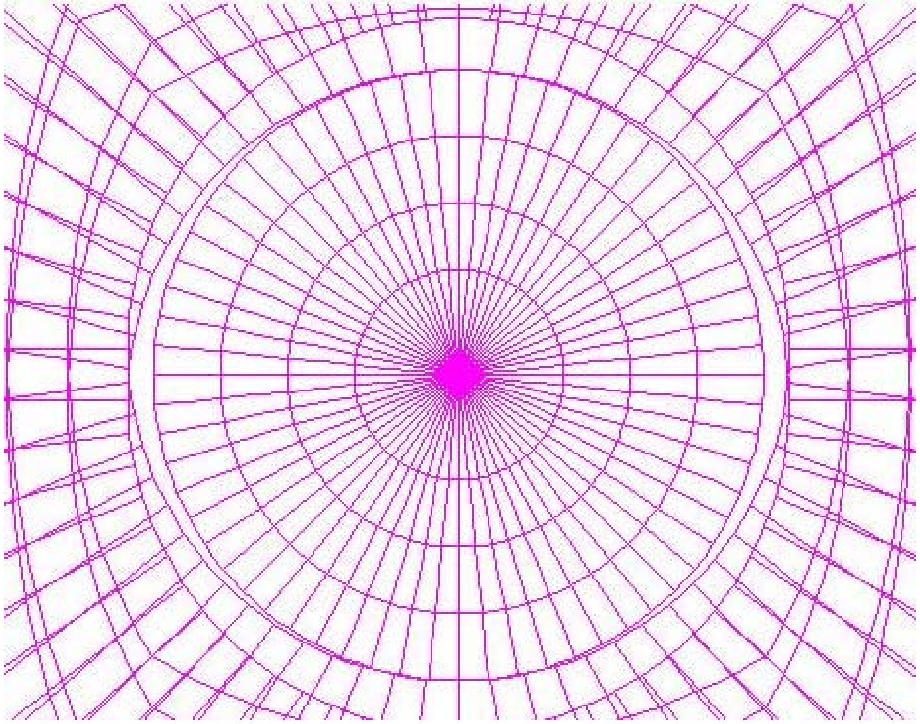


Figure 4.8 A Bolt in Position before GAP Elements Installed

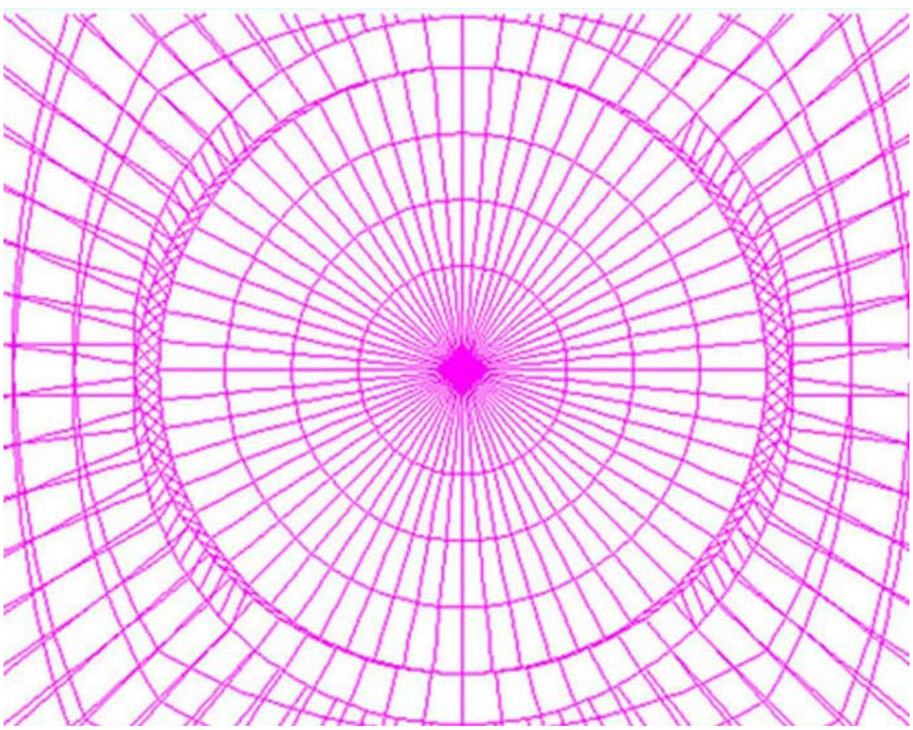


Figure 4.9 A Bolt in Position after GAP Elements Installed

shear deformation and rotation; therefore, the INFINITESIMAL option is not suitable. Consequently, the option SMALL SLIDING was used to define the contact behavior in the tangential direction when the surface of the bolt and the hole come into contact. The SMALL SLIDING option is also used in association with the NLGEOM analysis option so that the effect of geometric nonlinearity is included.

Several options are also available for simulating the interaction normal to a surface. The HARD option introduces contact pressure to the contact pair once the clearance becomes zero. The relationship between the contact pressure and the clearance can be modified by using the MODIFIED HARD option. The SOFTENED option offers some complex relationships between the pressure and the clearance such as an exponential function. To simulate the bearing stress in the model, the straightforward HARD option was used for the pressure-overclosure in the normal direction.

Once the necessary contacts are established in a simulation, all the structural elements are assembled. An example of a complete model is shown in Figures 4.10 and 4.11.

4.5 Boundary Conditions

As discussed in the mesh refinement section, boundary conditions are imposed on the welds by restraining every node on the side of the welds that is connected to the rigid column flange. Every node is restrained in the X- and Y-directions (in plane) except for two nodes that are also restrained in the Z-direction (out of plane). This method is used to minimize catenary effects.

The beam was modeled only to midspan length by using the shear-release boundary condition to take advantage of symmetry. Every node on the beam section that is on the centerline of the beam is restrained in the X-direction. In addition, the beam was also braced along the entire length to prevent lateral torsional buckling.

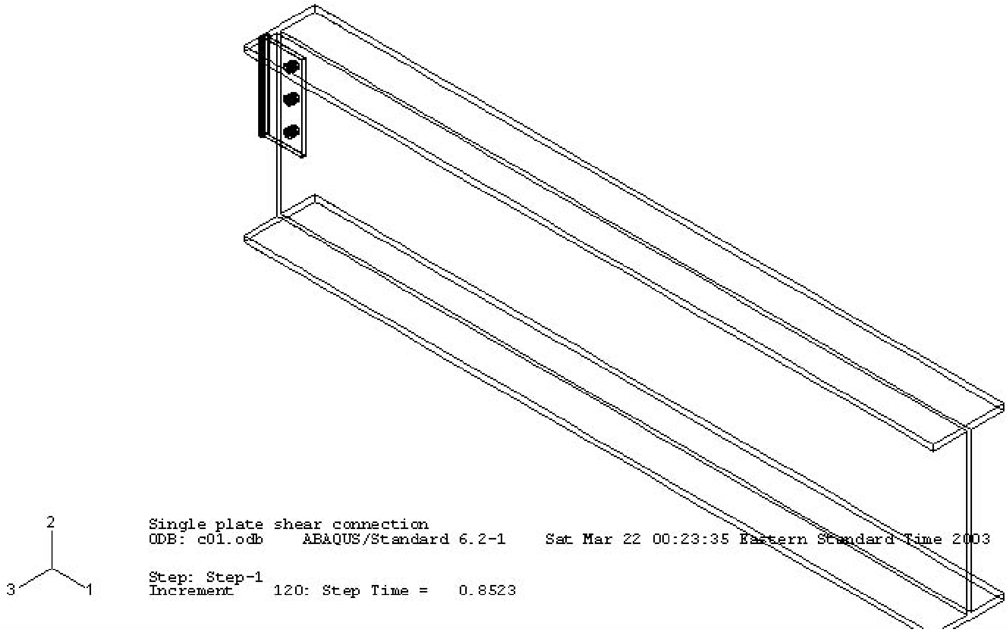


Figure 4.10 Assembled 3-Bolt Connection

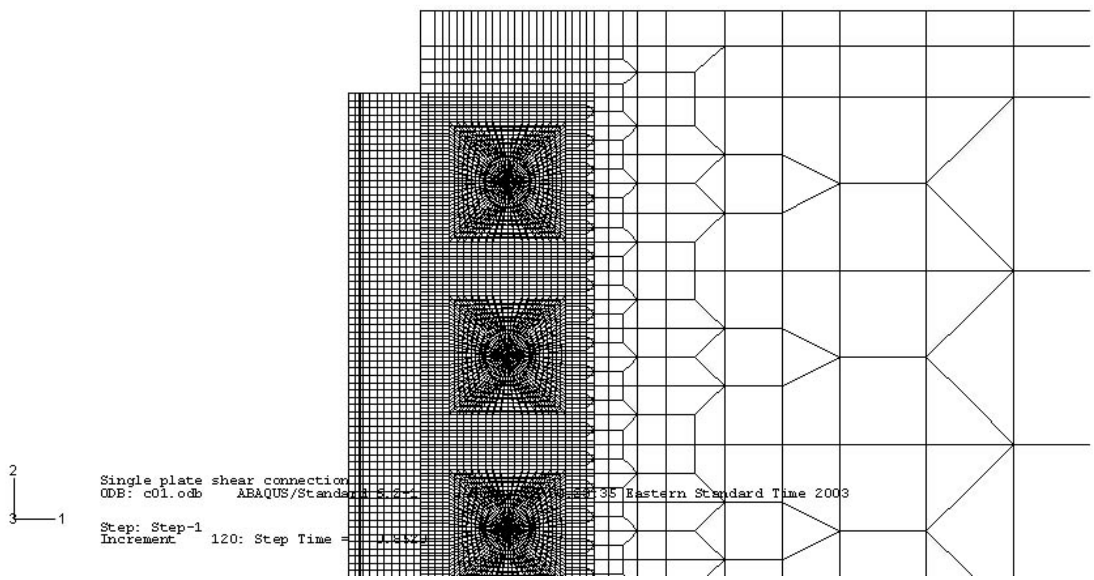


Figure 4.11 Closer Look at Assembled Connection with Mesh Exposed

To simulate the effect of a nut on the bolt, the bolt itself is restrained at both ends in the Z-direction. The boundary conditions on every node at each end keep the bolt in position as if it were locked to a plate and a beam web by a nut. This implementation bypasses the construction of the nut, which would introduce a complex contact problem without being necessary.

4.6 Loading and Analysis

Two types of loading were used in the simulations: a uniformly distributed load and a concentrated load. A uniformly distributed load was placed on the top flange elements to the plane of the beam web throughout the beam length. Alternately, when simulating a test, a concentrated load was placed on the top flange at the location used in the test. Application of the concentrated load required additional attention to prevent failure by beam web yielding or web crippling. In all test simulations, the beam web was strengthened with stiffeners on each side of the web at the location where the concentrated load was applied.

The STATIC method of analysis, which is a regular option in ABAQUS, was used. The NLGEOM option, as previously mentioned in the contact problems section, was used to include the nonlinear effect of the geometry. The rate of loading was linear.

4.7 Material Properties

An elastic perfectly-plastic strain relationship was assumed for the E70xx welds and high strength bolts, and an elastic perfectly-plastic strain relationship with an additional strain-hardening portion was used for the plate and beam material. The modulus of elasticity was taken as 29,000 ksi. Plate and beam material yield stresses varied between the simulations. The stress-strain curves used are shown in Figure 4.12 (Salmon and Johnson, 1996). The stress-strain relationship was converted into true stress and strain as required for ABAQUS input. The Poisson ratio used was 0.3 for every steel material.

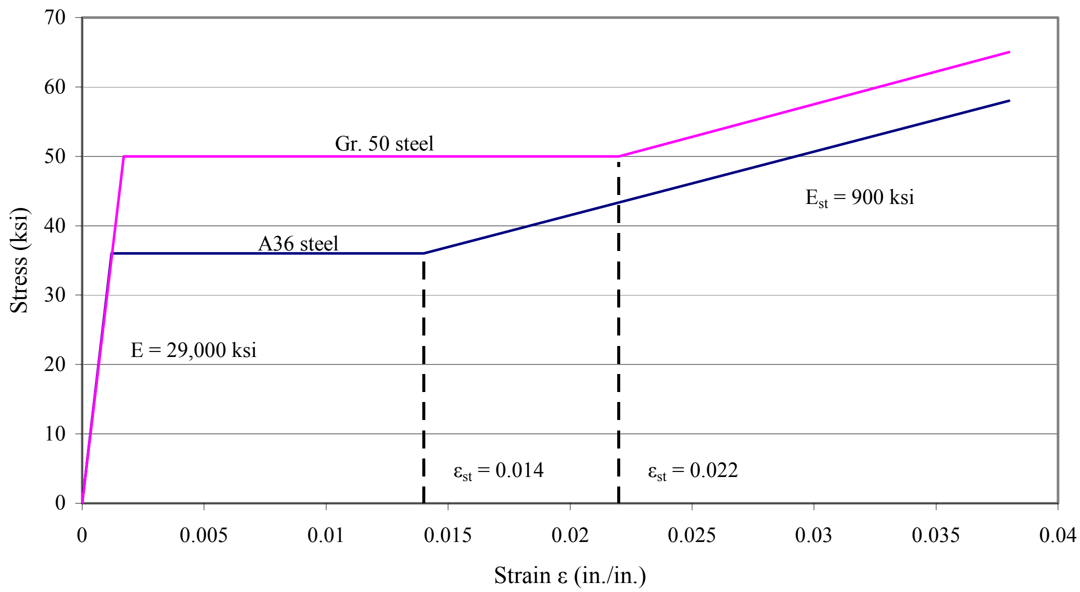


Figure 4.12 Stress-strain curves for A36 and Gr. 50 steel

4.8 Determination of the Strength for Each Limit State

The limit states of a single plate shear connection are shear yielding, shear rupture, block shear and bearing/tear-out of the plate, bearing/tear-out of the beam web, bolt rupture, and weld rupture. However, not every limit state is of interest, nor can every limit state be closely monitored. A summary of the methods used to monitor each limit state and concerns regarding them are:

Shear yielding of plate: This limit state is monitored using a shear versus rotation at bolt line plot as shown in Figure 4.4.

Shear rupture of plate: This limit state was not reported to have occurred in any of the tests reviewed in Chapter II. As stated in Chapter III, bolt movement caused by beam rotation is unlikely to bring about shear rupture failure. In addition, it is impossible to tear apart the two adjacent elements in the finite element models used in the study. Therefore, this limit state is not monitored.

Block shear: This limit state will govern only when the tension strength at the horizontal edge distance is smaller than the shear strength at the vertical edge distance. In all of the simulations, the two distances were of the same length. As a result, this limit state is not of interest.

Bearing/ tear-out: This limit state consists of two parts. The first part is bearing stress of the plate, which is the means to achieve the ductility for the shear tab connection; the limit state is not the cause of a connection failure. Bearing stress failure cannot be demonstrated with a plot, even though it is visible in the simulation. Therefore, this limit state is not monitored. The second part of the limit state is tear-out, which is a rupture limit state. With the same reasons stated with block shear, this limit state is also not monitored.

Bolt rupture: This is the most important limit state to be monitored. This limit state was monitored by observing the shift or fluctuation of shear stress at the centerline of every bolt in the connection. Failure is judged to have occurred when shear stress in the innermost element starts to decrease. Once one bolt has reached its maximum strength, instead of the bolt fracturing, the simulations show a relocation of force to other bolts. As a result, shear stress in the remaining bolts will continue to rise while shear stress in the failing bolt continues to fall. Moreover, the bolt is also judged to have fractured when the shear stress in the outer elements starts to exceed the shear stress in the innermost element. Bolt rupture detection is further explained in the next chapter.

Weld rupture: The limit state of weld rupture is monitored by using the moment at the weld line-beam end rotation diagram. The moment at the weld line is the summation of moments of all X-component reactions from every restrained node.

In summary, the limit states that were monitored in the connection simulations are shear yielding of the plate, bolt rupture, and weld rupture. The beam end rotation, along with the bending moment in the midspan section, was also recorded to observe the ductility of the connection.

Examples of the investigation of these limit states from the results of the simulations, including other useful information such as the behavior of the beam and movement of the point of inflection, are presented in Chapter V. The remaining results are presented in Appendix C.

Chapter V
Finite Element Analyses

5.1 Validation of Finite Element Models

5.1.1 Introduction

The first step in a finite element analysis is to verify how well the model predicts the behavior of the structure. To achieve this goal, eight finite element models, Models 1 through 8, were set up to simulate the connections tested by Astaneh et al. (1988) and by Sarkar (1992). The results of the simulations were then compared to the test results to validate the models. Four additional models were analyzed to verify assumptions used for the first eight models. Models 9 and 10 were created to examine the effect of beam size and length on the connection behavior. Models 11 and 12 were created to examine the effects of loading type and bolt strength on the connection behavior. Details of Models 1 through 12 are summarized in Tables 5.1 and 5.2.

Four plots and one table are constructed to facilitate the analysis of each simulation. The first plot, shear vs. rotation of the plate at the bolt line, is used to verify the behavior of the plate. The second plot, moment at weld line vs. beam end rotation, is for monitoring the weld behavior. The plot of shear vs. beam end rotation is used to observe the beam behavior. The plot of shear vs. distance to point of inflection from the weld line captures the movement of the eccentricity. A shear stress table is employed to demonstrate how a bolt is judged to have failed in a connection. The table format is used for this investigation rather than the plot format because data was not available to determine shear deformation. Further, the table format shows the instability of the bolt behavior more clearly, thus the failure of the bolt is easily seen. Starting with Model 8, because of utilization of the ABAQUS contact output request feature, the amount of force each bolt carries when the bolt failure occurs is also available.

Table 5.1 Details of Finite Element Models 1 through 8

Simulation	Beam		Bolts		Bolt Str.*	a-distance	PL Yield**	PL Dimensions	Reference Tests
	Size	Span (ft)	No.	Type	(ksi)	(in.)	(ksi)	(in.)	
1	W18x55 A36	12	3	A325X	110	2.75	35.5	3/8x4-1/4x9	Astaneh et al. (1988)
2	W18x55 A36	12	5	A325X	110	2.75	35.5	3/8x4-1/4x15	
3	W24x84 A36	16	7	A325X	110	2.75	35.5	3/8x4-1/4x21	
4	W18x55 Gr.50	12	3	A490X	140	2.75	35.5	3/8x3-7/8x8-1/4	
5	W18x55 Gr.50	12	5	A490X	140	2.75	35.5	3/8x3-7/8x14-1/4	
6	W12x35 A36	21	2	A325X	120	3.50	47.4	3/8x5x6	Sarkar (1992)
7	W18x76 A36	33	4	A325X	120	3.50	47.4	3/8x5x12	
8	W21x93 A36	25	6	A325X	120	3.50	47.4	3/8x5x18	

Table 5.2 Details of Finite Element Models 9 through 12

Simulation	Beam		Bolts		Bolt Str.*	a-distance	PL Yield**	PL Dimensions	Investigation
	Size	Span (ft)	No.	Type	(ksi)	(in.)	(ksi)	(in.)	
9	W24x84 Gr.50	12	5	A325X	110	2.75	35.5	3/8x4-1/4x15	Beam rotation (compare with Sim. 2)
10	W24x84 Gr.50	12	7	A325X	110	2.75	35.5	3/8x4-1/4x21	Beam rotation (compare with Sim. 3)
11	W18x76 A36	26	4	A325N	96	3.50	47.4	3/8x5x12	Bolt strength (compare with Sim. 7)
12	W21x93 A36	23.5	6	A325N	96	3.50	47.4	3/8x5x18	Bolt strength (compare with Sim. 8)

All bolts are 3/4-in. diameter bolts. All welds are 1/4-in.

* Assumed value ** Measured value

5.1.2 Discussion of Selected Results

To provide an overview of typical results, results of Models 1 and 11 are discussed in this section. Model 1 simulated the 3-A325 bolt connection with an A36 plate tested by Astaneh et al. (1988). Model 11 simulated the 4-A325 bolt connection with a Gr. 50 plate tested by Sarkar (1992). Results for the other simulations are in Appendix C.

Model 1

Results from Model 1 are shown in Figures 5.1 through 5.4 and Table 5.3. From Figure 5.1, the behavior of the plate can be observed. The initial yield point was reached when the shear force was approximately 50 kips, which is very close to the beam shear yield strength calculated using the shear formula for a rectangular plate (0.6x35.5 x0.375x9 /1.5 = 47.9 kips). The plate entered the inelastic region shortly after the shear force reached 50 kips. The plot then began to form a plateau after the shear force reached 80 kips.

The behavior of the welds is monitored in Figure 5.2. The corresponding value of shear force when the moment at the weld line reached its highest value is 93.1 kips.

Figure 5.3 demonstrates that the behavior of the beam remained elastic throughout the simulation. Figure 5.4 illustrates the movement of the point of inflection. The point of inflection moved toward the beam in the beginning of the simulation and then started moving toward the support once the shear force reached 60 kips. When the bolt was judged to have failed, the point of inflection was located almost exactly at the bolt line. This location of the point of inflection correlates with the moment at the weld line, which is equal to the shear force times the a-distance.

The bolt rupture strength of the connection was determined from the shear stress of the bolts in the connection. As shown in Table 5.3, the topmost bolt was judged to have failed after the shear reached 85.5 kips. The shear stress in the bolts clearly became unstable after the shear reached this value.

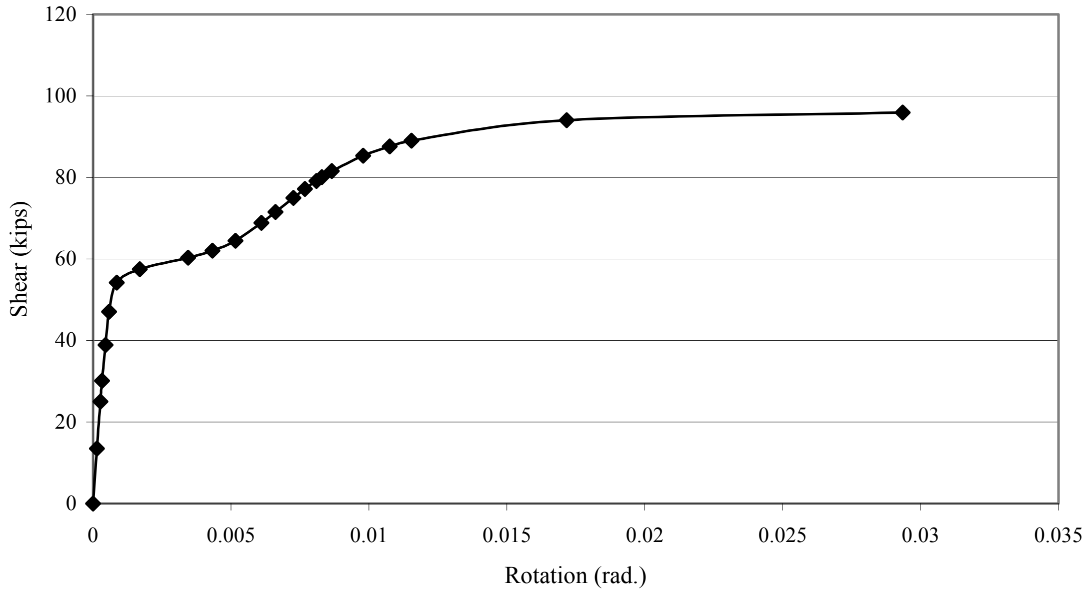


Figure 5.1 Shear vs. Rotation of Plate at Bolt Line of Model 1

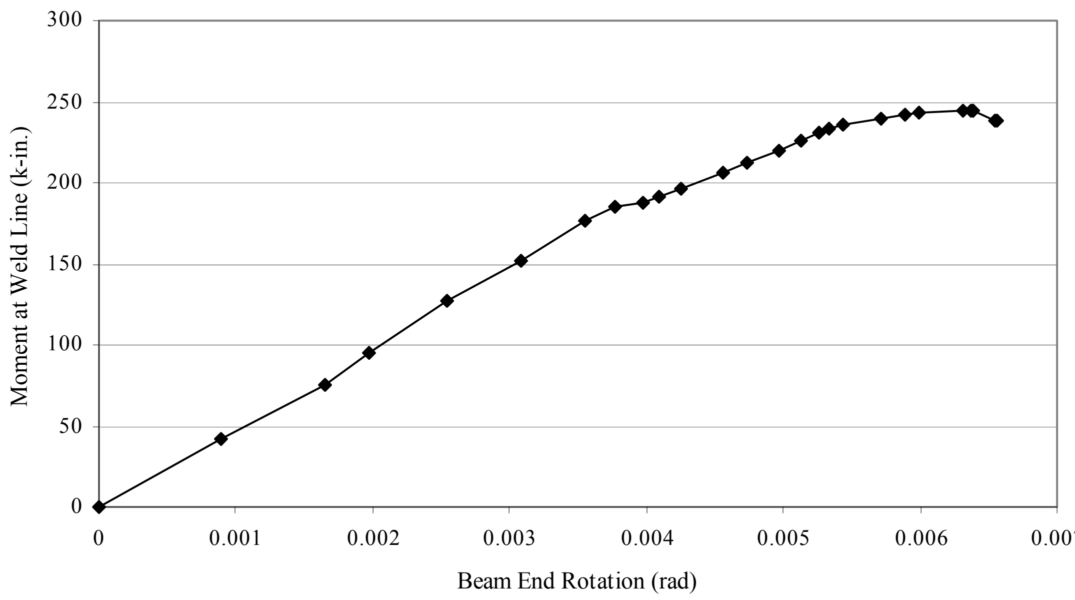


Figure 5.2 Moment at Weld Line vs. Beam End Rotation of Model 1

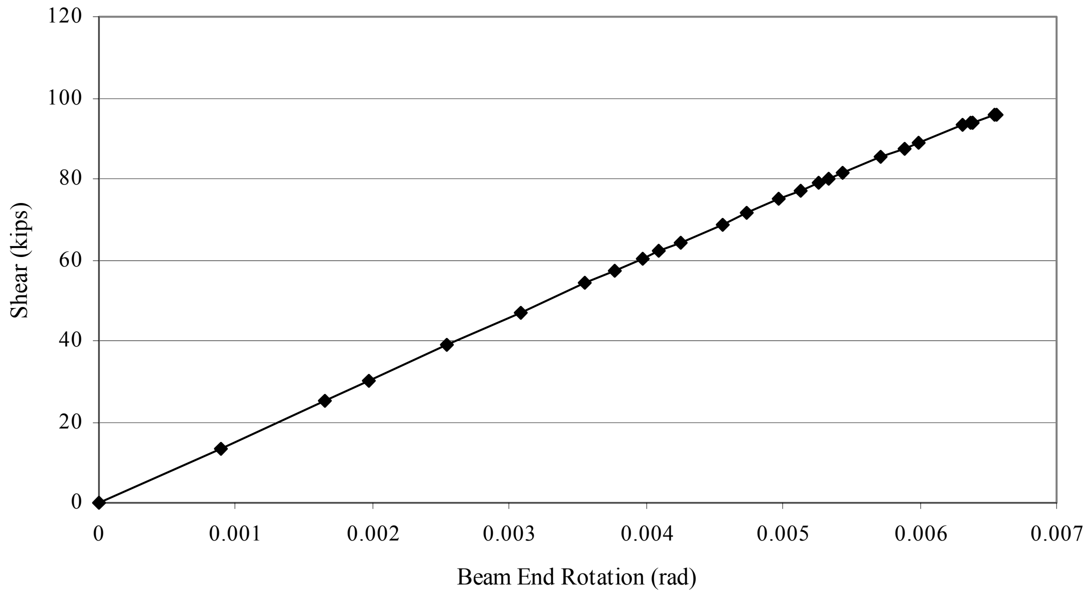


Figure 5.3 Shear vs. Beam End Rotation of Model 1

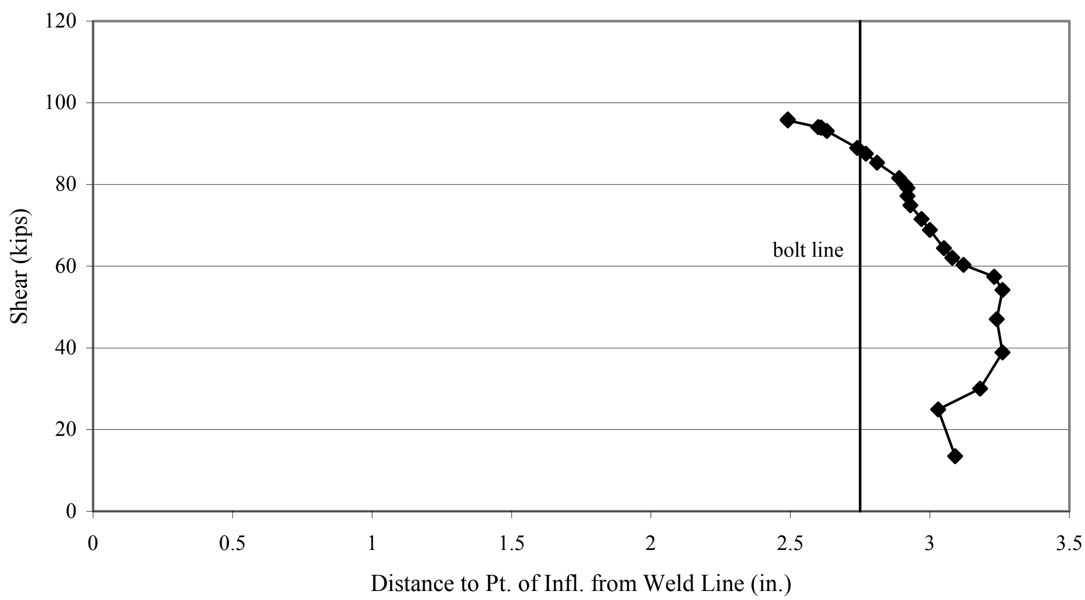


Figure 5.4 Shear vs. Distance to Point of Inflection from Weld Line of Model 1

Table 5.3 Shear Stress in Bolts of Model 1

Increment		84	86	88	90	92	94	96	98	100	102	104	106	108	110
Load		84.2	85.5	86.2	87.0	87.4	87.9	88.2	88.5	89.2	90.1	90.9	92.0	93.1	93.7
1st bolt	beam side	57.99	58.29	58.28	58.18	58.11	58.01	57.94	57.87	57.66	57.33	56.97	56.57	56.10	56.05
	plate side	58.84	59.34	59.63	59.88	60.01	60.17	60.26	60.35	60.54	60.70	60.82	60.90	61.11	61.39
2nd bolt	beam side	57.99	58.35	58.53	58.59	58.60	58.54	58.50	58.46	58.30	58.07	57.81	57.20	56.56	56.52
	plate side	58.40	58.91	59.21	59.53	59.70	59.87	59.94	60.02	60.19	60.44	60.62	60.70	60.82	60.99
3rd bolt	beam side	56.98	57.79	58.19	58.42	58.55	58.70	58.78	58.86	58.91	58.88	58.73	58.44	57.92	57.48
	plate side	57.78	58.18	58.42	58.70	58.86	59.06	59.16	59.28	59.58	59.86	60.07	60.30	60.44	60.43

Bolt shear rupture strength as determined from the FEM results

Indicates that the stress decreases in the next increment

Indicates that the stress in the outer element exceeds the stress in the innermost element

This result, judged to be the controlling limit state, compares to the value obtained from the experiment (94 kips).

Model 11

The purpose of Model 11 was to set up a simulation to make a comparison with Model 7 in terms of bolt strength. The connection configuration is identical to that in Model 7, except the bolt strength was reduced 20 percent to account for the effect of threads. The length of the beam was such that beam yielding at midspan could occur before bolt failure. The required beam length was determined by setting the moment at midspan of the beam, $M=wl^2/8$, equal to its full plastic moment strength, M_p. The value of w is calculated from the expected strength of the connection (bolt rupture without eccentricity) divided by the beam length to convert to a uniform load, resulting in a beam length of 26 ft. The simulation was then used to prove that single plate shear connections can provide the required rotation at the maximum bending strength of the beam.

The results of Model 11 are illustrated in Figures 5.5 through 5.8 and in Table 5.4. Similar to the plate behavior in Model 1, the plate reached the first yield point when the value of the shear force was 85 to 90 kips, the same value as calculated by using the beam shear yielding formula (0.6x47.4x0.375x12/1.5 = 85.3 kips). The plate then formed an inelastic plateau, but did not reach the strain hardening stage as the plate in Model 1.

The maximum moment at the weld line corresponds to a shear value of 102.5 kips, which is less than the ultimate strength of the weld.

The behavior of the beam is shown in Figure 5.7. Bolt shear rupture occurred in the connection just before the beam started to yield, which proves the ductility of the connection as intended with the precalculated beam length. Therefore, the idea of using this approach is implemented henceforth so that the ductility of the connection is justified.

The movement of the point of inflection throughout the simulation is shown in Figure 5.8. The point of inflection moved from the bolt line toward the beam until the shear force in the simulation reached 60 kips, then started moving toward the support. When bolt failure occurred, the point of inflection was approximately 6.5 in. from the support, or 3 in. from the bolt line.

When the bolt strength is reduced 20 percent to account for threads, the bolt shear rupture strength of the connection, as determined from Table 5.4, is 83 kips. This strength is very close to the test result (81.6 and 84.6 kips).

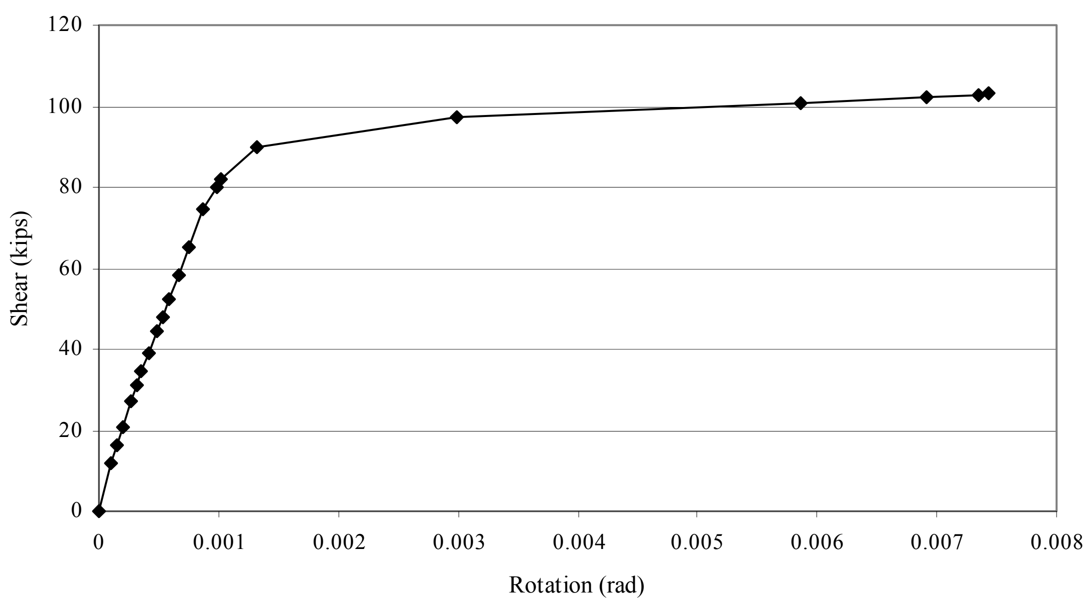


Figure 5.5 Shear vs. Rotation of Plate at Bolt Line of Model 11

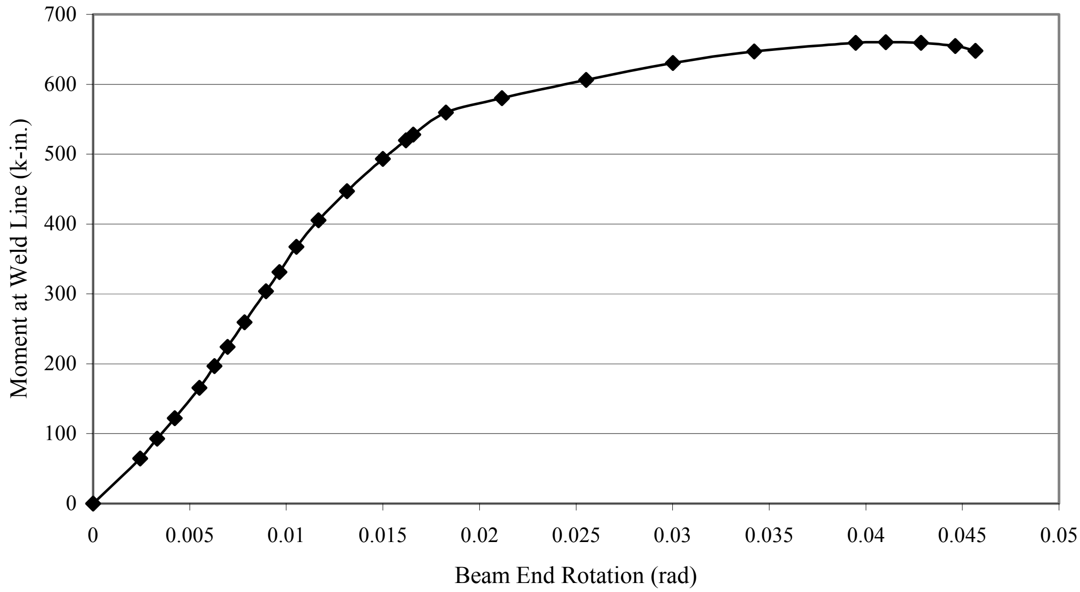


Figure 5.6 Moment at Weld Line vs. Beam End Rotation of Model 11

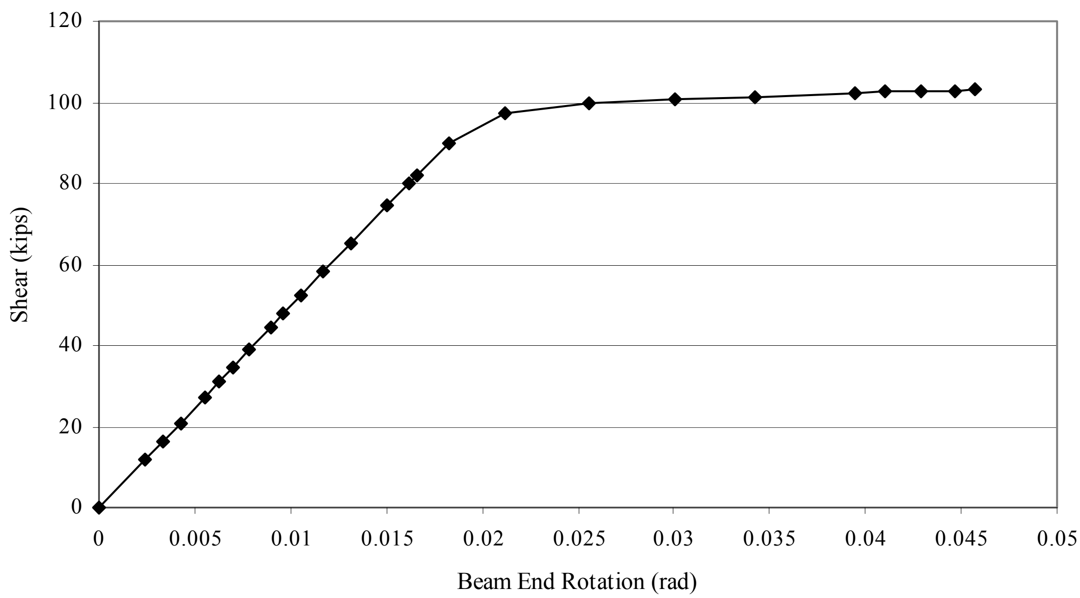


Figure 5.7 Shear vs. Beam End Rotation of Model 11

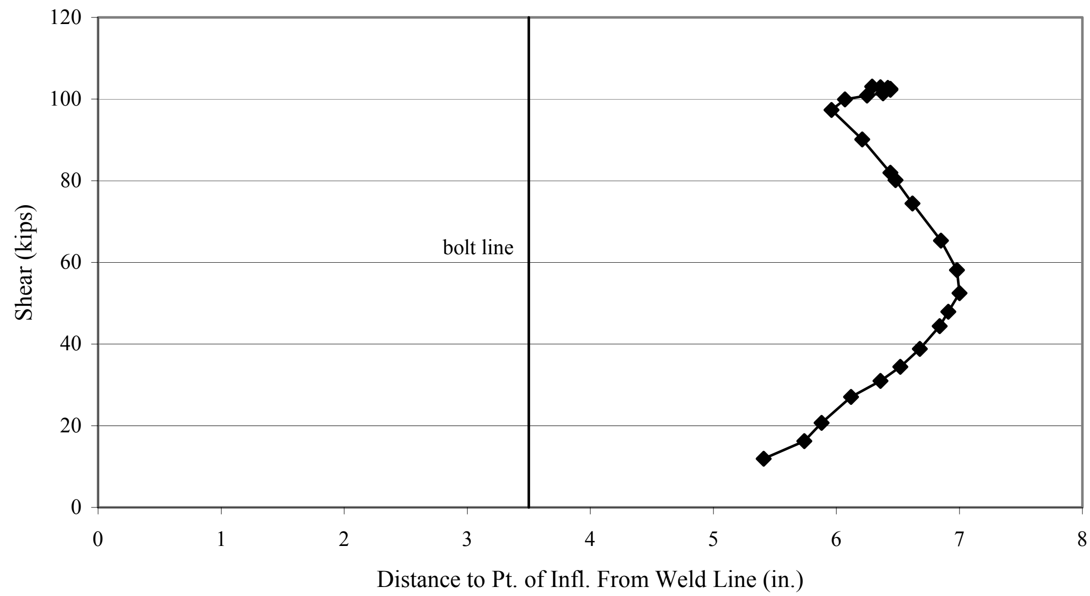


Figure 5.8 Shear vs. Distance to Point of Inflection from Weld Line of Model 11

Table 5.4 Shear Stress in Bolts of Model 11

Increment		54	56	58	60	62	64	66	68	70	72	74	76	78	80
Load		77.25	80.14	81.04	81.95	83.03	84.24	86.85	90.08	91.89	93.75	96.04	97.33	98.65	99.94
1st bolt	beam side	35.97	37.38	37.75	38.08	38.53	38.87	39.54	40.25	40.51	40.69	40.63	40.23	39.23	37.94
	plate side	35.25	36.67	37.04	37.39	37.87	38.25	39.10	40.02	40.49	40.86	41.16	41.09	40.47	39.26
2nd bolt	beam side	39.42	39.62	39.70	39.83	39.99	40.22	41.10	41.97	42.46	42.73	42.78	42.62	42.13	41.46
	plate side	40.24	40.58	40.73	40.93	41.29	41.63	42.62	43.59	44.25	44.86	45.22	45.23	45.04	44.77
3rd bolt	beam side	40.92	42.43	42.99	43.45	43.78	44.12	44.89	45.89	46.42	46.73	46.91	46.78	46.33	45.33
	plate side	41.46	43.08	43.66	44.25	44.82	45.40	46.37	47.92	49.02	50.13	51.62	52.29	52.69	52.63
4th bolt	beam side	40.19	41.17	41.42	41.62	41.88	42.27	43.04	43.82	44.19	44.80	45.41	45.76	45.72	45.02
	plate side	40.77	41.81	42.08	42.31	42.63	43.02	43.86	45.00	45.61	46.56	48.40	49.52	50.59	51.07

Bolt shear rupture strength

Indicates that the stress decreases in the next increment

Indicates that the stress in the outer element exceeds the stress in the innermost element

5.1.3 Overall Results and Discussion

The results of the twelve simulations are summarized in Table 5.5. Following are conclusions drawn from each aspect of the simulation results.

Table 5.5 Summary of Simulation Results for Models 1 through 12.

Model	Bolts in Simulation	Bolts in Test	Simulation Prediction (kips)	Test Results (kips)	Ratio of Prediction/Exp	Source
1	3-A325X	3-A325N	85.5	84, 94	1.02, 0.91	Astaneh et al. (1988)
2	5-A325X	5-A325N	131.6	137	0.96	
3	7-A325X	7-A325N	189.6	160	1.19	
4	3-A490X	3-A490N	84.7	79 (W)	1.07	
5	5-A490X	5-A490N	158.3	130	1.22	
6	2-A325X	2-A325N	57.7	51.8, 60.8 (W)	1.11, 0.95	Sarkar (1992)
7	4-A325X	4-A325N	109.1	81.6, 84.6	1.34, 1.29	
8	6-A325X	6-A325N	128.8	102, 109	1.26, 1.18	
9	5-A325X	5-A325N	140.3	137	1.02	Astaneh et al. (1988)
10	7-A325X	7-A325N	197.0	160	1.23	
11	4-A325N	4-A325N	83.0	81.6, 84.6	1.02, 0.98	Sarkar (1992)
12	6-A325N	6-A325N	104.1	102, 109	1.02, 0.96	

Note: The failure mode is bolt shear rupture unless indicated otherwise
W = Weld rupture

The performance of the model. Results of the ABAQUS simulations, especially the bolt strength, depend largely on the material properties used. The first eight simulations were set up using X-type bolt strengths to match test results reported by Astaneh et al. (1988). In Models 11 and 12, the bolt strength was reduced 20 percent to account for thread effects, that is, N-type bolts.

The predicted failure loads for Models 1, 2, 4, 6 and 9 are within 10 percent of the test loads reported by Astaneh et al. (1988) and Sarkar (1992). The predicted strengths of Models 3, 5, 7, 8, and 10 are approximately 20 percent greater than the test results. If the bolt strength used in Models 3, 5 and 10 is reduced 20 percent to account for threads in the shear plane, the results of the simulations are close to the experimental results. The results from Models 11 and 12, which have the same connection configuration as Models 7 and 8, respectively, show that the strength of the connection in the simulation depends on the material property of the bolts. The results from Models 11 and 12, with the bolt strength reduced 20 percent, are also close to the experimental results.

The behavior of the beam. Richard's proposed design model for the connection is a function of the beam (Richard, 1980), whereas, Astaneh's is not (Astaneh et al., 1993). Models were constructed with different beams to examine the effects of beam size and span.

Comparison of three 5-bolt connections with different beams is illustrated in Figure 5.9. Model 2 has a 12 ft span W18x55 A36 beam, Model 5 has a 12 ft span W18x55 Gr. 50 beam, and Model 9 has a 12 ft span W24x84 Gr. 50 beam. Beam rotation for all of the models, despite having the same connection configuration, is different. Models 2 and 5, which have the same beam section, have the same rotation path in the elastic region, whereas Model 9, which has a different beam, has a different rotation route.

Comparison of 7-bolt connections with different beams is demonstrated in Figure 5.10. Models 3 and 10 have the same connection geometry and beam size (W24x84), but the beam length and material used are different: 12 ft and Gr. 50 versus 16 ft and A36. The results in Figure 5.10 show that the rotation of the beam does not depend on the geometry of the single plate shear connection; it depends on the beam properties. Rotations obtained from the classical method, shown in Figures 5.9 and 5.10, are identical to the results from the simulations.

To further emphasize that the beam rotation is a function of beam properties, the rotations of the same beam sections with different connection configurations are summarized in Figures 5.11 and 5.12. The beam end rotations of Models 1, 2, 4, and 5 are plotted together in Figure 5.11. Despite being attached to 3- and 5-bolt connections, the beam end rotations are identical. The beam end rotations of Models 3, 9, and 10 are plotted in Figure 5.12. Again, in spite of different connection configurations, beams in Models 9 and 10 behave similarly. The beam in Model 3 rotates differently in the elastic region because it has a different length.

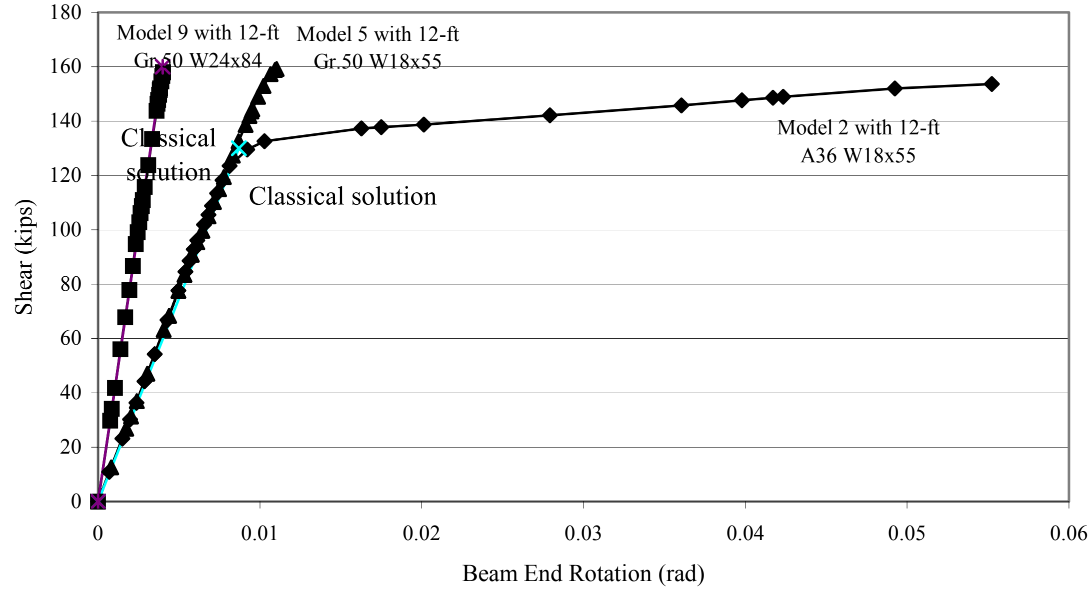


Figure 5.9 The 5-Bolt Connections with Different Beam Sections

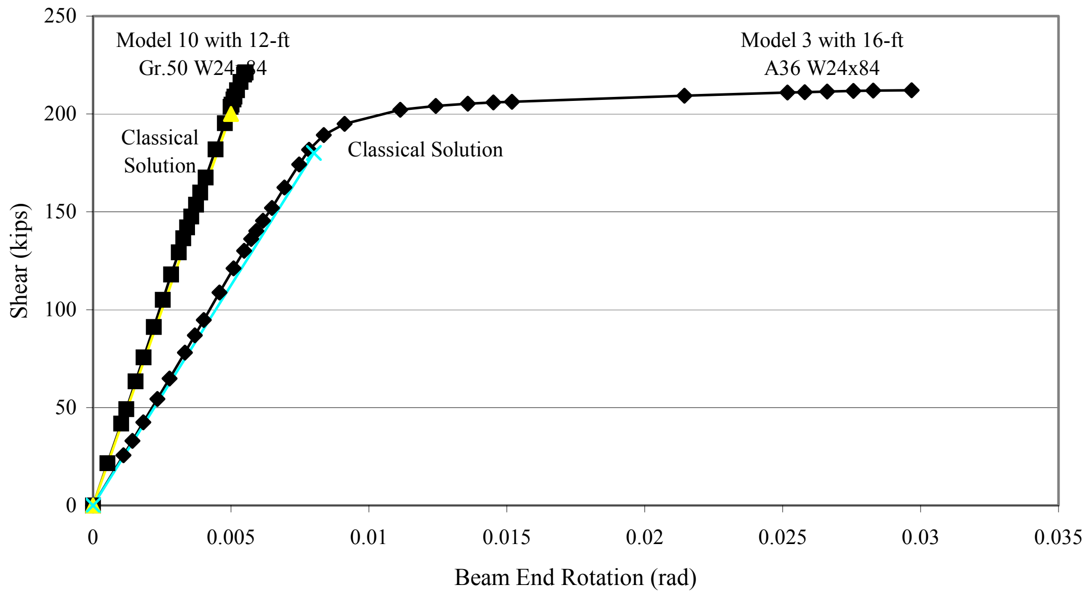


Figure 5.10 The 7-Bolt Connections with Different Beams

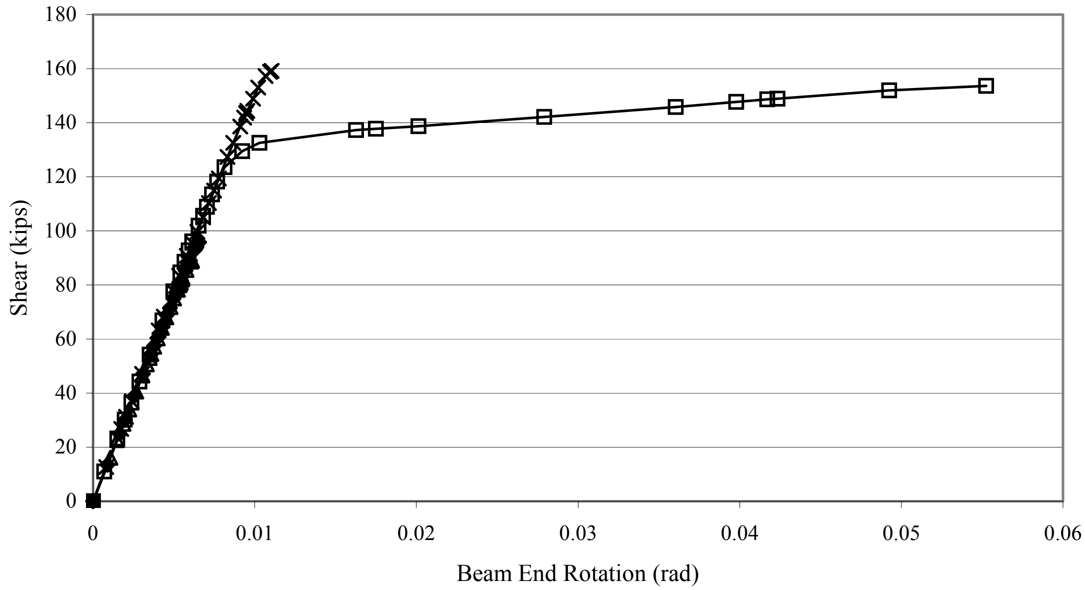


Figure 5.11 The 12-ft W18x55 with 3- and 5-Bolt Connections

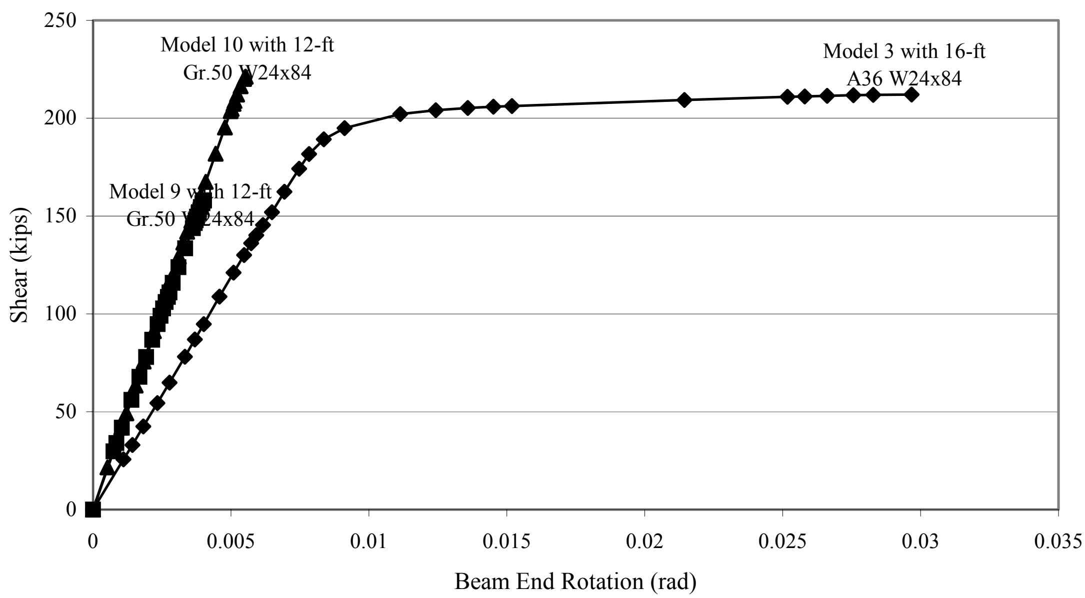


Figure 5.12 W24x84 with 5- and 7-Bolt Connections

From Figures 5.9 through 5.12, it is concluded that single plate shear connections behave as regular shear connections and do not affect the beam end rotation. The rotation of the beam is completely a function of the beam section and beam length.

The effect of type of bolts. Models 7 and 11, and Models 8 and 12, share the same connection configurations except for the bolt strength used. The bolt tensile strength used in Models 7 and 8 was 120 ksi, whereas the bolt tensile strength used in Models 11 and 12 was 96 ksi, a 20 percent decrease to account for the effect of threads in the shear plane. The results of Models 8 through 12 compared to direct shear strength of the connections (bolt area times bolt shear rupture strength times number of bolts) are summarized in Table 5.6.

Table 5.6 Bolt Shear Strength of Models 8 through 12 vs. Direct Shear Strength

Model	No. of Bolts	Bolt Tensile Strength (ksi)	Bolt Shear Strength (ksi)	Direct Shear Strength (kips)	Simulation Prediction (kips)	Ratio of Direct/ Simulation
7	4	120	72	127.0	109.1	0.86
11	4	96	58	101.6	83.0	0.82
8	6	120	72	190.5	128.8	0.68
12	6	96	58	152.4	104.1	0.68

Note: 3/4-in. dia. bolts

The results from the simulations show that, in spite of the different bolt strength, the ratios of the predicted strength to direct shear strength of the bolt group in the corresponding connections are nearly the same for connections with the same number of bolts. In Models 7 and 11, which simulate a 4-bolt connection, the predicted shear strengths are 86 and 82 percent of direct shear strengths, respectively. In Models 8 and 12, which simulate a 6-bolt connection, the predicted shear strengths in both models are 68 percent of direct shear strengths. From this observation, different bolt strengths used in connections can be treated by means of a factor.

The behavior of the plate. The plate behavior of Models 1 through 12 is summarized in Figure 5.13 with the corresponding values shown in Table 5.7. The values are calculated using the following.

The beam shear yielding of the plate is calculated by using the beam shear formula for any rectangular section:

$$R_n = 0.6F_yA_g/1.5 \tag{5.1}$$

where

A_g = gross area of plate (in.^{2})

The values of effective shear yielding are calculated by using a relationship developed from an observation of the simulation shear stress distribution (shown in Figure 5.14) in a plate cross section when the plate is about to reach the strain hardening stage:

$$R_n = 0.6F_y[(n\text{-}1)p + L_e]t \tag{5.2}$$

where

n = number of bolts

p = pitch (in.)

L_e = edge distance in vertical direction (in.)

The values of shear yielding and shear rupture limit states calculated according to the Specifications (AISC, 1999) are also included in the table for comparison. All the calculations exclude the strength reduction (ϕ) factor.

The shear yielding strength is calculated using:

$$R_n = 0.6F_yA_g \tag{5.3}$$

The shear rupture strength is calculated using:

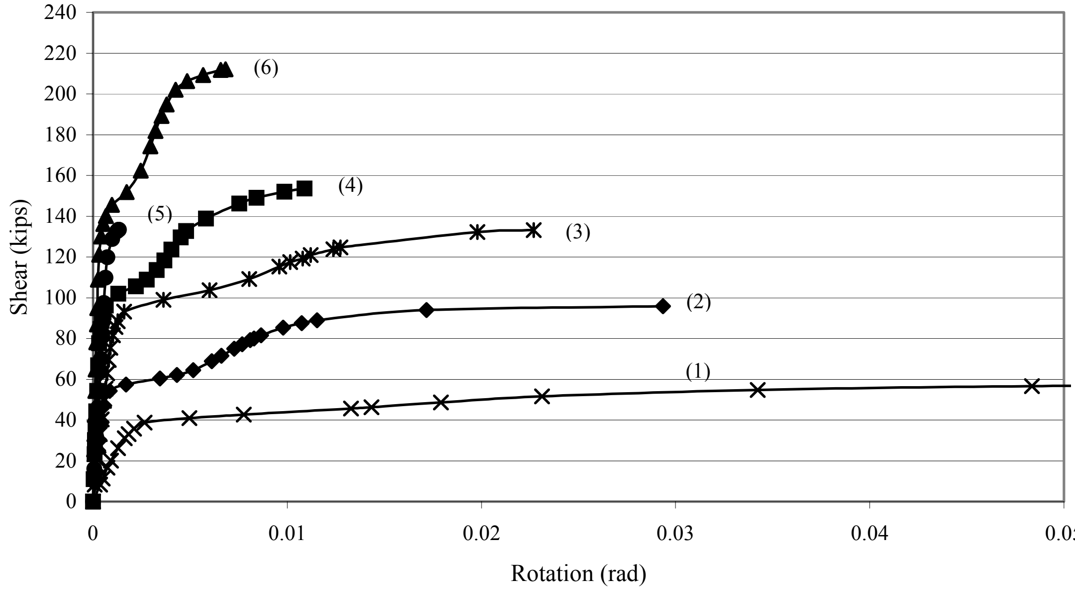
$$R_n = 0.6\ F_uA_{nv} \tag{5.4}$$

where

A_{nv} = net area subject to shear (in.^{2})

The plate bending strength shown in the table is calculated using the plastic section modulus of the plate. The bending strength is then divided by the a-distance, the assumed moment arm:

$$R_n = F_y(tL^2/4)(1/a) \tag{5.5}$$



(1) 3/8x5x6 in. (47.4 ksi) plate; 2-bolt connection (Model 6)
(2) 3/8x4-1/4x9 in. (35.5 ksi) plate; 3-bolt connection (Model 1)
(3) 3/8x5x12 in. (47.4 ksi) plate; 4-bolt connection (Models 7 and 11)
(4) 3/8x4-1/4x15-in. (35.5 ksi) plate; 5-bolt connection (Models 2 and 9)
(5) 3/8x5x18-in. (47.4 ksi) plate; 6-bolt connection (Models 8 and 12)
(6) 3/8x4-1/4x21-in. (35.5 ksi) plate; 7-bolt connection (Models 3 and 10)

Figure 5.13 Shear vs. Rotation at Bolt Line of Models 1 through 3 and 6 through 12

Table 5.7 Shear Strength of Plates in Figure 5.13

Model	Simulation			Prediction				
	PL Dimension (t, width, depth) (in.)	a distance (in.)	PL Strength (yield/ult.) (ksi)	Beam Shear Yielding (kips)	Effective Yielding (kips)	Shear Yielding (kips)	Shear Rupture (kips)	PL Bend Plastic (kips)
1	3/8x4-1/4x9	2.75	35.5/61	47.9	59.9	71.9	82.4	98.0
2, 9	3/8x4-1/4x15	2.75	35.5/61	79.9	107.8	119.8	137.3	272.3
3, 10	3/8x4-1/4x21	2.75	35.5/61	111.8	155.8	167.7	192.2	533.7
4	3/8x3-7/8x8-1/4	2.75	35.5/61	43.9	59.9	65.9	75.5	82.4
5	3/8x3-7/8x14-1/4	2.75	35.5/61	75.9	107.8	113.8	130.4	245.8
6	3/8x5x6	3.50	47.4/65	42.7	48.0	64.0	58.5	45.7
7, 11	3/8x5x12	3.50	47.4/65	85.3	112.0	128.0	117.0	182.8
8, 12	3/8x5x18	3.50	47.4/65	128.0	176.0	192.0	175.5	411.4

The plate in each connection started to yield when the shear force in the simulation reached the value of $0.6F_yA_g/1.5$. The plate then completely yielded and entered the hardening stage. The inelastic region is visible in the plot for the plates with yield strength of 35.5 ksi (lines 2, 4, and 6 in Figure 5.13). For the plates with yield strength of 47.4 ksi, the inelastic region can be seen in 6 in. and 12 in. plates (lines 1 and 3). In the case of the 18 in. plate, the bolts failed before the plate reached the inelastic behavior (line 6).

From the results and the predicted values shown in Figure 5.13 and Table 5.7, respectively, it is concluded that the behavior of the plate in the simulations can be described by the beam shear yielding formula in Equation 5.1 and the relationship developed from the shear stress distribution before the strain hardening stage in Equation 5.2.

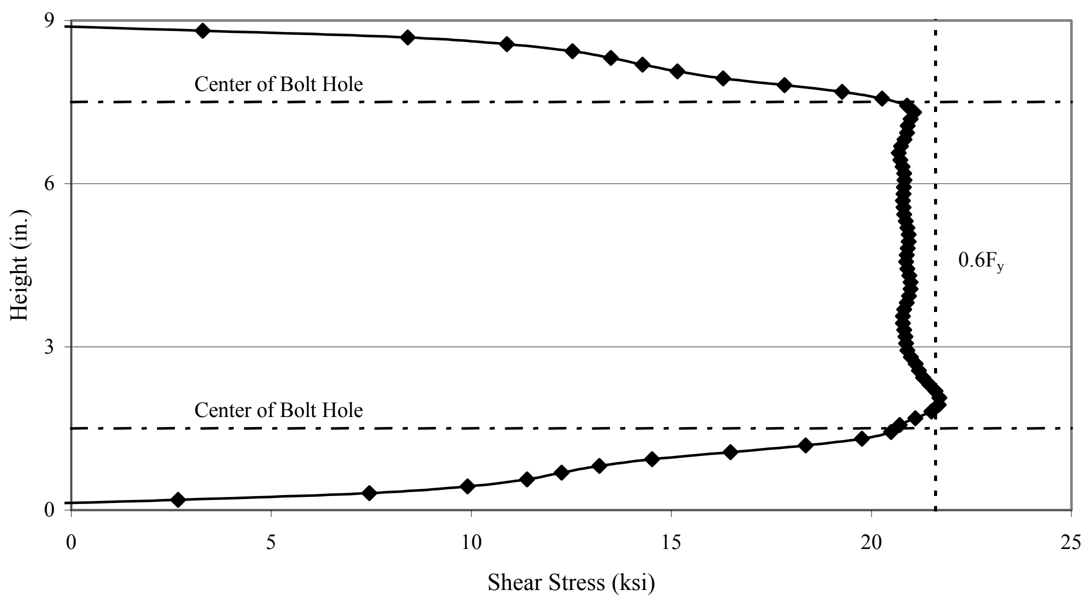


Figure 5.14 Example of Shear Stress Distribution in Plate before Strain Hardening Stage

5.2 Examination of Effect of a-distance and Plate Material on Bolt Group in Connections

5.2.1 Introduction

The test results reported by Astaneh et al. (1988) do not clearly show that the bolt groups experienced any eccentricity as predicted by the design model. The tests, upon which the design model was built, also had only one a-distance to observe. Therefore, it is not possible to conclude whether or not the bolt shear strength is a function of the a-distance.

Models 13 and 14 were set up to evaluate the shear strength of the bolt group under direct shear. In these special cases, the a-distance was set equal to 2.5 in. to minimize the effect that the distance itself might have on a bolt group. The beam was loaded with a concentrated load at 1 ft from the bolt line to simulate the effect of direct shear. Stiffeners were added on both sides of the beam web to prevent web yielding and web crippling. This configuration is referred to as the "pure shear" case. The bolt strength used in Model 13 was 110 ksi (X-type bolts) to make a comparison with previous simulations that were carried out using this bolt strength. The bolt strength used in Model 14 was 88 ksi to account for the effect of threads in the shear plane. Models 15 through 17 were set up with various a-distances and bolt configurations. The bolt strength used in Models 15 to 17 was 88 ksi. The results from Models 1, 9, and 10, which were used to validate the finite element models, are incorporated in the investigation to compare with results from Models 13 through 17 by means of a factor. The details of the models are shown in Table 5.8.

The test results by Sarkar (1992) indicate bolt strength reduction in the connections. A major difference in Astaneh's and Sarkar's research is the strength of the plate. The yield strength of the plate used in Astaneh's research was 35.5 ksi, a value for any standard A36 material, whereas the yield strength of the plate used in Sarkar's research was 47.4 ksi, a value close to Gr. 50 material. The results of the bolt shear rupture in Models 6 through 8, 11, and 12 also demonstrated strength reductions of the bolt groups. As a result, the effect of the strength of the base material on the shear strength of the bolt group should be investigated.

Table 5.8 Details of Finite Element Models 13 through 25

Simulation	Beam		Bolts		Bolt Str.	a-distance	PL Yield	PL Dimensions	Investigation
	Size	Span (ft)	No.	Type	(ksi)	(in.)	(ksi)	(in.)	
13	W18x55 A36	20	3	A325	110	2.50	36.0	3/8x4x9	Pure shear
14	W24x84 A36	17.5	7	A325	88	2.50	36.0	3/8x4x21	
15	W18x55 A36	20	3	A325	88	5.00	36.0	3/8x6-1/2x9	a-distance and bolt config. in A36 PL
16	W18x55 A36	12	5	A325	88	5.00	36.0	3/8x6-1/2x15	
17	W24x84 A36	17.5	7	A325	88	5.00	36.0	3/8x6-1/2x21	
18	W18x55 Gr.50	20	3	A325	88	2.50	50.0	3/8x4x9	Pure shear
19	W24x84 Gr.50	17.5	7	A325	88	2.50	50.0	3/8x4x21	
20	W18x55 Gr.50	28	3	A325	88	3.00	50.0	3/8x4-1/2x9	a-distance and bolt config. in Gr.50 PL
21	W18x55 Gr.50	17	5	A325	88	3.00	50.0	3/8x4-1/2x15	
22	W24x84 Gr.50	24	7	A325	88	3.00	50.0	3/8x4-1/2x21	
23	W18x55 Gr.50	28	3	A325	88	5.00	50.0	3/8x6-1/2x9	
24	W18x55 Gr.50	17	5	A325	88	4.00	50.0	3/8x5-1/2x15	
25	W24x84 Gr.50	24	7	A325	88	5.00	50.0	3/8x6-1/2x21	

All bolts are 3/4-in. diameter bolts

All welds are 1/4-in.

Two additional pure shear cases, Models 18 and 19, were created to measure shear strength of the bolt group in Gr. 50 plates under direct shear. Models 20 through 25 were set up with various a-distances and bolt configurations. The bolt strength used in Models 18 to 25 was 88 ksi to account for the effect of threads in the shear plane. The details of the models are shown in Table 5.8.

In addition to four plots and one table used to investigate the behavior of the connection in the simulation, a plot of bolt movement is also available, starting with Model 13. The plot is constructed by connecting coordinates of the bolt center, which change throughout a simulation. A dotted line connecting each bolt movement indicates locations of the bolts and a pattern of the movement when the bolt failure occurs. Two bolt movement plots are shown in Section 5.2.2 for discussion. The plots and tables for Models 13 through 25 can be found in Appendix C.

The results of the simulations including the bolt shear strength of the connection, the behavior of the plate, statics, and moment in the connections are summarized in the following section.

5.2.2 Results and Discussions

The results of Models 13 through 25 are summarized in Tables 5.9 and 5.10, respectively. Tables 5.9(a) and 5.10(a) show model numbers arranged such that the effect of the a-distance can be easily observed. The model numbers are replaced by the bolt shear strength of the connections in Tables 5.9(b) and 5.10(b), correspondingly. The ratio of the shear strength predicted by the simulations to the nominal direct shear strength of the bolt group is included in both tables for comparison.

Table 5.9 Schemes of Simulations for Investigating Effect of a-distance on Shear Strength of Bolt Group in A36-Plate Connections

(a) Simulation

No. of Bolts	Simulation Number					
	Pure Shear	a-distance (in.)				
		2.75	3.50	4.00	4.50	5.00
3	13	1				15
4						
5		2, 9				16
6						
7	14	3, 10				17

(b) Bolt Shear Strength of Corresponding Simulations

No. of Bolts	Bolt shear strength (kips)/ratio to nominal strength					
	Pure Shear	a-distance (in.)				
		2.75	3.50	4.00	4.50	5.00
3	86.8/0.99	85.5/0.98				66.9/0.96
4						
5		140.3/0.96				109.4/0.94
6						
7	158.9/0.97	197.0/0.96				150.7/0.92

Notes: Nominal strength	3-bolt	5-bolt	7-bolt
A325N (F_v = 52.8 ksi)	69.9 kips	116.5 kips	163.1 kips
A325X (F_v = 66.0 ksi)	87.6 kips	146.0 kips	204.4 kips

Table 5.10 Schemes of Simulations for Investigating Effect of a-distance on Shear Strength of Bolt Group in Gr. 50-Plate Connections

(a) Simulation

No. of Bolts	Simulation Number					
	Pure Shear	a-distance (in.)				
		3.00	3.50	4.00	4.50	5.00
3	18	20				23
4			11			
5		21		24		
6			12			
7	19	22				25

(b) Bolt Shear Strength of Corresponding Simulations

No. of Bolts	Bolt shear strength (kips)/ratio to nominal strength					
	Pure Shear	a-distance (in.)				
		3.00	3.50	4.00	4.50	5.00
3	69.3/0.99	58.7/0.84				57.1/0.82
4			83/0.82			
5		97.3/0.84		96.9/0.83		
6			104.1/0.68			
7	155/0.95	121.1/0.74				119.6/0.73

Notes: Nominal strength	3-bolt	5-bolt	7-bolt
F_u = 88 ksi (F_v = 52.8 ksi)	69.9 kips	116.5 kips	163.1 kips
	4-bolt	6-bolt	
F_u = 96 ksi (F_v = 57.6 ksi)	101.6 kips	152.4 kips	

The effect of the a-distance. In Tables 5.9(b) and 5.10 (b), the ratios of the bolt shear strength obtained from the simulations to the nominal shear strength of the bolt group in both plate materials do not vary with the a-distance, regardless of the bolt configuration. Thus, from the results shown in Tables 5.9(b) and 5.10(b), it is concluded that the a-distance does not have any effect on the shear strength of the bolt group.

The bolt shear strength. In A36-plate connections, bolt groups supporting beams with uniformly distributed load reached their nominal direct shear strength. In Gr. 50-plate connections, the bolt shear strength is significantly reduced under the normal loading condition as shown by the ratios in Table 5.10 (b); however, the bolt groups were able to reach the nominal strength under direct shear.

The shear strength of a bolt group in Gr. 50 plate is reduced once the demand for displacement becomes significant. In pure shear cases, the bolt groups were almost directly loaded with a point load on the beam at 1 ft. from the bolt line. As a result, the beam did not carry much bending moment nor require rotation. To accommodate rotation, the bolts were required to undergo much greater displacement, especially in the horizontal direction. The farther the bolts are from the center of rotation (the neutral axis of the beam), the more they displace. In the A36 material, it is not difficult for a bolt to plow through, but in the Gr.50 material, it is much harder for a bolt to displace, and therefore the vertical shear strength of the bolts is reduced.

Forces acting on bolts. Forces acting on bolts during the simulations for both plate materials are summarized in Tables 5.11 through 5.16, with the direction of the horizontal force illustrated in Figure 5.15. Tables 5.11 and 5.14 illustrate the vertical force, and Tables 5.12 and 5.15 the horizontal force acting on the bolts during the simulations, respectively. Tables 5.13 and 5.16 show the resultant, which is the vector sum of the corresponding horizontal and vertical forces in Tables 5.11, 5.12, 5.14, and 5.15. Bold numbers indicate which bolt(s) failed during the simulations. The force on each bolt is arranged such that the study of the position is carried out more easily. Dotted lines in the tables mark the position of the neutral axis of the beam with respect to the connection. Two dotted lines on the top and the bottom of the force indicate that the particular bolt is located at or close to the beam neutral axis. The single dotted line for case 12 indicates that the neutral axis of the beam is approximately halfway between the two bolts. The sum of the vertical forces on the bolts is the shear strength of the bolt group.

As indicated by bold numbers, the outer bolt(s) always failed first. The shear force carried by the bolts is nearly equal in every case. In the case of A36 plates, the vertical force is approximately 22.1 kips in A325N bolts (F_u = 88 ksi) and 28.3 kips in A325X bolts (F_u = 110 ksi). The horizontal force varies through the configuration; the farther the bolt is from the center of rotation, the greater the horizontal force. However, the force on each bolt is small, with the maximum value less than 6 kips in 7-bolt connections. The resultant demonstrates that all failed bolts reached their shear strength regardless of the bolt type used (23.3 kips for F_u = 88 ksi, and 29.2 kips for F_u = 110 kips). The vertical force carried by the bolts is reduced slightly due to the existence of the horizontal force. As a result, the shear strength of the bolt group, which is the sum of the vertical forces carried by the bolts, is almost the bolt nominal strength when A36-plate is used.

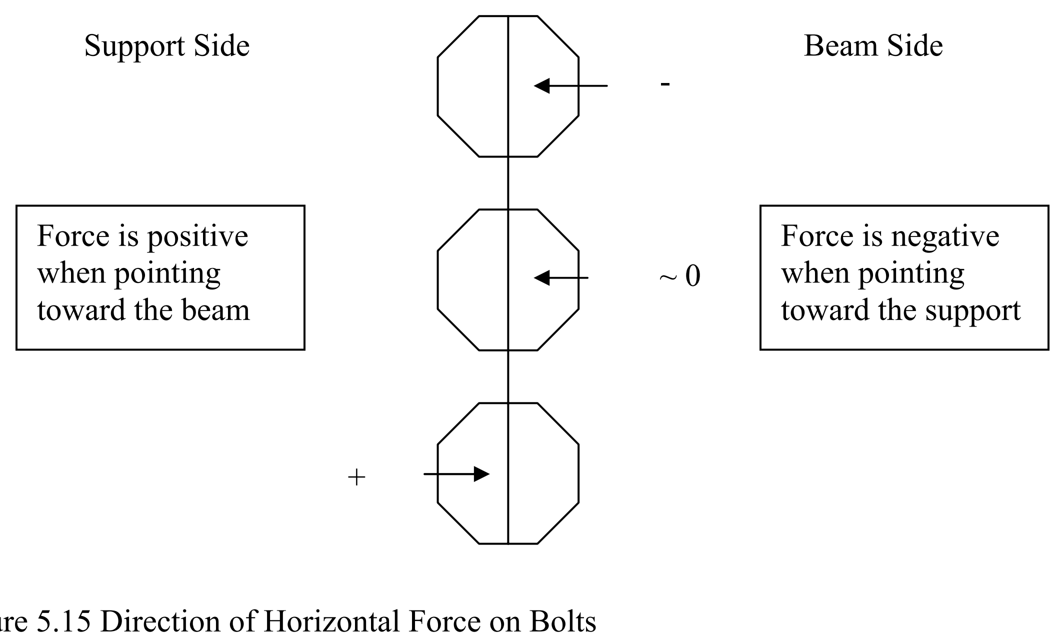


Figure 5.15 Direction of Horizontal Force on Bolts

Table 5.11 Vertical Force Acting on Bolts in A36-Plate Connections

Bolt	Vertical Shear Force, (kips)						
	case 13 3-bolt pure F_u=110	case 15 3-bolt a = 5" F_u=88	case 9 5-bolt a = 2.75" F_u=110	case 16 5-bolt a = 4" F_u=88	case 14 7-bolt pure F_u=88	case 10 7-bolt a = 2.75" F_u=110	case 17 7-bolt a = 5" F_u=88
1					**23.24**	**28.97**	**21.78**
2	**29.31**	**22.53**	**28.97**	**22.27**	**23.16**	**28.84**	**22.45**
3	**29.06**	22.38	**28.62**	22.30	22.86	**28.61**	22.33
4	28.47	22.01	28.16	22.01	22.76	**28.41**	21.81
5			27.71	21.61	22.56	28.08	21.36
6			26.83	21.26	22.48	27.51	21.07
7					22.83	26.62	19.87
Total	86.8	66.9	140.3	109.4	158.9	197.0	150.7

All plates are 3/8-in. thick

All bolts are 3/4-in. dia.

Bold numbers indicate that shear fracture occurs in that bolt

Indicates the position of the beam neutral axis

Table 5.12 Horizontal Force Acting on Bolts in A36-Plate Connections

Bolt	Horizontal Shear Force, (kips)						
	case 13 3-bolt pure F_u=110	case 15 3-bolt a = 5" F_u=88	case 9 5-bolt a = 2.75" F_u=110	case 16 5-bolt a = 4" F_u=88	case 14 7-bolt pure F_u=88	case 10 7-bolt a = 2.75" F_u=110	case 17 7-bolt a = 5" F_u=88
1					**-3.04**	**-3.97**	**-5.44**
2	**-1.26**	**-0.23**	**-2.18**	**-3.19**	**-1.91**	**-2.59**	**-5.98**
3	**-1.80**	1.10	**-1.02**	-2.02	-0.55	**-0.30**	-2.61
4	0.36	5.06	0.99	0.58	0.68	**1.88**	0.49
5			2.47	3.11	1.98	4.08	3.65
6			0.28	2.80	3.24	5.81	6.17
7					2.96	4.34	4.11

Bold numbers indicate that shear fracture occurs in that bolt

Indicates the position of the beam neutral axis

\- Minus sign indicates that the force points towards the support

Table 5.13 Resultant on Bolts in A36-Plate Connections

Bolt	Resultant, (kips)						
	case 13 3-bolt pure F_u=110	case 15 3-bolt a = 5" F_u=88	case 9 5-bolt a = 2.75" F_u=110	case 16 5-bolt a = 4" F_u=88	case 14 7-bolt pure F_u=88	case 10 7-bolt a = 2.75" F_u=110	case 17 7-bolt a = 5" F_u=88
Nominal	29.16	23.33	29.16	23.33	23.33	29.16	23.33
1					**23.44**	**29.24**	**22.45**
2	**29.34**	**22.53**	**29.05**	**22.50**	**23.24**	**28.96**	**23.23**
3	**29.12**	22.41	**28.64**	22.39	22.87	**28.61**	22.48
4	28.47	22.58	28.18	22.02	22.77	**28.47**	21.82
5			27.82	21.83	22.65	28.37	21.67
6			26.83	21.44	22.71	28.12	21.95
7					23.02	26.97	20.29

Bold numbers indicate that shear fracture occurs in that bolt

________ Indicates the position of the beam neutral axis

Table 5.14 Vertical Force Acting on Bolts in Gr. 50-Plate Connections

Bolt	Vertical Shear Force, (kips)									
	case 18 3-bolt pure F_u=88	case 20 3-bolt a = 3" F_u=88	case 23 3-bolt a = 5" F_u=88	case 11 4-bolt a =3.5" F_u=96	case 21 5-bolt a = 3" F_u=88	case 24 5-bolt a = 4" F_u=88	case 12 6-bolt a = 3.5" F_u=96	case 19 7-bolt pure F_u=88	case 22 7-bolt a = 3" F_u=88	case 25 7-bolt a = 5" F_u=88
1								**22.11**	**15.23**	**15.13**
2	**23.04**	**18.50**	**18.10**	**19.93**	**19.70**	**19.68**	**16.31**	22.22	17.50	17.48
3	**23.11**	**20.06**	19.76	21.21	19.54	19.45	18.78	22.30	17.78	17.63
4	23.11	20.10	19.26	20.86	19.26	19.15	17.73	22.35	16.36	16.18
5				21.08	19.36	19.21	17.38	22.20	17.23	16.96
6					**19.42**	**19.39**	17.97	21.99	17.42	17.15
7							15.94	**21.83**	**14.86**	**14.64**
Total	69.3	58.7	57.1	83.0	97.3	96.9	104.1	155.0	116.4	115.2

All plates are 3/8-in. thick

All bolts are 3/4-in. dia.

Bold numbers indicate that shear fracture occurs in that bolt

______ Indicates the position of the beam neutral axis

Table 5.15 Horizontal Force Acting on Bolts in Gr. 50-Plate Connections

Bolt	Horizontal Shear Force, (kips)									
	case 18 3-bolt pure F_u=88	case 20 3-bolt a = 3" F_u=88	case 23 3-bolt a = 5" F_u=88	case 11 4-bolt a =3.5" F_u=96	case 21 5-bolt a = 3" F_u=88	case 24 5-bolt a = 4" F_u=88	case 12 6-bolt a = 3.5" F_u=96	case 19 7-bolt pure F_u=88	case 22 7-bolt a = 3" F_u=88	case 25 7-bolt a = 5" F_u=88
1								**-6.30**	**-16.98**	**-16.92**
2	**-1.98**	**-15.00**	**-15.04**	**-15.67**	**-13.40**	**-13.33**	**-17.71**	-3.75	-14.37	-14.10
3	**-0.44**	**-12.47**	-12.09	-11.50	-7.59	-7.48	-12.43	-1.49	-6.60	-6.40
4	0.44	-1.31	-1.68	-0.86	-0.09	-0.06	-3.99	0.76	-0.22	-0.17
5				8.68	7.04	6.93	2.35	2.93	6.08	5.92
6					**13.37**	**13.24**	10.45	5.11	13.90	13.46
7							17.33	**6.85**	**17.11**	**17.06**

Bold numbers indicate that shear fracture occurs in that bolt
Indicates the position of the beam neutral axis
\- Minus sign indicates that the force points towards the support

Table 5.16 Resultant in Bolts in Gr. 50-Plate Connections

Bolt	Resultant, (kips)									
	case 18 3-bolt pure F_u=88	case 20 3-bolt a = 3" F_u=88	case 23 3-bolt a = 5" F_u=88	case 11 4-bolt a =3.5" F_u=96	case 21 5-bolt a = 3" F_u=88	case 24 5-bolt a = 4" F_u=88	case 12 6-bolt a = 3.5" F_u=96	case 19 7-bolt pure F_u=88	case 22 7-bolt a = 3" F_u=88	case 25 7-bolt a = 5" F_u=88
Nominal	23.33	23.33	23.33	25.45	23.33	23.33	25.45	23.33	23.33	23.33
1								**22.99**	**22.81**	**22.70**
2	**23.13**	**23.82**	**23.53**	**25.35**	**23.83**	**23.77**	**24.08**	22.53	22.64	22.46
3	**23.11**	**23.62**	23.17	24.13	20.96	20.84	22.52	22.35	18.97	18.76
4	23.11	20.14	19.33	20.88	19.26	19.15	18.17	22.36	16.36	16.18
5				22.80	20.60	20.42	17.54	22.39	18.27	17.96
6					**23.58**	**23.48**	20.79	22.58	22.29	21.80
7							23.55	**22.88**	**22.66**	**22.48**

Bold numbers indicate that shear fracture occurs in that bolt
Indicates the position of the beam neutral axis

In the case of Gr. 50 plates, the vertical force is reduced to approximately 19.3 kips in the 3- and 5-bolt connections. In 7-bolt connections, the outermost bolts, both top and bottom, failed with an average vertical force of only 15 kips, while the rest of the bolts carried approximately 17.3 kips. The investigation of the horizontal forces reveals that the magnitude of the force is much greater than that in the A36-plate connections. Therefore, the shear strength of the Gr. 50-connections is greatly reduced. However, the resultant in the failed bolts is equal to the shear rupture strength of the bolt (23.3 kips for F_u = 88 ksi, and 25.4 kips for F_u = 96 ksi). It should also be noted that the innermost bolts in some connections, most obvious in 7-bolt connections, did not reach their maximum shear strength when the bolt failure occurred in the outer bolts. This indicates that the extrapolation of the strength of larger bolt configurations based on current results might not give accurate results.

The results underline the effect of the base material on the bolt shear strength. Moreover, the less shear force carried by the top and bottommost bolts in the 7-bolt connection, which are 9 in. away from the center of gravity of the bolt group (also the beam neutral axis), states the importance of the bolt configuration and the position of the bolt group with respect to the beam neutral axis.

Bolt movement. Two examples of the bolt movement from the simulations, Models 20 (Figure 5.16) and 22 (Figure 5.17), are included for discussion. The plots of bolt movement illustrate that the bolts in a bolt group move together as if they were connected by a solid structure, contradicting the concept of instantaneous center, which assumes that each bolt rotates about one imaginary point defined as an instantaneous center. The bolt movement is essentially caused by beam rotation, therefore; every bolt in the group, which is attached to the beam, should move together. The displacement of the bolt, as shown in the two figures, is also a function of its distance from the center of rotation.

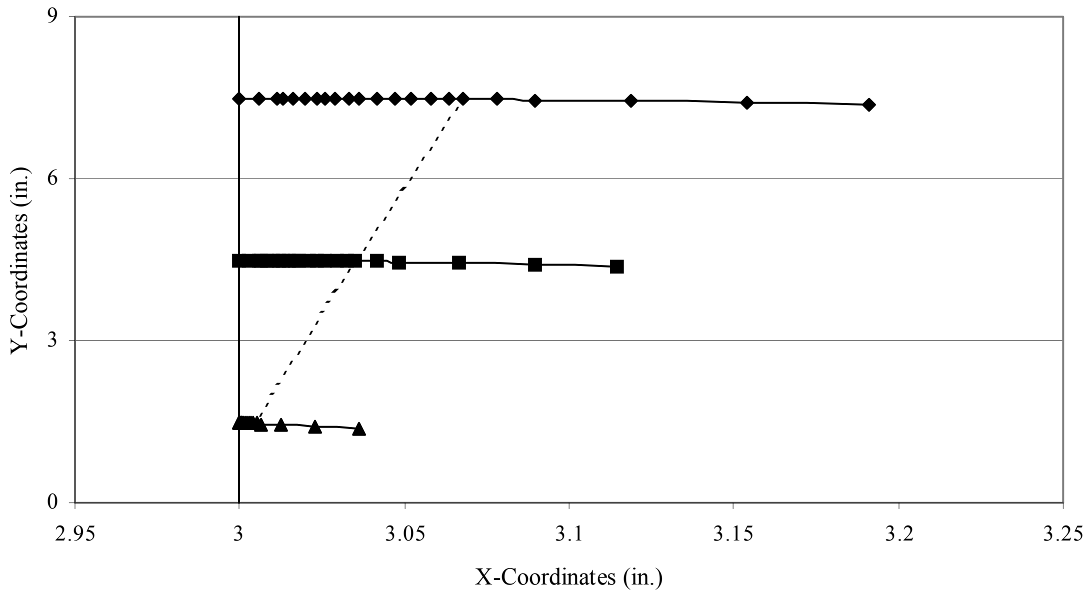


Figure 5.16 Movement of Bolt Group in Model 20 (3-A325N)

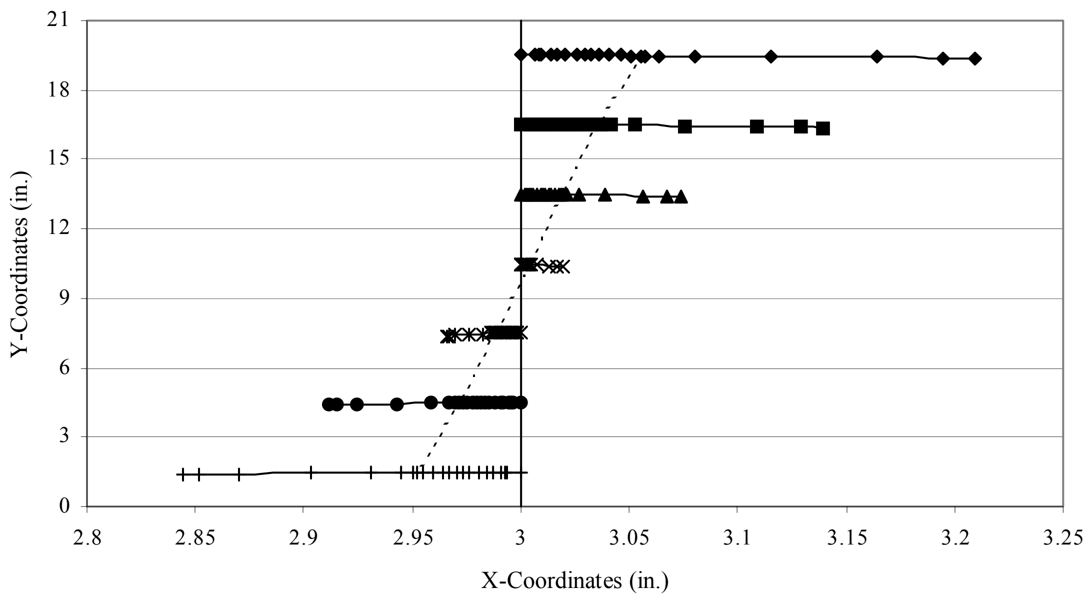


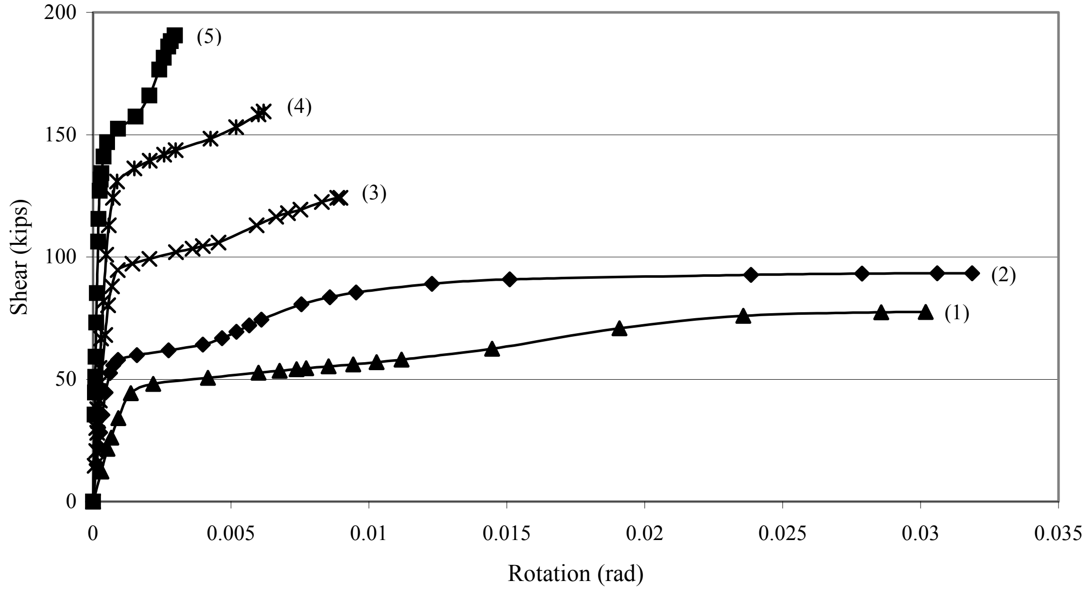
Figure 5.17 Movement of Bolt Group in Model 22 (7-A325N)

The behavior of the plate. The plate behavior in Models 13 through 25 is summarized in Figures 5.18 and 5.19 with corresponding values shown in Tables 5.17 and 5.18, respectively.

The behavior of the plate is as discussed in Section 5.1.2. The plate began to yield when the value of the shear force reached the beam shear strength calculated using Equation 5.1. The plate reached the strain hardening stage after the value of the shear force reached the value calculated using Equation 5.2, which was developed from the shear stress distribution in the plate cross section. The strain hardening stage is visible in the A36 plate plot, but is less visible in the Gr. 50 plate plot.

Plate bending strength calculated by using Equation 5.5 has the smallest strength among the limit states considered in small plates with large a-distance, as seen in both the 9 in. deep A36 and Gr. 50 plates with a-distance equal to 5 in. (Models 15 and 23). Nevertheless, in the simulations, both plates were able to reach the yielding strength (calculated by Equation 5.2) and the bolt groups did not fail.

Equilibrium. The moment at the weld line, the calculation to verify equilibrium, and the moment at the bolt line are demonstrated in Tables 5.19 and 5.20. The point of inflection, where the moment is zero, is the eccentricity of the entire connection (M/V) taken from the simulations when bolt failure occurred. The eccentricity on the bolt group (column 3) is calculated by subtracting the a-distance from the point of inflection. The value of the moment at the weld line (column 4) is the value when bolt failure occurred. The moment at the weld line is also calculated by multiplying the shear force by the eccentricity of the connection (column 5). The moment at the welds produced by the shear force is calculated by multiplying the shear force in the connection by the a-distance (column 7). The moment at the bolt line (column 8) is then calculated by subtracting the moment at the weld line with the moment from column 7. Another way of computing the moment at the bolt line (column 9) is simply multiplying the shear force by the eccentricity of the bolt group in column 3.

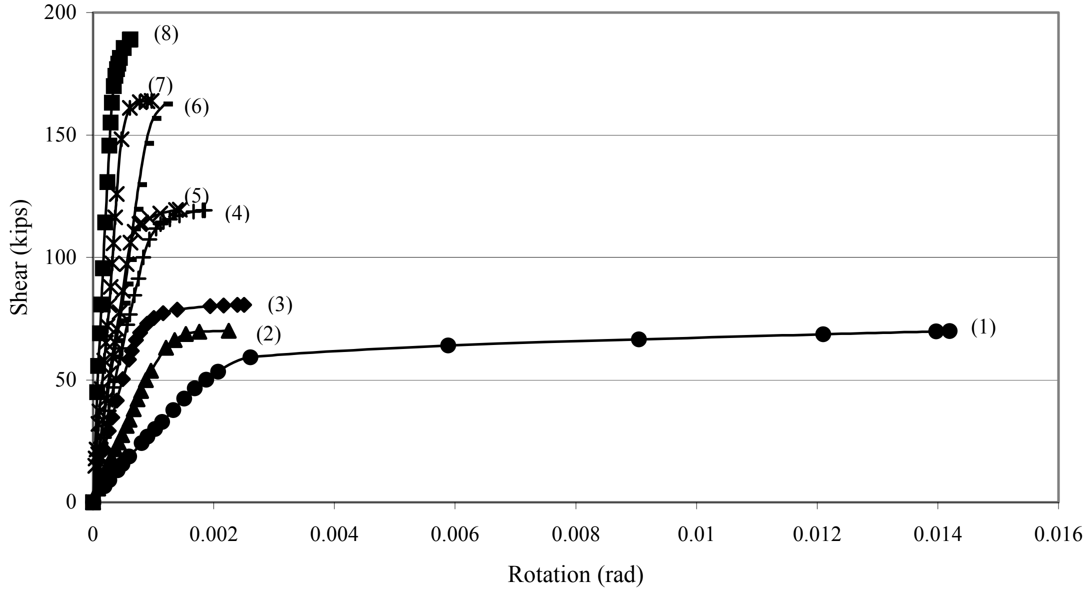


(1) 3/8x6-1/2x9 in. plate (a = 5); 3-bolt connection (Model 15)
(2) 3/8x4x9 in. plate (a = 2.5); 3-bolt connection (Model 13)
(3) 3/8x5-1/2x15 in. plate (a = 4); 5-bolt connection (Model 16)
(4) 3/8x6-1/2x21-in. plate (a = 5); 7-bolt connection (Model 17)
(5) 3/8x4x21 in. plate (a = 2.5); 7-bolt connection (Model 14)

Figure 5.18 Shear vs. Rotation at Bolt Line of Models 13 through 17 (A36 plate)

Table 5.17 Shear Strength of Plates in Figure 5.18

Model	Simulation			Prediction				
	PL Dimension (t, width, depth) (in.)	a distance (in.)	PL Strength (yield/ult.) (ksi)	Beam Shear Yielding (kips)	Effective Yielding (kips)	Shear Yielding (kips)	Shear Rupture (kips)	PL Bend Plastic (kips)
13	3/8x4x9	2.50	36/58	48.6	60.8	72.9	78.3	109.4
14	3/8x4x21	2.50	36/58	113.4	158.0	170.1	182.7	595.4
15	3/8x6-1/2x9	5.00	36/58	48.6	60.8	72.9	78.3	54.7
16	3/8x5-1/2x15	4.00	36/58	81.0	109.4	121.5	130.5	189.8
17	3/8x6-1/2x21	5.00	36/58	113.4	158.0	170.1	182.7	297.7



(1) 3/8x6-1/2x9 in. plate (a = 5); 3-bolt connection (Model 23)
(2) 3/8x4-1/2x9 in. plate (a = 3); 3-bolt connection (Model 20)
(3) 3/8x4x9 in. plate (a = 2.5); 3-bolt connection (Model 18)
(4) 3/8x5-1/2x15 in. plate (a = 4); 5-bolt connection (Model 24)
(5) 3/8x6-1/2x15 in. plate (a = 3); 5-bolt connection (Model 21)
(6) 3/8x6-1/2x21-in. plate (a = 5); 7-bolt connection (Model 25)
(7) 3/8x5-1/2x21-in. plate (a = 3); 7-bolt connection (Model 22)
(8) 3/8x4x21 in. plate (a = 2.5); 7-bolt connection (Model 19)

Figure 5.19 Shear vs. Rotation at Bolt Line of Models 18 through 25 (Gr. 50 plate)

Table 5.18 Shear Strength of Plates in Figure 5.19

Model	Simulation			Prediction				
	PL Dimension (t, width, depth) (in.)	a distance (in.)	PL Strength (yield/ult.) (ksi)	Beam Shear Yielding (kips)	Effective Yielding (kips)	Shear Yielding (kips)	Shear Rupture (kips)	PL Bend Plastic (kips)
18	3/8x4x9	2.50	50/65	67.5	84.4	101.3	87.8	151.9
19	3/8x4x21	2.50	50/65	157.5	219.4	236.3	204.8	826.9
20	3/8x4-1/2x9	3.00	50/65	67.5	84.4	101.3	87.8	126.6
21	3/8x4-1/2x15	3.00	50/65	112.5	151.9	168.8	146.3	351.6
22	3/8x4-1/2x21	3.00	50/65	157.5	219.4	236.3	204.8	689.1
23	3/8x6-1/2x9	5.00	50/65	67.5	84.4	101.3	87.8	75.9
24	3/8x5-1/2x15	4.00	50/65	112.5	151.9	168.8	146.3	263.7
25	3/8x6-1/2x21	5.00	50/65	157.5	219.4	236.3	204.8	413.4

Table 5.19 Calculations of Moment in Connections for A36-Plate Connections

Simulation	No of Bolts	Pt. of Infl. (e) (in.) (1)	a distance (in.) (2)	e_b (in.) (3)	Moment @ Weld (k-in.) (4)	Shear (kips) (5)	Ve (k-in.) (6)	Va (k-in.) (7)	Moment @ Bolt(i) (k-in.) (8)	Moment @ Bolt(ii) (k-in.) (9)
case 1	3	2.80	2.75	0.05	240	85.5	239	235	5	4
case 15	3	5.13	5.00	0.13	343	66.9	343	335	9	9
case 9	5	2.88	2.75	0.13	404	140.3	404	386	18	18
case 16	5	4.07	4.00	0.07	479	109.4	445	438	41	8
case 10	7	2.95	2.75	0.2	580	197.0	581	542	38	39
case 17	7	5.71	5.00	0.71	928	150.7	860	754	175	107

Table 5.20 Calculations of Moment in Connections for Gr. 50-Plate Connections

Simulation	No of Bolts	Pt. of Infl. (e) (in.) (1)	a distance (in.) (2)	e_b (in.) (3)	Moment @ Weld (k-in.) (4)	Shear (kips) (5)	Ve (k-in.) (6)	Va (k-in.) (7)	Moment @ Bolt(i) (k-in.) (8)	Moment @ Bolt(ii) (k-in.) (9)
case 20	3	5.91	3.0	2.91	345	58.7	347	176	169	171
case 23	3	7.97	5.0	2.97	455	57.1	455	286	170	170
case 7	4	5.59	3.5	2.09	610	109.1	610	382	228	228
case 11	4	6.41	3.5	2.91	532	83.0	532	291	242	242
case 21	5	5.16	3.0	2.16	502	97.3	502	292	210	210
case 24	5	6.15	4.0	2.15	596	96.7	595	387	209	208
case 8	6	8.03	3.5	4.53	1035	128.8	1034	451	584*	583
case 12	6	7.46	3.5	3.96	777	104.1	777	364	413*	412
case 22	7	7.53	3.0	4.53	876	116.4	876	349	527	527
case 25	7	9.53	5.0	4.53	1096	115.2	1098	576	520	522

(i) M_b = M @ Weld – Va
(ii) $M_b = Ve_b$
* Bolt strength used in Simulation no. 8 was 120 ksi whereas in Simulation no. 12 it was 96 ksi.

The exclusion of the effect of the a-distance is explained by the equilibrium check in the connections. The values of the moment at the bolt line are equal for the same connection configurations regardless of the a-distance. The values of the moment at the bolt line calculated in both approaches are also equal. It is also shown clearly by the calculations that the moment at the weld line is the summation of two components: the moment created by the shear force Va, and the moment at the bolt line (column 4 = column 7 + column 8).

In the case of A36-plate connections, the moment at the bolt line is negligible in the first five simulations listed in Table 5.19. The moments at the bolt line calculated using the two approaches are not equal in cases 16 and 17 because the eccentricity and the moment at the weld line used in the calculations were extrapolated. This is because the beams in both connections yielded as the bolt groups reached their shear strength. As previously described in the setup of Model 11 (section 5.1.2), the beam length used in the simulation was designed such that the beam would reach yielding as well as the bolt group reaching the shear strength (without eccentricity). When the beam yielded, the rotation increased rapidly with the small amount of load, resulting in the eccentricity immediately moving toward the beam. In general, the eccentricity moves toward the beam in the beginning of the simulation and starts moving toward the support when bolts begin to plow. Because the eccentricity in the two simulations moved toward the beam due to premature yielding of the beam, it was felt that the extrapolation of the eccentricity from the trend prior to yielding might reflect the behavior of the connection more effectively. The equilibrium check reveals that the extrapolation could not obtain the correct solution for the problem.

In the case of Gr. 50-plate connections, the moment at the bolt line is much greater. The evidence is strongly supported by the magnitude of the horizonal force, shown in Table 5.15, which can be used to calculate this moment. The moment is calculated by multiplying the horizontal force with its moment arm, which is the distance to the center of rotation. The summation of the moment produced by each horizontal force is the moment at the bolt line. The moment at the bolt line calculated using this approach for Models 20 through 22 is shown in Table 5.21.

Table 5.21 Moment at Bolt Line Calculated Using Horizontal Force for Models 20 through 22

Model	Horizontal Force (kips)	Moment Arm (in.)	Moment on Bolts by Horizontal Force (k-in.)	Moment @ Bolt Line from Table 5.20 (k-in.)
Model 20	15.00	6	90	
	12.47	3	37	
	1.31	0	0	
			127	171
Model 21	13.40	6	80	
	7.59	3	23	
	0.09	0	0	
	7.04	3	21	
	13.37	6	80	
			205	210
Model 22	16.98	9	153	
	14.37	6	86	
	6.60	3	20	
	0.22	0	0	
	6.08	3	18	
	13.90	6	83	
	17.11	9	154	
			514	527

The moment calculated by using horizontal force is close to the value obtained using equilibrium shown in the last column of Table 5.21, except for Model 20. A possible reason that causes the moment of the horizontal force in Model 20 to be small is that the magnitude of the force is reduced due to excessive deflection of the plate. As a result, the horizontal force acting on bolts can be used to calculate the moment at the bolt line, which must be considered along with the moment produced by the shear force (Va) in the weld design of connections that have large horizontal forces.

5.3 Examination of Effect of Plate Thickness

5.3.1 Introduction

Plate thicknesses in single plate shear connections should be limited relative to the bolt diameter, as suggested by Richard (1980) and later by Astaneh (1989).

To investigate the effect of plate thickness, Models 26 through 29 were set up with A36 and Gr. 50 plates, a-distance equal to 5 in., and the plate thicknesses equal to 1/2 and 3/4 in. In addition, Models 30 and 31 were created with 43 ksi yield stress plate material and plate thicknesses of 3/8 and 1/2 in. All bolts were A325 3/4-in. diameter bolts with a strength of 88 ksi. The details of Models 26 through 31 are shown in Table 5.22.

The results of the simulations, including the bolt shear strength, the behavior of the plate, and the moment in the connections, are summarized in Figure 5.20 and Tables 5.23 through 5.29. Results of the simulations are in Appendix C.

Table 5.22 Details of Finite Element Models 26 through 31

Simulation	Beam		Bolts		Bolt Str.	a-distance	PL Yield	PL Dimensions
	Size	Span (ft)	No.	Type	(ksi)	(in.)	(ksi)	(in.)
26	Gr.50 W18x55	28	3	A325	88	5.00	36.0	1/2x6-1/2x9
27	Gr.50 W18x55	28	3	A325	88	5.00	50.0	1/2 x6-1/2x9
28	Gr.50 W18x55	28	3	A325	88	5.00	36.0	3/4 x6-1/2x9
29	Gr.50 W18x55	28	3	A325	88	5.00	50.0	3/4 x6-1/2x9
30	Gr.50 W18x55	28	3	A325	88	3.00	43.0	3/8 x4-1/2x9
31	Gr.50 W18x55	28	3	A325	88	5.00	43.0	1/2 x6-1/2x9

5.3.2 Results and Discussions

The bolt shear strength of Models 15, 23, and 26 through 31 is summarized in Tables 5.23. The bolt shear strength of Models 15 and 23 is included for comparison.

Table 5.23 Schemes of Simulations for Investigating Effect of Plate Thickness on Shear Strength of Bolt Group

(a) Simulation

Plate Material	Simulation Number		
	Plate Thickness (in.)		
	3/8	1/2	3/4
A36	15	26	28
Fy = 43	30	31	
Gr.50	23	27	29

(b) Bolt Shear Strength of Corresponding Simulations

Plate Material	Bolt Shear Strength (kips)		
	Plate Thickness (in.)		
	3/8	1/2	3/4
A36	66.9	59.2	56.7
Fy = 43	58.1	58.4	
Gr.50	57.1	57.1	56.5

Note: Nominal strength (F_v = 52.8 ksi) = 69.9 kips

Effect of plate thickness. In the case of A36 plates, the bolt shear strength is reduced once the plate thickness is increased from 3/8 in. to 1/2 in. or 3/4 in. In the case of 43 ksi yield stress plates, the bolt shear strength is reduced in both plate thicknesses. In the case of Gr. 50 plates, the shear strength of the connection, which is already reduced in the 3/8-in. plate, does not change when the plate thickness is increased from 3/8 in. to 1/2 in., and only slightly further reduced when the plate thickness is increased to 3/4 in. The bolt shear strength in 43 ksi yield stress plate is close to that in Gr. 50 plate.

Forces acting on bolts. The vertical and horizontal forces, and resultant forces acting on the bolts, are shown in Tables 5.24 through 5.26, respectively. The magnitude

Table 5.24 Vertical Force Acting on Bolts in Models 26 through 31

Bolt	Vertical Shear Force, (kips)							
	case 15 A36 t = 3/8	case 26 A36 t = 1/2	case 28 A36 t = 3/4	case 23 Gr. 50 t = 3/8	case 27 Gr. 50 t = 1/2	case 29 Gr. 50 t = 3/4	case 30 Fy = 43 t = 3/8	case 31 Fy = 43 t = 1/2
1	**22.53**	**20.07**	**18.01**	**18.10**	**18.46**	**17.31**	**18.56**	**18.95**
2	22.38	19.50	19.93	19.76	19.62	**19.52**	19.84	20.01
3	22.01	19.58	19.19	19.26	19.04	19.66	19.67	19.47
Total	66.9	59.2	56.7	57.1	57.1	56.5	58.1	58.4

All bolts are 3/4-in. A325N (Fu = 88 ksi)

Bold numbers indicate the shear fracture occurs in that bolt

Every connection is constructed such that bolt no. 3 coincides with the beam neutral axis

Table 5.25 Horizontal Force Acting on Bolts in Models 26 through 31

Bolt	Horizontal Shear Force, (kips)							
	case 15 A36 t = 3/8	case 26 A36 t = 1/2	case 28 A36 t = 3/4	case 23 Gr. 50 t = 3/8	case 27 Gr. 50 t = 1/2	case 29 Gr. 50 t = 3/4	case 30 Fy = 43 t = 3/8	case 31 Fy = 43 t = 1/2
1	**-0.23**	**-12.63**	**-15.26**	**-15.04**	**-14.93**	**-16.20**	**-14.81**	**-14.69**
2	1.10	-6.69	-7.24	-12.09	-6.93	**-7.91**	-12.04	-7.07
3	5.06	-1.00	-0.19	-1.68	-0.15	0.32	-1.23	-0.40

- Minus sign indicates that the force points toward the support

Table 5.26 Resultant on Bolts in Models 26 through 31

Bolt	Resultant, (kips)							
	case 15 A36 t = 3/8	case 26 A36 t = 1/2	case 28 A36 t = 3/4	case 23 Gr. 50 t = 3/8	case 27 Gr. 50 t = 1/2	case 29 Gr. 50 t = 3/4	case 30 Fy = 43 t = 3/8	case 31 Fy = 43 t = 1/2
Nomina l	23.33	23.33	23.33	23.33	23.33	23.33	23.33	23.33
1	**22.53**	**23.71**	**23.61**	**23.53**	**23.74**	**23.71**	**23.74**	**23.98**
2	22.41	20.61	21.20	23.17	20.81	21.06	23.21	21.22
3	22.58	19.61	19.19	19.33	19.04	19.66	19.71	19.47

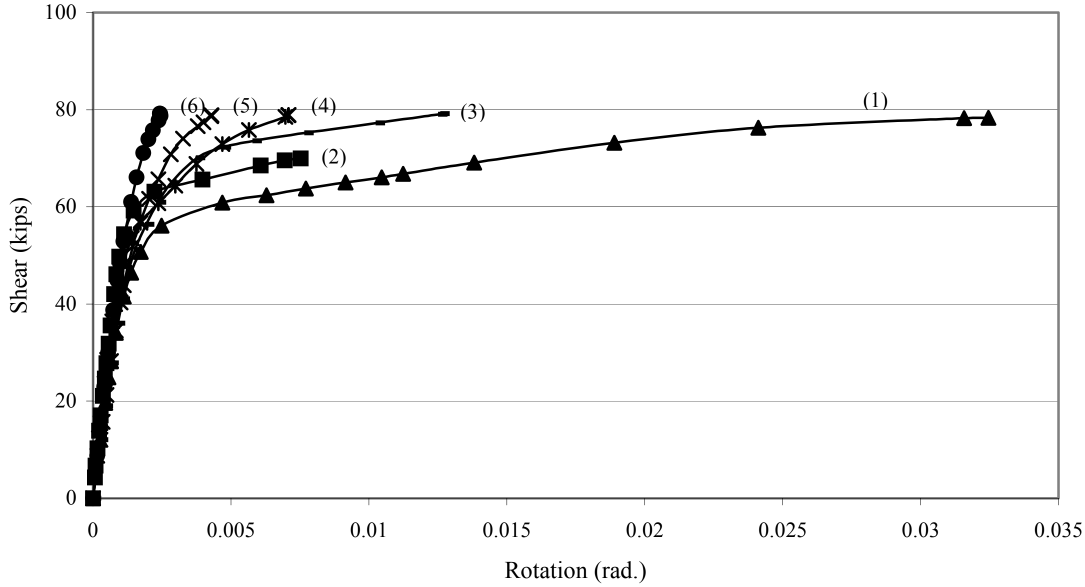
of the horizontal force in A36 plates with thickness greater than 3/8 in. reflects the reduced bolt shear strength. The horizontal force in plates with yield strength of 43 ksi has almost the same magnitude as that in the Gr. 50 plate. Most of the bolts in the simulations failed when the resultant reached the bolt shear strength.

The behavior of the plate. The behavior of the plates in Models 26 through 31 is summarized in Figure 5.20 and Table 5.27. The strain hardening stage of the plate is hardly visible in the plot because the shear strength of the plate is significantly strengthened by the increase of the plate thickness to 1/2 in. and 3/4 in.

Statics and equilibrium. Table 5.28 demonstrates the moment at the weld line, the moment at the bolt line, and the related calculations previously described (Section 5.2.2, equilibrium). The value of the moment at the bolt line increases in A36 plate connections once the thickness is greater than 3/8 in., and is almost equal to that in any Gr. 50 plate connection once the plate thickness is 3/4 in. In general, except for the 3/8-in A36 plate, the value of the moment at the weld line ranges from 120 k-in. to 170 k-in.

Bolt plowing. To help investigate the bolt plowing in the plates, the ratio of the contact area, which is the area between the failed bolt and the bolt hole, and the theoretical bearing area, $d_b t_p$, for each connection is shown in Table 5.29. The ratios in the 3/8-in., 1/2-in. A36 plates, and 3/8-in. Gr. 50 plate are approximately 70 percent. The ratio becomes smaller in 1/2-in. Gr. 50 and reduces to merely 60 and 50 percent in 3/4-in. A36 and Gr. 50 plates, respectively.

Considering the beam web (Gr. 50 W18x55, t_w = 0.39 in.) in association with the plate thickness, it is concluded that the bolts plowed through the 3/8-in. and 1/2-in. A36 plates, and 3/8-in. Gr. 50 plate. In these three cases, the ratio of the contact area to the bearing area was almost equal. The 3/8-in. A36 plates were ductile enough for the bolts to plow through with small horizontal force, whereas the 1/2-in. A36 and 3/8-in. Gr. 50 plates were less ductile and produced much greater horizontal forces acting on the bolts. The smaller ratio of contact area to bearing area in 1/2-in. Gr. 50 plate suggested



(1) The 1/2x6-1/2x9 in. A36 plate (a = 5) (Model 26)
(2) The 3/8x4-1/2x9 in. F_y = 43 ksi plate (a = 3) (Model 30)
(3) The 1/2x6 –1/2x9 in. F_y = 43 ksi plate (a = 5) (Model 31)
(4) The 3/4x6-1/2x9 in. A36 plate (a =5) (Model 28)
(5) The 1/2x6-1/2x9 in. Gr. 50 plate (a =5) (Model 27)
(6) The 3/4x6-1/2x9 in. Gr. 50 plate (a =5) (Model 29)

Figure 5.20 Shear vs. Rotation at Bolt Line of Models 26 through 31

Table 5.27 Shear Strength of Plates in Figure 5.20

Model	Simulation			Prediction				
	PL Dimension (t, width, depth) (in.)	a distance (in.)	PL Strength (yield, ult.) (ksi)	Beam Shear Yielding (kips)	Effective Yielding (kips)	Shear Yielding (kips)	Shear Rupture (kips)	PL Bend Plastic (kips)
26	1/2x6-1/2x9	5.00	36, 58	64.8	81.0	97.2	104.4	72.9
27	1/2x6-1/2x9	5.00	50, 65	90.0	112.5	135.0	117.0	101.3
28	3/4x6-1/2x9	5.00	36, 58	97.2	121.5	145.8	156.6	109.4
29	3/4x6-1/2x9	5.00	50, 65	135.0	168.8	202.5	175.5	151.9
30	3/8x4-1/2x9	3.00	43, 65	58.1	72.6	87.1	87.8	108.8
31	1/2x6-1/2x9	5.00	43, 65	77.4	96.8	116.1	117.0	87.1

Table 5.28 Calculations of Moment in Connections for Verifying Effect of Plate Thickness

Simulation	Material, Thickness (ksi), (in.)	Pt. of Infl. (e) (in.) (1)	a distance (in.) (2)	e_b (in.) (3)	Moment @ Weld (k-in.) (4)	Shear (kips) (5)	Ve (k-in.) (6)	Va (k-in.) (7)	Moment @ Bolt(i) (k-in.) (8)	Moment @ Bolt(ii) (k-in.) (9)
case 15	A36, 3/8	5.13	5	0.13	343	66.9	343	335	9	9
case 26	A36, 1/2	7.06	5	2.06	418	59.2	418	296	122	122
case 28	A36, 3/4	7.62	5	2.62	432	56.7	432	284	149	149
case 23	Gr50, 3/8	7.97	5	2.97	455	57.1	455	286	170	170
case 27	Gr50, 1/2	7.53	5	2.53	430	57.1	430	286	145	144
case 29	Gr50, 3/4	7.78	5	2.78	439	56.6	440	283	156	157
case 30	Fy43, 3/8	5.89	3	2.89	342	58.1	342	174	168	168
case 31	Fy43, 1/2	7.45	5	2.45	435	58.4	435	292	143	143

(i) M_b = M @ Weld – Va
(ii) $M_b = Ve_b$

Table 5.29 Contact Area of Failed Bolt in Each Plate

Plate		Gr. 50	Area in		
Material	Thickness (in.)	W18x55 web (in.)	contact ($in.^2$)	A/d_bt_p	Observation
A36	0.375	0.39	0.1953	0.69	Bolts plowed through plate w/o horizontal forces
A36	0.500	0.39	0.2709	0.72	Bolts plowed through plate w/ horizontal forces
A36	0.750	0.39	0.3332	0.59	Bolts might have plowed through beam web
Gr. 50	0.375	0.39	0.2073	0.74	Bolts plowed through plate w/ horizontal forces
Gr. 50	0.500	0.39	0.2518	0.67	Bolts might have plowed through plate/beam web
Gr. 50	0.750	0.39	0.2834	0.50	Bolts might have plowed through beam web

that the ductility was achieved through the plowing in the beam web as well as in the plate. The ductility of the plate was significantly reduced when the thickness of the plate became greater than 1/2 in. in A36 and 3/8 in. in Gr. 50 plates. The reductions of the ratio of the contact area and the bearing area in the plates indicate that bolt plowing occurred in the beam web, which was relatively softer. As a result, whether or not 3/4-in. A36, 1/2-in and 3/4-in. Gr. 50 plates are able to provide sufficient ductility to the entire system is inconclusive.

5.4 Double-Column Bolt Connections

5.4.1 Introduction

As previously discussed in Section 5.3.1, if the thickness of the plate cannot be increased because of plowing considerations, the capacity of double-column bolt connections will be limited by plate limit states. To have a meaningful study of the double-column configuration, the plate thickness was increased to 3/4 in. to reflect the strength of the additional bolt column.

Models 32 through 34 were set up as pure shear cases. Models 35 through 37 were set up with a-distance of 3 in., and Models 38 through 40 were created with varied a-distances. Models 41 and 42 were later constructed to examine the effect of plate thickness in double-column connections due to the relatively small amount of shear carried by the connections with 3/4 in. plates. The force redistribution did not occur among the bolt columns in 3/4 in. plates, causing the uppermost bolt in the second column to fail prematurely. Therefore, the two models were created with the plate thickness reduced to 1/2 in.

Details of Models 32 through 40 are summarized in Table 5.30. All the plates were Gr. 50 and bolts were A325 3/4-in. diameter bolts with strength of 88 ksi. The a-distance in double-column bolt connections is referred to as the distance from the weld line to the first column of the bolts. The spacing between the two columns is 3 in. The plate configuration for all models is shown in Figure 5.21.

The results of the simulations, including the bolt capacity, the behavior of the plate, and moment in the connections, are summarized in Figure 5.25, and Tables 5.31, and 5.34 through 5.38. Other plots and tables of the simulations used in the investigation are included in Appendix C.

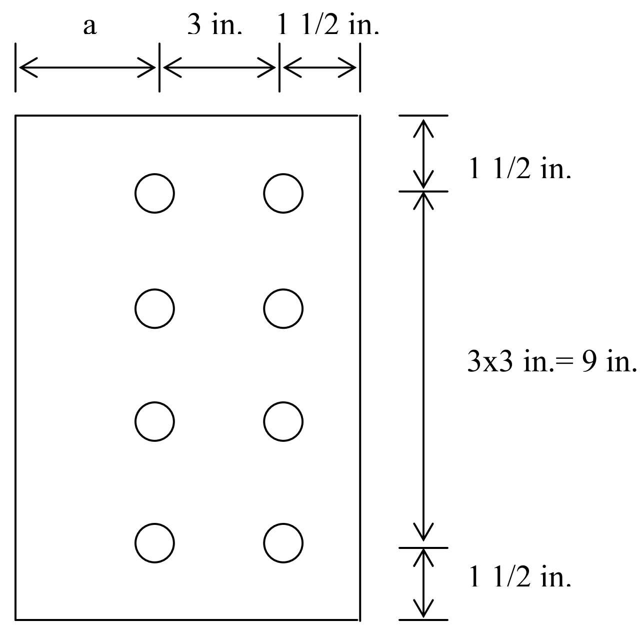
Table 5.30 Details of Finite Element Models 32 through 40

Simulation	Beam		Bolts		Bolt Str.	a-distance	PL Yield	PL Dimensions	Investigation
	Size	Span (ft)	No.	Type	(ksi)	(in.)	(ksi)	(in.)	
32	Gr.50 W12x35	9.5	4	A325	88	2.50	50.0	3/4x7x6	Pure shear
33	Gr.50 W18x55	14	6	A325	88	2.50	50.0	3/4x7x9	
34	Gr.50 W18x76	15.5	8	A325	88	2.50	50.0	3/4x7x12	
35	Gr.50 W12x35	9.5	4	A325	88	3.00	50.0	3/4x7-1/2x6	a-distance and bolt configuration
36	Gr.50 W18x55	14	6	A325	88	3.00	50.0	3/4x7-1/2x9	
37	Gr.50 W18x76	15.5	8	A325	88	3.00	50.0	3/4x7-1/2x12	
38	Gr.50 W12x35	9.5	4	A325	88	4.00	50.0	3/4x9-1/2x6	
39	Gr.50 W18x55	14	6	A325	88	5.00	50.0	3/4x8-1/2x9	
40	Gr.50 W18x76	15.5	8	A325	88	4.00	50.0	3/4x9-1/2x12	
41	Gr.50 W18x55	14	6	A325	88	3.00	50.0	1/2x7-1/2x9	Plate thickness
42	Gr.50 W18x76	15.5	8	A325	88	3.00	50.0	1/2x7-1/2x12	

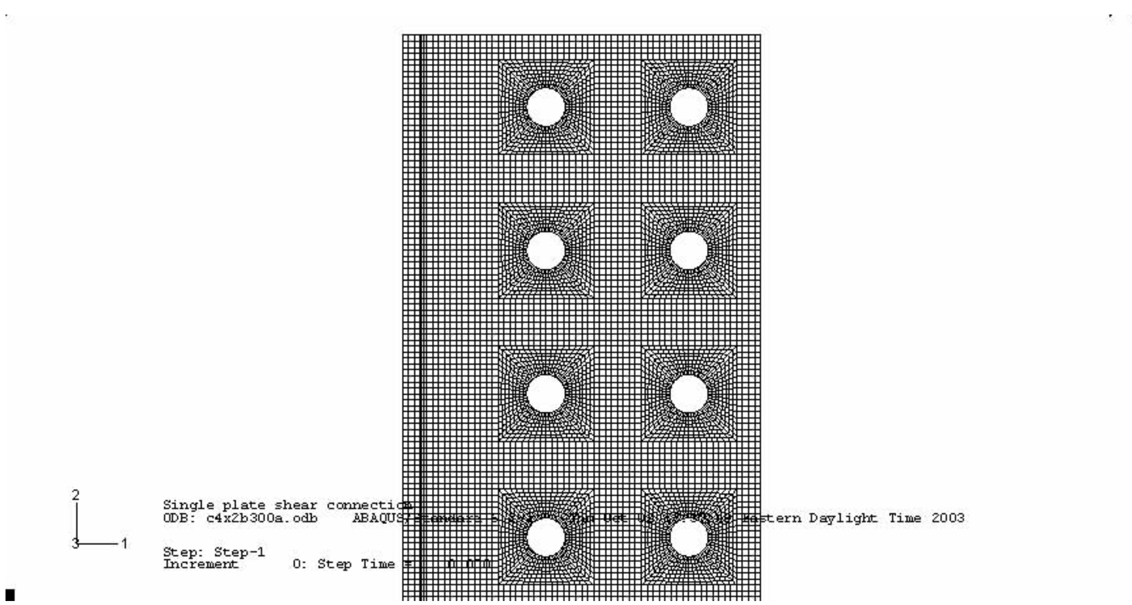
All bolts are 3/4-in. diameter bolts

All plates are Gr. 50 plate

All welds are 1/2-in.



(a) Plate Dimensions



(b) Finite Element Model

Figure 5.21 Plate for Double-Column Bolt Configuration

5.4.2 Results and Discussions

Results of Models 32 through 42 are summarized in Table 5.31. Discussion of the results follows.

Table 5.31 Schemes of Simulations for Investigating Double-Column Bolt Connections

(a) Simulation

No. of Bolts	Simulation Number		
	a-distance		
	3.00	4.00	5.00
2x2	35		38
3x2	36	39	
4x2	37		40

(b) Bolt Shear Strength of Corresponding Models

No. of Bolts	Predicted Strength (kips)		
	a-distance		
	3.00	4.00	5.00
2x2	86.4		47.4(Pl)
3x2	86.7	89.5	
4x2	104.4		108.5

Note: Pl = Plate failed.

The effect of a-distance. As shown in Table 5.31(b), the a-distance does not affect the bolt shear strength in double-column bolt connections. In the 2x2-bolt connection with a-distance of 5 in., plate failure caused the bolt strength reduction and the bolts did not rupture. In the 3x2- and 4x2-bolt connections with 3/4-in. plates, the bolt shear strength of the connections did not vary with the a-distance.

The bolt shear strength. The results show that the bolt shear strength is affected by the plate thickness, as the bolt shear strength of the connections with 1/2-in. plate is greater than that of 3/4-in. plate. To examine the bolt shear rupture strength, shear stress variation in bolts of Models 37 and 42 is presented in Tables 5.32 and 5.33, respectively.

In both cases, bolt shear stress of the innnermost elements started decreasing before the bolt failed. Eventually the shear stress in the outer elements exceeded the shear stress in the innermost element, which is normal in the simulations, and indicates bolt failure in Gr. 50 plates. In addition, bolt failure is confirmed by the resultant acting on bolts shown in Table 5.31.

The plate thickness clearly influences how much the force is distributed to each bolt column. In the 4x2 bolt connection with a 3/4 in. plate, the topmost bolt in the second column failed while the bolts in the first column carried the load up to only 11 kips on average. The total shear force that the connection carried was only 104 kips. In the 4x2 bolt connection with 1/2 in. plate, the bolts in the connection were able to carry the load up to 123 kips without failing. Nevertheless, at that particular load, the force began to be redistributed to the first bolt column, resulting in the reduction of the force carried by the second bolt column. The force redistribution was also the cause of the shear stress reduction of the bolts in the second column. This behavior was related to the bending in the plate, which is discussed below. The bolts in the connection did not fail until the shear force was 149 kips.

Forces acting on bolts. The horizontal and vertical forces, and the resultant acting on the bolts in the double-column connections, are presented in Tables 5.34 through 5.36. The vertical force shows that the bolts in the first column carry much less load than the bolts in the second column in Models 36, 37, 39, and 40, where the bolt failure occured without redistribution. In the case of Models 35, 41, and 42, which demonstrated the force redistribution, the forces acting on bolts shown in the tables are the forces when the redistribution and bolt failure occurred. The force redistribution, which occurred in these models, allowed the bolts in the first column to carry more vertical shear force than bolts in the second column in Models 35 and 41. However, they are almost equal in Model 42. The horizontal force varies with the distance from the bolt

Table 5.32 Shear Stress in Bolts of Model 37

Increment		16	18	20	22	24	26	28	30	32	34	36	38	40	42
Load		74.48	80.38	87.68	95.14	104.4	113.8	125.5	132.1	138.8	147.1	155.6	165.9	170.6	176.6
1st bolt	beam side	6.294	8.108	10.23	12.21	14.82	17.85	21.56	22.10	23.08	25.31	28.29	32.13	34.20	36.08
	plate side	5.553	7.278	9.211	11.24	13.74	16.61	20.67	22.27	23.94	26.18	28.76	31.90	33.57	35.03
2nd bolt	beam side	3.055	3.849	4.934	6.369	8.952	12.04	17.27	20.68	24.47	29.07	33.47	37.71	38.59	39.82
	plate side	2.994	3.761	4.814	6.202	8.532	11.47	16.31	19.40	22.74	27.04	31.33	35.61	37.22	39.24
3rd bolt	beam side	5.362	6.545	8.069	9.853	12.45	15.50	19.77	21.98	24.27	27.05	30.65	35.87	38.43	40.74
	plate side	5.375	6.506	7.982	9.674	12.02	14.59	18.05	20.17	22.40	25.49	29.43	34.50	37.05	39.70
4th bolt	beam side	4.871	6.006	7.872	10.12	12.95	16.04	20.65	23.60	27.10	31.97	30.00	38.57	39.20	39.79
	plate side	5.033	6.192	7.926	9.878	12.34	15.42	20.06	22.98	26.38	30.73	34.45	37.68	38.43	39.10
1st bolt	beam side	36.20	36.75	37.05	36.91	36.30	36.02	35.85	35.73	35.69	35.68	35.73	36.03	36.39	37.01
	plate side	35.13	36.33	37.61	37.99	38.15	38.25	38.01	37.80	37.61	37.58	37.68	37.99	38.35	39.13
2nd bolt	beam side	35.04	36.88	38.67	39.86	40.87	41.59	41.82	41.71	41.61	41.51	41.12	40.79	40.79	40.92
	plate side	32.91	34.95	36.99	39.03	40.91	42.21	43.17	43.51	43.73	44.01	44.23	44.39	44.45	44.70
3rd bolt	beam side	34.98	36.97	38.85	40.40	41.47	42.28	43.50	44.02	44.37	44.37	44.10	43.60	43.35	43.07
	plate side	34.06	36.31	38.40	40.19	41.72	42.65	43.72	44.28	44.71	45.23	45.73	46.03	46.13	46.28
4th bolt	beam side	35.31	37.04	38.71	39.73	40.69	41.58	41.41	42.42	42.38	42.24	42.08	42.14	42.21	42.03
	plate side	33.07	34.92	36.76	38.68	40.39	41.58	42.88	43.35	43.77	44.29	44.64	44.98	45.17	45.39

Bolt shear rupture strength as determined from the FEM results

Indicates that the stress decreases in the next increment

Indicates that the stress in the outer element exceeds the stress in the innermost element

Table 5.33 Shear Stress in Bolts of Model 42

Increment		28	30	32	34	36	38	40	42	44	46	48	50	52	54
Load		114.6	119.9	122.9*	129.7	137.7	140.7	144.5	146.7	149.3	150.7	153.4	154.9	156.7	157.6
1st bolt	beam side	22.51	25.08	26.63	30.66	35.93	37.36	38.54	38.86	39.22	39.37	39.76	40.05	40.39	40.55
	plate side	23.09	25.90	27.54	31.51	36.59	37.86	39.17	39.78	40.42	40.70	41.12	41.59	42.02	42.21
2nd bolt	beam side	17.35	20.11	21.74	26.45	31.57	33.24	35.37	36.78	38.27	39.01	39.92	40.22	40.59	40.77
	plate side	17.97	20.74	22.21	26.52	31.44	33.16	35.16	36.65	38.46	39.20	40.47	41.12	42.02	42.40
3rd bolt	beam side	19.90	21.83	22.96	25.61	31.07	33.56	36.64	37.79	39.03	39.40	39.96	40.29	40.69	40.86
	plate side	19.81	21.60	22.78	25.55	31.40	33.99	37.08	38.38	39.86	40.60	41.93	42.56	43.05	43.27
4th bolt	beam side	21.28	24.28	25.92	29.48	34.19	35.86	37.34	37.95	38.72	39.11	39.73	40.09	40.46	40.60
	plate side	22.07	24.90	26.40	29.68	34.23	35.99	37.79	38.75	40.07	40.78	41.84	42.24	42.69	42.94
1st bolt	beam side	37.33	37.38	37.45	37.82	37.42	37.23	36.96	36.93	36.88	36.89	36.95	37.06	37.22	37.30
	plate side	40.02	40.26	40.39	40.82	40.38	40.14	40.03	40.07	40.08	40.09	40.16	40.27	40.39	40.46
2nd bolt	beam side	38.82	38.97	39.06	39.27	39.60	39.74	39.92	40.04	40.20	40.29	40.45	40.55	40.67	40.73
	plate side	40.97	41.14	41.21	41.39	41.64	41.77	41.93	42.03	42.18	42.27	42.42	42.50	42.62	42.68
3rd bolt	beam side	39.45	39.67	39.79	39.91	39.97	39.98	39.98	39.98	40.01	40.04	40.10	40.16	40.24	40.28
	plate side	40.57	40.96	41.14	41.34	41.53	41.59	41.61	41.64	41.71	41.78	41.90	41.99	42.09	42.13
4th bolt	beam side	38.91	39.09	39.15	38.78	37.22	36.71	36.36	36.31	36.34	36.39	36.48	36.61	36.76	36.84
	plate side	40.57	40.80	40.88	40.56	38.80	38.18	37.83	37.72	37.72	37.77	37.87	38.03	38.23	38.33

* Indicates that force redistribution occurs

Bolt shear rupture strength as determined from the FEM results

Indicates that the stress decreases in the next increment

Indicates that the stress in the outer element exceeds the stress in the innermost element

Table 5.34 Vertical Force Acting on Bolts in Models 32 through 42

Bolt		Vertical Shear Force, (kips)													
		case 32 2x2-bolt pure t = 3/4	case 35 2x2-bolt a = 3" t = 3/4	case 35 2x2-bolt a = 3" t = 3/4	case 38 2x2-bolt a = 5" t = 3/4	case 33 3x2-bolt pure t = 3/4	case 36 3x2-bolt a = 3" t = 3/4	case 39 3x2-bolt a = 4" t = 3/4	case 34 4x2-bolt pure t = 3/4	case 37 4x2-bolt a = 3" t = 3/4	case 40 4x2-bolt a = 5" t = 3/4	case 41 3x2-bolt a = 3" t = 1/2	case 41 3x2-bolt a = 3" t = 1/2	case 42 4x2-bolt a = 3" t = 1/2	case 42 4x2-bolt a = 3" t = 1/2
1st col.	1					16.59	8.94	9.87	16.35	6.38	7.31	10.32	19.81	12.40	18.49
	2	**22.78**	17.90	**22.53**	13.16	16.17	8.92	10.20	15.44	4.26	6.31	9.96	21.36	10.26	17.45
	3	22.49	18.51	**22.68**	13.23	17.30	9.40	9.98	16.25	6.18	8.07	9.83	22.11	10.35	17.83
	4								17.33	6.16	7.36			12.01	18.15
Total		45.3	36.4	45.2	26.4	50.1	27.3	30.1	65.4	23.0	29.1	30.1	63.3	45.0	71.9
2nd col.	1					**22.19**	**19.90**	**19.90**	**22.71**	**20.35**	**20.13**	17.92	**16.03**	20.55	**20.06**
	2	21.97	20.14	20.89	10.77	22.10	19.97	19.90	22.47	20.52	19.93	17.01	17.98	19.37	20.49
	3	21.52	19.71	20.33	10.27	22.19	19.73	19.90	22.43	20.13	19.59	16.49	15.63	19.10	19.45
	4								22.83	20.40	19.79			18.90	17.37
Total		43.5	39.9	41.2	21.0	66.5	59.6	59.7	90.4	81.4	79.4	51.4	49.6	77.9	77.4
1st + 2nd		88.8	76.3	86.4	47.4	116.5	86.9	89.8	155.8	104.4	108.5	81.5	112.9	122.9	149.3

All plates are Gr. 50 plate

All bolts are 3/4-in. A325N (Fu = 88 ksi)

Bold numbers indicate that shear fracture occurs in that bolt

Indicates the position of the beam neutral axis

Table 5.35 Horizontal Force Acting on Bolts in Models 32 through 42

Bolt		Horizontal Shear Force, (kips)													
		case 32 2x2-bolt pure t = 3/4	case 35 2x2-bolt a = 3" t = 3/4	case 35 2x2-bolt a = 3" t = 3/4	case 38 2x2-bolt a = 5" t = 3/4	case 33 3x2-bolt pure t = 3/4	case 36 3x2-bolt a = 3" t = 3/4	case 39 3x2-bolt a = 4" t = 3/4	case 34 4x2-bolt pure t = 3/4	case 37 4x2-bolt a = 3" t = 3/4	case 40 4x2-bolt a = 5" t = 3/4	case 41 3x2-bolt a = 3" t = 1/2	case 41 3x2-bolt a = 3" t = 1/2	case 42 4x2-bolt a = 3" t = 1/2	case 42 4x2-bolt a = 3" t = 1/2
1 st col.	1					-5.97	-7.66	-7.53	-5.54	-8.39	-7.58	-4.65	-2.19	-6.56	-4.54
	2	**1.05**	-2.27	**0.84**	-0.64	-1.87	-2.25	-2.40	-1.70	-2.66	-2.43	-1.33	-1.15	-1.79	-0.74
	3	-2.01	-1.18	**-3.45**	-1.47	0.37	0.18	0.04	0.69	0.44	0.16	0.05	-1.99	1.22	2.58
	4								3.34	3.90	2.90			3.84	3.26
2 nd col.	1					**-6.59**	**-8.56**	**-8.39**	**-6.10**	**-9.54**	**-8.61**	-6.24	**-0.89**	-8.66	**-4.16**
	2	0.18	-2.50	0.80	-0.77	-2.66	-3.36	-3.46	-2.25	-3.94	-3.61	-2.95	-4.44	-4.02	-3.91
	3	-3.62	-1.90	5.54	-1.88	0.24	0.27	-0.02	0.70	0.51	0.23	-0.54	-6.05	0.03	-1.86
	4								3.18	4.69	3.47			3.29	-1.73

All plates are Gr. 50 plate

All bolts are 3/4-in. A325N (Fu = 88 ksi)

Bold numbers indicate that shear fracture occurs in that bolt

Indicates the position of the beam neutral axis

\- Minus sign indicates that the force points towards the support

Table 5.36 Resultant on Bolts in Models 32 through 42

Bolt		Resultant, (kips)													
		case 32 2x2-bolt pure t = 3/4	case 35 2x2-bolt a = 3" t = 3/4	case 35 2x2-bolt a = 3" t = 3/4	case 38 2x2-bolt a = 5" t = 3/4	case 33 3x2-bolt pure t = 3/4	case 36 3x2-bolt a = 3" t = 3/4	case 39 3x2-bolt a = 4" t = 3/4	case 34 4x2-bolt pure t = 3/4	case 37 4x2-bolt a = 3" t = 3/4	case 40 4x2-bolt a = 5" t = 3/4	case 41 3x2-bolt a = 3" t = 1/2	case 41 3x2-bolt a = 3" t = 1/2	case 42 4x2-bolt a = 3" t = 1/2	case 42 4x2-bolt a = 3" t = 1/2
Nominal		23.33	23.33	23.33	23.33	23.33	23.33	23.33	23.33	23.33	23.33	23.33	23.33	23.33	23.33
1 st col.	1					17.63	11.77	12.41	17.26	10.54	10.53	11.32	19.93	14.03	19.04
	2	**22.80**	18.04	**22.55**	13.18	16.28	9.20	10.48	15.53	5.02	6.76	10.05	21.39	10.41	17.47
	3	22.58	18.55	**22.94**	13.31	17.30	9.40	9.98	16.26	6.20	8.07	9.83	22.20	10.42	18.01
	4								17.65	7.29	7.91			12.61	18.44
2 nd col.	1					**23.15**	**21.66**	**21.60**	**23.51**	**22.48**	**21.89**	18.97	**16.05**	22.30	**20.49**
	2	21.97	20.29	20.91	10.80	22.26	20.25	20.20	22.58	20.90	20.25	17.26	18.52	19.78	20.86
	3	21.82	19.80	21.07	10.44	22.19	19.73	19.90	22.44	20.14	19.59	16.50	16.76	19.10	19.54
	4								23.05	20.93	20.09			19.18	17.46

All plates are Gr. 50 plate

All bolts are 3/4-in. A325N (Fu = 88 ksi)

Bold numbers indicate that the shear fracture occurs in that bolt

Indicates the position of the beam neutral axis

to the center of rotation as in single-column bolt connections. However, the magnitude of the force is smaller than those in the 3/4-in. Gr. 50 plate single-column bolt connection because the increased plate thickness caused the bolts to plow through the beam web more than in the plate.

Plate behavior. As discussed in the bolt shear strength section, force redistribution occurred when the bending stress in the plate reached a specific value. To examine the redistribution behavior, the plate behavior in Model 42 is presented in Figures 5.22 and 5.23. The redistribution started at a shear force of approximately 120 kips, when the shear stress in the bolts in the second column started decreasing. The shear force in the plate increased steadily until it reached the value calculated by using the effective shear yielding relationship (Equation 5.2). Slightly after the redistribution started, the moment at the weld line became constant and started decreasing when the shear reached the value calculated by the effective shear formula. The constant moment indicates that the center of the shear force in both columns started moving toward the support to maintain the same amount of moment while the shear force kept increasing. The bending stress distribution in the plate cross section when the redistribution occurred is demonstrated in Figure 5.24. The calculation of bending moment of the section based on the distribution divided by the center of the force (shown in Table 5.38) gives the amount of shear force when the redistribution occurs.

The following relationship gives the plastic moment capacity, which is reduced by the presence of shear force:

$$M_{pv} = \frac{wt^2}{4}\sqrt{\sigma_y^2 - 3\left(\frac{V}{wt}\right)^2} \tag{5.6}$$

By moving the plastic section modulus term, $wt^2/4$, to the left side, setting the bending stress, $M_{pv}/(wt^2/4)$, equal to $0.4\sigma_y$, and replacing the V/wt term with τ, the equation becomes:

$$0.4\sigma_y = \sqrt{\sigma_y^2 - 3(\tau)^2}$$

The shear stress corresponding to the bending stress of $0.4\sigma_y$ is then:

$$\tau \approx 0.5\sigma_y$$

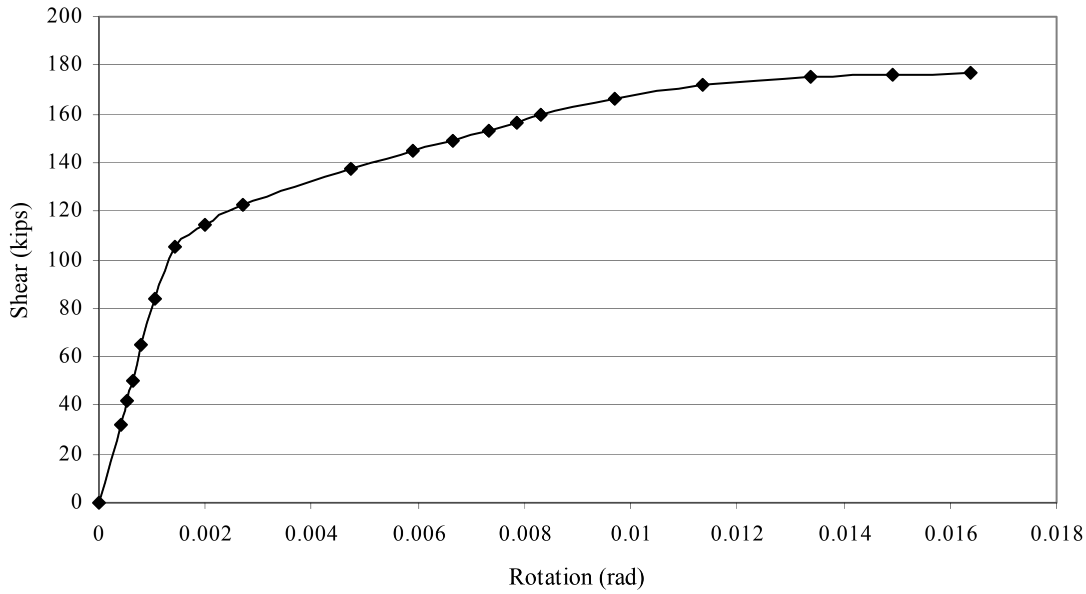


Figure 5.22 Shear vs. Rotation at Bolt Line of Model 42

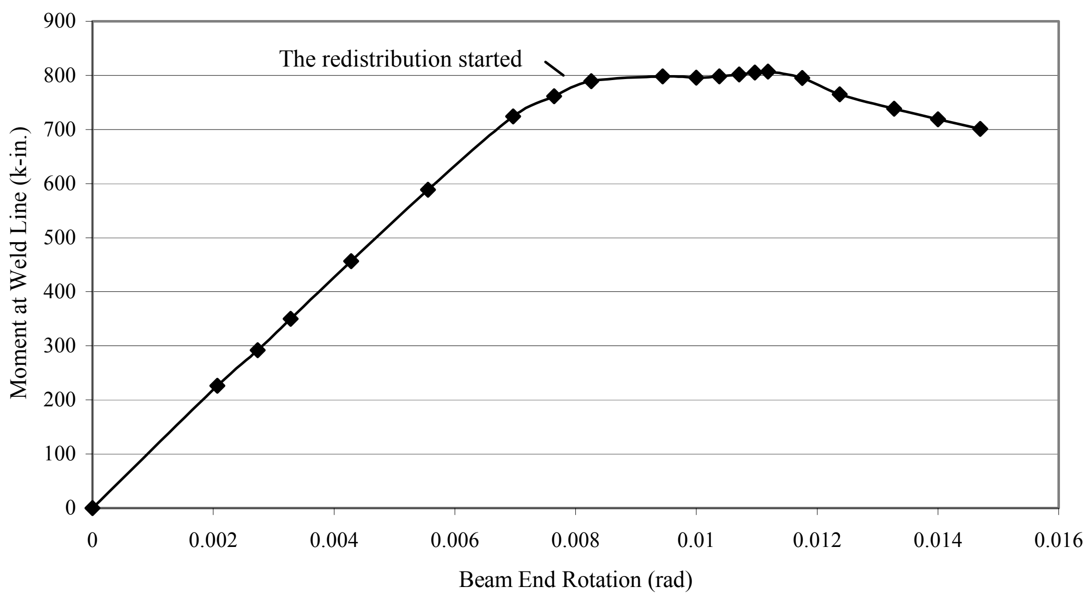


Figure 5.23 Moment at Weld Line vs. Beam End Rotation of Model 42

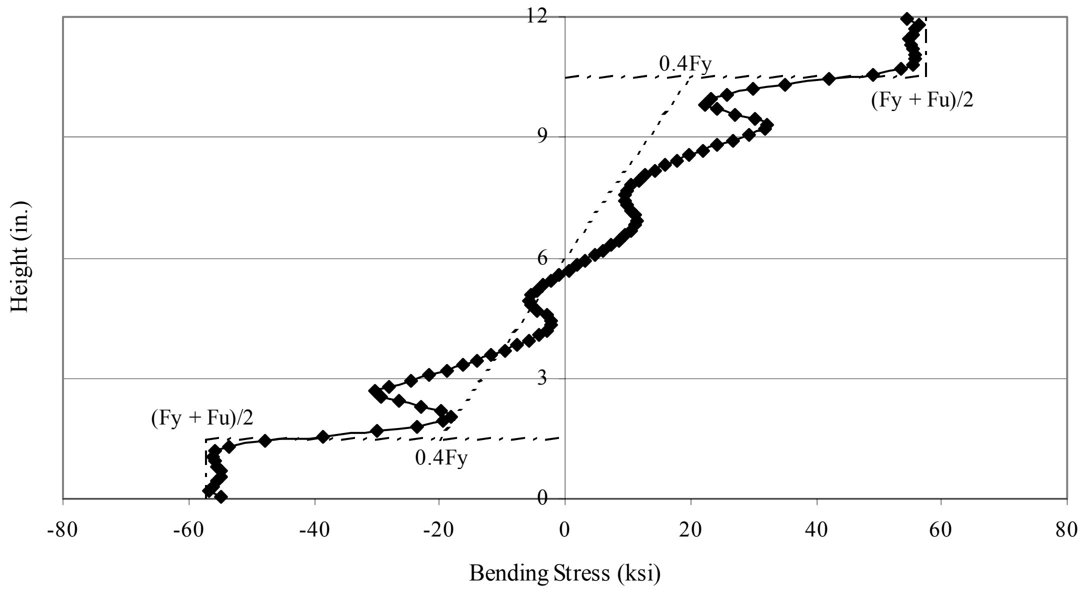


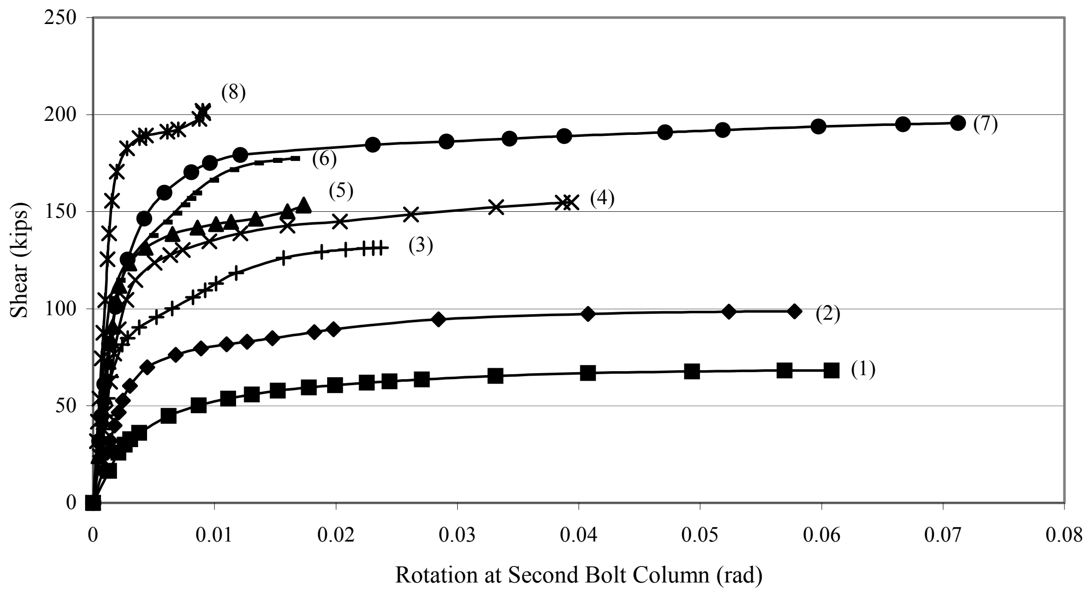
Figure 5.24 Bending Stress Distribution at Load Redistribution of Model 42

The calculation demonstrates that the bending stress of $0.4\sigma_y$ used in the stress distribution shown in Figure 5.24 is able to explain the behavior of the plate when the redistribution occurs.

The behavior of the plate in double-column connections, except for the pure shear cases, is summarized in Figure 5.25 and Table 5.37. The values of shear force at the redistribution calculated by the bending stress distribution in Figure 5.24 are included. The values calculated for the three models in which the redistribution occured are very close to the shear force at the beginning of the redistribution shown in Table 5.34.

Equilibrium. Calculations for verifying equilibrium and moment at the bolt line of double-column connections are summarized in Table 5.38. The calculations are based on the summation of the shear force in both bolt columns. The calculations of moment at the bolt group reveal that the a-distance also does not affect the behavior of the bolt group.

As in single-column bolt connections, the calculation for moment at the bolt line shows that the bolt shear strength is also not a function of the a-distance.



(1) The 3/4x9-1/2x6 in. plate (a = 5) (Model 38)
(2) The 3/4x7-1/2x6 in. plate (a = 3) (Model 35)
(3) The 1/2x7-1/2x9 in. plate (a = 3) (Model 41)
(4) The 3/4x8-1/2x9 in. plate (a = 4) (Model 39)
(5) The 3/4x7-1/2x9 in. plate (a = 3) (Model 36)
(6) The 1/2x7-1/2x12 in. plate (a = 3) (Model 42)
(7) The 3/4x9-1/2x12 in. plate (a = 5) (Model 40)
(8) The 3/4x7-1/2x12 in. plate (a = 3) (Model 37)

Figure 5.25 Shear vs. Rotation at Bolt Line of Models 35 through 42

Table 5.37 Shear Capacity of Plates in Figure 5.25

Model	Simulation		Prediction					
	PL Dimension (t, width, depth) (in.)	a distance (in.)	Beam Shear Yielding (kips)	Effective Yielding (kips)	Shear Yielding (kips)	Shear Rupture (kips)	PL Bend Plastic (kips)	PL Bend Redistr. (kips)
35	3/4x7-1/2x6	3.00	90.0	101.3	135.0	117.0	56.3	69.1
36	3/4x7-1/2x9	3.00	135.0	168.8	202.5	175.5	126.6	
37	3/4x7-1/2x12	3.00	180.0	236.3	270.0	234.0	225.0	
38	3/4x9-1/2x6	5.00	90.0	101.3	135.0	117.0	42.2	
39	3/4x8-1/2x9	4.00	135.0	168.8	202.5	175.5	108.5	
40	3/4x9-1/2x12	5.00	180.0	236.3	270.0	234.0	168.8	
41	1/2x7-1/2x9	3.00	90.0	112.5	135.0	117.0	84.4	78
42	1/2x7-1/2x12	3.00	120.0	157.5	180.0	156.0	150.0	120

Table 5.38 Calculations of Moment in Connections for Double-Column Bolt Connections

Simulation	No of Bolts	Pt. of Infl. (e) (in.) (1)	a distance (in.) (2)	e_b (in.) (3)	Moment @ Weld (k-in.) (4)	Shear 1st col (kips) (5)	Shear 2nd col (kips) (6)	Center of Force (c) (in.) (7)	Ve (k-in.) (8)	Vc (k-in.) (9)	Moment @ Bolt(i) (k-in.) (10)	Moment @ Bolt(ii) (k-in.) (11)
case 35	2x2	4.91	3	0.34	374	36.4	39.9	4.57	375	349	25	26
case 38	2x2	6.56	5	0.23	311	26.4	21.0	6.33	311	300	11	11
case 36	3x2	6.76	3	1.70	587	27.3	59.6	5.06	587	440	148	148
case 39	3x2	7.65	4	1.66	684	30.1	59.7	5.99	687	538	146	149
case 37	4x2	7.47	3	2.13	780	23.0	81.4	5.34	780	557	223	222
case 40	4x2	9.08	5	1.88	984	29.1	79.4	7.20	985	781	203	204
case 41	3x2	6.14	3	1.25	501	30.1	51.4	4.89	500	399	102	102
case 42	4x2	6.42	3	1.52	789	45.0	77.9	4.90	789	602	187	187

$$c = \frac{(Shear1stcol)(a) + (Shear2ndcol)(a+3)}{(Shear1stcol + Shear2ndcol)}$$

(i) $M_b = M$ @ Weld – Va

(ii) $M_b = Ve_b$

e_b is for entire bolt group (2 columns)

Moment @ Bolt is for entire bolt group (2 columns)

V is the sum of shear forces in both columns

5.5 Examination of Effect of Position of Connection with Respect to Beam

5.5.1 Introduction

The presence of the horizontal force in Gr. 50 plates, with its magnitude presumed to be a function of the location of the bolt with respect to the beam neutral axis, underlines the significance of the position of the connection. To investigate the effect of the position in a single column connection, Models 43 through 45 were created. Model 43 is a 3-bolt connection, Model 44 is a 5-bolt connection, and Model 45 is a 6-bolt connection. All plates used are Gr. 50 and bolts are A325 3/4-in diameter bolts. The details of Models 43 through 45 are shown in Table 5.39 and Figure 5.26. The results from these models are compared to results from Models 20, 22, 24, and 12.

In Model 43, the bolt group is positioned such that the centroid of the bolt group coincides with the beam neutral axis. In other words, the middle bolt is located at the beam neutral axis. The results from this model will be compared with those of Model 20 which has the same configuration except that the connection in Model 20 is installed at the top portion of the web, resulting in its bottommost bolt being aligned with the beam neutral axis.

The bolt group in Model 44 is arranged at the top portion of the beam web, therefore the fourth bolt in the connection (second from bottom) is located at the beam neutral axis. The results of the simulation are compared to those of Model 24, where the centroid of the bolt group is placed on the beam neutral axis, and those of Model 22, where the position of the top half of the bolt configuration is similar to the first four bolts in Model 44.

Model 45 is constructed with the bolt group at the top portion of the beam, which placed the fourth bolt (from the top) of the bolt group at the beam neutral axis. The results are compared to those of Model 12, where the bolt configuration is aligned with the beam neutral axis.

The results of the simulations, including the bolt shear strength of the connection and statics and moment in the connections, are summarized in Tables 5.40 through 5.44. The behavior of the plate in these models is not presented because it is the same as previously discussed in Section 5.2. Other plots and tables used to investigate the simulations are included in Appendix C.

Table 5.39 Details of Finite Element Models 43 through 45

Simulation	Beam		Bolts		Bolt Str. (ksi)	a-distance (in.)	PL Yield (ksi)	PL Dimensions (in.)
	Size	Span (ft)	No.	Type				
43	Gr.50 W18x55	28	3	3A325	88	3.00	50.0	3/8x4-1/2x9
44	Gr.50 W24x84	33	5	5A325	88	4.00	50.0	3/8x5-1/2x15
45	Gr.50 W24x84	28	6	6A325	96	3.50	50.0	3/8x5x18

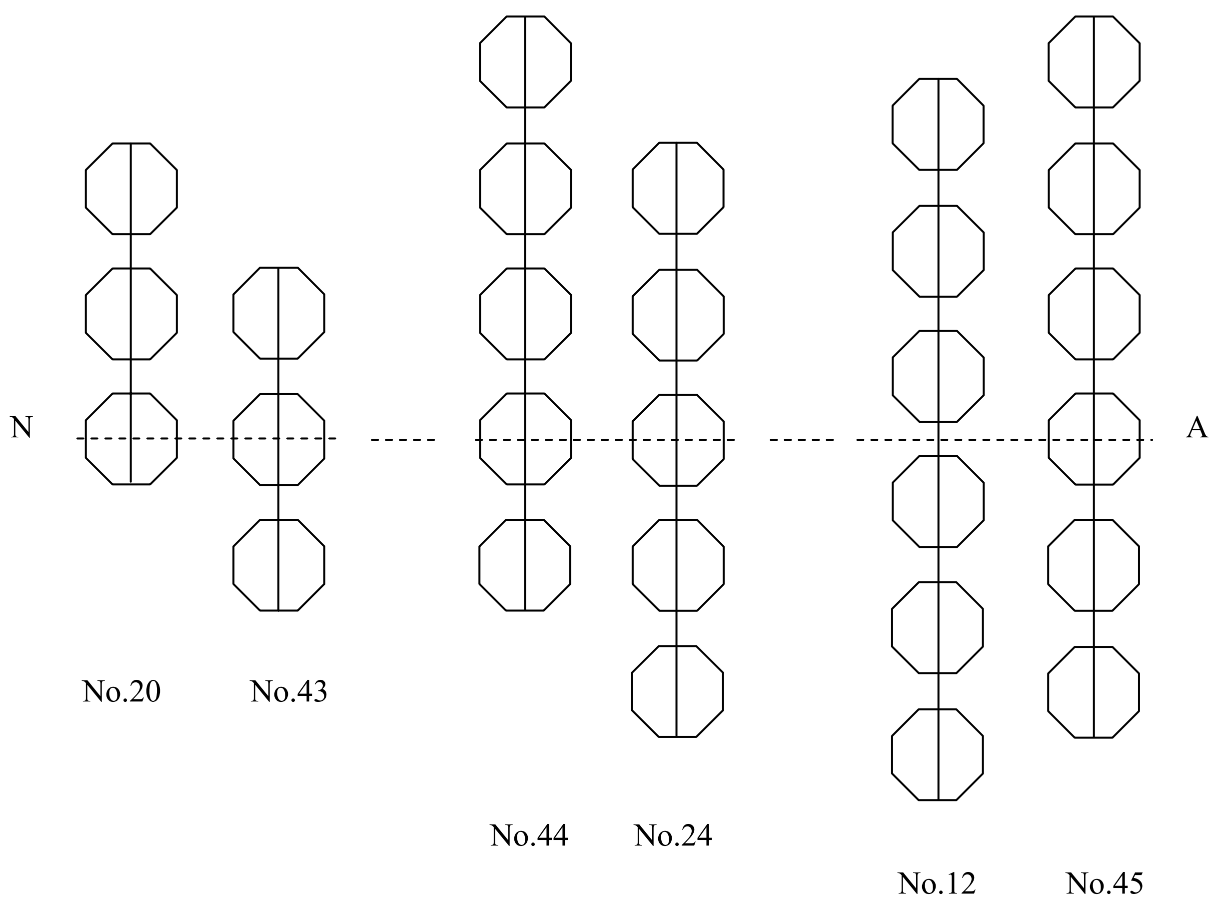


Figure 5.26 Position of Bolt Group with Respect to Beam Neutral Axis

5.5.2 Results and Discussions

The bolt shear strengths of Models 43 through 45 are summarized in Table 5.40. Bolt shear strengths of Models 12, 20, and 24 are included for comparison.

Table 5.40 Schemes of Simulations for Investigating Effect of Position on Shear Strength of Bolt Group

(a) Simulation

No. of Bolts	Simulation Number	
	Position	
	Mid	Top
3	43	20
5	24	44
6	12	45

(b) Bolt Shear Strength of Corresponding Models

No. of Bolts	Predicted Strength (kips)	
	Position	
	Mid	Top
3	59.5	58.7
5	96.9	84.8
6	104.1	101.5

The effect of position. The position of the bolt group with respect to the center of rotation has an effect on the shear strength of the connection. The effect is demonstrated more clearly by the horizontal and vertical forces, and the resultant acting on the bolts as shown in Tables 5.41 through 5.43.

The effect of the position was not apparent in 3-bolt connections because, even though the centroid of the bolt group in Model 43 was not placed at the beam neutral axis, as in Model 20, the distance of the farthest bolt from the beam neutral axis was only

6 in. The magnitude of the horizontal force acting on this bolt did not increase significantly from the one acting on the bolt located 3 in. from the center of rotation.

The effect of the position became significant in the 5-bolt connection. The bolt group positioned at the top of the beam in Model 44 carried a total vertical force of only 85 kips compared to 97 kips in Model 24. The vertical force carried by each bolt in Models 24 and 44 is different. The magnitude of the force in Model 24 is similar to those in Models 20 and 43, whereas in Model 44 the magnitude varies along the bolt configuration. The similarity of the force magnitude in Models 20, 24, and 43 is due to the same position of the bolts with respect to the center of rotation. The magnitude of the horizontal force on the farthest bolt in Model 44, which is 9 in. from the beam neutral axis, is greater than the vertical force. It can be clearly seen that, the farther the bolt is located from the center of rotation, the less the vertical strength because of the horizontal force component.

The results of Model 22, which is a 7-bolt connection, are included in Tables 5.41 through 5.43 for a comparison with those of Model 44 because of the similarity in the bolt positions with respect to the beam neutral axis. The comparison shows that the magnitude of the force acting on the bolts in the same position in Model 44 as those in Model 22 is almost equal. The size of the beam used in Models 22 and 44 is the same, but the length is different. This leads to the conclusion that the effect of the beam can be excluded from the shear capacity of the bolt group, as well as the moment.

The effect of the position was not very evident in Model 45 when compared to Model 12. This was because the position of the bolt groups in the two cases is only 1.5 in. different, that is, the distance of the outermost bolt in Model 12 is 7.5 in. from the center of rotation whereas in Model 45 the distance is 9 in. However, the magnitude of the horizontal force acting on the outermost bolts in both models is significant.

Equilibrium. The moment at the bolt line shown in Table 5.44 demonstrates very clearly, especially in the case of Model 44, that once the bolt group is positioned out of the alignment with the beam neutral axis, the moment it generates is much greater. This is because both the magnitude of the horizontal force and the distance to the center of roation, the force moment arm, are substantially increased.

Table 5.41 Vertical Force Acting on Bolts in Models 43 through 45

Bolt	Vertical Shear Force, (kips)						
	case 43 3-bolt a = 3" MID Fu=88	case 20 3-bolt a = 3" TOP Fu=88	case 24 5-bolt a = 4" MID Fu=88	case 44 5-bolt a = 4" TOP Fu=88	case 22 7-bolt a = 3" MID Fu=88	case 12 6-bolt a = 3.5" MID Fu=96	case 45 6-bolt a = 3.5" TOP Fu = 96
1				**15.00**	**15.23**		**15.17**
2		**18.50**	**19.68**	16.88	17.50	**16.31**	17.08
3	**19.83**	**20.06**	19.45	18.46	17.78	18.78	18.33
4	19.85	20.10	19.15	16.84	16.36	17.73	16.43
5	19.86		19.21	17.69	17.23	17.38	17.84
6			**19.39**		17.42	17.97	16.67
7					**14.86**	15.94	
Total	59.5	58.7	96.9	84.9	116.4	104.1	101.5

Bold numbers indicate that shear fracture occurs in that bolt

______ Indicates the position of the beam neutral axis

MID Indicates that the bolt group CG coincides with the beam neutral axis

TOP Indicates that the connection is installed at the top of the beam web

Table 5.42 Horizontal Force Acting on Bolts in Models 43 through 45

Bolt	Horizontal Shear Force, (kips)						
	case 43 3-bolt a = 3" MID Fu=88	case 20 3-bolt a = 3" TOP Fu=88	case 24 5-bolt a = 4" MID Fu=88	case 44 5-bolt a = 4" TOP Fu=88	case 22 7-bolt a = 3" MID Fu=88	case 12 6-bolt a = 3.5" MID Fu=96	case 45 6-bolt a = 3.5" TOP Fu = 96
1				**-17.35**	**-16.98**		**-18.18**
2		**-15.00**	**-13.33**	-15.75	-14.37	**-17.71**	-16.15
3	**-13.45**	**-12.47**	-7.48	-8.35	-6.60	-12.43	-8.03
4	-2.55	-1.31	-0.06	-0.50	-0.22	-3.99	-0.01
5	9.49		6.93	6.69	6.08	2.35	7.97
6			**13.24**		13.90	10.45	16.44
7					**17.11**	17.33	

\- Minus sign indicates that the force points towards the support

Table 5.43 Resultant Acting on Bolts in Models 43 through 45

Bolt	Resultant, (kips)						
	case 43 3-bolt a = 3" MID Fu=88	case 20 3-bolt a = 3" TOP Fu=88	case 24 5-bolt a = 4" MID Fu=88	case 44 5-bolt a = 4" TOP Fu=88	case 22 7-bolt a = 3" MID Fu=88	case 12 6-bolt a = 3.5" MID Fu=96	case 45 6-bolt a = 3.5" TOP Fu = 96
1				**22.94**	**22.81**		**23.68**
2		**23.82**	**23.77**	23.09	22.64	**24.08**	23.51
3	**23.96**	**23.62**	20.84	20.26	18.97	22.52	20.01
4	20.01	20.14	19.15	16.85	16.36	18.17	16.43
5	22.01		20.42	18.91	18.27	17.54	19.54
6			**23.48**		22.29	20.79	23.41
7					**22.66**	23.55	

Bold numbers indicate that shear fracture occurs in that bolt

_____ Indicates the position of the beam neutral axis

MID Indicates that the bolt group CG coincides with the beam neutral axis

TOP Indicates that the connection is installed at the top of the beam web

Table 5.44 Calculations of Moment in Connections for Verifying Effect of Position

Simulation	No of Bolts	Pt. of Infl. (e) (in.) (1)	a distance (in.) (2)	e_b (in.) (3)	Moment @ Weld (k-in.) (4)	Shear (kips) (5)	Ve (k-in.) (6)	Va (k-in.) (7)	Moment @ Bolt(i) (k-in.) (8)	Moment @ Bolt(ii) (k-in.) (9)
case 43	3	4.66	3	1.66	278	59.5	277	179	100	99
case 20	3	5.91	3	2.91	345	58.7	347	176	169	171
case 24	5	6.15	4	2.15	596	96.7	595	387	209	208
case 44	5	9.34	4	5.34	791	84.8	792	339	452	453
case 22	7	7.53	3	4.53	876	116.4	876	349	527	527
case 12	6	7.46	3.5	3.96	777	104.1	777	364	413	412
case 45	6	8.84	3.5	5.34	897	101.5	897	355	542	542

(i) M_b = M @ Weld – Va

(ii) $M_b = Ve_b$

5.6 Conclusions from Results of Simulations

From the results and analyses of the 45 simulations carried out, the following conclusions can be drawn:

1. Beams in the simulations were able to rotate and reach their maximum bending capacity.
2. Different bolt materials can be treated by means of a factor. The ratio of the bolt shear rupture strength of the failed bolt to the corresponding nominal shear strength of the bolt in each connection is summarized in Table 5.45.
3. The yielding behavior of the plate can be described using a beam shear formula and the formula developed from the observation of a shear distribution of the plate cross section at the point which the plate reaches the strain hardening stage.
4. The a-distance does not have any effect on a bolt group, as shown by both the bolt shear rupture and the moment at the bolt line.
5. Bolt groups in A36 plates had a small strength reduction whereas bolt groups in Gr. 50 plate had a much greater reduction. The strength reduction was caused by the horizontal force components acting on the bolts.
6. The horizontal force on bolts increases as the distance of the bolt from the center of rotation increases.
7. The bolt group in the connection moved altogether about the center of rotation; the movement was not as assumed by the instantaneous center concept. The amount of displacement of a bolt is a function of the distance from the center of rotation (beam neutral axis), not the distance to the instantaneous center, which is an imaginary point.
8. The moment at the bolt line is the summation of the moments created by the horizontal forces acting on each bolt in the connection.
9. The moment at the weld line is the summation of the moments created by the shear force (Va) and the moment created at the bolt group.
10. The 3/8-in. A36 plates were ductile enough for 3/4-in. diameter bolts to plow through and for beams to reach their bending capacity. The 1/2-in. A36 and 3/8-in. Gr. 50 plates were less ductile and produced horizontal forces acting on bolts, but still ductile enough to provide a rotation capacity to beams.

Table 5.45 Ratio of Vertical Force and Resultant to Nominal Strength on Failed Bolts

case	PL Thickness/Yield (in./ksi)	Vertical Force (kips)	Resultant (kips)	F_u (ksi)	F_uA_b (kips)	Ratio of Vertical Force to Nominal	Ratio of Resultant to Nominal
9	0.375/36	28.97	29.05	110	29.2	0.99	1.00
10	0.375/36	28.97	29.24	110	29.2	0.99	1.00
11	0.375/47	19.93	25.35	96	25.4	0.78	1.00
12	0.375/47	16.31	24.08	96	25.4	0.64	0.95
13	0.375/36	29.31	29.34	110	29.2	1.01	1.01
14	0.375/36	23.24	23.44	88	23.3	1.00	1.00
15	0.375/36	22.53	22.53	88	23.3	0.97	0.97
16	0.375/36	22.27	22.50	88	23.3	0.95	0.96
17	0.375/36	22.45	23.23	88	23.3	0.96	1.00
18	0.375/50	23.04	23.13	88	23.3	0.99	0.99
19	0.375/50	22.11	22.99	88	23.3	0.95	0.99
20	0.375/50	18.50	23.82	88	23.3	0.79	1.02
21	0.375/50	19.70	23.83	88	23.3	0.84	1.02
22	0.375/50	15.23	22.81	88	23.3	0.65	0.98
23	0.375/50	18.10	23.53	88	23.3	0.78	1.01
24	0.375/50	19.68	23.77	88	23.3	0.84	1.02
25	0.375/50	15.13	22.70	88	23.3	0.65	0.97
26	0.500/36	20.07	23.71	88	23.3	0.86	1.02
27	0.500/50	18.46	23.74	88	23.3	0.79	1.02
28	0.750/36	18.01	23.61	88	23.3	0.77	1.01
29	0.750/50	17.31	23.71	88	23.3	0.74	1.02
30	0.375/43	18.56	23.74	88	23.3	0.80	1.02
31	0.500/43	18.95	23.98	88	23.3	0.81	1.03
32	0.750/50	22.78	22.80	88	23.3	0.98	0.98
33	0.750/50	22.19	23.15	88	23.3	0.95	0.99
34	0.750/50	22.71	23.51	88	23.3	0.97	1.01
35	0.750/50	22.68	22.94	88	23.3	0.97	0.98
36	0.750/50	19.90	21.66	88	23.3	0.85	0.93
37	0.750/50	20.35	22.48	88	23.3	0.87	0.96
38*	0.750/50	13.23	13.31	88	23.3	0.57	0.57
39	0.750/50	19.90	21.60	88	23.3	0.85	0.93
40	0.750/50	20.13	21.89	88	23.3	0.86	0.94
41	0.500/50	16.03	16.05	88	23.3	0.69	0.69
42	0.500/50	20.06	20.49	88	23.3	0.86	0.88
43	0.375/50	19.83	23.96	88	23.3	0.85	1.03
44	0.375/50	15.00	22.94	88	23.3	0.64	0.98
45	0.375/50	15.17	23.68	96	25.4	0.60	0.93

*Bolts did not fail.

11. It is inconclusive whether or not the 3/4-in. A36 and the 1/2-in., and 3/4-in. Gr. 50 plates are ductile enough for 3/4-in. diameter bolts, because in the simulations plowing of bolts occurred in the beam web.
12. Plates with yield strength equal to 43 ksi caused the same horizontal force as Gr. 50 plates.
13. In double-column connections, 1/2-in. Gr. 50 plates were able to redistribute forces from the second column bolt to the first column, while in 3/4-in. Gr. 50 plates, the bolts in the second column failed when bolts in the first column carried an average shear force of only 11 kips as compared to their nominal strength, 23.3 kips.
14. Plates in double-column bolt connections started yielding under bending when the bending stress distribution of the plate cross section reached $0.4F_y$ for a triangular stress distribution ranging from the center of the topmost bolt to the center of the bottommost bolt, and $(F_y + F_u)/2$ for rectangular stress distribution of the area above and below the centers of topmost and bottommost bolts, respectively. Plates then started redistributing forces to the first column of bolts. As a result, the center of the force started shifting towards the support to maintain the same amount of moment.
15. The position of the connection with respect to the center of rotation (beam neutral axis) had an effect on bolt group strength because the magnitude of the horizontal forces acting on the bolts in less ductile plates was a function of the distance of the bolts to the center of rotation.

Conclusions drawn from the results of finite element analyses of the simulations in this chapter are further examined in Chapter VI.

Chapter VI
Conclusions

6.1 Conclusions on the Behavior of Single Plate Shear Connections

The results of the simulations in association with the history of the development of the current design model and fundamental analysis lead to the following conclusions on the behavior of single plate shear connections.

6.1.1 Behavior of Plate

1. The plate shear yielding behavior can be best predicted using the shear stress distribution in the plate cross section when the plate enters the strain hardening stage. The proposed relationship is:

$$R_n = 0.6F_y[(n-1)p + L_e]t \tag{6.1}$$

   The relationship in Equation 6.1 reflects the distribution where the shear stress decreases from $0.6F_y$ at the center of the topmost and the bottommost bolt holes to zero at the top and the bottom of the plate, respectively.

2. Force redistribution occurs in double-column bolt connections. The magnitude of the shear force when redistribution occurs is calculated from the moment in the plate divided by the distance to the center of force carried by the two bolt column. The moment is calculated assuming a triangular bending distribution between the center of the topmost and the bottommost bolts with the maximum value of $0.4F_y$, and a rectangular bending distribution in the area above the center of the topmost and below the center of the bottommost bolts with a maximum value of $(F_y + F_u)/2$, as shown in Figure 6.1. After the distribution, the moment remains constant because the center of the shear force moves toward the support while the shear force increases.

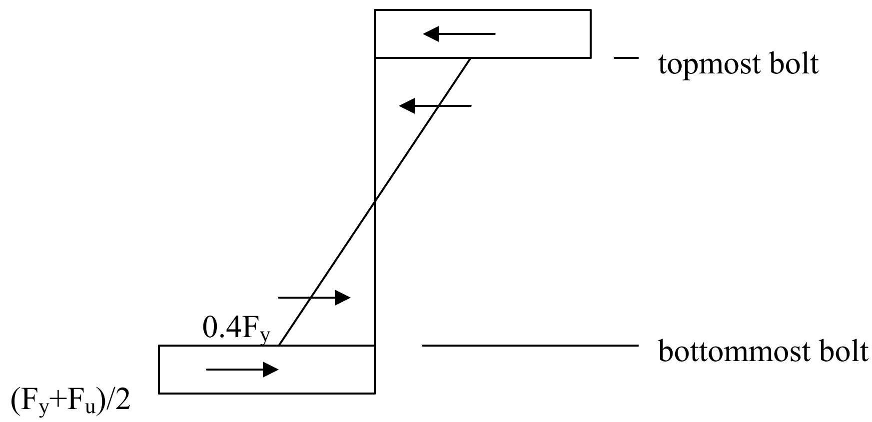


Figure 6.1 Bending Stress Distribution at the Force Redistribution

3. Plates are ductile enough for bolts to plow through without imposing significant horizontal force when the plate thickness satisfies the following limitation as suggested by Richard (1980):

$$t_p \leq \frac{d_b}{2} \tag{6.2}$$

4. Maximum plate thickness for A36 material when used with 3/4-in. dia. bolts according to Inequality 6.2 is 3/8 in. assuming the relationship between the plate material and thickness is linear, the equivalent thickness for Gr. 50 plate is:

$$t_{p,Gr.50} = \frac{36}{50} x 0.375 = 0.27 \text{ in.}$$

The calculation suggests that 3/4-in. dia. bolts in 1/2-in. A36 plates are predicted to behave similarly to those in 3/8-in. Gr. 50 plates. This is also observed in the results from the simulations. As a result, the limitation of Inequality 6.2 can be extended into the following inequality:

$$t_p \leq \frac{d_b}{2}\left(\frac{36}{F_y}\right) \tag{6.3}$$

With this modification to account for the strength of the plate material, the behavior of 3/4-in. dia. bolts in 3/8-in. Gr. 50 plates can be further explained.

Based on Inequality 6.3, 3/8-in. Gr. 50 plates used with 3/4-in. diameter bolts exceed the limitation (0.75/2x36/50 = 0.27 in. ≤ 0.375 in.).

5. From the simulation results, when plate thickness is greater than the limitation in Inequality 6.3, a resisting horizontal force is created by the plate, and the bolt shear strength of the connection is reduced. Considering 1/2-in. A36 and the 3/8-in. Gr. 50 plates with 3/4 in. diameter bolts:

   For 1/2-in. A36:

   $$\frac{t_p}{d_b} = \frac{0.5}{0.75} = 0.67$$

   For 3/8-in. Gr. 50:

   $$\frac{t_p}{d_b} = \frac{0.375}{0.75} x \frac{50}{36} = 0.7$$

   In both cases, plates are ductile enough for bolts to plow through. As a result, the following limitation is obtained:

   $$t_p \leq 0.7 d_b \left(\frac{36}{F_y} \right) \tag{6.4}$$

6. A further study is needed for plates thicker than recommended by Inequality 6.4 to ensure that the ductility can be achieved.
7. It should be noted that the plate shear rupture limit state becomes the governing limit state for plates instead of plate shear yielding once the plate material is changed from A36 to Gr. 50. Thus, it is recommended that shear rupture of the plate be further investigated.

6.1.2 Behavior of Bolt Group

1. Connections with either A325 or A490 bolts and plate thickness satisfying the limitation in Inequality 6.3 can be designed without considering eccentricity, that is, $e_b = 0$. As a result, the bolt shear strength is simply F_vA_bn.
2. Connections with either A325 or A490 bolts and plate thickness greater than the limitation in Inequality 6.3 must be designed considering eccentricity. The moment is created by horizontal forces, which arise from the bearing resistance of the base material that restricts the bolt movement. This moment is a function of the bolt configuration as shown in Figure 6.2; it is not a function of the cantilever distance of the plate from the support to the bolt line, or the a-distance.
3. The connection bolt shear strength is not a function of the a-distance regardless of plate thickness, that is, smaller or greater than the limitation in Inequality 6.3.
4. Bolts under direct shear, even used in plates thicker than the limitation in Inequality 6.3, do not have their capacity reduced.
5. Bolt configurations have an effect on a bolt group capacity in terms of the horizontal force. The horizontal force is a function of the location of the bolt with respect to the center of rotation; the greater the distance of the bolt from the center of rotation, the greater the force. As a result, the position of a bolt group with respect to the neutral axis of the beam has an impact on the shear capacity of the connection. The horizontal force reduces the capacity of a bolt to resist vertical force. To maximize the vertical shear capacity of the bolt group, it is recommended that the connection be designed such that the beam neutral axis coincides with the centroid of the bolt configuration.
6. The movement of the bolt group is not as assumed by the concept of instantaneous center.
7. Forces in double-column bolt connections are redistributed when the plate thickness used is not greater than 1/2 in. for Gr. 50 steel. When the plate thickness is greater than 1/2 in., the bolt(s) in the second column fails while the shear force carried by bolts in the first column is much less than the shear force in the second column.

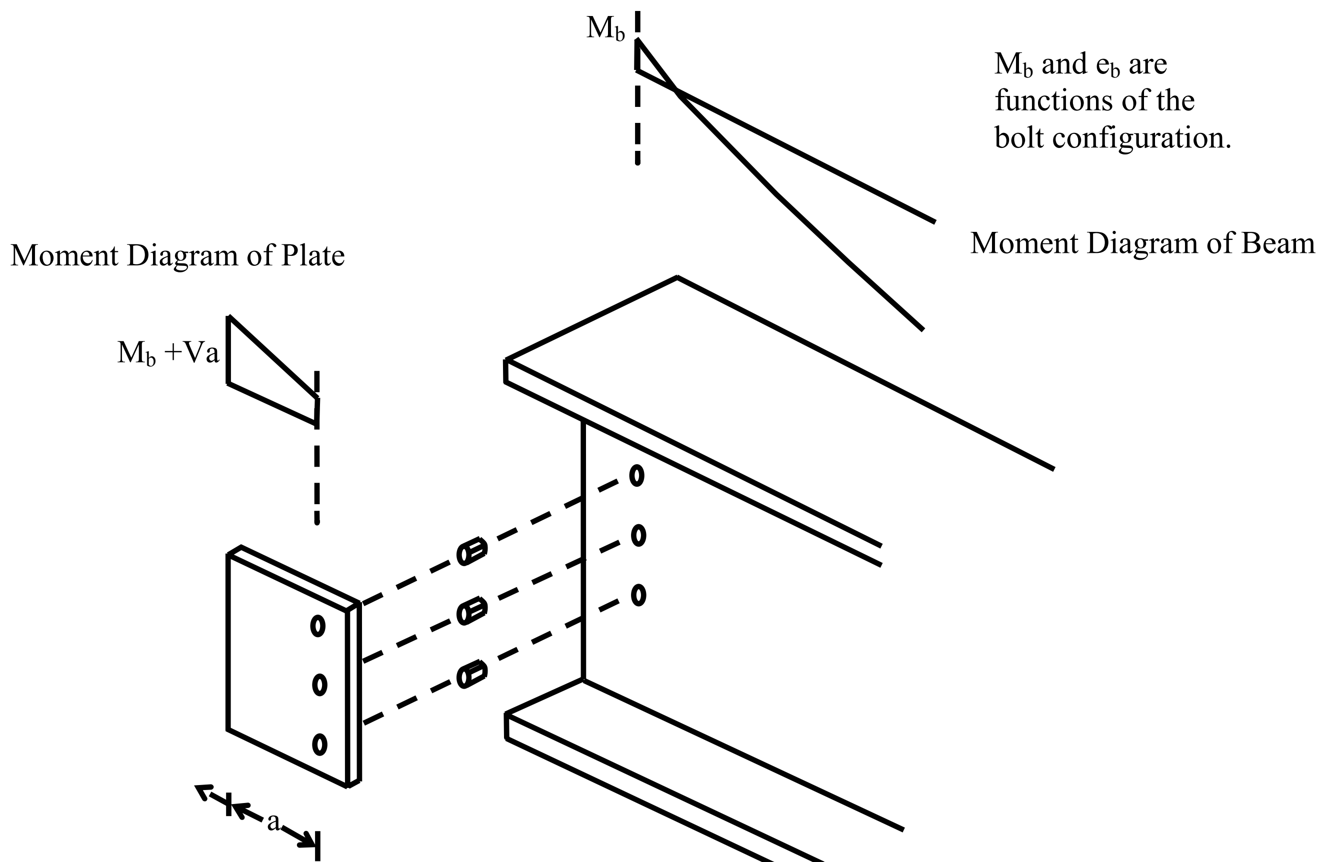


Figure 6.2 Moment Diagrams of Beam and Plate

6.1.3 Forces on Welds

1. Welds in a connection where the plate thickness satisfies Inequality 6.3 should be designed to accommodate a shear force equal to the amount carried by the bolt group in the connection with an eccentricity, e_w, equal to the a-distance of the plate.
2. Welds in a connection where the plate thickness is greater than the limitation of Inequality 6.3 should be designed to accommodate the summation of the moment produced by both the vertical shear force (Va) and the horizontal force (M_b) as shown in Figure 6.2.

6.1.4 Ductility

1. The ductility of the entire system is sufficiently provided when the plate thickness satisfies Inequality 6.3 or 6.4.
2. The beam rotation is a function of the beam size and length, and not the connection configuration.

6.1.5 Concept of Instantaneous Center

1. The horizontal component of the bolt resistance reduces shear strength of the bolt and is a function of the distance of the bolt from the center of rotation. The farther the bolt is from the center of the rotation (or the beam neutral axis), the greater the horizontal component.

6.2 Proposed Design Model for Single Plate Shear Connections

A design model for calculating bolt shear strength and moment on welds of single-column bolt configurations is proposed. The model, taking into account the effect of the plate material, is divided into two parts: a design when $t_p \leq (d_b/2)(36/F_y)$, and a design when $(d_b/2)(36/F_y) < t_p \leq (0.7d_b)(36/F_y)$.

1. When $t_p \leq \left(\frac{d_b}{2}\right)\left(\frac{36}{F_y}\right)$

The first case is developed from the results of 3/8-in. A36-plate connections where the horizontal forces on the bolts were found to be small. Even though the effect is not significant, a five percent reduction is applied to the bolt shear strength.

The amount of moment generated by the horizontal or plowing forces is negligible for the design of welds. Thus, the design moment for welds is simply the one generated by a shear force, Va.

2. When $\left(\frac{d_b}{2}\right)\left(\frac{36}{F_y}\right) < t_p \leq \left(0.7d_b\right)\left(\frac{36}{F_y}\right)$

The second case is developed from the results of 1/2-in. A36-plate connections and 3/8-in. Gr. 50-plate connections. Reduction of bolt shear strength is substantial in this case.

The results of 7-bolt connections in 3/8-in. Gr. 50 plate show that the innermost bolts do not carry the force (resultant) up to the maximum shear strength before bolt failure occurs in the outermost bolt(s). Therefore, the extrapolation of the bolt shear strength of a bolt configuration greater than seven is not recommended. In addition, the bolt shear strength of the 7-bolt connections from the simulations indicates large strength reduction (at least 30 percent less than the nominal shear strength). As a result, the use of a bolt configuration greater than seven when the plate thickness is greater than the first limitation (Inequality 6.3) is not recommended.

The design of welds must account for both the moment generated by a shear force, Va, and by a bolt group, M_b. The moment of a bolt group, M_b, is obtained by taking the moment of the factored horizontal force provided in Figure 6.3 with respect to the neutral axis. To accommodate the design tables in the Manual, the moment M_b is transformed into the eccentricity e_b.

The proposed design model is summarized as follows:

When $t_p \le \left(\frac{d_b}{2}\right)\left(\frac{36}{F_y}\right)$

$$V_n = 0.95 F_v A_b n \quad (6.5)$$

$$e_w = a \quad (6.6)$$

When $\left(\frac{d_b}{2}\right)\left(\frac{36}{F_y}\right) < t_p \le \left(0.7 d_b\right)\left(\frac{36}{F_y}\right)$

For $n \le 5$

$$V_n = 0.84\, F_v A_b n \quad (6.7)$$

For $5 \le n \le 7$

$$V_n = \Sigma\, r_n \quad (6.8)$$

where

(a) Bolts with distance to neutral axis greater than 6 in.

$$r_n = 0.64\, F_v A_b \quad (6.9)$$

(b) Bolts with distance to neutral axis within 6 in.

$$r_n = 0.70\, F_v A_b \quad (6.10)$$

$$e_w = a + M_b/V_n \quad (6.11)$$

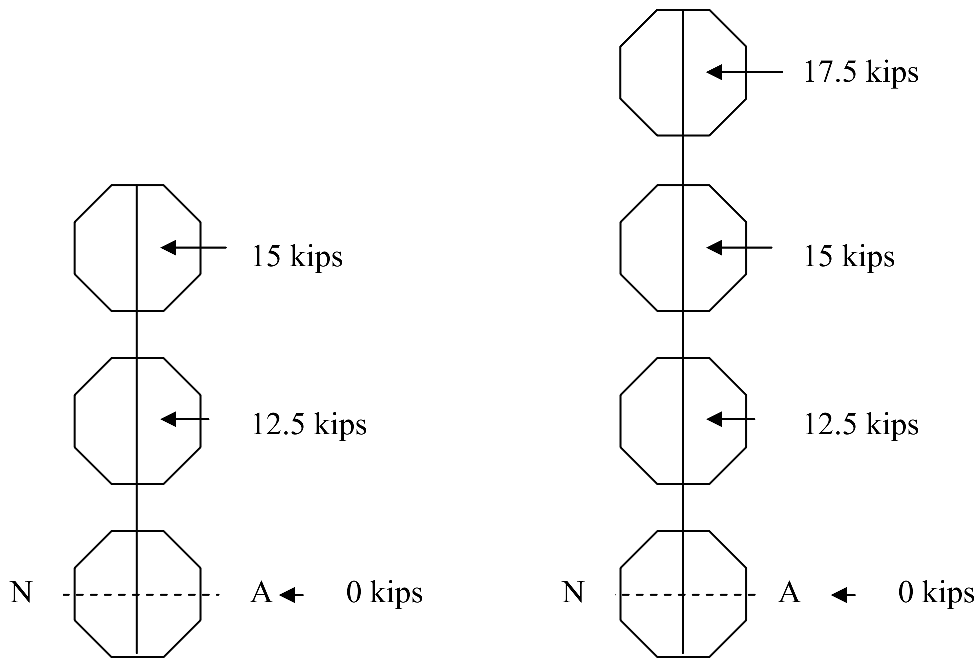
and

V_n = bolt shear strength (kips)

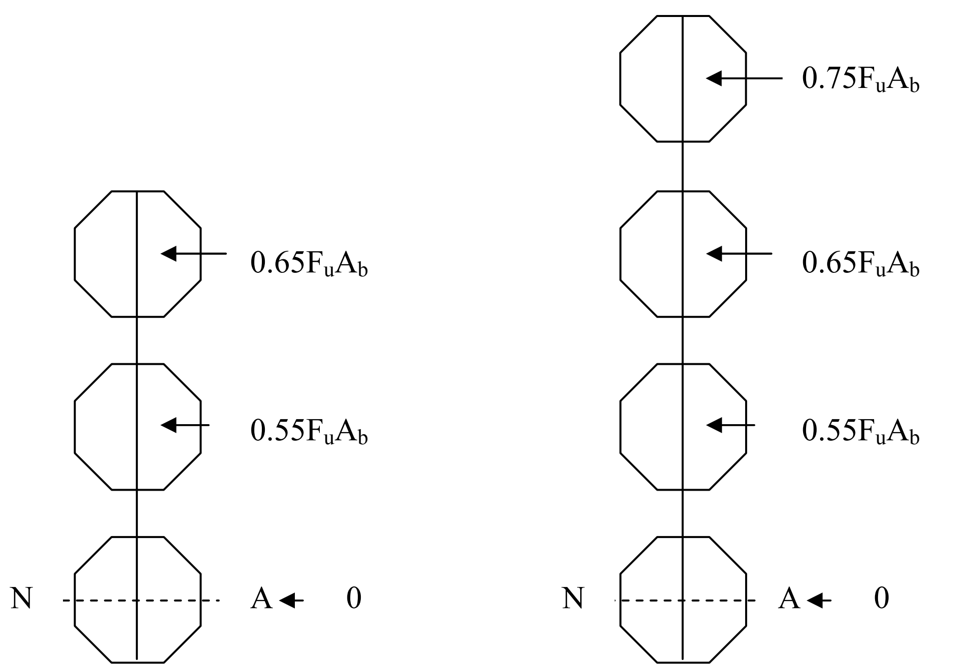
r_n = bolt shear strength of a single bolt (kips)

e_w = eccentricity on welds (in.)

M_b = moment on bolt group calculated using the horizontal forces shown in Figure 6.3 (k-in.)



(a) Average Horizontal Force Based on A325 3/4-in. Dia. Bolts



(b) Horizontal Force on Bolts

Figure 6.3 Horizontal Force on Bolts for 5- and 7-Bolt Configurations (Top Half Only)

The following limitations apply:

1. The connection must be placed such that its centroid coincides with the beam neutral axis.
2. To ensure ductility, plates must not be thicker than $(0.7d_b)(36/F_y)$.

In addition, the following relationship is proposed to be used in the calculation of the plate shear yielding limit state:

$$R_n = 0.6F_y[(n-1)p + L_e]t$$

where

F_y	=	yield strength of the plate (ksi)
n	=	number of bolts in the plate
p	=	spacing between bolts (in.)
L_e	=	edge distance in vertical direction (in.)
t	=	thickness of the plate (in.)

6.3 Predictions of Test Results Using Proposed Design Method

To evaluate a performance of the proposed design method, comparisons of the strength of connections tested by Astaneh and Sarkar and the values predicted by this method were made. Bolt shear strength used in calculations for the first six connections, which were tested by Astaneh, are as recommended by the Manuals (1989, 2001). Calculations for both N- and X-type bolts were carried out for these connections. Bolt shear strength used in calculations for the last three connections, which were tested by Sarkar, is 57.6 ksi (0.6x120x0.8). The results are shown in Table 6.1.

The proposed design model predicts the bolt shear strength of the connections close to the current design model for the 3- and 5-bolt connections. For the 7- and 9-bolt connections, the values predicted by using the proposed model are closer to the test results than those predicted by the current model. For the 2-, 4-, and 6-bolt connections, the proposed model yields better values than the current model.

Table 6.1 Predictions of Test Results by Proposed Design Method

Connection	Test Results (kips)	Predicted Strength, N-type (kips)		Predicted Strength, X-type (kips)	
		Current Model	Proposed Method	Current Model	Proposed Method
3-A325	94	59.6	60.4	74.5	75.5
5-A325	137	101.0	100.7	126.3	125.9
7-A325	160	125.6	141.0	157.0	176.3
3-A490	79	74.5	75.5	93.1	94.4
5-A490	130	124.4	125.9	155.5	157.4
9-A490	260	150.4	226.6	180.5	272.0
2-A325	52, 61 (Welds)	21.4	42.8		
4-A325	67, 82	82.3	85.5		
6-A325	102, 109	119.4	103.8		

Bolt shear strength used is 0.6x120x0.8 = 57.6 ksi

6.4 Suggestions for Future Research

Plate shear rupture is the limit state that should be further studied. According to the shear stress distribution obtained from this research, the portions of the plate above the topmost and below the bottommost bolts do not experience the shear stress entirely; the shear stress gradually decreases from the maximum value at the center of the bolt holes to zero at the top and the bottom of the plate.

Additional studies on double-column bolt configurations for single plate shear connections should be carried out. Even though it might be possible to use double-column bolt configurations with plates that satisfy the condition in Inequality 6.4, such as 3/4-in. dia. bolts with 1/2-in. A36 plates, it is not recommended due to insufficient investigation.

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Appendix A

Introduction

Appendix A contains calculations made to verify Astaneh's test results with current LRFD Design Specifications as shown in Table 3-11. Limit states considered in the calculations for each connection are:

1. Shear yielding of plate.
2. Shear rupture of plate.
3. Block shear of plate.
4. Bearing/Tear-out of plate.
5. Flexural yielding of plate.
6. Shear strength of bolts.
7. Shear strength of welds

The first four limit states are calculated according to current LRFD Design Specifications. Flexural yielding of plate capacity is shown in terms of bending strength, not shear, because no design model is available regarding this failure mode. Shear strength of the bolts is calculated based on current specifications, that is, eccentricity is calculated according to Astaneh's formulas, and the number of effective bolts is calculated using Table 7-17 in LRFD Manual (2001). The two methods of calculating eccentricity that use Table 8-5 in LRFD Manual (2001) for welds are presented. The first method was presented by Astaneh in 1989, whereas the second was presented by Astaneh et al. in 1993. Factors of safety, ϕ, are excluded from all calculations.

Material Properties

Material properties used in the calculations are:

1. For A36 plate, F_y = 35.5 ksi, and F_u = 61 ksi (as reported by Astaneh).
2. For A325 N bolts, F_v = 36/0.75 = 48 ksi (nominal per specification since was not indicated in the research).
3. For A490 N bolts, Fv = 45/0.75 = 60 ksi (nominal per specification since was not indicated in the research).
4. For E70xx welding, F_u = 0.6x70/0.75 = 56 ksi (nominal per specification since was not indicated in the research).

3-A325 Bolt Connection

Configuration:

- 3-A325 3/4-in diameter bolt
- Plate 3/8 x 4 1/4 x 0'-9 A36 with 1 1/2 in. edge distance and 3 in. bolt spacing.
- E70xx 0.25 in. weld

AISC Nominal Strength:

1. Shear yielding of plate

   $0.6F_yA_g$ = 0.6x35.5x0.375x9 = 71.9 kips
2. Shear rupture of plate

   $0.6F_uA_n$ = 0.6x61x0.375x{9-3(3/4+1/8)} = 87.5 kips
3. Block shear of plate

   Tension rupture = F_uA_n = 61x0.375x{1.5-0.5x(3/4+1/8)} = 24.3 kips

   Shear rupture = $0.6F_uA_n$ = 0.6x61x0.375x{7.5-2.5x(3/4+1/8)} = 72.9 kips ← governing

   Tension yielding = F_yA_g = 35.5x0.375x1.5 = 20.0 kips ← less than tension rupture

   ∴Block shear = 72.9 + 20 = 92.9 kips
4. Bearing/tear-out

   Bearing = $2.4F_ud_bt_p$ = 2.4x61x3/4x0.375 = 41.2 kips

   Tear-out, edge = $1.2F_ut_pL_c$ = 1.2x61x0.375x{1.5-0.5x(3/4+1/16)} = 30.0 kips ← less

   , interior = $1.2F_ut_pL_c$ = 1.2x61x0.375x{3-1x(3/4+1/16)} = 60.0 kips ← more

   ∴Bearing/tear-out = 1x30.0 + 2x41.2 = 112.4 kips
5. Flexural yielding of plate

   $F_yS_x = F_yt_pl^2/6$ = 35.5x0.375x$(9.0)^2$/6 = 179.7 k-in.
6. Shear strength of bolts

   Eccentricity of bolt group = | (n-1) – a | = | (3-1) – 2.75 | = 0.75 in. (must extrapolate)

   By calculation based on instantaneous center approach, number of bolts effective = 2.81

   ∴Shear strength of bolts = 2.81x(36/0.75)x0.442 = 59.6 kips for type N (63.6 kips full)

7. Shear strength of welds (I)

   Eccentricity of welds = Max(n:a) = 3 in.

   By interpolation a = e_w/l = 3/9 = 0.33, C = 2.22

   $\therefore$ Shear strength of weld = CC_1Dl = (2.22/0.75)x1x4x9 = 106.6 kips

   Shear strength of welds (II)

   Eccentricity of welds = (n-1) = 3-1 = 2 in.

   By interpolation a = e_w/l = 2/9 = 0.22, C = 2.58

   $\therefore$ Shear strength of weld = CC_1Dl = (2.58/0.75)x1x4x9 = 123.6 kips

Summary of Limit States

Type of Limit States	3-A325 bolt
1. Shear yielding of plate	71.9 kips
2. Shear rupture of plate	87.5 kips
3. Block shear of plate	92.9 kips
4. Bearing/tear-out of plate	112.4 kips
5. Flexural yielding of plate	179.7 k-in
6. Shear strength of bolts, N	59.6 kips (63.6)
eccentricity	0.75 in.
Coefficient C	2.81
7. Shear strength of weld (I)	106.6 kips
eccentricity	3 in.
Shear strength of weld (II)	123.6 kips
eccentricity	2 in.

5-A325 Bolt Connection

Configuration:

- 5-A325 3/4- in. bolt
- Plate 3/8 x 4 1/4 x 1'-3 A36 with 1 1/2 in. edge distance and 3 in. bolt spacing.
- E70xx 0.25 in. weld

AISC Nominal Strength:

1. Shear yielding of plate

   $0.6F_yA_g$ = 0.6x35.5x0.375x15 = 119.8 kips
2. Shear rupture of plate

   $0.6F_uA_n$ = 0.6x61x0.375x{15-5(3/4+1/8)} = 145.8 kips
3. Block shear of plate

   Tension rupture = F_uA_n = 61x0.375x{1.5-0.5x(3/4+1/8)} = 24.3 kips

   Shear rupture = $0.6F_uA_n$ = 0.6x61x0.375x{13.5-4.5x(3/4+1/8)} = 131.2 kips ← governing

   Tension yielding = F_yA_g = 35.5x0.375x1.5 = 20.0 kips ← less than tension rupture

   ∴Block shear = 131.2 + 20 = 151.2 kips
4. Bearing/tear-out

   Bearing = $2.4F_ud_bt_p$ = 2.4x61x3/4x0.375 = 41.2 kips

   Tear-out, edge = $1.2F_ut_pL_c$ = 1.2x61x0.375x{1.5-0.5x(3/4+1/16)} = 30.0 kips ← less

   , interior = $1.2F_ut_pL_c$ = 1.2x61x0.375x{3-1x(3/4+1/16)} = 60.0 kips ← more

   ∴Bearing/tear-out = 1x30.0 + 4x41.2 = 194.8 kips
5. Flexural yielding of plate

   $F_yS_x = F_yt_pl^2/6$ = 35.5x0.375x$(15.0)^2$/6 = 499.2 k-in.
6. Shear strength of bolts

   Eccentricity of bolt group = | (n-1) – a | = | (5-1) – 2.75 | = 1.25 in. (must extrapolate)

   By calculation based on instantaneous center approach, number of bolts effective = 4.69

   ∴Shear strength of bolts = 4.69x(36/0.75)x0.442 = 99.5 kips for type N (106.1 kips for full)

7. Shear strength of welds (I)

   Eccentricity of welds = Max(n:a) = 5 in.

   By interpolation a = e_w/l = 5/15 = 0.33, C = 2.22

   $\therefore$ Shear strength of weld = CC_1Dl = (2.22/0.75)x1x4x15 = 177.6 kips

   Shear strength of welds (II)

   Eccentricity of welds = (n-1) = 5-1 = 4 in.

   By interpolation a = e_w/l = 4/15 = 0.27, C = 2.42

   $\therefore$ Shear strength of weld = CC_1Dl = (2.42/0.75)x1x4x15 = 193.3 kips

Summary of Limit States

Type of Limit States	5-A325 bolt
1. Shear yielding of plate	119.8 kips
2. Shear rupture of plate	145.8 kips
3. Block shear of plate	151.2 kips
4. Bearing/tear-out of plate	194.8 kips
5. Flexural yielding of plate	499.2 k-in
6. Shear strength of bolts, N	101.0 kips (106.1)
eccentricity	1.25 in.
Coefficient C	4.69
7. Shear strength of weld (I)	177.6 kips
eccentricity	5 in.
Shear strength of weld (II)	193.3 kips
eccentricity	4 in.

7-A325 Bolt Connection

Configuration:

- 7-A325 3/4-in. bolt
- Plate 3/8 x 4 1/4 x 1'-9 A36 with 1 1/2 in. edge distance and 3 in. bolt spacing.
- E70xx 0.25 in. weld

AISC Nominal Strength:

1. Shear yielding of plate

   $0.6F_yA_g$ = 0.6x35.5x0.375x21 = 167.7 kips

2. Shear rupture of plate

   $0.6F_uA_n$ = 0.6x61x0.375x{21-7(3/4+1/8)} = 204.2 kips

3. Block shear of plate

   Tension rupture = F_uA_n = 61x0.375x{1.5-0.5x(3/4+1/8)} = 24.3 kips

   Shear rupture = $0.6F_uA_n$ = 0.6x61x0.375x{19.5-6.5x(3/4+1/8)} = 189.6 kips ← governing

   Tension yielding = F_yA_g = 35.5x0.375x1.5 = 20.0 kips ← less than tension rupture

   ∴Block shear = 189.6 + 20 = 209.6 kips

4. Bearing/tear-out

   Bearing = $2.4F_ud_bt_p$ = 2.4x61x3/4x0.375 = 41.2 kips

   Tear-out, edge = $1.2F_ut_pL_c$ = 1.2x61x0.375x{1.5-0.5x(3/4+1/16)} = 30.0 kips ← less

   , interior = $1.2F_ut_pL_c$ = 1.2x61x0.375x{3-1x(3/4+1/16)} = 60.0 kips ← more

   ∴Bearing/tear-out = 1x30.0 + 6x41.2 = 277.2 kips

5. Flexural yielding of plate

   $F_yS_x = F_yt_pl^2/6$ = 35.5x0.375x$(21.0)^2$/6 = 978.5 k-in.

6. Shear strength of bolts

   Eccentricity of bolt group = | (n-1) – a | = | (7-1) – 2.75 | = 3.25 in.

   By interpolation, number of bolts effective = 5.925

   ∴Shear strength of bolts = 5.92x(36/0.75)x0.442 = 125.6 kips for type N (148.5 kips for full)

7. Shear strength of welds (I)

Eccentricity of welds = Max(n:a) = 7 in.

By interpolation a = e_w/l = 7/21 = 0.33, C = 2.22

$\therefore$ Shear strength of weld = CC_1Dl = (2.22/0.75)x1x4x21 = 248.6 kips

Shear strength of welds (II)

Eccentricity of welds = (n-1) = 7-1 = 6 in.

By interpolation a = e_w/l = 6/21 = 0.29, C = 2.35

$\therefore$ Shear strength of weld = CC_1Dl = (2.35/0.75)x1x4x21 = 263.4 kips

Summary of Limit States

Type of Limit States	7-A325 bolt
1. Shear yielding of plate	167.7 kips
2. Shear rupture of plate	204.2 kips
3. Block shear of plate	209.6 kips
4. Bearing/tear-out of plate	277.2 kips
5. Flexural yielding of plate	978.5 k-in
6. Shear strength of bolts, N	125.6 kips (148.5)
eccentricity	3.25 in.
Coefficient C	5.92
7. Shear strength of weld (I)	248.6 kips
eccentricity	7 in.
Shear strength of weld (II)	263.4 kips
eccentricity	6 in.

3-A490 Bolt Connection

Configuration:

- 3-A490 3/4-in. bolt
- Plate 3/8 x 3 7/8 x 0'-8 1/4 A36 with 1 1/8 in. edge distance and 3 in. bolt spacing.
- E70xx 7/32- in. weld

AISC Nominal Strength:

1. Shear yielding of plate

   $0.6F_yA_g$ = 0.6x35.5x0.375x8.25 = 65.9 kips
2. Shear rupture of plate

   $0.6F_uA_n$ = 0.6x61x0.375x{8.25-3(3/4+1/8)} = 77.2 kips
3. Block shear of plate

   Tension rupture = F_uA_n = 61x0.375x{1.125-0.5x(3/4+1/8)} = 15.7 kips

   Shear rupture = $0.6F_uA_n$ = 0.6x61x0.375x{7.125-2.5x(3/4+1/8)} = 67.8 kips ← governing

   Tension yielding = F_yA_g = 35.5x0.375x1.125 = 15.0 kips ← less than tension rupture

   ∴Block shear = 67.8 + 15.0 = 82.8 kips
4. Bearing/tear-out

   Bearing = $2.4F_ud_bt_p$ = 2.4x61x3/4x0.375 = 41.2 kips

   Tear-out, edge = $1.2F_ut_pL_c$ = 1.2x61x0.375x{1.125-0.5x(3/4+1/16)} = 19.7 kips ← less

   , interior = $1.2F_ut_pL_c$ = 1.2x61x0.375x{3-1x(3/4+1/16)} = 60.0 kips ← more

   ∴Bearing/tear-out = 1x19.7 + 2x41.2 = 102.1 kips
5. Flexural yielding of plate

   $F_yS_x = F_yt_pl^2/6$ = 35.5x0.375x$(8.25)^2$/6 = 151.0 k-in.
6. Shear strength of bolts

   Eccentricity of bolt group = | (n-1) – a | = | (3-1) – 2.75 | = 0.75 in. (must extrapolate)

   By calculation based on instantaneous center approach, number of bolts effective = 2.81

   ∴Shear strength of bolts = 2.81x(45/0.75)x0.442 = 74.5 kips for type N (79.6 kips for full)

7. Shear strength of welds (I)

Eccentricity of welds = Max(n:a) = 3 in.

By interpolation a = e_w/l = 3/8.25 = 0.36, C = 2.13

$\therefore$ Shear strength of weld = CC_1Dl = (2.13/0.75)x1x3.5x8.25 = 81.9 kips

Shear strength of welds (II)

Eccentricity of welds = (n-1) = 3-1 = 2 in.

By interpolation a = e_w/l = 2/8.25 = 0.24, C = 2.51

$\therefore$ Shear strength of weld = CC_1Dl = (2.51/0.75)x1x3.5x8.25 = 96.7 kips

Summary of limit states

Type of Limit States	3-A490 bolt
1. Shear yielding of plate	65.9 kips
2. Shear rupture of plate	77.2 kips
3. Block shear of plate	82.8 kips
4. Bearing/tear-out of plate	102.1 kips
5. Flexural yielding of plate	151.0 k-in
6. Shear strength of bolts, N	74.5 kips (79.6)
eccentricity	0.75 in.
Coefficient C	2.81
7. Shear strength of weld (I)	81.9 kips
eccentricity	3 in.
Shear strength of weld (II)	96.7 kips
eccentricity	2 in.

5-A490 Bolt Connection

Configuration:

- 5-A490 3/4-in. bolt
- Plate 3/8 x 3 7/8 x 1'-2 1/4 A36 with 1 1/8 in. edge distance and 3 in. bolt spacing.
- E70xx 7/32 in. weld

AISC Nominal Strength:

1. Shear yielding of plate

   $0.6F_yA_g$ = 0.6x35.5x0.375x14.25 = 113.8 kips
2. Shear rupture of plate

   $0.6F_uA_n$ = 0.6x61x0.375x{14.25-5(3/4+1/8)} = 135.5 kips
3. Block shear of plate

   Tension rupture = F_uA_n = 61x0.375x{1.125-0.5x(3/4+1/8)} = 15.7 kips

   Shear rupture = $0.6F_uA_n$ = 0.6x61x0.375x{13.125-4.5x(3/4+1/8)} = 126.1 kips ← governing

   Tension yielding = F_yA_g = 35.5x0.375x1.125 = 15.0 kips ← less than tension rupture

   ∴Block shear = 126.1 + 15.0 = 141.1 kips
4. Bearing/tear-out

   Bearing = $2.4F_ud_bt_p$ = 2.4x61x3/4x0.375 = 41.2 kips

   Tear-out, edge = $1.2F_ut_pL_c$ = 1.2x61x0.375x{1.125-0.5x(3/4+1/16)} = 19.7 kips ← less

   , interior = $1.2F_ut_pL_c$ = 1.2x61x0.375x{3-1x(3/4+1/16)} = 60.0 kips ← more

   ∴Bearing/tear-out = 1x19.7 + 4x41.2 = 184.5 kips
5. Flexural yielding of plate

   $F_yS_x = F_yt_pl^2/6$ = 35.5x0.375x$(14.25)^2$/6 = 450.5 k-in.
6. Shear strength of bolts

   Eccentricity of bolt group = | (n-1) – a | = | (5-1) – 2.75 | = 1.25 in. (must extrapolate)

   By extrapolation, number of bolts effective = 4.69

   ∴Shear strength of bolts = 4.69x(45/0.75)x0.442 = 124.4 kips for type N (132.6 kips for full)

7. Shear strength of welds (I)

   Eccentricity of welds = Max(n:a) = 5 in.

   By interpolation a = e_w/l = 5/14.25 = 0.35, C = 2.16

   $\therefore$ Shear strength of weld = CC_1Dl = (2.16/0.75)x1x3.5x14.25 = 143.6 kips

   Shear strength of welds (II)

   Eccentricity of welds = (n-1) = 5-1 = 4 in.

   By interpolation a = e_w/l = 4/14.25 = 0.28, C = 2.38

   $\therefore$ Shear strength of weld = CC_1Dl = (2.38/0.75)x1x3.5x14.25 = 158.5 kips

Summary of limit states

Type of Limit States	5-A490 bolt
1. Shear yielding of plate	113.8 kips
2. Shear rupture of plate	135.5 kips
3. Block shear of plate	141.1 kips
4. Bearing/tear-out of plate	184.5 kips
5. Flexural yielding of plate	450.5 k-in
6. Shear strength of bolts, N	124.4 kips (132.6)
eccentricity	1.25 in.
Coefficient C	4.69
7. Shear strength of weld (I)	143.6 kips
eccentricity	5 in.
Shear strength of weld (II)	158.5 kips
eccentricity	4 in.

Appendix B

Introduction

Appendix B contains calculations made to verify Sarkar's test results with current LRFD Design Specifications as shown in Table 3-12. Limit states considered in the calculations are the same as previously listed in Appendix A.

Material Properties

Material properties used in the calculations are:

1. For A36 plate, F_y = 47.4 ksi, and F_u = 65 ksi (as reported by Sarkar).
2. For A325 N bolts, F_t = 120 ksi. Thus, F_v = 0.8x0.62x120x0.8 = 48 ksi (as reported by Sarkar, but also same as spec.).
3. For E70xx welding, F_u = 0.6x70/0.75 = 56 ksi (nominal per specification as not indicated in the research).

2-A325 Bolt Connection

Configuration:

- 2-A325 3/4-in diameter bolt
- Plate 3/8 x 5 x 0'-6 A36 with 1 1/2 in. edge distance and 3 in. bolt spacing.
- E70xx 0.25 in. weld

AISC Nominal Strength:

1. Shear yielding of plate

   $0.6F_yA_g$ = 0.6x47.4x0.375x6 = 64.0 kips
2. Shear rupture of plate

   $0.6F_uA_n$ = 0.6x65x0.375x{6-2(3/4+1/8)} = 62.2 kips
3. Block shear of plate

   Tension rupture = F_uA_n = 65x0.375x{1.5-0.5x(3/4+1/8)} = 25.9 kips

   Shear rupture = $0.6F_uA_n$ = 0.6x65x0.375x{4.5-1.5x(3/4+1/8)} = 46.6 kips ← governing

   Tension yielding = F_yA_g = 47.4x0.375x1.5 = 26.7 kips ← more than tension rupture

   ∴Block shear = 46.6 + 25.9 = 72.5 kips
4. Bearing/tear-out

   Bearing = $2.4F_ud_bt_p$ = 2.4x65x3/4x0.375 = 43.9 kips

   Tear-out, edge = $1.2F_ut_pL_c$ = 1.2x65x0.375x{1.5-0.5x(3/4+1/16)} = 32.0 kips ← less

   , interior = $1.2F_ut_pL_c$ = 1.2x65x0.375x{3-1x(3/4+1/16)} = 64.0 kips ← more

   ∴Bearing/tear-out = 1x32.0 + 1x43.9 = 75.9 kips
5. Flexural yielding of plate

   $F_yS_x = F_yt_pl^2/6$ = 47.4x0.375x$(6.0)^2$/6 = 106.7 k-in.
6. Shear strength of bolts

   Eccentricity of bolt group = | (n-1) – a | = | (2-1) – 3.5 | = 2.5 in.

   By interpolation, number of bolts effective = 1.01

   ∴Shear strength of bolts = 1.01x48x0.442 = 21.4 kips for type N (42.4 kips full)

7. Shear strength of welds (I)

Eccentricity of welds = Max(n:a) = 3.5 in.

By interpolation a = e_w/l = 3.5/6 = 0.58, C = 1.54

$\therefore$ Shear strength of weld = CC_1Dl = (1.54/0.75)x1x4x6 = 49.4 kips

Shear strength of welds (II)

Eccentricity of welds = (n-1) = 2-1 = 1 in.

By interpolation a = e_w/l = 1/6 = 0.17, C = 2.71

$\therefore$ Shear strength of weld = CC_1Dl = (2.71/0.75)x1x4x6 = 86.6 kips

Summary of Limit States

Type of Limit States	2-A325 N bolt
1. Shear yielding of plate	64.0 kips
2. Shear rupture of plate	62.2 kips
3. Block shear of plate	72.5 kips
4. Bearing/tear-out of plate	75.9 kips
5. Flexural yielding of plate	106.7 k-in
6. Shear strength of bolts, N	21.4 kips (42.4)
eccentricity	2.5 in.
Coefficient C	1.01
7. Shear strength of weld (I)	49.4 kips
eccentricity	3.5 in.
Shear strength of weld (II)	86.6 kips
eccentricity	1 in.

4-A325 Bolt Connection

Configuration:

- 4-A325 3/4- in. bolt
- Plate 3/8 x 5 x 1' A36 with 1 1/2 in. edge distance and 3 in. bolt spacing.
- E70xx 5/16 in. weld

AISC Nominal Strength:

1. Shear yielding of plate

   $0.6F_yA_g$ = 0.6x47.4x0.375x12 = 128.0 kips
2. Shear rupture of plate

   $0.6F_uA_n$ = 0.6x65x0.375x{12-4(3/4+1/8)} = 124.3 kips
3. Block shear of plate

   Tension rupture = F_uA_n = 65x0.375x{1.5-0.5x(3/4+1/8)} = 25.9 kips

   Shear rupture = $0.6F_uA_n$ = 0.6x65x0.375x{10.5-3.5x(3/4+1/8)} = 108.8 kips ← governing

   Tension yielding = F_yA_g = 47.4x0.375x1.5 = 26.7 kips ← more than tension rupture

   ∴Block shear = 108.8 + 25.9 = 134.7 kips
4. Bearing/tear-out

   Bearing = $2.4F_ud_bt_p$ = 2.4x65x3/4x0.375 = 43.9 kips

   Tear-out, edge = $1.2F_ut_pL_c$ = 1.2x65x0.375x{1.5-0.5x(3/4+1/16)} = 32.0 kips ← less

   , interior = $1.2F_ut_pL_c$ = 1.2x65x0.375x{3-1x(3/4+1/16)} = 64.0 kips ← more

   ∴Bearing/tear-out = 1x32.0 + 3x43.9 = 163.7 kips
5. Flexural yielding of plate

   $F_yS_x = F_yt_pl^2/6 = 47.4x0.375x(12.0)^2/6$ = 426.6 k-in.
6. Shear strength of bolts

   Eccentricity of bolt group = | (n-1) – a | = | (4-1) – 3.5 | = 0.5 in. (must extrapolate)

   By calculation based on instantaneous center approach, number of bolts effective = 3.88

   ∴Shear strength of bolts = 3.88x48x0.442 = 82.3 kips for type N (84.9 kips for full)

7. Shear strength of welds

   Eccentricity of welds = Max(n:a) = 4 in.

   By interpolation a = e_w/l = 4/12 = 0.33, C = 2.22

   $\therefore$ Shear strength of weld = CC_1Dl = (2.22/0.75)x1x5x12 = 177.6 kips

   Shear strength of welds (II)

   Eccentricity of welds = (n-1) = 4-1 = 3 in.

   By interpolation a = e_w/l = 3/12 = 0.25, C = 2.48

   $\therefore$ Shear strength of weld = CC_1Dl = (2.48/0.75)x1x5x12 = 198.4 kips

Summary of Limit States

Type of Limit States	4-A325 N bolt
1. Shear yielding of plate	128.0 kips
2. Shear rupture of plate	124.3 kips
3. Block shear of plate	134.7 kips
4. Bearing/tear-out of plate	163.7 kips
5. Flexural yielding of plate	426.6 k-in
6. Shear strength of bolts, N	82.3 kips (84.9)
eccentricity	0.5 in.
Coefficient C	3.88
7. Shear strength of weld (I)	177.6 kips
eccentricity	4 in.
Shear strength of weld (II)	198.4 kips
eccentricity	3 in.

6-A325 Bolt Connection

Configuration:

- 6-A325 3/4-in. bolt
- Plate 3/8 x 5 x 1'-6 A36 with 1 1/2 in. edge distance and 3 in. bolt spacing.
- E70xx 5/16 in. weld

AISC Nominal Strength:

1. Shear yielding of plate

   $0.6F_yA_g$ = 0.6x47.4x0.375x18 = 192.0 kips
2. Shear rupture of plate

   $0.6F_uA_n$ = 0.6x65x0.375x{18-6(3/4+1/8)} = 186.5 kips
3. Block shear of plate

   Tension rupture = F_uA_n = 65x0.375x{1.5-0.5x(3/4+1/8)} = 25.9 kips

   Shear rupture = $0.6F_uA_n$ = 0.6x65x0.375x{16.5-5.5x(3/4+1/8)} = 170.9 kips ← governing

   Tension yielding = F_yA_g = 47.4x0.375x1.5 = 26.7 kips ← less than tension rupture

   ∴ Block shear = 170.9 + 25.9 = 196.8 kips
4. Bearing/tear-out

   Bearing = $2.4F_ud_bt_p$ = 2.4x65x3/4x0.375 = 43.9 kips

   Tear-out, edge = $1.2F_ut_pL_c$ = 1.2x65x0.375x{1.5-0.5x(3/4+1/16)} = 32.0 kips ← less

   , interior = $1.2F_ut_pL_c$ = 1.2x65x0.375x{3-1x(3/4+1/16)} = 64.0 kips ← more

   ∴ Bearing/tear-out = 1x32.0 + 5x43.9 = 251.5 kips
5. Flexural yielding of plate

   $F_yS_x = F_yt_pl^2/6$ = 47.4x0.375x$(18.0)^2$/6 = 959.9 k-in.
6. Shear strength of bolts

   Eccentricity of bolt group = | (n-1) – a | = | (6-1) – 3.5 | = 1.5 in.

   By interpolation, number of bolts effective = 5.63

   ∴ Shear strength of bolts = 5.63x48x0.442 = 119.4 kips for type N (127.3 kips for full)

7. Shear strength of welds

Eccentricity of welds = Max(n:a) = 6 in.

By interpolation a = e_w/l = 6/18 = 0.33, C = 2.22

$\therefore$ Shear strength of weld = CC_1Dl = (2.22/0.75)x1x5x18 = 266.4 kips

Shear strength of welds (II)

Eccentricity of welds = (n-1) = 6-1 = 5 in.

By interpolation a = e_w/l = 5/18 = 0.28, C = 2.38

$\therefore$ Shear strength of weld = CC_1Dl = (2.38/0.75)x1x5x18 = 286.1 kips

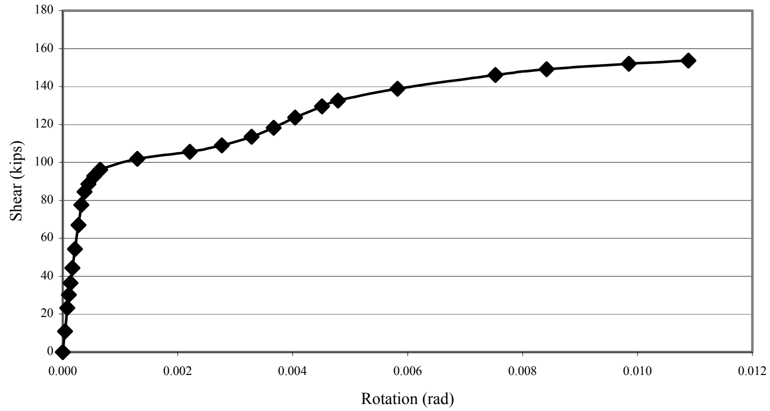
Summary of Limit States

Type of Limit States	6-A325 N bolt
1. Shear yielding of plate	192.0 kips
2. Shear rupture of plate	186.5 kips
3. Block shear of plate	196.8 kips
4. Bearing/tear-out of plate	251.5 kips
5. Flexural yielding of plate	959.9 k-in
6. Shear strength of bolts, N	119.4 kips (127.3)
eccentricity	1.5 in.
Coefficient C	5.63
7. Shear strength of weld (I)	266.4 kips
eccentricity	6 in.
Shear strength of weld (II)	286.1 kips
eccentricity	5 in.

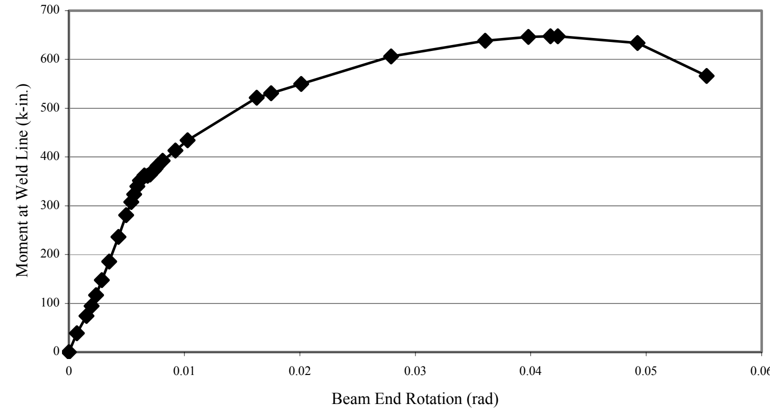
Appendix C

Introduction

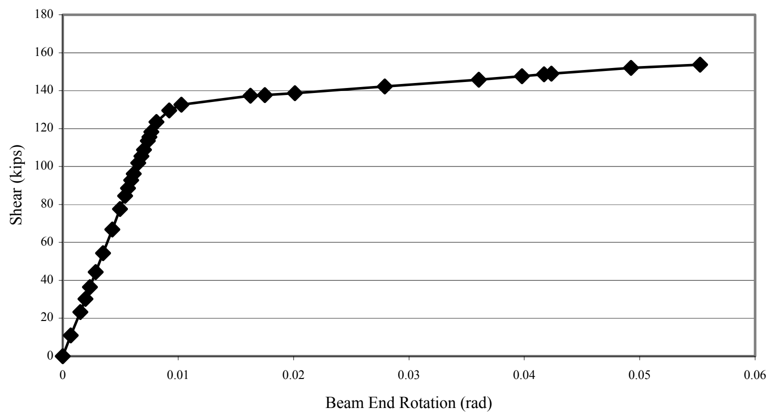
Appendix C contains the results of the simulations, except for Models 1 and 11, which are shown in Chapter V. The results are plots of shear force vs. rotation at the bolt line, moment at weld line vs. beam end rotation, shear vs. beam end rotation, and shear vs. distance to point of inflection from weld line, and a table that illustrates the shear stress in the bolts. Starting with Model 13, the results include a plot of bolt movement throughout the simulation.



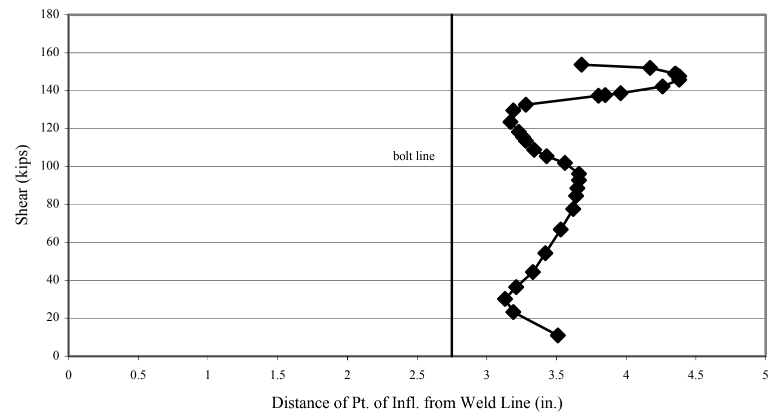
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C1 Results from Model 2

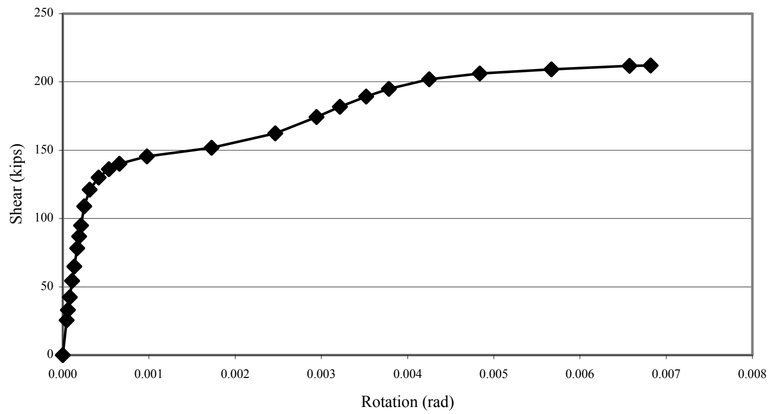
Table C1 Shear Stress in Bolts of Model 2

Increment		92	94	96	98	100	102	104	106	108	110	112	114	116	118	120
Load		130.7	131.6	132.7	133.9	134.2	134.9	135.3	135.7	136.2	136.4	137.0	137.3	137.7	138.4	140.1
1st bolt	beam side	55.26	55.65	55.63	55.44	55.34	55.10	54.87	54.51	54.02	53.65	52.8	52.29	51.78	51.04	50.39
	plate side	55.96	56.08	56.11	56.02	55.96	55.84	55.73	55.57	55.33	55.15	54.72	54.48	54.17	53.71	52.65
2nd bolt	beam side	55.32	55.64	56.00	56.42	56.59	56.87	57.01	57.04	57.06	57.04	56.88	56.74	56.53	55.92	54.69
	plate side	55.91	56.27	56.74	56.98	57.05	57.13	57.19	57.29	57.40	57.43	57.47	57.47	57.54	57.68	57.72
3rd bolt	beam side	54.58	54.98	55.55	56.20	56.41	56.83	57.09	57.51	57.92	58.06	58.36	58.52	58.66	58.63	58.33
	plate side	55.27	55.66	56.21	56.94	57.21	57.6	57.76	57.97	58.20	58.36	58.73	58.96	59.23	59.63	60.19
4th bolt	beam side	53.72	54.16	54.56	54.99	55.18	55.59	55.68	55.71	55.72	55.70	55.70	55.64	55.4	54.92	54.14
	plate side	54.43	54.81	55.26	55.84	55.89	55.98	56.04	56.09	56.15	56.21	56.51	56.67	56.75	56.95	57.40
5th bolt	beam side	52.57	52.99	53.34	53.38	53.39	53.46	53.41	53.13	52.77	52.52	52.01	51.73	51.34	50.83	50.53
	plate side	53.27	53.69	54.03	54.13	54.03	53.85	53.77	53.57	53.20	53.00	52.64	52.47	52.24	52.10	52.42

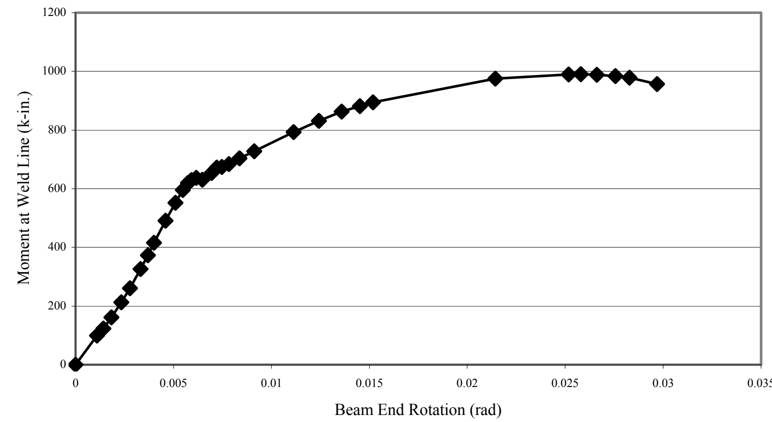
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

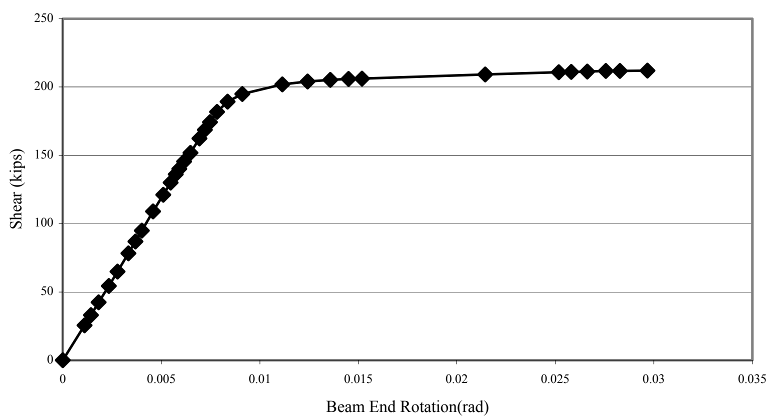
Indicates that the stress in outer element exceeds the stress in the innermost element



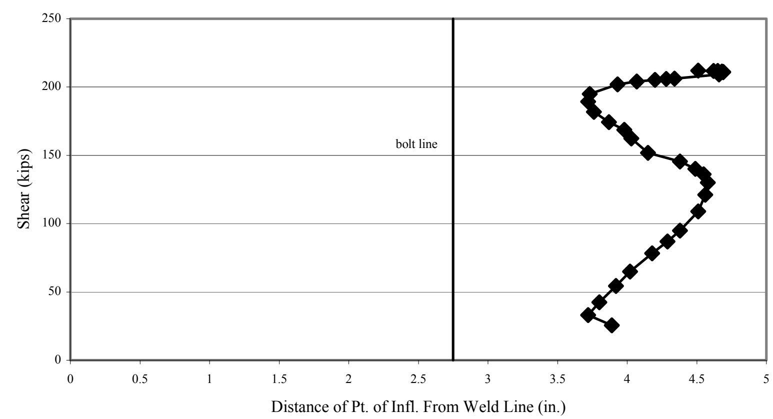
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C2 Results from Model 3

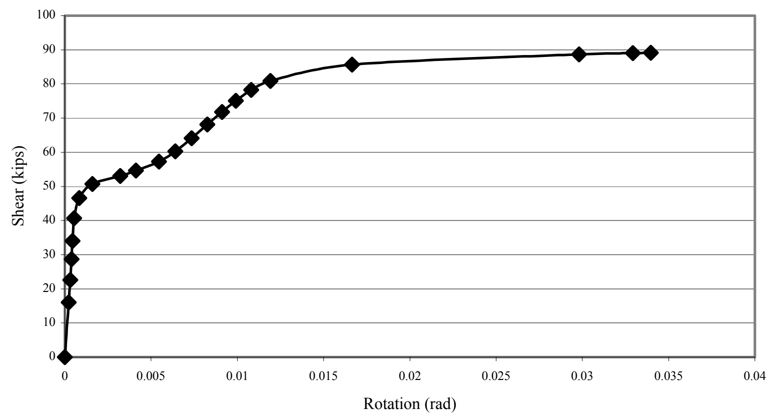
Table C2 Shear Stress in Bolts of Model 3

Increment		70	80	90	92	94	96	98	100	102	104	106	108	110	112	114
Load		164.6	177.5	185.4	188.0	188.8	189.6	190.0	191.1	193.4	194.7	197.6	199.2	201.2	203.3	204.5
1st bolt	beam side	50.16	52.83	54.46	54.72	54.73	54.74	54.74	54.73	54.58	54.39	53.77	53.34	52.55	51.31	50.35
	plate side	50.64	53.35	55.37	56.08	56.3	56.52	56.63	56.89	57.43	57.67	57.99	58.03	57.77	57.19	56.64
2nd bolt	beam side	51.88	54.47	55.57	55.85	55.91	56.00	56.03	56.10	56.14	56.12	55.85	55.45	54.77	53.81	52.94
	plate side	51.97	54.80	56.23	56.67	56.8	56.94	57.02	57.25	57.82	58.18	58.95	59.22	59.35	59.05	58.67
3rd bolt	beam side	51.86	54.71	56.21	56.59	56.72	56.84	56.90	57.03	57.24	57.32	57.32	57.23	56.85	56.12	55.35
	plate side	51.99	55.15	56.92	57.48	57.69	57.88	57.98	58.22	58.74	59.07	59.92	60.36	60.87	61.18	61.11
4th bolt	beam side	50.97	54.46	55.97	56.34	56.47	56.59	56.67	56.85	57.20	57.31	57.40	57.35	57.08	56.61	56.10
	plate side	51.14	54.95	56.70	57.24	57.44	57.63	57.76	58.06	58.70	59.04	59.81	60.31	60.92	61.50	61.73
5th bolt	beam side	49.82	53.34	54.95	55.27	55.38	55.50	55.55	55.67	55.89	55.96	55.94	55.83	55.53	54.91	54.24
	plate side	50.01	53.83	55.64	56.09	56.26	56.43	56.52	56.73	57.22	57.49	58.16	58.63	59.23	59.83	59.94
6th bolt	beam side	48.15	51.85	53.11	53.54	53.67	53.79	53.85	53.96	54.13	54.21	54.05	53.92	53.43	52.57	51.83
	plate side	48.34	52.22	53.37	54.18	54.35	54.51	54.59	54.77	55.16	55.40	55.98	56.40	56.89	57.03	56.94
7th bolt	beam side	46.79	49.83	51.52	51.97	52.08	52.20	52.28	52.44	52.82	52.98	53.19	53.19	52.96	52.53	51.88
	plate side	46.96	50.07	52.04	52.45	52.59	52.73	52.82	53.01	53.45	53.67	54.07	54.30	54.71	55.09	54.89

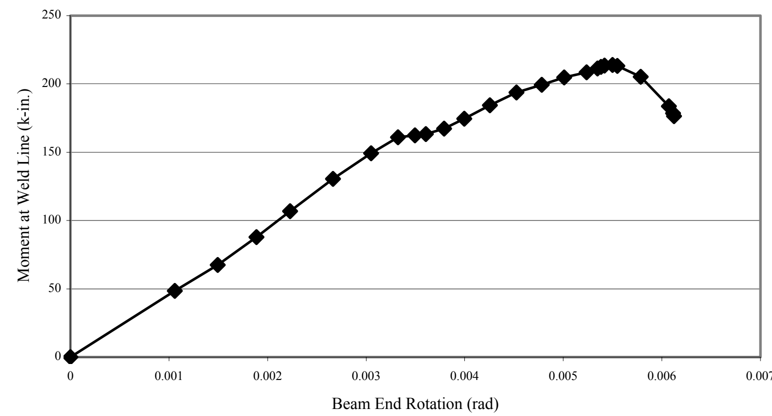
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

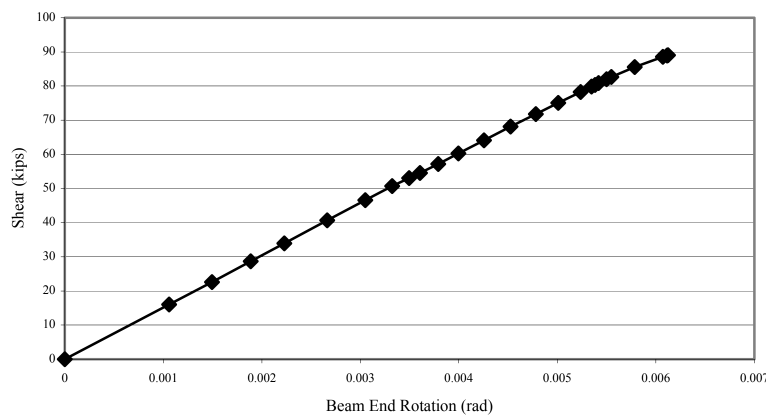
Indicates that the stress in outer element exceeds the stress in the innermost element



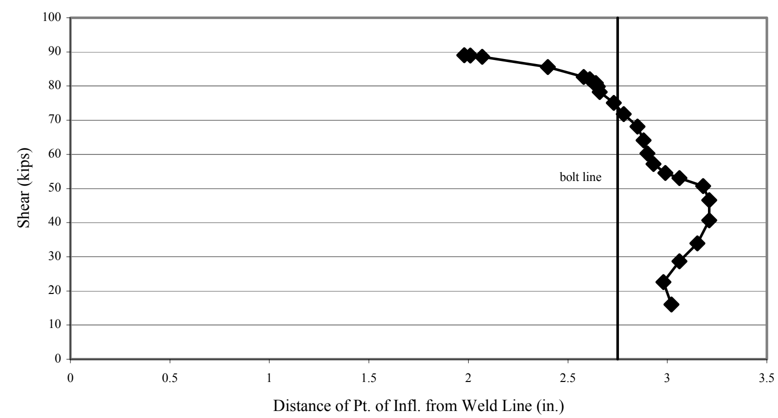
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C3 Results from Model 4

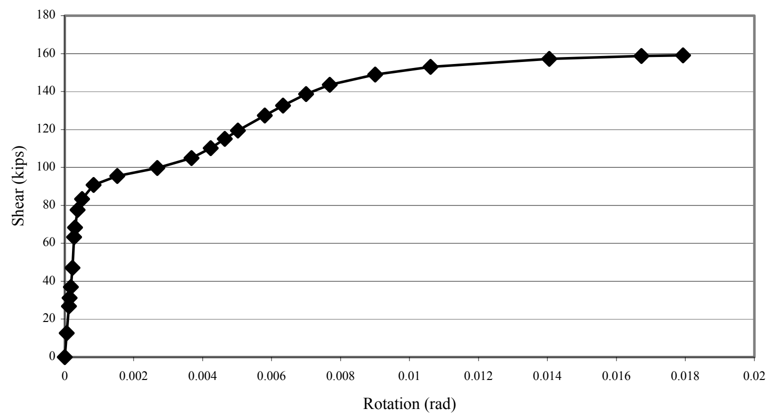
Table C3 Shear Stress in Bolts of Model 4

Increment		82	84	86	88	90	92	94	96	98	100	102	104	106	108	110
Load		80.36	80.86	82.00*	82.64	83.00	83.20	83.66	84.68	85.26	85.58	86.31	86.72	87.23	87.51	87.81
1st bolt	beam side	59.62	59.97	60.89	61.41	61.72	61.89	62.30	63.38	64.10	64.58	66.02	66.78	67.99	68.6	69.36
	plate side	60.47	60.88	61.93	62.55	62.91	63.11	63.61	64.94	65.88	66.53	68.55	69.75	71.69	72.89	74.12
2nd bolt	beam side	58.98	59.39	60.12	60.58	60.84	61.00	61.34	62.16	62.74	63.12	64.35	65.2	66.39	67.15	68.00
	plate side	59.20	59.60	60.48	61.02	61.32	61.51	61.93	62.93	63.68	64.18	65.84	67.00	68.80	70.11	71.55
3rd bolt	beam side	50.59	50.79	51.15	51.32	51.4	51.43	51.52	51.61	51.57	51.52	51.34	51.23	51.07	50.94	50.78
	plate side	50.60	50.79	51.14	51.33	51.41	51.44	51.54	51.64	51.61	51.55	51.38	51.30	51.21	51.12	51.04

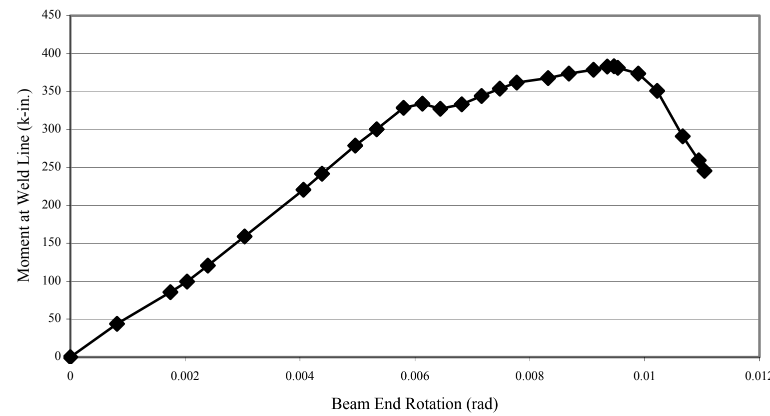
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

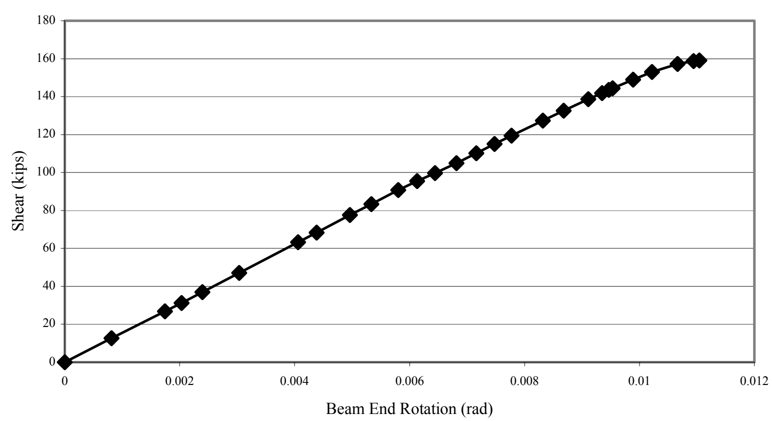
Indicates that the stress in outer element exceeds the stress in the innermost element



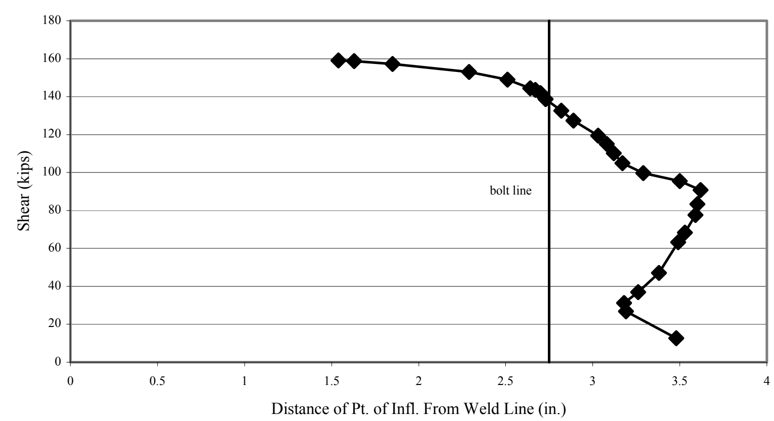
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C4 Results from Model 5

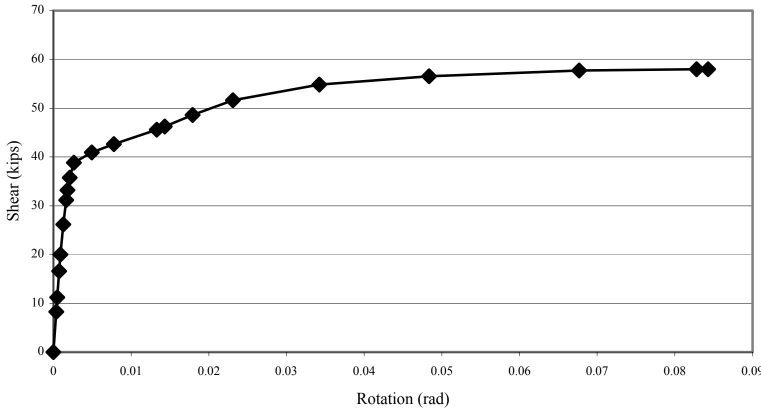
Table C4 Shear Stress in Bolts of Model 5

Increment		100	120	130	140	150	152	154	156	158	160	162	164	166	168	170
Load		143.5	153.0	155.4	157.3	158.3	158.5	158.6	158.6	158.7	158.8	158.8	158.9	158.9	158.9	159.0
1st bolt	beam side	62.52	67.89	70.32	72.70	74.54	74.57	74.57	74.57	74.57	74.52	74.50	74.46	74.45	74.42	74.40
	plate side	63.68	70.65	74.32	76.60	77.83	78.01	78.14	78.22	78.31	78.47	78.53	78.67	78.73	78.85	78.95
2nd bolt	beam side	62.78	68.39	70.90	73.41	74.28	74.26	74.24	74.22	74.19	74.14	74.12	74.06	74.04	73.99	73.96
	plate side	63.28	70.41	74.22	76.49	77.51	77.74	77.92	78.01	78.10	78.32	78.40	78.49	78.51	78.50	78.48
3rd bolt	beam side	61.99	67.23	69.94	72.33	73.38	73.41	73.42	73.42	73.42	73.41	73.41	73.39	73.38	73.37	73.35
	plate side	62.31	69.10	73.4	75.71	77.00	77.29	77.49	77.59	77.71	77.93	78.01	78.17	78.20	78.26	78.27
4th bolt	beam side	59.47	64.95	67.34	69.99	71.55	71.74	71.74	71.74	71.74	71.75	71.76	71.77	71.78	71.79	71.80
	plate side	59.77	66.64	70.4	73.81	75.21	75.51	75.63	75.71	75.79	75.96	76.01	76.14	76.19	76.28	76.37
5th bolt	beam side	52.10	53.03	53.17	53.42	53.65	53.70	53.74	53.76	53.78	53.86	53.83	53.86	53.86	53.88	53.91
	plate side	52.11	53.05	53.28	53.69	54.08	54.16	54.23	54.27	54.31	54.38	54.40	54.46	54.47	54.51	54.56

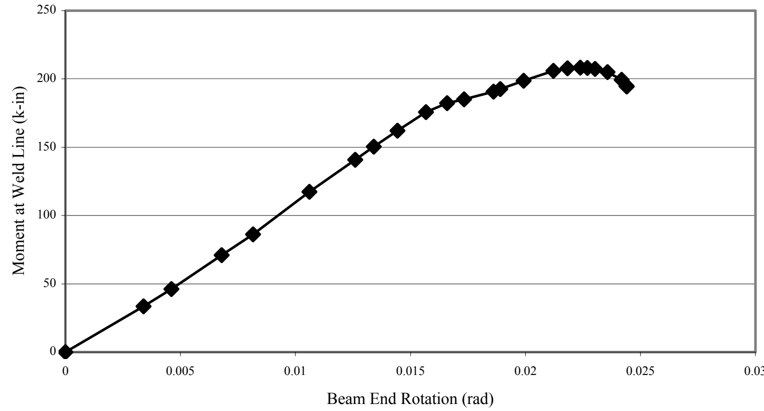
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

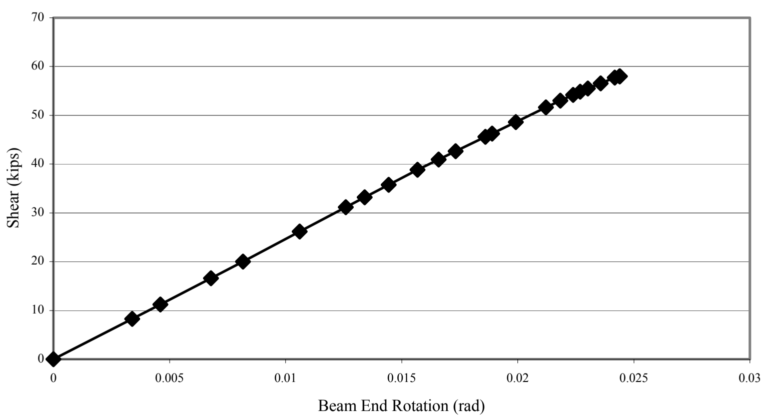
Indicates that the stress in outer element exceeds the stress in the innermost element



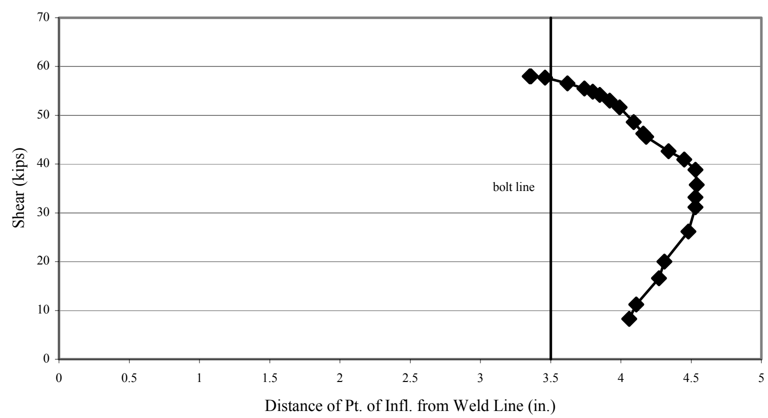
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C5 Results from Model 6

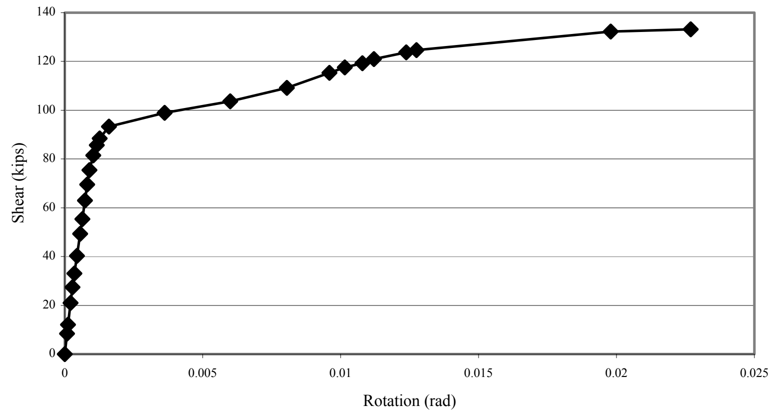
Table C5 Shear Stress in Bolts of Model 6

Increment		80	82	84	86	88	90	92	94	96	98
Load		57.17	57.52	57.71	$\boxed{57.88}$	57.94	58.00	58.00	58.01	58.01	58.01
1st bolt	beam side	56.25	57.29	58.13	59.02	59.25	60.62	60.69	60.94	61.09	61.17
	plate side	60.07	61.31	62.21	$\boxed{62.99}$	$\boxed{63.36}$	63.89	63.90	63.93	63.94	63.95
2nd bolt	beam side	62.48	63.27	63.47	63.87	64.02	65.10	65.14	65.25	65.32	65.36
	plate side	61.44	61.61	61.64	61.70	61.69	61.79	61.79	61.81	61.81	61.82

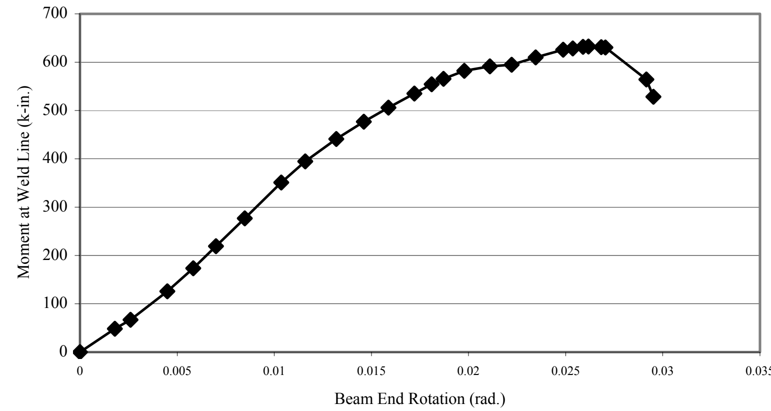
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

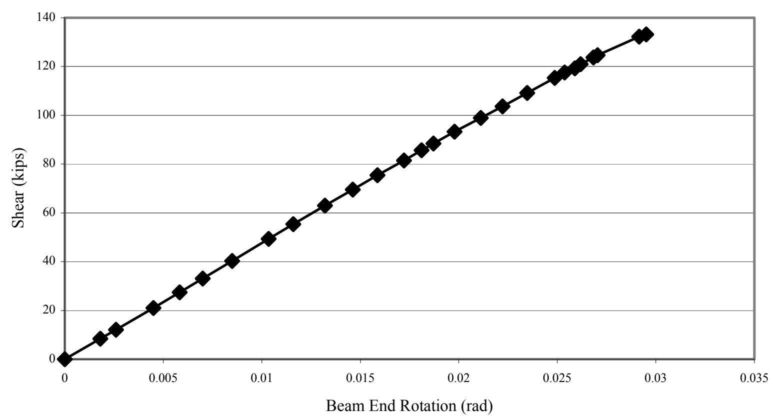
Indicates that the stress in outer element exceeds
the stress in the innermost element



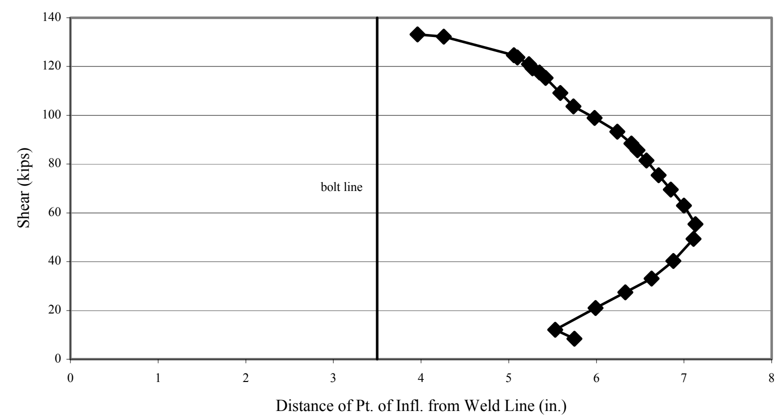
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C6 Results from Model 7

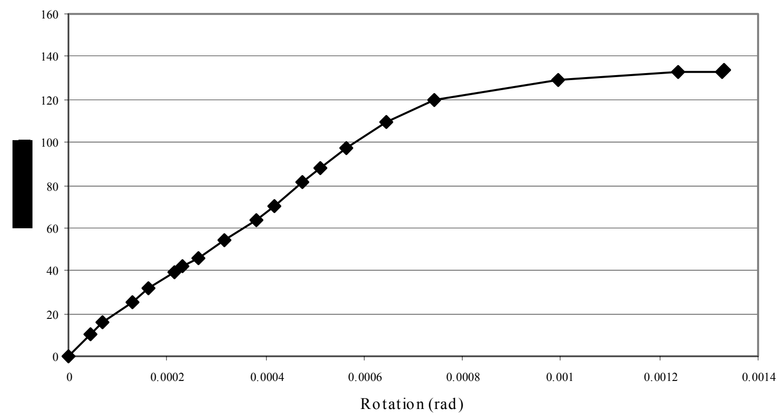
Table C6 Shear Stress in Bolts of Model 7

Increment		80	82	84	86	88	90	92	94	96	98	100	102	104	106	108	110
Load		105.6	106.7	109.1	110.5	113.6	115.4	117.5	119.7	121.0	123.8	124.6	125.5	126.6	127.7	129.1	129.9
1st bolt	beam side	53.47	53.89	54.69	55.12	56.06	56.74	57.60	58.36	58.71	59.16	59.36	59.57	59.63	59.78	59.95	60.00
	plate side	52.10	52.55	53.49	54.16	55.83	57.00	58.40	59.82	60.59	62.07	62.49	63.08	63.85	64.51	65.28	65.69
2nd bolt	beam side	55.37	55.97	57.05	57.64	59.10	59.58	60.05	60.24	60.27	60.08	60.04	59.95	59.85	59.82	59.89	59.97
	plate side	55.57	56.14	57.21	57.82	59.52	60.33	61.37	62.31	62.81	63.99	64.32	64.64	65.03	65.43	66.16	66.57
3rd bolt	beam side	57.21	57.62	58.95	59.51	60.68	61.14	61.70	62.27	62.61	62.89	62.87	62.74	62.49	62.30	61.65	61.25
	plate side	57.59	57.99	59.31	60.01	61.36	62.07	62.79	63.66	64.23	65.63	66.03	66.45	66.90	67.39	67.74	67.84
4th bolt	beam side	56.08	56.69	58.10	58.67	59.51	59.95	60.46	61.11	61.41	61.99	62.20	62.30	62.38	62.39	62.28	62.10
	plate side	56.55	57.21	58.73	59.41	60.45	61.24	62.01	62.87	63.34	64.43	64.84	65.27	65.77	66.30	67.03	67.41

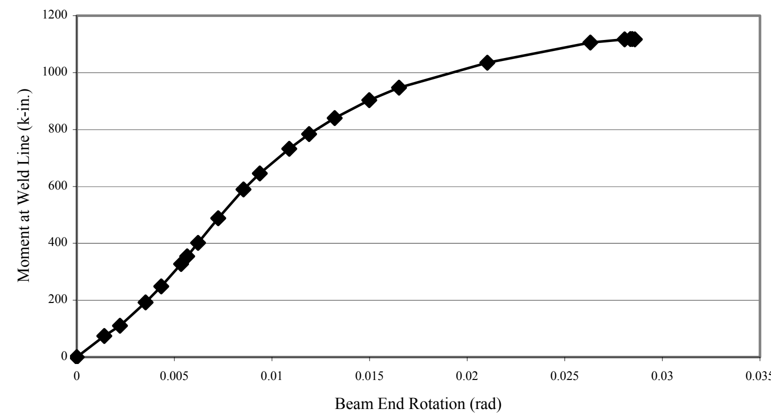
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

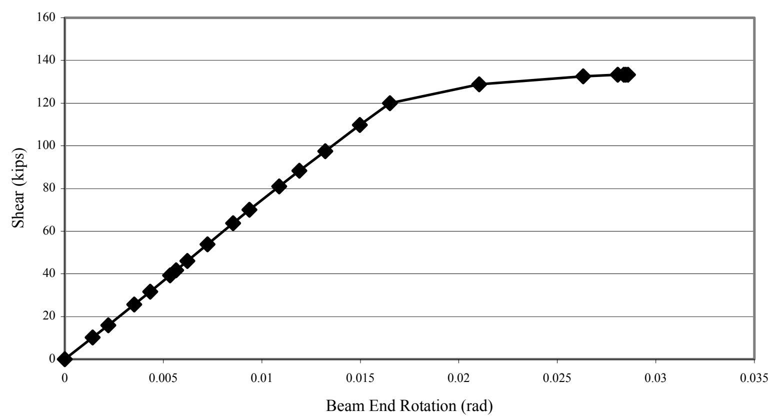
Indicates that the stress in outer element exceeds
the stress in the innermost element



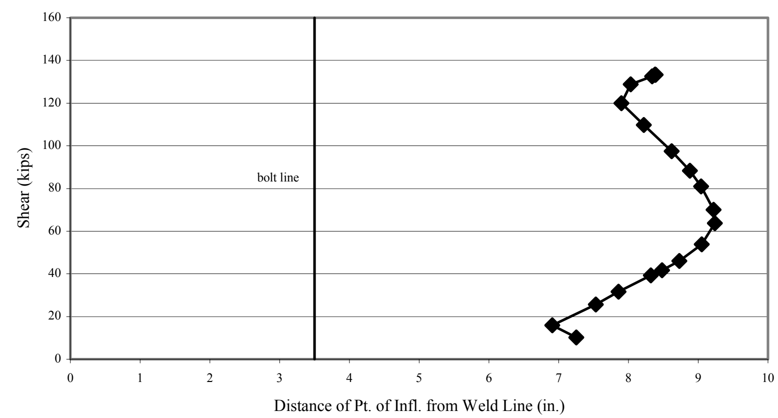
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C7 Results from Model 8

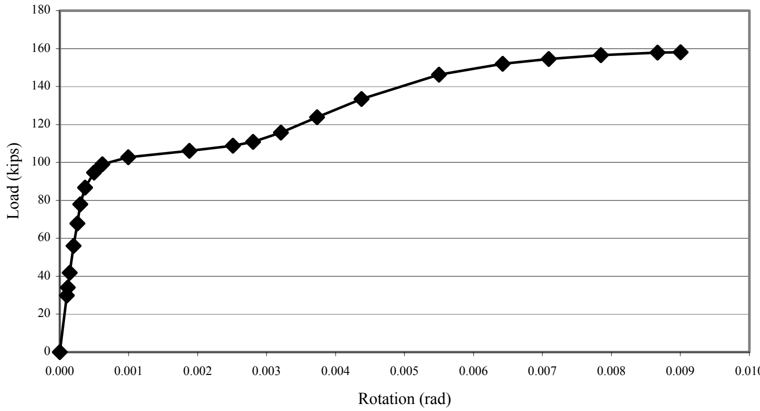
Table C7 Shear Stress in Bolts of Model 8

Increment		60	62	64	66	68	70	72	74	76	78	80	82	84	86	88
Load		119.9	125.6	127.4	127.8	128.8	130.0	130.5	131.8	132.5	132.8	133.2	133.2	133.3	133.3	133.4
1st bolt	beam side	39.60	40.14	39.83	39.67	39.17	38.33	37.78	36.48	35.93	35.75	35.69	35.75	35.94	36.02	36.17
	plate side	37.60	39.16	39.29	39.27	38.73	37.83	37.32	35.67	34.54	34.19	33.76	33.69	33.74	33.80	33.93
2nd bolt	beam side	41.77	43.15	43.26	43.25	43.31	43.43	43.60	43.76	43.92	43.94	43.84	43.80	43.73	43.69	43.62
	plate side	40.90	41.67	41.47	41.39	41.34	41.54	41.68	42.03	42.88	43.05	43.11	43.13	43.09	43.06	43.00
3rd bolt	beam side	47.01	49.17	49.83	49.99	50.47	51.14	51.34	51.26	51.25	51.25	51.22	51.22	51.19	51.17	51.13
	plate side	47.52	49.94	50.52	50.62	51.09	51.77	51.82	51.67	51.48	51.43	51.35	51.34	51.29	51.26	51.21
4th bolt	beam side	45.56	49.95	51.62	51.99	52.95	53.97	54.55	55.74	56.32	56.57	56.84	56.93	57.04	57.08	57.13
	plate side	46.09	50.44	52.68	53.12	54.19	55.26	55.88	57.20	57.85	58.13	58.55	58.69	58.82	58.86	58.91
5th bolt	beam side	42.73	44.38	44.87	45.00	45.29	45.99	46.49	47.67	48.35	48.65	49.03	49.12	49.24	49.28	49.35
	plate side	43.30	44.41	44.69	44.7	44.74	45.06	45.44	46.69	47.71	48.19	48.67	48.80	48.98	49.04	49.13
6th bolt	beam side	39.92	41.64	42.28	42.46	42.72	42.84	42.86	42.95	43.07	43.12	43.06	43.01	42.98	42.98	42.98
	plate side	37.38	39.23	40.11	40.38	40.65	40.99	41.14	41.49	41.61	41.69	41.63	41.58	41.55	41.55	41.55

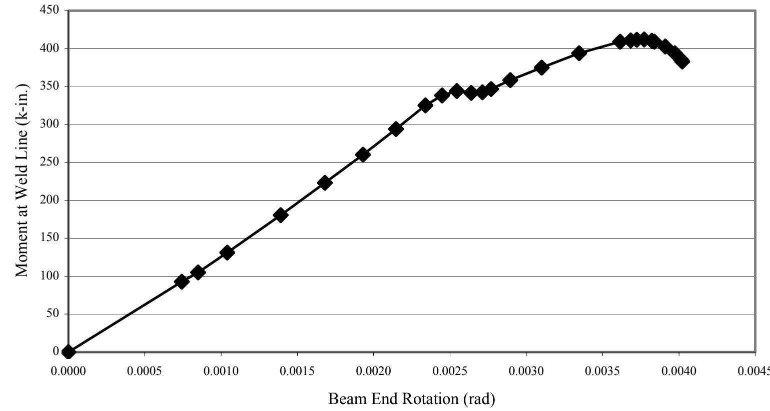
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

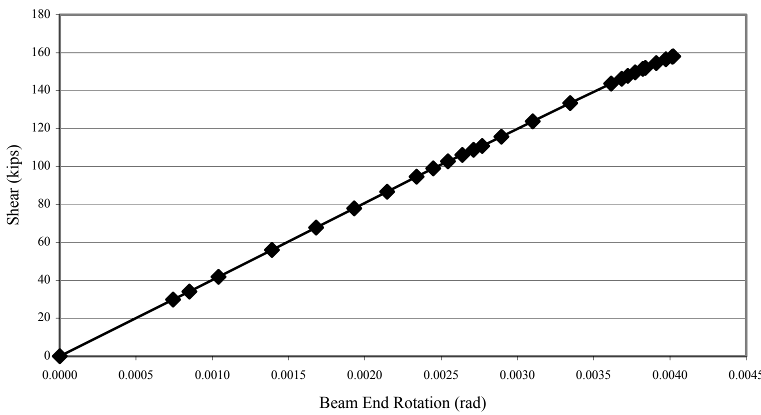
Indicates that the stress in outer element exceeds
the stress in the innermost element



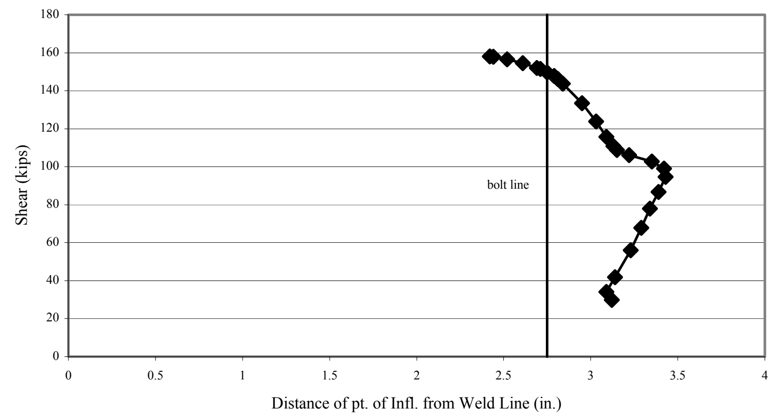
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C8 Results from Model 9

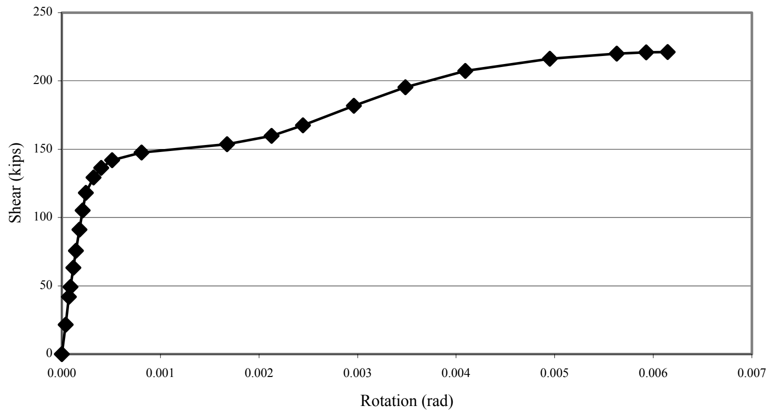
Table C8 Shear Stress in Bolts of Model 9

Increment		62	64	66	68	70	72	74	76	78	80
Load		136.0	140.3	141.6	143.0	146.1	147.8	148.8	151.0	152.2	152.9
1st bolt	beam side	55.24	55.42	55.42	55.35	54.88	54.47	54.22	53.42	52.92	52.64
	plate side	58.18	59.77	60.32	60.86	61.77	62.08	62.19	62.39	62.61	62.59
2nd bolt	beam side	55.42	55.90	55.98	55.95	55.70	55.41	55.21	54.40	53.94	53.74
	plate side	57.63	59.07	59.62	60.21	61.35	61.80	61.98	62.22	62.36	62.51
3rd bolt	beam side	55.21	55.72	55.83	55.88	55.79	55.52	55.34	54.66	54.3	54.07
	plate side	57.18	58.44	58.85	59.39	60.77	61.43	61.69	62.03	62.17	62.27
4th bolt	beam side	54.63	55.27	55.48	55.59	55.52	55.31	55.16	54.63	54.17	53.91
	plate side	56.59	57.84	58.37	58.91	60.04	60.37	61.03	61.56	61.72	61.80
5th bolt	beam side	54.35	55.03	55.3	55.55	55.98	56.04	56.12	56.10	56.06	56.00
	plate side	55.95	56.95	57.39	57.80	58.78	59.36	59.84	60.82	61.37	61.67

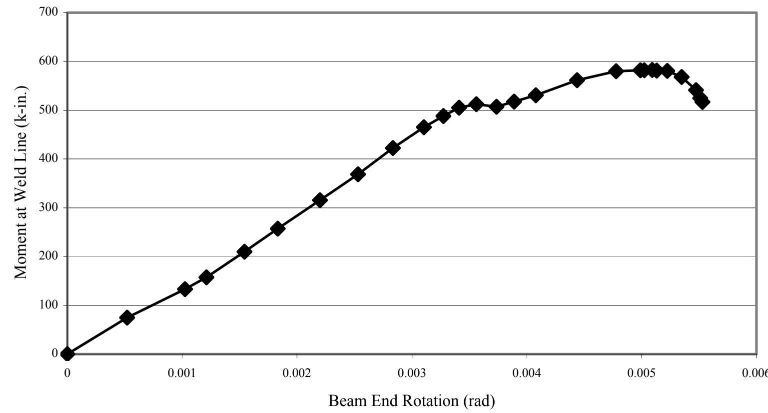
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

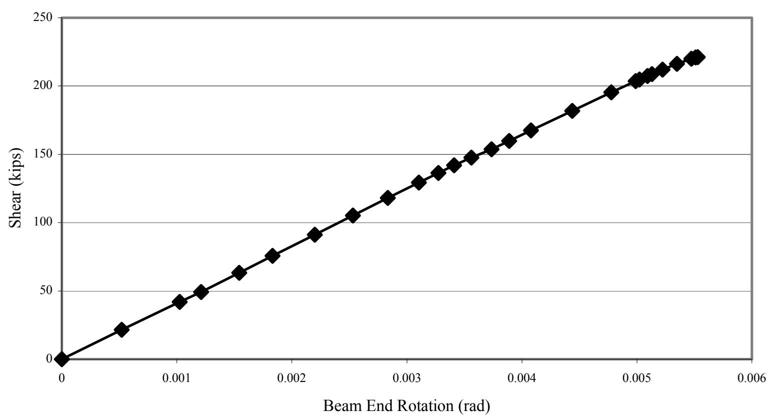
Indicates that the stress in outer element exceeds the stress in the innermost element



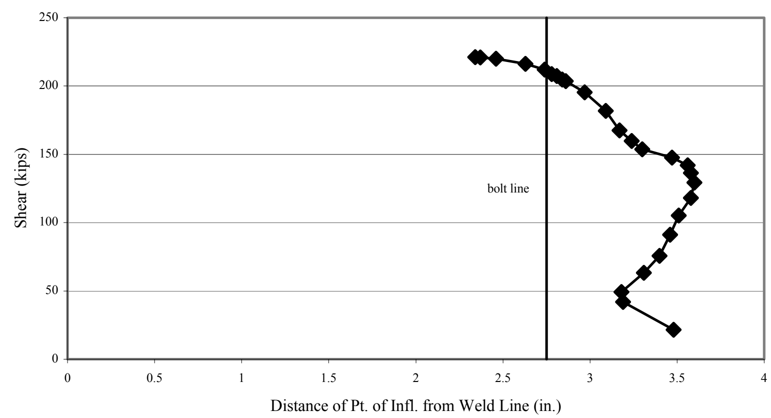
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C9 Results from Model 10

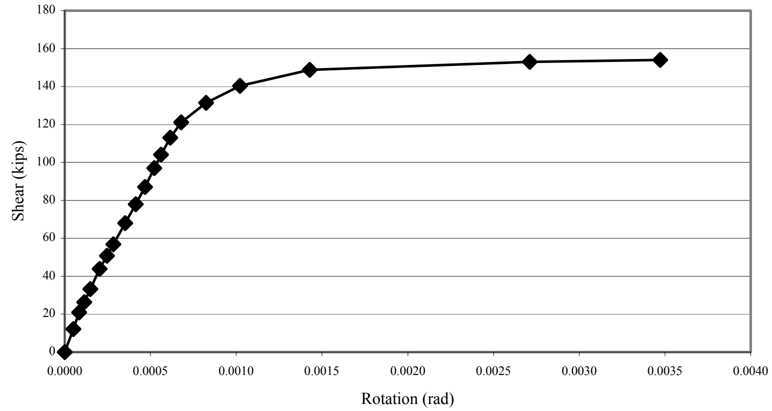
Table C9 Shear Stress in Bolts of Model 10

Increment		72	74	76	78	80	82	84	86	88	90
Load		186.9	189.8	193.4	197.0	201.6	205.3	207.9	210.5	212.0	212.8
1st bolt	beam side	54.57	54.77	54.99	55.06	54.98	54.31	53.66	52.92	52.60	52.42
	plate side	57.12	57.79	58.76	59.88	61.08	61.6	61.75	61.91	62.10	62.19
2nd bolt	beam side	54.88	55.32	55.71	55.83	55.73	55.3	54.83	54.27	53.87	53.64
	plate side	56.87	57.57	58.49	59.45	60.73	61.54	61.85	61.99	62.10	62.20
3rd bolt	beam side	54.89	55.26	55.76	55.97	55.97	55.61	55.23	54.6	54.22	54.01
	plate side	56.68	57.21	58.04	59.06	60.47	61.46	61.90	62.13	62.23	62.30
4th bolt	beam side	54.68	55.03	55.54	55.74	55.69	55.38	55.10	54.49	54.19	53.97
	plate side	56.53	57.11	57.96	58.87	60.12	61.11	61.6	61.89	61.99	62.05
5th bolt	beam side	54.16	54.48	54.88	55.13	55.16	55.02	54.77	54.22	53.89	53.70
	plate side	56.10	56.67	57.42	58.33	59.54	60.55	61.10	61.38	61.50	61.55
6th bolt	beam side	53.27	53.72	54.18	54.48	54.54	54.46	54.35	53.94	53.66	53.44
	plate side	55.01	55.60	56.38	57.29	58.42	59.54	60.16	60.66	60.86	60.91
7th bolt	beam side	52.8	53.18	53.59	54.00	54.49	54.87	55.10	55.36	55.45	55.47
	plate side	54.34	54.96	55.55	56.16	57.01	57.93	58.65	59.48	59.96	60.25

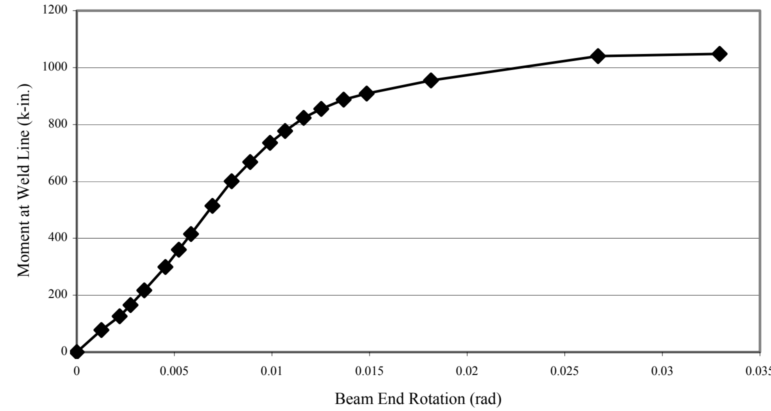
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

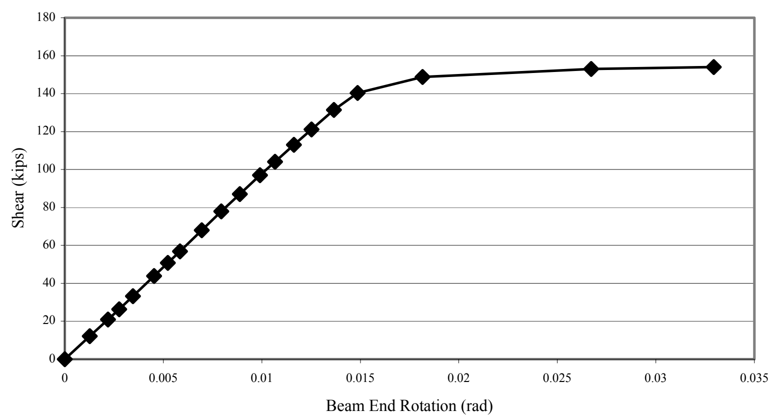
Indicates that the stress in outer element exceeds the stress in the innermost element



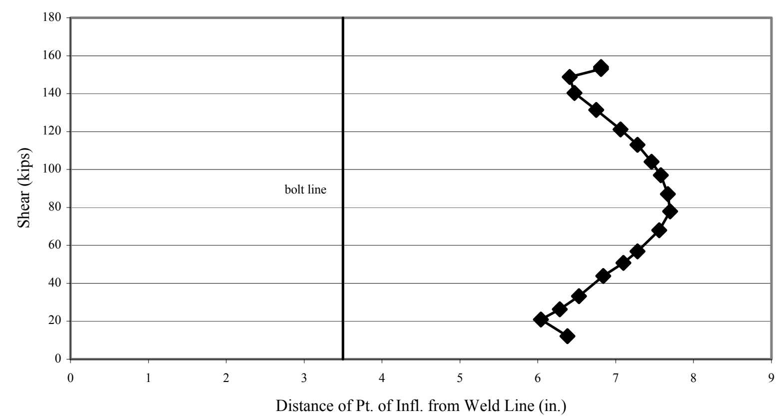
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C10 Results from Model 12

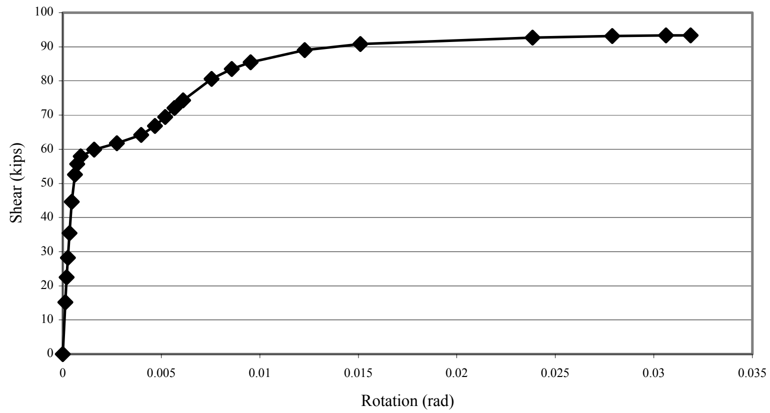
Table C10 Shear Stress in Bolts of Model 12

Increment		42	44	46	48	50	52	54	56	58	60	62	64	66	68	70	72
Load		95.01	97.03	101.6	104.1	109.9	113.1	117.1	121.2	126.2	131.4	133	134.6	138.3	140.4	142.9	145.5
1st bolt	beam side	28.95	29.44	30.64	31.31	32.79	33.60	34.48	35.30	36.23	36.95	37.23	37.47	37.95	38.08	38.00	37.47
	plate side	28.75	29.23	30.56	31.39	33.35	34.45	35.54	36.50	37.76	38.84	39.15	39.45	40.14	40.42	40.62	40.29
2nd bolt	beam side	34.86	35.04	35.52	35.83	36.38	36.61	37.09	37.77	38.65	39.32	39.56	39.79	40.22	40.53	40.57	40.61
	plate side	35.54	35.96	36.24	36.43	37.07	37.66	38.49	39.30	40.67	41.87	42.31	42.70	43.36	43.61	43.85	44.19
3rd bolt	beam side	33.35	34.42	36.59	38.01	40.57	41.58	42.34	42.71	42.97	43.36	43.52	43.67	43.74	43.55	43.19	42.66
	plate side	33.96	34.92	37.01	38.24	41.29	42.64	43.62	44.22	44.96	46.20	46.80	47.58	49.17	50.00	50.68	50.99
4th bolt	beam side	32.17	33.24	35.64	36.95	39.54	40.67	42.09	43.17	43.58	44.26	44.37	44.50	44.73	44.77	44.53	43.88
	plate side	33.06	34.06	36.36	37.60	40.19	41.44	43.03	44.29	45.21	47.05	47.78	48.59	50.19	50.98	51.99	52.75
5th bolt	beam side	34.21	34.84	36.08	36.36	37.02	37.61	38.49	39.15	40.19	40.86	41.02	41.34	42.00	42.14	42.19	42.12
	plate side	35.17	35.78	36.98	37.47	38.12	38.60	39.65	40.80	42.50	43.69	44.02	44.60	45.77	46.20	46.83	47.27
6th bolt	beam side	28.72	29.23	30.55	31.27	32.90	33.78	34.67	35.71	37.02	38.18	48.50	38.76	39.21	39.47	39.62	39.52
	plate side	29.03	29.49	30.77	31.49	33.33	34.43	35.63	36.87	38.52	40.11	40.58	40.95	41.66	42.05	42.27	42.36

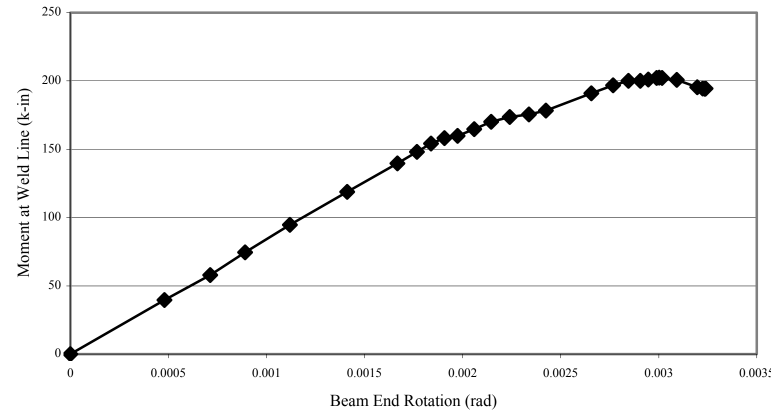
Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

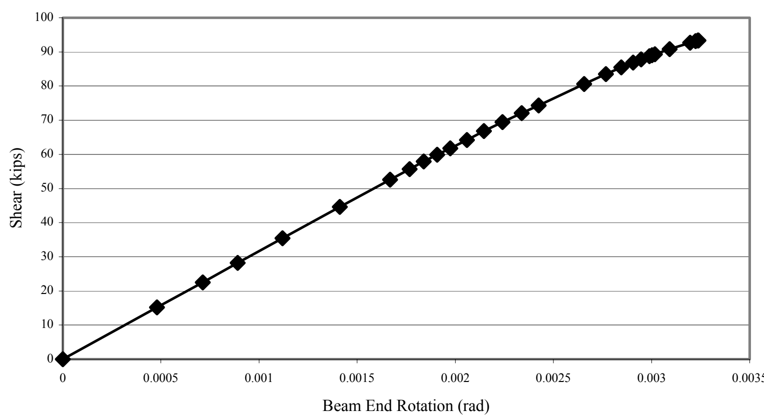
Indicates that the stress in outer element exceeds the stress in the innermost element



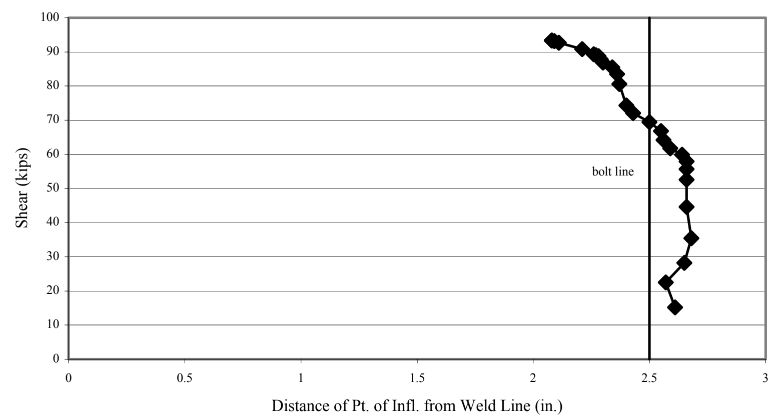
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C11 Results from Model 13

Table C11 Shear Stress in Bolts of Model 13

Increment		72	74	76	78	80	82	84	86	88	90	92	94	96	98	100
Load		83.05	83.42	83.54	83.80	84.40	84.73	85.48	86.84	87.77	88.73	89.02	89.32	90.00	90.38	90.86
1st bolt	beam side	57.71	57.96	57.99	58.06	58.21	58.29	58.44	58.45	58.25	57.92	57.80	57.71	57.50	57.31	57.01
	plate side	58.74	58.88	58.92	59.02	59.25	59.39	59.68	60.21	60.48	60.77	60.87	60.98	61.15	61.26	61.42
2nd bolt	beam side	57.50	57.78	57.86	58.05	58.23	58.33	58.52	58.66	58.51	58.27	58.17	58.09	57.81	57.61	57.31
	plate side	58.34	58.48	58.53	58.62	58.87	59.01	59.29	59.85	60.18	60.54	60.65	60.04	60.87	60.96	61.13
3rd bolt	beam side	56.53	56.76	56.84	57.02	57.40	57.64	58.24	58.81	59.09	59.22	59.20	59.18	59.13	59.10	58.99
	plate side	57.12	57.36	57.44	57.64	58.14	58.44	58.74	59.25	59.63	60.02	60.12	60.21	60.40	60.51	60.65

Bolt shear capacity

Indicating that the stress decreases in the next increment

Indicating that the stress in outer element exceeds1 the stress in the innermost element

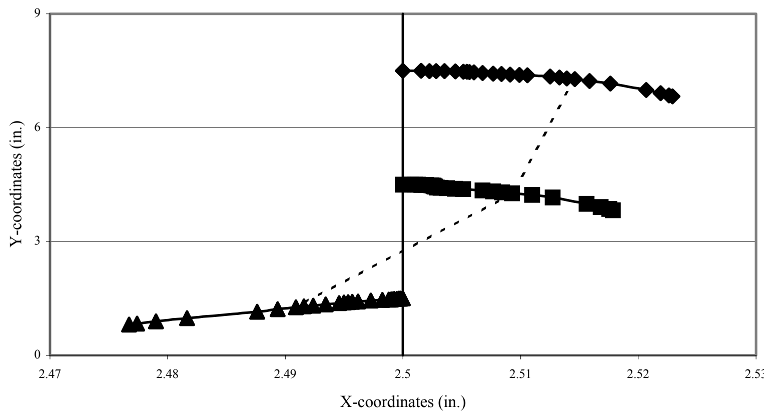
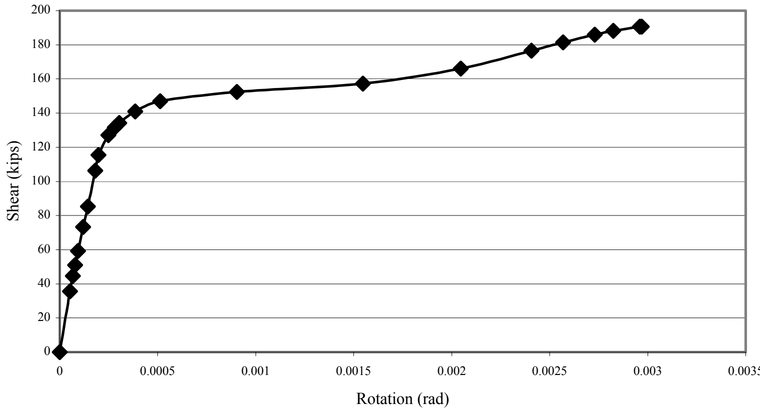
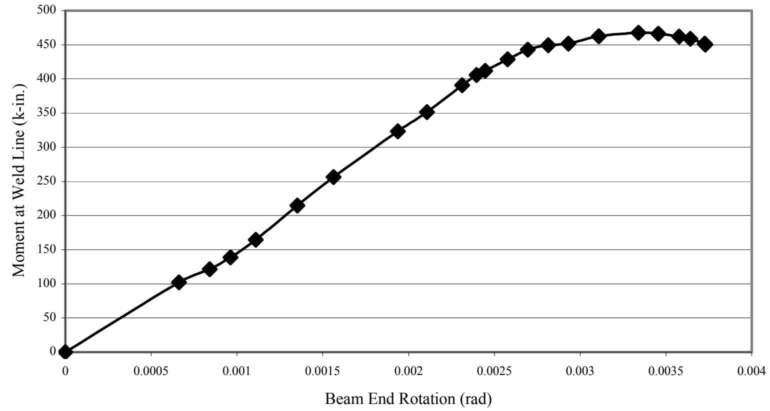


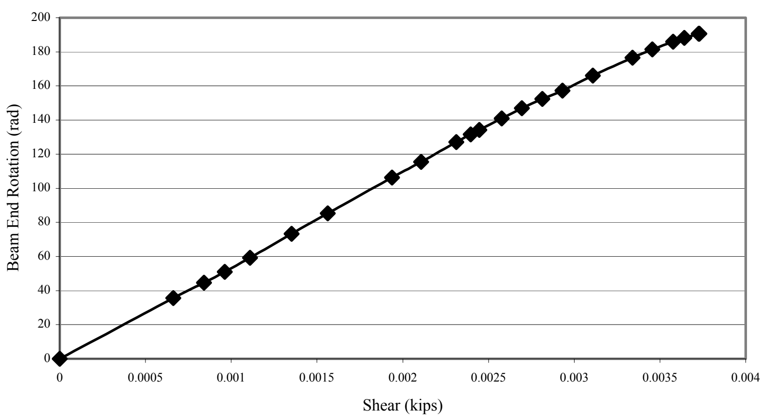
Figure C12 Model 13 - Bolt Movement



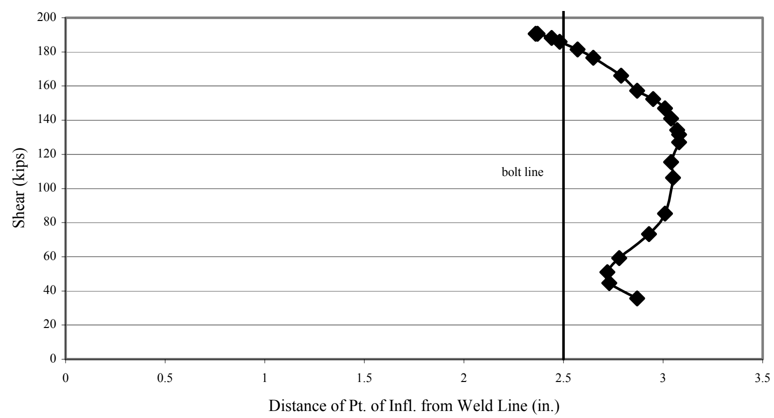
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C13 Results from Model 14

Table C12 Shear Stress in Bolts of Model 14

Increment		50	52	54	56	58	60	62	64	66	68	70	72	74	76
Load		144.8	147.0	149.6	152.4	153.9	157.3	158.9	160.8	163.1	166.1	169.7	173.3	175.4	176.6
1st bolt	beam side	41.07	41.44	41.81	42.20	42.44	42.65	42.70	42.68	42.59	42.27	41.52	40.70	40.32	40.20
	plate side	42.61	43.19	43.97	44.95	45.61	46.77	47.22	47.63	48.00	48.31	48.43	48.34	48.29	48.25
2nd bolt	beam side	41.82	42.11	42.54	42.90	43.06	43.19	43.22	43.21	43.17	42.94	42.34	41.53	41.06	40.91
	plate side	43.15	43.57	44.21	44.97	45.46	46.48	46.86	47.35	47.85	48.27	48.57	48.61	48.52	48.49
3rd bolt	beam side	42.21	42.48	42.78	43.03	43.12	43.25	43.30	43.36	43.31	43.16	42.69	41.85	41.37	41.19
	plate side	43.45	43.88	44.43	45.03	45.35	46.11	46.48	46.91	47.52	48.21	48.69	48.83	48.77	48.69
4th bolt	beam side	42.05	42.29	42.59	42.88	43.05	43.27	43.32	43.35	43.34	43.12	42.59	41.81	41.33	41.12
	plate side	43.30	43.65	44.12	44.61	44.88	45.70	46.19	46.81	47.54	48.27	48.71	48.78	48.67	48.60
5th bolt	beam side	41.62	41.83	42.08	42.46	42.67	43.97	43.15	43.20	43.28	43.11	42.42	41.56	41.10	40.89
	plate side	42.76	43.09	43.51	44.07	44.40	45.28	45.72	46.37	47.16	48.04	48.52	48.58	48.49	48.45
6th bolt	beam side	41.15	41.45	41.81	42.19	42.35	42.63	42.68	42.76	42.82	42.76	42.33	41.61	41.17	40.95
	plate side	42.24	42.66	43.21	43.83	44.16	44.96	45.41	46.11	46.82	47.65	48.15	48.39	48.37	48.33
7th bolt	beam side	40.28	40.48	40.76	41.18	41.44	41.98	42.19	42.39	42.63	42.84	42.75	42.15	41.69	41.41
	plate side	41.49	41.74	42.08	42.56	42.95	43.81	44.22	44.76	45.55	46.70	47.94	48.44	48.57	48.57

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

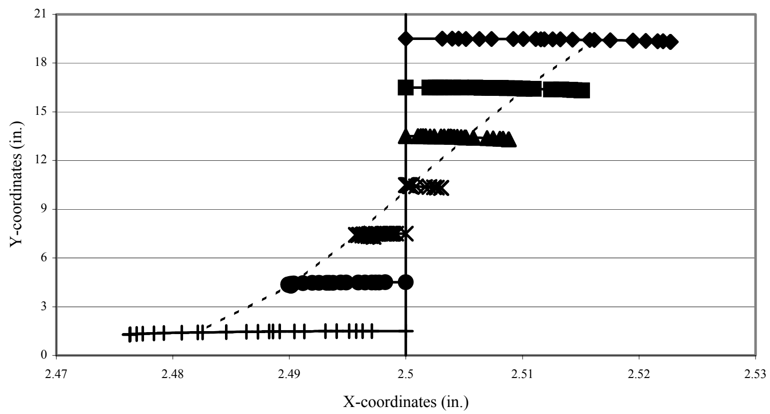
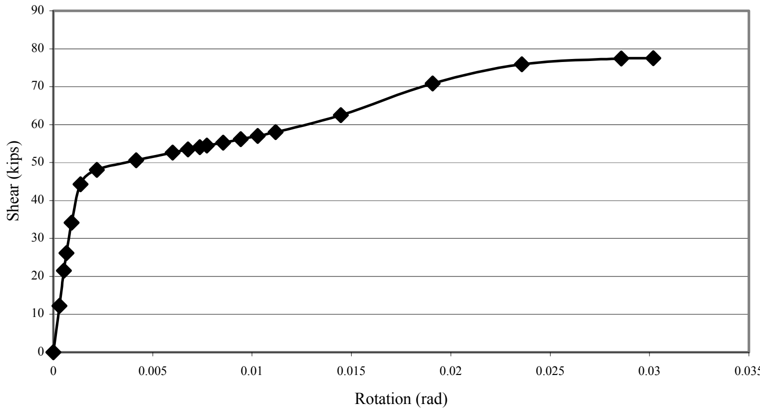
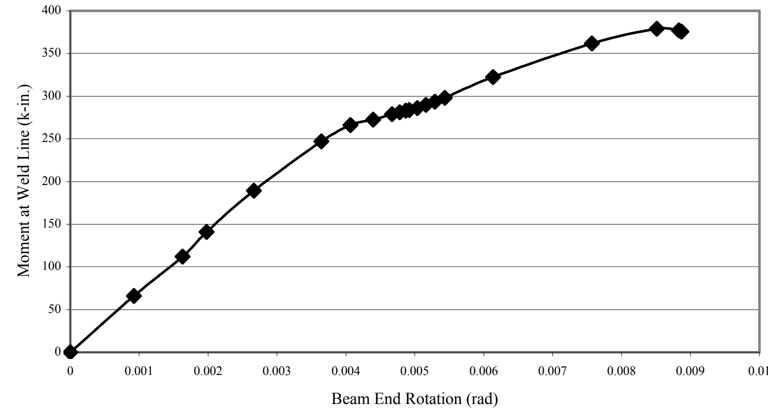


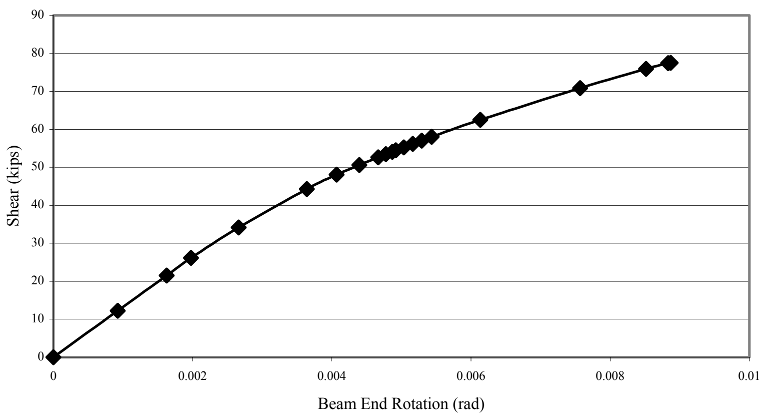
Figure C14 Model 14 - Bolt Movement



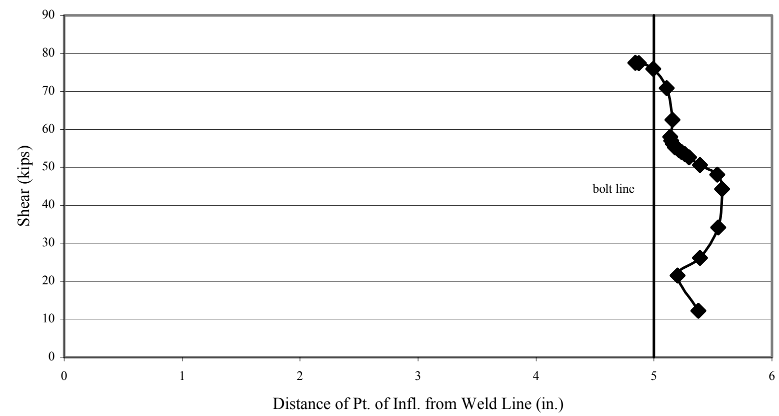
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C15 Results from Model 15

Table C13 Shear Stress in Bolts of Model 15

Increment		48	50	52	54	56	58	60	62	64	66	68	70	72	74
Load		55.26	55.70	56.14	56.58	57.02	57.45	58.00	59.23	60.76	61.62	62.50	64.47	66.91	69.41
1st bolt	beam side	39.24	39.48	39.74	39.99	40.24	40.54	40.94	41.49	42.00	42.29	42.60	43.21	43.82	43.78
	plate side	39.85	40.12	40.40	40.66	40.97	41.31	41.74	42.49	43.24	43.64	43.99	44.73	45.89	47.59
2nd bolt	beam side	40.23	40.56	40.91	41.77	41.36	41.52	41.72	42.07	42.47	42.70	42.91	43.50	44.15	44.35
	plate side	40.60	40.96	42.32	41.60	41.84	42.03	42.29	42.88	43.47	43.61	43.87	44.56	45.61	47.08
3rd bolt	beam side	38.61	38.82	39.03	39.26	39.49	39.69	39.90	40.39	41.07	41.48	41.84	42.65	43.62	43.78
	plate side	38.78	38.98	39.18	39.38	39.61	39.85	40.08	40.57	41.43	41.79	42.13	42.96	44.25	45.88

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

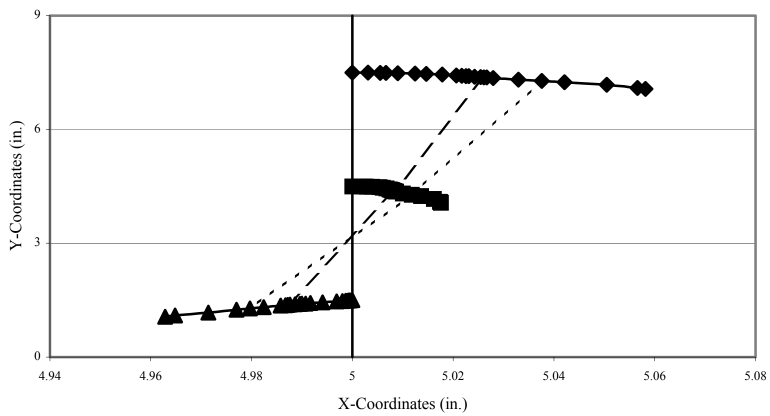
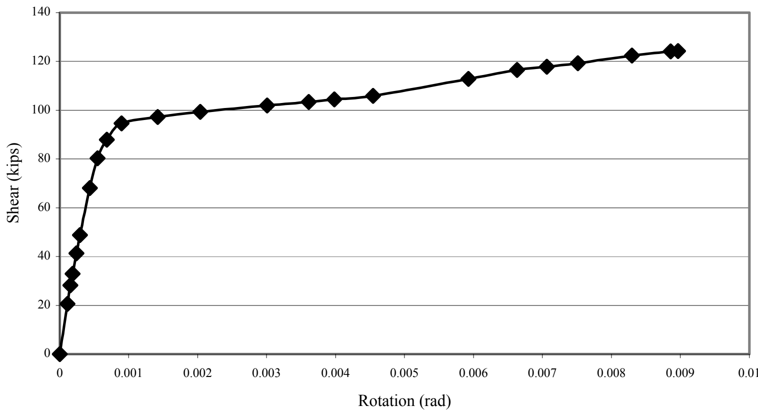
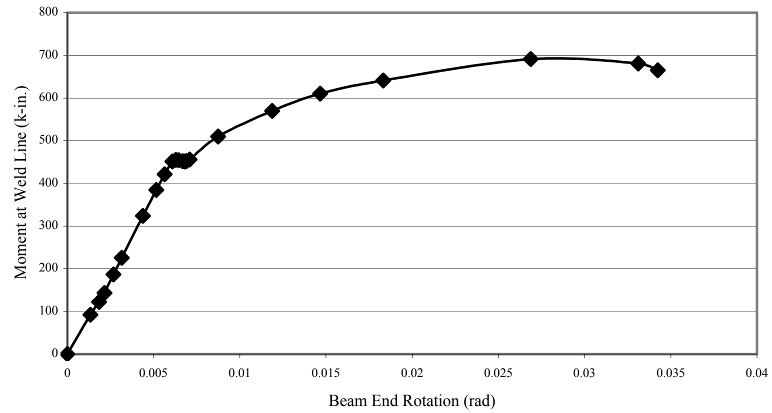


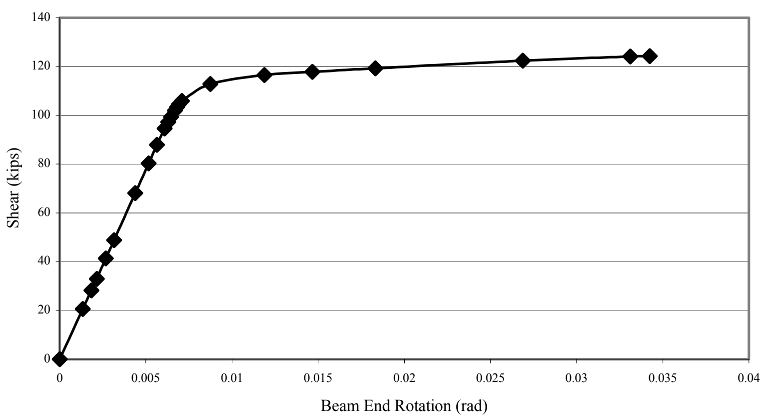
Figure C16 Model 15 - Bolt Movement



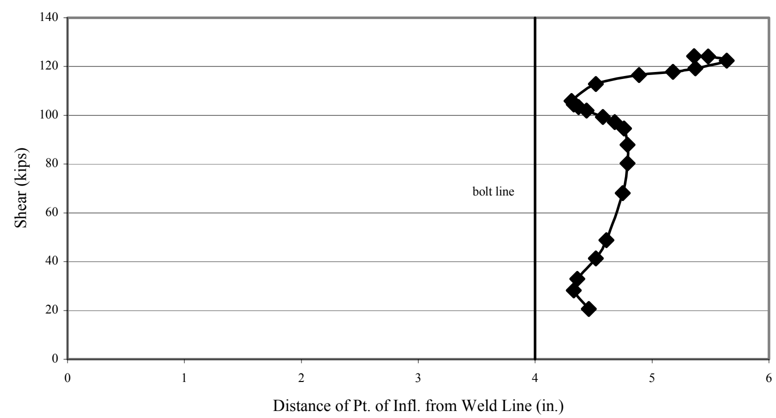
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C17 Results from Model 16

Table C14 Shear Stress in Bolts of Model 16

Increment		54	56	58	60	62	64	66	68	70	72	74	76	78	80
Load		103.9	104.4	104.9	105.9	107.7	109.4	111.6	112.9	113.6	115.1	116.0	116.5	116.8	117.1
1st bolt	beam side	42.48	42.60	42.74	42.95	43.38	43.67	43.58	43.34	43.11	41.95	40.96	40.04	39.51	38.95
	plate side	43.60	43.76	43.91	44.18	44.78	45.40	46.21	46.50	46.62	46.21	45.63	44.97	44.53	43.94
2nd bolt	beam side	42.97	43.09	43.21	43.41	43.65	43.89	44.10	44.09	44.08	43.74	43.19	42.78	42.43	41.95
	plate side	43.95	44.11	44.27	44.53	44.89	45.29	45.95	44.39	46.64	47.32	47.56	47.54	47.47	47.29
3rd bolt	beam side	42.92	43.02	43.12	43.29	43.53	43.79	44.13	44.32	44.37	44.38	44.21	44.00	43.81	43.51
	plate side	43.86	43.98	44.10	44.31	44.62	44.96	45.58	46.13	46.51	47.43	48.05	48.31	48.42	48.51
4th bolt	beam side	42.30	42.40	42.50	42.68	42.96	43.21	43.58	43.69	43.67	43.45	43.11	42.67	42.37	41.93
	plate side	43.19	43.30	43.41	43.65	43.99	44.31	44.89	45.37	45.66	46.38	46.81	46.95	46.94	46.89
5th bolt	beam side	41.29	41.43	41.54	41.76	42.14	42.60	43.08	43.23	43.21	42.71	42.06	41.45	41.06	40.58
	plate side	42.38	42.51	42.63	42.87	43.32	43.85	44.44	44.69	44.84	45.72	44.90	44.79	44.65	44.38

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

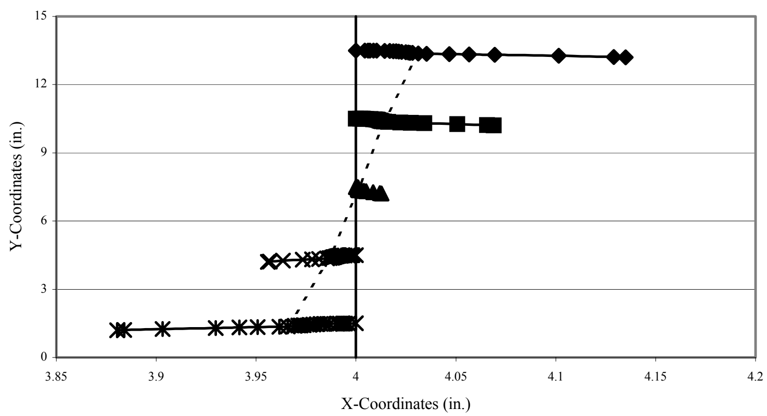
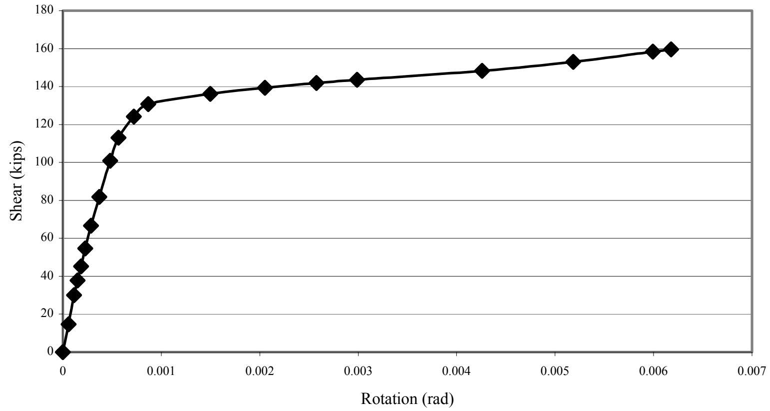
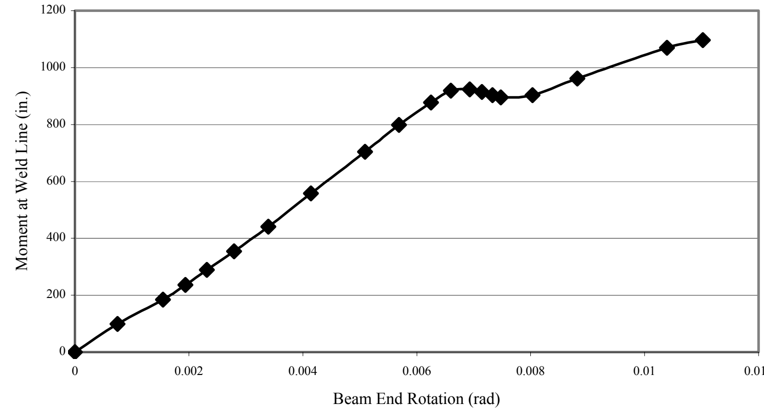


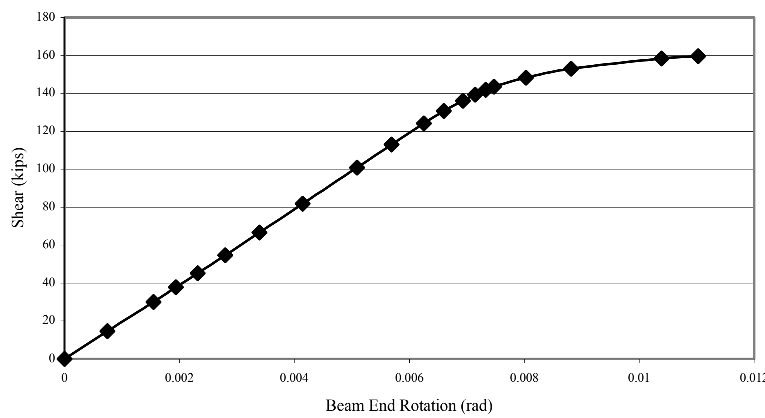
Figure C18 Model 16 - Bolt Movement



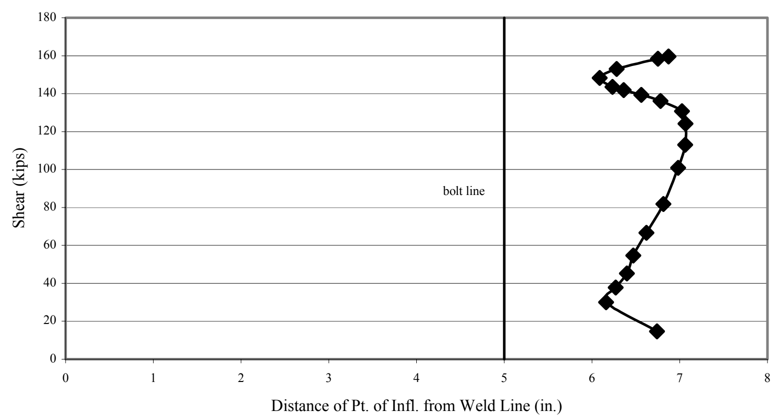
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C19 Results from Model 17

Table C15 Shear Stress in Bolts of Model 17

Increment		60	62	64	66	68	70	72	74	76	78	80	82	84	86
Load		143.6	144.8	146.0	147.2	148.3	149.5	150.7	151.8	153.0	154.5	156.3	157.3	158.3	159.5
1st bolt	beam side	40.81	41.00	41.26	41.47	41.64	41.79	41.89	41.94	41.88	41.71	41.12	40.60	39.88	38.93
	plate side	41.64	42.00	42.33	42.72	43.13	43.57	43.98	44.41	44.78	44.93	44.58	44.11	43.35	42.26
2nd bolt	beam side	41.21	41.52	41.73	41.84	41.93	42.00	42.01	42.00	41.98	41.95	41.76	41.56	41.27	40.86
	plate side	42.56	43.09	43.59	44.06	44.43	44.71	44.92	45.12	45.24	45.37	45.47	45.42	45.27	44.92
3rd bolt	beam side	42.29	42.47	42.60	42.71	42.79	42.84	42.88	42.90	42.94	42.99	43.05	43.01	42.94	42.70
	plate side	43.74	44.00	44.23	44.48	44.71	44.90	45.06	45.24	45.48	45.85	46.48	46.87	47.20	47.54
4th bolt	beam side	42.15	42.30	42.42	42.54	42.65	42.74	42.82	42.90	42.97	43.07	43.20	43.28	43.32	43.24
	plate side	43.50	43.72	43.91	44.10	44.29	44.44	44.56	44.71	44.94	45.29	46.00	46.53	47.14	47.83
5th bolt	beam side	41.00	41.20	41.38	41.56	41.73	41.89	42.03	42.16	42.27	42.37	42.46	42.48	42.52	42.44
	plate side	42.15	42.38	42.61	42.86	43.10	43.33	43.55	43.79	44.09	44.55	45.32	45.80	46.33	46.81
6th bolt	beam side	39.87	40.04	40.28	40.52	40.71	40.87	41.01	41.12	41.17	41.20	41.18	41.17	41.07	40.76
	plate side	40.88	41.06	41.35	41.64	41.93	42.20	42.40	42.58	42.80	43.14	43.60	43.92	44.13	44.22
7th bolt	beam side	39.59	39.64	39.69	39.76	39.78	39.88	40.07	40.21	40.28	40.22	40.02	39.93	39.81	39.48
	plate side	40.08	40.16	40.22	40.38	40.54	40.86	41.18	41.48	41.70	41.91	42.17	42.30	42.46	42.25

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

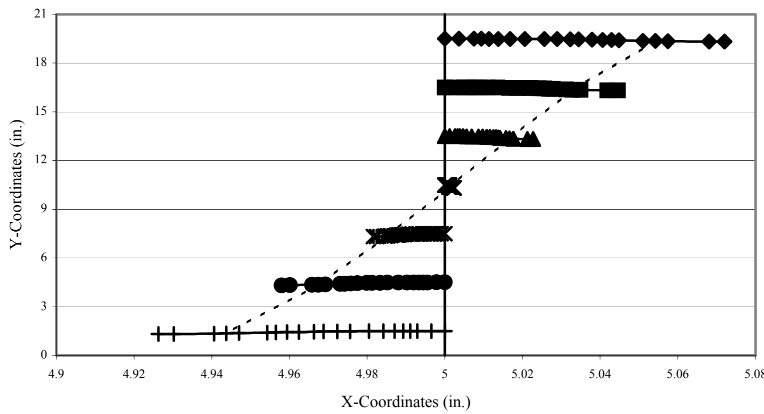
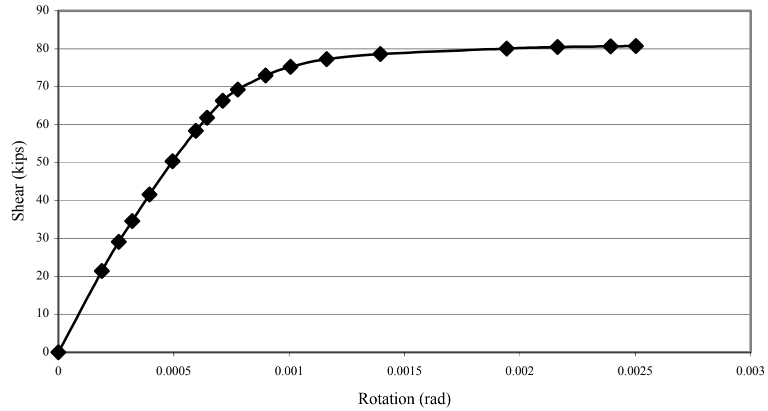
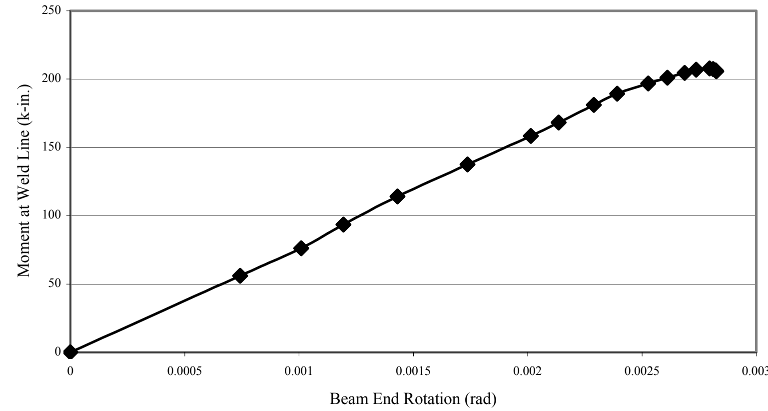


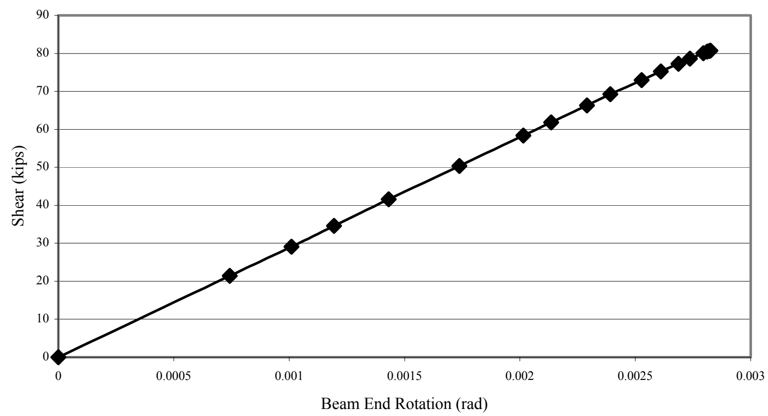
Figure C20 Model 17 - Bolt Movement



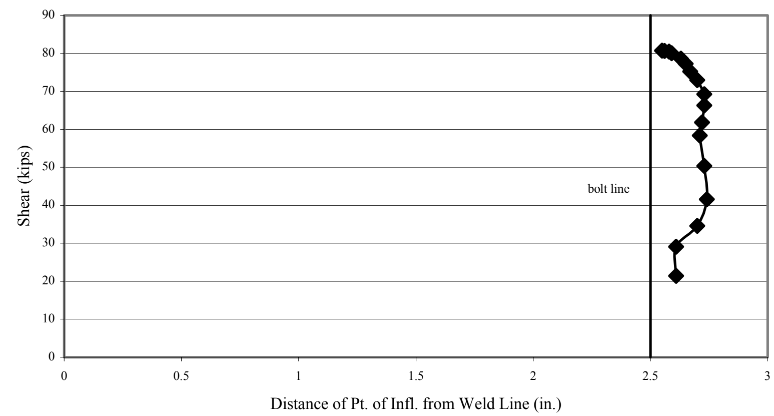
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C21 Results from Model 18

Table C16 Shear Stress in Bolts of Model 18

Increment		22	24	26	28	30	32	34	36	38	40	42	44	46	48
Load		54.32	58.33	60.59	61.86	64.71	66.32	67.23	69.26	70.40	72.98	74.43	75.24	76.25	77.28
1st bolt	beam side	38.22	39.39	40.07	40.49	41.29	41.70	41.94	42.28	42.30	41.40	40.42	39.82	39.41	39.45
	plate side	39.27	41.03	41.81	42.30	43.39	43.91	44.31	45.51	46.28	47.44	47.41	47.28	47.15	47.06
2nd bolt	beam side	38.62	39.76	40.42	40.78	41.66	42.19	42.38	42.60	42.69	41.83	40.85	40.32	40.01	40.25
	plate side	39.31	41.15	41.86	42.28	43.28	43.99	44.43	45.75	46.48	47.64	47.57	47.39	47.19	47.06
3rd bolt	beam side	38.72	39.85	40.50	40.88	41.73	42.14	42.33	42.65	42.70	41.94	40.98	40.39	40.01	40.16
	plate side	39.47	41.25	41.96	42.37	43.46	43.99	44.33	45.69	46.42	47.64	47.62	47.43	47.23	47.14

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

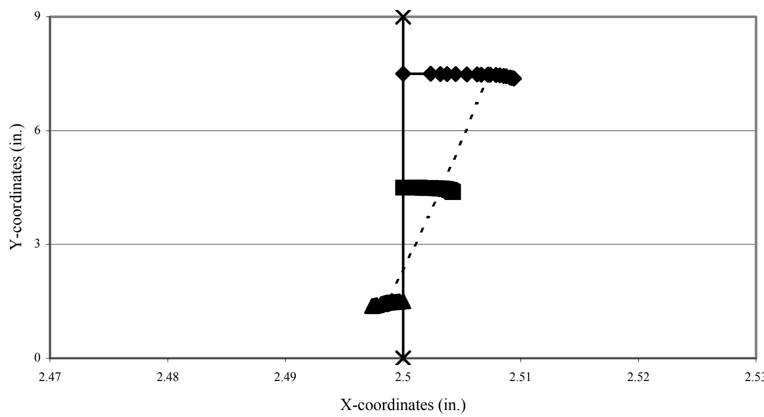
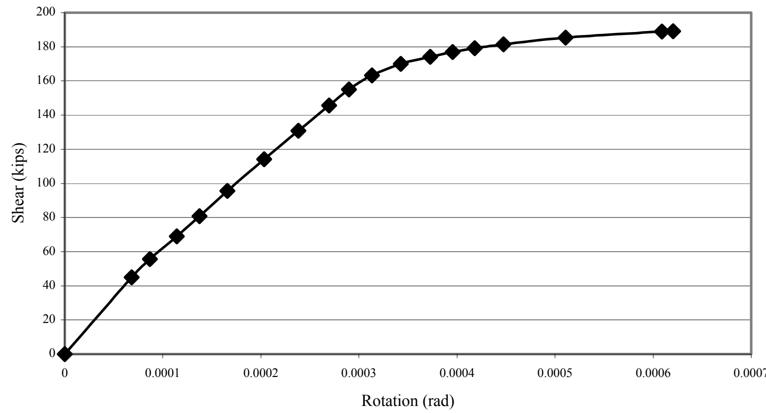
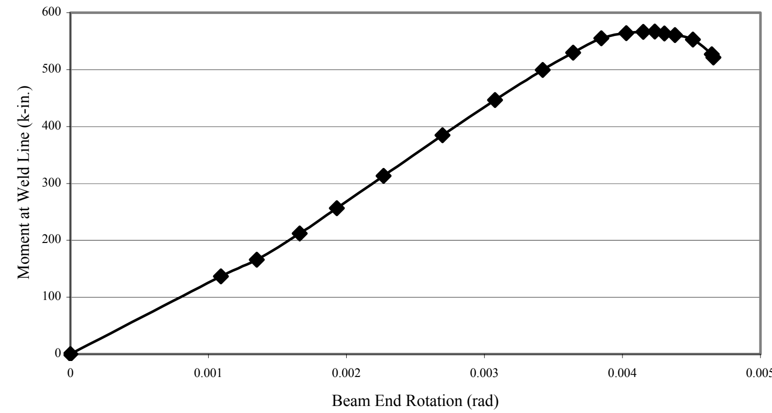


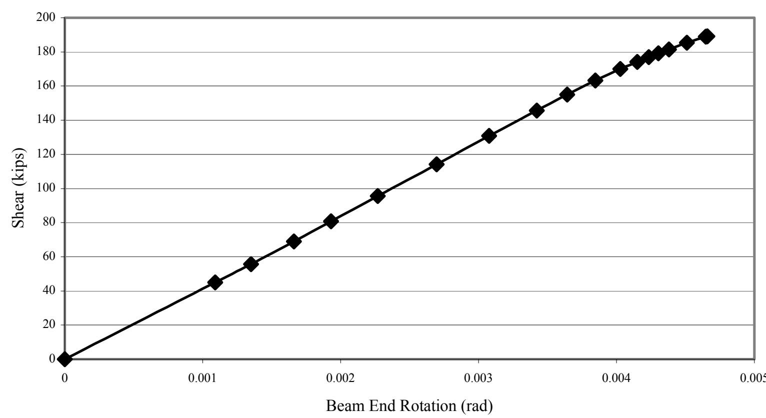
Figure C22 Model 18 - Bolt Movement



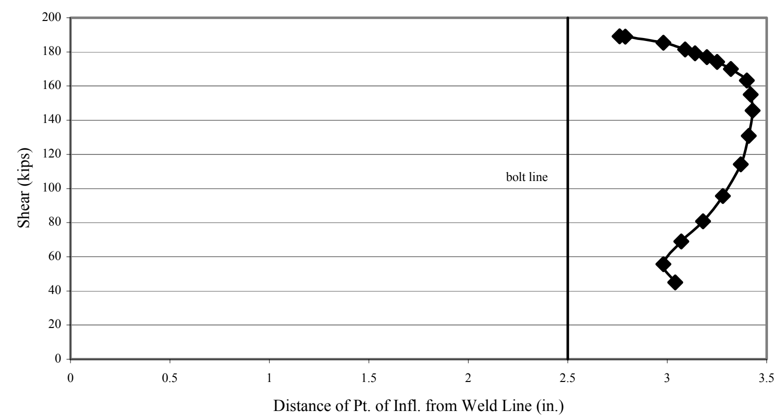
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C23 Results from Model 19

Table C17 Shear Stress in Bolts of Model 19

Increment		24	26	28	30	32	34	36	38	40	42	44	46	48	50
Load		114.2	124.8	130.8	138.1	145.7	149.9	155.0	160.2	163.2	166.2	169.9	172.0	174.2	176.6
1st bolt	beam side	35.07	36.91	37.36	37.98	38.34	38.53	38.71	38.81	38.75	38.62	38.27	37.91	37.54	37.49
	plate side	36.04	38.33	39.01	39.78	40.74	41.47	42.56	43.99	44.75	45.26	45.58	45.74	45.90	46.15
2nd bolt	beam side	34.96	37.38	37.99	38.69	39.25	39.55	40.06	40.34	40.48	40.18	39.52	39.18	38.84	38.69
	plate side	35.05	37.93	39.34	40.24	41.31	42.07	43.26	44.77	45.68	46.49	46.90	46.98	46.98	47.05
3rd bolt	beam side	34.71	37.58	38.45	39.29	40.03	40.46	40.91	41.27	41.18	40.75	39.95	39.49	38.96	38.68
	plate side	35.07	38.28	39.64	40.81	41.74	42.59	43.60	45.59	46.66	47.33	47.55	47.54	47.55	47.51
4th bolt	beam side	34.88	37.71	38.67	39.38	40.14	40.65	41.11	41.44	41.38	40.96	40.05	39.51	38.97	38.64
	plate side	35.20	38.49	39.80	40.89	41.87	42.61	43.72	45.73	46.78	47.45	47.72	47.72	47.64	47.55
5th bolt	beam side	35.25	37.80	38.48	39.11	39.94	40.32	40.62	40.96	41.07	40.85	40.08	39.58	39.12	38.79
	plate side	35.44	38.38	39.66	40.60	41.76	42.51	43.48	44.90	46.01	46.97	47.41	47.50	47.43	47.41
6th bolt	beam side	35.50	37.73	38.21	38.81	39.35	39.53	39.82	40.04	40.16	40.11	39.55	39.15	38.87	38.70
	plate side	35.84	38.42	39.50	40.24	41.21	41.86	42.93	44.30	45.17	45.89	46.42	46.62	46.73	46.85
7th bolt	beam side	35.27	37.11	37.61	38.27	38.97	39.13	39.33	39.47	39.47	39.28	38.86	38.66	38.40	38.04
	plate side	36.04	38.20	38.90	39.06	40.70	41.33	42.42	43.44	44.08	44.82	45.46	45.68	45.73	46.12

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

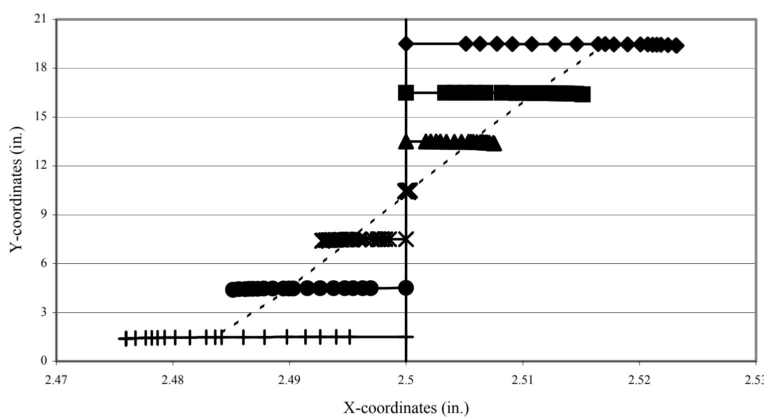
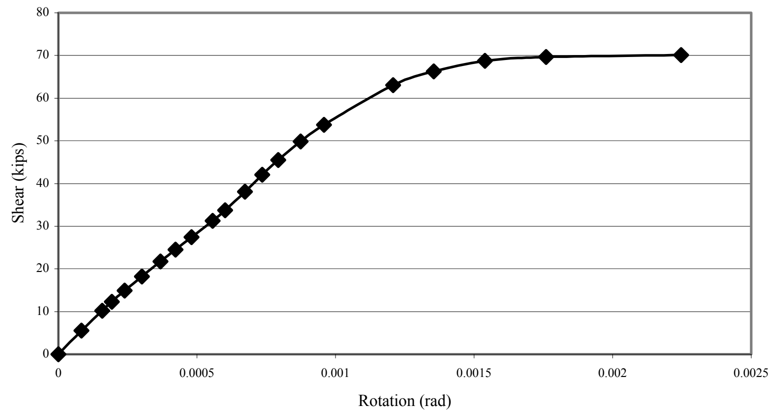
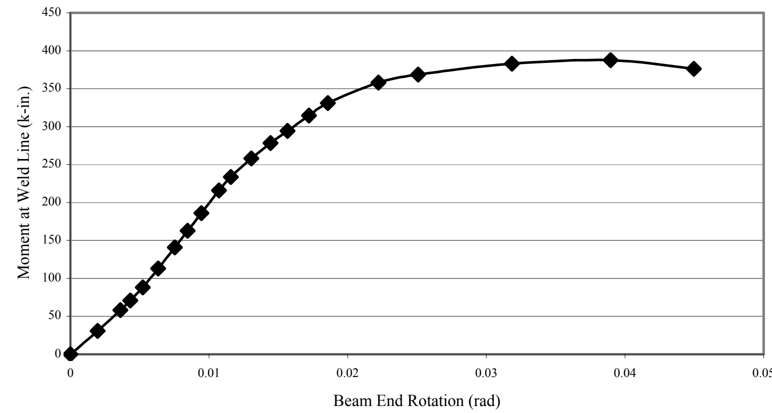


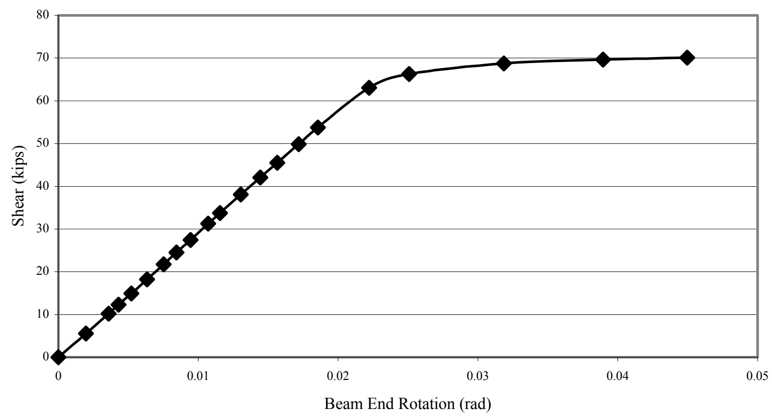
Figure C24 Model 19 - Bolt Movement



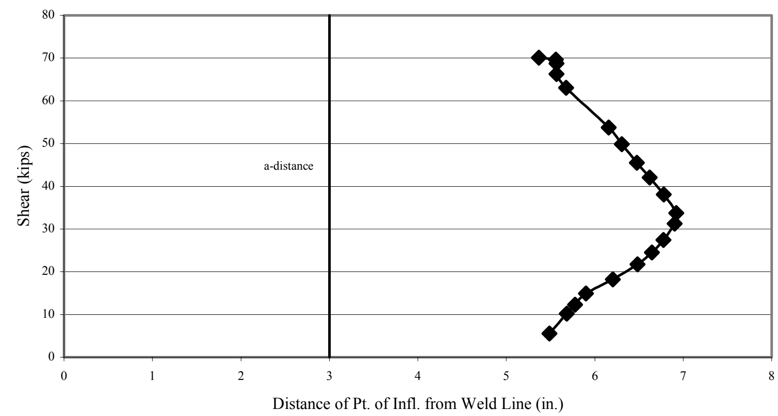
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C25 Results from Model 20

Table C18 Shear Stress in Bolts of Model 20

Increment		56	58	60	62	64	66	68	70	72	74	76	78	80	82
Load		49.88	52.36	53.76	56.90	58.66	60.85	63.08	63.77	64.48	65.35	66.24	67.35	67.97	68.60
1st bolt	beam side	25.79	27.15	28.03	30.48	31.66	33.03	33.78	33.95	33.99	33.73	33.63	33.45	33.42	33.40
	plate side	27.10	28.65	29.62	32.01	33.12	34.27	34.91	35.01	34.99	34.64	34.35	33.83	33.59	33.27
2nd bolt	beam side	34.85	35.18	35.34	36.26	36.72	36.95	36.94	36.85	36.67	36.37	36.23	36.33	36.62	36.99
	plate side	35.82	36.57	36.82	37.52	37.99	38.46	38.65	38.69	38.70	38.58	38.44	38.21	38.02	37.78
3rd bolt	beam side	35.62	37.54	38.46	39.63	40.38	41.23	42.10	42.26	42.37	42.31	41.61	40.21	39.72	39.73
	plate side	35.82	38.04	39.15	41.06	41.86	43.04	44.79	45.37	45.94	46.90	47.35	47.00	46.63	46.25

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

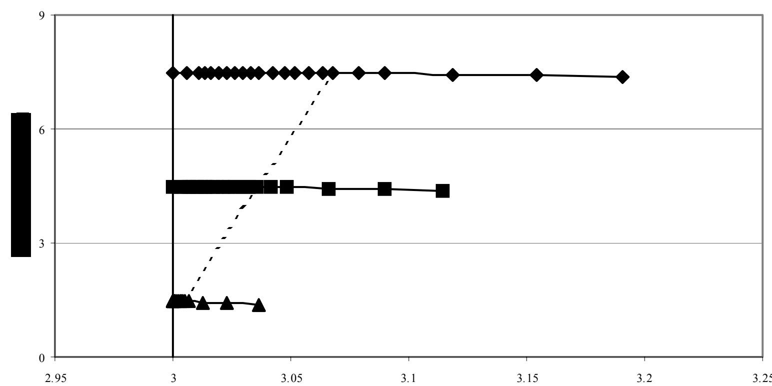
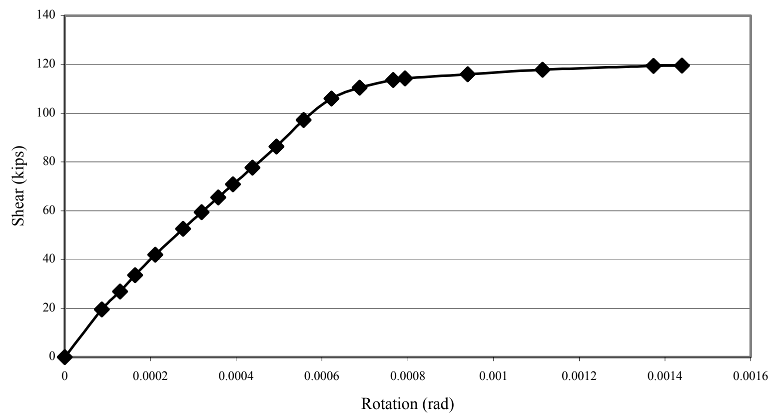
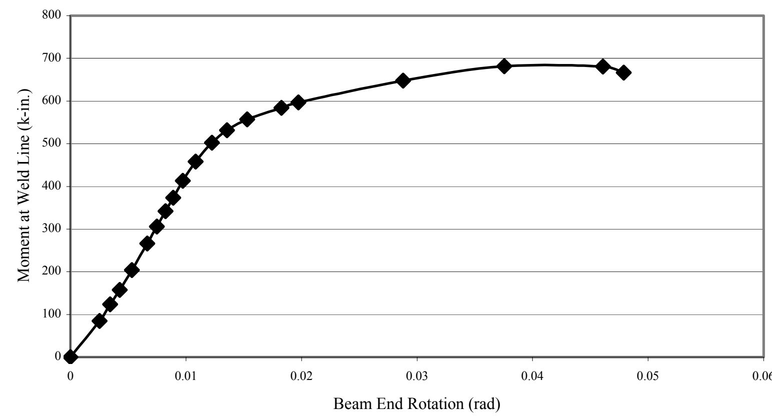


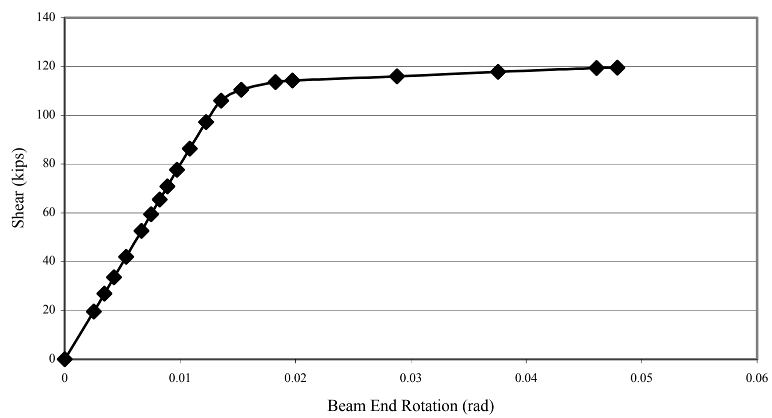
Figure C26 Model 20 - Bolt Movement



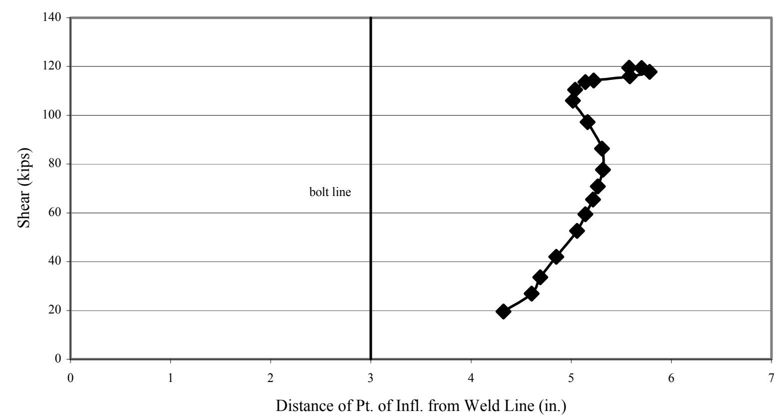
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C27 Results from Model 21

Table C19 Shear Stress in Bolts of Model 21

Increment		34	36	38	40	42	44	46	48	50	52	54	56	58	60
Load		74.24	77.69	81.97	86.34	91.75	97.29	100.4	106.0	107.7	110.5	111.6	113.6	113.9	114.3
1st bolt	beam side	30.65	31.27	32.27	33.46	35.34	36.51	36.67	36.39	36.04	35.34	35.01	34.45	34.15	33.89
	plate side	31.38	31.92	32.96	34.15	35.92	37.39	37.91	38.12	37.80	37.12	36.78	35.86	35.51	35.13
2nd bolt	beam side	33.18	34.45	35.74	36.26	36.73	37.17	37.76	38.98	39.33	38.67	38.21	37.00	36.68	36.45
	plate side	33.28	34.72	36.16	37.33	37.89	38.78	39.81	41.94	42.60	42.84	42.60	41.51	41.05	40.68
3rd bolt	beam side	29.46	31.34	33.54	35.94	38.28	39.67	40.31	41.84	42.35	42.60	42.23	40.73	40.47	40.25
	plate side	29.75	31.66	33.73	36.13	38.82	41.08	41.75	43.68	44.75	46.83	47.46	47.37	47.28	47.19
4th bolt	beam side	32.21	33.61	35.09	36.41	37.09	38.07	38.88	40.50	40.83	40.75	40.35	39.11	38.93	38.93
	plate side	32.48	33.84	35.54	37.00	38.18	38.99	39.76	42.06	42.96	44.13	44.27	43.85	43.75	43.72
5th bolt	beam side	30.51	31.21	32.35	33.76	35.68	37.07	37.38	37.14	36.89	36.39	36.25	36.04	36.00	35.95
	plate side	30.89	31.46	32.27	33.44	35.45	37.25	37.95	38.49	38.37	38.06	37.92	37.64	37.48	37.29

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

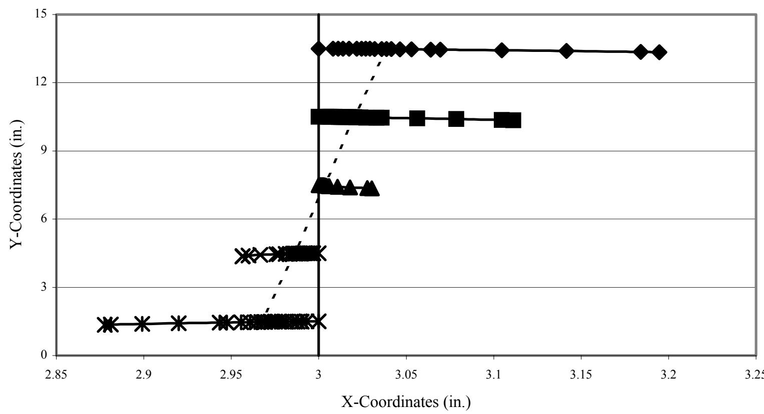
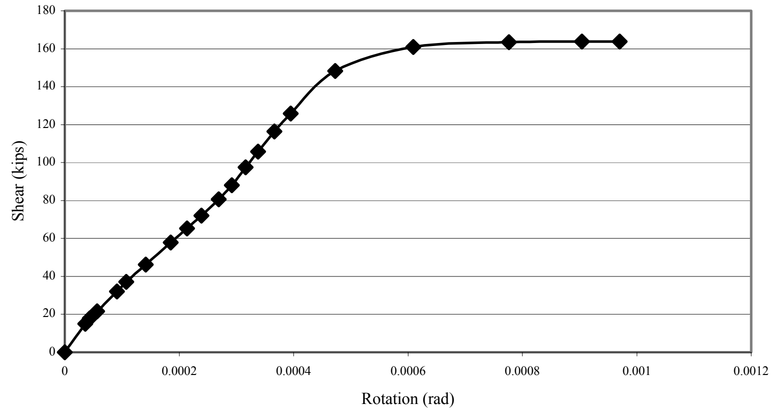
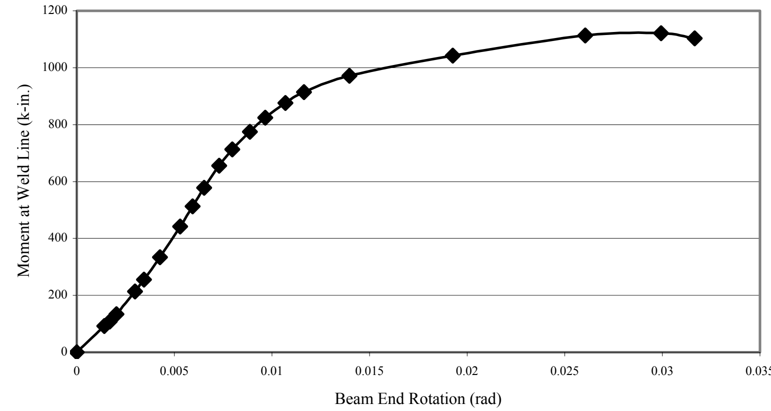


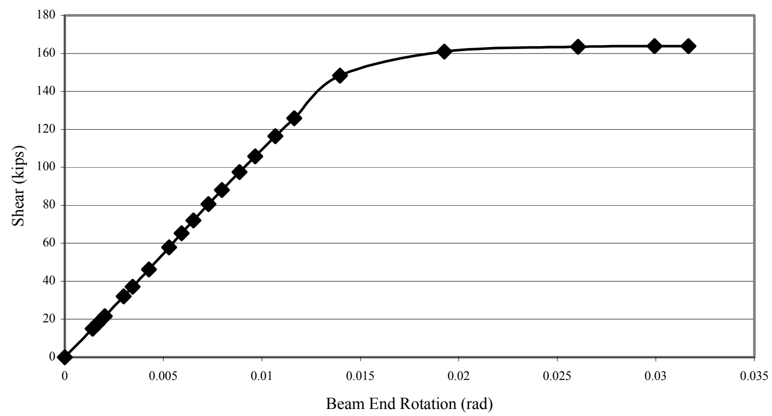
Figure C28 Model 21 - Bolt Movement



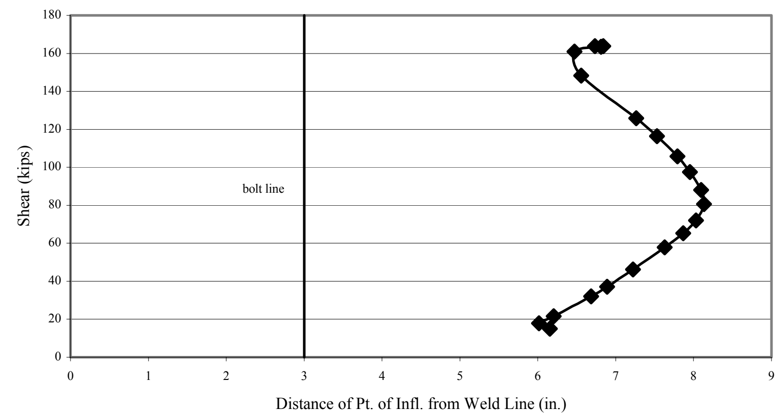
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C29 Results from Model 22

Table C20 Shear Stress in Bolts of Model 22

Increment		44	46	48	50	52	54	56	58	60	62	64	66	68	70
Load		88.02	92.24	97.49	102.8	105.8	112.6	116.4	121.1	125.9	131.8	137.9	144.8	148.2	152.4
1st bolt	beam side	21.91	22.91	24.09	25.32	26.10	27.65	28.51	29.61	30.49	31.36	32.07	32.94	33.45	33.68
	plate side	21.91	22.56	23.47	24.50	25.12	26.47	27.24	28.37	29.52	30.77	31.92	33.18	33.82	34.48
2nd bolt	beam side	28.07	28.65	29.00	29.37	29.76	31.22	32.23	33.50	34.59	35.41	35.92	36.23	38.46	36.63
	plate side	28.02	28.90	29.24	29.59	29.91	31.10	32.00	33.23	34.44	35.45	36.11	36.68	37.07	37.40
3rd bolt	beam side	27.61	29.41	31.54	33.53	34.33	36.12	36.46	36.84	37.21	37.68	38.12	38.36	38.29	37.69
	plate side	28.08	29.86	31.90	33.84	34.87	36.73	37.46	37.93	38.48	39.41	40.78	42.23	42.78	42.97
4th bolt	beam side	24.93	26.04	27.66	29.52	30.70	33.50	35.39	36.98	38.40	39.30	40.22	41.13	41.31	40.51
	plate side	24.70	25.95	27.66	29.79	31.02	33.87	35.74	37.79	39.39	40.87	42.02	44.55	46.10	47.47
5th bolt	beam side	26.88	28.22	30.02	31.95	33.05	35.05	36.05	36.62	37.52	38.75	39.80	40.25	40.15	39.53
	plate side	27.37	28.74	30.56	32.47	33.54	35.83	36.86	37.76	38.58	39.86	41.59	43.45	44.08	44.62
6th bolt	beam side	28.59	29.34	30.03	30.69	31.13	32.56	33.38	34.41	35.27	35.98	36.40	36.82	36.98	37.02
	plate side	28.46	29.46	30.30	30.73	31.13	32.46	33.10	34.06	35.04	36.10	36.82	37.61	37.97	38.46
7th bolt	beam side	22.19	23.23	24.48	25.87	26.71	28.63	29.65	30.89	31.92	32.88	33.67	34.51	34.91	35.13
	plate side	22.11	22.77	23.78	24.96	25.66	27.16	28.05	29.26	30.39	31.69	33.00	34.47	35.09	35.69

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

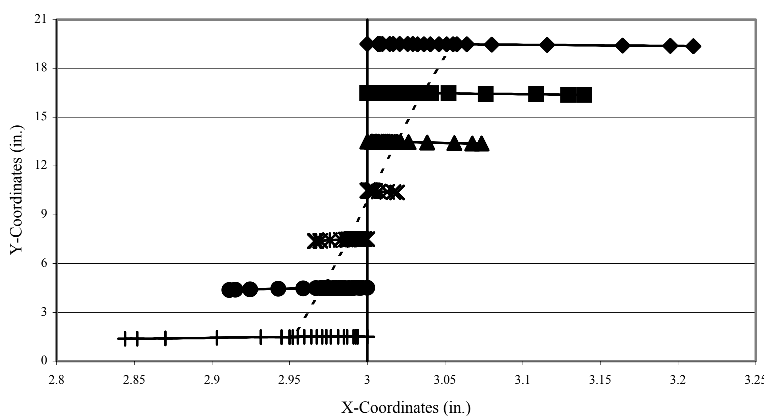
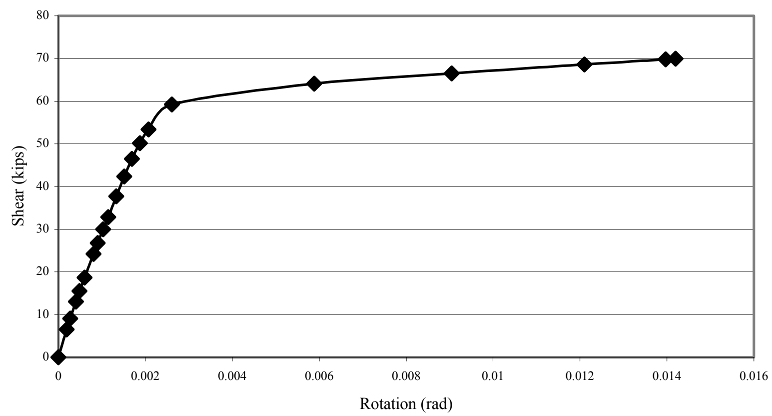
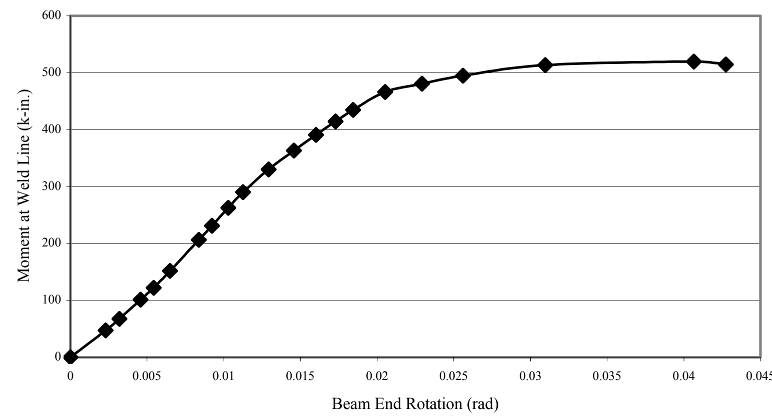


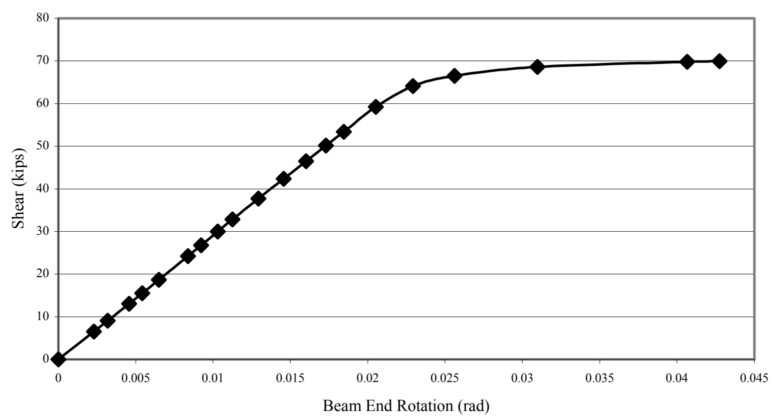
Figure C30 Model 22 - Bolt Movement



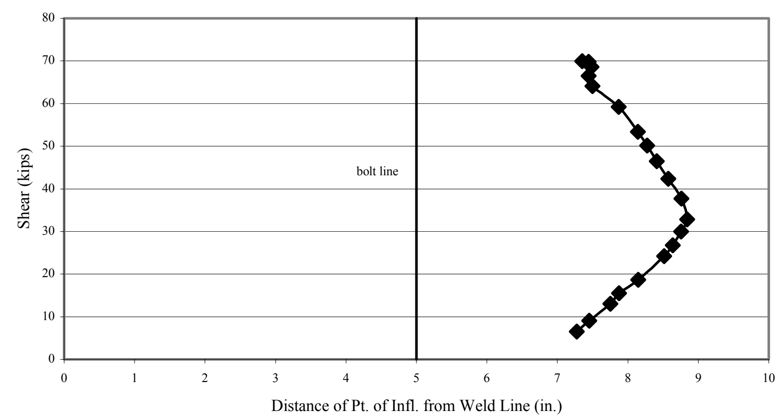
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C31 Results from Model 23

Table C21 Shear Stress in Bolts of Model 23

Increment		52	54	56	58	60	62	64	66	68	70	72	74	76	78
Load		50.13	51.75	53.40	57.12	59.21	61.80	62.46	63.28	64.12	64.59	65.17	65.77	66.50	66.92
1st bolt	beam side	26.61	27.54	28.61	31.27	32.72	34.58	35.36	36.12	36.61	36.66	36.59	36.46	36.13	35.84
	plate side	27.74	28.76	29.93	32.64	33.98	35.65	36.40	37.20	37.76	37.87	37.83	37.58	37.15	36.80
2nd bolt	beam side	34.95	35.20	35.42	36.27	36.85	37.19	37.32	37.43	37.50	37.55	37.57	37.50	37.22	37.03
	plate side	35.91	36.38	36.83	37.56	38.16	38.79	38.95	39.16	39.47	39.65	39.86	39.96	39.89	39.79
3rd bolt	beam side	35.78	36.93	38.08	39.54	40.35	41.13	41.13	41.21	41.27	41.32	41.37	41.33	40.96	40.57
	plate side	35.97	37.41	38.62	40.99	41.93	43.11	43.16	43.35	43.78	44.13	44.51	45.02	45.48	45.53

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

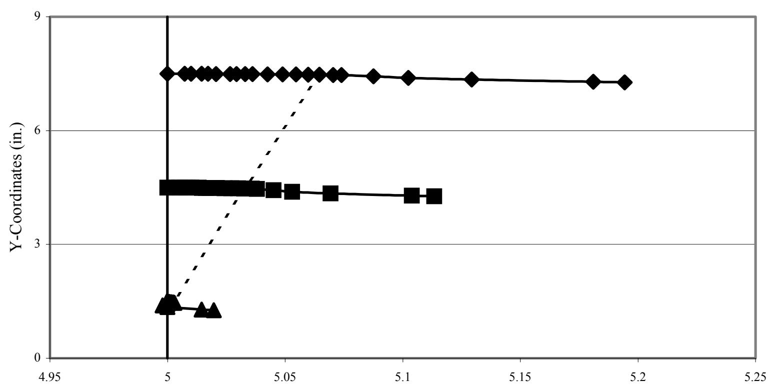
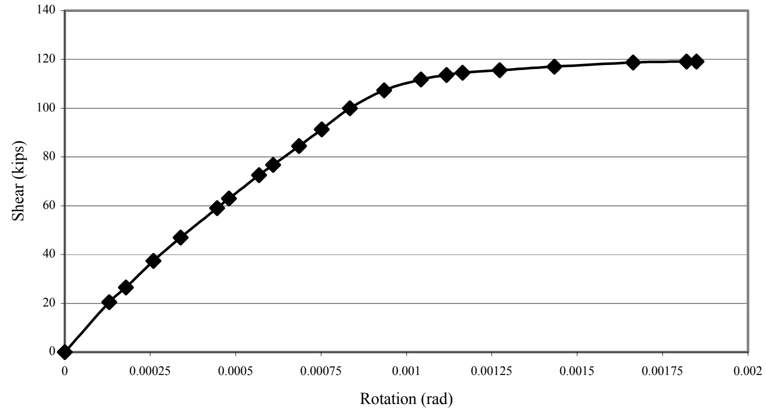
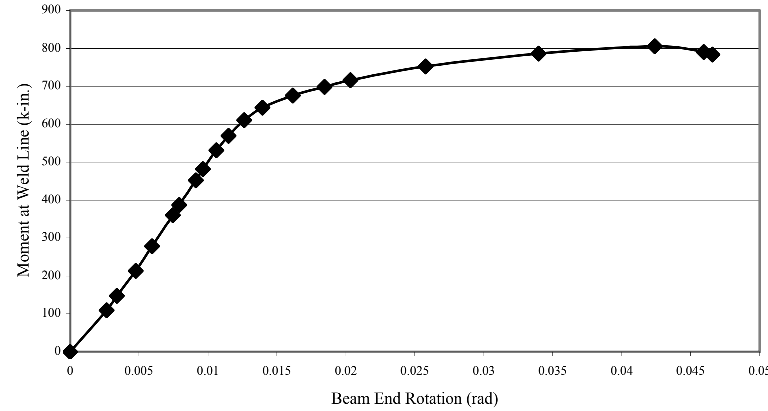


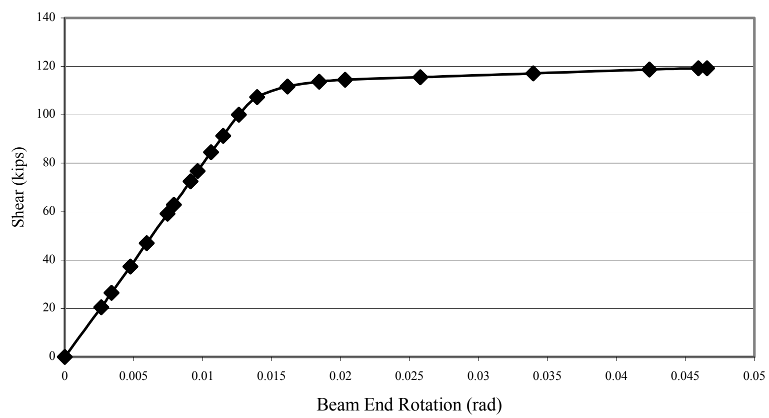
Figure C32 Model 23 - Bolt Movement



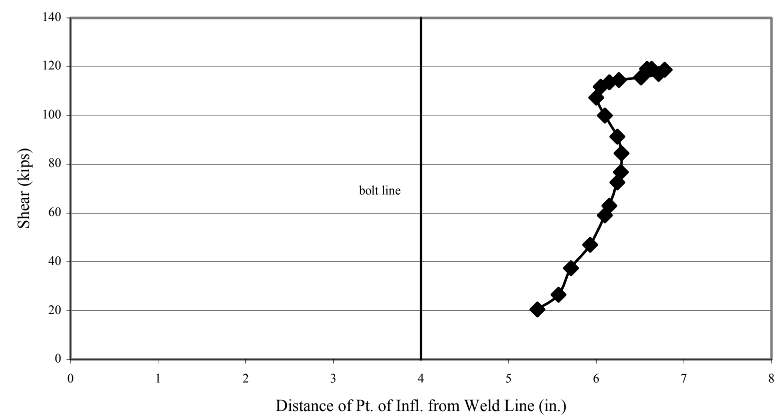
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C33 Results from Model 24

Table C22 Shear Stress in Bolts of Model 24

Increment		28	30	32	34	36	38	40	42	44	46	48	50	52	54
Load		72.53	75.26	76.79	80.25	84.52	88.9	91.36	96.89	100.0	105.6	107.4	109.5	111.7	113.0
1st bolt	beam side	30.51	31.01	31.27	32.05	33.04	34.43	35.25	36.49	36.70	36.46	36.15	35.67	35.01	34.62
	plate side	31.40	31.75	31.97	32.71	33.84	35.14	35.91	37.40	37.94	38.23	37.98	37.49	36.88	36.29
2nd bolt	beam side	32.48	33.61	34.22	35.17	36.11	36.48	36.72	37.18	37.81	38.99	39.39	39.18	38.27	37.53
	plate side	32.58	33.71	34.36	35.55	36.90	37.68	37.87	38.72	39.72	41.83	42.52	43.03	42.66	42.05
3rd bolt	beam side	28.66	30.16	30.96	32.69	34.96	37.08	38.11	39.59	40.22	41.71	42.27	42.70	42.18	41.29
	plate side	28.92	30.44	31.27	32.96	35.14	37.56	38.63	41.00	41.65	43.48	44.43	46.04	47.46	47.55
4th bolt	beam side	31.44	32.55	33.17	34.53	35.84	36.80	37.03	37.94	38.77	40.40	40.75	40.99	40.32	39.64
	plate side	31.73	32.81	33.42	34.79	36.32	37.57	38.11	38.88	39.65	41.88	42.72	43.85	44.30	44.14
5th bolt	beam side	30.30	30.83	31.12	31.99	33.24	34.76	35.61	37.02	37.38	37.18	36.93	36.50	36.21	36.06
	plate side	30.66	31.23	31.56	32.12	33.07	34.45	35.40	37.21	37.93	38.55	38.45	38.18	37.97	37.85

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

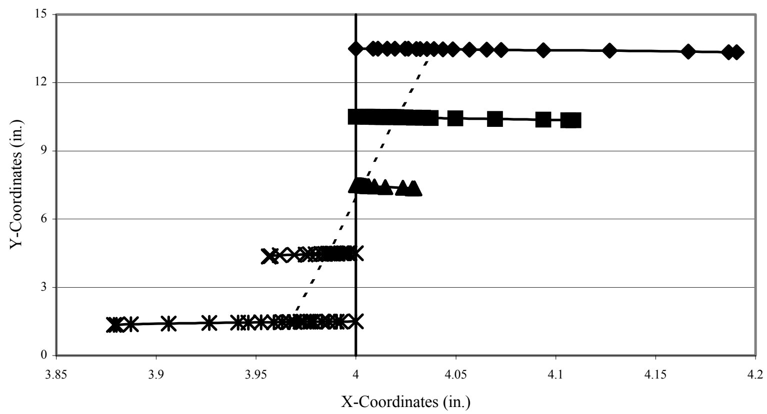
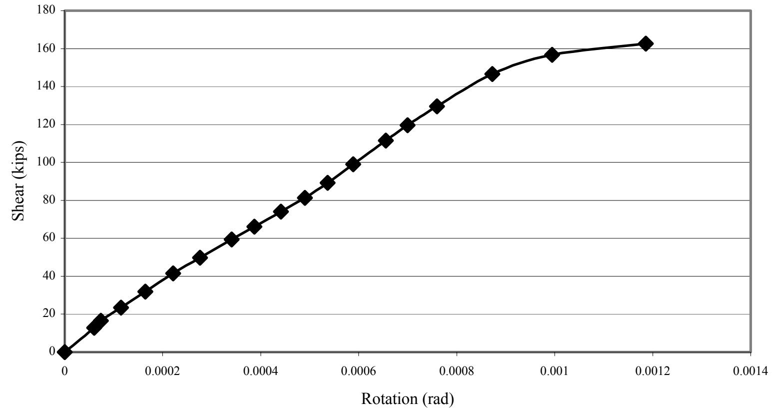
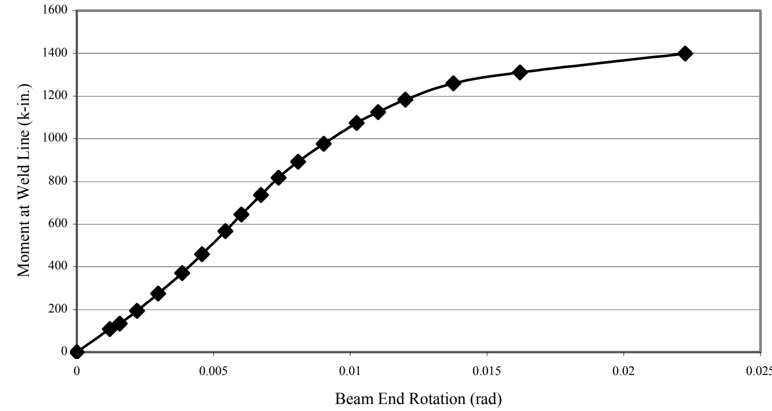


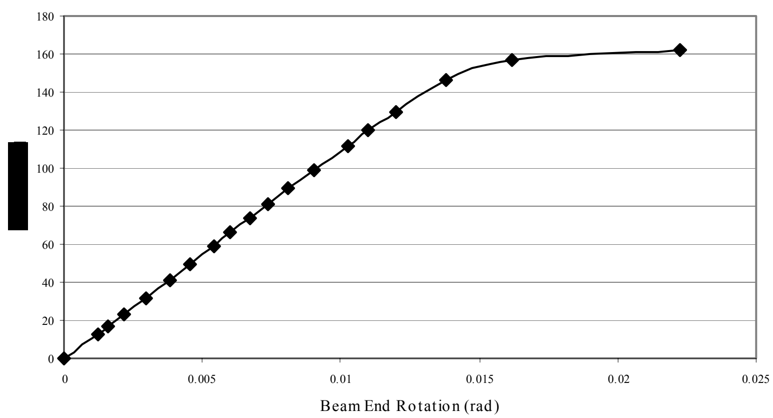
Figure C34 Model 24 - Bolt Movement



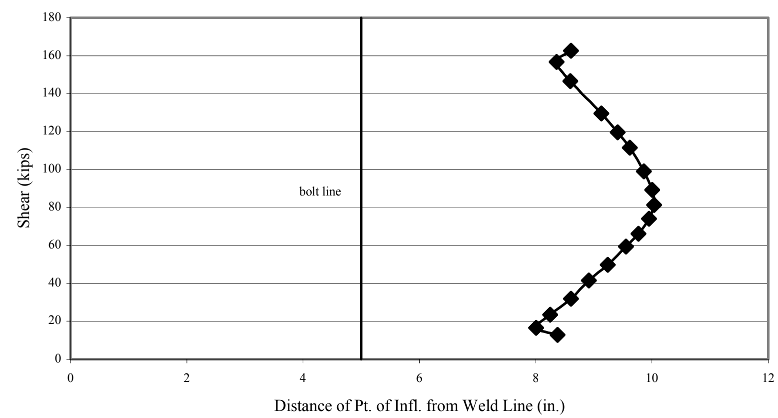
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C35 Results from Model 25

Table C23 Shear Stress in Bolts of Model 25

Increment		48	50	52	54	56	58	60	62	64	66	68	70	72	74
Load		99.07	105.3	111.6	115.2	119.6	124.1	129.6	135.3	138.5	142.5	146.6	151.6	154.4	155.1
1st bolt	beam side	24.45	25.99	27.57	28.40	29.49	30.45	31.25	32.06	32.40	32.87	33.50	33.92	33.76	33.62
	plate side	23.85	25.10	26.44	27.21	28.26	29.35	30.52	31.73	32.30	33.03	33.80	34.68	34.85	34.82
2nd bolt	beam side	29.53	30.01	31.16	32.01	33.15	34.24	35.18	35.76	35.97	36.18	36.38	36.61	36.63	36.62
	plate side	29.76	30.22	31.17	31.95	33.04	34.16	35.29	36.00	36.29	36.58	36.97	37.43	37.56	37.54
3rd bolt	beam side	32.37	34.34	36.00	36.43	36.78	37.13	37.61	38.05	38.25	38.49	38.47	37.98	37.43	37.30
	plate side	32.69	34.84	36.58	37.36	37.88	38.33	39.08	40.29	41.04	41.94	42.72	43.09	43.06	42.97
4th bolt	beam side	28.40	30.75	33.23	34.99	36.63	37.94	39.02	39.82	40.35	40.96	41.33	40.84	39.88	39.58
	plate side	28.48	31.08	33.60	35.33	37.37	38.79	40.59	41.45	42.21	43.58	45.27	47.32	47.65	47.66
5th bolt	beam side	30.53	32.71	34.65	35.63	36.46	37.11	38.23	39.26	39.79	40.17	40.23	39.76	39.10	38.93
	plate side	31.09	33.22	35.41	36.44	37.43	38.28	39.28	40.54	41.59	42.73	43.69	44.55	44.91	44.99
6th bolt	beam side	30.14	30.91	32.15	32.91	33.91	34.81	35.65	36.23	36.45	36.73	36.97	36.99	37.08	37.13
	plate side	30.44	30.98	32.08	32.83	33.69	34.62	35.68	36.63	36.98	37.44	37.89	38.45	38.77	38.87
7th bolt	beam side	24.63	26.27	28.16	29.19	30.45	31.60	32.65	33.52	33.91	34.41	34.97	35.33	35.30	35.29
	plate side	23.93	25.33	26.86	27.69	28.83	30.00	31.28	32.61	33.27	34.13	35.04	35.81	36.06	36.11

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

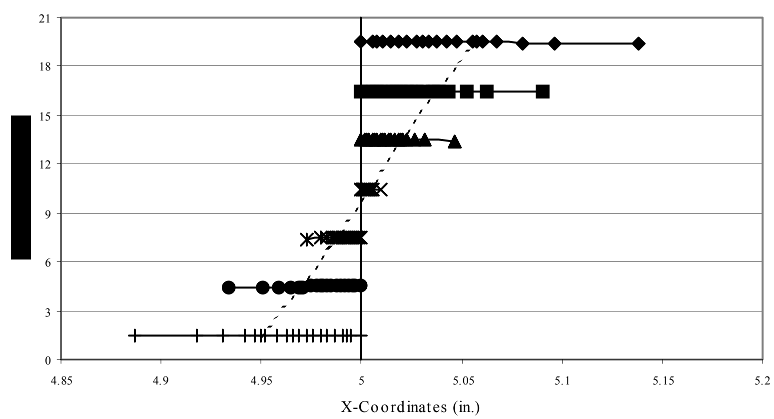
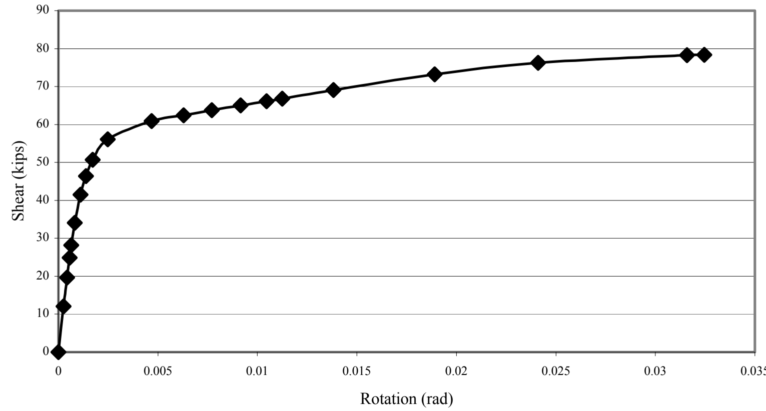
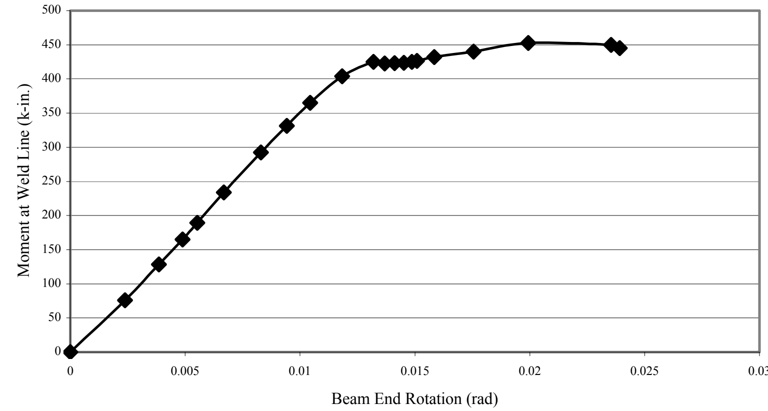


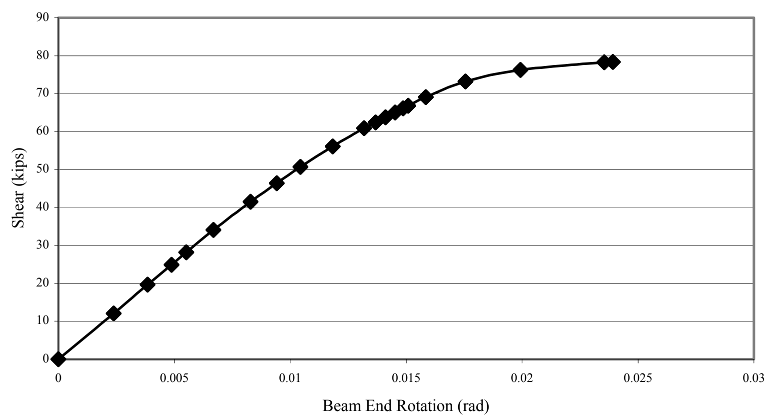
Figure C36 Model 25 - Bolt Movement



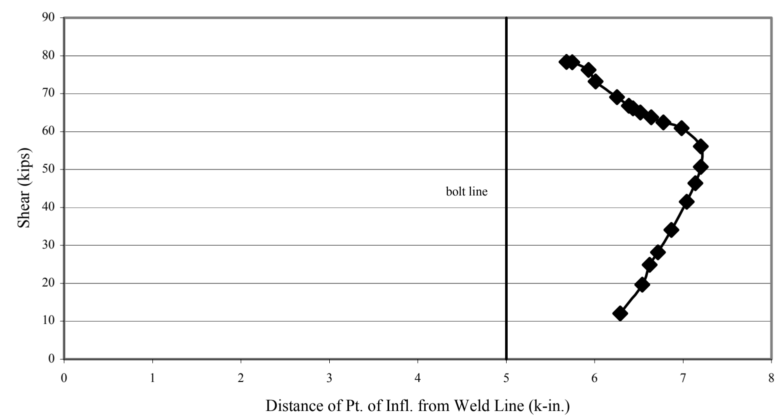
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C37 Results from Model 26

Table C24 Shear Stress in Bolts of Model 26

Increment		36	38	40	42	44	46	48	50	52	54	56	58	60	62
Load		56.10	59.16	60.88	61.84	62.39	63.06	63.75	64.13	65.01	65.50	66.10	66.44	66.79	67.58
1st bolt	beam side	36.05	37.21	38.55	39.16	39.38	39.55	39.65	39.75	39.95	40.09	40.24	40.38	40.59	40.88
	plate side	37.73	38.67	40.10	41.01	41.44	41.86	42.22	42.40	42.78	42.92	43.21	43.43	43.66	44.12
2nd bolt	beam side	37.90	38.64	39.23	39.70	40.03	40.44	40.79	41.03	41.64	41.88	42.08	42.19	42.37	42.70
	plate side	39.73	40.48	41.21	41.79	42.18	42.66	43.12	43.43	44.25	44.60	44.98	45.15	45.33	45.85
3rd bolt	beam side	39.78	40.84	41.05	41.07	41.07	41.14	41.28	41.33	41.47	41.65	41.82	41.85	41.86	41.94
	plate side	40.84	42.72	43.07	43.14	43.17	43.28	43.44	43.53	43.81	44.13	44.65	44.83	44.98	45.40

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

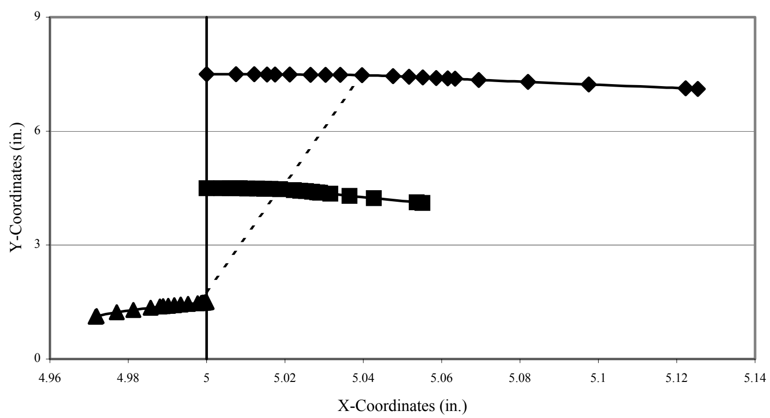
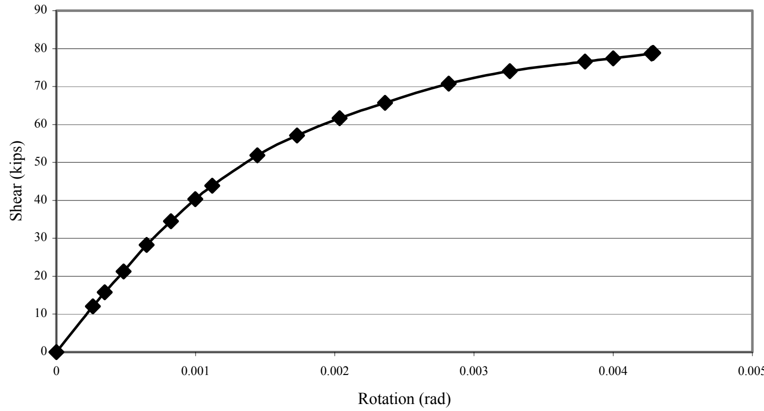
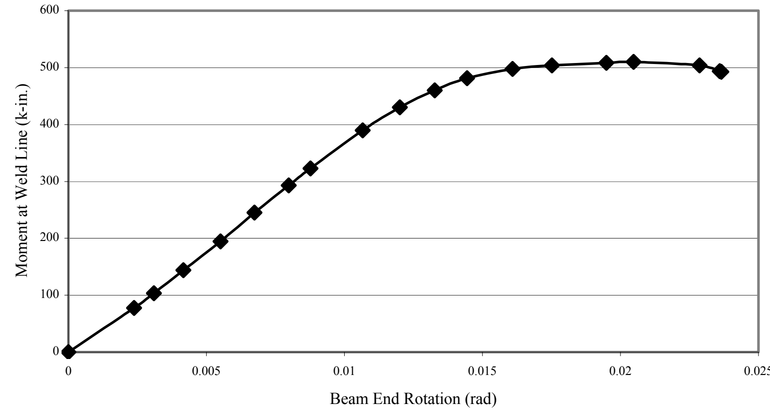


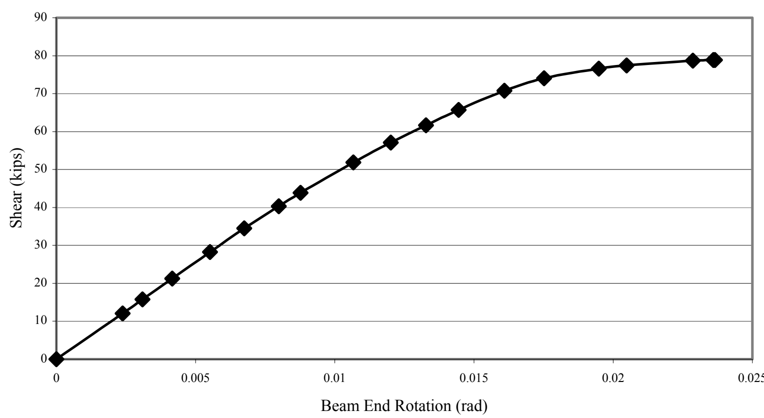
Figure C38 Model 26 - Bolt Movement



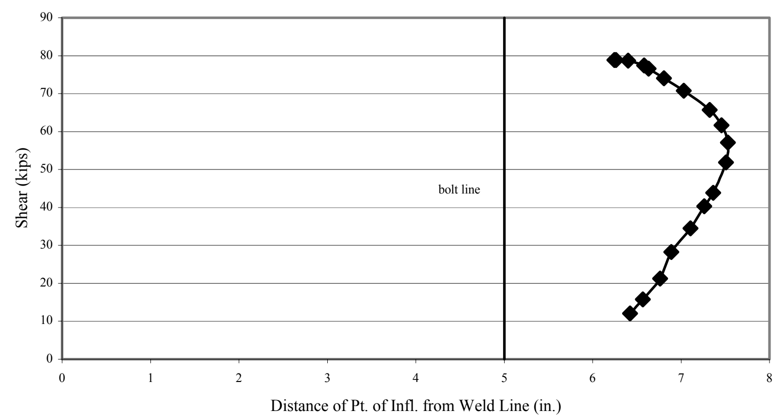
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C39 Results from Model 27

Table C25 Shear Stress in Bolts of Model 27

Increment		22	24	26	28	30	32	34	36	38	40	42	44	46	48
Load		36.31	40.33	42.59	43.87	46.73	51.88	53.49	57.12	59.15	61.68	64.25	65.70	68.97	70.80
1st bolt	beam side	27.42	30.16	30.68	30.80	31.19	31.51	31.65	31.84	31.76	31.91	32.39	32.80	34.49	35.58
	plate side	28.12	31.27	32.31	32.50	33.23	34.01	33.85	33.54	33.32	33.33	33.62	34.06	36.26	37.88
2nd bolt	beam side	27.19	30.14	31.87	32.79	34.89	36.95	37.28	38.11	38.51	38.77	39.09	39.07	38.18	38.26
	plate side	27.35	30.33	32.08	33.05	35.19	38.20	38.90	40.17	41.33	42.33	43.12	43.31	43.49	43.60
3rd bolt	beam side	25.25	27.70	29.12	30.09	32.36	36.77	37.99	40.07	40.80	41.80	42.72	42.78	41.42	40.36
	plate side	25.86	28.41	29.81	30.78	32.99	37.38	38.68	41.17	42.60	43.60	45.31	46.39	47.37	47.18

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

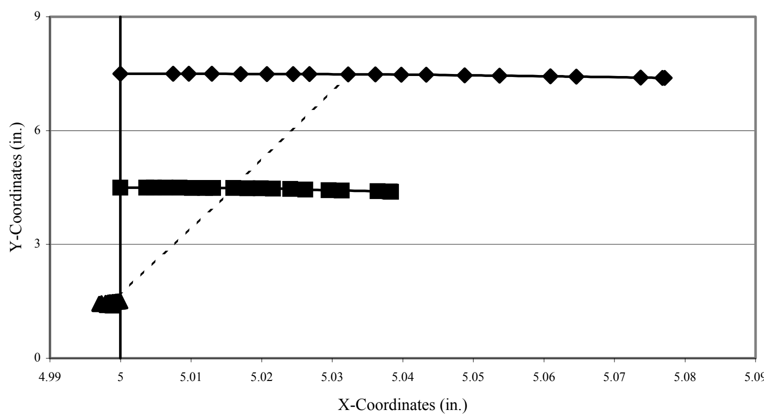
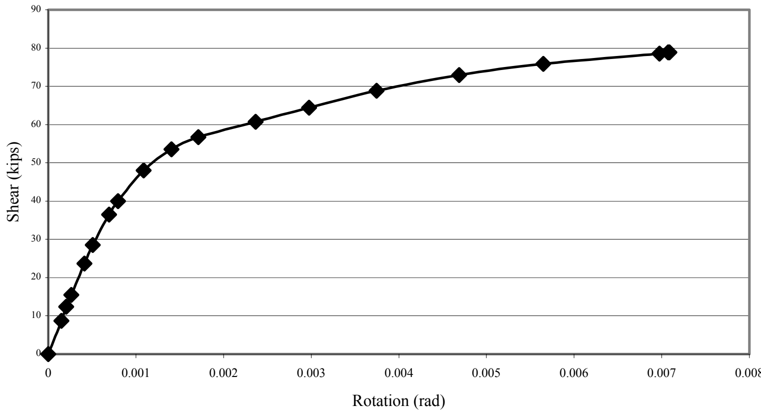
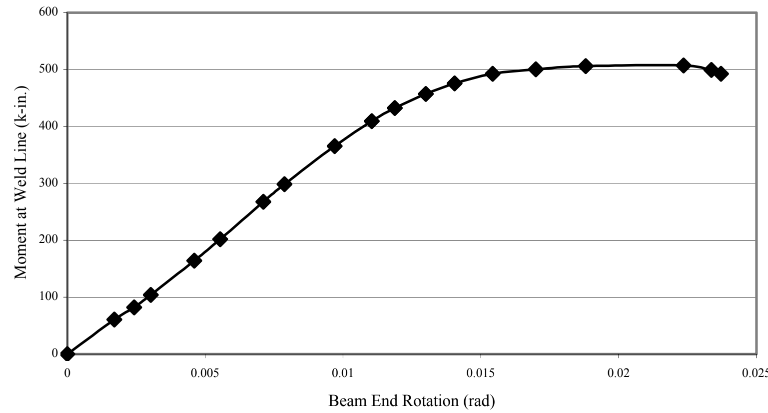


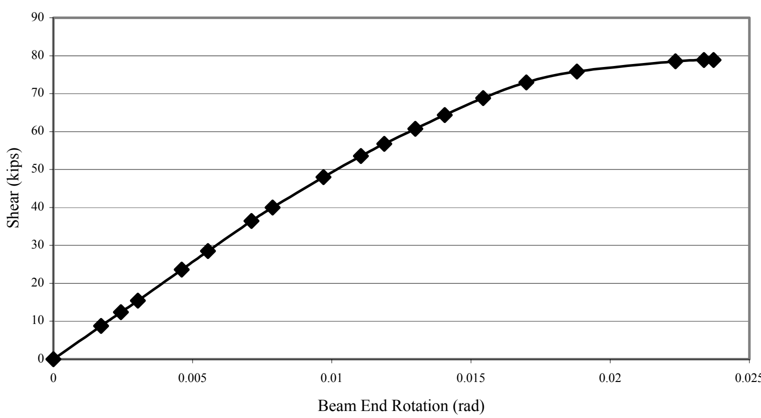
Figure C40 Model 27 - Bolt Movement



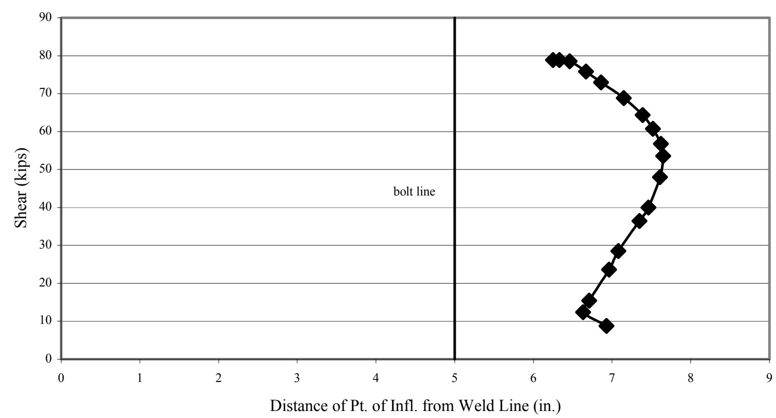
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C41 Results from Model 28

Table C26 Shear Stress in Bolts of Model 28

Increment		30	32	34	36	38	40	42	44	46	48	50	52	54	56
Load		42.86	48.01	51.56	53.55	55.59	56.73	59.31	60.76	62.56	64.39	66.60	68.87	71.19	72.93
1st bolt	beam side	30.21	30.48	30.68	31.10	31.57	31.82	32.68	33.18	33.92	34.59	35.54	36.70	37.93	38.90
	plate side	30.71	31.54	31.69	31.69	31.58	31.62	32.13	32.54	33.12	33.84	35.07	37.02	39.05	40.29
2nd bolt	beam side	32.14	36.24	37.68	38.24	38.77	38.92	39.26	39.37	39.77	40.22	40.63	40.93	41.44	41.98
	plate side	30.93	34.95	37.06	38.16	39.53	39.94	40.79	41.10	41.68	42.23	42.81	43.58	44.23	44.64
3rd bolt	beam side	31.23	35.32	38.12	39.71	40.96	41.42	42.51	43.06	43.78	44.48	44.51	44.14	43.76	43.63
	plate side	31.14	35.11	37.73	39.05	40.41	41.11	42.75	43.63	44.50	45.39	46.30	46.86	47.37	47.54

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

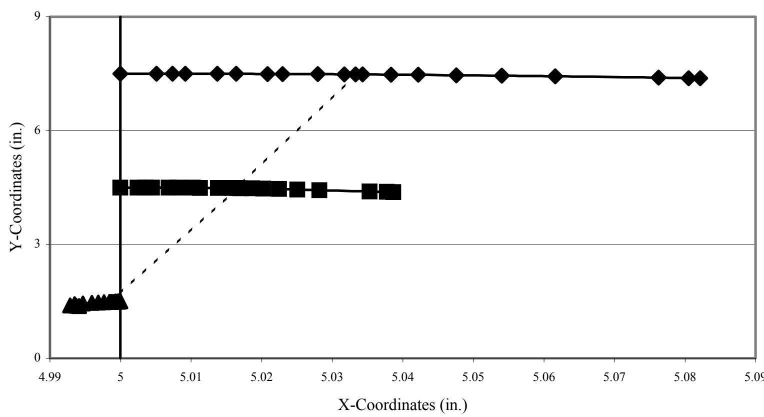
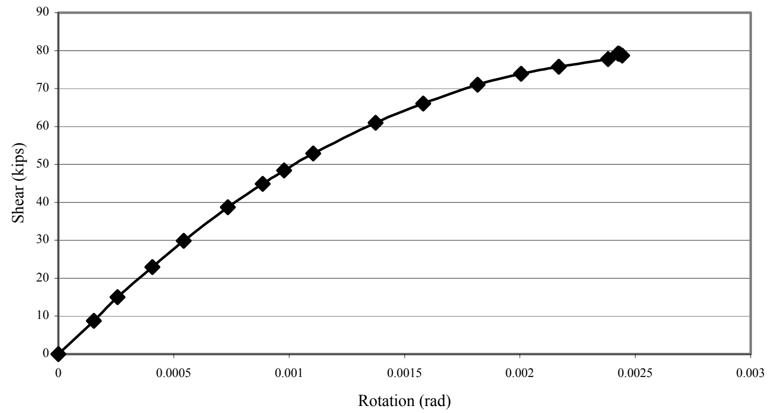
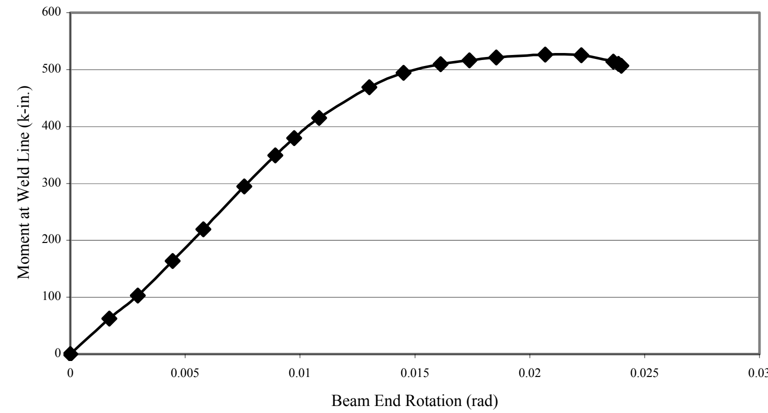


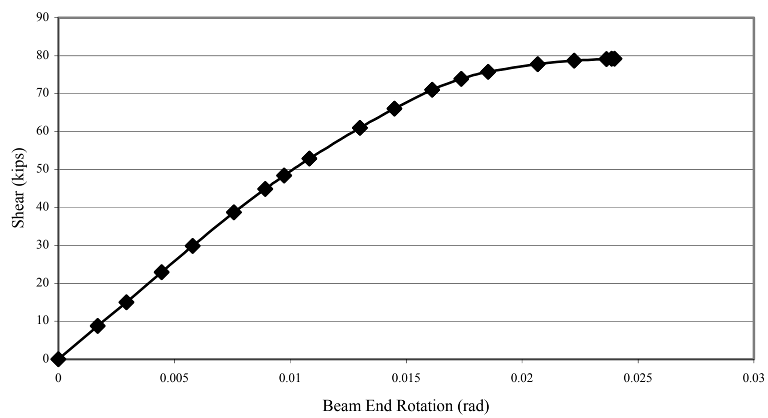
Figure C42 Model 28 - Bolt Movement



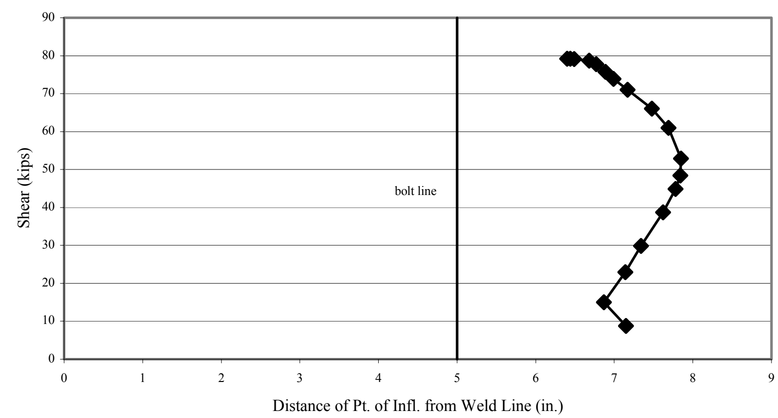
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C43 Results from Model 29

Table C27 Shear Stress in Bolts of Model 29

Increment		16	18	20	22	24	26	28	30	32	34	36	38	40	42
Load		29.88	35.54	38.71	42.65	44.86	47.12	48.39	51.26	52.87	56.49	60.97	63.49	66.07	69.26
1st bolt	beam side	22.65	27.43	28.68	28.63	28.56	28.43	28.28	28.29	28.54	29.11	30.41	31.34	32.71	34.79
	plate side	21.97	26.76	28.43	29.93	30.13	30.30	30.09	29.54	29.28	28.52	28.46	29.14	30.53	33.16
2nd bolt	beam side	21.36	26.07	28.35	31.64	33.62	35.40	35.97	36.88	37.20	37.72	38.06	38.63	39.03	39.39
	plate side	19.78	24.07	26.31	29.53	31.51	33.33	34.18	35.95	36.99	38.86	40.02	40.52	40.98	42.18
3rd bolt	beam side	21.73	24.88	27.36	30.74	32.84	34.91	36.25	38.50	39.74	41.71	43.44	44.67	44.62	43.72
	plate side	19.59	23.05	25.76	29.25	31.48	33.79	35.15	37.82	39.06	41.76	43.85	44.99	46.03	46.52

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

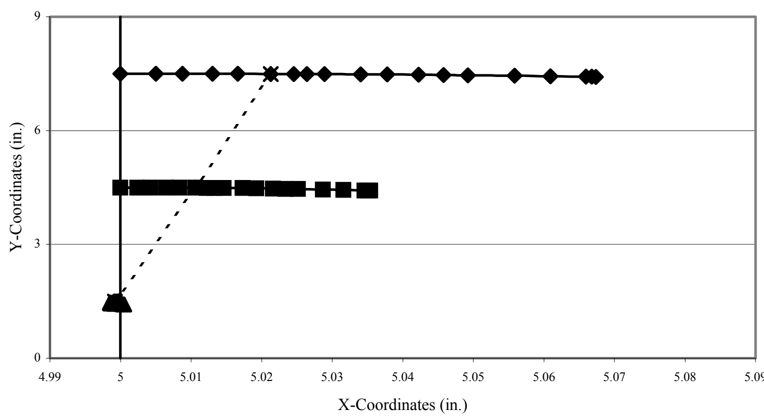
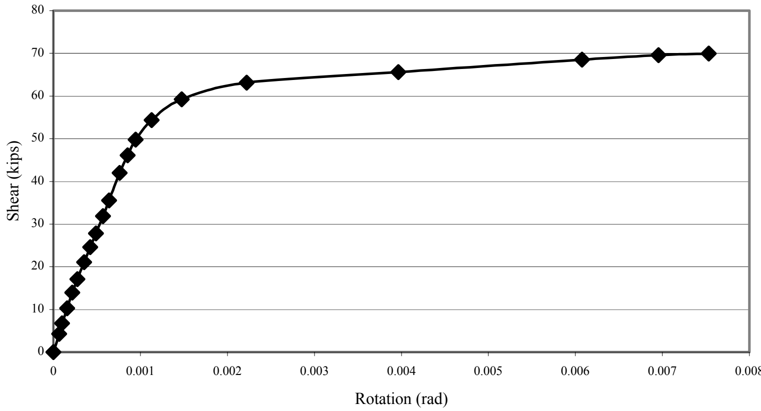
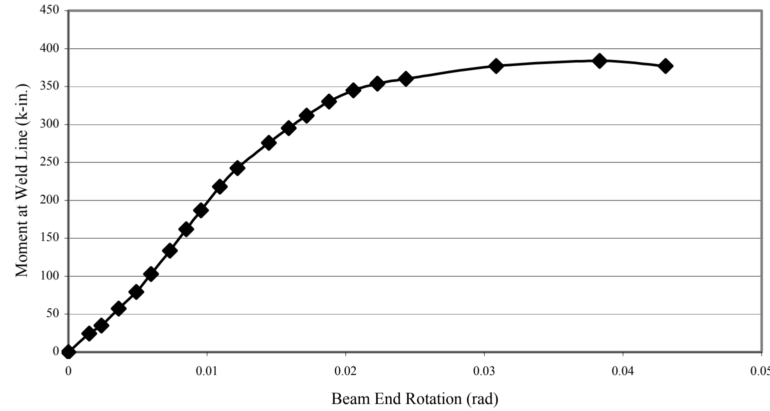


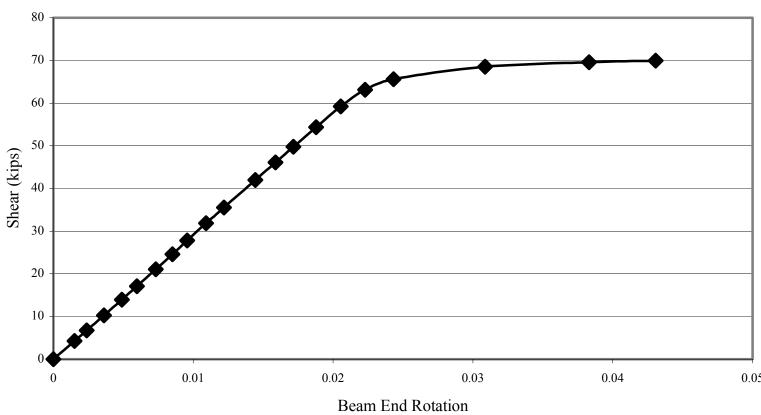
Figure C44 Model 29 - Bolt Movement



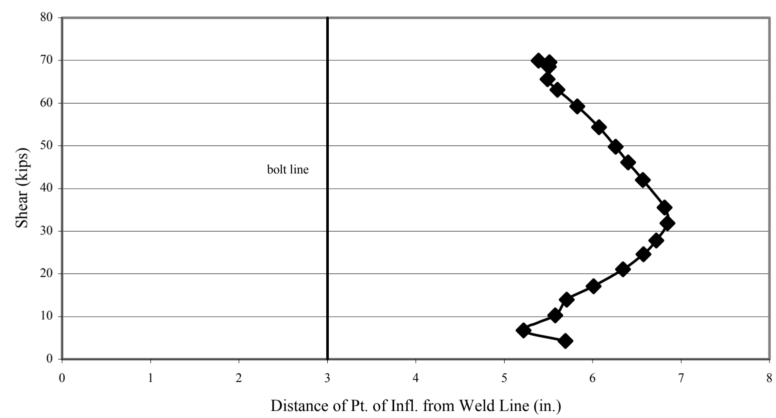
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C45 Results from Model 30

Table C28 Shear Stress in Bolts of Model 30

Increment		50	52	54	56	58	60	62	64	66	68	70	72	74	76
Load		48.45	49.76	52.70	54.35	58.07	59.22	60.40	61.84	62.68	63.14	63.61	64.67	65.26	65.60
1st bolt	beam side	25.93	26.49	28.44	29.55	32.34	33.08	33.70	34.25	34.50	34.62	34.74	34.95	34.81	34.69
	plate side	26.92	27.61	29.67	30.88	33.56	34.23	34.78	35.29	35.53	35.65	35.77	35.98	35.74	35.58
2nd bolt	beam side	34.68	34.92	35.36	35.51	36.47	36.88	37.21	37.41	37.45	37.50	37.59	37.73	37.60	37.49
	plate side	35.55	35.81	36.66	37.01	37.79	38.20	38.52	38.96	39.16	39.37	39.58	39.95	39.97	39.92
3rd bolt	beam side	34.24	35.42	37.64	38.70	40.08	40.52	40.95	41.60	41.99	42.17	42.28	42.49	42.55	42.54
	plate side	34.47	35.63	38.13	39.44	41.56	42.08	42.64	43.54	44.02	44.27	44.58	45.43	46.09	46.46

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

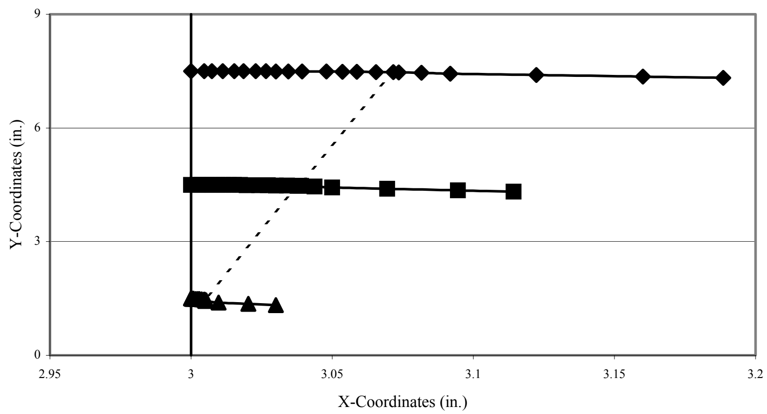
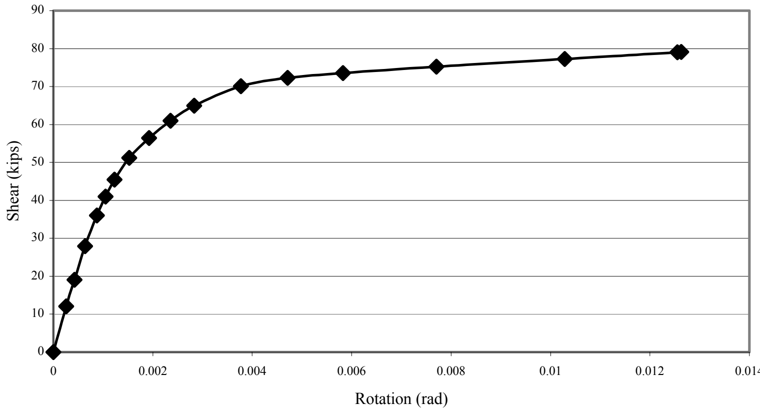
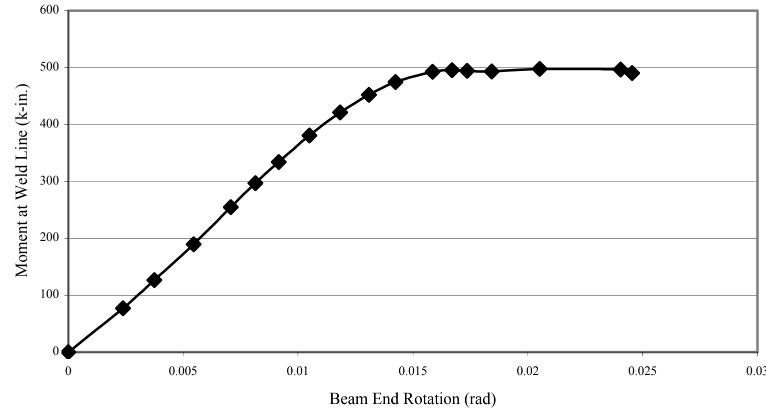


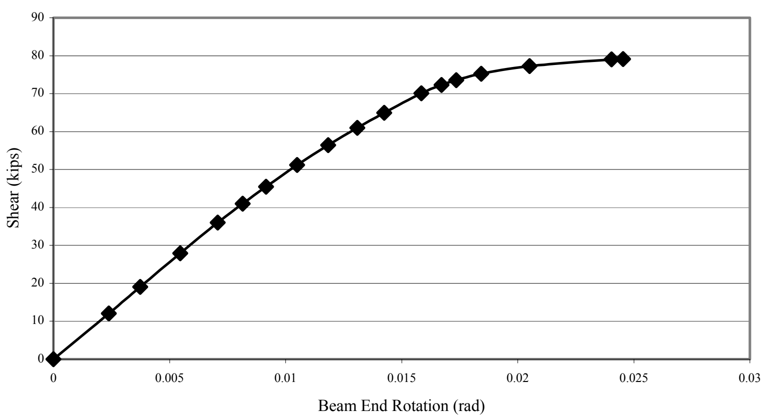
Figure C46 Model 30 - Bolt Movement



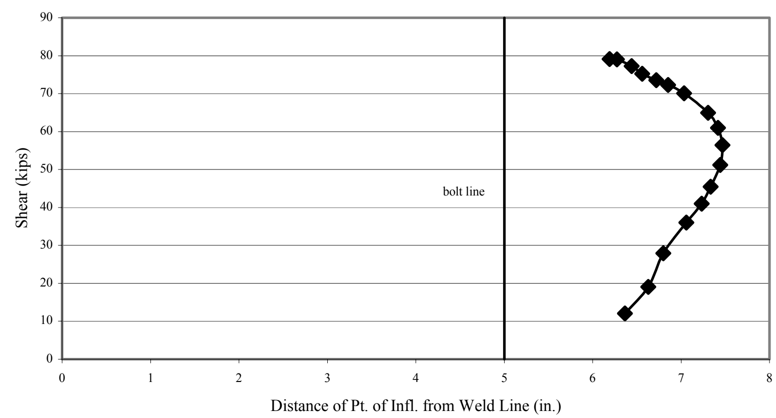
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C47 Results from Model 31

Table C29 Shear Stress in Bolts of Model 31

Increment		24	26	28	30	32	34	36	38	40	42	44	46	48	50
Load		45.50	48.30	51.16	52.77	56.39	58.43	60.95	63.50	64.98	68.25	70.08	71.11	72.27	72.85
1st bolt	beam side	31.44	31.92	32.01	32.16	32.70	32.74	32.81	33.12	33.36	34.52	35.64	36.29	37.23	37.72
	plate side	33.04	33.95	34.49	34.48	34.45	34.35	34.33	34.55	34.82	36.47	38.09	39.05	40.31	40.97
2nd bolt	beam side	33.92	35.82	36.86	37.23	37.98	38.46	38.76	39.16	39.29	38.77	38.56	38.60	38.92	39.17
	plate side	34.20	36.23	37.78	38.58	39.81	40.66	41.90	42.88	43.29	43.63	43.74	43.88	44.10	44.25
3rd bolt	beam side	31.15	33.48	35.98	37.33	39.72	40.47	41.40	42.32	42.76	42.26	41.17	40.56	40.14	39.98
	plate side	31.73	33.99	36.47	37.97	40.74	42.00	43.18	44.40	45.42	47.21	47.32	47.19	46.97	46.83

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

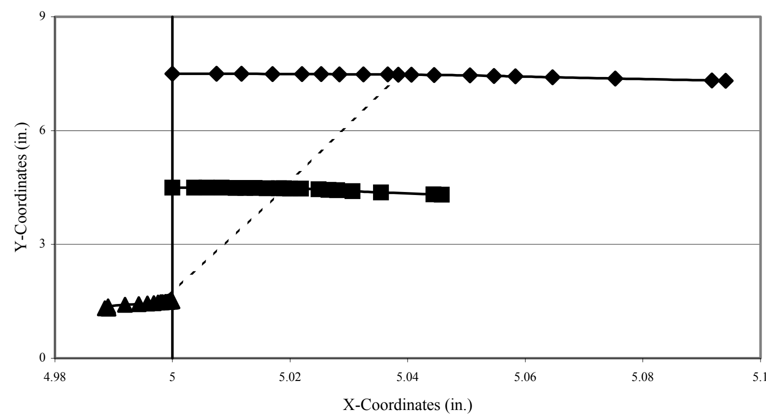
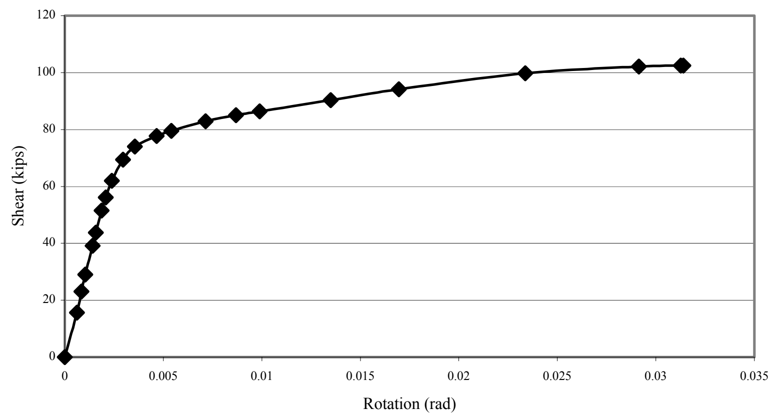
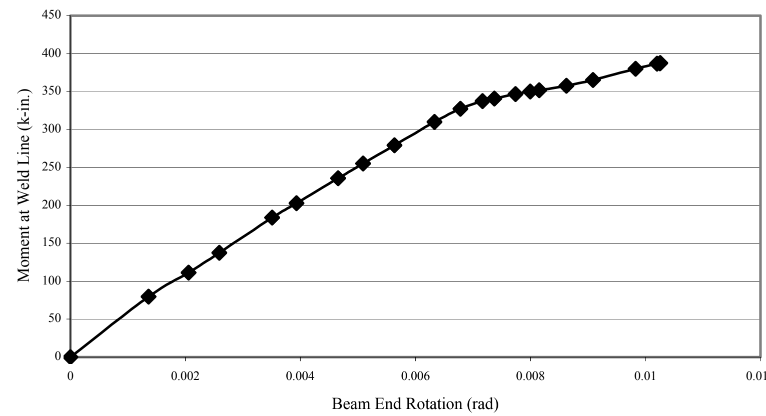


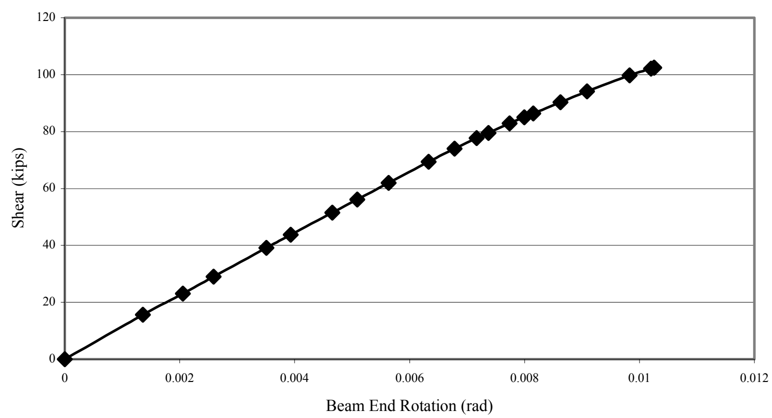
Figure C48 Model 31 - Bolt Movement



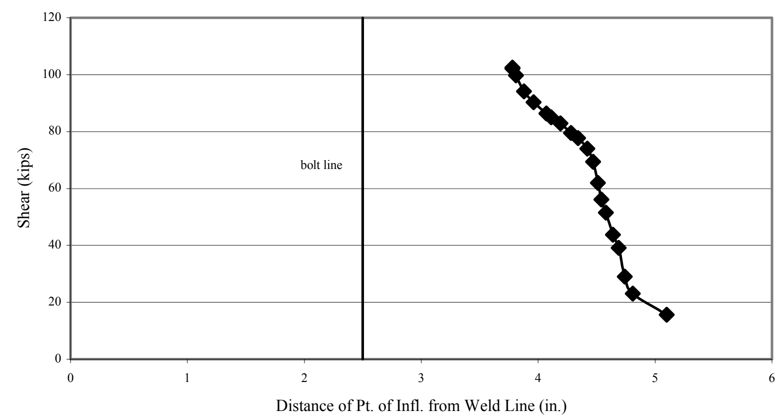
(a) Shear vs. Rotation at Outer Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C49 Results from Model 32

Table C30 Shear Stress in Bolts of Model 32

Increment		48	50	52	54	56	58	60	62	64	66	68	70	72	74
Load		79.54	81.66	82.86	83.53	85.03	85.88	86.36	87.43	88.76	89.51	90.27	90.7	91.67	92.87
1st bolt	beam side	40.65	42.38	43.46	43.82	44.61	45.14	45.40	45.89	46.36	46.48	46.60	46.58	46.54	46.32
	plate side	39.81	41.37	42.41	42.72	43.64	44.31	44.51	44.94	45.42	45.61	45.73	45.79	45.94	46.09
2nd bolt	beam side	41.02	42.17	42.78	43.16	44.03	44.62	44.92	45.62	46.28	46.63	46.88	46.98	47.10	47.16
	plate side	40.58	41.55	42.07	42.36	43.03	43.49	43.90	44.25	44.59	44.79	44.90	44.99	45.21	45.41
1st bolt	beam side	43.23	43.56	43.74	43.94	44.50	44.89	45.02	45.27	45.57	45.77	45.96	46.07	46.30	46.49
	plate side	42.35	42.85	43.16	43.43	43.98	44.18	44.31	44.51	44.69	44.78	44.88	44.94	45.09	45.26
2nd bolt	beam side	42.63	43.09	43.30	43.44	43.92	44.27	44.41	44.65	44.87	45.03	45.20	45.27	45.50	45.63
	plate side	42.59	42.96	43.17	43.29	43.74	43.95	44.02	44.18	44.29	44.37	44.46	44.49	44.68	44.90

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

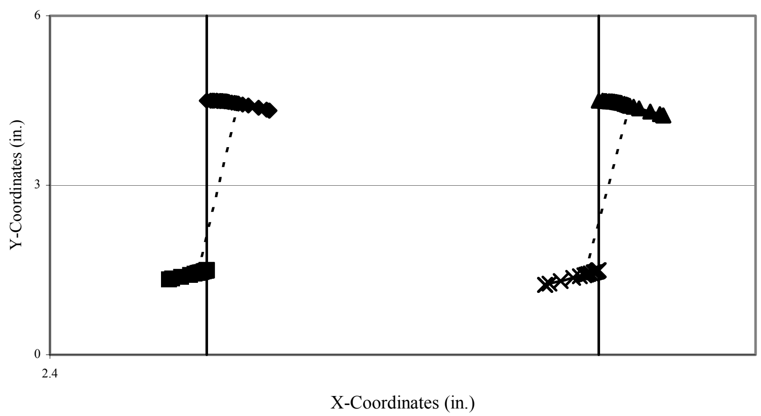
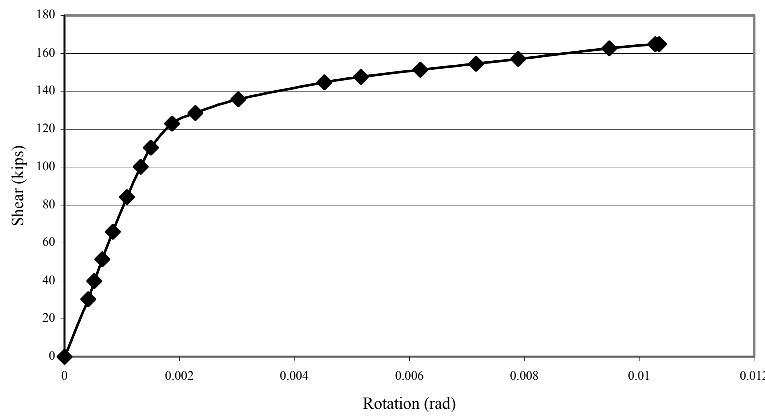
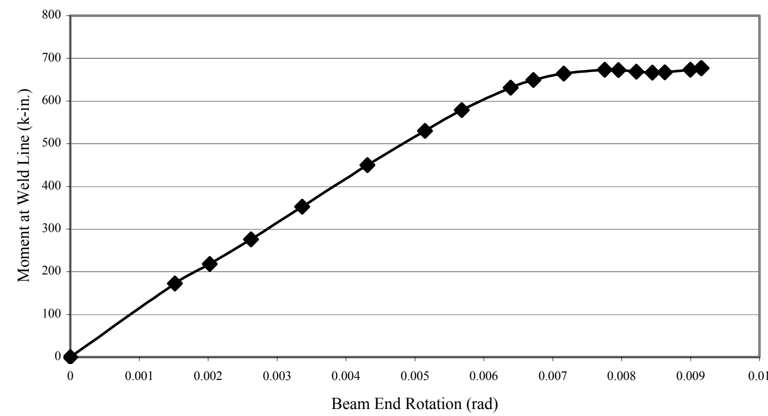


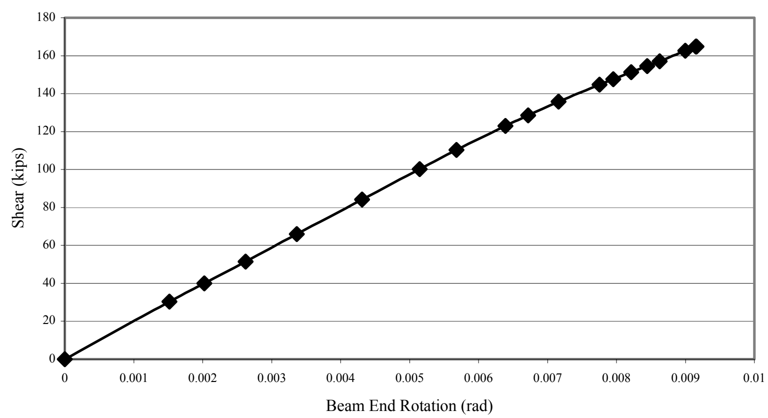
Figure C50 Model 32 - Bolt Movement



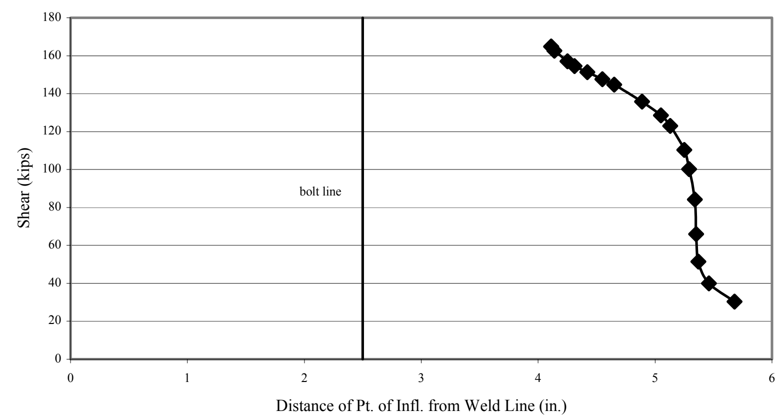
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C51 Results from Model 33

Table C31 Shear Stress in Bolts of Model 33

Increment		24	26	28	30	32	34	36	38	40	42	44	46	48	50
Load		100.2	105.2	110.3	116.5	123.0	126.6	128.6	133.2	135.8	141.6	144.8	146.7	147.7	150.0
1st bolt	beam side	28.84	31.29	33.67	36.29	37.85	38.69	39.22	40.48	41.33	43.51	44.23	44.52	44.54	44.2
	plate side	27.22	29.53	31.76	34.35	36.7	37.94	38.74	40.54	41.17	43.17	44.03	44.58	44.86	45.44
2nd bolt	beam side	28.5	30.58	32.95	35.82	38.95	40.32	40.99	42.32	43.08	44.92	45.23	45.14	45.08	44.46
	plate side	26.64	28.71	30.91	33.81	36.95	38.61	39.36	41.14	41.89	43.88	44.85	45.25	45.38	45.55
3rd bolt	beam side	30.45	32.8	35.21	38.12	40.59	41.54	42.06	42.97	43.38	45.08	45.28	45.2	45.14	44.77
	plate side	28.99	31.43	33.92	37.2	39.61	41.01	41.74	42.65	43.19	44.48	45.18	45.56	45.67	45.86
1st bolt	beam side	40.03	40.64	41.13	41.41	41.75	42.04	42.23	42.32	42.31	42.56	42.69	42.66	42.64	42.62
	plate side	40.33	40.95	41.52	42.18	42.85	43.22	43.46	44.01	44.28	45.01	45.39	45.7	45.86	46.35
2nd bolt	beam side	41.62	42.29	42.77	43.59	44.07	44.06	44.06	44.04	44	43.81	43.67	43.52	43.44	43.28
	plate side	41.02	41.95	42.75	43.56	44.52	44.88	45.04	45.38	45.53	45.82	45.98	46.07	46.11	46.29
3rd bolt	beam side	42.02	42.55	43	43.99	44.7	44.79	44.79	44.79	44.75	44.51	44.2	44.02	43.97	43.81
	plate side	42.31	42.85	43.51	44.21	44.99	45.33	45.49	45.86	46.03	46.28	46.41	46.51	46.53	46.66

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

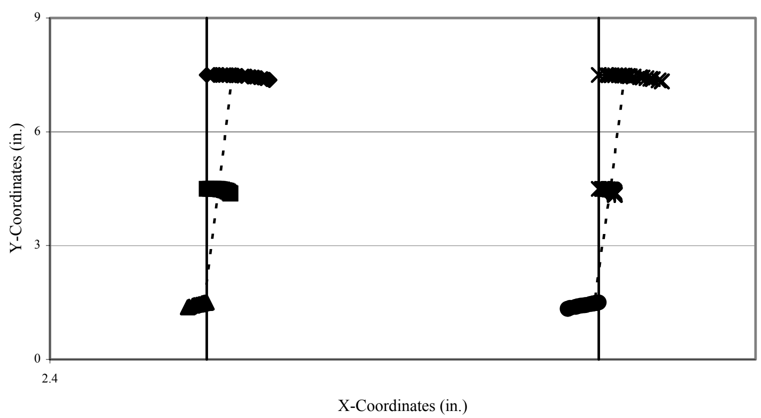
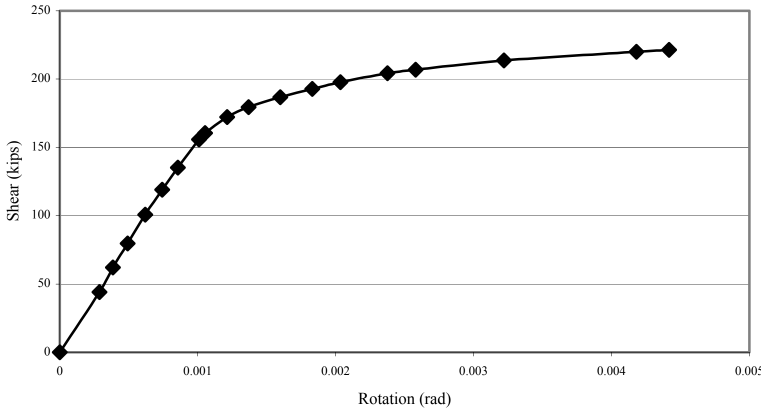
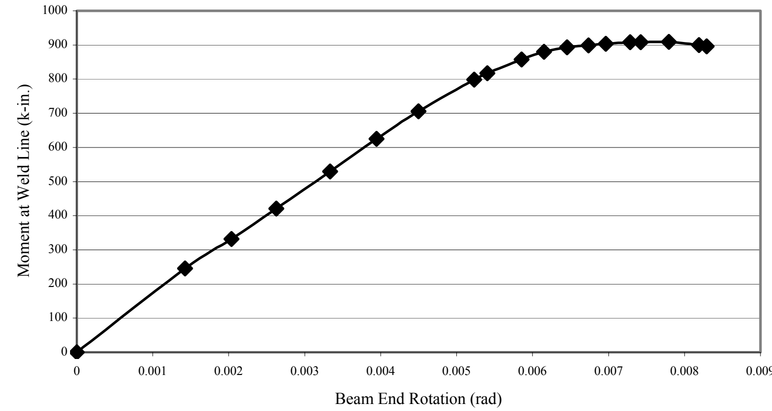


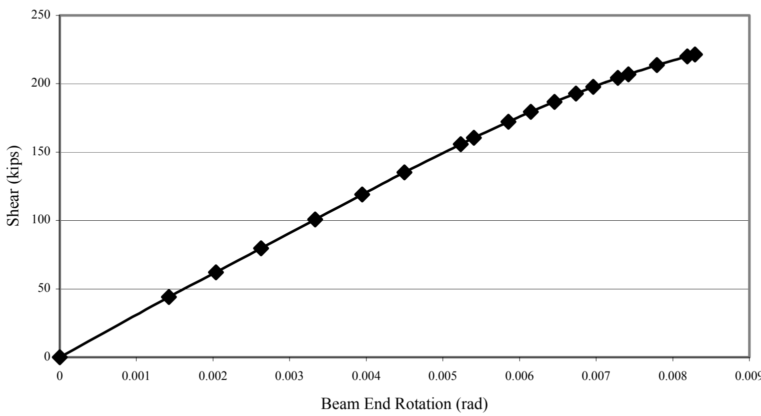
Figure C52 Model 33 - Bolt Movement



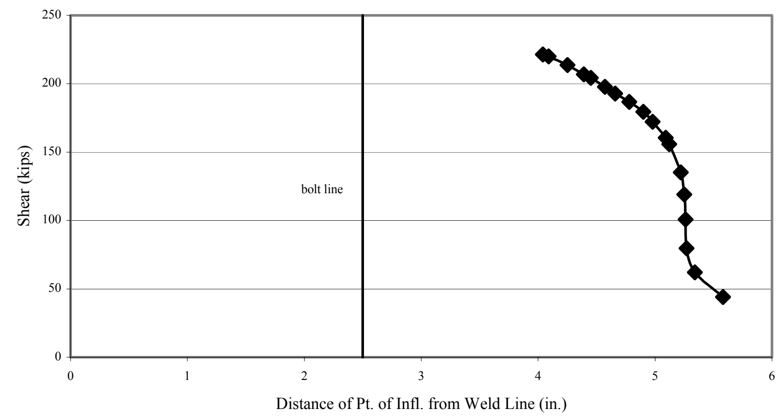
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C53 Results from Model 34

Table C32 Shear Stress in Bolts of Model 34

Increment		24	26	28	30	32	34	36	38	40	42	44	46	48	50
Load		135.3	148.4	155.8	158.1	160.4	165.7	172.2	175.8	179.6	184.2	186.7	191.5	192.9	194.4
1st bolt	beam side	27.46	32.69	35.40	36.15	36.91	37.81	38.81	39.31	39.50	40.19	40.68	41.49	41.76	42.12
	plate side	26.03	30.97	33.65	34.34	35.05	36.60	38.44	39.20	39.83	41.01	41.59	42.75	43.30	43.54
2nd bolt	beam side	27.25	31.19	33.84	34.74	35.72	37.52	39.75	40.59	41.40	42.33	42.83	44.36	44.62	44.73
	plate side	25.55	29.43	32.11	32.97	33.87	35.93	38.13	39.46	40.56	41.68	42.37	43.60	44.03	44.43
3rd bolt	beam side	28.32	32.89	35.73	36.58	37.40	39.27	40.85	41.55	42.19	43.02	43.58	44.01	43.98	43.97
	plate side	26.86	31.59	34.45	35.35	36.26	37.93	40.01	41.12	42.01	43.06	43.58	44.45	44.67	44.87
4th bolt	beam side	31.24	35.50	37.62	38.32	38.89	39.90	41.28	41.84	42.53	43.35	43.60	43.82	43.90	43.95
	plate side	28.98	33.00	35.35	36.11	36.83	38.69	40.85	41.83	42.52	43.45	43.95	44.81	45.11	45.35
1st bolt	beam side	40.24	40.86	41.04	41.09	41.11	41.17	41.24	41.20	40.96	40.78	40.76	40.86	40.89	40.89
	plate side	40.94	42.11	42.82	43.01	43.17	43.49	43.72	43.90	44.15	44.52	44.75	45.19	45.36	45.53
2nd bolt	beam side	41.72	43.00	43.69	43.85	43.87	43.85	43.80	43.78	43.65	43.50	43.46	43.30	43.23	43.16
	plate side	41.61	43.20	43.91	44.14	44.41	44.86	45.35	45.57	45.76	45.95	46.06	46.24	46.30	46.37
3rd bolt	beam side	41.79	42.80	43.67	43.95	44.18	44.56	44.73	44.73	44.68	44.47	44.34	44.05	43.91	43.76
	plate side	41.99	43.14	43.72	43.92	44.15	44.63	45.10	45.38	45.66	45.86	45.97	46.17	46.22	46.29
4th bolt	beam side	41.79	43.19	43.86	43.99	44.07	44.13	44.22	44.23	44.20	44.17	44.12	43.83	43.72	43.62
	plate side	41.83	43.48	44.23	44.40	44.60	45.03	45.57	45.77	45.99	46.22	46.32	46.53	46.60	46.67

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

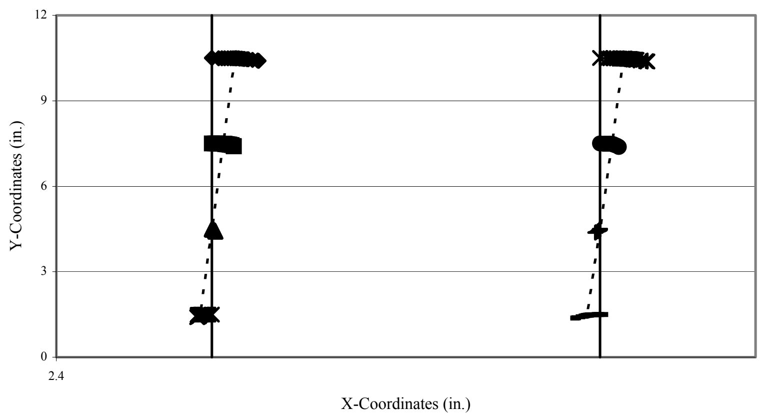
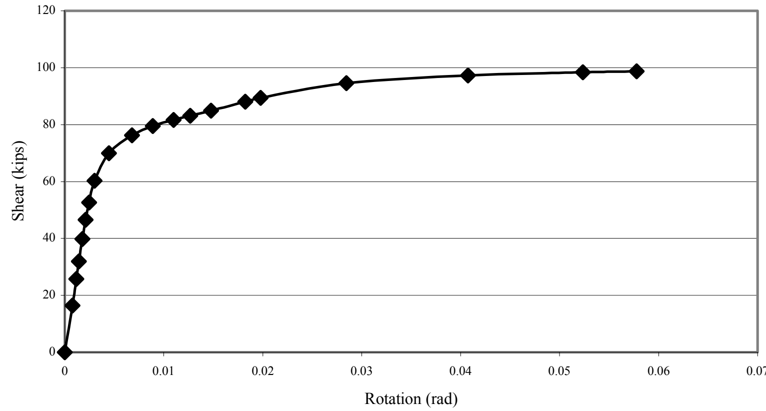
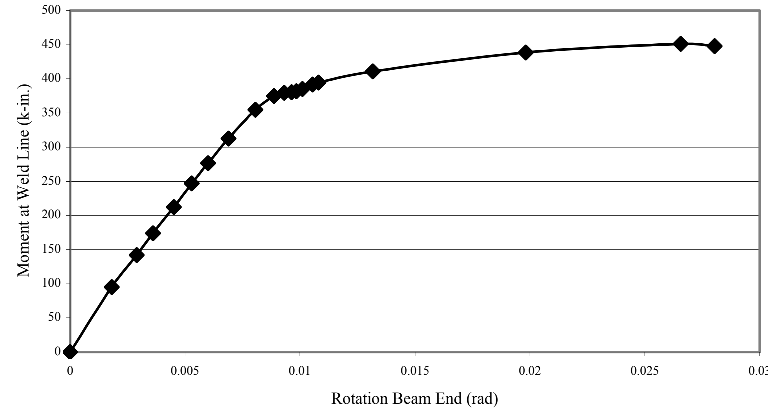


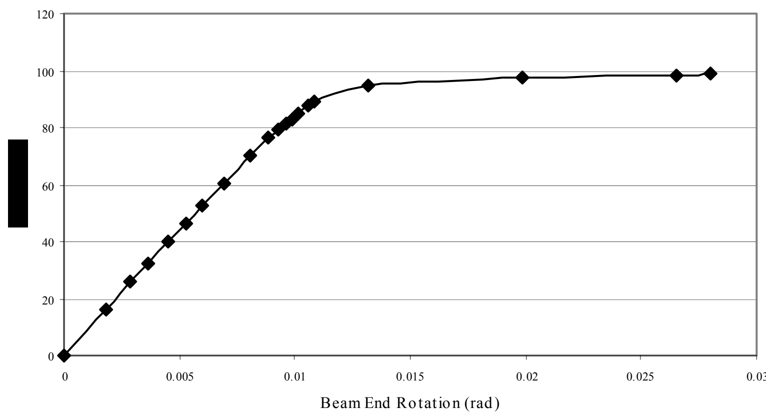
Figure C54 Model 34 - Bolt Movement



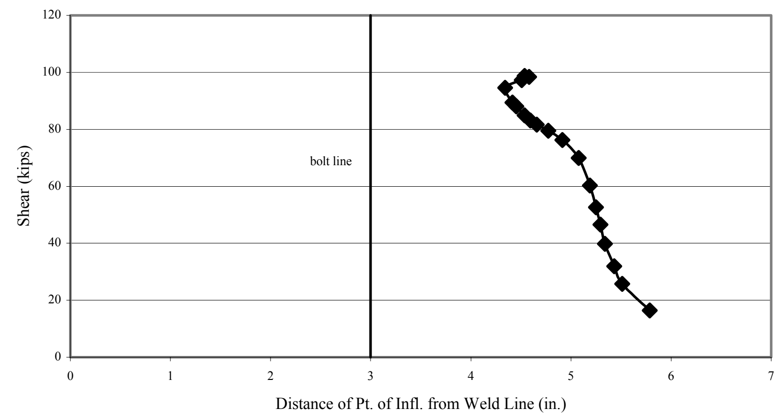
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C55 Results from Model 35

Table C33 Shear Stress in Bolts of Model 35

Increment		32	34	36	38	40	42	44	46	48	50	52	54	56	58
Load		69.91	71.86	76.26*	78.73	79.47	80.86	81.64	82.08	83.07	83.63	84.88	86.43	88.01	88.49
1st bolt	beam side	35.31	36.84	39.13	41.05	41.76	43.00	43.72	43.98	44.52	44.92	45.60	46.15	46.31	46.30
	plate side	33.28	34.94	37.83	40.29	41.03	42.10	42.74	43.04	43.70	44.26	44.82	45.37	45.88	45.99
2nd bolt	beam side	35.59	36.98	39.72	41.32	41.89	43.11	43.60	43.94	44.51	44.47	45.40	45.99	46.29	46.32
	plate side	34.14	35.61	39.14	40.87	41.48	42.56	42.97	43.30	43.74	43.88	44.21	44.58	44.92	45.00
1st bolt	beam side	40.79	41.31	42.30	42.66	42.75	42.89	42.94	42.98	43.04	43.10	43.26	43.65	44.32	44.46
	plate side	39.62	40.17	41.31	41.95	42.16	42.55	42.77	42.91	43.13	43.25	43.47	43.78	44.12	44.21
2nd bolt	beam side	40.81	41.19	41.76	41.58	41.51	41.42	41.43	41.49	41.72	41.90	42.34	42.57	42.82	42.94
	plate side	40.60	41.02	41.62	41.49	41.44	41.47	41.57	41.65	41.87	42.05	42.59	42.85	43.12	43.22

* Indicates that the force redistribution occurs

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

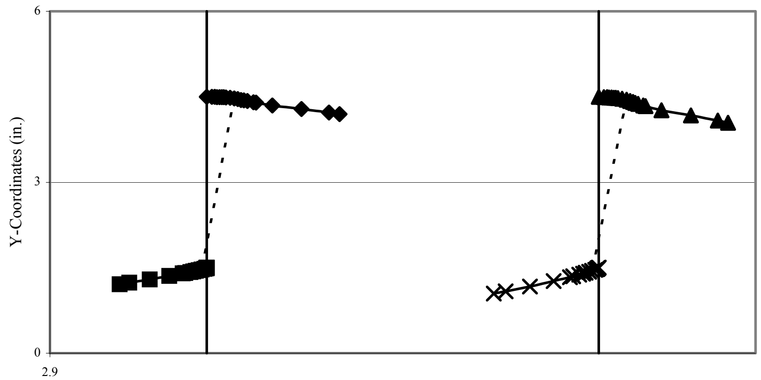
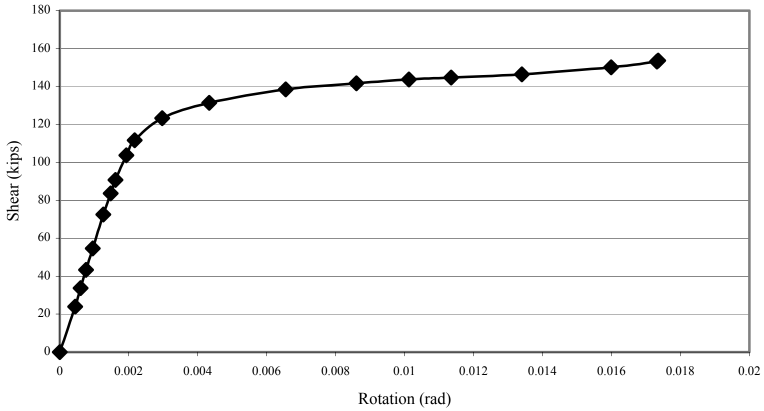
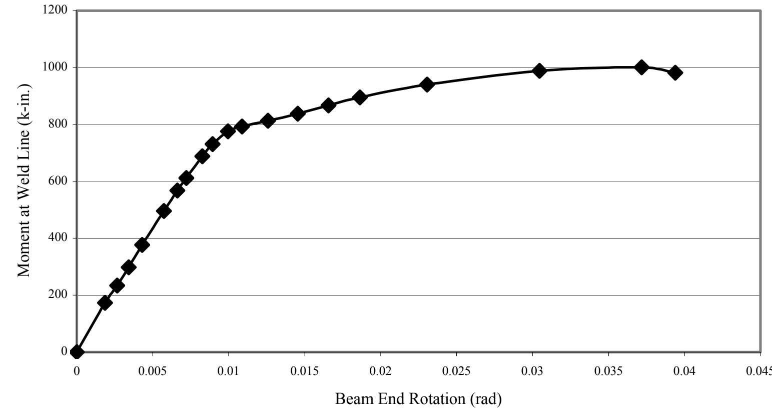


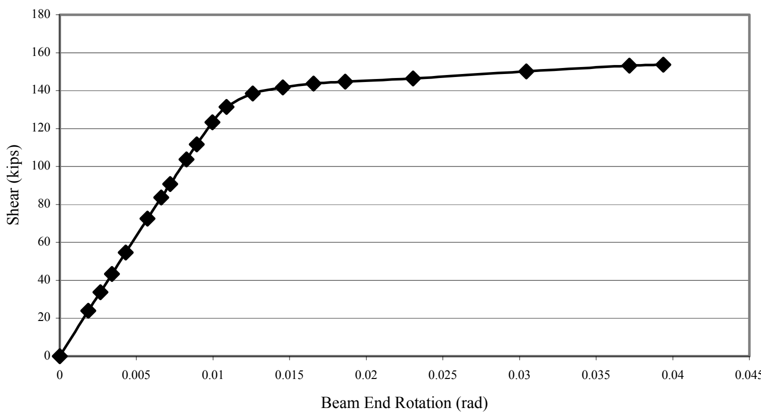
Figure C56 Model 35 - Bolt Movement



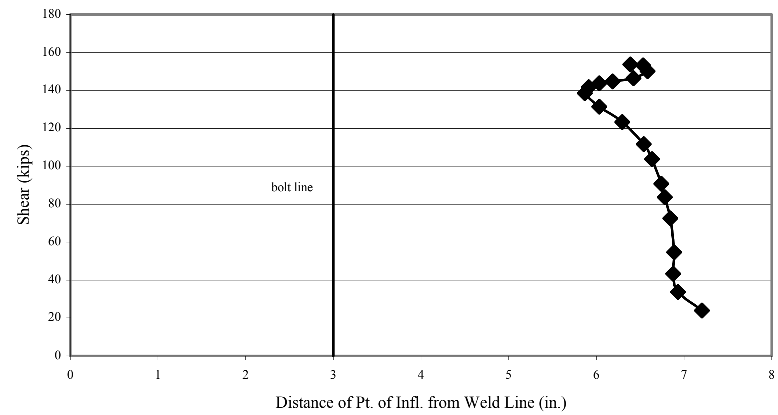
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C57 Results from Model 36

Table C34 Shear Stress in Bolts of Model 36

Increment		18	20	22	24	26	28	30	32	34	36	38	40	42	44
Load		64.53	72.55	78.06	83.69	86.86	90.78	94.79	103.8	108.9	111.7	116.9	123.3	126.9	131.4
1st bolt	beam side	10.91	13.90	16.08	18.44	19.70	21.44	23.63	28.70	31.80	32.38	34.31	36.80	37.71	38.43
	plate side	10.00	13.03	15.11	17.56	18.78	20.52	22.57	27.35	29.91	31.28	34.08	36.37	37.43	38.45
2nd bolt	beam side	9.676	12.19	14.68	17.47	19.09	21.33	23.72	27.99	30.35	31.89	34.61	38.06	39.58	41.37
	plate side	9.327	11.83	13.99	16.46	17.84	19.64	21.62	25.56	27.95	29.48	32.14	35.76	37.88	40.16
3rd bolt	beam side	12.84	16.16	18.39	19.96	20.86	22.30	23.54	27.62	30.63	32.34	35.68	39.10	40.75	42.17
	plate side	12.25	14.78	16.48	17.84	18.81	20.16	21.53	26.02	29.20	31.04	34.58	37.96	39.67	41.74
1st bolt	beam side	36.16	36.99	37.24	37.25	37.25	37.26	37.14	37.20	37.37	37.49	37.65	38.19	38.88	39.76
	plate side	34.68	36.49	37.67	38.22	38.55	38.84	39.04	39.25	39.31	39.33	39.39	40.09	40.73	41.80
2nd bolt	beam side	35.29	38.28	39.62	40.62	41.14	41.54	41.89	42.96	43.22	43.25	43.03	42.81	42.66	42.59
	plate side	33.09	36.18	37.92	39.70	40.60	41.23	41.90	43.16	43.66	43.90	44.37	44.49	44.58	44.73
3rd bolt	beam side	35.91	38.66	40.20	41.19	41.69	42.18	42.61	43.68	44.41	44.63	44.72	44.62	44.49	44.27
	plate side	34.98	38.11	39.68	41.28	41.97	42.50	42.96	44.03	44.60	44.96	45.40	45.83	45.97	46.08

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

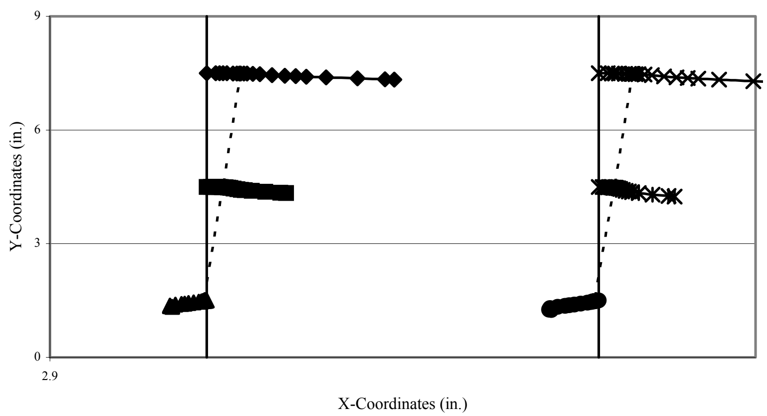
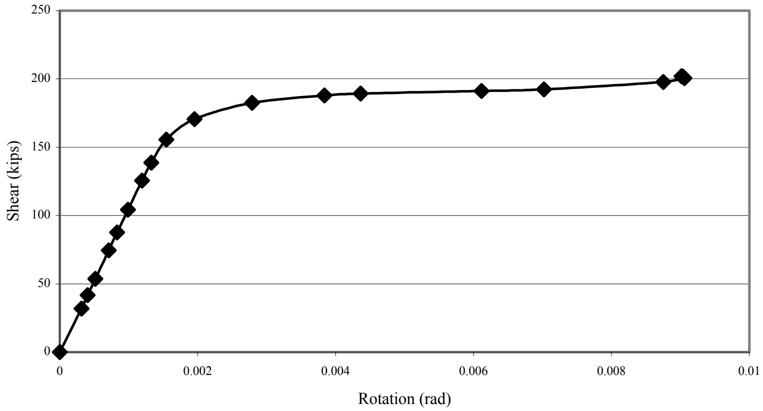
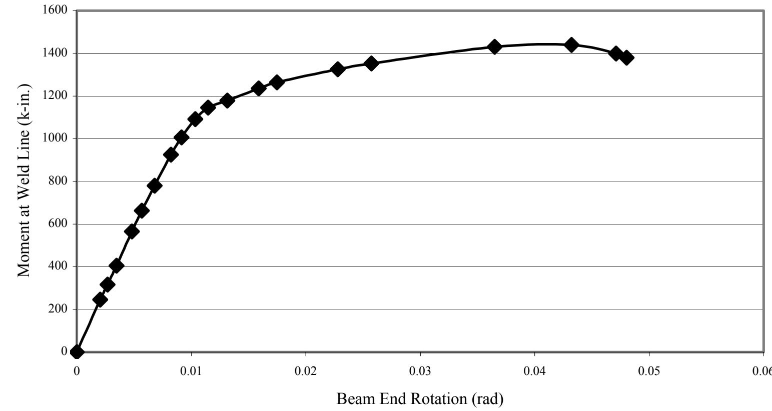


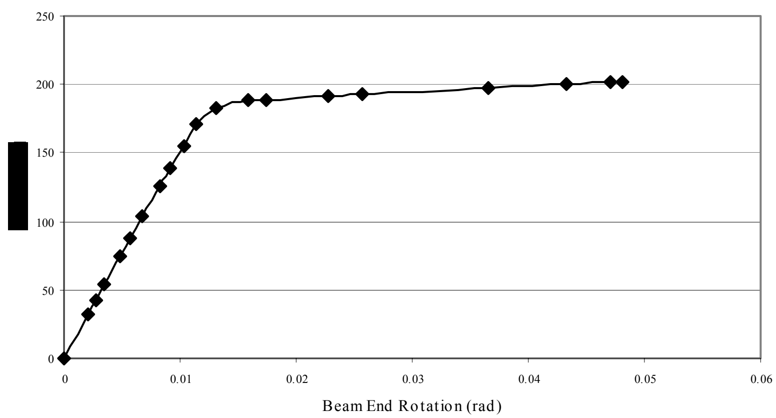
Figure C58 Model 36 - Bolt Movement



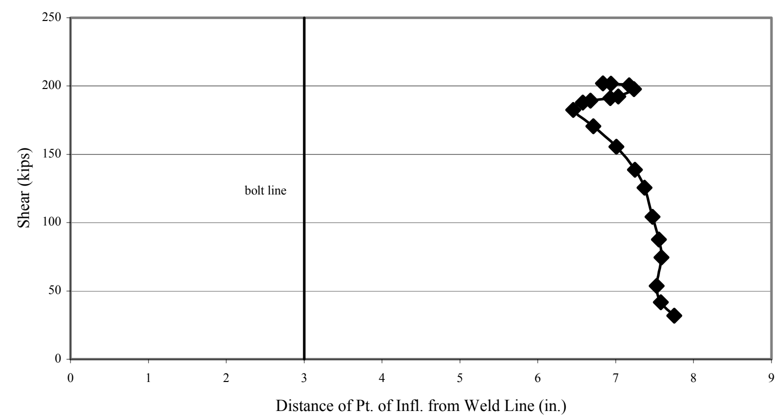
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C59 Results from Model 37

Table C35 Shear Stress in Bolts of Model 37

Increment		16	18	20	22	24	26	28	30	32	34	36	38	40	42
Load		74.48	80.38	87.68	95.14	104.4	113.8	125.5	132.1	138.8	147.1	155.6	165.9	170.6	176.6
1st bolt	beam side	6.294	8.108	10.23	12.21	14.82	17.85	21.56	22.10	23.08	25.31	28.29	32.13	34.20	36.08
	plate side	5.553	7.278	9.211	11.24	13.74	16.61	20.67	22.27	23.94	26.18	28.76	31.90	33.57	35.03
2nd bolt	beam side	3.055	3.849	4.934	6.369	8.952	12.04	17.27	20.68	24.47	29.07	33.47	37.71	38.59	39.82
	plate side	2.994	3.761	4.814	6.202	8.532	11.47	16.31	19.40	22.74	27.04	31.33	35.61	37.22	39.24
3rd bolt	beam side	5.362	6.545	8.069	9.853	12.45	15.50	19.77	21.98	24.27	27.05	30.65	35.87	38.43	40.74
	plate side	5.375	6.506	7.982	9.674	12.02	14.59	18.05	20.17	22.40	25.49	29.43	34.50	37.05	39.70
4th bolt	beam side	4.871	6.006	7.872	10.12	12.95	16.04	20.65	23.60	27.10	31.97	30.00	38.57	39.20	39.79
	plate side	5.033	6.192	7.926	9.878	12.34	15.42	20.06	22.98	26.38	30.73	34.45	37.68	38.43	39.10
1st bolt	beam side	36.20	36.75	37.05	36.91	36.30	36.02	35.85	35.73	35.69	35.68	35.73	36.03	36.39	37.01
	plate side	35.13	36.33	37.61	37.99	38.15	38.25	38.01	37.80	37.61	37.58	37.68	37.99	38.35	39.13
2nd bolt	beam side	35.04	36.88	38.67	39.86	40.87	41.59	41.82	41.71	41.61	41.51	41.12	40.79	40.79	40.92
	plate side	32.91	34.95	36.99	39.03	40.91	42.21	43.17	43.51	43.73	44.01	44.23	44.39	44.45	44.70
3rd bolt	beam side	34.98	36.97	38.85	40.40	41.47	42.28	43.50	44.02	44.37	44.37	44.10	43.60	43.35	43.07
	plate side	34.06	36.31	38.40	40.19	41.72	42.65	43.72	44.28	44.71	45.23	45.73	46.03	46.13	46.28
4th bolt	beam side	35.31	37.04	38.71	39.73	40.69	41.58	41.41	42.42	42.38	42.24	42.08	42.14	42.21	42.03
	plate side	33.07	34.92	36.76	38.68	40.39	41.58	42.88	43.35	43.77	44.29	44.64	44.98	45.17	45.39

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

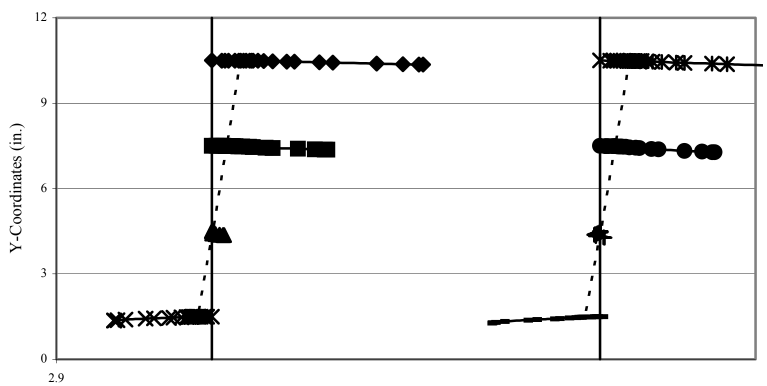
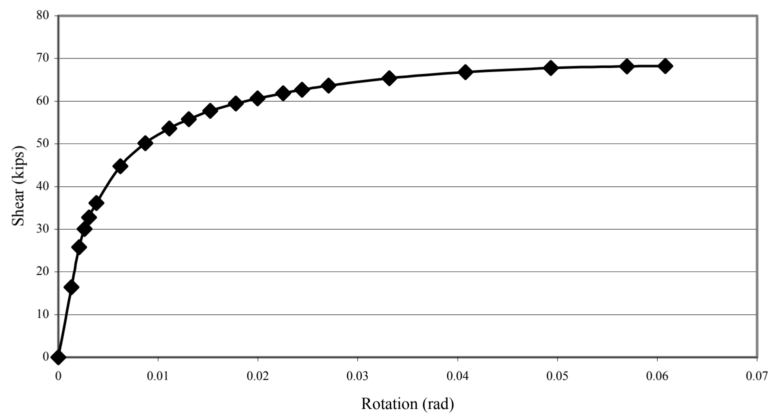
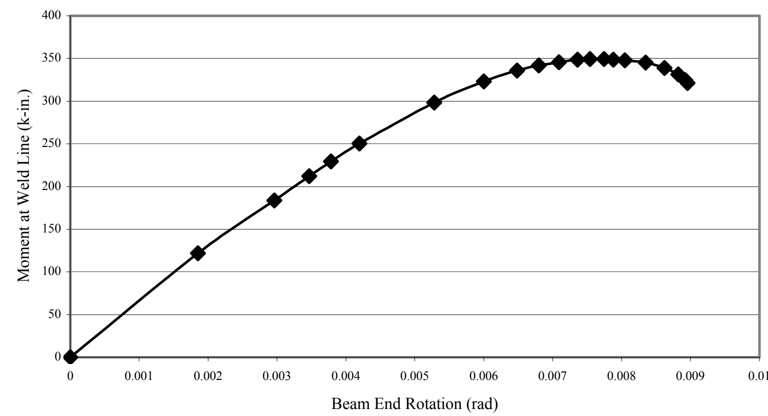


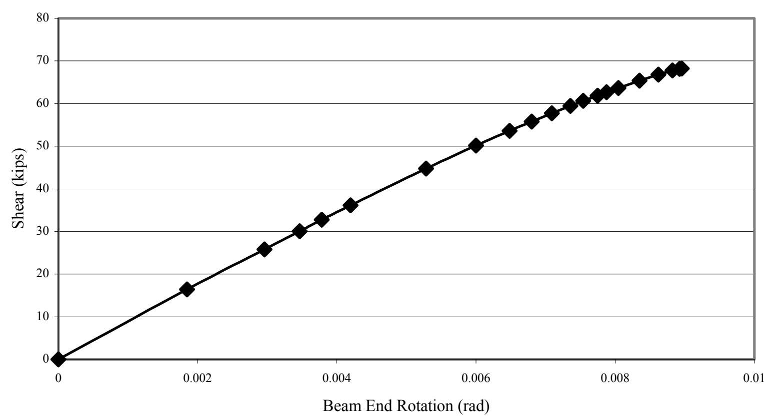
Figure C60 Model 37 - Bolt Movement



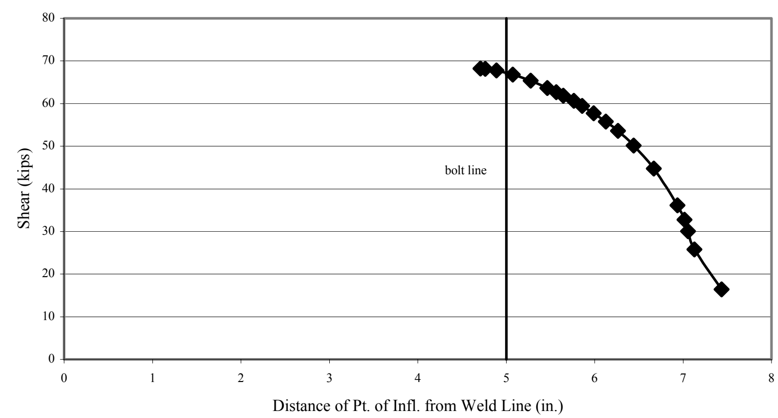
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C61 Results from Model 38

Table C36 Shear Stress in Bolts of Model 38

Increment		14	16	18	20	22	24	26	28	30	32	34	36	38	40
Load		31.37	32.73	34.4	36.12	39.97	44.74	47.43*	50.17	51.72	53.63	54.70	55.80	57.04	57.73
1st bolt	beam side	15.98	16.85	18.12	19.29	22.39	27.05	29.88	33.05	34.92	37.06	38.12	39.19	40.37	41.11
	plate side	14.49	15.32	16.50	17.75	20.84	25.62	28.55	31.78	33.82	36.19	37.59	38.85	40.19	41.10
2nd bolt	beam side	15.77	16.65	17.73	19.14	22.67	27.15	30.01	33.19	35.02	37.12	38.06	38.81	39.75	40.59
	plate side	14.07	14.98	16.06	17.42	21.00	25.52	28.28	31.33	33.05	35.36	36.65	37.58	38.90	39.99
1st bolt	beam side	19.93	20.49	21.18	21.78	22.71	23.78	24.13	24.29	24.18	23.95	23.58	23.07	22.38	21.80
	plate side	18.17	18.73	19.41	19.96	20.79	21.72	22.10	22.36	22.30	22.17	21.84	21.39	20.88	20.40
2nd bolt	beam side	19.82	20.40	20.99	21.46	22.31	23.08	23.16	22.88	22.65	22.43	22.12	21.75	21.59	21.31
	plate side	18.14	18.74	19.36	19.88	20.78	21.64	21.77	21.49	21.26	21.08	20.83	20.52	20.43	20.25

* Indicates that the plate fails

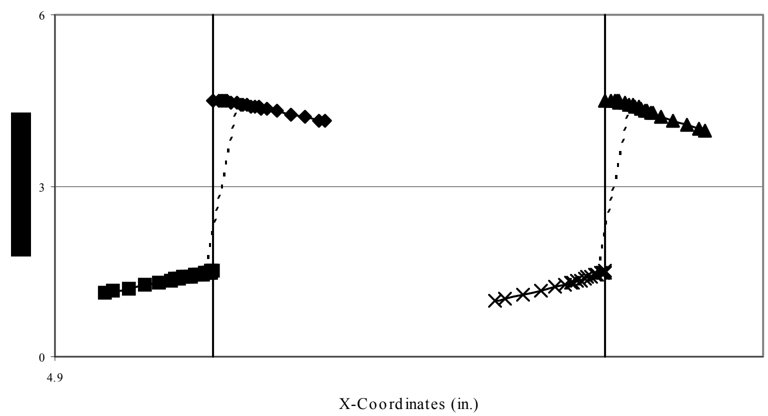
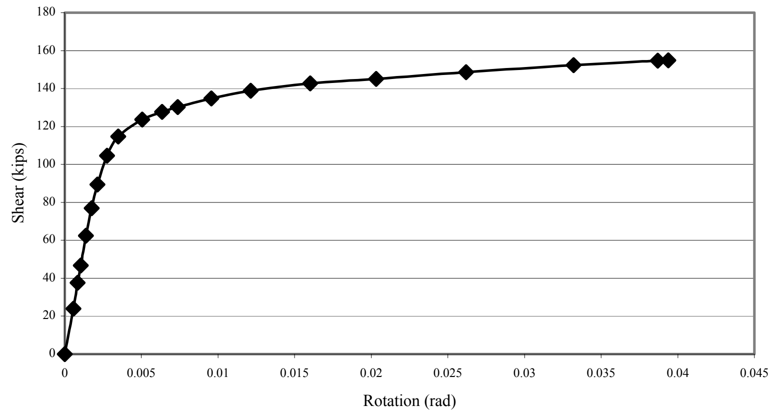
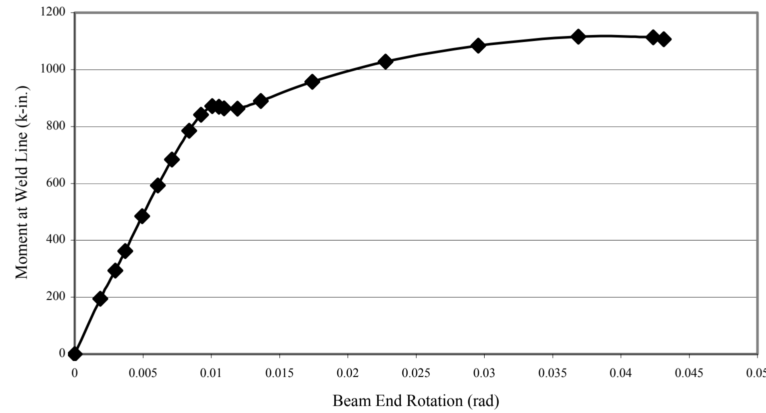


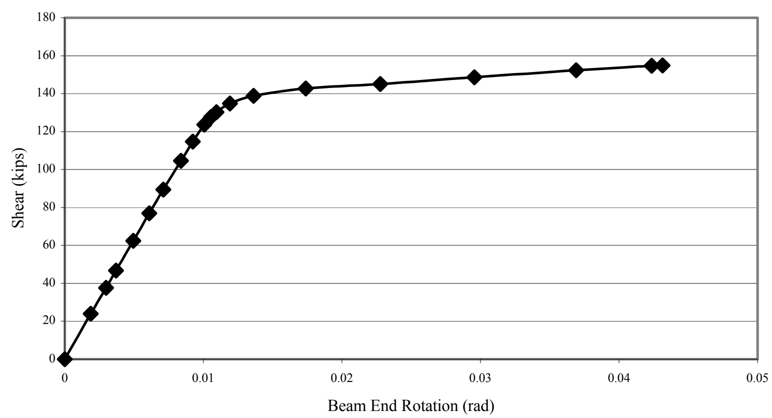
Figure C62 Model 38 - Bolt Movement



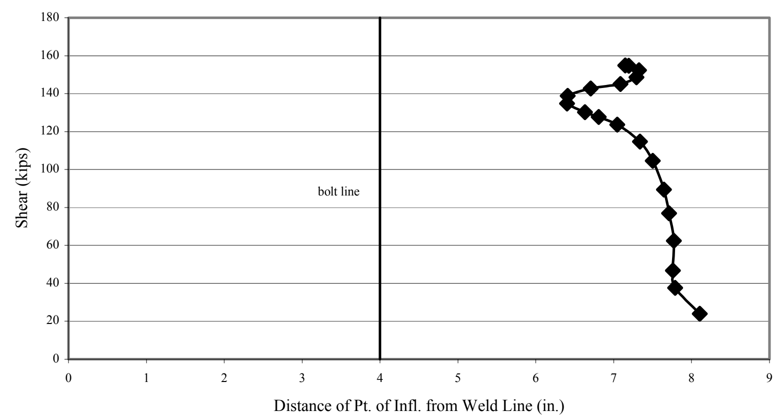
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C63 Results from Model 39

Table C37 Shear Stress in Bolts of Model 39

Increment		16	18	20	22	24	26	28	30	32	34	36	38	40	42
Load		62.39	66.84	76.86	82.5	89.47	96.60	104.6	109.1	114.7	120.4	123.6	125.4	127.7	128.9
1st bolt	beam side	10.43	12.18	16.05	18.41	21.51	25.33	30.03	32.46	34.94	37.22	38.40	38.92	39.53	39.89
	plate side	9.626	11.25	15.22	17.54	20.68	24.24	28.67	30.92	33.82	36.55	37.80	38.35	39.14	39.60
2nd bolt	beam side	9.996	11.22	15.38	18.16	22.01	25.64	29.18	31.34	34.19	37.40	39.01	39.96	41.11	41.63
	plate side	9.609	10.93	14.64	17.10	20.26	23.34	26.74	28.90	31.75	35.14	37.07	38.43	40.07	40.81
3rd bolt	beam side	13.03	14.95	18.85	20.33	22.26	24.91	29.15	31.76	35.54	38.76	40.51	41.22	42.01	42.41
	plate side	12.42	13.85	16.78	18.08	20.12	23.14	27.67	30.50	34.43	37.86	39.57	40.59	41.74	41.19
1st bolt	beam side	35.17	36.21	37.18	37.38	37.43	37.45	37.56	37.81	38.26	39.26	40.28	40.76	41.23	41.43
	plate side	33.32	34.73	36.89	37.89	38.62	39.12	39.53	39.63	39.91	40.69	41.51	42.07	42.88	43.34
2nd bolt	beam side	33.57	35.32	38.81	39.91	41.04	41.70	42.53	42.91	43.09	43.05	42.96	42.90	42.85	42.83
	plate side	31.36	33.09	36.67	38.50	40.47	41.69	42.63	43.13	43.48	43.74	43.79	43.78	43.82	43.86
3rd bolt	beam side	34.15	36.00	39.15	40.59	41.57	42.45	43.17	43.62	44.16	44.23	44.01	43.70	43.21	42.92
	plate side	33.08	35.08	38.62	40.23	41.70	42.74	43.60	44.00	44.55	44.83	44.71	44.54	44.26	44.09

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

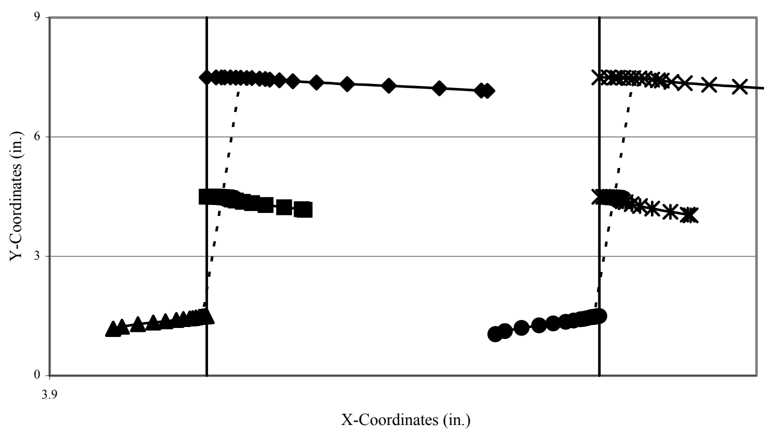
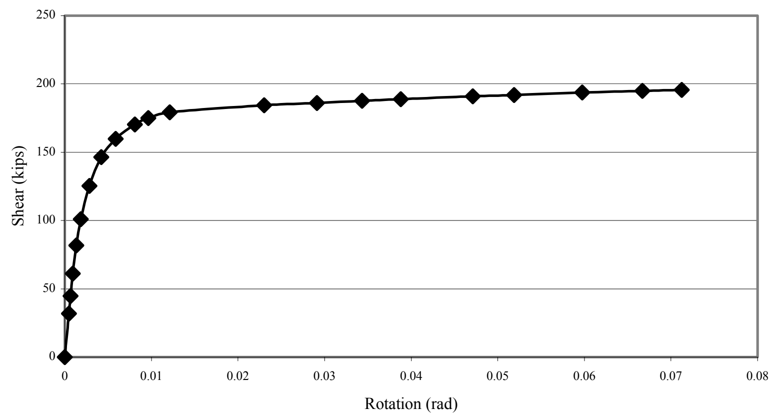
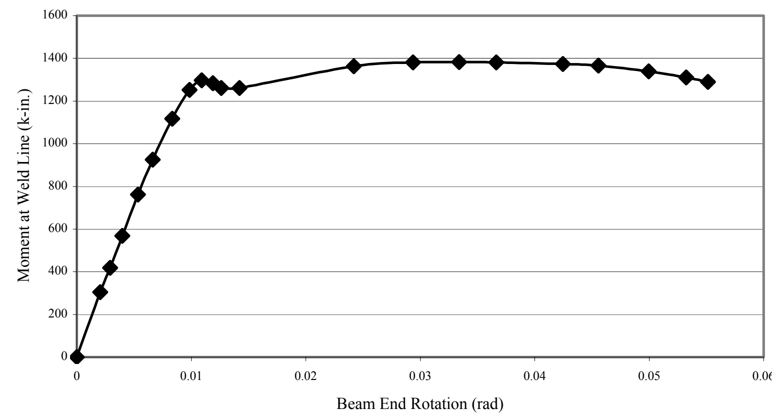


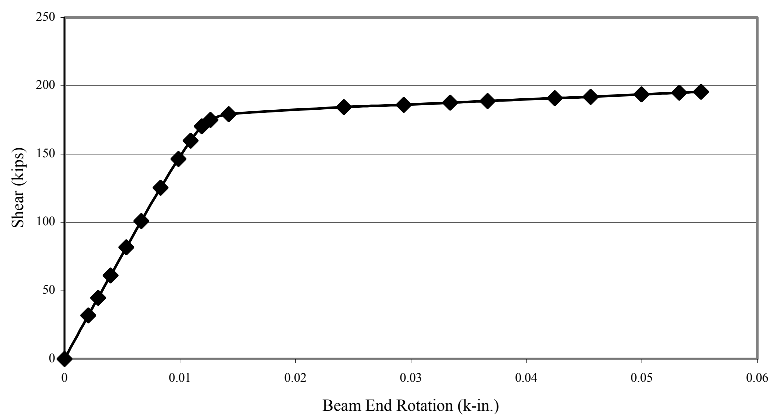
Figure C64 Model 39 - Bolt Movement



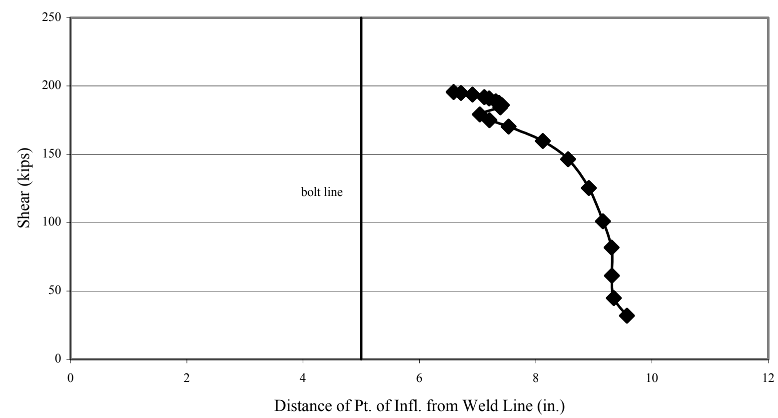
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C65 Results from Model 40

Table C38 Shear Stress in Bolts of Model 40

Increment		16	18	20	22	24	26	28	30	32	34	36	38	40	42
Load		81.84	87.74	101.0	108.5	125.3	134.7	146.4	153.0	159.7	163.5	170.3	172.4	175.0	177.7
1st bolt	beam side	8.792	10.49	14.25	16.41	22.18	26.51	32.53	35.54	38.19	39.31	40.29	40.63	41.20	41.84
	plate side	7.988	9.720	13.35	15.57	21.24	25.35	31.10	34.33	37.29	38.58	40.66	41.30	42.34	43.49
2nd bolt	beam side	6.133	7.248	10.75	13.09	20.19	24.84	30.30	32.99	36.15	37.70	40.23	41.15	42.07	43.14
	plate side	6.020	7.133	10.39	12.66	18.99	23.15	28.18	30.87	33.90	35.51	39.29	40.61	41.55	42.54
3rd bolt	beam side	8.763	10.33	14.52	17.12	21.97	24.30	28.46	31.79	35.96	38.20	41.24	41.96	42.70	43.74
	plate side	8.679	10.15	13.75	15.78	19.86	22.44	27.29	30.93	35.10	37.21	40.67	41.62	42.43	43.22
4th bolt	beam side	7.453	8.821	12.63	15.23	22.22	26.89	32.06	35.01	38.23	40.11	42.04	42.47	43.22	43.81
	plate side	7.543	8.911	12.65	15.08	21.33	25.35	29.98	32.85	36.67	39.17	42.67	43.44	43.87	44.35
1st bolt	beam side	36.58	37.01	37.23	37.10	36.89	37.10	37.79	38.75	39.91	40.50	41.05	40.88	40.57	40.06
	plate side	35.69	36.73	38.10	38.37	38.96	39.25	40.07	40.76	41.85	42.65	43.98	44.02	43.77	43.51
2nd bolt	beam side	36.08	37.47	39.72	40.49	41.63	42.17	42.41	42.57	42.86	43.04	43.55	43.65	43.76	43.81
	plate side	34.01	35.58	38.74	40.12	42.10	42.78	43.32	43.60	43.97	44.20	44.67	44.89	45.13	45.30
3rd bolt	beam side	36.03	37.61	40.18	40.98	42.18	42.75	43.56	43.81	43.94	43.99	43.98	43.97	43.93	43.79
	plate side	35.21	36.97	39.92	40.95	42.45	43.06	43.61	43.90	44.13	44.27	44.48	44.54	44.60	44.62
4th bolt	beam side	36.08	37.50	39.64	40.51	41.98	42.91	43.76	44.09	43.80	43.25	41.80	41.55	41.17	40.48
	plate side	33.98	35.54	38.35	39.89	42.12	43.06	44.05	44.49	44.45	43.85	42.31	41.99	41.87	41.64

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

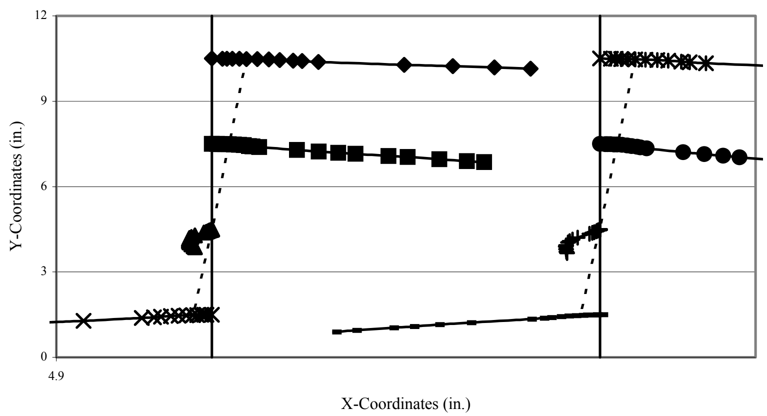
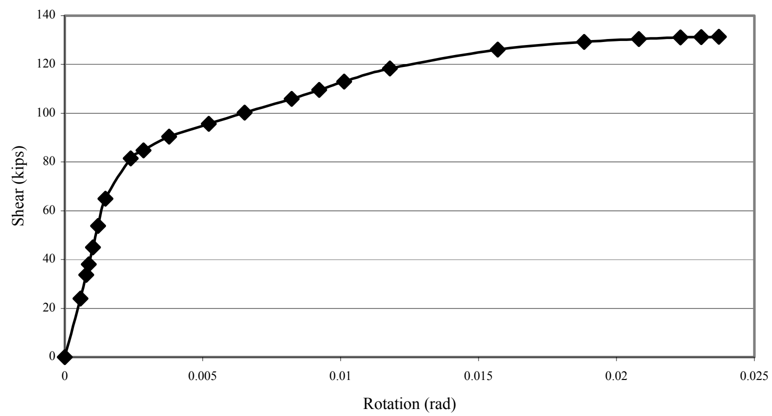
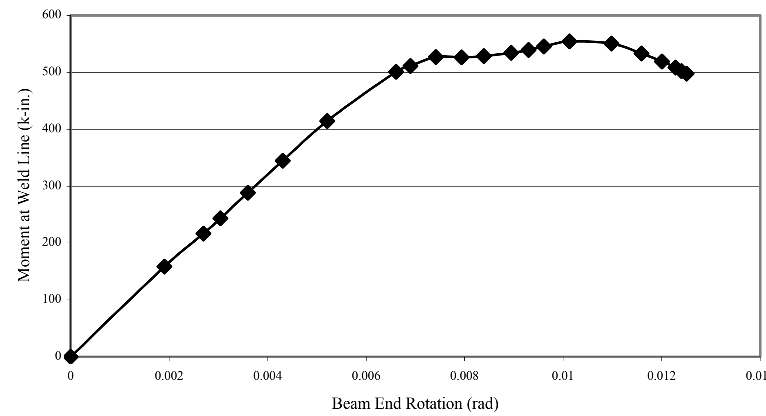


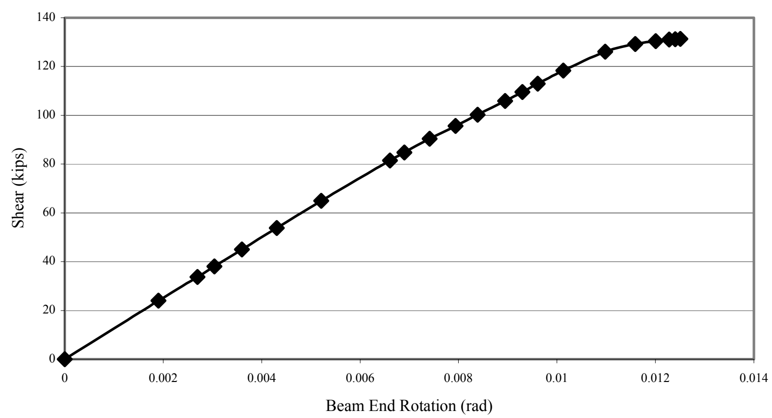
Figure C66 Model 40 - Bolt Movement



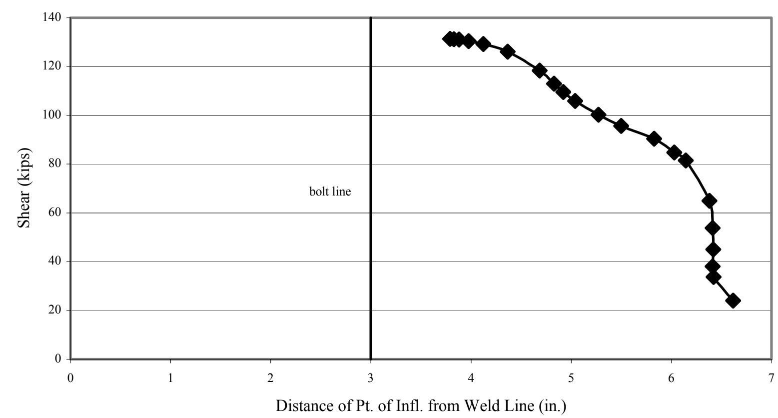
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C67 Results from Model 41

Table C39 Shear Stress in Bolts of Model 41

Increment		34	36	38	40	42	44	46	48	50	52	54	56	58	60
Load		87.57	90.43	92.03	95.64	97.68	100.2	102.8	105.9	107.7	109.6	110.6	112.9	117.1	118.4
1st bolt	beam side	26.83	29.23	30.95	34.03	35.17	36.45	37.53	38.76	39.32	39.81	40.14	40.88	42.15	42.47
	plate side	27.61	29.88	31.47	34.46	35.50	36.98	38.10	39.61	40.42	41.07	41.63	42.87	44.49	45.15
2nd bolt	beam side	25.90	27.77	29.16	32.46	34.49	36.95	39.08	40.47	40.99	41.61	41.94	42.65	43.44	43.56
	plate side	25.72	27.60	28.95	32.36	34.47	36.94	39.31	41.42	42.32	43.17	43.56	44.43	46.40	46.89
3rd bolt	beam side	25.72	27.79	29.74	33.91	36.14	38.31	39.88	40.85	41.33	41.88	42.14	42.94	43.80	43.97
	plate side	26.11	28.30	30.15	34.21	36.33	38.81	40.80	42.43	42.94	43.52	43.83	44.65	46.19	46.67
1st bolt	beam side	37.20	37.30	36.86	36.06	35.61	35.06	34.62	33.94	33.63	33.40	33.35	33.27	33.47	33.55
	plate side	38.82	38.92	38.44	37.59	37.11	36.58	36.11	35.48	35.19	34.99	34.94	34.87	35.01	35.07
2nd bolt	beam side	37.30	37.34	37.21	37.21	37.21	37.36	37.52	37.74	37.84	37.94	37.99	38.13	38.53	38.66
	plate side	38.19	38.24	38.08	38.03	38.01	38.14	38.31	38.63	38.77	38.95	39.05	39.31	39.95	40.11
3rd bolt	beam side	36.26	36.17	35.66	34.65	34.38	34.15	33.85	33.94	34.01	34.05	34.08	34.21	34.87	34.98
	plate side	36.79	36.67	36.11	34.99	34.71	34.53	34.26	34.30	34.46	34.61	34.70	34.91	35.76	36.06

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

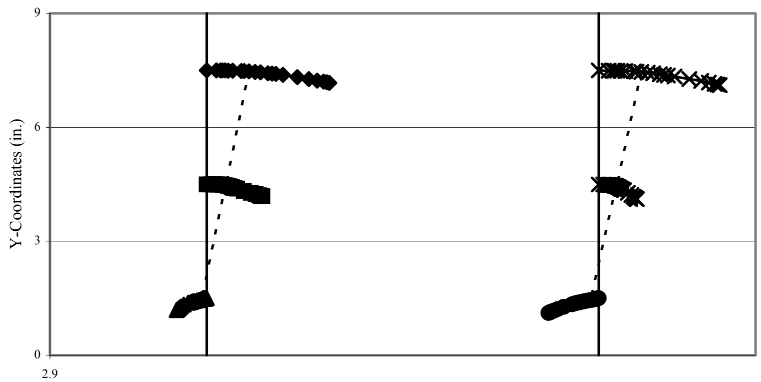
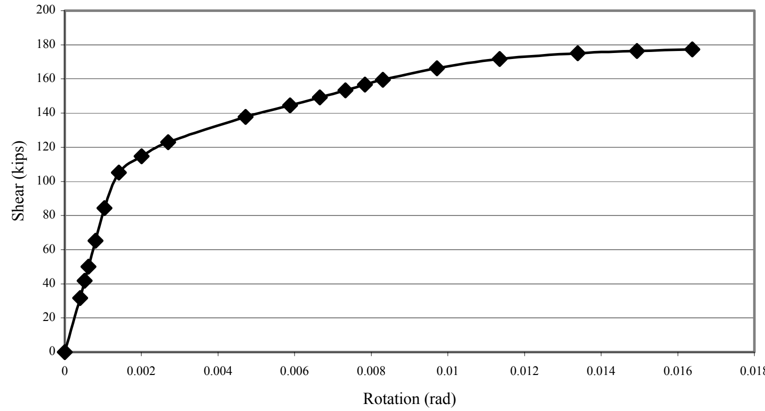
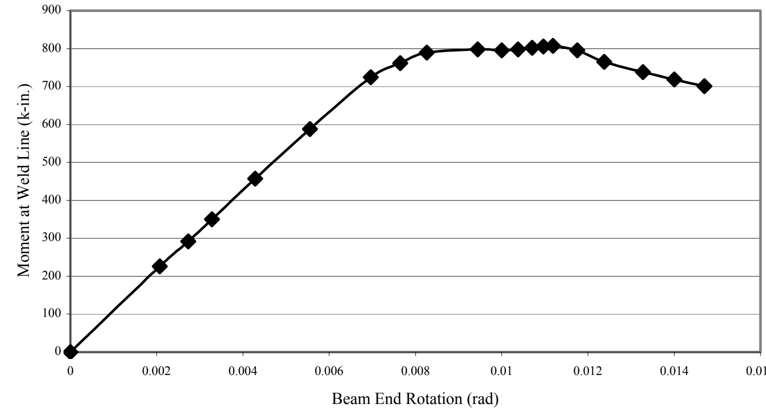


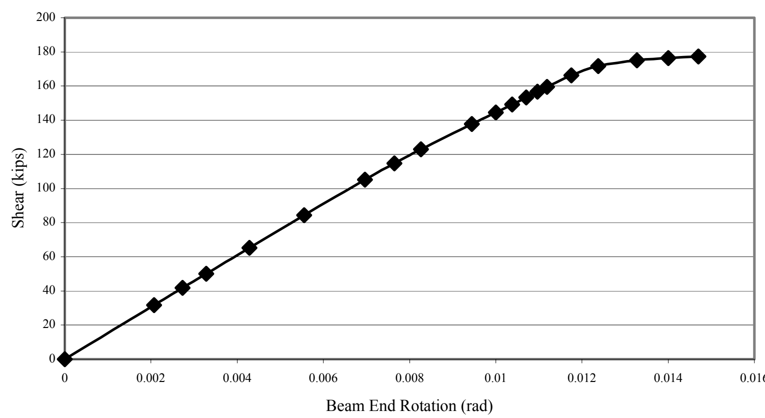
Figure C68 Model 41 - Bolt Movement



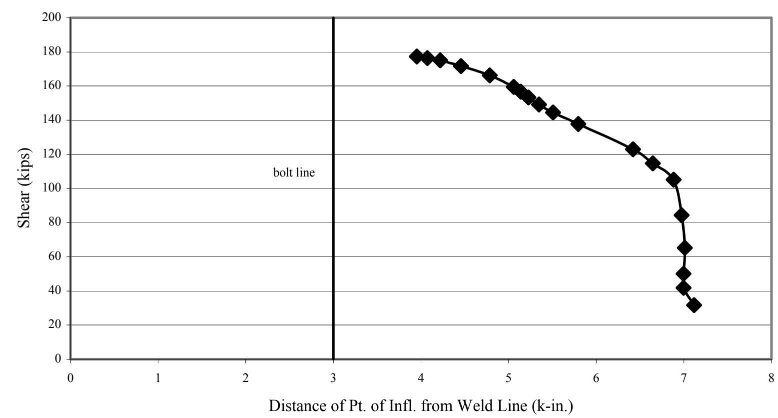
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C69 Results from Model 42

Table C40 Shear Stress in Bolts of Model 42

Increment		28	30	32	34	36	38	40	42	44	46	48	50	52	54
Load		114.6	119.9	122.9*	129.7	137.7	140.7	144.5	146.7	149.3	150.7	153.4	154.9	156.7	157.6
1st bolt	beam side	22.51	25.08	26.63	30.66	35.93	37.36	38.54	38.86	39.22	39.37	39.76	40.05	40.39	40.55
	plate side	23.09	25.90	27.54	31.51	36.59	37.86	39.17	39.78	40.42	40.70	41.12	41.59	42.02	42.21
2nd bolt	beam side	17.35	20.11	21.74	26.45	31.57	33.24	35.37	36.78	38.27	39.01	39.92	40.22	40.59	40.77
	plate side	17.97	20.74	22.21	26.52	31.44	33.16	35.16	36.65	38.46	39.20	40.47	41.12	42.02	42.40
3rd bolt	beam side	19.90	21.83	22.96	25.61	31.07	33.56	36.64	37.79	39.03	39.40	39.96	40.29	40.69	40.86
	plate side	19.81	21.60	22.78	25.55	31.40	33.99	37.08	38.38	39.86	40.60	41.93	42.56	43.05	43.27
4th bolt	beam side	21.28	24.28	25.92	29.48	34.19	35.86	37.34	37.95	38.72	39.11	39.73	40.09	40.46	40.60
	plate side	22.07	24.90	26.40	29.68	34.23	35.99	37.79	38.75	40.07	40.78	41.84	42.24	42.69	42.94
1st bolt	beam side	37.33	37.38	37.45	37.82	37.42	37.23	36.96	36.93	36.88	36.89	36.95	37.06	37.22	37.30
	plate side	40.02	40.26	40.39	40.82	40.38	40.14	40.03	40.07	40.08	40.09	40.16	40.27	40.39	40.46
2nd bolt	beam side	38.82	38.97	39.06	39.27	39.60	39.74	39.92	40.04	40.20	40.29	40.45	40.55	40.67	40.73
	plate side	40.97	41.14	41.21	41.39	41.64	41.77	41.93	42.03	42.18	42.27	42.42	42.50	42.62	42.68
3rd bolt	beam side	39.45	39.67	39.79	39.91	39.97	39.98	39.98	39.98	40.01	40.04	40.10	40.16	40.24	40.28
	plate side	40.57	40.96	41.14	41.34	41.53	41.59	41.61	41.64	41.71	41.78	41.90	41.99	42.09	42.13
4th bolt	beam side	38.91	39.09	39.15	38.78	37.22	36.71	36.36	36.31	36.34	36.39	36.48	36.61	36.76	36.84
	plate side	40.57	40.80	40.88	40.56	38.80	38.18	37.83	37.72	37.72	37.77	37.87	38.03	38.23	38.33

* Indicates that the force redistribution occurs

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

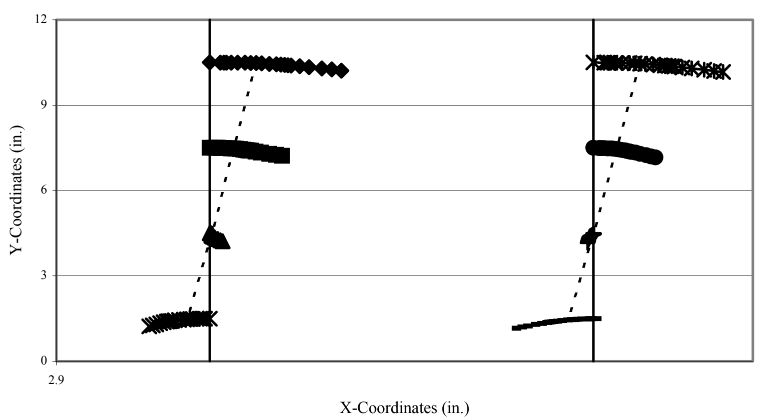
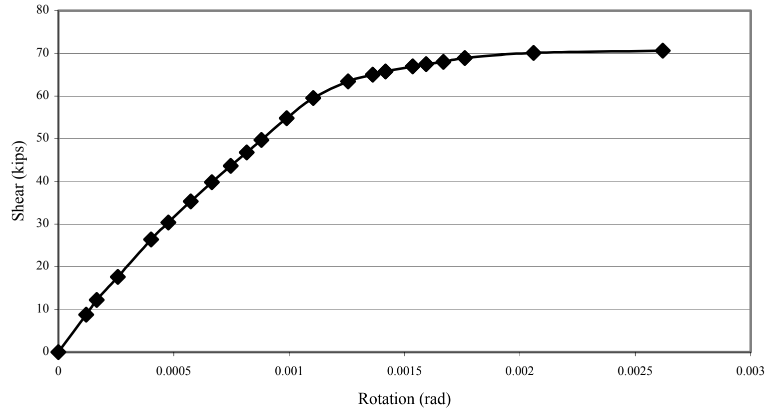
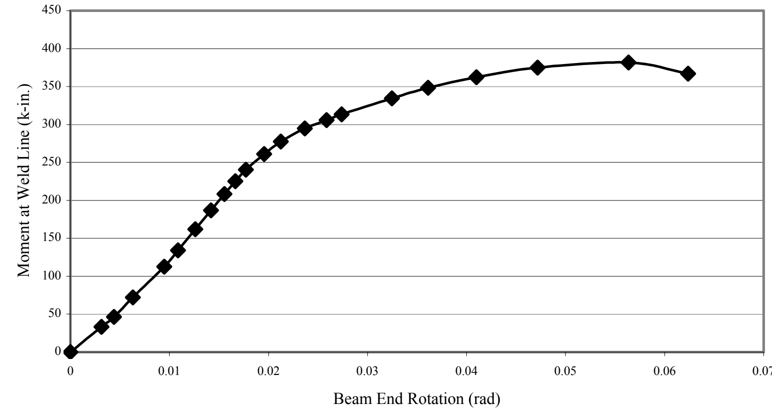


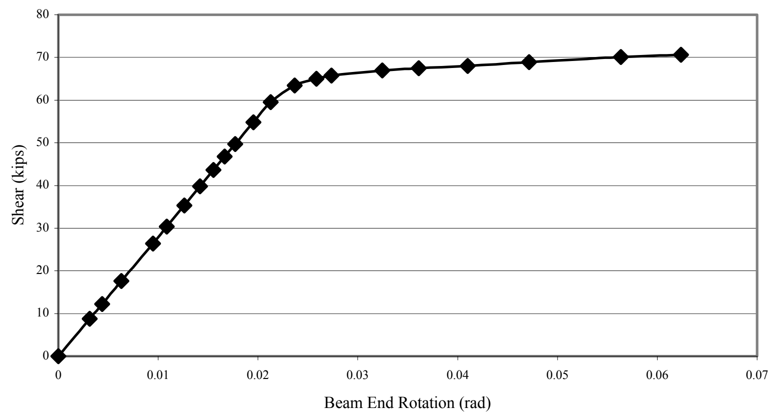
Figure C70 Model 42 - Bolt Movement



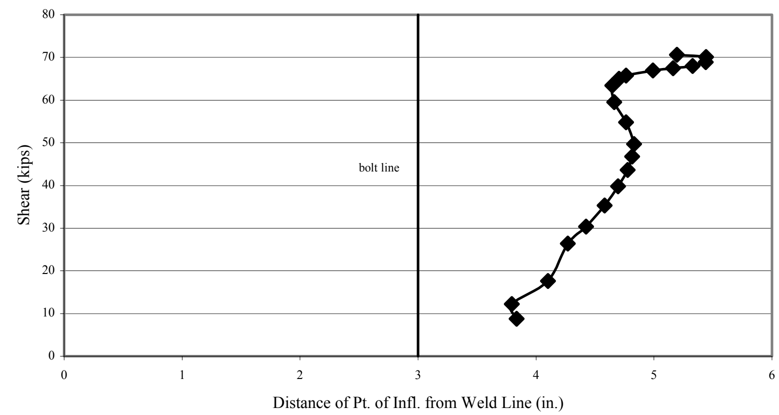
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C71 Results from Model 43

Table C41 Shear Stress in Bolts of Model 43

Increment		38	40	42	44	46	48	50	52	54	56	58	60	62	64
Load		47.69	49.73	52.25	54.83	56.28	59.54	61.37	63.43	64.46	65.04	65.37	65.77	66.18	66.41
1st bolt	beam side	31.73	32.38	33.38	34.85	35.63	36.44	36.32	38.76	35.12	34.68	34.34	33.91	33.21	32.80
	plate side	32.57	33.41	34.36	35.58	36.26	37.46	37.54	37.11	36.44	35.91	35.49	34.94	34.13	33.70
2nd bolt	beam side	32.19	33.94	36.10	37.93	38.54	39.84	40.61	41.40	41.50	41.45	41.23	40.89	40.18	39.64
	plate side	32.45	34.13	36.37	38.59	39.74	41.43	42.50	44.13	44.92	45.30	45.46	45.56	45.31	44.98
3rd bolt	beam side	34.38	35.30	36.08	36.87	37.33	38.36	39.39	40.17	40.39	40.49	40.48	40.40	40.26	40.23
	plate side	34.70	35.64	36.60	37.35	37.82	38.93	40.04	41.40	42.17	42.76	43.09	43.40	43.78	43.93

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

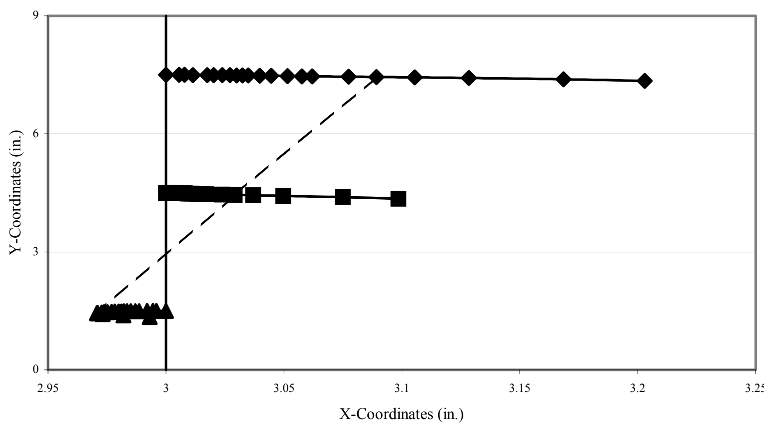
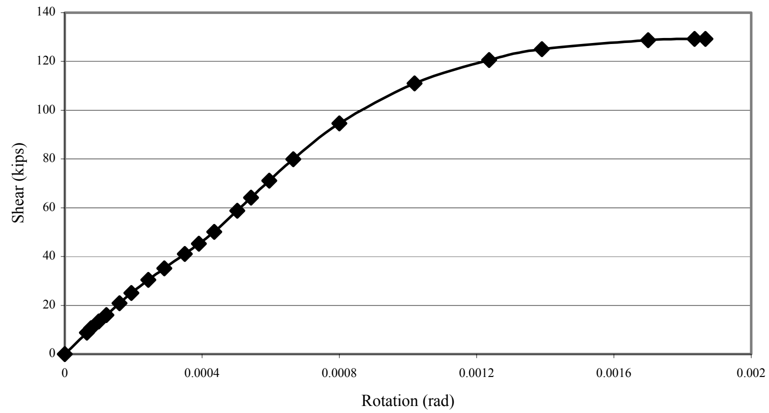
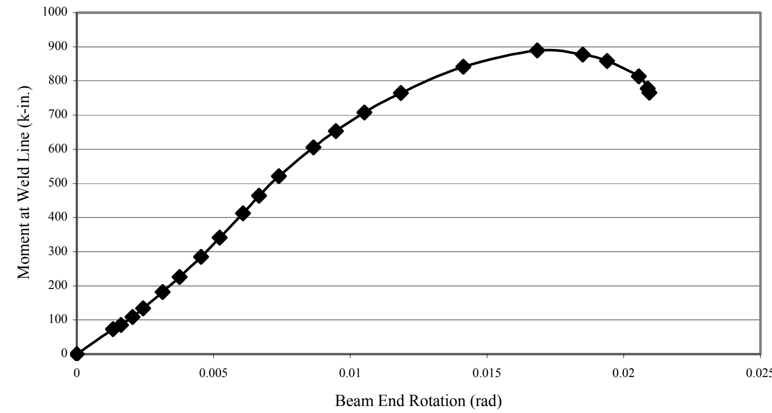


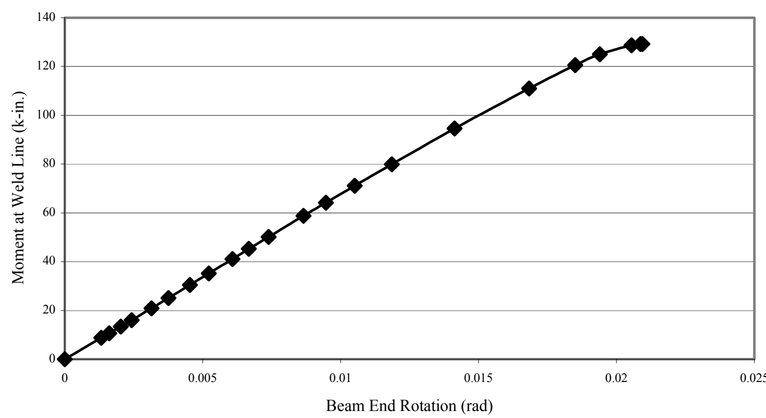
Figure C72 Model 43 - Bolt Movement



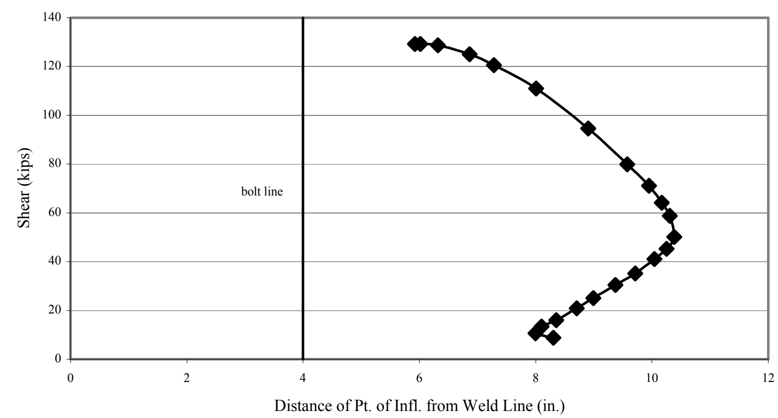
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C73 Results from Model 44

Table C42 Shear Stress in Bolts of Model 44

Increment		52	54	56	58	60	62	64	66	68	70	72	74	76	78
Load		64.22	67.31	71.93	75.04	79.88	84.83	87.62	91.06	94.58	96.57	101.0	109.0	111.1	113.1
1st bolt	beam side	21.37	22.08	22.93	23.94	25.34	26.73	27.53	28.35	29.14	29.55	30.46	32.62	33.19	33.85
	plate side	21.13	21.68	22.44	23.26	24.50	25.94	26.83	27.84	28.87	29.45	30.72	33.44	34.18	35.11
2nd bolt	beam side	26.30	26.43	27.03	28.35	30.42	32.36	33.25	33.97	34.49	34.73	35.12	36.24	36.64	37.13
	plate side	26.63	26.67	27.00	27.86	29.34	30.99	31.97	32.84	33.51	33.85	34.52	36.03	36.47	37.02
3rd bolt	beam side	28.95	30.62	32.84	34.31	35.50	36.18	36.41	36.65	36.80	36.97	37.51	37.04	37.15	37.31
	plate side	29.38	31.05	33.17	34.91	36.48	37.12	37.47	38.05	38.72	39.13	40.25	40.85	41.14	41.44
4th bolt	beam side	25.54	26.84	28.40	30.41	33.22	36.25	37.56	38.61	39.28	39.68	40.59	40.62	40.21	39.57
	plate side	25.42	26.73	28.44	30.69	33.61	36.88	38.31	39.85	40.89	41.37	42.87	46.96	47.40	47.50
5th bolt	beam side	27.59	29.22	31.02	33.03	34.83	36.21	36.67	37.59	38.59	39.11	40.10	39,79	39.56	39.26
	plate side	28.07	29.72	31.57	33.53	35.63	37.27	37.90	38.69	39.73	40.39	42.16	44.90	45.61	46.00

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

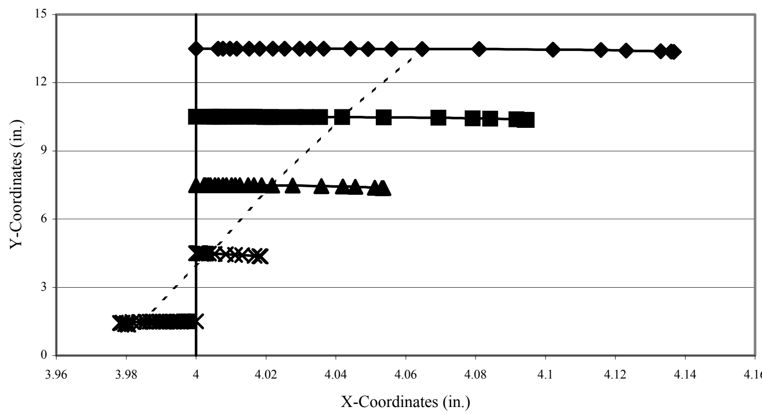
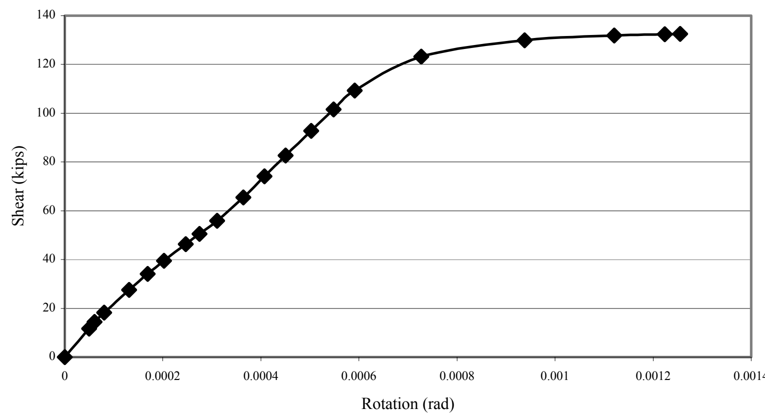
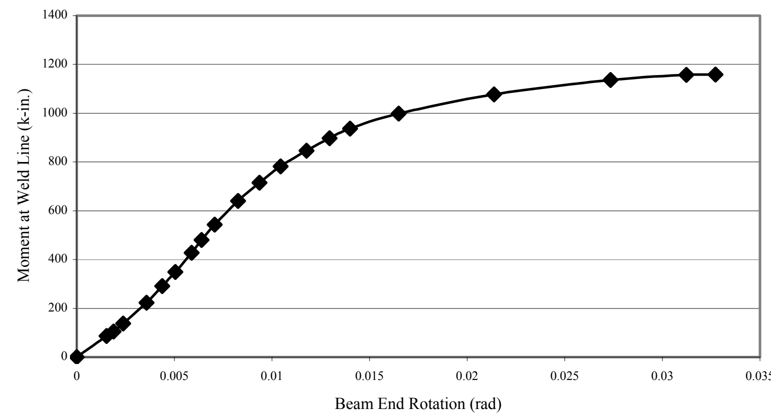


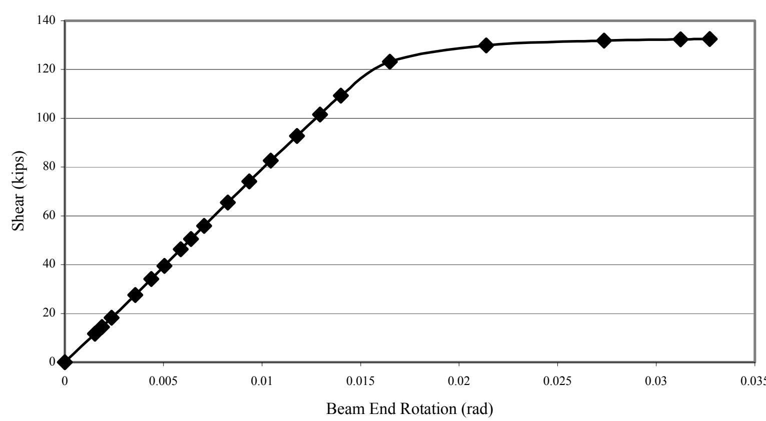
Figure C74 Model 44 - Bolt Movement



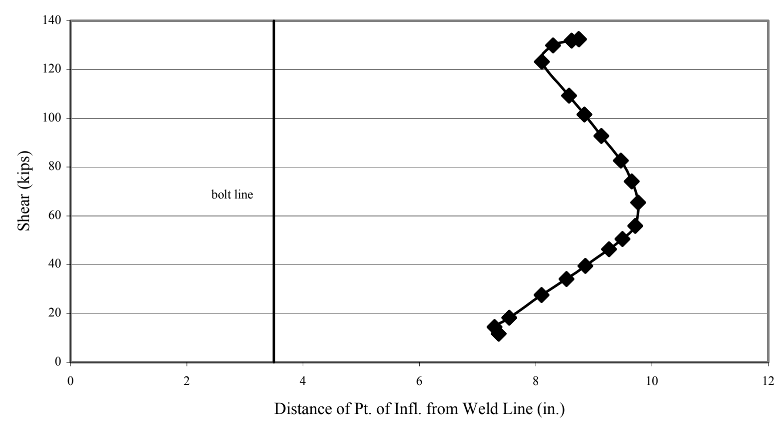
(a) Shear vs. Rotation at Bolt Line



(b) Moment at Weld Line vs. Beam End Rotation



(c) Shear vs. Beam End Rotation



(d) Shear vs. Distance to Point of Inflection from Weld Line

Figure C75 Results from Model 45

Table C43 Shear Stress in Bolts of Model 45

Increment		48	50	52	54	56	58	60	62	64	66	68	70	72	74
Load		82.61	85.72	92.72	96.66	101.5	106.5	109.3	112.8	116.3	120.7	123.2	125.7	127.1	128.9
1st bolt	beam side	17.62	17.89	18.26	18.38	19.13	20.53	21.29	22.37	23.51	24.48	24.42	23.58	22.71	21.12
	plate side	23.57	24.14	25.41	26.23	27.53	29.37	30.34	31.63	32.93	33.98	33.87	33.03	32.19	30.96
2nd bolt	beam side	23.94	24.26	24.82	25.28	26.05	26.87	27.39	28.11	28.89	29.85	30.07	29.36	28.71	27.51
	plate side	29.48	29.71	30.51	31.14	32.19	33.76	34.49	35.38	36.29	37.38	37.88	38.00	37.80	36.93
3rd bolt	beam side	26.74	27.83	30.11	31.23	32.21	33.03	33.47	34.05	34.15	34.59	34.56	34.47	34.50	33.97
	plate side	31.71	33.19	36.34	37.83	39.17	40.04	40.26	40.61	41.02	42.04	42.57	42.96	43.03	42.97
4th bolt	beam side	24.19	25.02	27.34	28.92	31.01	33.06	34.48	35.70	36.67	37.95	39.05	30.13	40.26	40.18
	plate side	26.97	28.14	31.44	33.36	36.09	38.67	40.29	41.84	43.11	44.81	45.69	47.14	48.19	49.92
5th bolt	beam side	26.05	27.10	29.67	31.18	32.45	33.55	34.12	35.13	36.10	36.43	36.64	36.95	37.16	37.54
	plate side	30.78	32.06	35.11	36.83	38.37	39.79	40.36	41.02	41.72	42.26	42.97	43.97	44.77	45.54
6th bolt	beam side	24.09	24.54	25.70	26.59	27.84	29.22	29.92	30.71	31.42	32.12	32.55	32.51	32.39	32.07
	plate side	29.03	29.24	30.32	31.28	32.69	34.25	35.05	35.95	36.75	37.45	37.95	38.39	38.62	38.84

Bolt shear rupture strength as determined from FEM

Indicates that the stress decreases in the next increment

Indicates that the stress in outer element exceeds the stress in the innermost element

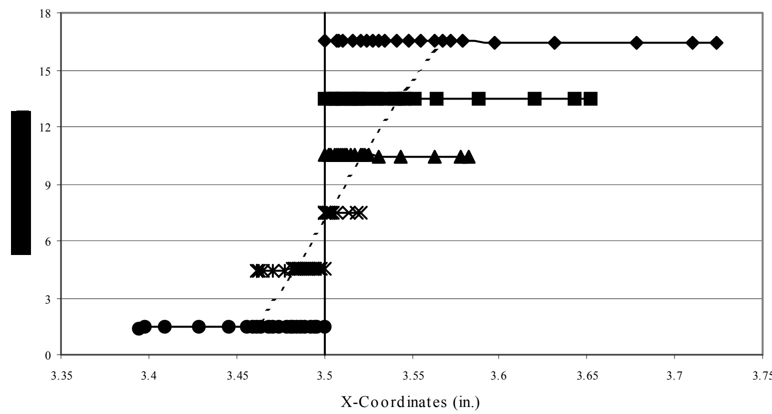


Figure C76 Model 45 - Bolt Movement

Vita

Aphinat Ashakul was born on August 1, 1972, in Bangkok, Thailand to Mr. Argon and Mrs. Nittaya Ashakul. He is the oldest child of the family with two younger sisters. In 1990, he received a high school diploma and succeeded in the entrance examination to enter Chulalongkorn University, one of the most renowned universities in Thailand. Having participated in numerous extracurricular activities, he won a scholarship in 1992. In 1993, he became the vice president of the 10th Engineering Academic Exhibition, the largest student-run event, held every three years. He was also a member of the Voluntary Engineering Student Camp in His Majesty's Patronage. In 1994, he graduated from the university with a bachelor's degree in civil engineering. In 1995, he started working at the Consultant One Hundred and Ten Co., Ltd. as a design engineer before switching to EEC-IE Co., Ltd. in 1996. In 1997, he began the pursue of his academic goal by entering Illinois Institute of Technology and achieved the master of structural engineering degree in 1999. In 2000, he continued his study for the Ph.D. in civil engineering at Virginia Polytechnic Institute and State University.