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The Effects of Bolt Pretension, In-Plane Eccentricity, and Friction on the Ductility of Block Shear Connections

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The Effects of Bolt Pretension, In-Plane Eccentricity, and Friction on the Ductility of Block Shear Connections

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1.1 Introduction

Block shear is a phenomenon found in short bolted connections of tension members or coped beams. It involves tension rupture on a plane between bolt holes normal to the applied load and shear failure on a plane between bolt holes parallel with the load. Figure 1.1 shows typical block shear failure paths for a coped beam connection (a), an angle tension member (b), and a channel tension member (c). In this figure P indicates direction of loading, and T and S are the tension and shear planes, respectively.

Block shear failures were first observed in coped beams (Birkemoe and Gilmor, 1978), but in the 1978 collapse of the roof of the Hartford Civic Center, an angle was found in the wreckage that failed in block shear (Smith and Epstein, 1980). Block shear failure of tension connections was not a consideration at that time. Figure 1.1b illustrates similarities between the block shear failure paths of an angle in tension to that of a coped beam and channel shape. This observed failure prompted block shear studies of various tension connections.

Unlike beam connections where forces are coplanar, angles connected by a single leg are not only subjected to tension, but also to a moment due to the inherent eccentricity of the shape. Studies have shown that the eccentricity built into these types of
connections plays a role in determining the failure mode and load (Thacker, 1987; Chamarajanagar, 1990; Adidam, 1990; Gross, 1994). Forces are always concentrated in the connection, but when tension members are not attached through all of their elements, for instance the outstanding leg of an angle, forces are contained within the connected regions. The term 'shear lag' has been used to describe the process of transferring concentrated forces through the connection and into the unconnected elements. The forces carried by the connected elements are transferred through shear to the unconnected elements. Shear lag reduces the capacity of these tension members and must be considered in the design of such connection. It has been suggested that the reduction factor for shear lag, \( U = 1 - \frac{x}{l} \), where \( x \) is the member eccentricity and \( l \) is the connection length be used as a reduction factor in block shear as well (Epstein and Adidam, 1991).

Initially, block shear studies of tension connections only considered angles, until a 1996 study that was conducted on a limited number of structural tee connections (Twilley, 1996). The highlight of this study was the failure of one particular specimen which occurred in a previously undocumented manner. In a follow up paper, this failure was termed as 'alternate path' block shear (Epstein, 1996). This path involved tension on the transverse sections of the flange of the tee, and shear on a longitudinal section in the web of the tee. Figure 1.2 shows (a) the expected and (b) observed block shear failure paths. Figure 1.3 is a photo of a tee section that failed along the alternate shear path (Epstein, 1996).

In several studies of angles in the 1980's, the finite element (FE) method was shown to be a valuable tool in the investigation of this failure phenomenon (Thacker, 1987; Chamarajanagar, 1990; Gulia, 1988; Ricles and Yura, 1983). Early models created
were idealized as either elastic, or elastic-perfectly plastic, but still provided good representations of approximate failure patterns and relative failure loads. Thus, they were used to predict trends and provide a means of comparison of relative magnitudes between sections.

Connection eccentricity causes the development of internal moments, quantified simply by the load, $P$, multiplied by the eccentricity, $e$. However, a 1998 study using structural tees found there were additional factors which influence the internal moment (McGinnis 1998). The length of the connection and the rotational stiffness of the connection were also shown to influence the internal moment in these connections. Subsequent studies have been conducted to attempt to quantify the effects of connection length on the block shear capacity and apply appropriate factors (Cunningham et al. 1995, Driver et al. 2006).

Block shear is a complicated phenomenon with a variety of variables effecting both the capacity and ductility of the connection. The level of pretensioning in the bolts of the connection could play a role in the performance of the connection. Clamping forces induced by the bolts creates compressive forces normal to the surface of the plate. This leads to a complex triaxial stress field. This is compounded by the stress localization which is caused by the geometric imperfection of the bolt hole. Through the combination of the triaxial stresses and geometric stress localization, the capacity of the connection could be affected by the level of tensioning found in the bolts.

In-plane eccentricity causes a change in the stress/strain distribution due to the addition of a moment. This would alter the levels of tensile stress/strain on one end of the tension plane compared to the other. It also results in a larger shear stress/strain on
the shear leg closer to the line of the applied load. These effects are illustrated in Fig. 1.4. The current design code (AISC, 2011) does not consider non-uniform stresses to develop in any tensile or shear plane with the exception of coped beams with two or more rows of bolts as outlined by Ricles and Yura (1983). In spite of this, in-plane eccentricity and stress concentration could have an effect on either capacity or ductility.

Friction between the connected plates also acts as a means of load transfer between the connected parts. Slip-critical connections rely on this load transfer mechanism and require surface preparation to achieve the level of friction required. If the surface was not properly prepared, the capacity of the load transfer through friction may not be met and may lower the capacity of the connection.

The focus of this study will be to investigate the effects of in-plane eccentricity and bolt tension on the capacity and ductility of the block shear failure. Finite element modeling and destructive testing will be performed on connections designed to fail in block shear with varied levels of eccentricity. Bolts will be tensioned to two levels defined in the AISC steel manual, snug-tight and fully pretensioned. Failure loads and deformation will be compared between connections with different eccentricity and level of pretension to quantify their respective effects on the behavior of the connection. Additionally, one test including Teflon tape will be included to quantify the isolated effect of friction on these connections.
1.2 Literature Review

In 1978, Birkemoe and Gilmor conducted tests on bolted, coped beam web connections. They noted that a failure, termed block shear, could govern (Birkemoe and Gilmor, 1978). A design equation to treat this failure mode was proposed. It involves the ultimate shear strength applied to the net shear area, and the ultimate tension strength applied to the net tension area, as shown in Fig. 1.1a. They proposed the connection’s block shear load capacity as

\[ P_{BS} = (0.3A_{nv} + 0.5A_{nt})F_u \]  

(1.1)

where:

- \( F_u \) = tensile strength of the material,
- \( A_{nv} \) = net shear area, and
- \( A_{nt} \) = net tensile area.

This equation was included in the ninth edition of the Allowable Stress Design (ASD) specification (AISC, 1989), which is still used as the referenced standard in some state building codes.

Yura et al. (1982) reported on additional tests conducted on coped and uncoped beam web connections with single and double rows of bolts. They noted that lower than expected capacities were observed when the double row bolts were utilized, compared to 1978 AISC predicted capacity. Ricles and Yura (1983) continued testing of beam web connections in 1983 and generated a linear finite element model that corroborated their test results that two rows of bolts caused a reduction of capacity.
Based on the results of tests performed on tension gusset plates, Hardash and Bjorhovde (1985) suggested a set of design equations using the yield strength of the gross section on one plane and the ultimate strength of the net section on the other. This approach, with minor changes, is currently used by both Load and Resistance Factor Design (LRFD) and ASD to calculate the nominal block shear capacity, $R_n$. The fourteenth edition of American Institute of Steel Construction’s (AISC) combined ASD/LRFD (AISC, 2011) has governing block shear equations as:

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt} \quad (1.2)$$

where:

$\varphi = 0.75$ (LRFD)

$\Omega = 2.00$ (ASD)

$A_{gv} = $ gross area subject to shear,

$A_{nv} = $ net area subject to shear,

$A_{nt} = $ net area subject to tension,

$F_u = $ specified minimum tensile strength,

$U_{bs} = $ adjustment factor for non-uniform tension stress

$F_y = $ specified minimum yield strength.

$U_{bs}$ in Eq. 1.2 is set as 0.5 when tension stress is non-uniform, and 1.0 in all other cases. The commentary section of the fourteenth edition defines non-uniform tension stresses to occur only in coped beam connections with multiple rows of bolts, which explains the lower than predicted values observed by Yura et al. (1982).

Thacker (1987), and Epstein and Thacker (1991) used the dimensions of an angle that had failed in block shear in the Hartford Civic Center collapse, to develop a non-
linear finite element model which demonstrated block shear as the failure mode. By varying the bolt hole stagger patterns, they also suggested that the direction of a staggered path affects the failure load when shear lag is present in tension members.

Gulia (1988), and Epstein and Gulia (1993) used a non-linear finite element model to show that gage was a significant variable affecting the block shear failure load. Based on the results of his model, he stated that the increase in effective width for a stagger path was not justified. Also in 1988, Madugula and Mohan reviewed the results of several angles in eccentric tension caused by unconnected legs. They documented 13 block shear failures out of 61 tests of angles of various sizes. They found that the ultimate strength defined by the specifications were excessively conservative if the connections were made with fitted bolts.

Adidam (1990), Epstein and Adidam (1991) and Epstein (1992) completed the testing of a full-scale series of double row, staggered and unstaggered bolted connections of structural angles in tension. They concluded that factors of safety in AISC’s ASD and LRFD are inadequate for block shear failures of angles. To correct this, they suggested the use of the shear lag factor, \( U = 1 - \frac{x}{l} \). The shear lag factor was being used as a net area reduction factor to account for the effects of the inherent eccentricity on net section rupture in eccentric tension connections. They recommended this factor be utilized as the \( U_{bs} \) term of the code equations for block shear. The study showed the effect of unconnected legs of angels in tension connections. The eccentricity increased as the area of the unconnected leg increases, resulting in reduced capacity for block shear in addition to net section failures. Concurrently, Chamarajanagar (1990) and Epstein and
Chamarajanagar (1996) used non-linear finite element models to verify these conclusions.

Gross (1994) investigated the performance of angles in tension made from high strength steel. He suggested that the block shear code equations may also be inadequate for high strength steel. As in the University of Connecticut studies, he recommended the use of the shear lag factor in the block shear equations.

Cunningham et al. (1995) worked to develop a unified failure-load equation for block shear in tension members. Utilizing 77 tests conducted between 1982 and 1992 they derived a series of nine equations for block shear failure load. These nine equations provided variations with factors such as the effects from in-plane eccentricity and the aspect ratios of the connection gross and net shear and tension planes. These were factors that were not considered by the current code. One of the nine equations is as follows:

\[
P_{\text{pred}} = 0.55A_{nt}F_{yt} + \left[1.55 \left(\frac{l_{nv}}{l_{nt}}\right)^{-0.25} - 0.1e_s\right]A_{nv}F_y
\]

(1.3)

where:

- \(l_{nv}\) = length of the net shear plane,
- \(l_{nt}\) = length of the net tension plane, and
- \(e_s\) = in-plane shear eccentricity.

They concluded that the variation in the capacity ratios can be significantly reduced by incorporating the ‘aspect ratio’ of the block and the in-plane load eccentricity measured to the bolt line, and failure loads can be predicted with equal accuracy using either the material yield strength or the ultimate strength.

The first edition of AISC's LRFD (AISC, 1986) specified that, for block shear, two equations should be checked and the larger value be used. The second edition of
AISC's LRFD (AISC, 1994) specified that the block shear equation with the larger rupture term should govern. In 1996, Epstein (1996) showed that this minor change did not change the capacity for most connections, but could reduce the design capacity in some connections by as much as 20%.

Until 1996, work on block shear in tension connections had been primarily focused on angles. Epstein and Twilley (1996) conducted a limited testing program of tee sections in tension with the stem of the tee as the unconnected leg. In this investigation, a previously undocumented block shear failure path was discovered. As a result of this study, the National Science Foundation, in conjunction with AISC, commissioned a full-scale study of tension connections in structural tees.

In 1998, as a first step in that investigation, McGinnis (1998) and Epstein and McGinnis (2000) used finite element analyses to predict the mode of failure of the structural tee tension members. They compared the finite element model results to the results of the previous tests (Twilley, 1996). The models accurately predicted the failure modes of the tests, including the previously undocumented alternate failure path. McGinnis' also noted that for large out-of-plane eccentricities, a compressive zone developed in the end of the web at the lead bolt holes. It was found that the moments induced by this eccentricity were not simply the load multiplied by eccentricity, $P_e$. The force couple developed in the reaction at the bolt holes causes opposing moments, thus reducing the moments. Suggestions were made about the inclusion of the shear lag factor, $U$, into the block shear equations for tees.

D'Aiuto (1999) continued the investigation of the nature of the bending moments in an eccentric tension member noted by McGinnis (1998). He used finite element
analyses as an aid in identifying the magnitude of these moments and determining their impact on the design codes. D'Aiuto compared his finite element results to previous data and was able to develop an analytical expression for the moments in structural tees by varying the geometric parameters, connection length, connection eccentricity, and connection stiffness of structural tees. The relationship developed was defined as \( M = Pe - Rl \), with \( Pe \) being the axial force multiplied by out-of-plane eccentricity and \( Rl \) was the moment induced by the force couple along the connection length. This was the product of the connection length \( l \), and the out-of-plane reaction, \( R \) defined as

\[
R = \frac{\left[ \frac{Pe}{2EI} (L-l) \right]}{\left[ \frac{t^2}{6EI (3L-4t)} + \frac{t}{AGtwd} + \frac{l}{K_\theta} \right]} \quad (1.4)
\]

where:

- \( L \) = member length,
- \( E \) = modulus of elasticity,
- \( I \) = governing moment of inertia,
- \( G \) = modulus of rigidity,
- \( t_w \) = thickness of the web,
- \( d \) = section depth, and

\[
K_\theta = \frac{E_p I_p}{l (L_p - l)^2}, \text{ which is the connection stiffness} \quad (1.5)
\]

with:

- \( E_p, I_p \) and \( L_p \) = modulus of elasticity, moment of inertia and length of the connecting plate, respectively.

D'Aiuto also incorporated the moments into the AISC LRFD interaction equations and compared the results to tension equations that use shear lag coefficients.
Orbison et al. (1999) conducted tests on single row bolted tension connections on A36 angles with strain gages placed on the tension plane. They concluded that the net tension in the equations from AISC’s second edition of LRFD (AISC 1994) governed due to the concentration of stress at the bolt hole on the tension plane. This causes the net tension plane to fail prematurely when edge distances are small. They found that small eccentricities, both in-plane and out-of-plane, had little effect on this stress concentration but also stated that larger eccentricities could play a significant role in magnifying these stress concentrations.

Stamberg (2000) and Epstein and Stamberg (2002) performed destructive tension tests on 50 structural tee sections. The specimens had varying out-of-plane eccentricities and connection lengths, with the chosen geometries designed to allow the failures to transition from net section to block shear failure. These tests demonstrated that as connection lengths decreased or eccentricities increased, the efficiency of the connections decreased. The computed design capacities for all 50 specimens were governed by either the net section or ‘alternate path’ block shear failure. Specimens with large web depth to width ratios were noted to experience local web buckling before failure initiation, as suggested by McGinnis (1998).

Barrett (2002) continued on the work at the University of Connecticut presented by D’Aiuto (1999). He utilized finite element analysis to investigate moments induced within steel angles in tension. He concluded that the equation presented by D’Aiuto, \( M = Pe-Rl \), with \( R \) defined in Eq. 1.4 not only provided an accurate representation of the moment in tees, but also accurately predicted the moments in angles.
Topkaya (2004) conducted a parametric finite element study on block shear in gusset plates and angles in tension. He compared the results of over 500 finite element models varying edge, end and bolt spacing, ultimate strength and both in- and out-of-plane eccentricity. He noted that in plane eccentricity in configurations with longer connection length experienced a decrease in capacity. Out of plane eccentricity had no appreciable effect.

Driver et al. (Driver et al., 2004; 2006) attempted to take block shear test data from 205 tests conducted in 17 research studies and produce a unified equation that not only handled block shear in tension members, like the one proposed by Cunningham et al. (1995) (Equation 1.3) but one which also accurately predicted loads in coped beams. The equation developed was

\[
P_u = R_t A_{nt} F_{tu} + R_v A_{g v} \left( \frac{F_v + F_u}{2\sqrt{3}} \right) \tag{1.6}
\]

Where \( R_t \) and \( R_v \) are tension and shear area mean stress correction factors which vary based on the connected part. For angle and tees, both \( R_t \) and \( R_v \) are 0.9. They proposed that this equation replace the current AISC’s equations.

Huns et al. (2006) tried to determine the progression of block shear in gusset plates, and created a finite element model, which included material damage. Through their analysis they found rupture of the tension plane always precedes failure of the shear plane. They also found peak loads to be reached prior to the rupture of the tension plane, confirming that the inclusion of shear capacity of the connection is valid.

Using finite element analysis, Clements and Teh (2012) investigated the location of the active shear planes in block shear connections. Working off of the work of Hardash and Bjorhovde (1985) and Huns et al., (2006), they found that shear failure does
not occur on either the gross section or the net section as previously thought, but instead it occurs on an ‘active shear plane’, located between the net and gross sections. In addition, they also found that yielding is always the mechanism for shear failure.

1.3 Report Structure

The next chapter discusses the details of the experiments performed, comprised of specimen drawings, material properties, instrumentation, and loading protocols.

Chapter Three presents the observations and outlines collected data from the experiments. This chapter includes the general observations, load-displacement responses, and strain data. A comparison of the results is provided as well.

Chapter Four describes the analytical studies. This will cover the formation of the ABAQUS finite element model and the results produced from the models. A comparison between the analytical and tested results is given.

In Chapter Five, recommended improvements to the current design code are presented, based on the results of both the experimental and analytical data.

Chapter Six summarizes the content of the document. Appendices A, B, and C contain the Figures, Tables, and Sample Calculations used in this report, respectively.
CHAPTER TWO – EXPERIMENTAL STUDY

2.1 Introduction

The destructive tests performed as part of this study are detailed within this chapter. This chapter will describe the concept and purpose, details of the specimens, design considerations, material characteristics, instrumentation, test setup, and loading protocols.

2.2 Destructive Tests

A series of tests was performed to determine the effects of in-plane eccentricity and bolt tension on the block shear capacity and ductility of connections utilizing the University of Connecticut’s 400 kip load frame. The in-plane eccentricity noted is in reference to the loading of the plate. The testing configuration required various components: two test plates, two chucks which would be secured to the platens of the load frame, four side plates to provide a means of securing the test plates to the chucks, and two large pins to connect the side plates to the chuck. Figure 2.1 shows a schematic the setup of the test.
Two types of loading were investigated in this study: centric and eccentric. Eccentric loading was applied through a second set of holes in the side plates, as seen in Fig. 2.2. Throughout this thesis, references are made to the offset and non-offset sides of the connection on the test plates. In cases of eccentric loading, the offset side refers to side of the connection opposite the load, and the non-offset side is the side of the connection facing the load. This terminology is used as a standard and is carried to centrically loaded plates as well. Offset and non-offset sides are illustrated in Fig. 2.2.

Specimens were labeled with two or three letters in accordance to eccentricity and level of pretension. The first letter denoted the loading eccentricity, C for centric and E for eccentric loading. Bolt tension was noted as P for pretensioned and S for snug-tight. The test that included Teflon tape was denoted with the letter T. Specimens included: CP: centric pretensioned, CS: centric snug-tight, EP: eccentric pretensioned, ES: eccentric snug-tight and CPT for the centric pretensioned with Teflon.

2.3 Specimen Details, Design Considerations, and Material Characteristics

In order to determine the effects of pretensioning, in-plane eccentricity and friction on block shear, a series of five experiments were developed. Two levels of bolt pretension were used: Snug-tight and fully pretensioned. At each of these pretensioning forces, two levels of eccentricity were investigated, one set centrically loaded, and the second set eccentrically loaded at 1.5 in. Eccentric loading was achieved with a second set of bolt holes in the side plate, offset by the 1.5 in. as shown if Fig. 2.2. In addition,
one additional pair was centrically loaded and fully pretensioned and included the addition of Teflon on the interaction surface between the two plates.

In all cases the test plates were made of $\frac{1}{4}$ in. A572 Gr. 50 steel, and measured 60 in. x 18 in. The connection utilized four $\frac{3}{4}$ in. A325 bolts in a box pattern, giving a nominal bolt shear resistance of 106 kips and a bearing strength of 142 kips per AISC code Equations (J3-1) and (J3-6b), respectively (AISC, 2011). Bolt spacing and gage were both set to 2.5 in., and an edge distance of 1.25 in. was used. On the opposite end of the plate, a 6-bolt (3x2) pattern was used to ensure failure in the other connection. Figure 2.3 shows the design of the test plates used. All holes were standard sized and drilled using a digital readout milling machine with readout accuracy of 0.0005 in. Plate’s edges were squared and zeroed to the machine’s coordinate system using an edge finder and the plate was secured to the table, shown in Fig. 2.4. The mill table was then translated until the coordinates of the bolt holes were reached, and a series of holes were manually drilled until the finished dimensions for the holes were met. First a center drill was used to set the center of each bolt hole (Fig. 2.5), and approximate locations were marked to verify that the machine coordinates were correct. Next, a pilot hole of 0.5 in. was drilled (Fig. 2.6) to reduce the stress on the final $\frac{13}{16}$ in. drill, shown in Fig. 2.7. The block shear capacity of the connection, calculated using Eq. 1.2, taken from the AISC Steel Construction Manual. It was determined to be 88.6 kips.

Specimens were attached to a side plate made of A572 Gr. 50 steel measuring 24 in. x 8 in. x 1.5 in. The side plate was machined to accept the connection at two levels of eccentricity (Fig. 2.2). The same machining process utilized for the test plates was also used in the machining of the connection region (Fig. 2.8). Centric holes were positioned
with an end distance of 4 in., while eccentric holes were added at an eccentricity of 1.5 in. positioned 1.25 in. above the existing centric holes. This was utilized as the level of eccentricity for the test as it was the largest eccentricity capable of fitting within the holes of the centric loading condition. Two pairs of side plates were produced; one with a 4-hole pattern to accept the test connection, and a second with a 6-hole pattern to accept the other end of the test plate. An 8 in. x 2 in. x \( \frac{3}{4} \) in. A572 Gr. 50 spacer was welded to the side plate at the edge nearest the connection for the plate to bear on the side plate on the opposite end of the chuck to prevent out of plane bending (Fig. 2.9).

To connect the side plate to the chuck in such a way that rotation was uninhibited, a pin connection was utilized. In order to carry the load from both of the test plates, a 2.25 in. diameter A354 BD07A pin was used (Fig. 2.10). Holes in both the side plate and chuck were machined to an oversize of only \( \frac{1}{32} \) in. To bring the hole to its finished dimension of \( 2\frac{9}{32} \) in., the parts were placed in an automated Computer Numeric Control (CNC) mill, as the bit used had to be continuously cooled (Fig. 2.11). A hole was centered on the chuck between existing holes from a previous experiment which utilized the 400 kip load frame (Fig. 2.12). This allows the side plates and therefore the connection to rotate, which would not have been possible if the plates were connected directly to the chuck. Chucks were secured in the load platens with serrated wedges intended to increase clamping force as the load increased. Figure 2.13 shows the end connector assembly.

Material properties for the plate specimens were determined using test coupon dimensions outlined in ASTM E8: Standard Test Methods for Tension Testing of Metallic Materials (ASTM, 2011). Since the test plate was 0.25 in. thick, a plate type flat
dog-bone shaped coupon was used, with a specified final width of 1.5 +0.125/-0.25 in. The 18 in. long specimens were cut into 2 in. strips (Fig. 2.14). Using an end mill (Fig. 2.15), the coupon was machined down to the proper dimensions and specifications as outlined in ASTM E8 (ASTM, 2011). The finished specimen width at the necked section was 1.5185 in. with a finished gage length of 8.000 in. Specimen was instrumented with two uniaxial strain gauges, type FLA-5-11-3L, located at mid length, was then loaded into the University of Connecticut’s 60 kip load frame and tested (Fig. 2.16). Yield stress and ultimate stress were found to be 62.4 ksi and 75.0 ksi respectively. Material testing reports provided for the test plate steel reported range for yield stress to be 62.0 ksi to 67.6 ksi and an ultimate stresses range of 72.9 ksi to 78.9 ksi, corroborating our results.

2.4 Test Setup

To test the plates in tension, the first chuck was loaded into the top platen of the 400 kip SATEC load frame. The chuck was secured by two serrated wedges loaded from the top of the platen. Once the top chuck was centered and plumbed, the pin was inserted through the chuck and the 6-hole pattern side plates were installed (Fig. 2.17). On the adjustable middle platen, another set of serrated wedges were used to secure the second chuck. Once held in place, the pin and 4-hole pattern side plates were installed. The load frame was now prepared to receive the specimen.

With the gauges installed, the test plates were ready to be loaded into the load frame. Bolts were inserted into the 6-hole pattern side plate. In order to minimize the likelihood of bolt banging, test plates were hung from the 6-bolts prior to tensioning the
bolts to ensure the bolts were engaged (Fig. 2.18). All bolts in the 6 bolt pattern were installed as snug-tight. To assure uniformity, a torque wrench set to 100 foot-pounds was used. 100 foot-pounds was determined to adequately describe a snug-tight connection, assuming the average iron worker is capable of applying 75-125 pounds of force and the average lever arm on a spud wrench 12 in. – 16 in.

Bolts were then inserted into the 4-hole side plate and the height of the middle platen adjusted so that the test plate could be attached. Once the bolts were through each of the test plates, the jaws of the middle platen were relaxed, releasing the bottom chuck which was now attached to the side plates. This not only allowed for the bolts to be engaged, but also allowed the bottom chuck to be centered and plumbed with the top. Once plumbness was achieved, the bottom jaws were reengaged and the connection bolted (Fig. 2.19). Bolts in this connection were either snug-tight or pretensioned. 100 foot-pounds of torque was used for the snug-tight condition, as was done for the top connection. For pretensioned bolts, the torque required was found to be 240 foot-pounds. This was determined by the use of direct tension indicating washers (DTIs). Standard washers, not DTIs, were utilized in the each of the tests as the DTIs were larger than the standard washers and limited the access to the tension plane for the application of the strain gauges.

DTIs are specially designed washers which are calibrated to the bolt size and grade to indicate when the bolt is fully pretensioned. DTIs have raised nipples designed to collapse as the load on the bolt is increased (Fig. 2.20). The bolt is considered to be pretensioned when three of the five gaps collapse to 0.015 in. or less. Prior to testing, a preliminary test was performed with bolts installed with DTIs. Each bolt was torqued to
a snug tight condition and the gap in each DTI was checked. Bolts were loaded in 10 foot-pound increments of torque and the gap checked at each level until all 8 bolts were determined to be fully pretensioned. The torque required to close the final gap was 240 foot-pounds, and was used in all subsequent tests as the ‘pretensioning’ torque.

The final step of preparation involved adding a whitewash to the plate that did not have the instrumentation installed. The whitewash was created using lime and water. The surface was prepared by first rubbing the surface with a course sand paper and then given an acetone wash; the surface was not taken below the patina. The whitewash solution was then added to the surface and dried using a heat gun until a solid distinguishable coat of white was present (Fig. 2.21). This coating will help to visualize the damage as it occurred.

For the CPT test, a few modifications were made to the setup method outlined above. Prior to installing the plates to the first side plate, Teflon tape was added to the back of each of the test plates behind the test connection. The tape was 2 in. wide and abrasion resistant with a very low coefficient of friction. Surface was washed with acetone and tape was added parallel to the direction of loading such that any surface on the test plate that would be in contact with the side plate was covered in tape (Fig. 2.22). Tape was directly abutted to the adjacent strip ensuring that the entire surface was covered and even, and tape was removed from bolt hole locations (Fig. 2.23). The surface of the side plates were also ground flat to prevent any burrs from catching and tearing the tape. The set up of the bottom chuck also differed with the Teflon test. The bottom chuck and side plate assembly was hung from the test plate and plumbed and centered and the bottom jaws shut. Prior to the bolts being tightened, the machine frame
was lowered, so that the bolts were positioned on the opposite end of the bolt hole. This was done so that the effect of the Teflon coating would be more pronounced.

### 2.5 Loading Protocol

A displacement controlled method was utilized in testing the plates. In each of the five tests, a constant rate of displacement of 2 mm/min (0.0787 in./min) was used. Throughout the duration of the test, photographs were taken at 10 kip intervals until deformation was noticeable. Photographs were subsequently taken more frequently. This provided documentation of the progression of the failure. Plates were tested through rupture of the tension plane.

### 2.6 Instrumentation

Instrumentation was included in order to record strains, displacements and load. Data was recorded using LabView 8.5 interfacing through a NI cDAQ-9178 8-slot USB chassis. Voltage readings were recorded through the NI-9234 IEPE accelerometer/microphone module, capable of recording voltages in a +/- 5 Volt range. Strains were recorded through an NI-9235 8 channel 120 Ohm quarter bridge strain gauge module. The FlexTEST 40 MTS controller for the SATEC 400k load frame records a variety of data including actuator force and displacement readings, actuator force and displacement commands, machine time, and is able to power and record data on up to 4 linear variable differential transformers (LVDTs). The MTS controller had the
capability of sending up to 8 voltage signals to an external module. As the NI-9234 module used was only capable of receiving 4 signals, it was decided that only actuator force, actuator displacement and two LVDT readings would be used. The standard voltage output of the MTS controller was +/− 10 Volts, but the output was scaled down in order to match the +/− 5 Volts of the NI-9234. The LVDTs used had an input voltage of 10 Volts and a stroke of 0.35 in. LVDTs were installed by attaching a L 2.5x2.5x 1/8 to the back of one of the test plates with C-clamps positioned above the side plate. The angle had two holes drilled 6 in apart to accept the LVDTs. The angle was positioned so that each LVDT was 1 in. from the edge of the 4-hole side plate to measure connection displacement (Fig. 2.24).

Uniaxial strain gauges were manufactured by Tokyo Sokki Kenkyujo Co., gauge type FLA-5-11-3L a 5mm x 1.5mm 119.5 Ohm prewired strain gauge with a strain limit of 5%. The rosettes used were manufactured by Micro-Measurements, gauge type CEA-06-125UR-120, a 0-45-90 120.0 Ohm rosette with a strain limit of 5%. Uniaxial gauges had a gauge factor of 2.1; rosettes had a gauge factor of 2.075. Prior to loading the test plates into the load frame, instrumentation was installed onto one of the two plates for each test. Each plate had a total of 8 strain gauges attached. Surface preparation included grinding the patina off the steel to expose the virgin metal. The fresh metal was then roughened with fine sandpaper, and washed clean with acetone. The tension plane was equipped with two uniaxial gauges, positioned parallel to the line of loading. Gauges were placed approximately at the 1/3 points along the net tension plane, to try to capture the greatest difference in tensile stress without interference from the washer. Spacing between gauges was approximately 11/16 in. Each shear plane was equipped with one 0-
45-90 rosette. The rosette was placed midway on the gross shear plane, as preliminary tests run showed a higher degree of damage on the gross rather than net shear plane. The rosette was positioned such that the 45° gauge was perpendicular to the loading direction, therefore the 0° and 90° gauges were positioned at +/− 45° from the normal of the shear plane facing towards the opposite shear plane. Figure 2.25 shows the surface preparation and final arrangement of gauges installed on the test plate. X-Y coordinates for the centerline of each gauge was measured with respect to the plate edges for use in validating the finite element model.

LabView 8.5 was used to log the data from each experiment. A while loop with a manual controlled stop function was created to place all of the virtual instruments (VIs) required for the test procedure. The DAQ Assistant VI was used to set up both the voltage and strain modules. This VI allowed for the communication with and configuration of each of the channels of the voltage and strain modules. Here, scaling factors were applied to each of the channels of the voltage module in order to convert them from voltages to the units they represent. For the strain gauges, the gauge factor, resistance of both the gauge and lead wire, source voltage and initial gauge voltage were input in this VI. Continuous samples were collected at a rate of 1664 Hz for both modules, as the specified minimum sampling rate for the NI-9234 was 1652 Hz. It was found that at this sampling rate, file sizes for the approximately 20 minute tests became unmanageably large. To combat this, each of the DAQ Assistant VI’s output their data into two other VI’s. The first VI wrote the raw data collected from the DAQ Assistant VI into a file as a backup, the second VI performed a simple sample compression, applying a reduction factor of 26, effectively reducing the sampling rate to 64 Hz. These average
values were then fed into secondary VI for writing to output files, creating four files for each test, raw and compressed data for both measured strains and voltages. Figure 2.26 shows the cDAQ assembly; a schematic of the arrangement and wiring of the VIs is shown in Fig. 2.27.
CHAPTER THREE – EXPERIMENTAL RESULTS
AND OBSERVATIONS

3.1 Introduction

The results and observations of the experiments detailed above will be discussed in this chapter. This will include the observed behavior, measured data, analyses of the experimental data, and the design recommendations that were developed based on the measured data.

3.2 Preparation Methods of Data for Analysis

3.2.1 Load Displacement Data

Seven forms of load-displacement relationships were developed to compare the behavior of the connection.

The first three used the raw data directly collected from the load frame displacement and the two LVDTs. The fourth utilized an average reading of the two
LVDTs. It is important to note that the load displacement relationship based on the frame displacement captures the behavior of the entire testing assembly, not just the connection. This is shown in the discrepancy between the frame displacements and the LVDT readings. The three load-displacement relationships obtained by the LVDT measurements (one for each LVDT and one for the average of the two) are able to demonstrate the behavior of the connection. If the slopes of the load-displacement relationships change at the same rate, it shows that the connection was displacing uniformly and no rotation was present, which is to be expected from a centrically loaded test. If the slope of the load-displacement relationships change at different rates, it shows that the connection was not displacing uniformly meaning rotation was present. It was also noticed that in all cases as the load increased both the frame and the LVDT displacement relationships began to displace at the same rate. This is due to the strain localization within the connection. As the load increases the material begins to plastically deform; these plastic deformations, located in the connection, are much larger than the elastic deformations of the other components of the test setup. This means the frame displacement is viable to describe the displacement of the connection once plastic deformation has begun.

Unfortunately, the LVDTs were only able to be utilized in capturing the full plastic behavior at the onset of the softening as the LVDTs only had a stroke of 0.35 in. Knowing that the frame displacement can be used to describe plasticity, the load-displacement relationship of the frame can be appended to the average LVDT load-displacement relationship. This provides the fifth load-displacement relationship. This is able to describe the behavior of the connection through the entire test. The sixth curve
shows the displacement of the connection with the slip removed. Bolts were not able to be fully engaged prior to the test beginning, and thus some slip was evident in each of the tests. This was done using two methods based on the type of slip experienced. For bolt banging, or a more sudden slip, the slip was removed by shifting the load-displacement relationship after the slip to match the curve before the slip, giving one seamless curve. For a more gradual slip the translation method could not be used, as the slopes before and after slip did not match. In this instance, the curve was created by removing all parts of the curve prior to the slip occurring; the remaining curve was completed by continuing the elastic portion of the curve until zero force was reached, and the entire curve was shifted to the origin. The last of the load-displacement relationships was a linearized frame displacement relationship. This curve was created using the same method for gradual slip. Figure 3.1 shows an example of the formation of the linearized relationship. The raw curve is shown in (a). Relationship (b) shows the removal of the bolt bang, while (c) removes the slip and (d) shows the final translation to the origin.

To enable quantifying the effect of the variables in question on the ductility of the connection, both the combined curve without slip and the linearized frame displacement relationship were cut at their point of rupture and the a bilinear relationship was created for each. These relationships were generated by creating two linear regions, one describing the elastic region and another for the plastic. An effective ‘elastic loading rate’, $E$, of the test was obtained by finding the point on the elastic region where the load was half the maximum load, and dividing it by the displacement at that interval. The final values of the plastic region were determined by the maximum load for the test, $F_u$, and the displacement at rupture, $d_u$. Values for the yield force and displacement at yield,
$F_y$ and $d_y$, were determined by equating the energy of both the bilinear curve and the curve from the test. The total energy, $\eta$, up to rupture was found as the area under the load-displacement relationship using the trapezoidal rule. The force and displacement at yield can be found by solving these two equations of Eq. 3.1 simultaneously

$$\begin{align*}
\{ & \eta = 0.5\left[F_y d_y + (F_y + F_u)(d_u - d_y)\right] \} \\
& F_y = Ed_y
\end{align*}$$

(3.1)

With the bilinear curves, the displacement ductility of the connection is able to be determined. Ductility, $\mu$, is defined as the ratio of displacement at failure (displacement at rupture of the tension plane) to the displacement at yield. This would provide a normalized method of determining the ductility of each of the specimens. Displacement ductility values were found for both connection load-displacement relationships and frame load-displacement relationships to help verify the relationships between each of the tests. Figure 3.2 illustrates the formation of the bilinear curve.

### 3.2.2 Strain Data

Six graphs were created to compare the relationship of strains and loads for all of the plates.

For the load-strain relationship on the tension plane, only one graph was required, formed using the data directly from the uniaxial strain gauges and the load cell. This plot shows the relationship between strains on the tension plane to load on the connection. This graph was used to identify whether the stress distribution on the tension plane is uniform, or if an imbalance of stress is present.
To capture the true behavior of the shear planes, the strains acquired by the rosettes was required to be combined using Mohr’s circle formulations. From the rosettes, the directional strains were found for the inclined gauge nearer the tension plane, \( \varepsilon_u \), the horizontal middle gauge, \( \varepsilon_h \), and the inclined gauge nearest the edge of the plate, \( \varepsilon_l \). Figure 3.3 shows the position of the directional strain gauges within each of the rosettes on the plate. The first two graphs created were the values of the maximum and minimum principal strains present within the shear planes versus the load. Using the principals of Mohr’s circle for plan strain and the relation between directional strains in a 0-45-90 rosette to shear strains, principal strains were found using Eq. 3.2

\[
\varepsilon_{\text{max, min}} = \frac{\varepsilon_u + \varepsilon_l}{2} \pm \sqrt{\frac{(\varepsilon_u - \varepsilon_h)^2 + (\varepsilon_h - \varepsilon_l)^2}{2}} \tag{3.2}
\]

In addition to finding the principal strains, the orientation of the principal axis should be known in order to obtain a full understanding of the strains within the shear plane. This created the third graph for the shear plane plotting load vs. principal axis orientation. Orientation to the principal axis was measured off the normal to the shear plane in the direction of the opposite shear plane rotated towards the tension plane. Figure 3.3 also illustrates the direction measured of \( \theta_p \). As the orientation of the 0° and 90° gauges in the rosette were rotated 45 degrees off of the baseline, the equation for orientation of the principal axis, in degrees, was modified to:

\[
\theta_p = 45 - \frac{1}{2} \tan^{-1} \frac{2\varepsilon_h - (\varepsilon_u + \varepsilon_l)}{\varepsilon_u - \varepsilon_l} \tag{3.3}
\]

With the conversion of the raw data \( \varepsilon_u, \varepsilon_h, \) and \( \varepsilon_l \) to the more useful parameters \( \varepsilon_{\text{max, min}} \) and \( \theta_p \), the last two and most critical graphs were able to be created. The first of these two graphs was the value of maximum shear strain vs. load. Maximum shear strain is defined as \( \gamma_{\text{max}} = (\varepsilon_{\text{max}} - \varepsilon_{\text{min}})/2 \). The relation between maximum shear strain and the
orientation of the principal axis could then be used to find the value of the shear strain on
the coordinate axis of the plate. The Von Mises equivalent strain was also calculated
using the formulation in Eq. 3.4:

\[ \varepsilon_e = \frac{1}{1+\nu} \sqrt{0.5(\varepsilon_{\text{max}} - \varepsilon_{\text{min}})^2} \]  

(3.4)

Where \( \nu = 0.3 \) is the Poisson’s ratio. Using the Mohr’s circle relationship, shear
strain is defined as \( \gamma_{xy} = \gamma_{\text{max}} \sin(2\theta_p) \).

3.3 Experimental Data

3.3.1 CP: Centrically Loaded, Fully Pretensioned Test

3.3.1.1 General Observations

Measured dimensions to the location of the strain gauges on the CP test are listed
in Table 3.1. Prior to the test, the frame was displaced until the force was reading one
kip, at that level of preload, the test began and progressed through the rupture of both
tension planes as described in the previous chapter. The initial few minutes of the test
were quiet, as the load carried by the plate began to increase, slowly at first and then at a
steady rate of about 1.3 sec/kip. After about three and a half minutes, a loud bang was
heard. This was the bolt bang associated with the friction between the 4-hole side plates
and test plates giving away. The load dropped from 102.5 kips to 89.1 kips as the bolts
became fully engaged after slipping 0.0109 in. Loading after the slip resumed at the
same rate as before.

The first noticeable signs of damage of the connection occurred near the bearing
of the bolts nearest the edge of the plate, as shown in Fig. 3.4. Shortly after, damage was
visible below the interior bolts as well as on the tension plane. A bulge began to form beneath the connection as the localization of stress began to deform the tension and shear planes. In Fig. 3.5, this bulge can be seen as well as the damage on the tension plane. The rosettes which were once rectangular were now out of square, clearly showing shear deformation within the plane (Fig. 3.5).

As the test progressed, the load began to stabilize. Compared with loading in the elastic region, where it took the load about 1.3 seconds to increase 1 kip, the time required to increase the load for the last 20 kips before peak load was about 10 sec/kip. Peak load for the test was 238 kips for the two plates, or 119 kips per connection. Frame displacement at the time of peak load was 0.759 in. After the peak load was reached the load began to decrease; slowly at first and then accelerating until the tension plane ruptured on one of the plates (Fig. 3.6). Rupture of the tension plane was sudden and displayed a cup-and-cone style ductile failure mechanism. Displacement of the frame at rupture was 0.939 in. The connection was able to displace 0.180 in. after the peak load before rupture occurred. Shortly after the rupture of the first plate, the second plate ruptured.

After the rupture of both tension planes the load stabilized at 171 kips as the shear planes began yielding. The specimen was unloaded when the total frame displacement was 1.172 in. After unloading the test was concluded. Figure 3.7 shows the CP specimen after the test was finished.
3.3.1.2 Load-Displacement Response

The curves shown in Fig. 3.6 are the frame and LVDT based load-displacement relationships. The difference between the frame and LVDT relationships is evident, clearly showing that the load displacement relationship based on the frame displacement captures the behavior of the entire testing assembly, not just the connection. The bolt bang mentioned can be seen in both the frame displacement relationship as well as in the LVDT relationships, showing that there was a degree of slip present in the connection at approximately 100 kips. The slip registered by the LVDTs was also larger than the one shown by the frame, 0.00541 in. compared to 0.00109 in., which verifies the frame displacement accounts for elasticity of the entire model. The three LVDT based curves also fall on top of one another, showing the uniform displacement. The combined load displacement curve is shown in Fig. 3.9, with the connection slip still evident. Fig. 3.10 shows the bilinearization of the last two load displacement curves, the linearized frame and connection load-displacement relationships. Ductility values \( \mu \) for the CP test were 2.230 in/in for the curve based on frame displacement and 32.70 in/in for the connection displacement.

3.3.1.3 Measured Strains

The tension plane strain behavior is presented in Fig. 3.11. The stress in the tension plane is uniform, as is expected from a centrically loaded connection. A similar relationship exists between the behavior of the tensile strain and load as was seen
between the connection displacement and load of Fig. 3.8-3.10. Both plots plateau; however, the softening of the curve is not as evident in the tensile strain plot.

Plots of the shear plane principal axis orientation, major principal strain and minor principal strain are presented in Fig. 3.12, 3.13 and 3.14, respectively. The plot for the load vs. principal axis orientation shows agreement between the behaviors of the two shear planes. Although the plots do not fall on top of one another, likely due to the gauges located differently on the shear planes, the two plots still behave in a similar manner, as a centric test on a symmetric connection is expected to behave. The plot of the major principal strain looks very much alike to the plot of the tensile strain in the tension plane, the minor principal strain is also similar; however, the strains are lower and are compressive rather than tensile. The presence of a tensile major principal strain and a compressive minor principal strain leads to a large maximum shear strain, shown in Fig. 3.15. Behavior of the Von Mises strain in Fig. 3.16 resembles that of the maximum shear strain. The reorientation of the principal axis at the onset of the bolt bang is notable. The bang is not noticeable on the other strain load plots as the bang occurred while the connection was still elastic, unloading along the same curve it was loading on. But the redistribution of forces caused by the bang causes a visible reorientation of both pairs of principal axes. The behavior of each of the shear plane principal axes differs as well. The offset side recovers to the orientation present before the bang occurred. The non-offset side recovers to a different orientation after the bang; this could suggest that an uneven slip occurred between the offset and non-offset sides. Comparing LVDT displacements confirmed this, showing a slip of 0.00734 in. on the offset side and 0.00542 in. on the non-offset side.
3.3.2   CS: Centrically Loaded, Snug-Tight Test

3.3.2.1 General Observations

Measured dimensions to the location of the strain gauges on the CS test are listed in Table 3.1, strain gauge locations can be seen in Fig. 3.17. Beginning the testing of the centric snug-tight test took a while before the load began increasing at the elastic loading rate of about 1.4 sec/kip. No bang was noted during the test, and the load continued to climb at the same rate.

First signs of damage in the connection occurred beneath the bolt hole of the offset side edge bolt. No damage was noticed on the non-offset side at the time of damage initiation of the offset side, see Fig. 3.18. As the damage progressed, deformations became visible beneath the non-offset side edge bolt hole and the offset side interior bolt hole (Fig. 3.19). Damage then became visible on the non-offset side bolt hole and tension plane, and grew down the shear planes (Fig. 3.20). Loading slowed to a rate of 11.3 sec/kip as the test approached its peak load of 245 kips, 122.5 kips per connection. Frame displacement at the time of peak load was 0.951 in. Rupture of the tension plane was sudden (Fig. 3.21), and displayed a ductile cup and cone fracture seen in Fig. 3.22. Displacement at the time of rupture was 1.050 in. Rupture of the second plate occurred shortly after the first tension rupture. Initial stabilization of the shear planes occurred at a load of 179 kips. Frame displacement at onset of unloading was 1.277 in. Figure 3.23 shows the specimen after being removed from the load frame.
3.3.2.2 Load-Displacement Response

Figure 3.24 shows the load-displacement relationships for the CS test based on the frame and LVDT displacements. It can be seen that the 3 plots for the LVDTs were not stacked as was seen in the CP test. This could be due to the test plate having a slight skew at the beginning of the test. If the offset side of the plate was initially slightly lower than the non-offset side, the LVDT would capture displacement on the offset side as it is brought to square before the non-offset side begins registering movement. This would make the offset side LVDT appear to have larger displacements at the same relative load, as is seen in Fig. 3.24. However, during the elastic loading portion the two LVDT based curves are loading at the same rate, showing that the connection was in fact not rotating. Figure 3.25 shows the connection load displacement curve, combining the average LVDT and frame load displacement curves. Figure 3.26 shows the bilinearization of the two measured curves of the CS test. Ductility values for the CS plate were 2.151 in./in. for the curve based on frame displacement and 6.895 in./in. for the curve based on connection displacement.

3.3.2.3 Measured Strains

The tension plane strain behavior is presented in Fig. 3.27. This plot shows that the discrepancy seen in the LVDT readings was not due to rotation as the plots of tensile strain coincide with one another. The behavior of the tensile strain follows the same plateau behavior seen in the load displacement curves. The plots in Figures 3.28 – 3.30
show values for shear plane principal axis orientation, major principal strain and minor principal strain, respectively. The plot of the principal axis further shows the misalignment in the plate. Orientation of the principal axis is different between the two shear planes up until a load of about 87 kips. Referring to the LVDT load displacement curves, at a load of 87 kips, both the LVDTs begin loading at the same rate, showing that prior to that load there was likely an uneven force distribution caused by a slight skew in the plate. Major and minor principal strains both show higher strains in the non-offset side compared to the offset side, which is unexpected as the damage was first visible on the offset side. This could again be due to the rosettes used on the shear planes being positioned in different locations, and to the fact that the rosette was not stacked, making directional strains being read from different locations on the shear plane. Major principal strains were tensile and higher than their compressive minor principal strain counterparts, similar to what was seen in the CP test. The maximum shear strain and Von Mises strain present in the shear planes can be seen in Fig. 3.31 and 3.32, respectively.

3.3.3  EP: Eccentrically Loaded, Fully Pretensioned Test

3.3.3.1  General Observations

Strain gauge locations for the EP test can be seen in Fig. 3.33, and dimensions are listed in Table 3.1. Testing of the eccentrically loaded pretensioned connection began relatively quickly and began gaining load at a rate of about 1.7 sec/kip. After about 3 minutes, a bolt bang was occurred as the connection slipped, dropping the load from 65 kips to 30 kips. The volume of the bang was similar to that of the CP test. Loading then resumed at the same rate prior to the bang.
Before any damage was visible, the connection could be seen to begin to rotate, visible in Fig. 3.34. First damage was visible below the non-offset side of the connection. Damage was expected to initiate from the non-offset side as the moment induced by the eccentricity of the connection causes a force couple in the bolts, increasing the force on the non-offset side and decreasing the force on the offset side. Damage began to progress up through the non-offset side shear plane as a bulge was becoming visible below the non-offset side bolts; all before any damage was seen on either the tension or offset side shear planes (Fig. 3.35). The tension plane and the offset side edge bolt then both began showing signs of damage. The damage on the tension plane was observed to occur more rapidly than the damage on the offset side shear plane. The entire tension plane was experiencing plastic damage before any appreciable damage on the shear plane between bolts on the offset side was noted, seen in Fig. 3.36. The maximum load achieved during the test was 233 kips, a load of 116.5 kips in the connection, at a frame displacement of 0.977 in. Rupture of the tension plane began at a frame displacement of 1.031 in. and was a slow, two step process that did not release significant noise as opposed to the centered connection. Damage began on the non-offset side and over the course of about five seconds, slowly ‘unzipped’ across the tension plane. Fig. 3.37 shows the tension plane of the EP plate in the process of rupturing the tension plane. The rupture mechanism of the tension plane also occurred in two stages. The damage near the initiation site of the non-offset side showed a ductile cup-and-cone style rupture. After the cup-and-cone style rupture progressed along about a third of the way across the tension plane, the failure mechanism transformed to a 45° ductile shear failure for the remainder of the failure. The rupture mechanism of the second plate’s
tension plane was purely cup-and-cone, and the load stabilized to 170 kip after the failure. Softening began to occur as the shear planes began to fail, as the load began to steadily decline. Unloading of the specimen began after a maximum displacement of 1.374 in. was reached. The rotation of the connection at the end of the test is seen in Fig. 3.38. Figure 3.39 shows the EP specimen after the test was concluded. Regions of cup-and-cone failure are labeled as I, and regions of 45° ductile shear failure are labeled as II.

3.3.3.2 Load-Displacement Response

The load-displacement relationships for the frame and LVDT displacements are presented in Fig. 3.40. The plots of the two LVDT displacements show the rotation that was occurring during the testing. The tearing of the tension plane can also be seen in the load displacement curve. The initial sharp drop denotes the cup-and-cone failure, but then towards the bottom of the drop a more gradual sloping region is seen, which is the act of the remainder of the tension plane failing by tearing.

The loading rate differs from one LVDT to the other; this differential slope is what shows the rotation seen in Fig. 3.38. The load-displacement relationship for the connection displacement is shown in Fig. 3.41. The elastic portion of this curve was based on the average of the two LVDT readings. The rotation experienced during the testing caused the non-offset LVDT to disengage during the elastic portion of the load-displacement relationship. In order to extrapolate an average LVDT reading for the portion of the curve where it was not present, the following equation was used:

\[ u_{avg} = \alpha u_{off} \]  \hspace{1cm} (3.5)
\[ \alpha = \frac{45720u_{avg}^3 - 5055.7u_{avg}^2 + 161.48u_{avg} + 0.7401}{1.889655} u_{off} < 0.050326 \\
\geq \frac{0.7401}{0.050326} u_{off} \geq 0.050326 \] (3.6)

Where \( u_{avg} \) is the LVDT average displacement reading and \( u_{off} \) is the offset side LVDT reading. The relation in Eq. 3.6 was found by performing a regression analysis of the ratio of \( u_{avg}/u_{off} \) compared to \( u_{off} \) to find an equation for \( \alpha \). Bilinear curves for the EP test are shown in Fig. 3.42. Ductility values for the EP plate were 2.143 in./in. for the curve based on frame displacement and 6.739 in./in. for the curve based on connection displacement.

### 3.3.3.3 Measured Strains

Tensile strain for the EP tension plane is shown in Fig. 3.43. Here it is seen that the in plane eccentricity has resulted in an imbalance of tensile forces on the tension plane. At a given load, the strain on the offset side of the tension plane is much higher than that on the non-offset side. This explains the unzipping seen in the test, as the rupture initiated from the non-offset side. Figure 3.44 shows the plot of principal axis for the test. Both plots show a lowering of orientation to horizontal as the load increases, with the non-offset side being consistently lower than the offset side orientation. This seems logical as the load being experienced by the non-offset side is higher. The plots of the major principal strain (Fig. 3.45) and minor principal strain (Fig. 3.46) confirm that the strains experienced by the non-offset side are higher than the strains on the offset side. Figures 3.47 and 3.48 show the maximum shear strain and Von Mises strain on the shear planes. The difference between the offset and non-offset maximum shear strains on the shear plane is also a lot more pronounced than the difference between the tensile
strains on the tension plane. This helps to explain why the damage was visible on the non-offset side shear plane before damage was seen on the tension plane.

### 3.3.4 ES: Eccentrically Loaded, Snug-Tight Test

#### 3.3.4.1 General Observations

Dimensions to the strain gauges for the ES test are listed in Table 3.1, and Fig. 3.49 shows the location of the gauges. Loading of the plate began relatively quickly, however before a steady loading rate could be reached, a small bang was heard after about two minutes bringing the load from 21 kips to 16 kips. The bang was not as loud as the bangs heard in other tests. After the bang a steady loading rate of 1.9 sec/kip was reached.

Rotation was evident in the connection prior to damage initiation, as seen in Fig. 3.50. First signs of damage on the plate were evident below both of the bolt holes on the non-offset side. Damage progressed up the non-offset side shear plane and across the tension plane as seen in Fig. 3.51. Damage continued down the offset side shear plane, and loading slowed to a rate of 8.2 sec/kip. Maximum load reached during the test was 233 kips, 116.5 kips in each connection, at a frame displacement of 0.988 in. Rupture of the tension plane occurred in the same method as was seen in the EP test, with the slow tearing of the tension plane from the non-offset side to the offset side (Fig. 3.52). Frame displacement at the onset of rupture was 1.039 in. The second tensile plane ruptured shortly after the first and the load stabilized at 171 kips. Softening of the curve caused by the onset of the yielding of the shear plane began at a frame displacement of 1.325 in. Maximum frame displacement reached during the test was 1.465 in. At the conclusion of
the test the rotation of the plate was apparent as is seen in Fig. 3.53. The plate after the test is shown in Fig. 3.54.

3.3.4.2 Load-Displacement Response

The plots of the frame and LVDT based load-displacement relationships are shown in Fig. 3.55. The bang mentioned in the previous section is hardly noticeable when viewed on the frame based displacement relationship, but is easily visible on the LVDT based relationships. The same tearing behavior of the tension plane is also seen in the behavior of the frame load-displacement relationship. Errors with the non-offset side LVDT caused the readings to be unreasonable early in the test, so the same method of extrapolating an average LVDT reading from the offset side LVDT was used. Because the non-offset side did not have enough data points to draw a strong enough correlation between the average and non-offset side readings, the use of Eq. 3.5 and 3.6 from the EP test were used, as rotations in the connections were similar. The combination of this calculated curve and the frame displacement curve are shown in Fig. 3.56. The bilinear curves for the ES test are shown in Fig. 3.57. Ductility values for the frame based displacement were 1.883 in./in., and 3.255 in./in. when based on the connection displacement.
3.3.4.3 Measured Strains

Figure 3.58 shows a plot of the tensile strain vs. load. Again the imbalance of strain present in the tension plane corroborates what was seen during the test by the tearing of the tension plane from non-offset side to offset side. The orientation of the principal axes in the ES test (Fig. 3.59) shows the offset side maintaining a consistent orientation of around $50^\circ \pm 5^\circ$, while the non-offset side begins the test with a larger angle of orientation, $\sim 60^\circ$, and transitions to a lower orientation of only $35^\circ$ over the course of 50 kips. This reorientation could be due to the reorientation of the plate by the rotation of the connection. The plots of the major principal strain (Fig. 3.60) and minor principal strain (Fig. 3.61) both show that the strains on the non-offset side were much higher than the ones on the offset side. In the plot for the minor principal strain, the minor principal strain actually becomes tensile rather than compressive in the range of 130 kips to 170 kips. Maximum shear strain is shown in Fig. 3.62. This again corroborates the observations made on the tension plane becoming damaged before the offset side shear plane. At 180 kips, virtually no shear strain is present on the offset side shear plane ($\sim 800 \ \mu \varepsilon$), but the tensile strain on the non-offset side tension plane was $\sim 14,000 \ \mu \varepsilon$. Von Mises strain follows the same behavior and is presented in Fig. 3.63.
3.3.5 CPT: Centrically Loaded, Fully Pretensioned Test with Teflon

3.3.5.1 General Observations

The positioning of the strain gauges for the CPT test is shown in Fig. 3.64 and dimensions to the gauges are listed in Table 3.1. As was stated in Section 2.4, bolts in the tested connection were placed on the opposite side of the bolt holes, to ensure that slip would occur. Blue tape was added to the side plate as a reference to the starting edge of the test plate. As the test began, hardly any force was being read by the load frame, however displacements continued to increase. Slip was clearly occurring at the end prepared with the Teflon, made evident by the blue tape as the gap increased (Fig. 3.65). After slipping about 0.1 in., the connection began to take load, and loading progressed at a slower rate of 2.5 sec/kip. When the load registered 35 kips, slip in the plates continued and no additional load was carried, until the plates had slipped an additional 0.059 in. After this second slip, loading began to progress at a rate of 1.6 sec/kip. No bangs were heard during these two initial progressions of slip, but as the plates began to load after the second slip, two quiet bangs were heard. These were likely from the top 6-bolt connection as no Teflon was applied to the interfacing surface and no bang was noted in the LVDT data.

First signs of damage were noted below the edge bolt holes, and the damage began to spread first to the shear planes (Fig. 5.66). Damage then progressed to the tension plane, as the loading rate slowed to 11.0 sec/kip. Peak load for the CPT test was 240 kips, 120 kips in each connection, and occurred at a frame displacement of 1.070 in. Rupture of the tension plane was sudden, and occurred at a frame displacement of 1.238...
in, with the second plate rupturing soon after. The load stabilized 171 kips, but began to lose load as the shear planes yielded. Two shear failures occurred; the first at a frame displacement of 1.445 in. bringing the load to 123 kips, and a second at 1.551 in. bringing the load to about 65 kip. Load then began to increase, and the test was concluded at a displacement of 1.761 in. Figure 3.67 shows the CPT plate after the test.

3.3.5.2 Load-Displacement Response

Load-displacement relationships based on the frame and LVDT readings are shown in Fig. 3.68. The regions of slip as well as the two small bangs noted in the previous two sections are easily visible. The wire leading the offset side LVDT was cut during the second region of slip, but as the test was centrically loaded, and no rotation was visible the readings of the non-offset LVDT was determined to be viable. The offset side LVDT also experienced a strange jump between loads of 108 kips and 127 kips, however readings during this time were able to be translated over, matching with the valid data. The offset side LVDT data was also used in the formation of the connection load displacement curve seen in Fig. 3.69. Initial frame displacement was used as the stroke of the LVDT had not activated until after the first slip. Bilinear curves for the CPT test are shown in Fig. 3.70. Ductility values for the two bilinear curves were 2.045 in./in. based on frame displacement and 12.56 in./in. based on connection displacement.
3.3.5.3 Measured Strains

Figure 3.71 shows the relationship between tensile strain and load in the CPT plate. Strains on the plate are uniform, showing that the eccentricity included within the test plates, both for the Teflon test as well as the two eccentric tests, did not affect the strains on the tension plane. Figures 3.72-76 show the behavior of the shear plane through plots of the principal axis orientation, major principal strain, minor principal strain, maximum shear strain, and Von Mises strain. Plots for minor principal fall on top one another for the two planes, while there is a slight discrepancy between the maximum principal strain as the load increases, showing a slightly higher strain on the non-offset side. This relationship is also transferred to the maximum shear and Mises equivalent strains. Orientations of the principal axes follow the same trends, but at slightly different values likely caused by minor variation in relative placement.

3.4 Comparison of Test Results

Tables 3.2 and 3.3 contain the bilinear values based on frame displacement and connection displacement, respectively, for all tests. Figures 3.77 and 3.78 present all load-displacement relationships for connection displacements and frame displacements, respectively. All test results were compared using a similar method. Percent change in the values in question ($F_y$, $F_u$, $\mu$, etc.) were found for the variable of interest (pretensioning, eccentricity or friction) for both the connection and frame based bilinear values. These changes were averaged together to assess the effect the variable had on that particular configuration. In the cases of pretensioning and eccentricity, the changes
for each of the two pairs were averaged to determine if a universal trend existed for the variable in question. Changes in values less than \( \pm 1\% \) were determined to not be affected by the variable, where changes greater than \( \pm 10\% \) were considered to be affected. Changes between these two margins suggest that the variable may have an effect.

3.4.1 Effects of Pretensioning

This section compares the results of the CS and ES tests to the results of the CP and EP tests, respectively. Table 3.4 contains the effects of pretensioning on the values of the bilinear curve. Tables 3.5 and 3.6 show the changes in the frame and connection based values, respectively, for both centric and eccentric loading cases. During the tests it was noted that both tests with pretensioned bolts experienced bolt banging. These are evident in the load-displacement relationships for the centric test (Fig. 3.9) and eccentric test (Fig. 3.41). By comparison, the CS test did not bang (Fig. 3.25), instead the bolts slipped, similar to the CPT test. Although the ES test did have a bang seen in Fig. 3.56, the relative volume of the bang and the load on the connection at the time of the bang compared to the pretensioned tests were both lower. The lower clamping force by the snug-tight bolts results in a lower force required to cause slip in the connection when the coefficient of friction is held constant. This makes it so less energy is stored by the friction force prior to slip. This results in a quieter bang.

The level of pretension was found to affect some aspects of the load-displacement and ductility values. It was found that in the tests with pretensioned bolts, the slopes of the elastic region were higher when compared to their snug-tight counterparts. Slope of
the elastic region was found to increase 162% over the two comparisons; 228% in the
centrically loaded case, and 96.5% for the eccentrically loaded case. It is possible that
the pretensioned bolts allow for additional load transfer through friction, and where this
would not increase the capacity of the connection, it could be observed as an increase in
the loading rate in the elastic region of the curve. This was also observed in the loading
rates of each of the tests described earlier in the chapter. There was no discernible
difference when comparing the two types of bilinear curves between the yielding forces
for the level of pretension. However, it was found that the displacement at yield was
found to be lower (-42.5%) when pretensioned bolts were used, due to the higher slope of
the elastic region. There seemed to be a decrease in the slope of the plastic region with
the use of pretensioned bolts, with an overall loss of 8.28%. This decrease in plastic
region slope would suggest a more ductile connection, assuming no major effects on
yield and ultimate strengths. Values based for the centric condition decreased more
significantly, reducing the plastic region slope by 18.4%. However, the eccentric case
saw a slight increase in the plastic region slope, 1.84%, with the addition of pretension
forces. There was a decrease of 2.78% observed in ultimate load in the centric loading
case. This gives credence to the increase in triaxial stresses caused by the clamping force
of the bolts. No change was noted in the ultimate capacity of the eccentric loading
condition. It is likely the effects of eccentricity had more of an effect than that of the
pretensioning. Rupture displacement dropped by 10.4% in the centric loading case with
the addition of pretensioning, and by 13.7% for the eccentrically loaded plates. This is
significant because it means the addition of pretension limits the mobility of the
connection. There is evidence to help support this in the decrease of energy absorbed by
the connection. Energy absorbed decreased 8.11% across the two tests, showing that the increase in pretensioning force limits the energy a connection is capable of absorbing prior to fracture. Displacement ductility of the connection was noted to increase with the addition of pretensioning. Connections with centric loading had connection ductility values increase 189% with the addition of the pretension. This large jump is due to the increase in the elastic loading rate, and a decrease in the plastic loading rate. Eccentric test connection ductility values increased 60.4% when pretensioned. The jump was not as pronounced because the plastic region slopes did not change as drastically as in the centric loading condition.

Pretensioning did not have an effect on the behavior of the strains in either the tension or shear planes.

3.4.2 Effects of In-Plane Eccentric Loading

This section compares the results of the CS and CP tests to the results of the ES and EP tests. Tables 3.7 and 3.8 show the changes in the frame and connection based values, respectively, for both snug-tight and pretensioned loading cases with a change of in-plane eccentricity. Table 3.9 contains the combined effects on these parameters on the bilinear curve. The addition of in plane eccentricity in the loading greatly affected the model. The most obvious effect was the rotation experienced by both of the tests with eccentric loading. The rotation present in the eccentric test is shown in Fig. 3.53. This rotation had an effect on every measured aspect of both the load displacement and strain behavior of the tests.
The addition of eccentricity decreased the rate of loading in the elastic region. Values for the elastic loading rate dropped 52.1% with pretensioned bolts when eccentricity was introduced, and dropped 46.2% under snug-tight conditions. This is likely due to the localization of stress during the elastic loading, placing more force on one half of the connection than the other. Values for the yield force tended to decrease with the addition of eccentricity, likely again due to stress localization. Yield force values for pretensioned bolts lowered 9.64%, while snug-tight connections were more pronounced, lowering 10.3%. This caused an increase in displacement at yield, averaging 173% over both cases. Slope of the plastic region increased with the addition of eccentric loading on the connection. This is also caused by the rotation of the connection. During the elastic loading, most of the strain was localized on the non-offset side. This left the offset side still relatively intact when the connection transitioned to the plastic region. This is likely the cause of the increased plastic region slope. Pretensioned connections plastic region slope increased 81.4%, snug-tight connections had an increase of only 44.1%. Ultimate loads tended to drop when eccentricity was added. Pretension ultimate capacity dropped 2.06%, and snug-tight capacity dropped 4.49%. The drop in ultimate load is likely due to the premature rupture of the tension plane due to the imbalance of force on the plane. Energy absorbed was by the connection was not found to affected by the presence of eccentricity very much, only increasing 1.72% across the two tests. Displacement at rupture was found to increase only 16.3% across both tests. Although increasing, the increase is not as significant as the increase in yield displacement, causing a decrease in ductility. Ductility decreased 41.7% in the pretensioned connections when eccentricity was included and dropped 32.6% in snug-
tight connections. The lower ductility present in these eccentrically loaded connections means that there is less warning prior to failure.

Eccentricity also affected the behavior of the strains in each of the legs of the connection. In the centrically loaded tests it was found that the strains in the tension plane (Fig. 3.11, 3.27) and shear plane (Fig. 3.13-16, 3.29-32) were uniform from one side to the other, regardless of the level of pretension. Minor variations were present, but the general behavior of each side remained the same. By comparison, the appearances of the eccentric tests are much different. Tensile strains (Fig. 3.43, 3.58) on the non-offset side begin to increase during the elastic region of loading, opposed to the slight divergence seen in the CS test where the difference only occurs in the plastic region. Shear strains (Fig. 3.45-48, 3.60-63) are even more dramatic, as the non-offset side goes plastic and strains become too large to read even before the offset side begins to experience plastic deformation.

### 3.4.3 Effects of Reduced Friction

This section compares the results of the CP test to the CPT test. Table 3.10 contains the effects of a reduction in friction on the values of the bilinear curve, Table 3.11 shows these changes separated into frame and connection based values. The most evident consequence of reduced friction is the loss of the bolt bang. Where the CP test experienced bolt banging, the CPT simply displaced until bolts were engaged.

It was found that the loading rate of the elastic region decreased when friction was reduced. The slope of the elastic region was found to decrease 42.1%. This would
support the theory of elastic loading via friction transfer. A slight increase of 3.98% in yield load was noted when friction was removed. This could suggest that traction forces present between the two plates may cause an early onset of yielding. The decrease in elastic loading rate and increase in yield load would explain the 120% increase in the yield displacement. Slope of the plastic region was increased to 6.26%. This could help to corroborate the idea that traction forces induce damage on the plate during elastic loading. An increase in ultimate displacement of 14.7% was observed, likely due to the change in elastic region loading rate. Energy absorption was found to increase 11.7% as friction was reduced, making the performance better under dynamic and impact loading. The differential increase of the yield and ultimate displacements resulted in a 35.4% decrease of ductility. This loss of ductility with the reduction of inter-plate friction is concerning. This suggests that unsatisfactory preparation of slip-critical surfaces has the ability to make a connection more vulnerable to a sudden failure.

The reduction in friction did not affect the strains in the plate. This was expected as the plate was still centrically loaded. The major mechanism for force transfer in each case was bearing of the bolts on the bolt holes.
CHAPTER FOUR - ANALYTICAL STUDIES

4.1 Introduction

This chapter focuses on the finite element (FE) analytical modeling of the specimens and the validation of the models using the experimental results. One base model was created, and calibrated with the results of the CPT test. The materials, interactions, loading scheme and mesh were then utilized in the configuration of the other test models.

4.2 Finite Element Modeling

4.2.1 Introduction and Scope

As was shown in Chapter 3, bolt tension, eccentricity, and interface friction all play a role in determining the behavior of a block shear connection. Changes were noted in both the load-displacement and load-strain response of the connection with these parameters. It was decided that a finite element model would help to confirm the data collected from the destructive test.

The finite element modeling software used was ABAQUS 6.11-1 (Dassault, 2011). ABAQUS was chosen due to its ability to handle the non-linear material
behavior, contact modeling, and high degree of geometric nonlinearity that would be required of the models being created. The model the connection, a three dimensional finite element model was created of the test plate, side plate, and bolts to simulate the load on one of the plates, the same test performed in the tests on the load frame. The bolts and the immediate area in the vicinity of the connection for both the test and side plates were modeled using solid elements, while the remainder of the side and test plates were modeled as beam elements (Fig. 4.1). Beam elements were only extended to the location of the center of the pins for application of the boundary conditions. The test plate was further divided into two regions: the connection region and the non-connection region. The plate was positioned such that the bolts were fully engaged and the model could be matched to the linearized connection displacement curve described in Chapter 3.

4.2.2 Modeling Method

4.2.2.1 Material Model

There were three materials used in the creation of the model. Material mass density for all parts in the model was set to the density of mild steel: 283.6e-6 kip/in$^3$. The side plate was modeled as a perfectly elastic material, with the modulus of elasticity equal to that of the test plate steel: 28,721 ksi, and Poisson’s ratio of 0.3. Bolt material was also modeled as elastic, with a slightly lower elastic modulus of 27,721 ksi typical of high strength bolt alloy. Material for the test plate was modeled as elastic-plastic using true stress-logarithmic strain relationship calculated from the stress-strain relationship from the coupon test.

\[ \varepsilon = \ln(1 + e) \quad (4.1) \]
\[ \sigma = s(1 + e) \] \hspace{1cm} (4.2)

Eq. 4.1 and 4.2 were used to convert the stress-strain \((s,e)\) relationship to a true stress-logarithmic strain \((\sigma,\varepsilon)\) relationship. A comparison of the stress-strain to the true stress-logarithmic strain curves is available in Fig. 4.2. The modulus of elasticity for the plate was found to be 28,721 ksi and a Poisson’s ratio of 0.3 was utilized. Table 4.1 outlines the plastic logarithmic strain-true stress associated with the test plate material model. Plastic logarithmic strain \((\varepsilon_p)\) was found by removing the elastic strain \((\varepsilon_e = \sigma/E)\) from the total strain \((\varepsilon)\) by using Eq. 4.3.

\[ \varepsilon_p = \varepsilon - \frac{\sigma}{E} \] \hspace{1cm} (4.3)

Plasticity was defined using classical metal plasticity and utilized isotropic hardening. Isotropic hardening involves the uniform growth of the yield surface in all directions as plastic straining occurs. The Hill’s anisotropic potential function was applied to the material. This allows for manipulation of material strength in all three normal and mixed directions. Using this, the shear capacities were increased 10%. Mass proportional damping with \(\alpha = 100\) was used to control the displacement of failed nodes.

To model the damage of the material, criteria had to be set for damage initiation, damage evolution and removal of failed elements. A ductile damage criterion was used to model the damage in the material. This models the damage progress caused by the formation, propagation and coalescence of voids in the material. Material damage model was defined using a FE model of the test coupon and comparing the load-strain relations. Damage initiates in the material when the plastic strain reaches a predetermined level. Damage was defined to initiate when the Mises plastic strain of the material reached a value of 0.7. Damage can then be set to evolve, based either on displacement (strain) or
through energy; softening of the curve can be set to either a linear or exponential function. Damage evolution was defined as energy based, with linear softening. Fracture energy was defined as 0.01 kip-in. Material failure is then achieved by removal of failed elements from the mesh. This is done by setting a threshold elements were removed from the model when the stress in the element had fully degraded.

4.2.2.2 Contact Interactions and Constraints

In order to properly define the motion of the nine parts, a series of constraints and interactions had to be defined. A tie constraint was made from the surface of the non-connection region to the connection region using surface to surface discretization. This allows the two regions of the plate to be continuous at their intersection. In order to connect the solid and beam regions of both the test plate and side plate, two definitions had to be made for each connection. First, a kinematic type coupling constraint was made from the end point of the beam element to a reference point defined at the same point. This enabled the reference point to move with all of its degrees of freedom tied to the end of the beam element. A tie type rigid body was then created from the reference point to the adjacent surface of the solid element. This caused the entire surface of the solid object to be tied to the translation and rotation of the reference point, and subsequently the movement of the end of the beam element. This ensured a seamless connection between beam and solid elements.

The solid element of the side plate and test plate were connected with the solid bolt elements. This was achieved by a contact definition. The general contact option for
explicit analysis was used. This defines the contact between all of the surfaces in the model which will come into contact, rather than having to define 17 identical surface-to-surface definitions. Three sets of contact properties were developed to implement in the model. The first involved the contact between the bolt shafts and plate regions, the second handled the contact surface between the two plates, and the third handled contact between the bolt heads and plate. For each, both normal and tangential behavior had to be defined. For the normal direction, the pressure-overclosure relationship was defined as a “hard” contact for all contact properties. This method utilizes the Lagrange multiplier method of constraint enforcement. Tangential behavior for each property was defined using the penalty method of friction enforcement. The friction coefficient for the surfaces in contact with the bolt shaft was defined as 0.55 and the interface of the bolt heads to plate as 0.95. Friction for the plate to plate contact differed between the standard model, \( \mu=0.3 \), and the model with Teflon, where the friction was set to 0.05.

4.2.2.3 Boundary Conditions and Loading

Boundary conditions for the model consisted of pin supports at the unsupported ends of each of the beam elements. The bottom support had translation restraints in all three directions as well as rotational restraints to resist any out-of-plane bending as well as any torsion the model may experience. The rotational degree of freedom for in-plane bending was kept free. The top support was restrained for lateral motion both in and out
of plane, with the same out-of-plane and torsional restraints on rotation. On this support, both the in-plane rotation and axial translation degrees of freedom were not constrained.

Two loading definitions were created to account for the pretension in the bolts and the loading of the connection. Bolt pretension was defined with the use of the predefined stress field. To do this, a set was defined containing all of the elements constituting the shaft of the bolt. A uniform normal stress field directed along the axis of the bolt was defined. This would cause the bolt to shrink and clamp onto the plates, and the amount of pretension was able to be calibrated with the magnitude of the stress field. It was determined that a predefined stress field of 166 ksi was required to achieve the 28 kips defined for the pretension force (AISC, 2011). Predefined stress field for the snug-tight definition was set as 70 ksi to achieve the same proportion of prestress as the torque used. The model was loaded by assigning a point velocity definition to the end of the beam element constituting the side plate. Time history of the velocity is shown in Fig. 4.3. \( V_{\text{max}} \) was defined such that total displacement after the test would be \( \frac{15}{16} \) in. This final displacement was defined as it would capture both the failure, and post-failure behavior.

4.2.2.4 Meshing and Element Types

Elements used for meshing the beams were the ABAQUS B32 element. This is a three-node, beam element with quadratic formulation. Each node has six degrees of freedom, three translational and three rotational. Transverse shear in the element is accounted for using Timoshenko beam theory.
Solid elements used were C3D8R. This is an eight node linear brick element with reduced integration. Each node has six degrees of freedom. Kinematic formulation for these elements differed, and was based on the region where they were located. Bolt heads and the connection region were computed using the standard average strain method. Kinematic formulation for the bolt shaft and plate exterior were set to orthogonal formulation. Orthogonal formulation utilizes the centroidal strain operator and modifies the hourglass shape vectors slightly to reduce computational time. This technique was used in these regions as the computed strains were not as critical as in the connection region. Side plate solid elements were formulated using centroid formulation. This technique uses the centriodal strain operator, and replaces the hourglass shape vectors with hourglass base vectors. Using the base vectors in lieu of the shape vectors reduces computation by a factor of three, but can lead to hourglass strain being developed. This formulation was chosen for the side plate due to the large number of elements in this region, and the fact that strains within the side plate were not of concern. Elements in the connection region had hourglass formulation using relaxed stiffness. This uses the integral viscoelastic approach and was chosen for its performance in preventing the collapse of elements in contact with the bolts.

Meshes for the connection region and side plate are shown in Fig. 4.4 and 4.5 respectively. The assembly mesh is shown in Fig. 4.6.
4.2.2.5 Analysis Method

ABAQUS/Explicit was used to run the analysis to deal with the material degradation and failure. A dynamic step was used and no rate dependent properties were assigned, creating a quasi-static analysis. A quasi-static analysis in ABAQUS/Explicit attempts to create a static analysis through dynamics by minimizing the energy associated with a dynamic analysis. This requires using as long a time step as possible to minimize the inertial effects. A time step of five seconds was chosen as a balance between computational efficiency and limiting the dynamic effects of the loading. Nonlinear geometric assumptions were made in order to agree with the true-stress logarithmic-strain values used in the material property definition. A mass scaling factor of 4 was also applied on the regions of the bolts and the connection region of the test plate. The quadratic bulk viscosity parameter was increased from the standard value of 1.2 to a value of 2.4. The quadratic bulk viscosity parameter is used to help prevent the collapse of elements under compressive forces.

4.2.3 Analytical Results

Results from the FE modeling were used as a means of comparing trends seen in the destructive testing. Failure progressed similarly in the finite element model than in the destructive tests. In the destructive tests, after the tensile plane failed, the shear planes maintained the load before they began to fail; in the finite element model, shear plane did not maintain the load as long; this is likely due to the dynamics of the FE model. In
eccentrically loaded cases, damage progression in the tension plane of the finite element model matched the destructive tests, slowly tearing the tension plane from non-offset side to offset side. Despite this, comparisons can still be made between the FE models to verify trends seen in the destructive tests. Figure 4.7 gives a comparison of all FE load-displacement relationships.

4.2.3.1 CPT: Centric Loading, Pretensioned Test with Teflon

The load-displacement comparison between the destructive test and the FE model is presented in Fig. 4.8. The softening observed in the FE model was not as pronounced as was seen in the destructive test, but still showed softening compared to the CP test (Fig. 4.7). The larger displacement at rupture of the connection compared to the case with friction was not seen. Peak load predicted by the finite element model was also slightly higher, 247 kips, than was seen in the destructive test. Strain data was taken from the centroid of the element where the strain gauges were centered. The relation comparing the strains on the tensile plane between the destructive test and the FE model show the same relationship (Fig. 4.9). A comparison of maximum shear strains on the shear planes show correlation with the destructive test (Fig. 4.10). The agreement seen in both the tensile and shear strains validate the FE model. Figure 4.11 shows the deformed shape of the connection after failure. The deformation of the bolt holes matches the deformed shape witnessed in the destructive testing. Tension plane rupture occurred at the shortest region between the bolt holes and the shear planes showed failure extending along the active shear plane, located between the net and gross planes. The distinctive ‘dog bone’
shape of the interior bolt holes is present in the FE model, and dislocations at the edge bolt holes also match the destructive test.

4.2.3.2 CP: Centric Loading, Pretensioned Test

The load-displacement comparison between the destructive test and the FE model is presented in Fig. 4.12. The elastic loading rate has been captured, but the FE model did not experience the same degree of softening as was experienced by the destructive test. FE results also predict a higher ultimate load, 250 kip, and later failure than was observed in the destructive test. No shear strength was observed after the rupture of the tensile plane as was seen in the destructive tests. Plots of the tensile strains and maximum shear plane strains for the FE model are shown in Fig. 4.13 and 4.14 respectively. Strains on the tension plane in the FE model undergo the same progression as was observed in the destructive test, corroborating the results. Shear plane maximum shear strains in the FE model show the same relations as the tensile strains. Damage to the plate after testing, presented in Fig. 4.15, shows the same post failure appearance as the plate from the follows the same patterning as was seen in the destructive test. The ‘dog bone’ shape of the inner bolt holes and the ‘cat eye’ pattern seen in the edge bolt holes were the same on the destructively tested plate.
4.2.3.3 CS: Centric Loading, Snug-Tight Test

The load-displacement comparison between the destructive test and the FE model is presented in Fig. 4.16. The softening of the curve of snug-tight bolts compared to pretensioned was seen, but was not as pronounced as in the destructive test. The FE model did not predict the increased ultimate load compared to the pretensioned case, rather it predicted the same ultimate load prediction, and the rupture of the connection also occurred at the same level of displacement. Plots of the tensile strains and maximum shear plane strains for the FE model are shown in Fig. 4.17 and 4.18 respectively. Strain behaviors of the tensile plane of the FE model show the same behavior as those in the destructive test, falling directly on the values. Shear strains also show the same relationship. Fig 4.19 illustrates that the damage seen in the plate of the finite element model match the results seen in from the destructive tests.

4.2.3.4 EP: Eccentric Loading, Pretensioned Test

Creating the EP test required taking the same parameters found in the CP model and moving the connection to the eccentric level. The load-displacement comparison between the destructive test and the FE model is presented in Fig. 4.20. The reduction in elastic stiffness is evident, as are the multiple stages of softening in the elastic region, but the response predicted by the FE model were much stiffer than that of the destructive test. Reduction of capacity and increase in displacement at failure were both predicted by the finite element model, though the reduction of the ultimate load was not as severe as that
of the destructive test. The FE model also accurately demonstrated the tearing of the
tension plane witnessed in the destructive test. Strains were found concentrated on the
shear plane nearest the line of loading, and tensile strains were found to be higher on that
side of the tension plane as well. Plots of the tensile strains and maximum shear plane
strains for the FE model are shown in Fig. 4.21 and 4.22 respectively. Predictions of the
strain behavior for both the tensile and shear strains were accurate when compared to the
destructive data. Figure 4.23 illustrates the failed shape of the FE plate. Damage is
nearly identical, showing little damage on the offset side edge bolt hole, and a near
vertical stretch pattern in the interior non-offset side bolt hole.

4.2.3.5 ES: Eccentric Loading, Snug-Tight Test

Testing of the ES test required taking the same parameters found in the CS model
and moving the connection to the eccentric level. The load-displacement comparison
between the destructive test and the FE model is presented in Fig. 4.24. The softening of
the curve was still pronounced, and had a softer curve compared to the EP condition, but
the FE prediction was stiffer than the destructive counterpart. Like in the destructive test,
the ES test in the FE model had both the lowest ultimate force and the largest ultimate
displacement. Plots of the tensile strains and maximum shear plane strains for the FE
model are shown in Fig. 4.25 and 4.26 respectively. The results demonstrate the same
trends seen in the destructive testing, following the progression of shear and tensile
strains in the connection. Like in the FE model for the EP condition, strains were found
to be concentrated on the non-offset side. This is able to solidify that in-plane eccentric
loading on tension members is able to create the same imbalance of strains on the tensile plane as double-columned bolts in coped beam connections. Fig. 4.27 shows the damage in the FE plate, in agreement with the damage seen after the destructive test.
CHAPTER FIVE – DESIGN

RECOMMENDATIONS

5.1 Introduction

This chapter will discuss the findings of this study with respect to the block shear failure of connections. It will compare the results of both the destructive and analytical tests and discuss the implications on current design practice. Recommended alterations to the current design code will also be made.

5.2 Recommended Alterations to the Current Design Code

Design values as calculated using Eq. 1.2, the current AISC design equation, give overly conservative values for high-strength steel. In order to improve the efficiency of construction, load predictions given ought to predict the actual capacity of the connection. In the past, this has been measured using a ‘professional factor’ which was defined as the ratio of the actual capacity to the predicted capacity. Design equations should yield a professional factor of 1.0, conservative estimates are greater than one, non-conservative estimates are less than one. Predicted capacity based on Eq. 1.2 yields an
ultimate capacity of 88.6 kips when using the material properties derived from the coupon test. The average ultimate strength for all five tests was 118.9 kips. This yields a factor across all five tests, as AISC design capacity is the same for all five, of 1.34. The fact that the shear planes held after the rupture of the tension plane allows for both the shear and tension capacities of the equation to be checked independently. Of the 88.6 kip block shear capacity of the connection, 57.0 kips is taken by shear, and 31.6 kips is taken by tension. Across the five tests, shear plane stabilization after tensile rupture occurred at 86.2 kips, leaving the remaining 32.7 kips to be taken by tension. Comparing these to the separated value shows that it is the shear capacity of the material which is underestimated by a factor of 0.66; nearly identical to the 0.6 reduction factor seen in Eq.1.2. This suggests that a lower reduction factor for the shear strength of the connection may be appropriate. Equation 1.1, from the Ninth Edition ASD by AISC, the calculated capacity was even more conservative, coming to 44.3 kips, a professional factor of 2.68. Shear capacity was predicted as 28.5 kips (0.33 reduction) and tensile capacity was 15.8 kips (0.48 reduction). These reductions again mirror the 0.3 and 0.5 reductions taken by the equation for shear and tensile capacity, respectively. By comparison, Eq. 1.3 proposed by Cunningham et al. (1995) predicts an ultimate capacity of 110.4 kips for the centric connection, and 94.5 kips based on the eccentric condition. This yields a professional factor of 1.09 for the centric connections and 1.23 for the eccentric connections. Overly conservative design equations result in overdesigned connections which ultimately raise the cost for both the fabricator and the client. As the higher strength steels become more prevalent in design, design equations must change to account for this, so the safest, most efficient structures can be created.
5.2.1 Recommendations on Pretensioning

Pretensioning forces had a negative effect on capacity of the centrically loaded tests. While the reduction seen was small, only 2.78%, the full effect of the triaxial stresses and stress concentration has on other types of failures, like net section rupture, is unclear. Although not a significant reduction, the designer and fabricators should be aware that higher levels of pretension have the ability to reduce the capacity of the rupture of connections in tension.

The designer should be aware of the reduction in energy absorbed prior to failure in pretensioned connections. This could be detrimental in members experiencing impact or dynamic loading, causing premature failure. This suggests that the members prone to impact or dynamic loading ought to be constructed with snug-tight joints, to decrease the likelihood of failure under this loading. The smaller displacement required to yield the connection helps corroborate this.

The displacement ductility of the connection did increase under pretensioning. This could pose issues if a pretensioned joint was called for and the bolts were not properly tensioned. The loss of ductility would result in failures that happen with less warning than more ductile connections. Care must be taken to ensure that the proper degree of tension is installed on the bolts, whether snug-tight or pretensioned.
5.2.2 Recommendations on In-Plane Eccentricity

In-plane eccentricity had a tremendous effect on the behavior of the block shear connection. Not only were capacities seen to reduce as much as 5%, but the distribution of stresses and progression of damage and failure were also affected.

The plot of the tensile strains for these two eccentric tests (Fig. 3.43, 3.58) clearly show a non-uniform distribution of stress on the tension plane of eccentrically loaded tension members, which is not predicted by the AISC code. This non-uniform tension plane stress is supposed to give such a member a reduced $U_{bs}$ value of 0.5. The use of a reduction of 0.5 is likely overly conservative, especially when the existing equation predicts lower capacities for high-strength steel, as in this case. The use of another, less conservative reduction value, such as the shear lag factor, may be more appropriate. This could account for the 5% reduction in ultimate capacity.

In addition to the reduction of capacity with the addition of in-plane eccentricity, displacement ductility was also found to reduce. This reduction in ductility would lead to less warning before a failure. This has the potential of being catastrophic when coupled with the capacity reduction. If the eccentricity in the loading is not accounted for, these connections would have lower than expected capacities that are also less ductile, potentially causing a failure before the problem is even noticed.
5.2.3 Recommendations on Reduced Friction

Care should be taken in the preparation of surfaces intended for slip-critical connections. No loss of load capacity was observed when the friction was reduced, but there was a significant loss in the ductility of the connection. This loss of ductility would create a connection with less of a warning before failure. The loss in friction was extreme in the case presented here; however it does show a trend of lowering ductility.

The reduction of friction also showed an interesting trend of increasing the energy absorbed by the connection prior to failure. This is interesting as it provides a simple way to help to increase the performance of structures under dynamic loading, such as blast loading. If the reduction of friction enables more energy to be absorbed, a simple way to decrease the likelihood of catastrophic failure under a high energy loading situation would be to reduce the friction between connected elements in all connections, with the exception of slip-critical ones. If this is proven valid, this could provide a remarkably cheap and simple means of helping make structures more resilient to high energy loading situations.
CHAPTER SIX – SUMMARY AND CONCLUSIONS

6.1 Summary

Block shear is a relatively recent failure phenomenon characterized by a combination of failure modes: tensile rupture and shear yielding. The capacity of these connections is affected by a variety of material and geometric variables. A series of experiments was developed in order to quantify the influences of bolt pretension, in-plane eccentricity, and friction.

Five tests with identical connection geometry were tested under varying conditions of bolt tensioning and in-plane eccentricity. Load-displacement data was gathered for both the global behavior as well as the localized effects. Strain data was also used to illustrate the force distributions in the connections.

Data from both the load displacement and strain readings were then utilized to calibrate a finite element model of the connection, including the bolts. This model included criteria for material degradation and failure. The tensioning forces seen in the bolts, cases of eccentric loading and inter-plate friction were all investigated.
6.2 Observations

In the course of the investigation, it was found through the destructive and finite element testing that:

1. Capacities predicted by the current AISC design code underestimate the capacity of the connection, with a professional factor of 1.34. The majority of this discrepancy comes from the shear term of the block shear equation.

2. Centrically loaded plates had uniform strains on both the tension and shear planes, while eccentrically loaded did not.

3. Eccentrically loaded plates rotated under loading demanding larger plastic strains on the near shear leg, and an imbalance of strains on the tension plane, warranting a reduction factor in accordance to today’s code.

4. Pretensioning caused increased elastic loading rate, and ductility, but lower displacements at both yield and failure. Maximum load, plastic region slope, and energy absorbed before fracture also tended to decrease. No effect was seen on the yield level.

5. Eccentric loading reduced elastic loading, yield load and ductility, while increasing the plastic loading rate as well as the displacements at yield and rupture. A reduction in ultimate load was also noted.

6. In the absence of friction, elastic loading rate and ductility were seen to decrease. Displacements at yield and rupture were also seen to increase, in addition to the energy absorbed. There was no effect on ultimate load.
6.3 Conclusions

The following conclusions were made based on the observations of the analytical and destructive tests:

1. With high strength steels becoming more prevalent in design, new equations which better predict block shear behavior should be used. Alternatively existing equations could be modified to better account for shear capacity.

2. Designers should be aware of the potential drop in capacity when pretensioned bolts are used due to induced triaxial stress.

3. The presence of non-uniform stress on the tensile planes of eccentrically loaded tension members requires a reduction factor, according to the AISC code. This is corroborated by a decrease in ultimate load. A reduction similar to $U_{bs}=0.5$ used for double-columned, coped beam connections should be used. The coefficient used on coped beams is likely excessive for this condition; a lesser reduction, such as the shear lag factor, is recommended.

4. The increase in energy absorption in connections with reduced friction could provide an easier method of making structures capable of withstanding blast and other dynamic loads. It is recommended that the effects of reduced friction in connections be studied more in order to gain a better understanding of the mechanics involved.
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Figure 4.18. CS Finite Element Shear Plane Maximum Shear Strains
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Figure 4.20. EP Destructive vs. FE Load-Displacement Relation
Figure 4.21. EP Finite Element Tension Plane Strains

Figure 4.22. EP Finite Element Shear Plane Maximum Shear Strains
Figure 4.23. EP Post-Failure Finite Element Damage

Figure 4.24. ES Destructive vs. FE Load-Displacement Relation
Figure 4.25. ES Finite Element Tension Plane Strains

Figure 4.26. ES Finite Element Shear Plane Maximum Shear Strains
Figure 4.27. ES Post-Failure Finite Element Damage
# Appendix B – Tables

Table 3.1. Strain Gauge Positions.

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<td>9 5/8</td>
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### Table 3.2. Frame Displacement Bilinear Curve Ductility Values

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### Table 3.3. Connection Displacement Bilinear Curve Ductility Values

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### Table 3.4. Effects of Pretension on Ductility

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<td>↓ -39.23%</td>
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148
Table 3.5. Pretension Changes in the Frame Values

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### Figure 3.7: Eccentricity Changes in Frame Values

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<td>222.1</td>
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<td>Yield Displacement (in)</td>
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<td>0.504</td>
<td>19.73%</td>
<td>0.381</td>
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<tr>
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<td>39.86%</td>
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<td>Maximum Load (kip)</td>
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<td>232.8</td>
<td>-4.94%</td>
<td>238.1</td>
</tr>
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<td>Failure Displacement (in)</td>
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<td>0.950</td>
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### Figure 3.8: Eccentricity Changes in Connection Values

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<th>Change</th>
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<td>216.6</td>
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<td><strong>Yield Displacement (in)</strong></td>
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<tr>
<td><strong>Slope of Plastic Region (kip/in)</strong></td>
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<td><strong>Maximum Load (kip)</strong></td>
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<td>232.8</td>
<td>-4.94%</td>
<td>238.1</td>
</tr>
<tr>
<td><strong>Failure Displacement (in)</strong></td>
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<td>0.769</td>
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<td><strong>Ductility (in/in)</strong></td>
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Table 3.9. Effects of Eccentricity on Ductility

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<th>Combined Effects of Eccentricity</th>
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<td>Slope of Elastic Region (E)</td>
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<td>↓ -9.64%</td>
<td>↓ -9.99%</td>
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<td>↑ 101.38%</td>
<td>↑ 244.40%</td>
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<td>Slope of Plastic Region (m)</td>
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<td>Maximum Load (Fu)</td>
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<td>↑ 1.50%</td>
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Table 3.10. Effects of Reduced Friction on Ductility

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<td>Yield Displacement (in)</td>
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<td>Slope of Plastic Region (kip/in)</td>
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Table 3.11: Friction Changes in Frame and Connection Values

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Table 4.1. Plastic Strain Model of the Test Plate
Appendix C – Sample Calculations

Block Shear Strength, AISC 13th:

\[ F_y = 62.4 \text{ ksi} \]
\[ F_u = 75 \text{ ksi} \]

gage distance, g = 2.5 in.
space, s = 2.5 in.
edge distance, e = 1.25 in.
hole diameter, d = 0.8125 in.
plate thickness, t = 0.25 in.

\[ \text{Ant} = (g-d)t = 0.421875 \text{ in}^2 \]
\[ \text{Agv} = 2(s+c)t = 1.875 \text{ in}^2 \]
\[ \text{Anv} = \text{Agv} - 3d^2t = 1.265625 \text{ in}^2 \]
\[ U_{bs} = 1 \]

\[ R_n = 0.60F_u\text{Anv} + U_{bs}F_u\text{Ant} \leq 0.60F_y\text{Agv} + U_{bs}F_u\text{Ant} \]  
(Eq. J4-5) (AISC, 2011)

\[ R_n = 88.6 \text{ kips} \leq 101.8 \text{ kips} \]
\[ R_n = 88.6 \text{ kips} \]

Block Shear Strength, AISC ASD 9\textsuperscript{th}:

\[ P_{bs} = (0.3\text{Anv} + 0.5\text{Ant})F_u \]  
(Eq. J4-1/J4-2) (AISC, 1989)

\[ P_{bs} = 44.3 \text{ kips} \]
Block Shear Strength, Cunningham et al.:

\[
\text{length of net shear plane, } l_n = \frac{A_n}{t} = 5.0625 \text{ in} \\
\text{length of net tension plane, } l_t = \frac{A_t}{t} = 1.6875 \text{ in} \\
\text{in-plane shear eccentricity, } e_{\text{es}} = 0 \text{ in} \\
\text{eccentric } e_{\text{ec}} = 1.5 \text{ in}
\]

\[P_{\text{pred}} = 0.55 \cdot A_n \cdot F_u + [1.55 (l_n / l_t)^{\cdot.25} \cdot 0.1 \cdot e_{\text{es}}] \cdot A_n \cdot F_y \quad \text{(Cunningham et al., 1995)}\]

- Centric: \[P_{\text{pred}} = 110.4 \text{ kips}\]
- Eccentric: \[P_{\text{pred}} = 94.5 \text{ kips}\]

Bolt Shear Strength, AISC 13th:

(based on all 4 bolts)

Threads were assumed not excluded from the shear plane

\[F_n = 54 \text{ ksi}\]
\[A_b (4 @ 0.75 \text{ in}) = 1.767 \text{ in}^2\]

\[R_n = F_n \cdot A_b \quad \text{(J3-1) (AISC, 2011)}\]
\[R_n = 95.4 \text{ kips} > 88.6 \text{ kips} \quad \text{OK}\]

Bolt Bearing Strength, AISC 13th:

- Clear distance, \(l_c\) = \[5.0625 \text{ in}\]
- Plate thickness, \(t\) = \[0.25 \text{ in}\]
- Bolt diameter, \(d\) = \[4 @ 0.75" = 3 \text{ in}\]

\[R_n = 1.5 \cdot l_c \cdot t \cdot F_u \leq 3.0 \cdot d \cdot t \cdot F_u \quad \text{(J3-6b) (AISC, 2011)}\]

\[R_n = 142.4 \text{ kips} \leq 168.8 \text{ kips}\]

\[R_n = 142.4 \text{ kips} > 88.6 \text{ kips} \quad \text{OK}\]