

Parametric Exploration of Block Shear Formulations

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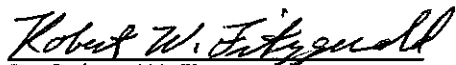
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Abstract

Block shear is a mode of failure in which a steel member fails in tension along one plane and shear on a perpendicular plane along the fasteners. The design process for block shear has been at the center of debate since it first appeared in the 1978 AISC *Specification* and has evolved over the years. This thesis investigated the block shear design equations as they have progressively changed from the 1978 provisions for Allowable Stress Design (ASD) to the 2005 provisions for Load and Resistance Factor Design (LRFD). Block shear strength capacities were calculated for multiple designs involving coped beams, angles, and structural tees. These analytical values were compared to physical test findings available in the literature. The results of this study compare the different strength predictions to one another, as well as benchmark the AISC provisions to actual physical testing strengths. The comparisons were also used to determine whether the ASD and LRFD specifications follow similar trends. Good agreement between the predicted capacities and the results from physical testing was observed for a majority of the geometries investigated. However, capacity predictions based on increasing the number of rows of bolts for a coped beam and changing the length of the outstanding leg for an angle or tee connection were found to disagree with the test results. A finite element study was also completed to further explore the influence of changing the length of the outstanding leg of tee connections because these geometries showed considerable disagreement between the calculated capacities and the test data.

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1. Introduction

Connections are a very important component of any structure. In order to design for them, specification equations are used to predict the capacity a connection can withstand. They must consider loads from the most extreme event possible for that particular location of the structure. The equations used must be constantly tested to find any flaws which will not predict capacities correctly. Once the equations can accurately predict capacity for all possible connection geometries, then they can be considered efficient and safe.

Block shear is one mode of failure for steel connections. This failure occurs when a bolted connection between two steel members fails by a combination of shear and tension along perpendicular planes. Over the years, new research has paved the way towards a better understanding of block shear behavior. As a result, the concepts and equations for block shear have been changing since they were first introduced in the 1978 *ASD Specification*. Even though many changes have already been made, some engineers believe additional changes are required because the current options are insufficient in accurately predicting block shear. This thesis focused on examining the level of accuracy of the block shear equations at every phase of their evolution.

Using previous research conducted by other engineers, concepts and geometries were selected for further studies, and these studies involved a two-step process. First, past work was used to compute the block shear capacities for each form of the block shear equations. Then the calculated capacities were compared to the physical testing completed in relative research. Comparison of these values allowed for a satisfactory assessment of the various equations. This comparison also confirmed which equation most accurately predicts block shear capacity for a given connection. In the completion of this evaluation, results revealed when the developed equations provided the most accurate estimates for block shear capacity, as well as whether or not further changes need to be implemented.

A finite element model was also created for the geometries of tees with changing web lengths. In research completed by Epstein and Stamberg (2002) it was found that the experimental trends did not correlate well with the equations. The model was used to investigate the distribution of the stresses in the connection and to explore the influence of changing the length of the outstanding leg.

This thesis begins with a background investigation regarding the origin of block shear and how the equations were introduced. Then, it covers the evolution of the block shear equations throughout the years. Next, research of previous work related to block shear failures in coped beams, angles, and tees is reviewed. This research was used to choose specific geometries for further exploration in this thesis. After the geometries are chosen, the method of work is covered. Lastly, the results and conclusions are discussed.

The results and conclusions found in this thesis suggested gaps and flaws in the block shear equations. With these revelations, future researchers can confirm the extent of the observed limitations and further refine the equations. If the specifications can accurately predict the capacities for all types of geometries, then newly designed structures will be safer as the chance of failure will be reduced.

2. Background

2.1. American Institute of Steel Construction

The American Institute of Steel Construction (AISC) is a not-for-profit technical institute that supports the steel design and construction community. They strive to make steel the construction material of choice by taking part in specification and code development, research, education, technical assistance, quality certification, standardization, and market development. AISC first published their steel construction manual in 1927 and has since been the guidelines for structural steel design. (AISC, 2005)

Until AISC introduced the Load and Resistance Factor Design (LRFD) *Specification* in 1986, the design of steel structures was based on Allowable Stress Design (ASD) techniques. ASD compared actual and allowable stresses, while LRFD compares the required strength to actual strengths. The difference between using strengths and stresses is not very significant. In fact, the newest specification uses Allowable Strength Design (ASD), which replaces the stress design. This effectively eliminated the major difference between ASD and LRFD. The current specification has a unified design method. It combines the previously separate ASD and LRFD into one specification, which makes it possible to design by either method.

2.2. Tearing Failures at Rivet Holes

Before block shear was considered in the specifications, bolted connections were rarely seen. In the 1950's, developments in welding and bolting were increasing in importance in steel connections, but riveted connections were still the most commonly employed method. "Riveting is a method of joining together pieces of metal by inserting ductile metal pins, called rivets, into holes of the pieces to be connected and forming a head at the end of each rivet to prevent the metal pieces from coming apart." (Bresler, Lin, 1966) While block shear was an unknown concept, steel designers were still aware of similar phenomena known as shear-out and tear-out. A shear-out failure can be seen in Figure 1.

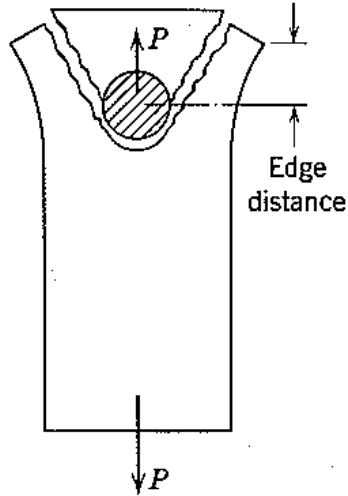


Figure 1: Shear-out failure (This figure was taken from Bresler & Lin, 1966)

This failure was not considered problematic as long as a sufficient edge distance was provided beyond the rivets. Therefore, the shear-out stresses were never calculated. To calculate a sufficient edge distance to prevent shear-out an equation was provided by the 1959 AISC *Specification* (Equations 1 & 2). These variables, e and a , are illustrated in Figure 2.

$$e = a + \frac{1}{2} * D * \cos(\theta)$$

Equation 1: Distance to center of hole

$$a = \frac{R}{2 * F_v * t}$$

Equation 2: Distance to edge of hole

- R=Rivet load on plate
- t=Plate thickness
- F_v =Allowable shear stress in the plate (13 ksi)
- $\theta=40^\circ$

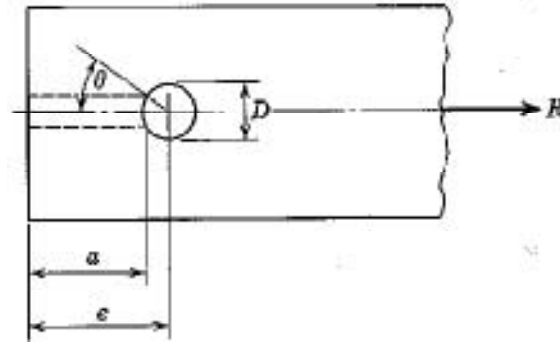


Figure 2: Edge distance to prevent shear-out failures (This figure was taken from Bresler & Lin, 1966)

Tear-out was the failure of the plate between rivets. It is more economical to space rivets as close as possible to allow for a more compact joint and minimize material. However, the spacing needs to be controlled so that the cross section of the plate is not greatly reduced. To prevent tear-out, an edge distance of more than two hole diameters was used. For spacing of the rivets, the specifications called for an optimum spacing; this is obtained when the tensile strength in the plates is equal to the combined rivet shearing or bearing strength. This optimum spacing was usually hard to calculate exactly, but if the design does not deviate from the optimum much, then it is still a good design. (Bresler, Lin, 1966)

These two failures, shear-out and tear-out, are very similar ideas to current design standards. They employ minimum edge distances and spacing to prevent failures like modern specifications. Both failures show that there was very little understanding of the future block shear phenomena.

2.3. Block Shear Failure at Bolt Holes

A block shear mode of failure occurs in steel members when there is a combination of shear yield or rupture through the line of bolts parallel to the applied load and tensile yield or rupture perpendicular to the load (see Figure 3). (Gross, et. al, 1995) Failure by tension rupture and shear yield is more common than shear rupture and tension yield due to the small amount of ductility in tension in comparison to shear. While block shear can be associated with welded joints, it is more commonly linked to bolted connections. (Kulak, Grondin, 2002) With the use of high-strength bolts becoming a more common occurrence in structural steel elements, there is a higher probability of block shear

failures. Due to the strength of these bolts, a connection can have fewer bolts and the area within the fasteners decreases. These factors result in a 'block' of material becoming either partially or completely removed from the rest of the element.

The block shear equations do not account for the fact that the net section cannot fully provide fracture strength when all of the elements of a tension member section are not attached to the connecting element. (Geschwindner, 2004) Shear lag accounts for these eccentricities in the connection as well as the length of the connection. (Munse, Chesson, 1963) Using an ANSYS finite element model, Figures 4a and 4b visually show shear lag. A 3"x12"x $\frac{1}{2}$ " grade 50 steel plate with two bolt holes was modeled in ANSYS. The bottom plane was assigned a frictionless surface and each bolt hole was constrained as a compression only support. Then a force of either 36 kips or 50 kips was applied to the end of the specimen without the bolt holes. Observing Figures 4a and 4b it can be seen that different elements are fully engaged at different times. This gradual development of the cross section describes shear lag.

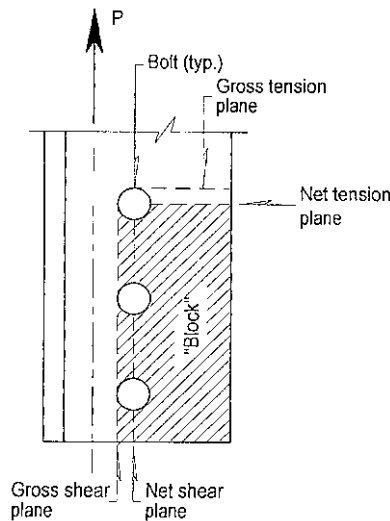


Figure 3: Typical block shear failure geometry (This figure is taken from Orbison, et. al, 1999)

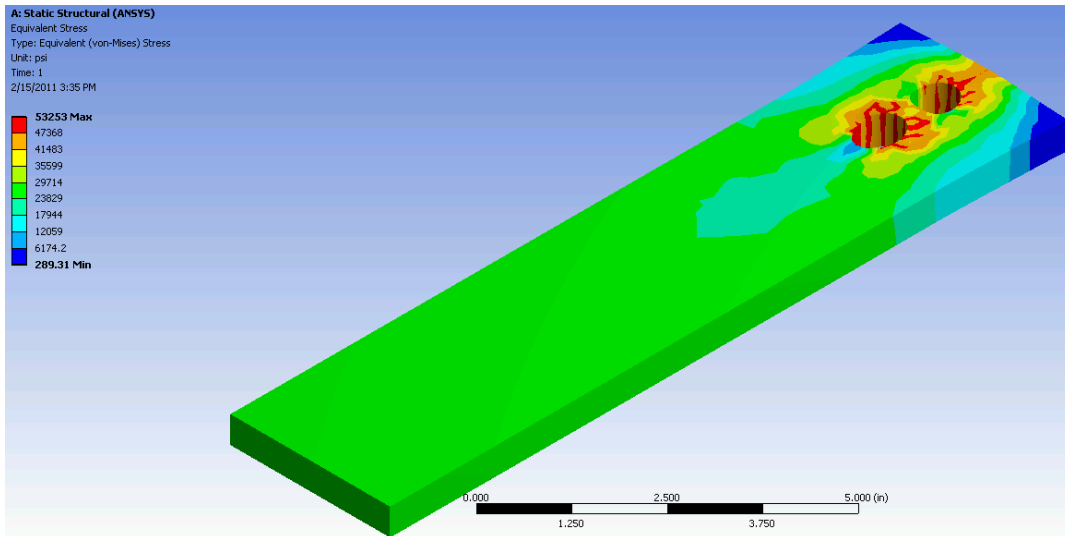


Figure 4a: Shear lag (36 kip force)

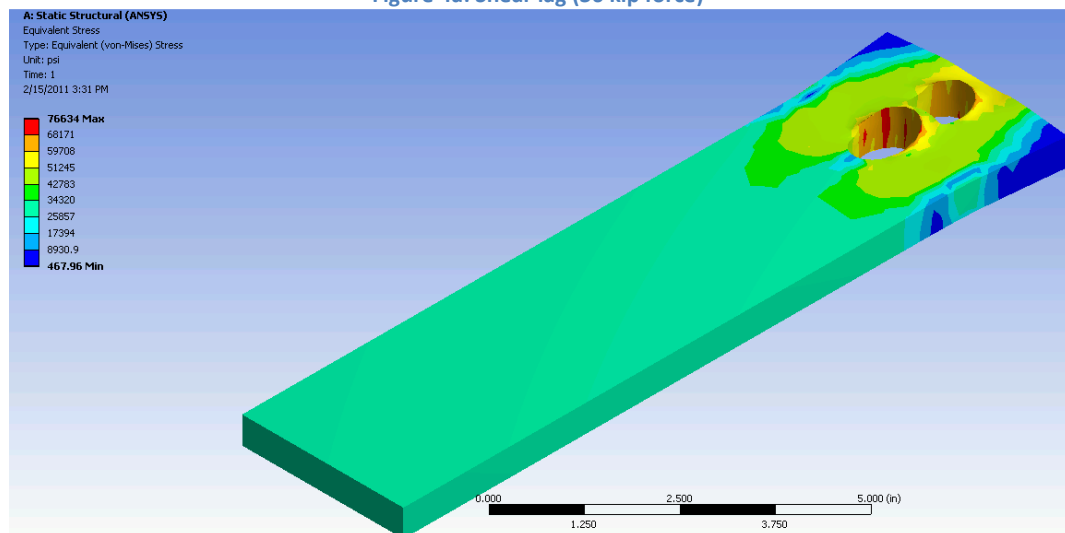


Figure 4b: Shear Lag (50 kip force)

It is common in construction to remove the top flange of a beam (cope) to make a beam to girder connection easier. Peter C. Birkemoe and Michael I. Gilmor first observed the block shear mode of failure in coped beams in 1978. The design standards of their time (CAN/CSA-S16-74) had significantly increased the allowable bolt bearing stress, which would allow for more failures by block shear. Birkemoe and Gilmor conducted tests for double angle shear connections on one coped and one uncoped beam. (Franchuk, et. al, 2002) Birkemoe and Gilmor used a universal loading machine with a special loading fixture to test their connections in shear. Two - 18 inch specimens were studied and both had the same connection geometry (see Figure 5). Their results revealed that the existence of the

cope in the beam reduced the strength by 24%. (Franchuk, et. al, 2002) Birkemoe and Gilmore (1978) recommended that an equation which combines tensile strength on one plane and shear strength on the perpendicular plane be incorporated into the design standards, but their paper did not develop nor present an actual equation.

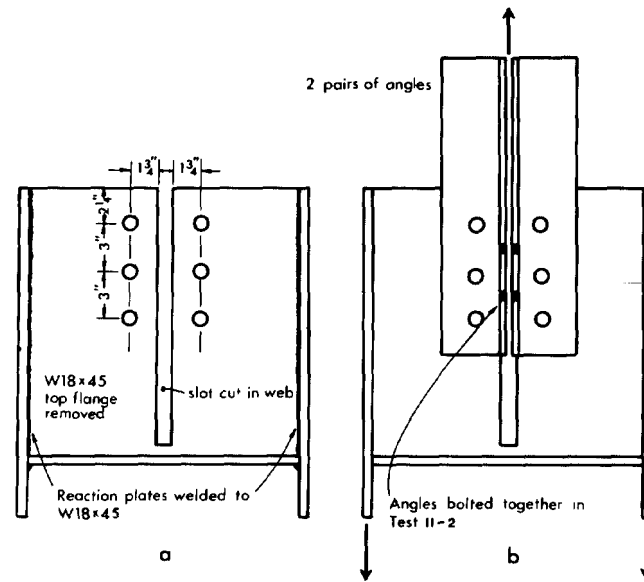


Figure 5: Birkemoe and Gilmore testing geometry (This figure was taken from Birkemoe & Gilmore, 1978)

2.4. Historical Background

Adjustments to the equations were made due to the increase in attention given to block shear as a result of specific tragedies over time. This is common for a number of changes in building standards. The progressive collapse of the Ronan Point in 1968 is one example of how tragedy leads to change. The Ronan Point was an apartment complex where an explosion on the eighteenth floor blew out one of the outer walls. This wall was the only support for the walls directly above and caused a partial collapse of the four floors above. The impact of these floors falling onto the eighteenth floor initiated the progressive failure of the floors below until the entire corner completely collapsed. The building had no fail-safe mechanisms, so the load had no alternative paths to follow, which ultimately led to the collapse. This tragedy brought about new design criteria in the United States as well as new guidelines for the Portland Cement Association (PCA) and Prestressed Concrete Institute (PCI). (Rouse, Delatte, 2003) For example, new codes now require minimum amounts of ductility and redundancy within

structures. Similar to this incident, the collapse of the Hartford Civic Center roof and World Trade Center Building 7 also caused the engineering world to react by investigating how block shear is addressed in design codes.

The Hartford Civic Center collapsed in 1978 due to design deficiencies. Subsequent investigation following the collapse revealed that there were at least three major design deficiencies and an underestimation of the dead load of 20%. The roof should have been designed for a dead load capacity of 140 psf, but on the day of the collapse there was only about 66 to 73 psf of snow accumulation. The three deficiencies are as followed: (Martin, 2010)

- The top layer's exterior compression members on the east and the west faces were overloaded by 852%.
- The top layer's exterior compression members on the north and the south faces were overloaded by 213%.
- The top layer's interior compression members in the east-west direction were overloaded by 72%.

After the collapse, an angle was found within the wreckage that had failed in block shear. (Epstein, Thacker, 1991) Before this event there was no consideration for block shear in tension connections. (Epstein, McGinnis, 2000) This discovery sparked an interest in research of block shear of angles and tees in tension.

Shortly after the event Epstein and Thacker created finite element models to observe block shear failure in angles. They studied the same geometry found in the Hartford Civic Center along with variations in stagger. Results showed that stagger may not always increase tension capacity as previous AISC specifications predicted. (Epstein, Thacker, 1991) Following these results, Epstein was interested in exploring other tension connections. Together, Epstein and McGinnis created finite element models for tees in block shear. These studies produced a good correlation between physical testing, computer models, and AISC specification equations. (Epstein, McGinnis, 2000)

The collapse of the World Trade Center Building 7 is another tragedy that triggered a need to investigate design codes. Building 7 survived the north tower of the Twin Towers collapsing in 2001.

After the north tower fell, Building 7 continued to burn for hours before it eventually collapsed. Fires caused thermal expansion of the long support beams which caused the beam-to-column connections to fail. Once one connection failed, all of the connections began to give way which generated a successive failure and total collapse. There were suspicions that block shear failure may have been the reason for the connection failure. This design of shear connections, in the case of a fire, raised concern and created the need to reevaluate the performance of tall structures with long spans within fire conditions. (Selamet, Garlock, 2010) Researchers used evidence from the collapse and full-scale fire tests to prove that connections are usually the weakest link. Connections are normally only required to transfer shear and moment, but in a fire condition they can be subjected to additional compressive or tensile forces. During fire conditions beams experience large deflections which cause conditions in which there is a high possibility for connection failure. Studies of the Building 7 connections found that at ambient temperatures the bolts remained intact and the beam web fractured in block shear, but at elevated temperatures the ultimate failure was by bolt shear fracture. The findings disclaimed the suspicions that block shear was the mode of failure. (Yu, et. al, 2007)

2.5. AISC Equations for Block Shear Capacity

The treatment of block shear has undergone many adjustments through the years, but the equations for strength remain very similar to the original version in the 1978 ASD document of record (see Table 1). (Epstein, D’Aiuto, 2002) The design specifications are intended to provide criteria that are as accurate as possible with the knowledge available at that time. The first concepts of block shear were based on experimental data from tests done on coped beams, and these provisions were introduced in the 1978 ASD *Specifications*. This document offered two possible equations, one in the specification and one in the commentary. The next version, 1989 ASD *Specifications*, brought one equation out of the commentary and into the specification. This specification also took into account that block shear can be considered for welded connections. In the 1986 LRFD *Specification*, block shear was taken as the larger

of two equations that were located in the commentary. Then, in 1993, the equations were brought into the specification and altered how the controlling equation was chosen. The larger of the shear and tension fracture values was combined with the opposite yield term, either shear or tension yield. For the 1999 LRFD *Specification*, the same equations were used, but an upper limit was placed on strength. This was similar to the equation used in 1978. The most current edition, 2005 ASD/LRFD *Specification*, uses tension fracture strength for all conditions. There is also a term added for the influence of non-uniform tension. Even though all these changes have been subtle, they show a growing understanding of the behavior of block shear. (Geschwindner, 2004)

The shear lag equation has also gone through a number of changes. Shear lag is the behavior of a tension member at a connection where not all of the cross-sectional elements contribute to the load path. The partial contribution causes the element within the load path to be overloaded and the unconnected cross-section is not fully stressed. Therefore, the entire member is not being used to its full capacity, and is accounted for by a reduction in the net area. The factor incorporates any eccentricities in the connection and the length of the connection. (Munse, Chesson, 1963)

Shear lag was first introduced along with block shear in the 1978 ASD *Specification* as 'C'. The 1989 ASD and 1986 LRFD codes changed the modifier, shear lag, to 'U'. Each specification gives a table with special cases and the value for shear lag that can be used. These tables are reasonable lower bounds, but Equation 3 shown below may be used in all cases. Munse and Chesson established this equation through their research. (Geschwindner, 2004) Then in the 1986 LRFD and 1989 ASD *Specifications*, provisions were made for shear lag in members connected by welds. In 1993, the *Specification* added an upper limit of $U=0.9$ for shear lag. This limit was removed in the 2005 *Specification* but added that for single angles, double angles, and WTs there must be a shear lag value of at least 0.6. Using this shear lag value helps to better predict block shear. (Geschwindner, 2004)

$$\text{Shear Lag} = C_t = U = 1 - \bar{x}/l$$

Equation 3: Shear Lag

Table 1: Block Shear Equations

SPECIFICATION	EQUATION
1978 ASD	$V_a = 0.3F_u(A_{nv} + A_{nt})$ OR $V_a = 0.3F_u A_{nv} + 0.5F_u A_{nt}$
1989 ASD	$V_a = 0.3F_u(A_{nv} + A_{nt})$ OR $V_a = 0.3F_u A_{nv} + 0.5F_u A_{nt}$
1986 LRFD	$\phi R_n = \phi[0.6F_y A_{gv} + F_u A_{nt}]$ $\phi R_n = \phi[0.6F_u A_{nv} + F_y A_{gt}]$ $\phi = 0.75$
1993 LRFD	When $F_u A_{nt} \geq 0.6F_u A_{nv}$: $\phi R_n = \phi[0.6F_y A_{gv} + F_u A_{nt}]$ When $0.6F_u A_{nv} \geq F_u A_{nt}$: $\phi R_n = \phi[0.6F_u A_{nv} + F_y A_{gt}]$ $\phi = 0.75$
1999 LRFD	When $F_u A_{nt} \geq 0.6F_u A_{nv}$: $\phi R_n = \phi[0.6F_y A_{gv} + F_u A_{nt}] \leq$ $\phi[0.6F_u A_{nv} + F_u A_{nt}]$ When $0.6F_u A_{nv} \geq F_u A_{nt}$: $\phi R_n = \phi[0.6F_u A_{nv} + F_y A_{gt}] \leq$ $\phi[0.6F_u A_{nv} + F_u A_{nt}]$ $\phi = 0.75$
2005 LRFD/ASD	$\phi R_n = \phi[0.6F_y A_{gv} + U_{bs} F_u A_{nt}]$ $\phi R_n = \phi[0.6F_u A_{nv} + U_{bs} F_u A_{nt}]$ $\phi = 0.75$

2.6. Identification of Experimental Data

In order to define a scope for this thesis, research was done to find specimens that commonly fail in block shear. By viewing past research and experiments, connection geometries were chosen. The research found some of the more critical connections that would be beneficial for further study.

2.6.1. Coped Beams

Some specimens that are known to fail in block shear are coped beams, angles, and structural tees. These are the three types of members that were explored in this thesis. Coped beams are wide flanges with the top flange removed at the ends of the beam to make connections easier. This practice increases the vulnerability of the beam to a block shear failure. It was believed that the equations given

by specifications in the past did not accurately predict block shear failure for coped beams. (Franchuk, et. al, 2004) Finite element models created by Ricles and Yura (1983) showed that coped beams in tension had a non-uniform tensile stress distribution. The current edition of the specifications presented in AISC 2005 *Steel Manual*, added a term to the equations which incorporates the influence of the non-uniform tension. (Geschwinder, 2004) It is now known that the beams fail by fracture in the tension zone first and a gross yielding along the vertical shear plane. Typically there are no signs of any fracture along the shear plane. (Ricles, Yura, 1983)

The variable in Franchuk, Cameron, Driver, and Grondin's (2003) testing which exhibits the most fluctuation in capacity is the gross shear area of a connection. In order to represent this, two coped beam geometries were examined, each with a different number of bolts which will increase the length of connection (shear area). It was found that the equations give a good prediction of the block shear capacity with changing shear area. This cannot be said for coped beams with two-line connections. When using the block shear equations presented in the AISC 1999 *Steel Manual* equations, it was found that results were exceedingly unconservative. To characterize this trend, a comparison of two coped beam geometries with one and two lines of bolts was researched. (Franchuk, et. al, 2003)

2.6.2. Angles

Block shear can be a potential failure mode in angle connections, especially when the connection is short. (Kulak, Grondin, 2002) Generally angles failing in block shear have a failure mechanism of rupture in the tension plane and yielding in the shear plane. (Gross, et. al, 1995) Some elements that have been observed to have an effect on block shear in angles are out-of-plane eccentricity, grade of steel, length of connection, and edge distance. The significance of out-of-plane eccentricity is not agreed upon by all. Gross, Orbison, and Ziemian (1995) observed out-of-plane bending in all of their tests on angles and found, that as the outstanding leg increased in length, the maximum load capacity decreased. To explore this phenomenon, they used angles with two different lengths of the outstanding leg to observe

the effect of out-of-plane eccentricity. However, Orbison, Wagner, and Fritz (1999) stated that out-of-plane eccentricity does not appear to reduce the load capacity significantly. Both of these groups of researchers found that the equations and testing show a decrease in failure load as the length of the outstanding leg is increased. Although Epstein (1992) noted that block shear equations predicted an increased failure load with an increased length of the outstanding leg, his testing found that the failure load decreased.

Edge distance and connection length can also be significant factors in influencing the block shear capacity in angles. To study changes in edge distance, three angles with varying edge distances were studied by Orbison, Wagner, and Fritz (1999). For altering connection lengths two sets of angles had been investigated, one with two bolts and the other with three bolts, increasing the length of the connection. (Orbison, et. al, 1999) Epstein (1992) also found the equations and the experiments agreed that an increase in the length of connection increases the block shear strength. Gross, Orbison, and Ziemian (1995) investigated three different block shear equations for angles and concluded that the ASD equations were the most accurate for their specimens. They also found that the 1993 LRFD *Specification* was accurate, but the governing failure mechanism did not agree with the one observed experimentally. It was also discovered that the 1986 LRFD *Specification* significantly overestimated the failure loads of angles made of grade 50 steel. To investigate this phenomenon, two sets of two angles were tested with the same geometries but two angles were A36 steel and the other two were grade 50 steel. All tests were limited by a ductile rupture of the tension plane preceded by significant necking. When the tension plane ruptured, there was yielding of the shear plane. (Gross, et. al, 1995)

2.6.3. Tees

Orbison, Wagner, and Fritz (1999) also performed experiments on structural tee sections. Through their work, it was concluded that edge distance and connection length can be significant influences on block shear capacity. They found that by increasing either edge distance or connection length the load

capacity increases as well. To represent the effect of connection length, two sets of three WTs had the same geometry but the numbers of bolts were increased from two to four. For changes in edge distances, three sets of three WTs were investigated with the edge distances differing throughout each set. In their testing they observed the tees developing in and out-of-plane flexural deformation. (Orbison, et. al, 1999)

Epstein and Stamberg (2002) varied the eccentricities of the tees by changing the lengths of the unbolted leg. They found that as the eccentricities increased, the efficiency of the connection decreased. Their experiments revealed that the tees failing in block shear were failing along an alternate block shear path through the front bolts and up through the web. It was also found that the sections with the largest eccentricities were subject to local buckling failures before the block shear failure load was reached. This load buckling suggested that for tensile loading there should be a limit on depth-to-width ratio for the web of a tee. (Epstein, Stamberg, 2002)

2.7. Research Objectives

Using the geometries from these past experiments, the block shear design equations were investigated as they have changed over the years. From the allowable stress design to the current load and resistance factor design specifications there have been five notable equation changes. Each equation was used for all the geometries and compared to the physical testing from the past work. The resistance factor (ϕ) provided in the LRFD codes were not applied to the equations in order to compare calculated capacities to the ultimate loads found from the testing. For the ASD equations a factor of safety value equal to two was built into the equation, so all the capacities determined from ASD provisions were multiplied by two to determine the ultimate capacity. (Ricles, Yura, 1983) Using the results from the equations and comparing them to the experimental data allowed for a meaningful comparison. It showed how the equations compare and how closely those results correlate to the

physical testing strengths. The results illustrate which equations are conservative and which are the most accurate. All these objectives were achieved by observing angles, tees, and coped beams.

3. Methodology

This thesis investigated the ways in which the block shear design equations have changed over the years. The hope was that after the work was completed the information could be used to further refine the block shear equations to better predict capacity for all possible scenarios.

3.1. AISC Block Shear Criteria

There have been five key revisions to the AISC provisions for block shear since the equations were formally introduced in the 1978 *Specification*. This work focused on the versions of the equations that were published in 1978 ASD, 1986 LRFD, 1993 LRFD, 1999 LRFD, and 2005 LRFD/ASD steel design manuals. These equations can be seen in Table 1.

Table 2: Summary of Geometries

Geometry	Constants	Variables
Coped Beams	W16x31 F _y =53 ksi Bolt Diameter= $\frac{3}{4}$ "	Length of Connection Number of Lines in Connection
Angles	Lx6x4x $\frac{5}{16}$ A36 Steel Bolt Diameter=1"	Edge Distance
Angles	Uniform Leg Thickness= $\frac{1}{4}$ " Bolt Diameter= $\frac{3}{4}$ "	Outstanding Leg Length Grade of Steel Length of Connection
Tees	WT7x11 A36 Steel Bolt Diameter=1"	Edge Distance Length of Connection
Tees	Grade 50 Steel Bolt Diameter= $\frac{3}{4}$ "	Outstanding Leg Length

3.2. Connection Geometries

The block shear equations were used to check the strength of multiple geometries for coped beams, angles, and tees. A summary of all the geometries used in this thesis are shown in Table 2. Geometries were chosen based on past experimentation and published findings.

3.2.1. Coped Beams

For coped beams, Franchuck, Driver, and Grondin (2003) found that there are two critical elements to be explored. The first element is changes in shear areas. To replicate this, the two geometries shown in Figure 6a were used. The second issue is the change from one to two line connections seen in Figure 6b. All coped beam geometries used W16x31 beams with yield strengths of 53 ksi and bolt diameters of $\frac{3}{4}$ inches.

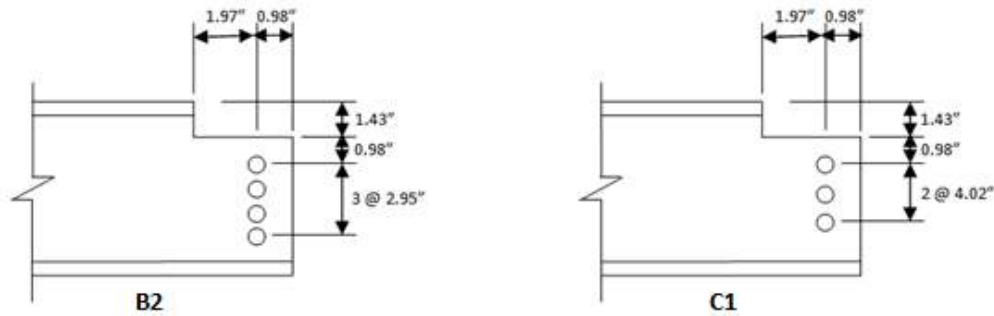


Figure 6a: Coped beams (changing connection lengths)

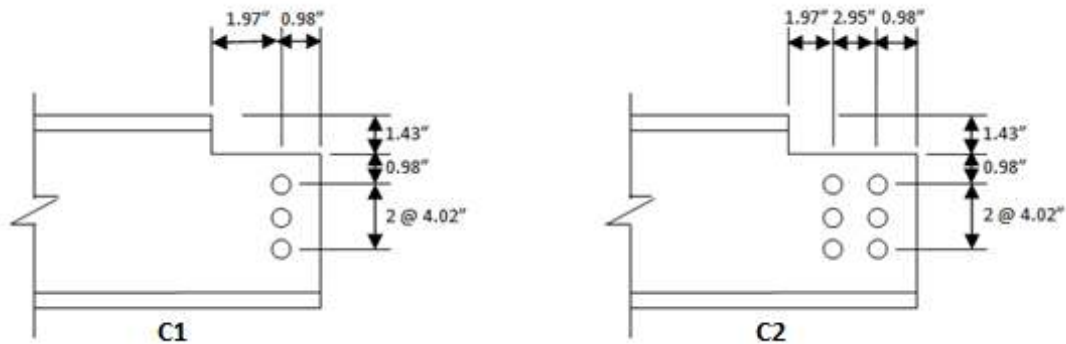


Figure 6b: Coped beams (one and two line connections)

3.2.2. Angles

Angles studied by Orbison, Wagner, and Fritz (1999) were found to have an increase in capacity as the edge distance was increased. The geometries used to portray this effect consisted of L6x4x $\frac{5}{16}$ angles, A36 steel, and 1-inch diameter bolts (Figure 7a).

Gross, Orbison, and Ziemian (1995) discovered that outstanding leg length, grade of steel, and length of connection can also all affect block shear load capacity. For changes in outstanding leg length

and grade of steel, two specimens of A36 steel and two specimens of grade 50 steel were investigated. The four examples shown in Figure 7b and Figure 7c provided comparisons for 3 inch and 4 inch leg lengths. Each used $\frac{3}{4}$ -inch diameter bolts and uniform leg thicknesses of $\frac{1}{4}$ inch. For studying changes in the length of connections, two angles with an outstanding leg length of three inches and two angles with an outstanding leg length of four inches were used (see Figures 7d & 7e). These connections also used $\frac{3}{4}$ -inch diameter bolts and had a uniform leg thickness of $\frac{1}{4}$ inch.

Last, a connection from the *1953 Specification* was used. By considering this connection, geometries used presently can be compared to historical geometries. In the *1953 Specification* several options for connections are illustrated with their capacities listed. The capacity depended on the size of the W-shape to which it was connected. This geometry used two $L4 \times 3\frac{1}{2} \times \frac{3}{8}$ angles, A36 steel, and $\frac{3}{4}$ " bolt diameters (see Figure 7f). For these specimens the capacities were compared to published capacity in the *1953 Specification* in lieu of test data. The published capacity for the 1953 angles was 21.6 kips.

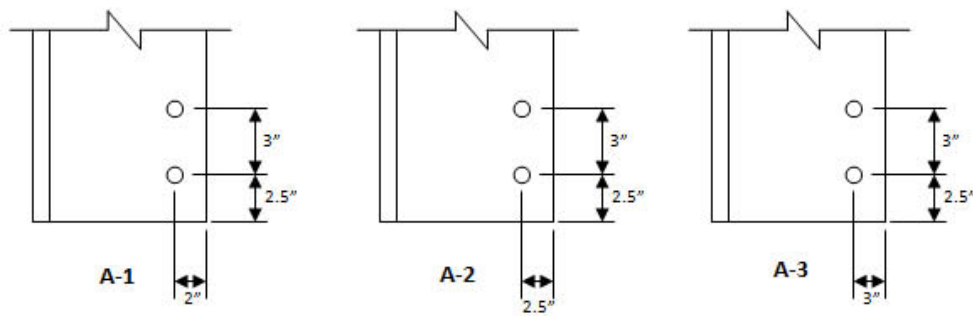


Figure 7a: Angles (changing edge distances)

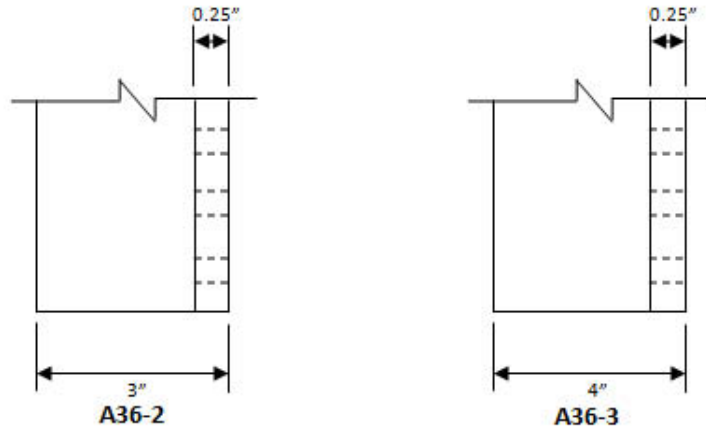


Figure 7b: Angles - A36 steel (changing outstanding leg length)

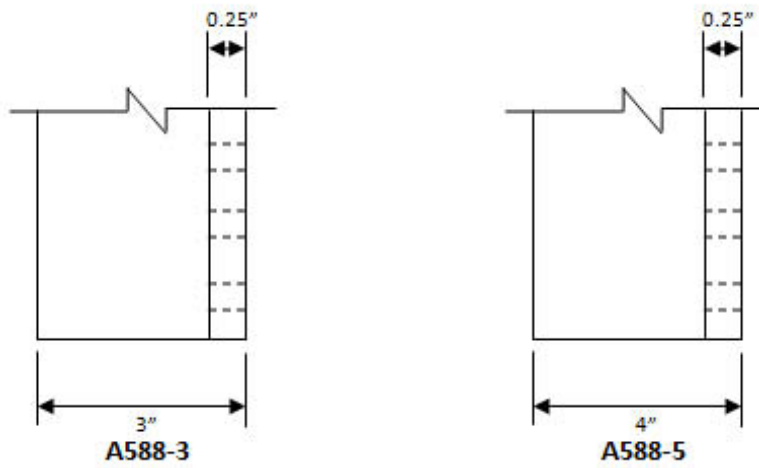


Figure 7c: Angles - Grade 50 steel (changing outstanding leg length)

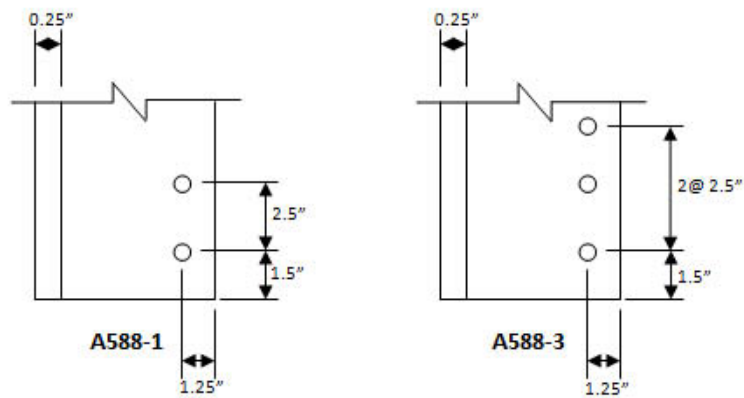


Figure 7d: Angles - Outstanding leg 3" (changing connection lengths)

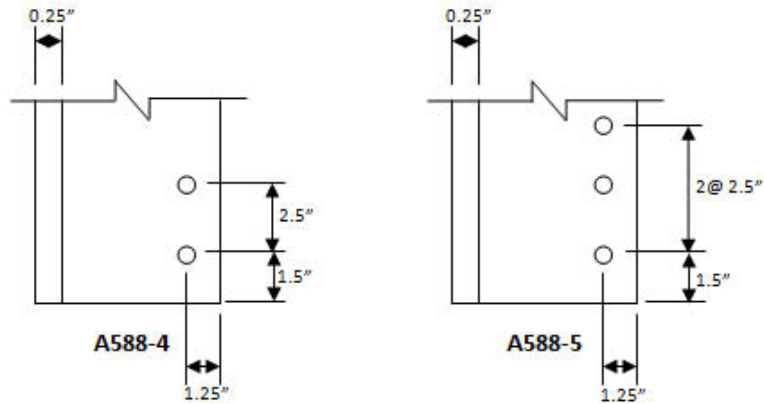


Figure 7e: Angles - Outstanding leg 4" (changing connection lengths)

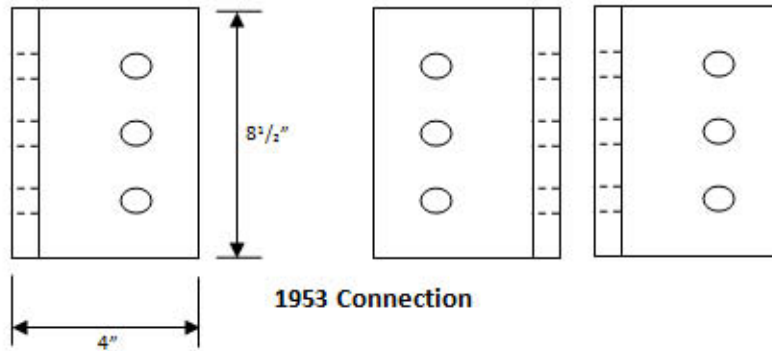


Figure 7f: Angles - 1953 Connection

3.2.3. Tees

Orbison, Wagner, and Fritz (1999) also studied structural tees for block shear. They found that edge distance and connection length are important variables for block shear strength. To model the changes in edge distances, three sets of WT structural tees with a varying number of bolts were considered with increasing edge distances (see Figures 8a, 8b, & 8c). For the lengths of connections, two sets of three tees were used. In each set the number of bolts increased from two to four (see Figures 8d & 8e). All of Orbison, Wagner, and Fritz's tee specimens used WT7x11 shapes, A36 steel, and 1-inch diameter bolts.

Epstein and Stamberg (2002) also did experimental work with structural tees. They studied the effect of changing the length of the unbolted leg of a tee in tension. This work looked at a group of three tees: WT6x8, WT6x8-1, and WT6x8-2. The designations -1 and -2 identify that the depth of the tee is 1 inch or 2 inches shorter than a standard WT6x8. The geometry of these specimens is shown in

Figure 8f. These specimens are all 53 inches long, used 3/4-inch diameter bolts, and were made of grade 50 steel.

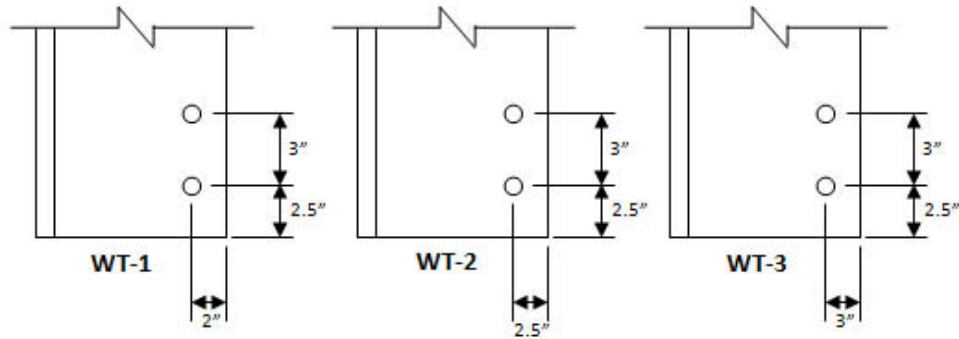


Figure 8a: Tees - 2 bolt connection (changing edge distances)

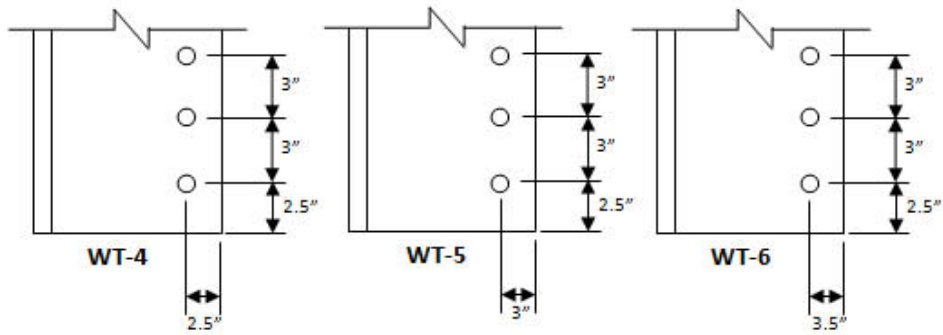


Figure 8b: Tees - 3 bolt connection (changing edge distances)

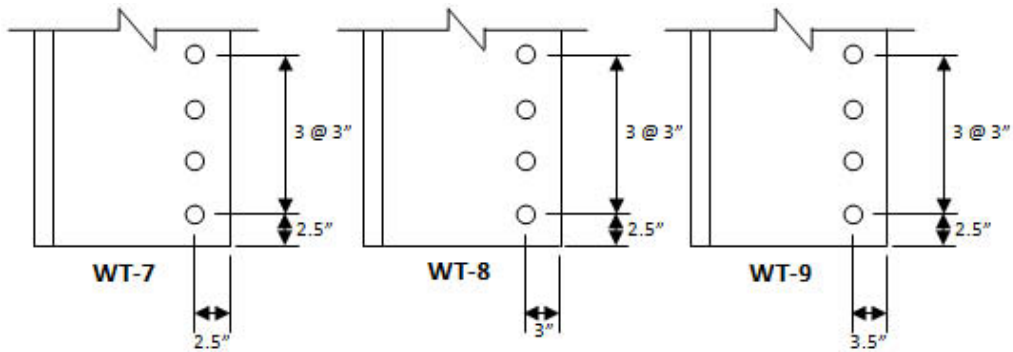


Figure 8c: Tees - 4 bolt connection (changing edge distances)

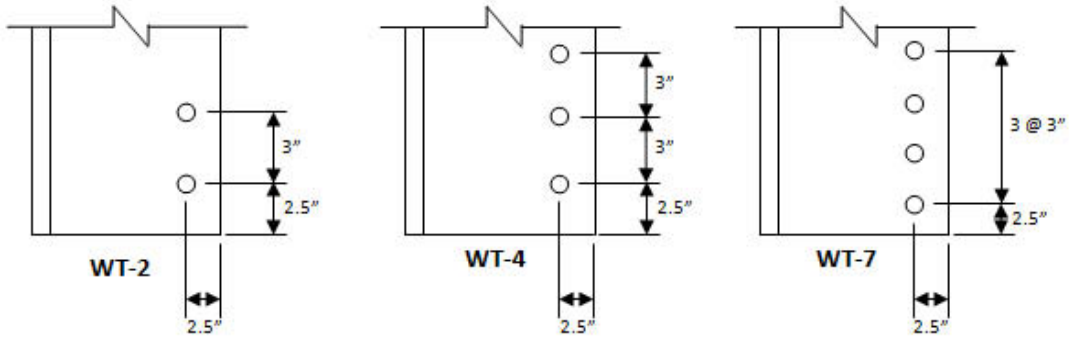


Figure 8d: Tees - 2.5" edge distance (changing connection lengths)

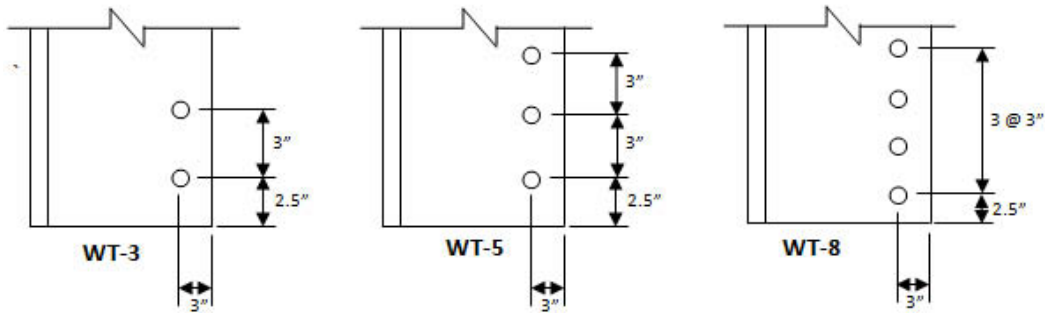


Figure 8e: Tees - 3" edge distance (changing connection lengths)

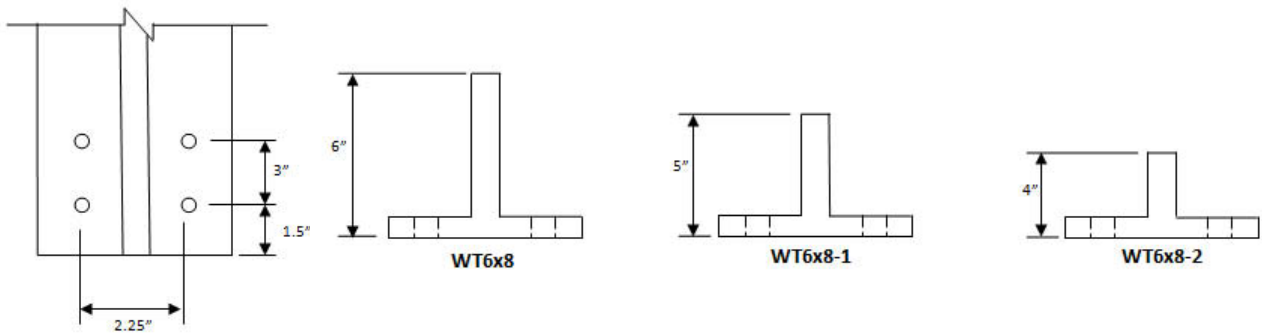


Figure 8f: Tees (changing length of outstanding legs)

3.3. Calculated Block Shear Capacities

Each of the connection geometries, shown in Figures 5a through 7f was analyzed using the set of six block shear equations presented in Table 1. Excel spreadsheets (see Appendices D through R) were utilized to calculate the block shear capacity of each individual specimen according to the different versions of the specification. A spreadsheet was developed for each form of the equation and, using the connection geometries, block shear capacities were determined. Equations 4 through 8 mathematically define the relevant tension and shear areas for input into the block shear equations.

$$A_{gt} = (\text{Edge Distance}) * (\text{Thickness of Bolted Element})$$

Equation 4: Gross Tension Area

$$A_{nt} = A_{gt} - \{(\text{Thickness of Bolted Element}) * \left[(\text{How Many Bolts in Tension}) * \left(\text{Bolt Diameter} + \frac{1}{8} \right) \right] \}$$

Equation 5: Net Tension Area

$$A_{gv} = (\text{Connection Length}) * (\text{Thickness of Bolted Element})$$

Equation 6: Gross Shear Area

$$A_{nv} = A_{gv} - \{(\text{Thickness of Bolted Element}) \left[(\text{How Many Bolts in Shear}) * \left(\text{Bolt Diameter} + \frac{1}{8} \right) \right] \}$$

Equation 7: Net Shear Area

$$A_e = A_{nt} * U$$

Equation 8: Effective Net Area

$$\bar{x} = \frac{\left(\frac{1}{2} * \text{thickness leg with bolts} * \text{area of leg} \right) + \left(\frac{1}{2} * \text{height leg without bolt} * \text{area of leg} \right)}{\text{Total area of specimen}}$$

Equation 9: Center of Gravity

In order to account for any eccentricities and connection lengths, shear lag was incorporated in the block shear calculations. The shear lag factor, 'U', modifies the net tension area (A_{nt}) into an effective net area (see Equation 8). Equation 1 shows the equation used for shear lag. ' \bar{x} ' is the distance from the face of a member to the center of gravity of the member and 'L' is the length of the connection. For most specimens, because they are defined in the *Steel Construction Manual*, \bar{x} was determined from a table. In all other cases, \bar{x} was manually calculated by assuming each cross section is comprised of simple rectangles and using Equation 9 to locate the centroid of the composite area.

When the focus of study is the unbolted leg, the block shear equations show no change in capacity, but once shear lag is introduced there is a change. This is because the equations for block

shear only focus on the connection and do not take any other portion of the specimen into consideration. Shear lag accounts for the eccentricities within the member and also takes the connection length into consideration. As the length of the connection is increased, shear lag increases the efficiency of the connection. (Munse, Chesson, 1963)

3.4. Comparisons of Calculated Capacities with Experimental Data

The capacity values obtained from the various AISC provisions were considered nominal strength values and were not adjusted for design practice. This means that for the LRFD equations, ' ϕ ' was not used, and for the ASD equations, the built-in factor of safety of two was multiplied out of the equation. (Ricles, Yura, 1983) Excluding resistance and safety factors allowed the findings to be compared to the ultimate loads from the physical testing done by previous engineers.

The calculated capacities and the published experimental results were easily compared using professional factors; this is the ratio of experimental failure load to predicted failure load. (Epstein, Stamberg, 2002) When this value is less than one, the load prediction is unconservative. Values greater than one are conservative predictions. The closer the value is to one, the better the equation predicted the failure load. (Gross, Orbison, and Ziemian, 1995) When using professional factors it is expected that the results should show a sensible amount of scatter. Graphs, tables, and charts were produced from these values in order to visually show any patterns or trends observed from the work.

3.5. Finite Element Analysis of Tees

To provide another visual, a finite element model was built using *ANSYS Workbench 2.0* (2009). The objective was to gain a better understanding of the stresses present in structural tees as the stem was shortened. Epstein and Stamberg (2002) found through their experiments that as the length of the unbolted leg increased, the capacity increased. However, using the equations the opposite trend is found. The equations show that as the length of the unbolted leg is increased the capacity should decrease because the centroid distance, \bar{x} , increased and shear lag, U , value decreased (see Equation 3),

which caused the net tension area to decrease. When these values decrease, the block shear capacity decreases as well. Creating a computer model of this phenomenon allowed for a better look into what is actually happening within the connection. It was expected that the ability to visualize these stresses would help to comprehend the apparent divergence between the trends of experimentally derived and calculated capacities.

4. Results

The background research established a baseline understanding of what is expected from the AISC equations for block shear capacity. Development of the Excel spreadsheets offered a deeper study of each individual equation and its components. The multiple geometry changes provided insight into the application of each equation to compute capacity. Last, reflection on the background research, spreadsheet calculations, and published experimental data led to a finite element model to investigate the effects of changing the length of an unbolted leg in a structural tee geometry.

4.1. Observed Similarities between Calculated and Experimental Results

For the several geometry changes tested, the majority showed a good correlation between the equations and experimental testing. This means that the block shear equations are correctly predicting the capacities of the geometries.

4.1.1. Increasing Edge Distance

The first geometry observed was increasing the edge distance of the connection. This was studied for angles and tees. For both sets of specimens, as the edge distance was increased, the block shear capacity was increased for both the equations and the experiments. Typical comparisons for angles and tees can be seen in Figures 9 and 10, respectively. More figures showing the capacities for tees with changing edge distances can be seen in Appendix B. The capacities increase because, as the edge distance is increased, the gross tension area (A_{gt}), net tension area (A_{nt}), and effective areas (A_e) increase as well. As these area values increase so does the computed block shear capacity.

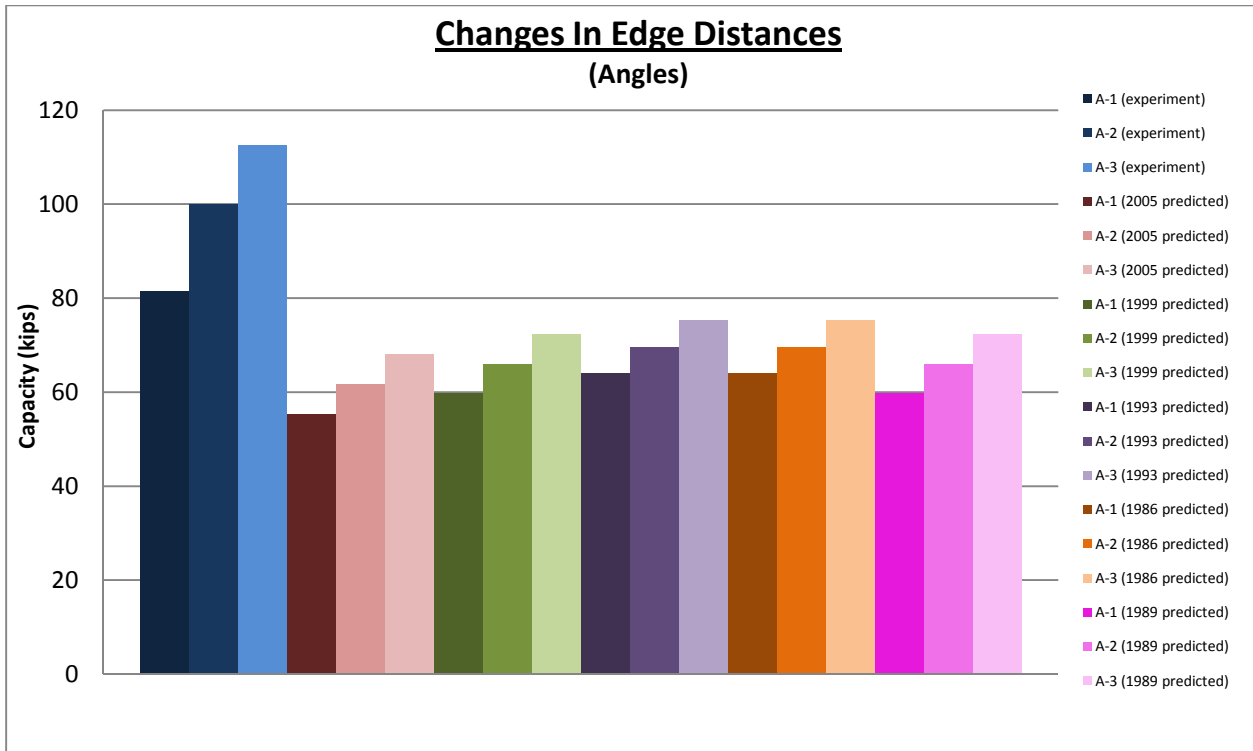


Figure 9: Capacities of angles with a given size and changes in the edge distances

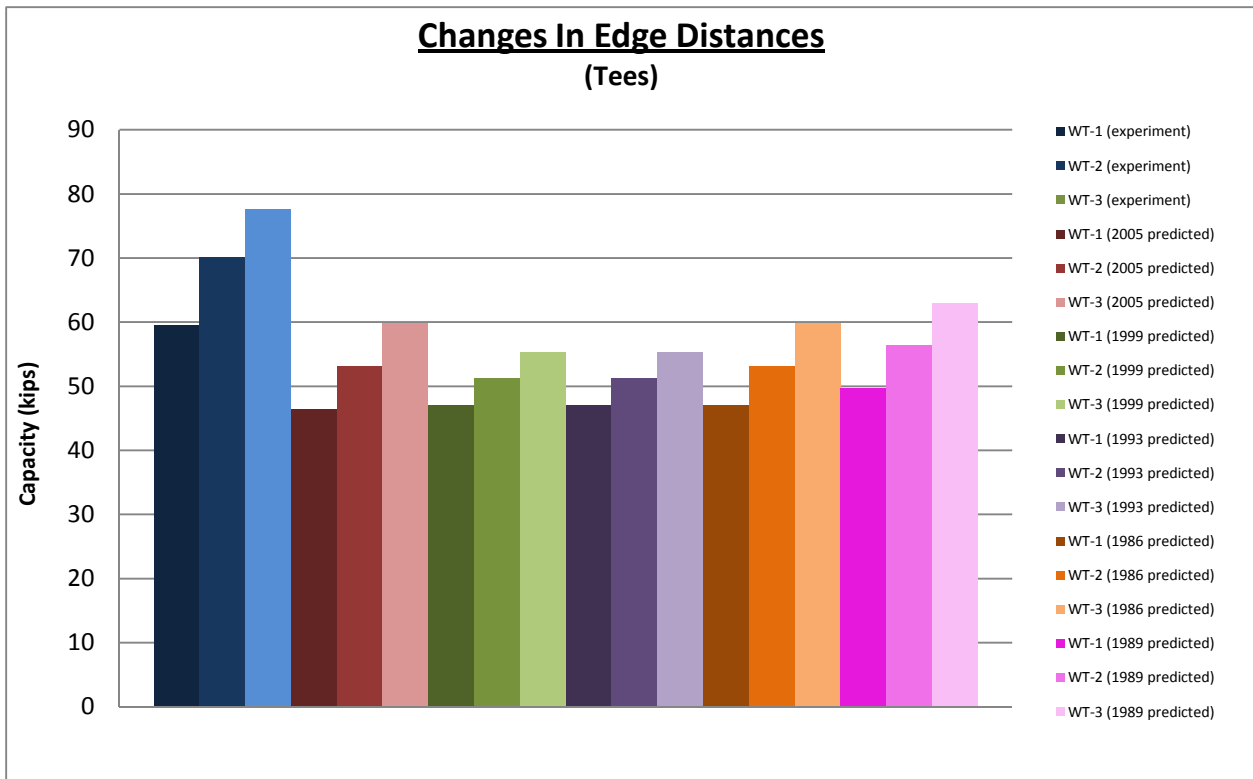


Figure 10: Capacities of tees with a given size and changes in the edge distances

4.1.2. Increasing Length of Connection

Another geometry tested was increasing the length of the connection by adding additional fasteners. This condition was studied in angles, coped beams, and tees. For all three types of elements, the capacity increased for both the equations and experiments when the length of connection was increased. Comparisons of calculated and experimental results can be seen in Figures 11, 12, and 13. Additional figures showing these comparisons can be seen in Appendices A and B. The capacities increase because, as the length of connection is increased, the gross shear area (A_{gv}) and net shear area (A_{nv}) increase as well, and the block shear capacity increases with these increased areas.

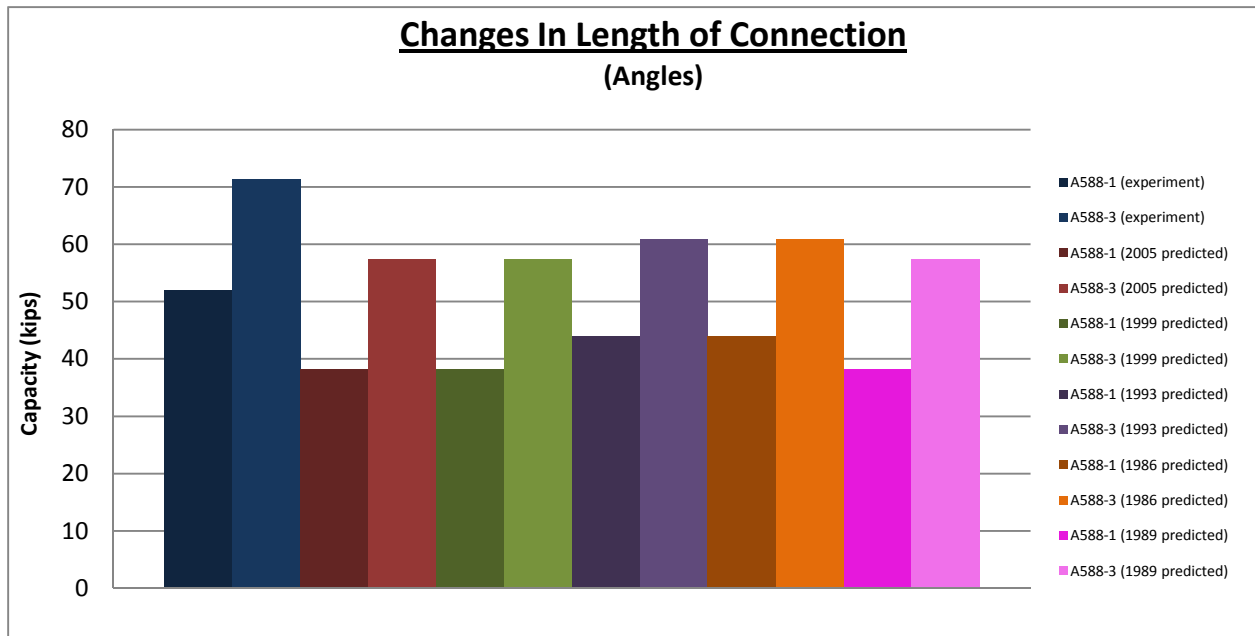


Figure 11: Capacities of angles with a given size and changes in the length of connection

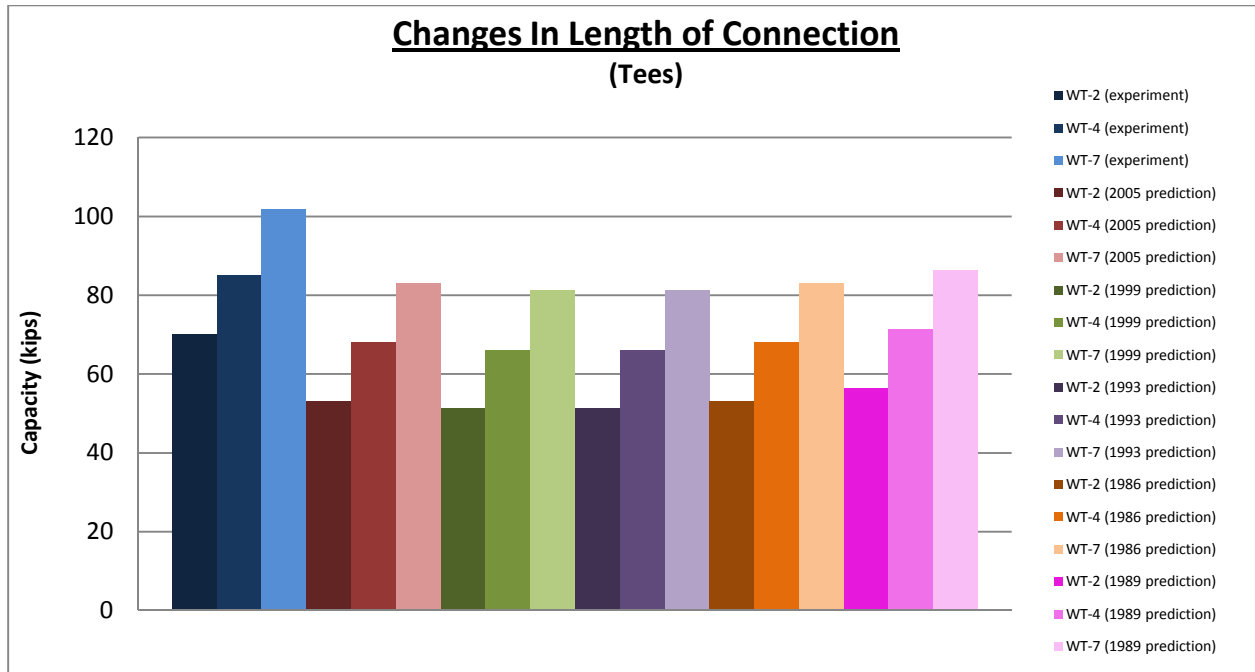


Figure 12: Capacities of tees with a given size and changes in the length of connection

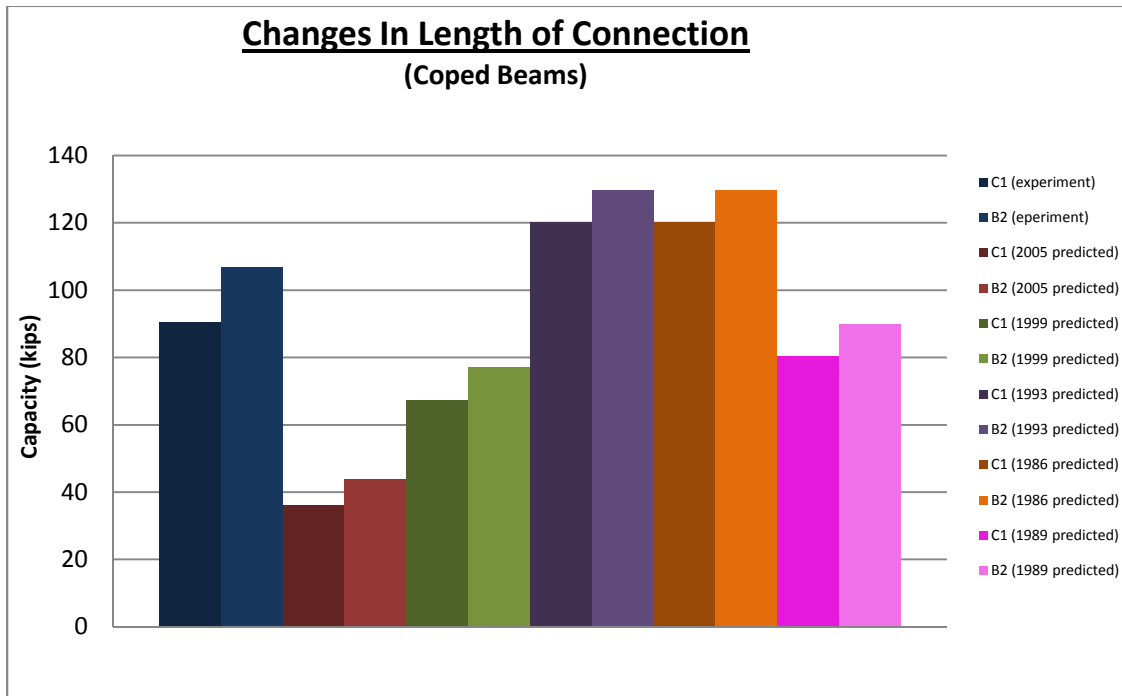


Figure 13: Capacities of coped beams with a given size and changes in the length of connection

4.1.3. Grade of Steel

The grade of steel was the last variable for which the calculated capacities and experimental data showed similar trends. The effect of change in steel from A36 to grade 50 was examined for angles. For both the AISC equations and experimental data, angles made of grade 50 steel had a higher capacity

than those produced from A36 steel. This relationship can be seen in Figure 14, and additional comparisons are provided in Appendix A. Since block shear is a tension or shear rupture failure, higher ultimate tensile stress (F_u) for grade 50 steel increases the capacity over that for A36 steel.

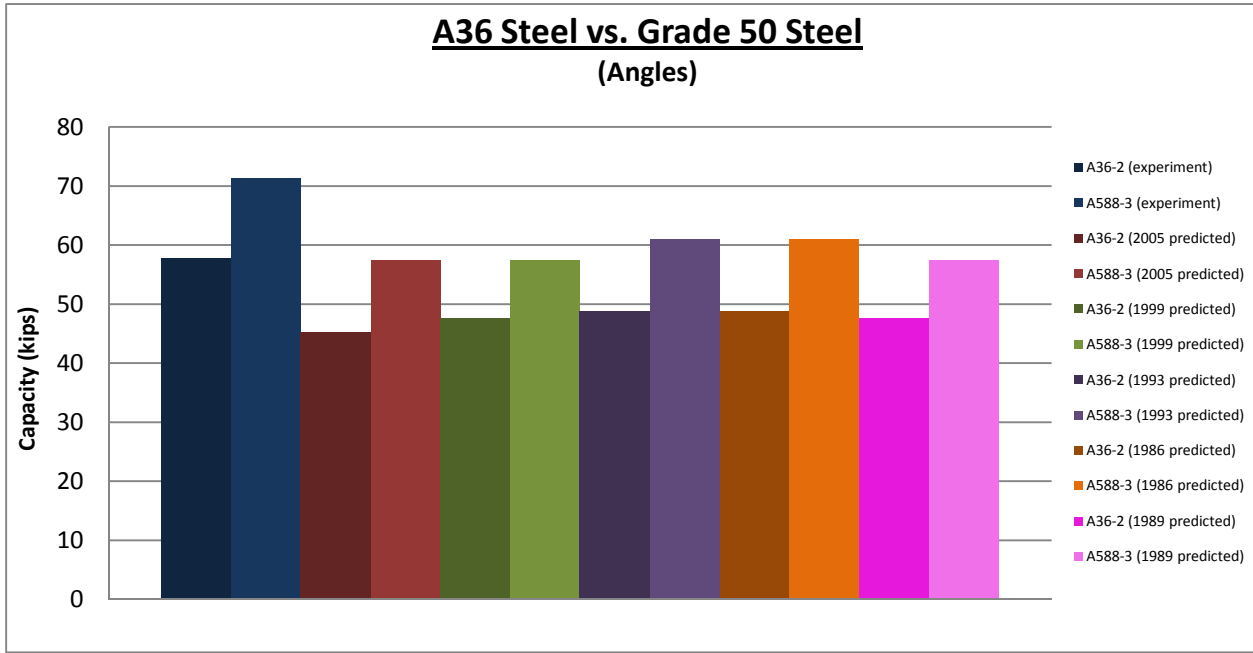


Figure 14: Capacities of angles with a given size and changes in the grade of steel

4.2. Observed Differences between Calculated and Experimental Results

4.2.1. Angles

When observing an angle's block shear capacity while changing the outstanding leg length, it was found that not all of the equations agree with the experimental trends. Figure 15 shows that the experimental data has a decrease in capacity with a decrease in the outstanding leg length, while not all the equations display this same trend (also see Appendix A). The experimental data indicated that block shear capacity decreased as the outstanding leg length was increased. The 2005, 1999, and ASD versions of the block shear equations agreed with this trend. Conversely, the 1993 and 1986 *Specifications* did not agree with this trend. In both these years, the predicted capacity remains the same despite changing the length of the angle's outstanding leg. Closer inspection revealed that the equation $R_n = 0.6 * F_u * A_{nv} + F_y * A_{gt}$ governed for all cases, and it does not account for shear lag. When there is no change in the connection geometry, the only mechanism to effect a change in the block shear equation is the shear lag modifier. As noted previously, shear lag takes into account the fact that all of the available cross-sectional area is not effective in resisting the applied load. As the length of the angle's outstanding leg is increased, the distance, \bar{x} , between the angle's center of gravity and the plane of the connection is increased according to Equation 9.

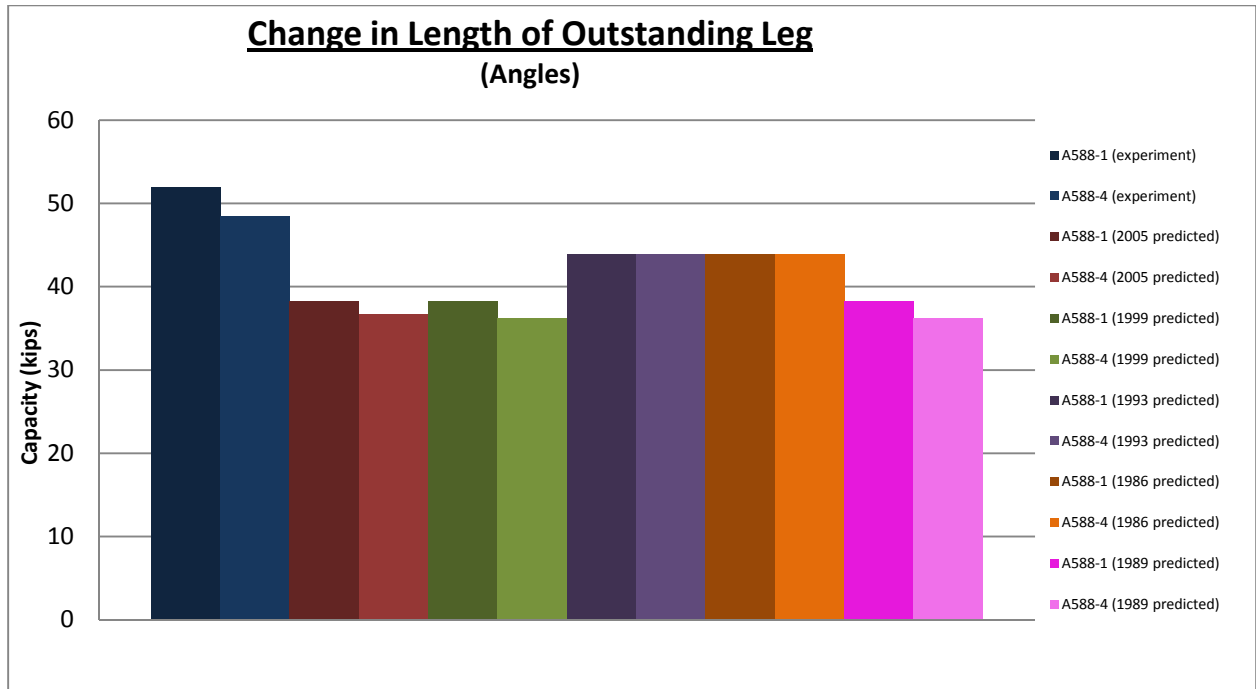


Figure 15: Capacities of angles with a given size and changes in the length of outstanding leg

4.2.2. Coped Beams

Coped beams were studied for one and two lines of bolts. The experimental data showed that the block shear capacity increased with an increase in the number of rows. Figure 16 illustrates that the experimental data only agrees with the 2005 *Specification* equations. In all the other years, the calculated capacity remained the same for one and two lines of bolts. The earlier AISC provisions only considered the shear and tension areas associated with the first line of bolts and no modifiers were provided to capture the shear lag introduced by additional rows of bolts. When the 2005 block shear equations were created, a new variable called U_{bs} was defined. U_{bs} is a reduction factor that approximates the non-uniform stress distribution in the tensile plane due to the presence of multiple rows of bolts. For coped beams with multiple rows of bolts, the tensile stress distribution is non-uniform because the row of bolts that is closest to the end of the beam carries the most shear load.

Figure 17 shows the different cases for the block shear tensile stress distribution and the corresponding U_{bs} values. (AISC, 2005)

- $U_{bs}=1$ when the tension stress is uniform (angles, gusset plates, and most coped beams)
- $U_{bs}=0.5$ when the tension stress is non-uniform (multiple-row coped beam connections)

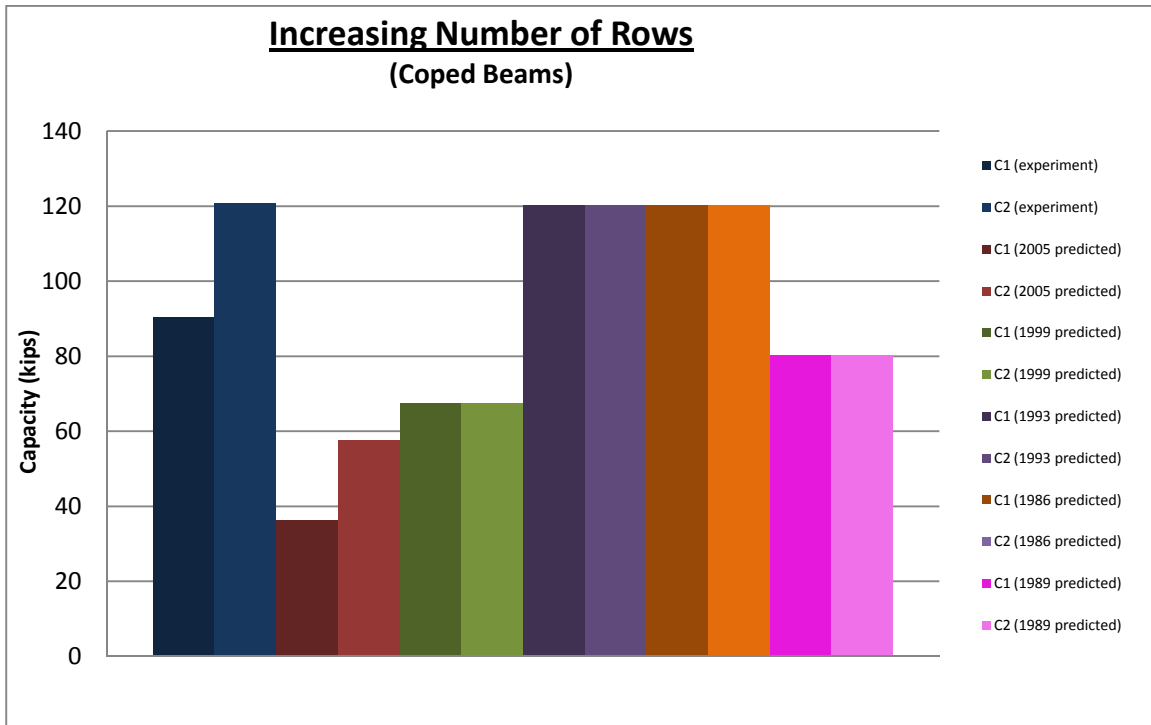


Figure 16: Capacities of coped beams with a given size and increasing the number of rows of bolts

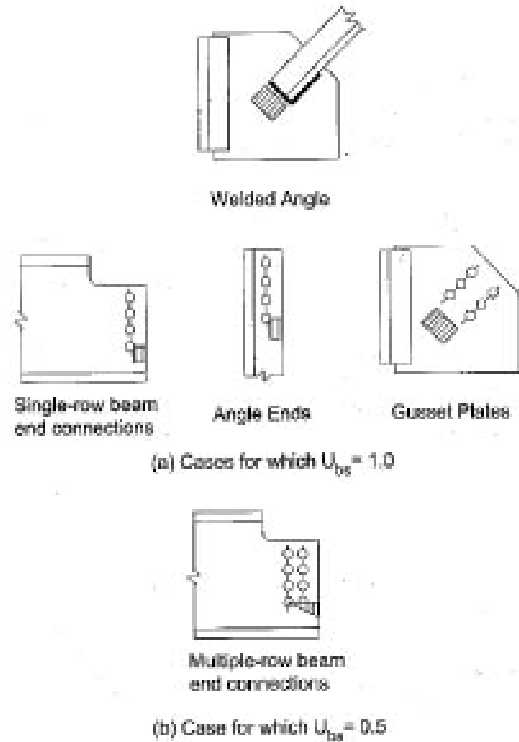


Figure 17: Block shear tensile stress distribution (This figure was taken from AISC, 2005)

4.2.3. Tees

Epstein and Stamberg (2002) had tested the block shear capacity of structural tees while changing the length of the unbolted leg or web. Their research showed that as the length of the unbolted leg increased, capacity decreased, but the AISC equations predicted an opposite effect. Figure 18 illustrates that this thesis followed a similar trend to the observations of Epstein and Stamberg. For the ASD and 1999 equations, the trend in predicted capacity is the complete opposite of the experimental data. In 1986 and 1993, the capacity remains the same for all three specimens because the equation $R_n = 0.6 * F_u * A_{nv} + F_y * A_{gt}$ governs and it does not consider shear lag. When the connection geometry has no change, shear lag is the only way for the block shear equation to account for a change. The changing element is the center of gravity, which is accounted for in the modifier for shear lag. Then in 2005, WT6x8 and WT6x8-1 have the same calculated capacity, while the equations predict an increased capacity for the WT6x8-2.

The capacity for the first two is the same because the connections and shear lag effects are similar to $U=0.6$ according to Equation 3. The 2005 provisions specify a maximum allowable shear lag factor of 0.6, so the analysis of both the WT6x8 and WT6x8-1 specimens default to this value.

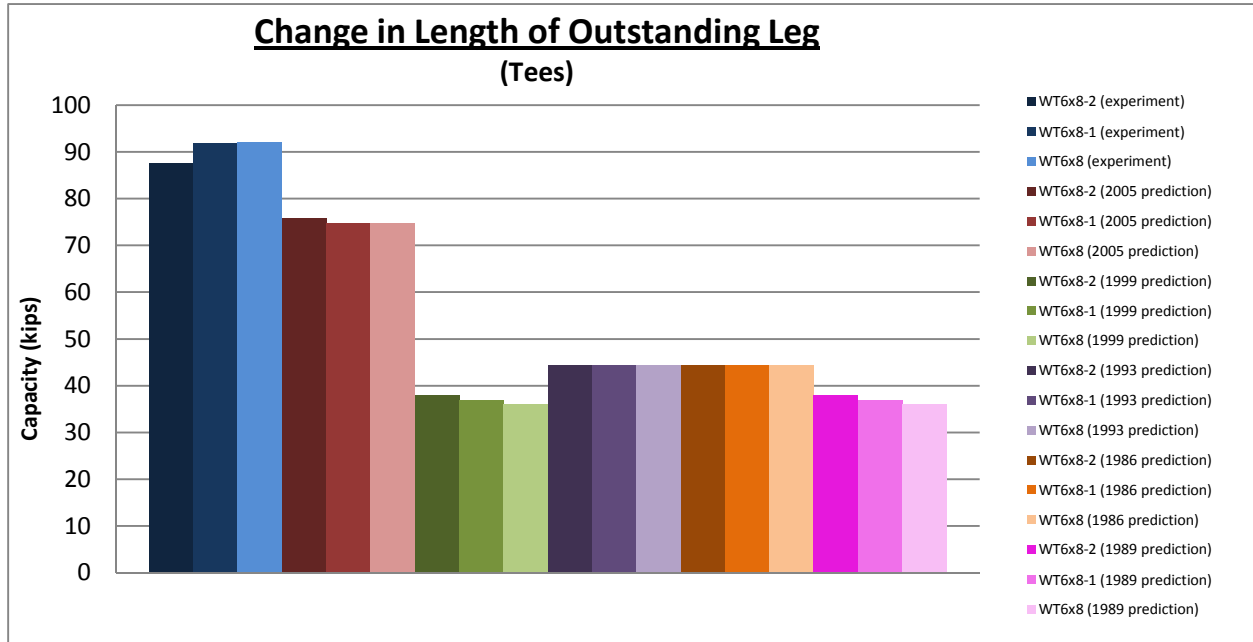


Figure 18: Capacities of tees with a given size and changes in the length of outstanding leg

4.3. Professional Factors

To better depict the relationship between the predicted capacities and the experimental capacities, professional factors were computed. Professional factors are the ratio of the experimental failure load to the predicted failure load. When this value is less than one, the load prediction is unconservative. Values greater than one are conservative predictions.

4.3.1. Professional Factor ≥ 1.0

For the majority of the cases the professional factor is greater than one, which means that the AISC equations provided a conservative prediction, i.e. the predicted capacity was smaller than that obtained from testing. These specimens showed professional factors ranging from 1 to 2.55. This is the expected scatter that is prevalent in most testing. The professional factors for all the specimens can be seen in Figures 19, 20, & 21. Table 3 also shows a summary of the professional factors and the coefficient of

variation. The higher the percentage of the coefficient of variation, the more likely it is that the equation will be inaccurate when predicting capacity.

The coped beam with two rows of bolts (C2) for the 1986 and 1993 equations have a professional factor of one. For both versions of the specification, the same equation governed and changing the number of rows created no difference in the predicted capacity. Specimen C1 was a coped beam with one row of three bolts, while C2 had two rows each of three bolts. Both specimens, C1 and C2, had the same predicted capacity, but a larger experimental capacity was indicated for geometry C2. A professional factor equal to one indicates that the 1986 and 1993 block shear provisions were quite accurate for this geometry. Application of the resistance factor $\phi=0.75$ would then provide a design margin for possible under-strength due to deviations in material strength and connection geometry.

The following comparisons showed a large professional factor ($PF>2$):

- All coped beams for the 2005 equations.
- WT6x8 and WT6x8-1 for the 1999, 1993, 1986, and ASD equations.
- WT6x8-2 for the 1999 and ASD equations.

These cases with a large professional factor demonstrate that the equations seem to be more conservative than usual. Large professional factors may indicate the equations are near the bounds of applicability. Another possible cause is over-strength of the test specimens due to higher material strengths than the minimum prescribed values of F_y and F_u . Conservative predictions are good for design because there is a smaller chance that the connection will be under-designed. Excessive conservatism is not desirable. It can cause higher prices because the connection is being over-designed and more steel than needed is being used.

Table 3: Summary of Professional Factors

Professional Factors					
Specimen	# of Cases	Max	Min	Mean	Coefficient of Variation (%)
A-1	5	1.47	1.27	1.350	6.20
A-2	5	1.26	1.44	1.508	4.93
A-3	5	1.65	1.49	1.550	4.25
A588-1	5	1.36	1.19	1.292	7.21
A588-3	5	1.24	1.17	1.212	3.16
A588-4	5	1.34	1.11	1.244	9.86
A588-5	5	1.19	1.10	1.154	4.27
A36-2	5	1.28	1.19	1.220	3.01
A36-3	5	1.18	1.07	1.112	4.09
1953 CONNECTION	5	0.13	0.12	0.122	3.66
B2	5	2.44	0.82	1.332	50.02
C1	5	2.50	0.75	1.294	55.66
C2	5	2.10	1.00	1.478	32.83
WT-1	5	1.28	1.20	1.252	2.42
WT-2	5	1.37	1.24	1.324	4.02
WT-3	5	1.40	1.23	1.326	55.32
WT-4	5	1.29	1.19	1.254	3.27
WT-5	5	1.37	1.23	1.310	4.58
WT-6	5	1.48	1.30	1.396	5.77
WT-7	5	1.25	1.18	1.228	2.33
WT-8	5	1.37	1.26	1.320	3.67
WT-9	5	1.45	1.30	1.380	4.86
WT6x8	5	2.55	1.23	2.094	25.76
WT6x8-1	5	2.48	1.23	2.062	24.75
WT6x8-2	5	2.31	1.16	1.944	24.18

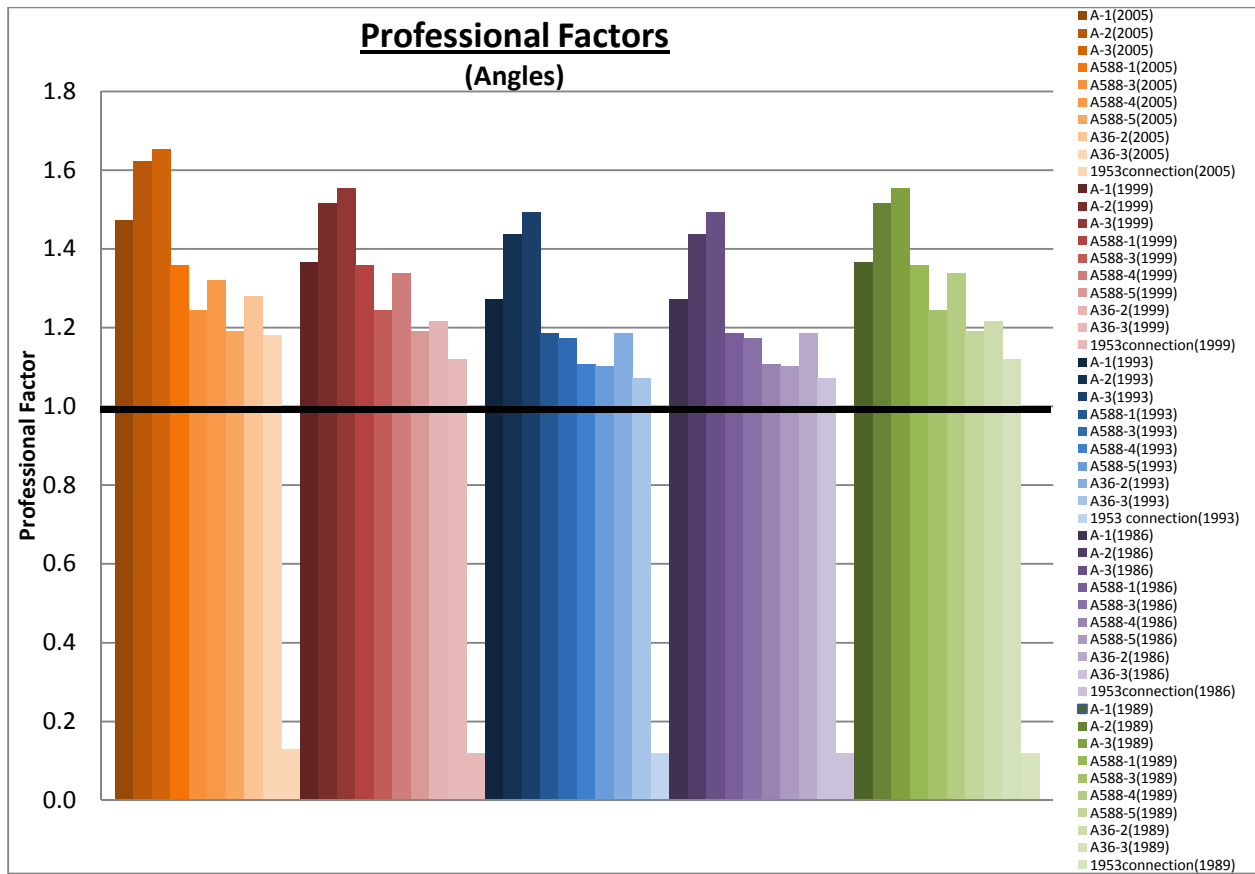


Figure 19: Professional factors of all angle specimens

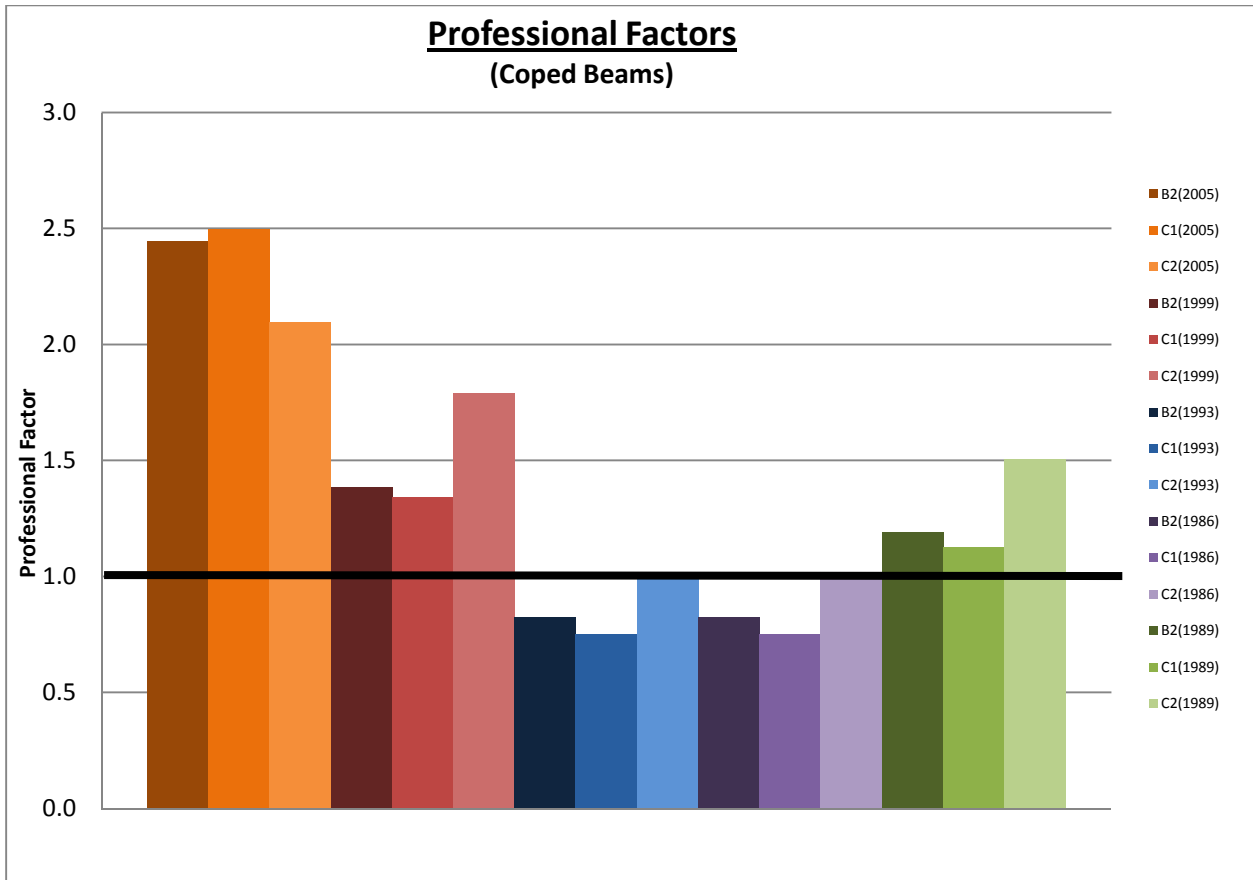


Figure 20: Professional factors of all coped beam specimens

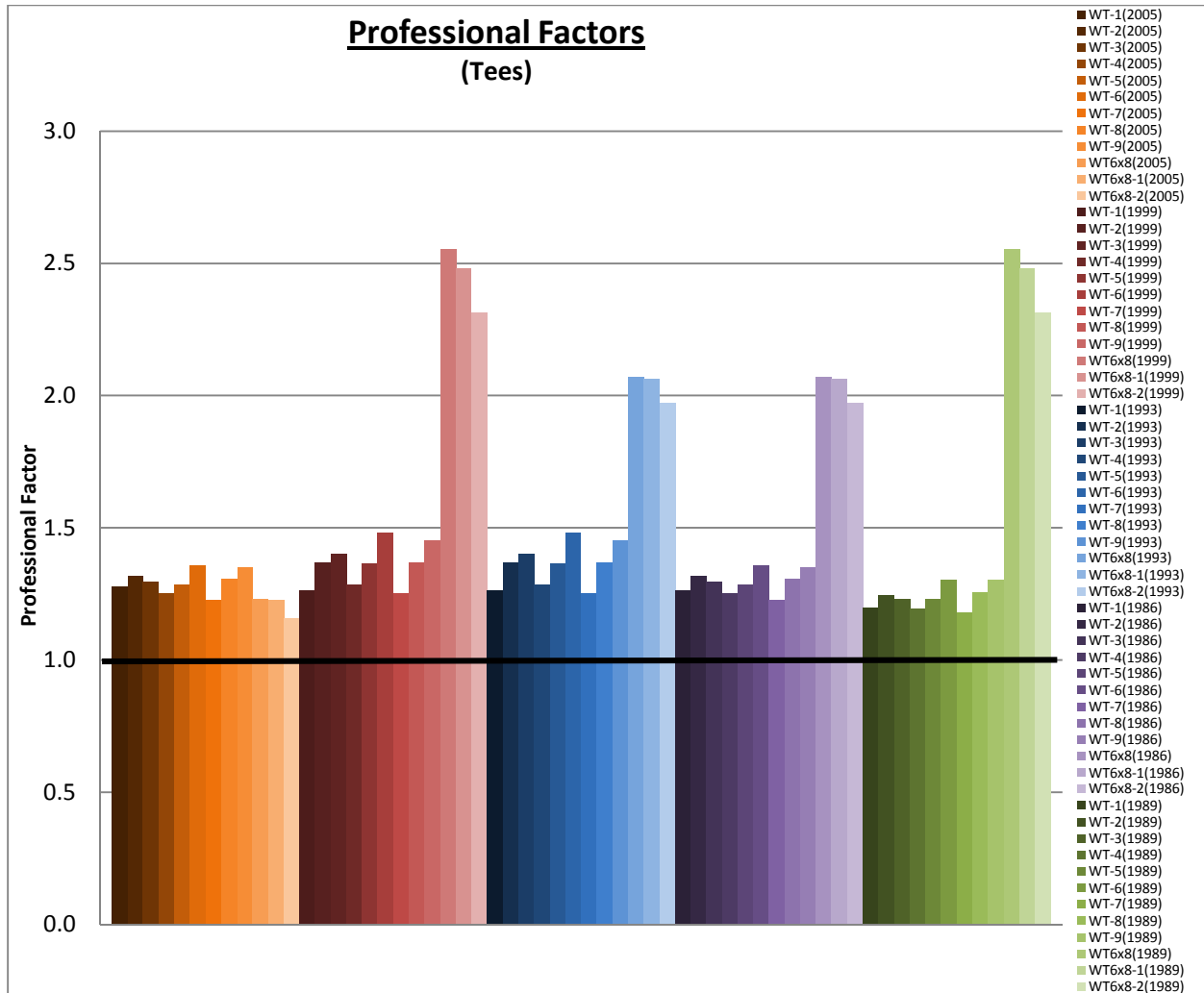


Figure 21: Professional factors of all tee specimens

4.3.2. Professional Factors < 1.0

Figures 19, 20, and 21 also show some specimens that have professional factors less than one. In these cases, the predicted block shear capacity was larger than the experimental capacity, and the AISC equation is considered unconservative. While this is not an ideal situation, it is not too significant if the professional factor remains above 0.75. This is because a safety factor of $\phi=0.75$ will be applied, reducing the predicted value down closer to the experimental capacity. All of the 1953 angle connections were found to be unconservative. The capacities used for comparison were given in the 1953 *Specification*. When these connections were used, block shear was not a known form of failure. Using the block shear equations for these connections now predicts that the angles could hold much

more than the given capacity in the code. Comparison of block shear capacity with given capacity indicates that these connections would exhibit some other failure mode.

Comparisons of experimental data for coped beams with a single line of bolts (B2 & C1) with the block shear provisions of the 1993 and 1986 *Specifications* also resulted in professional factors less than one. In the 1993 and 1986 *Specifications* two equations define block shear capacity, and the same equation governed the calculated capacities. The governing equation defined failure as tension and shear rupture. The next equation modification in the 1999 *Specification* introduced a maximum value for block shear capacity based on combine shear and tensile rupture. Consideration of this maximum block shear capacity resulted in professional factors greater than one when the experimental data was compared with the provisions of the 1999 and 2005 *Specifications*.

4.4. ANSYS Finite Element Model

While studying the calculated capacities and experimental data for the various connection geometries, structural tees with a changing length of the unbolted leg was the only geometry that could not be rationalized. Research showed that as the length of the unbolted leg increased, capacity decreased, but the AISC equations predicted an opposite effect. To better understand this trend, a finite element model was created using *ANSYS (ANSYS Workbench 2.0 Framework, 2009)*.

4.4.1. Finite Element Model

The tees were modeled as a solid element with holes to account for the bolt holes. A frictionless surface was defined for the top of the flange and the nodes along the inside of the bolt hole were defined as compression only supports. Next, a point force was positioned at the center of gravity on the end furthest from the connection. The point force was a ramped force starting at zero that increased linearly in value up to the failure capacity found in the experiments done by Epstein and Stamberg (2002).

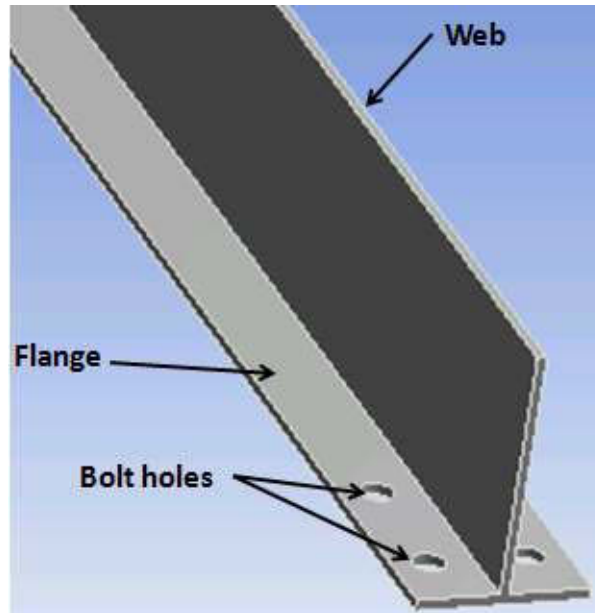


Figure 22: Finite element model

4.4.2. Finite Element Stress Results

Figures 23a through 23g show the stresses in the WT6x8 specimen modeled in ANSYS starting with a 30kip force that was increased by increments of 10 kips until the failure load of 92.133 kips was reached. The figures depicting the stress contours are shown with a plan view of the connection on the left and a lateral view on the right. Additional figures showing the models for the WT6x8-1 and WT6x8-2 specimens can be seen in Appendix C.

All three specimens followed similar trends. The stress contours show that the front two bolts initially take a large amount of stress. Then, the stress levels begin to rise in the web, indicating the forces propagated into the web and the back bolts began to resist more of the applied force. When the force approached the failure load observed experimentally the stress becomes more evenly distributed throughout the whole section.

These trends in the stress patterns shows local yielding at the low levels of force and then a redistribution of stresses as the applied force grew larger. Epstein and Stamberg (2002) found that these tees would fail by an alternate block shear path: tearing through the front bolt holes and up through the web. The stress contours produced in the ANSYS model displayed a similar path.

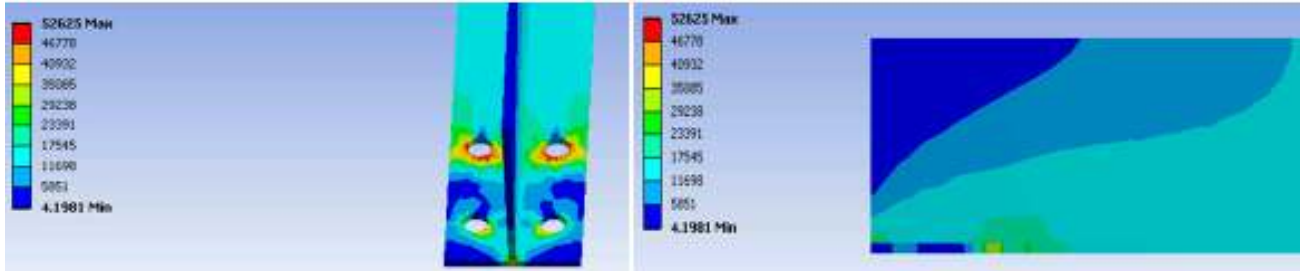


Figure 23a: WT6x8 30 kip force

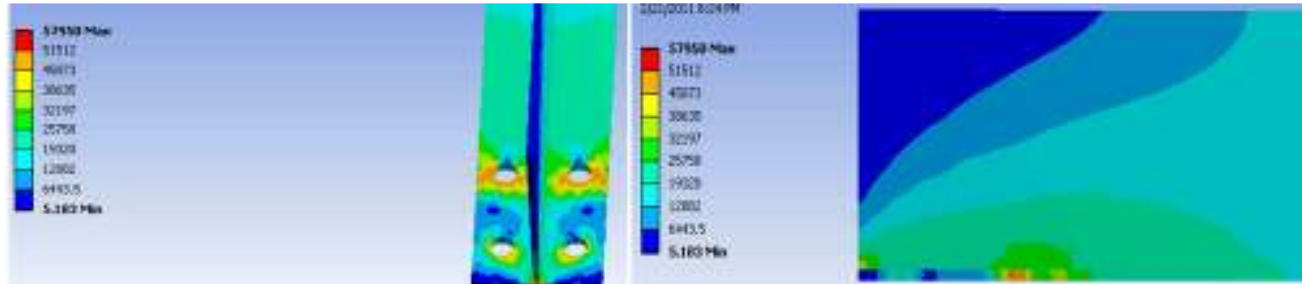


Figure 23b: WT6x8 40 kip force

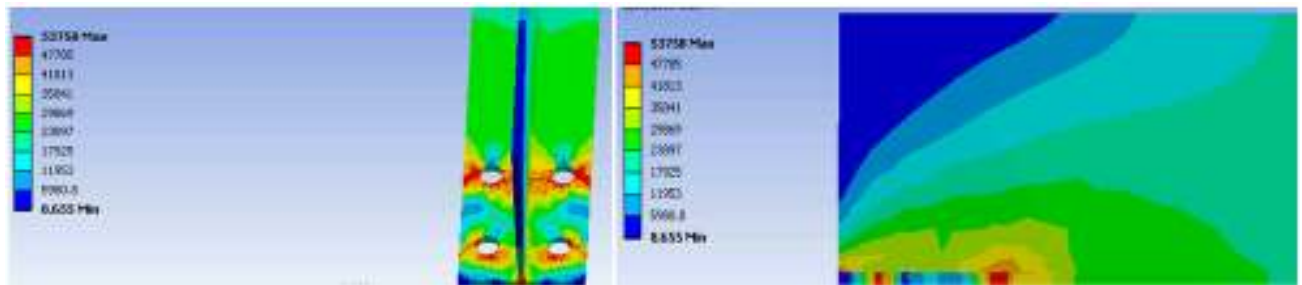


Figure 23c: WT6x8 50 kip force

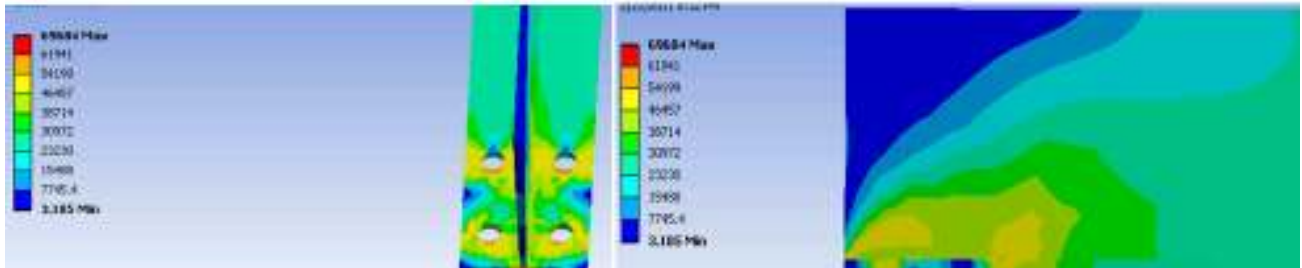


Figure 23d: WT6x8 60 kip force

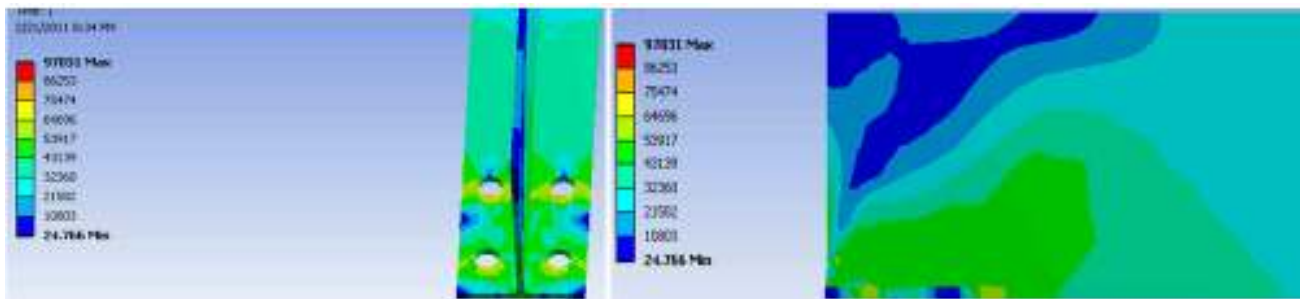


Figure 23e: WT6x8 70 kip force

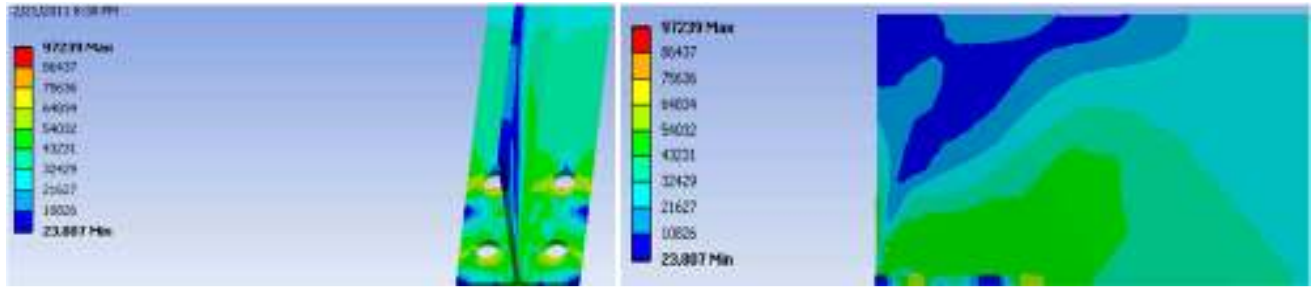


Figure 23f: WT6x8 80 kip force

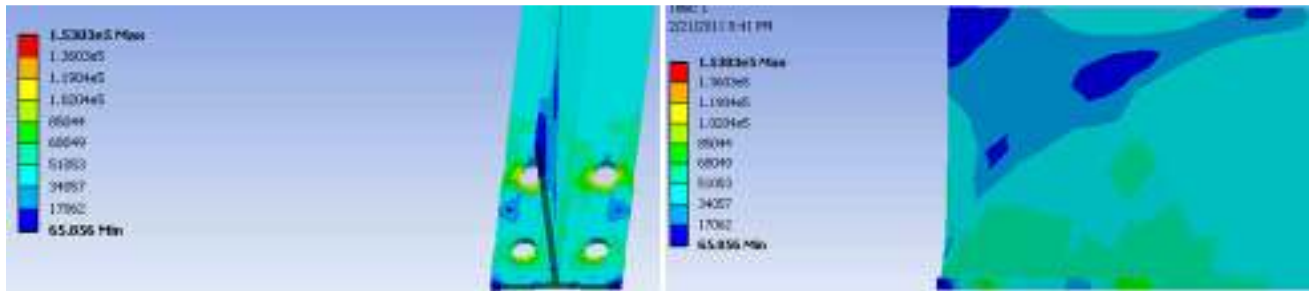


Figure 23g: WT6x8 92.133 kip force

4.4.3. Discussion

Table 4 suggests that, for these structural tees, block shear is the mode of failure. There is a large variation between the 2005 block shear capacity and the ASD block shear capacity. Also, both the 2005 and ASD *Specifications* show an opposite trend to the experimental data. This data gives reason to perform more finite element testing for these geometries since the model and AISC equations give very different results. Observing the stress pictures between 30 kips and 70 kips there are obvious differences in the distribution of stresses along the cross sections and fasteners. At 30 kips the stress within the cross section is concentrated at the point where the web meets the flange, and most of the stress along the fasteners is on the lead bolts. Then at 70 kips across the cross section the stress has propagated out along the flange and web. Along the fasteners at 70 kips, the stress is evenly distributed along all four bolts.

Using the time and stress tables and ramped forces, forces versus maximum stress graphs were created. These graphs are shown in Figures 25a through 25c. All three geometries show the same trend: the maximum stress is fairly constant and then grows rapidly after a certain load level is applied.

This is because the specimen was modeled as a bilinear stress and strain material (see Figure 24). Figure 24 shows that the material is elastic for low levels of stress and plastic at levels of stress greater than the yield strength (50,000 psi). The critical load level is approximately 50 ksi of force for all three specimens. At 50 ksi the grade 50 steel is at its yield level. The linear rise in stress with force corresponds to strain hardening. This occurs because the material modeled was a non-perfect plastic model. The tangent modulus is 210304.637 psi, which is the slope of the plastic region. In the strain hardening region the specimen will continue to take load and the deformation quickly becomes large. The specimen is in the fracturing region above 65 ksi. In design, the specimen should never be in this region. This is why a safety factor and the shear lag modifier are used. They both help to keep the capacity within a safe level.

Table 4: Summary of variable in tees with changing the length of outstanding legs

Specimen	Ultimate Load (kips)	$F_y * A_g$ (kips)	$F_u * A_e$ (kips)	Block Shear Capacities	
				2005 Specification (kips)	ASD Specification (kips)
WT6x8	92.133	115.95	87.75	74.83	36.07
WT6x8-1	91.8	104.95	87.75	74.83	36.99
WT6x8-2	87.6	93.95	96.85	75.78	37.89

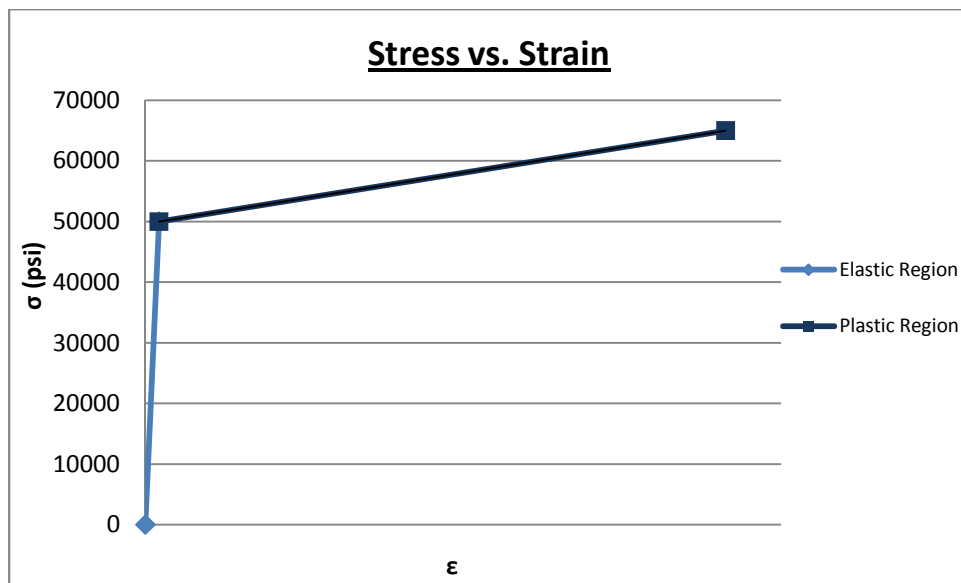


Figure 24: Stress vs. Strain for finite element model material

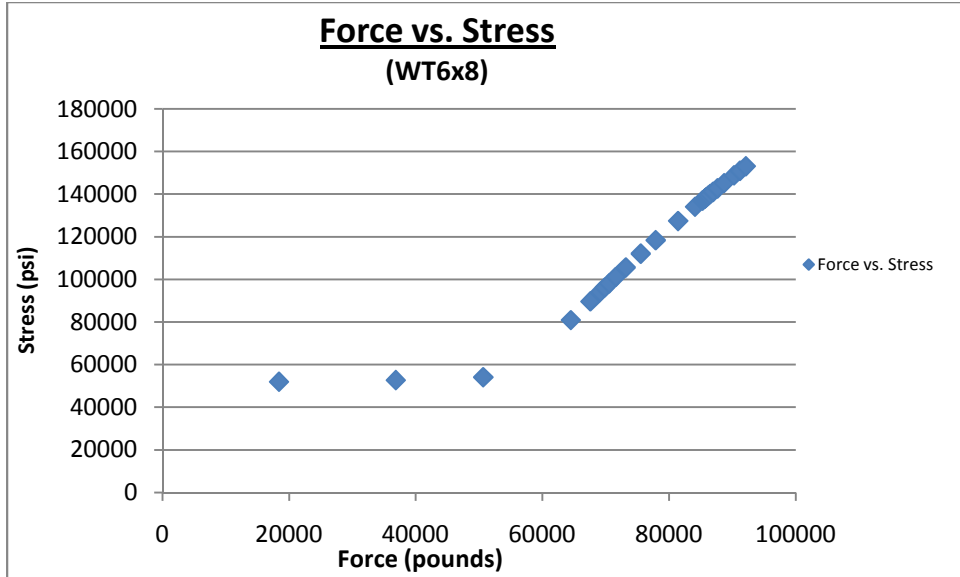


Figure 25a: Force vs. Stress (WT6x8)

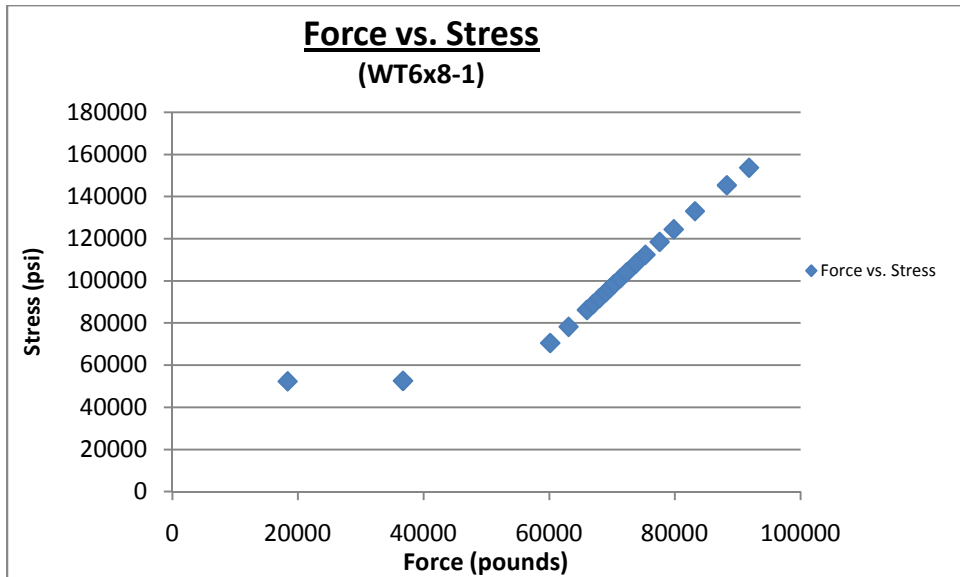


Figure 25b: Force vs. Stress (WT6x8-1)

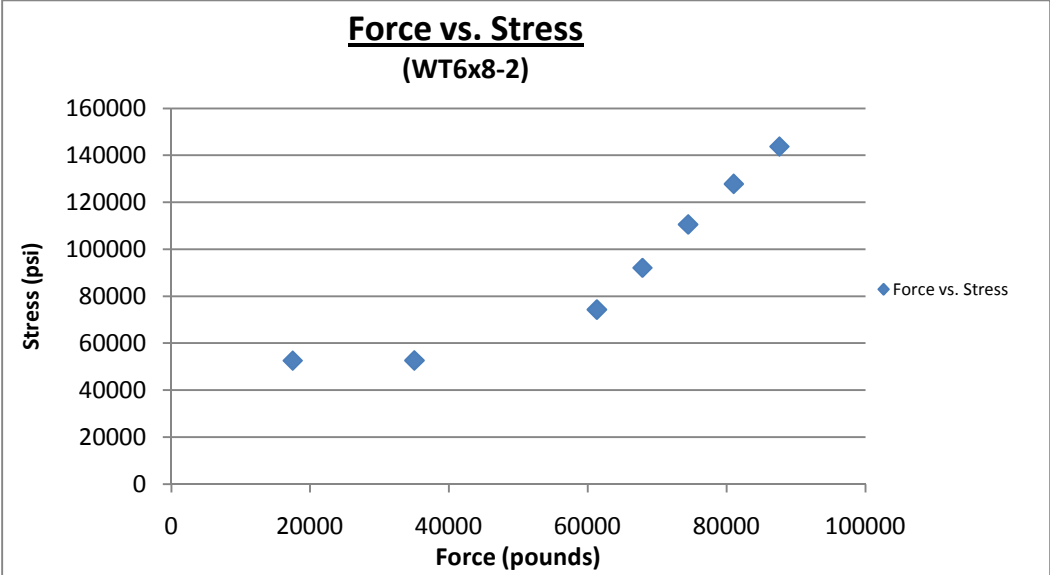


Figure 25c: Force vs. Stress (WT6x8-2)

5. Conclusions

Block shear has been an evolving concept since it was introduced in the 1978 AISC *Specification*.

This thesis reviewed the block shear equations by comparing their predicted capacities to experimental data. These comparisons were made using coped beams, angles, and structural tees. Overall, the calculated capacities followed similar trends as observed in the experimental data for nearly all of the geometry changes. This shows that the equations do an excellent job at predicting capacity. Looking at the 1953 connection allowed for a view of how connection designs have changed over the years. Connections are now designed to fail differently and to have a higher capacity. This demonstrates how connection technology and the engineering science base have evolved throughout the years.

The block shear equations have also shown evolution throughout the years. As the research base grew with the equations, changes were made to address the new knowledge approximately in the next specification. The first situation which showed a correction was coped beams with a single line of bolts. In the 1986 and 1993 *Specifications*, the professional factors were less than one, but in 1999 a maximum value for block shear capacity was implemented. In this work, it was observed that the professional factor became greater than one. Also, for coped beams with one and two line connections, all equations had predicted that they would both have the same capacity. In the 2005 equations, the value U_{bs} was added so that the two line connections would have a larger capacity than the single line connections. Lastly, for angles with a change in the outstanding leg length, the 1986 and 1993 equations did not include shear lag. Then, in the 1999 and 2005 equations, shear lag was included, solving the problem. This can be seen viewing the professional factors increasing from 1.19 to 1.36 between the 1993 equation to the 1999 equation, respectively.

There was a big evolution of the specifications going from ASD to LRFD. ASD was the most widely used specifications during the publications of the 1978 and 1989 ASD manual. Their block shear equations showed a good correlation for the majority of the geometries. Then, the first LRFD

Specification was published in 1986 and was not generally used and had its own flaws. In 1993, the second LRFD manual was beginning to be used more extensively and it too had flaws with block shear equations. Once LRFD had become the standard for design, the imperfections that were apparent in the 1986 and 1993 *Specifications* were discovered. The problems were remedied in the 1999 and 2005 LRFD *Specifications*. The block shear equations are always working towards reliable and accurate predictions, and the current LRFD equations are now used in almost all situations. These observations were made by viewing the professional factors. They show that the equations have become more refined over time.

One geometry, which was unable to be rationalized, was that of the structural tees with changes to the length of the unbolted legs. The experimental data and calculated capacities displayed opposite trends for the capacity versus length of the unbolted leg. There should be further testing done in order to understand why this occurs. First, similar specimens should be tested following the same testing procedures. This retesting will rule out any problems with the original research. Next, more specimens will need to be tested. In order to choose a group of specimens to be tested, a large number of groups will need to be analyzed before any physical testing can be done. Groups of tees with different lengths of the unbolted legs will have to be evaluated to ensure they will fail by block shear before any other form of failure. Then, these groups will be physically tested, and the ones that fail by block shear will be studied.

If the experiments are consistent with the testing already completed, then the equations may be predicting capacity incorrectly. For these specimens the connection geometry along the fasteners is not changing, so shear lag is very important for predicting block shear capacity as affected by changing the length of the outstanding leg. This variable and shear lag effects should be examined for structural tees. If shear lag is not properly being calculated for the tees, then the capacity will not be accurate.

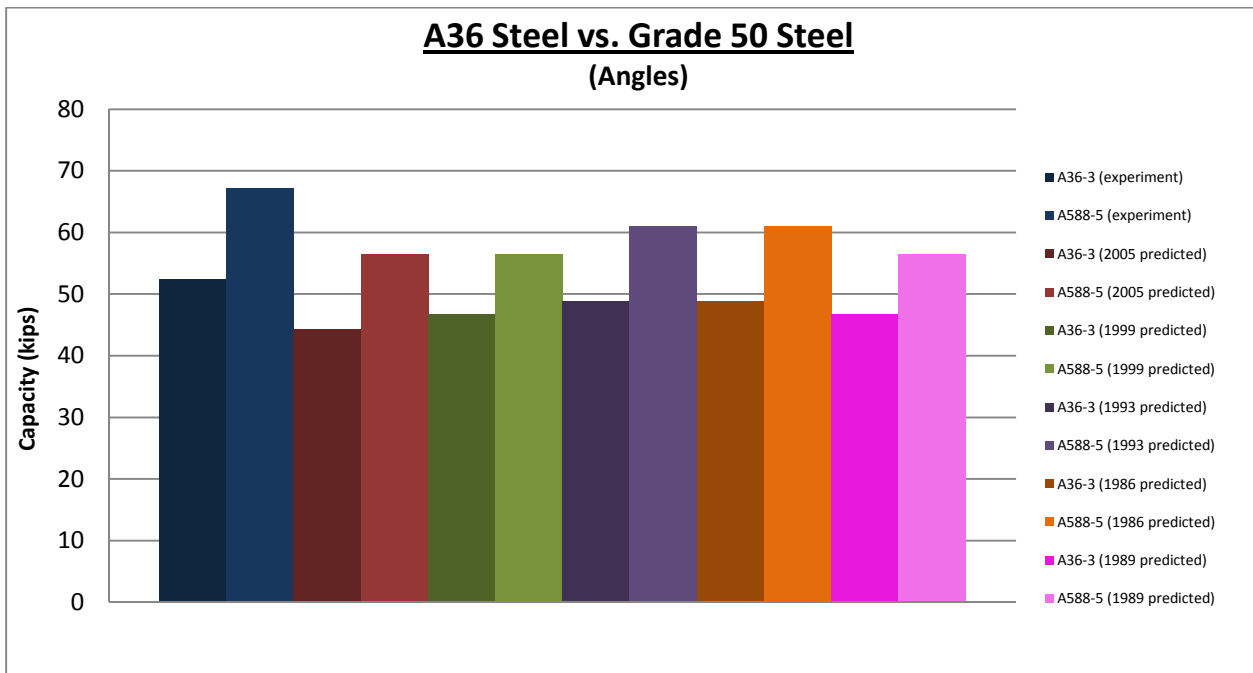
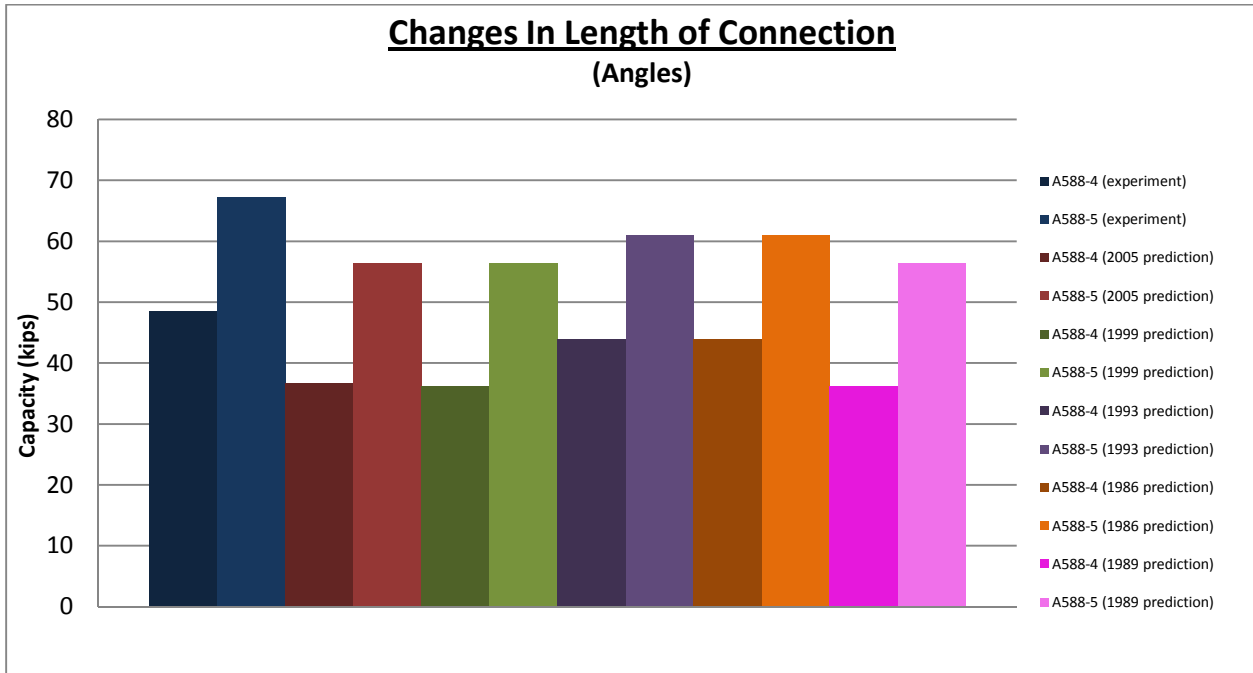
Prediction abilities are enhanced with continued research of connection failures, and the improved prediction capabilities allow for safer construction and structures. It is also important to not grossly over predict capacities, since overly conservative designs require more materials and increase cost without adding value.

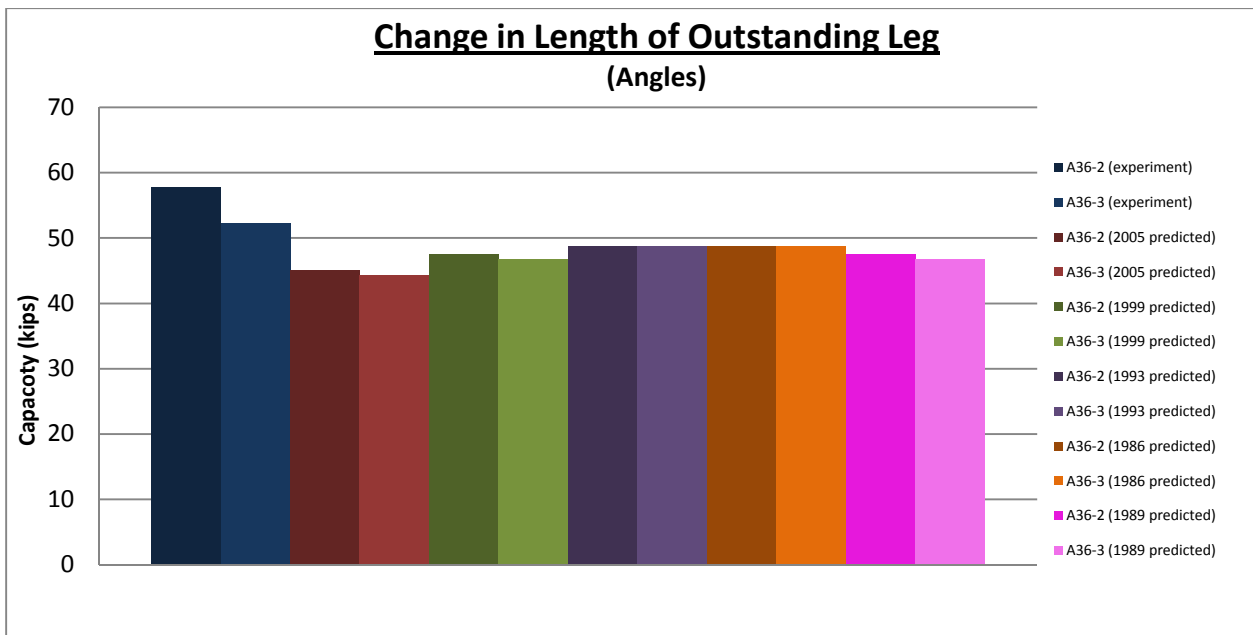
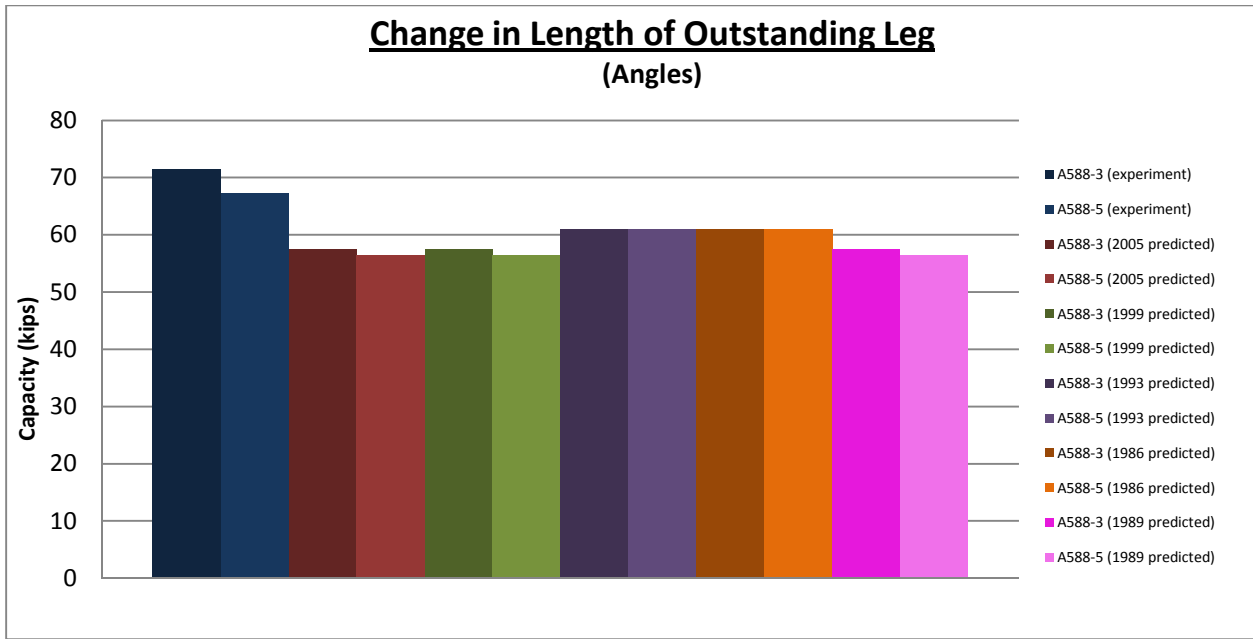
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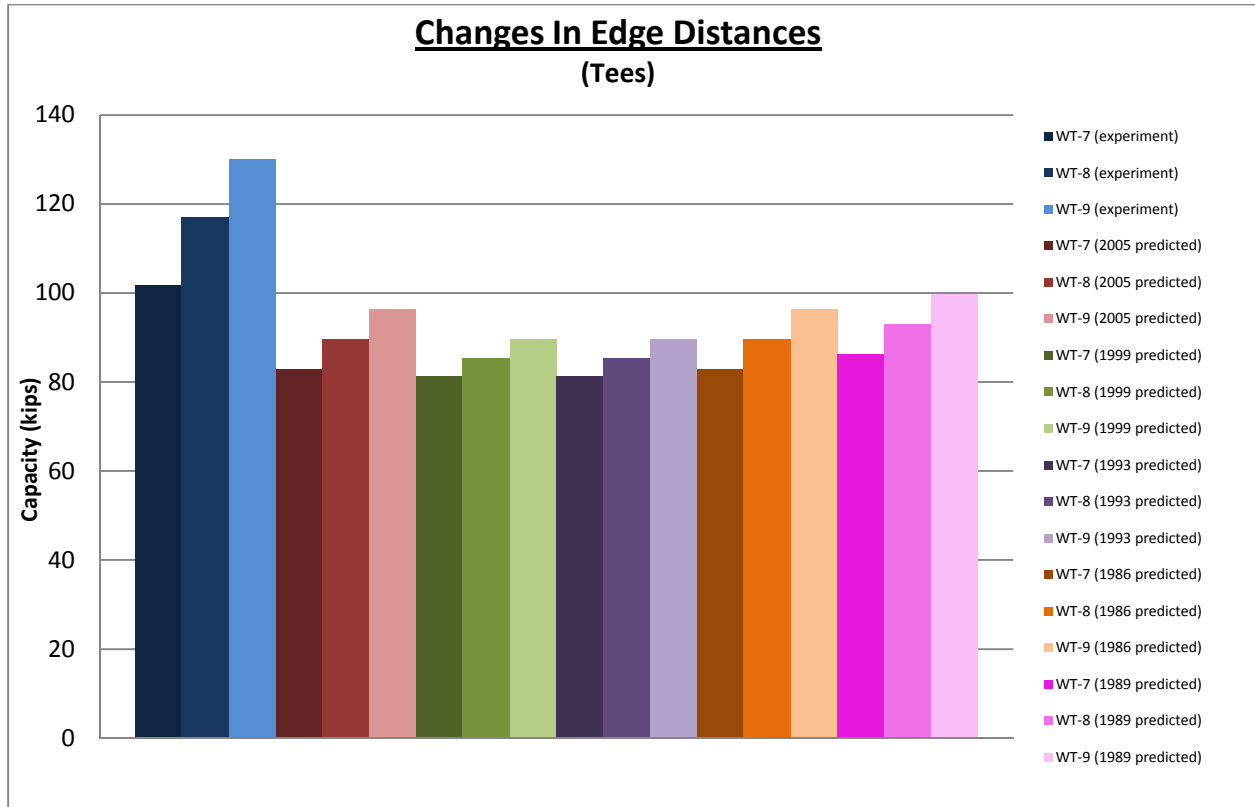
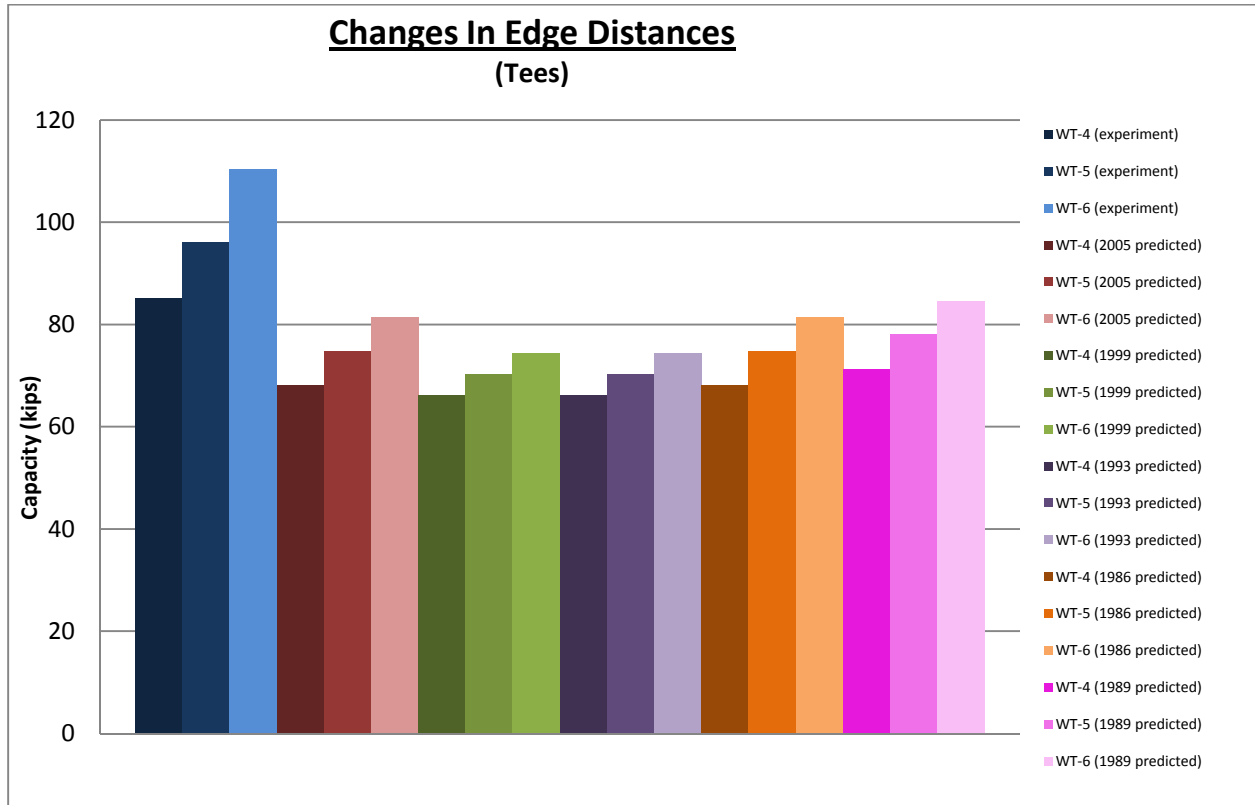
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Appendix A: Graphs (Angles)

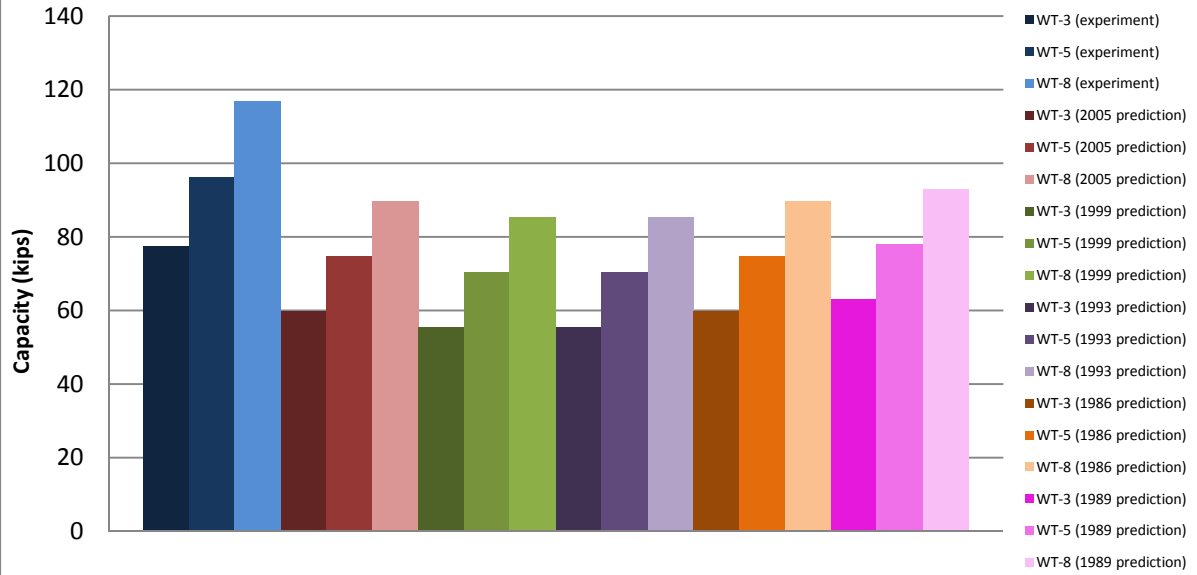




Appendix B: Graphs (Tees)

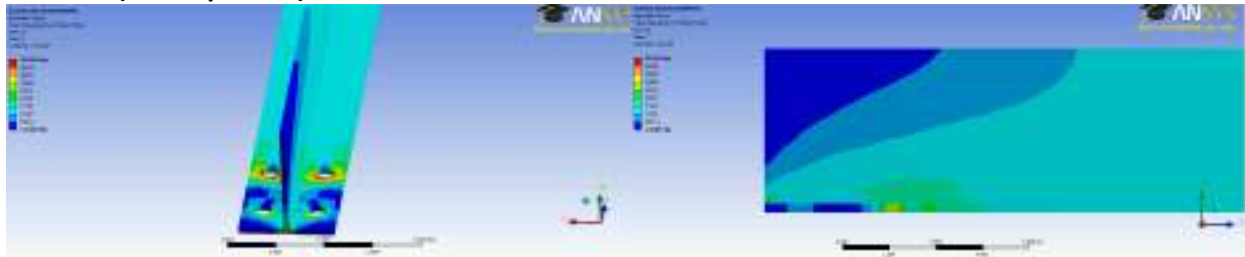


Changes In Length of Connection (Tees)

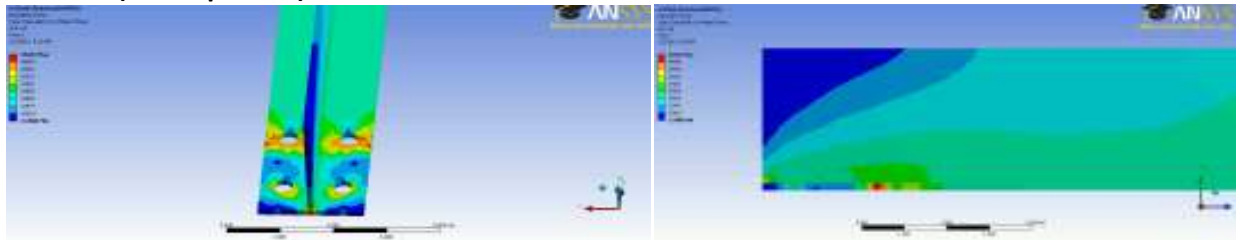


Appendix C: Stress Contours from Finite Element Analysis

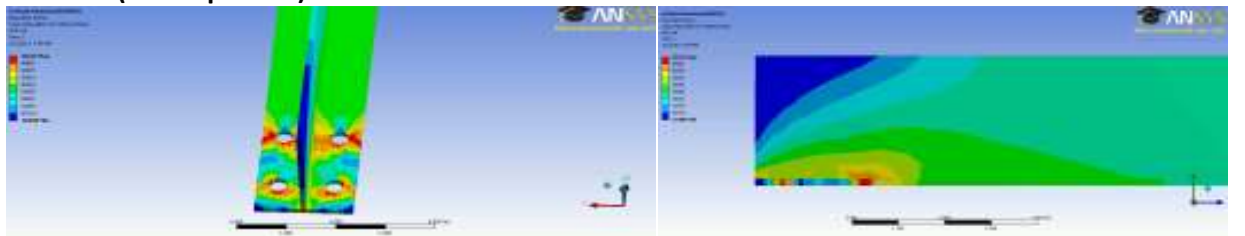
WT6x8-1 (30000 pounds)



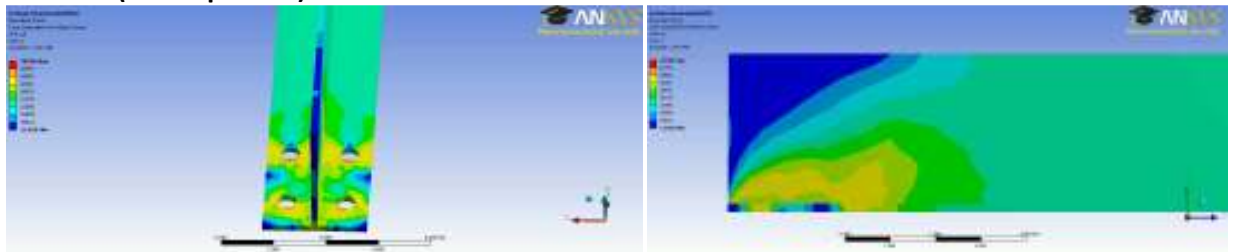
WT6x8-1 (40000 pounds)



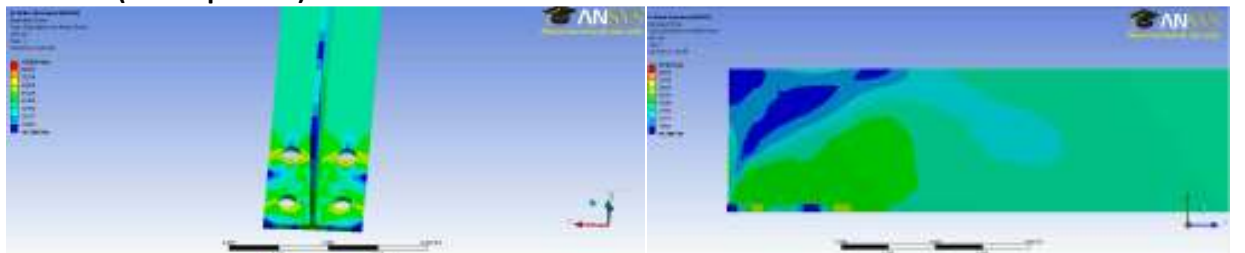
WT6x8-1 (50000 pounds)



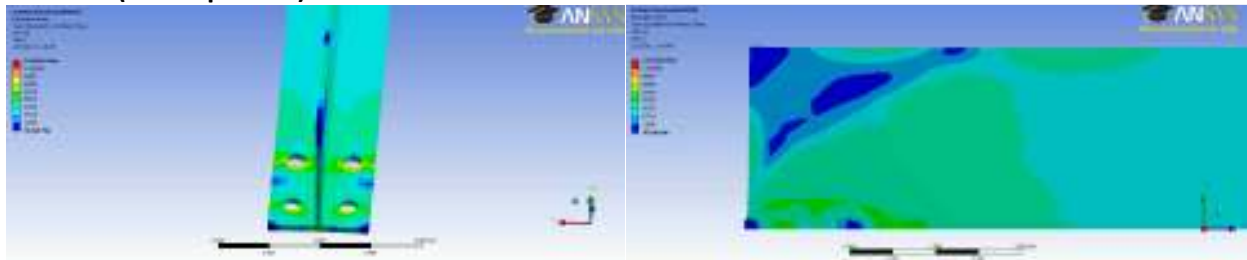
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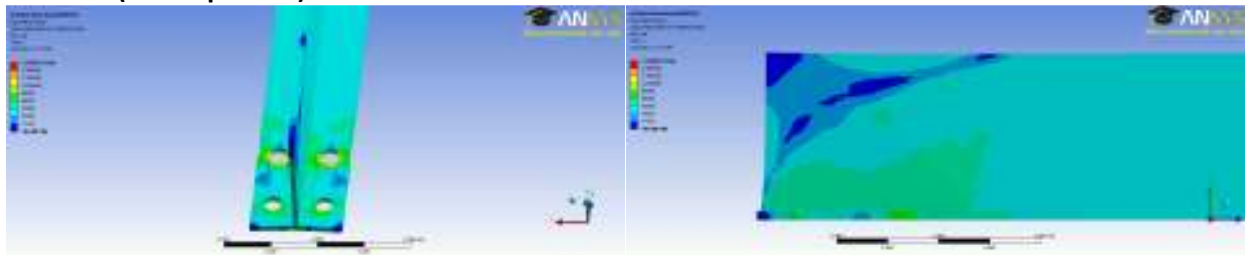
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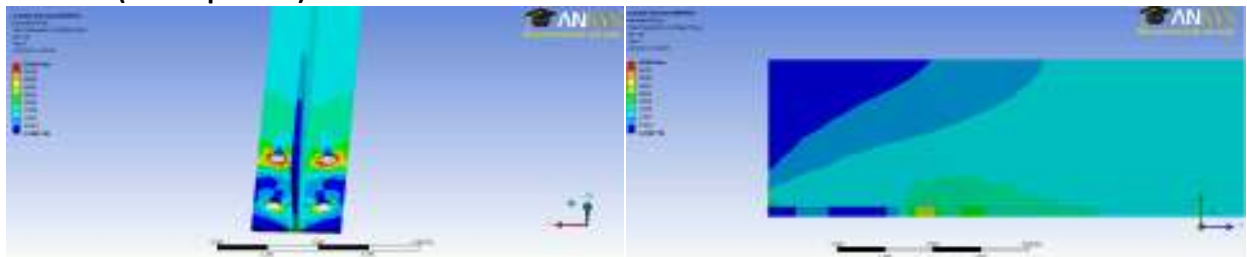
WT6x8-1 (80000 pounds)



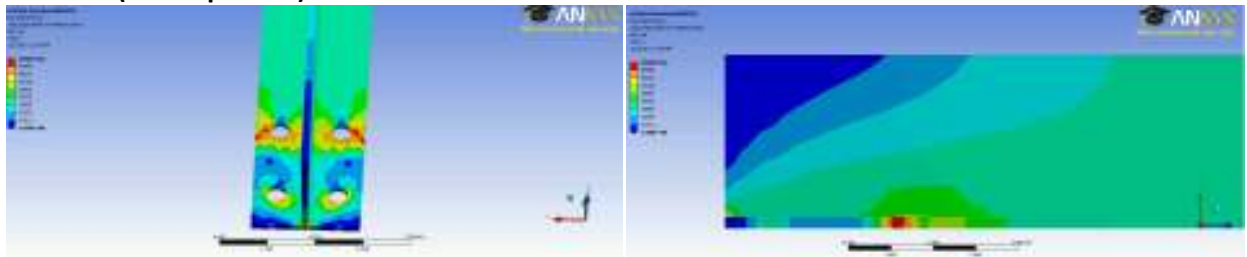
WT6x8-1 (91800 pounds)



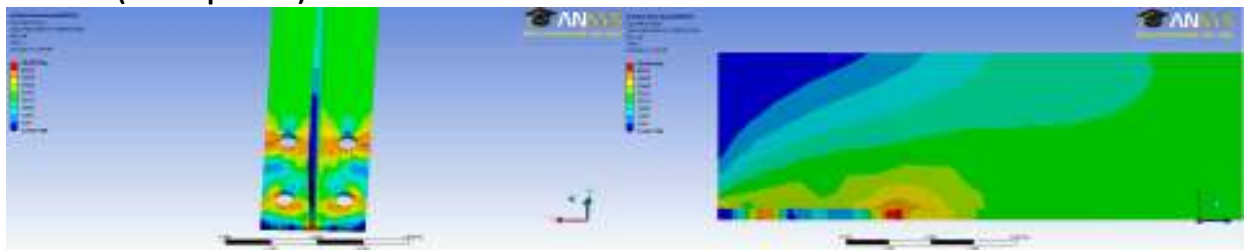
WT6x8-2 (30000 pounds)



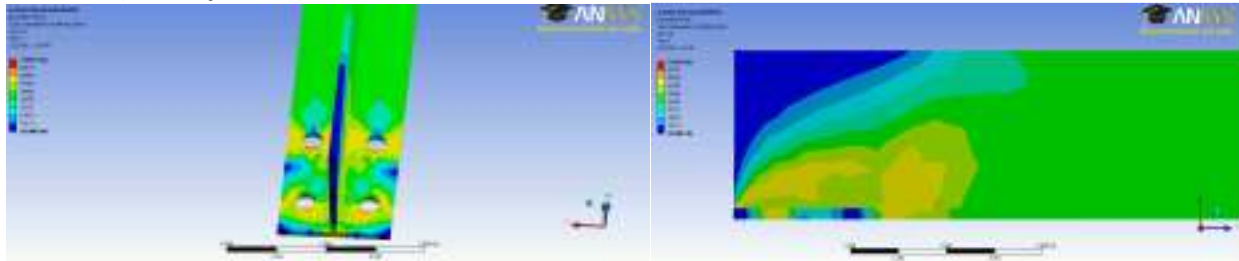
WT6x8-2 (40000 pounds)



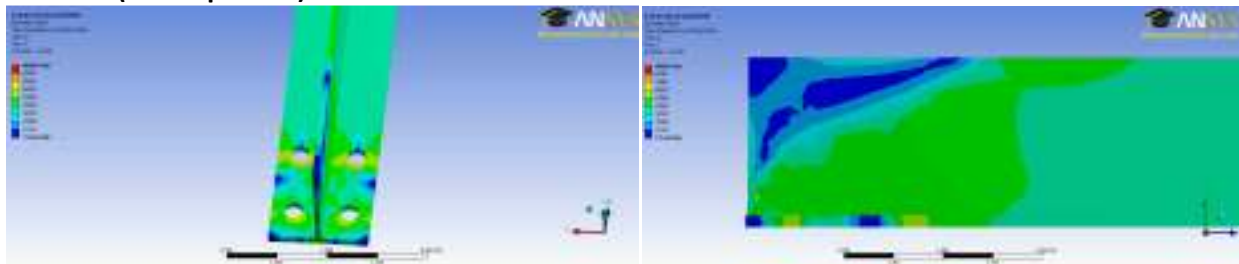
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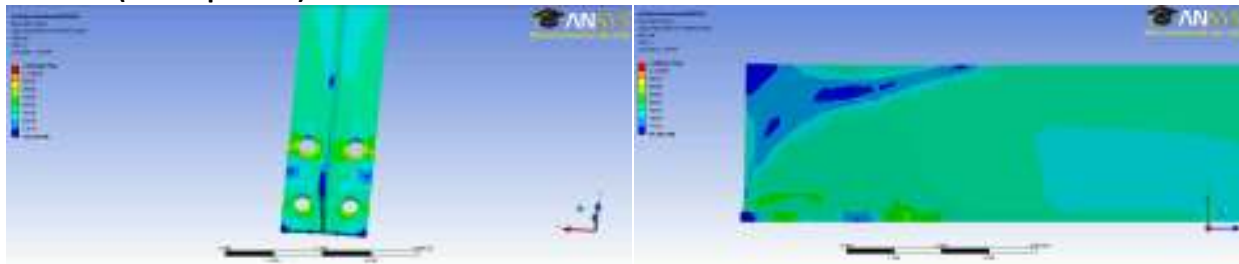
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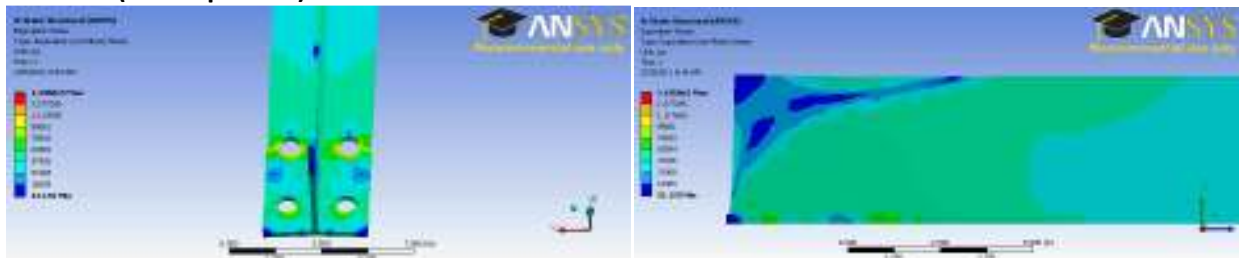
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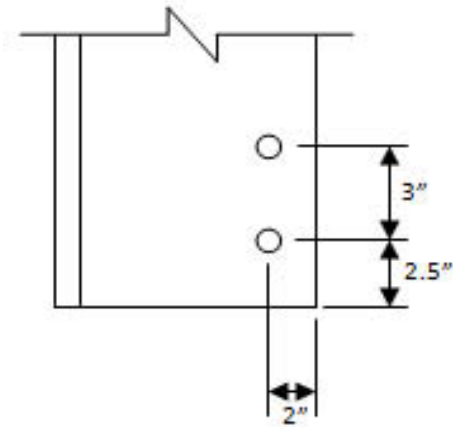
WT6x8-2 (80000 pounds)



WT6x8-2 (87600 pounds)

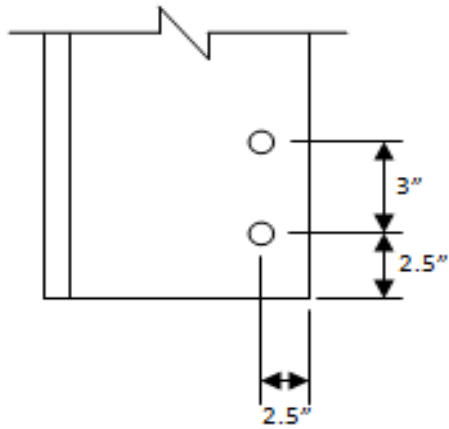


Appendix D: 1986 LRFD Excel Spreadsheets (Angles)



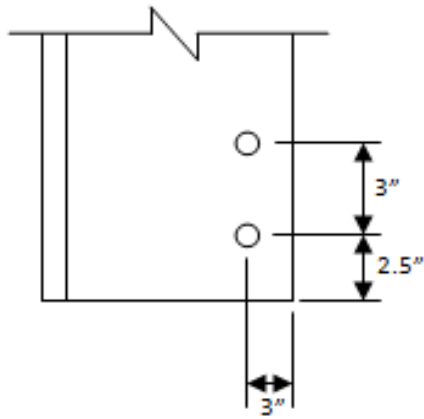
A-1

Shape:	L6x4x ⁵ / ₁₆		
Edge Distance:	2 inches		
Length of Connection:	5.5 inches		
Thickness:	0.3125 inches		
# Bolts in Tension:	0.5		
# Bolts in Shear:	1.5		
F _u :	58 ksi		
F _y :	36 ksi		
Bolt Diameter:	1 inches		
\bar{x} =	0.908 inches		
L =	3 inches		
A _{gt} =	0.63 in ²		
A _{nt} =	0.45 in ²		
A _{gv} =	1.72 in ²		
A _{nv} =	1.19 in ²		
U =	0.70		
A _e =	0.31 in ²		
R _n =	55.29 k		
R _n =	63.96 k	governs	
experimental failure load =	81.4 k		
PF =	1.27	CONSERVATIVE	



A-2

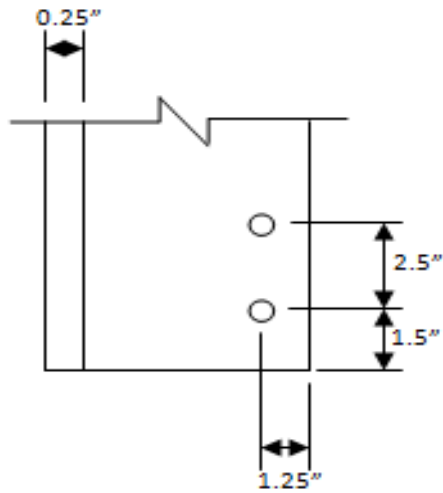
Shape:	L6x4x ⁵ / ₁₆	
Edge Distance:	2.5 inches	
Length of Connection:	5.5 inches	
Thickness:	0.3125 inches	
# Bolts in Tension:	0.5	
# Bolts in Shear:	1.5	
F_u :	58 ksi	
F_y :	36 ksi	
Bolt Diameter:	1 inches	
\bar{x} =	0.908 inches	
L =	3 inches	
A_{gt} =	0.78 in ²	
A_{nt} =	0.61 in ²	
A_{gv} =	1.72 in ²	
A_{nv} =	1.19 in ²	
U =	0.70	
A_e =	0.42 in ²	
R_n =	61.61 k	
R_n =	69.59 k	governs
experimental failure load =	99.99 k	
PF =	1.44	CONSERVATIVE



A-3

Shape:	L6x4x ⁵ / ₁₆	
Edge Distance:	3 inches	
Length of Connection:	5.5 inches	
Thickness:	0.3125 inches	
# Bolts in Tension:	0.5	
# Bolts in Shear:	1.5	
F_u :	58 ksi	
F_y :	36 ksi	
Bolt Diameter:	1 inches	
\bar{x} =	0.908 inches	
L =	3 inches	
A_{gt} =	0.94 in ²	
A_{nt} =	0.76 in ²	
A_{gv} =	1.72 in ²	
A_{nv} =	1.19 in ²	
U =	0.70	
A_e =	0.53 in ²	
R_n =	67.93 k	
R_n =	75.21 k	governs
experimental failure load =	112.4 k	
PF =	1.49	CONSERVATIVE

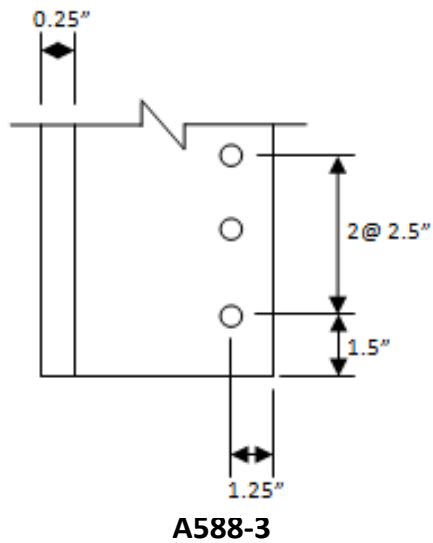
Outstanding Leg= 3"



A588-1

Edge Distance:	1.25 inches	
Length of Connection:	4 inches	
Thickness:	0.25 inches	
# Bolts in Tension:	0.5	
# Bolts in Shear:	1.5	
F_u :	70 ksi	
F_y :	50 ksi	
Bolt Diameter:	0.75 inches	
\bar{x} =	0.736 inches	
L =	2.5 inches	
A_{gt} =	0.31 in ²	
A_{nt} =	0.20 in ²	
A_{gv} =	1.00 in ²	
A_{nv} =	0.67 in ²	
U =	0.71	
A_e =	0.14 in ²	
R_n =	40.03 k	
R_n =	43.84 k	governs
experimental failure load =	52 k	
PF =	1.19	CONSERVATIVE

Outstanding Leg= 3"

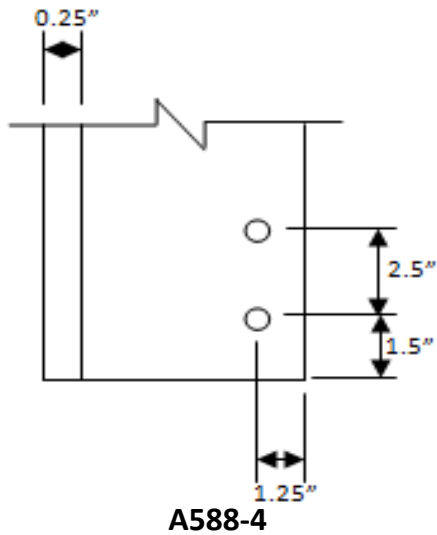


Edge Distance: 1.25 inches
 Length of Connection: 6.5 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

ϕ = 0.736 inches
 L = 5 inches
 A_{gt} = 0.31 in²
 A_{nt} = 0.20 in²
 A_{gv} = 1.63 in²
 A_{nv} = 1.08 in²
 U = 0.85
 A_e = 0.17 in²
 R_n = 60.88 k
 R_n = 60.91 k **governs**
 71.4 k
 PF = 1.17 **CONSERVATIVE**

experimental failure load=

Outstanding Leg= 4"

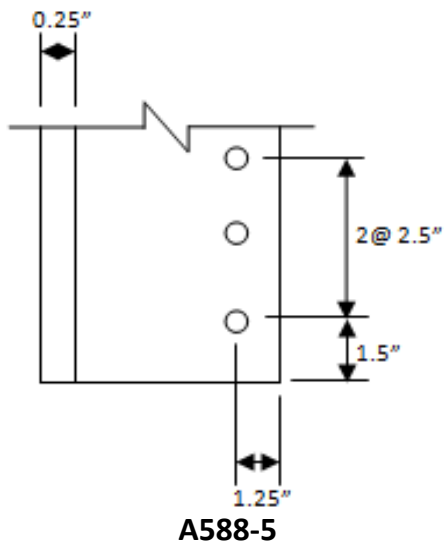


Edge Distance: 1.25 inches
 Length of Connection: 4 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

ϕ = 1.09 inches
 L = 2.5 inches
 A_{gt} = 0.31 in²
 A_{nt} = 0.20 in²
 A_{gv} = 1.00 in²
 A_{nv} = 0.67 in²
 U = 0.56
 A_e = 0.11 in²
 R_n = 38.02 k
 R_n = 43.84 k **governs**
 48.5 k
 PF = 1.11 **CONSERVATIVE**

experimental failure load =

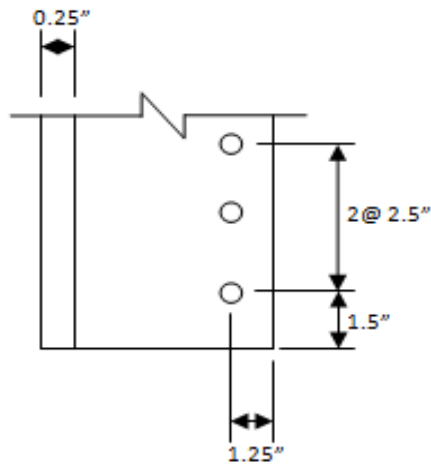
Outstanding Leg= 4"



Edge Distance: 1.25 inches
 Length of Connection: 6.5 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 1.09 inches
 L = 5 inches
 A_{gt} = 0.31 in²
 A_{nt} = 0.20 in²
 A_{gv} = 1.63 in²
 A_{nv} = 1.08 in²
 U = 0.78
 A_e = 0.16 in²
 R_n = 59.87 k
 R_n = 60.91 k **governs**
 experimental failure load = 67.2 k
 PF = 1.10 **CONSERVATIVE**

Outstanding Leg= 3"

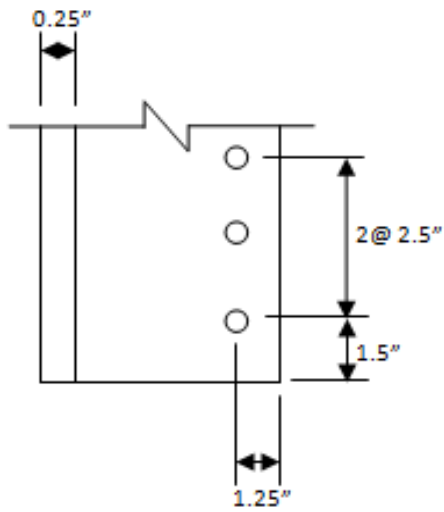


A36-2

Edge Distance:	1.25 inches	
Length of Connection:	6.5 inches	
Thickness:	0.25 inches	
# Bolts in Tension:	0.5	
# Bolts in Shear:	2.5	
F_u :	58 ksi	
F_y :	36 ksi	
Bolt Diameter:	0.75 inches	
ϕ =	0.736	inches
L=	5	inches
A_{gt} =	0.31	in ²
A_{nt} =	0.20	in ²
A_{gv} =	1.63	in ²
A_{nv} =	1.08	in ²
U=	0.85	
A_e =	0.17	in ²
R_n =	45.15	k
R_n =	48.77	k
experimental failure load=	57.8	k
PF=	1.19	CONSERVATIVE

governs

Outstanding Leg= 4"

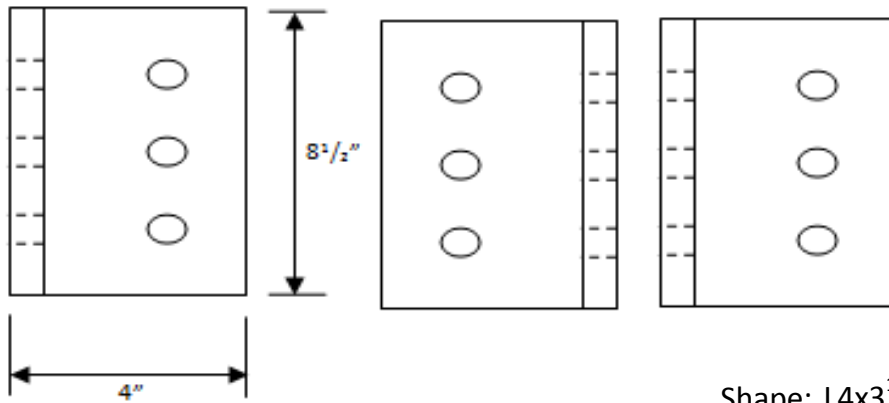


A36-3

Edge Distance:	1.25 inches	
Length of Connection:	6.5 inches	
Thickness:	0.25 inches	
# Bolts in Tension:	0.5	
# Bolts in Shear:	2.5	
F_u :	58 ksi	
F_y :	36 ksi	
Bolt Diameter:	0.75 inches	
ϕ	1.09	inches
L	5	inches
A_{gt}	0.31	in ²
A_{nt}	0.20	in ²
A_{gv}	1.63	in ²
A_{nv}	1.08	in ²
U	0.78	
A_e	0.16	in ²
R_n	44.31	k
R_n	48.77	k
experimental failure load	52.3	k
PF	1.07	CONSERVATIVE

governs

1953 CONNECTION



Shape: $L4 \times 3 \frac{1}{2} \times \frac{3}{8}$

Edge Distance: 1.75 inches

Length of Connection: 7.25 inches

Thickness: 0.38 inches

Bolts in Tension: 0.5

Bolts in Shear: 2.5

F_u : 58 ksi

F_y : 36 ksi

Bolt Diameter: 0.75 inches

\bar{x} = 0.947 inches

L = 6 inches

A_{gt} = 0.66 in^2

A_{nt} = 0.49 in^2

A_{gv} = 2.72 in^2

A_{nv} = 1.90 in^2

U = 0.84

A_e = 0.41 in^2

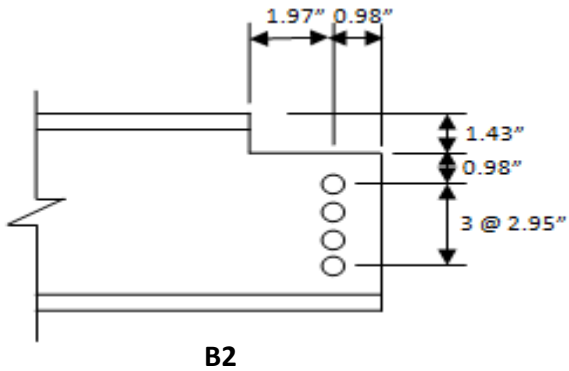
R_n = 165.53 k

R_n = 179.38 k **governs**

experimental failure load = 21.6 k

PF = 0.12 **UNCONSERVATIVE**

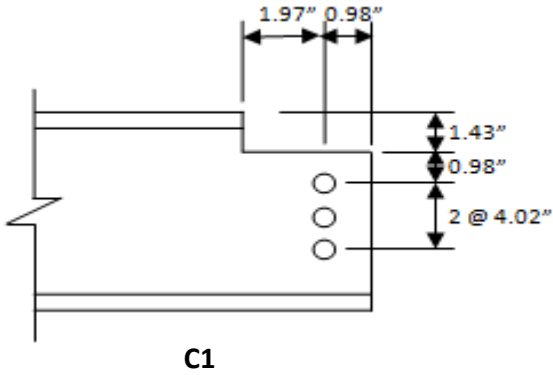
Appendix E: 1986 LRFD Excel Spreadsheets (Coped Beams)



Shape:	W16x31
Edge Distance:	0.98 inches
Length of Connection:	9.83 inches
Thickness:	0.275 inches
# Bolts in Tension:	3.5
# Bolts in Shear:	0.5
F_u :	74.4 ksi
F_y :	53.2 ksi
Bolt Diameter:	0.75 inches

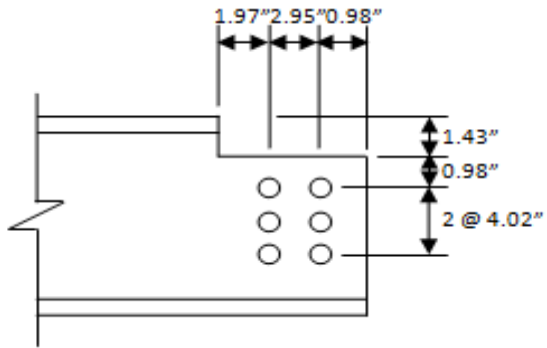
\bar{x}	=	0	inches
L	=	8.85	inches
A_{gt}	=	0.27	in ²
A_{nt}	=	-0.57	in ²
A_{gv}	=	2.70	in ²
A_{nv}	=	2.58	in ²
U	=	1.00	
A_e	=	-0.57	in ²
R_n	=	43.68	k
R_n	=	129.64	k
experimental failure load	=	106.7842	k
PF	=	0.82	UNCONSERVATIVE

governs



Shape: W16x31
 Edge Distance: 0.98 inches
 Length of Connection: 9.02 inches
 Thickness: 0.275 inches
 # Bolts in Tension: 3.5
 # Bolts in Shear: 0.5
 F_u : 74.8 ksi
 F_y : 53.1 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 0 inches
 L = 8.04 inches
 A_{gt} = 0.27 in²
 A_{nt} = -0.57 in²
 A_{gv} = 2.48 in²
 A_{nv} = 2.36 in²
 U = 1.00
 A_e = -0.57 in²
 R_n = 36.19 k
 R_n = 120.24 k **governs**
 experimental failure load = 90.3732 k
 PF = 0.75 **UNCONSERVATIVE**

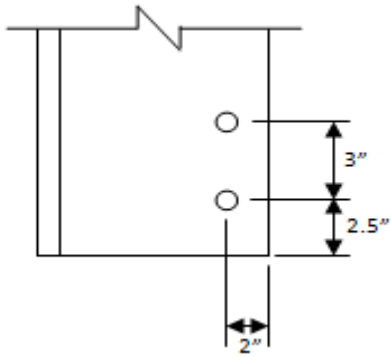


C2

Shape: W16x31
 Edge Distance: 0.98 inches
 Length of C: 9.02 inches
 Thickness: 0.275 inches
 # Bolts in Tension: 3.5
 # Bolts in Shear: 0.5
 F_u : 74.8 ksi
 F_y : 53.1 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 0 inches
 L = 8.04 inches
 A_{gt} = 0.27 in²
 A_{nt} = -0.57 in²
 A_{gv} = 2.48 in²
 A_{nv} = 2.36 in²
 U = 1.00
 A_e = -0.57 in²
 R_n = 36.19 k
 R_n = 120.24 k **governs**
 experimental failure load = 120.7224 k
 PF = 1.00 **CONSERVATIVE**

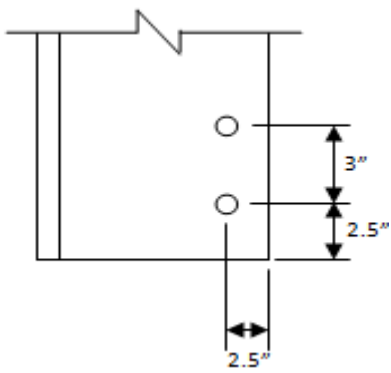
Appendix F: 1986 LRFD Excel Spreadsheets (Tees)



WT-1

Shape:	WT7x11
Edge Distance:	2 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

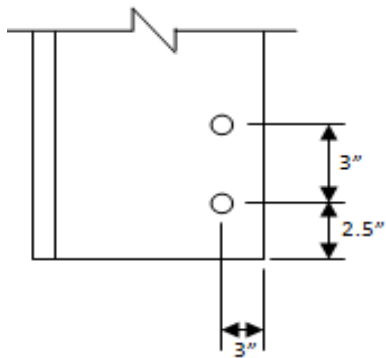
\bar{x} =	0	inches	
L =	3	inches	
A_{gt} =	0.46	in ²	
A_{nt} =	0.33	in ²	
A_{gv} =	1.27	in ²	
A_{nv} =	0.88	in ²	
U =	1.00		
A_e =	0.33	in ²	
R_n =	46.50	k	
R_n =	47.08	k	governs
experimental failure load =	59.5	k	
PF =	1.26	CONSERVATIVE	



WT-2

Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

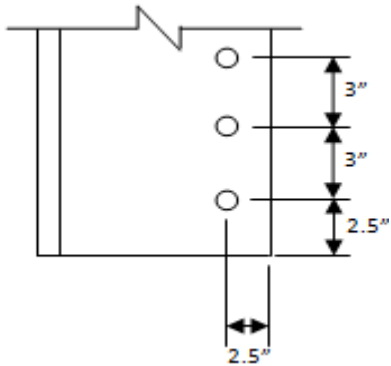
\bar{x} =	0	inches	
L =	3	inches	
A_{gt} =	0.58	in ²	
A_{nt} =	0.45	in ²	
A_{gv} =	1.27	in ²	
A_{nv} =	0.88	in ²	
U =	1.00		
A_e =	0.45	in ²	
R_n =	53.17	k	governs
R_n =	51.22	k	
experimental failure load =	70.1	k	
PF =	1.32		CONSERVATIVE



WT-3

Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

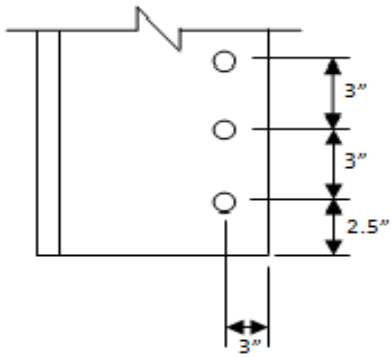
\bar{x} =	0	inches	
L =	3	inches	
A_{gt} =	0.69	in ²	
A_{nt} =	0.56	in ²	
A_{gv} =	1.27	in ²	
A_{nv} =	0.88	in ²	
U =	1.00		
A_e =	0.56	in ²	
R_n =	59.84	k	governs
R_n =	55.36	k	
experimental failure load =	77.6	k	
PF =	1.30		CONSERVATIVE



WT-4

Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

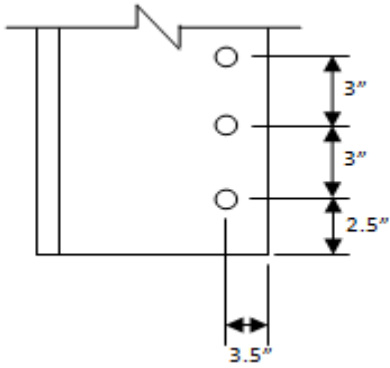
\bar{x} =	0	inches	
L =	6	inches	
A_{gt} =	0.58	in ²	
A_{nt} =	0.45	in ²	
A_{gv} =	1.96	in ²	
A_{nv} =	1.31	in ²	
U =	1.00		
A_e =	0.45	in ²	
R_n =	68.07	k	governs
R_n =	66.22	k	
experimental failure load =	85.2	k	
PF =	1.25	CONSERVATIVE	



WT-5

Shape: WT7x11
 Edge Distance: 3 inches
 Length of Connection: 8.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

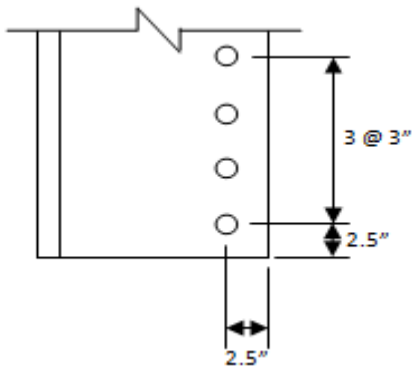
\bar{x} = 0 inches
 L = 6 inches
 A_{gt} = 0.69 in²
 A_{nt} = 0.56 in²
 A_{gv} = 1.96 in²
 A_{nv} = 1.31 in²
 U = 1.00
 A_e = 0.56 in²
 R_n = 74.74 k **governs**
 R_n = 70.36 k
 experimental failure load = 96.1 k
 PF = 1.29 **CONSERVATIVE**



WT-6

Shape:	WT7x11
Edge Distance:	3.5 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

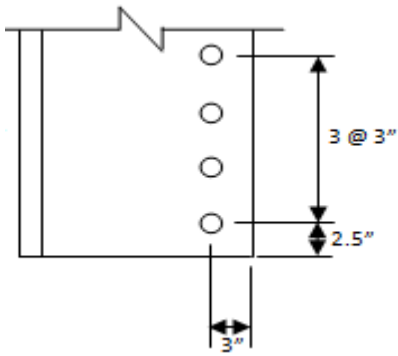
\bar{x}	=	0	inches	
L	=	6	inches	
A_{gt}	=	0.81	in^2	
A_{nt}	=	0.68	in^2	
A_{gv}	=	1.96	in^2	
A_{nv}	=	1.31	in^2	
U	=	1.00		
A_e	=	0.68	in^2	
R_n	=	81.41	k	governs
R_n	=	74.50	k	
experimental failure load	=	110.4	k	
PF	=	1.36		CONSERVATIVE



WT-7

Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	11.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	3.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

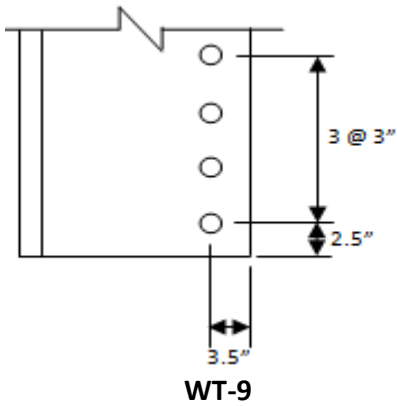
\bar{x} =	0	inches	
L =	9	inches	
A_{gt} =	0.58	in^2	
A_{nt} =	0.45	in^2	
A_{gv} =	2.65	in^2	
A_{nv} =	1.74	in^2	
U =	1.00		
A_e =	0.45	in^2	
R_n =	82.98	k	governs
R_n =	81.23	k	
experimental failure load =	101.8	k	
PF =	1.23		CONSERVATIVE



WT-8

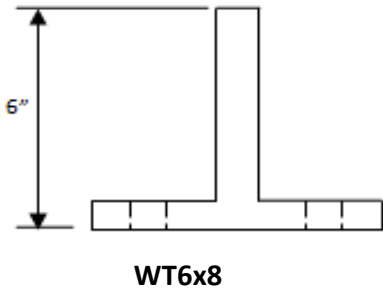
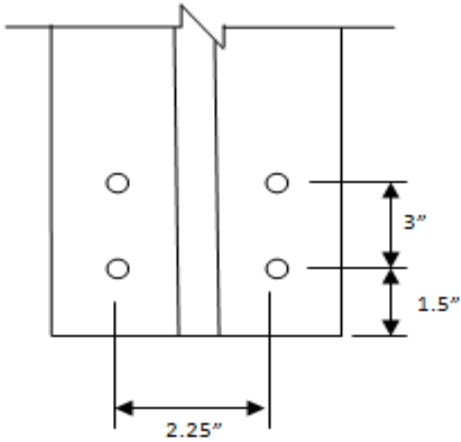
Shape: WT7x11
 Edge Distance: 3 inches
 Length of Connection: 11.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 3.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

$\bar{x} = 0$ inches
 $L = 9$ inches
 $A_{gt} = 0.69$ in²
 $A_{nt} = 0.56$ in²
 $A_{gv} = 2.65$ in²
 $A_{nv} = 1.74$ in²
 $U = 1.00$
 $A_e = 0.56$ in²
 $R_n = 89.65$ k **governs**
 $R_n = 85.37$ k
 experimental failure load = 116.99 k
 PF = 1.30 **CONSERVATIVE**



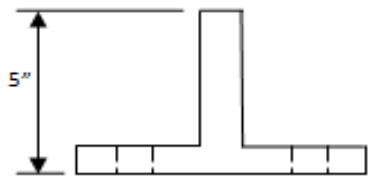
Shape: WT7x11
 Edge Distance: 3.5 inches
 Length of Connection: 11.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 3.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

$\bar{x} = 0$ inches
 $L = 9$ inches
 $A_{gt} = 0.81$ in²
 $A_{nt} = 0.68$ in²
 $A_{gv} = 2.65$ in²
 $A_{nv} = 1.74$ in²
 $U = 1.00$
 $A_e = 0.68$ in²
 $R_n = 96.32$ k **governs**
 $R_n = 89.51$ k
 experimental failure load = 129.98 k
 PF = 1.35 **CONSERVATIVE**



Shape: WT6x8
 Edge Distance: 0.87 inches
 Length of Connection: 4.5 inches
 Thickness: 0.265 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 65 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

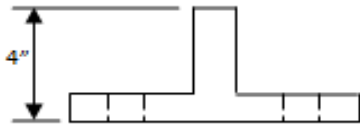
ϕ = 1.74 inches
 L = 3 inches
 A_{gt} = 0.23 in²
 A_{nt} = 0.11 in²
 A_{gv} = 1.19 in²
 A_{nv} = 0.84 in²
 U = 0.42
 A_e = 0.05 in²
 R_n = 38.90 k
 R_n = 44.47 k **governs**
 experimental failure load = 92.13 k
 PF = 2.07 **CONSERVATIVE**



WT6x8-1

Shape:	WT6x8-1
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

\bar{x} =	1.37	inches	
L =	3	inches	
A_{gt} =	0.23	in ²	
A_{nt} =	0.11	in ²	
A_{gv} =	1.19	in ²	
A_{nv} =	0.84	in ²	
U =	0.54		
A_e =	0.06	in ²	
R_n =	39.82	k	
R_n =	44.47	k	governs
experimental failure load =	91.8	k	
PF =	2.06	CONSERVATIVE	

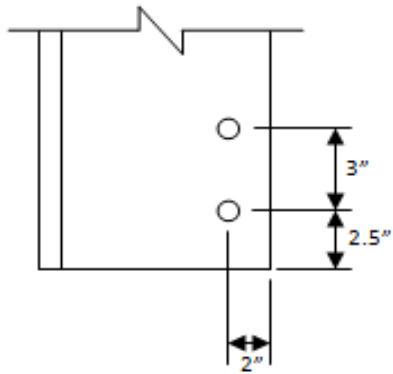


WT6x8-2

Shape:	WT6x8-2
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

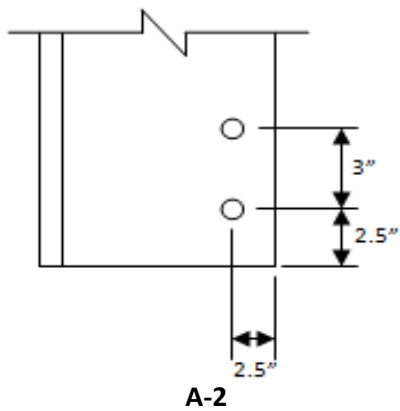
\bar{x} =	1.007	inches	
L=	3	inches	
A_{gt} =	0.23	in ²	
A_{nt} =	0.11	in ²	
A_{gv} =	1.19	in ²	
A_{nv} =	0.84	in ²	
U=	0.66		
A_e =	0.08	in ²	
R_n =	40.72	k	
R_n =	44.47	k	governs
experimental failure load=	87.6	k	
PF=	1.97	CONSERVATIVE	

Appendix G: 1989 & 1979 ASD Excel Spreadsheets (Angles)



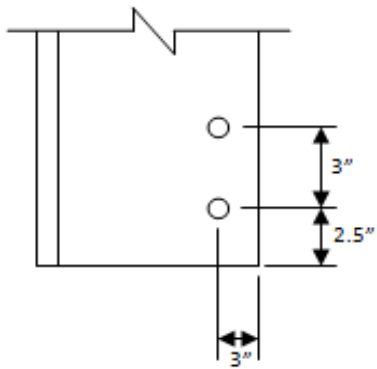
A-1

Shape:	L6x4x ⁵ / ₁₆		
Edge Distance:	2 inches		
Length of Connection:	5.5 inches		
Thickness:	0.3125 inches		
# Bolts in Tension:	0.5		
# Bolts in Shear:	1.5		
F_u :	58 ksi		
F_y :	36 ksi		
Bolt Diameter:	1 inch		
\bar{x} =	0.908 inches		
L=	3 inches		
A_{gt} =	0.63 in ²		
A_{nt} =	0.45 in ²		
A_{gv} =	1.72 in ²		
A_{nv} =	1.19 in ²		
U=	0.70		
A_e =	0.31 in ²		
V_a =	52.36 k		
V_a =	59.63 k	governs	
experimental failure load=	81.4 k		
PF=	1.37	CONSERVATIVE	



Shape: L6x4x⁵/₁₆
 Edge Distance: 2.5 inches
 Length of Connection: 5.5 inches
 Thickness: 0.3125 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u: 58 ksi
 F_y: 36 ksi
 Bolt Diameter: 1 inches

\bar{x} = 0.908 inches
 L = 3 inches
 A_{gt} = 0.78 in²
 A_{nt} = 0.61 in²
 A_{gv} = 1.72 in²
 A_{nv} = 1.19 in²
 U = 0.70
 A_e = 0.42 in²
 V_a = 56.15 k
 V_a = 65.95 k **governs**
 experimental failure load = 99.99 k
 PF = 1.52 **CONSERVATIVE**

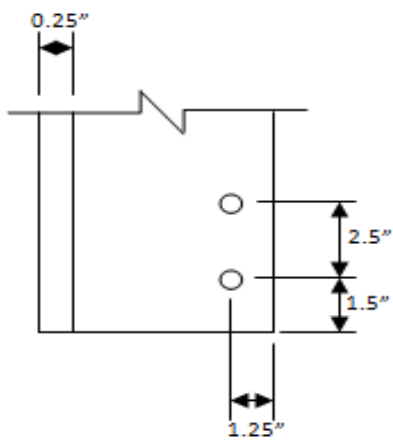


A-3

Shape: L6x4x⁵/₁₆
 Edge Distance: 3 inches
 Length of Connection: 5.5 inches
 Thickness: 0.3125 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

\bar{x} = 0.908 inches
 L = 3 inches
 A_{gt} = 0.94 in²
 A_{nt} = 0.76 in²
 A_{gv} = 1.72 in²
 A_{nv} = 1.19 in²
 U = 0.70
 A_e = 0.53 in²
 V_a = 59.95 k
 V_a = 72.27 k **governs**
 experimental failure load = 112.4 k
 PF = 1.56 **CONSERVATIVE**

Outstanding Leg= 3"

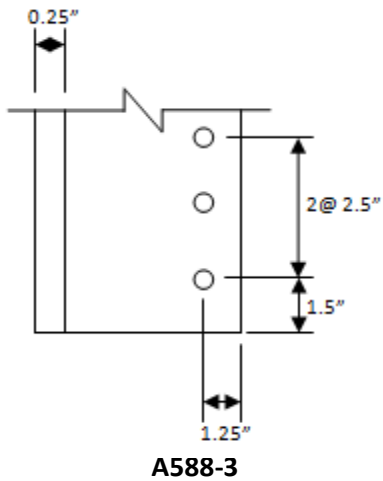


A588-1

Edge Distance: 1.25 inches
 Length of Connection: 4 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 0.736 inches
 L = 2.5 inches
 A_{gt} = 0.31 in²
 A_{nt} = 0.20 in²
 A_{gv} = 1.00 in²
 A_{nv} = 0.67 in²
 U = 0.71
 A_e = 0.14 in²
 V_a = 34.24 k
 V_a = 38.25 k **governs**
 experimental failure load = 52 k
 PF = 1.36 **CONSERVATIVE**

Outstanding Leg= 3"

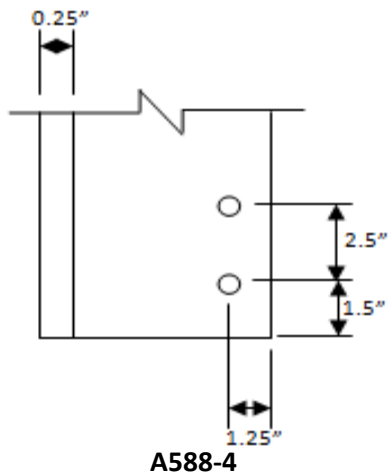


Edge Distance: 1.25 inches
 Length of Connection: 6.5 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 0.736 inches
 L = 5 inches
 A_{gt} = 0.31 in²
 A_{nt} = 0.20 in²
 A_{gv} = 1.63 in²
 A_{nv} = 1.08 in²
 U = 0.85
 A_e = 0.17 in²
 V_a = 52.56 k
 V_a = 57.41 k **governs**
 PF = 1.24 **CONSERVATIVE**

experimental failure load=

Outstanding Leg= 4"



Edge Distance: 1.25 inches
 Length of Connection: 4 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 1.09 inches

L = 2.5 inches

A_{gt} = 0.31 in²

A_{nt} = 0.20 in²

A_{gv} = 1.00 in²

A_{nv} = 0.67 in²

U = 0.56

A_e = 0.11 in²

V_a = 33.03 k

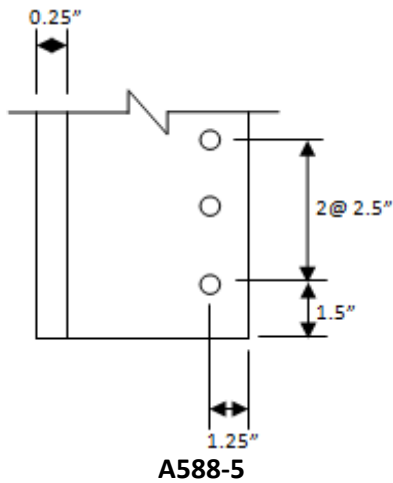
V_a = 36.24 k **governs**

48.5 k

PF = 1.34 **CONSERVATIVE**

experimental failure load =

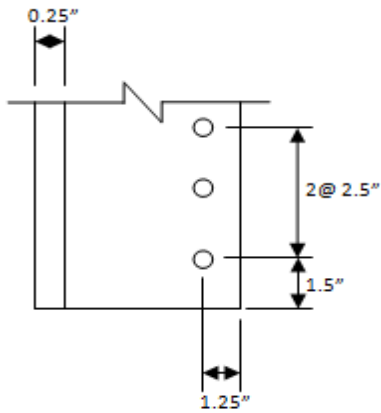
Outstanding Leg= 4"



Edge Distance: 1.25 inches
 Length of Connection: 6.5 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 1.09 inches
 L = 5 inches
 A_{gt} = 0.31 in²
 A_{nt} = 0.20 in²
 A_{gv} = 1.63 in²
 A_{nv} = 1.08 in²
 U = 0.78
 A_e = 0.16 in²
 V_a = 51.95 k
 V_a = 56.40 k **governs**
 experimental failure load = 67.2 k
 PF = 1.19 **CONSERVATIVE**

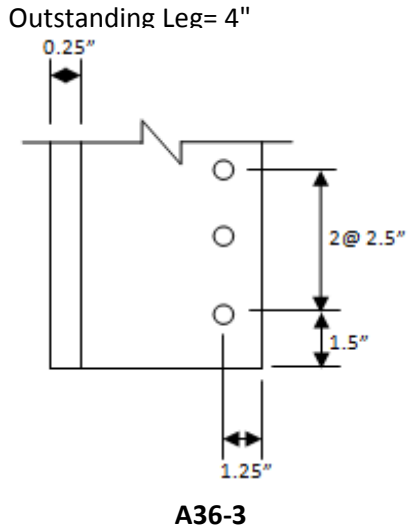
Outstanding Leg= 3"



A36-2

Edge Distance:	1.25 inches
Length of Connection:	6.5 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	0.75 inches

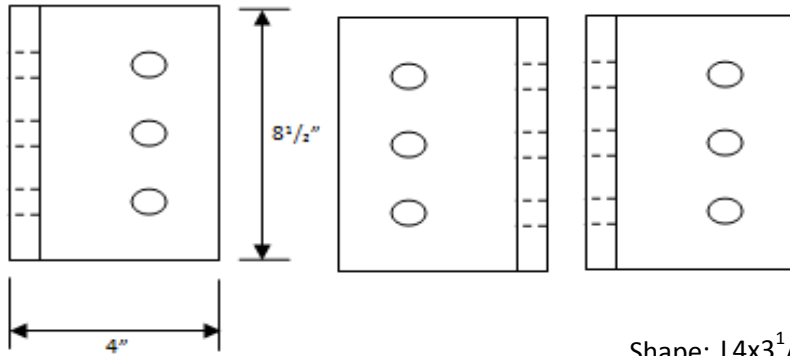
\bar{x} =	0.736	inches	
L=	5	inches	
A_{gt} =	0.31	in ²	
A_{nt} =	0.20	in ²	
A_{gv} =	1.63	in ²	
A_{nv} =	1.08	in ²	
U=	0.85		
A_e =	0.17	in ²	
V_a =	43.55	k	
V_a =	47.57	k	governs
experimental failure load=	57.8	k	
PF=	1.22	CONSERVATIVE	



Edge Distance:	1.25 inches
Length of Connection:	6.5 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	0.75 inches

\bar{x} =	1.09	inches	
L=	5	inches	
A_{gt} =	0.31	in ²	
A_{nt} =	0.20	in ²	
A_{gv} =	1.63	in ²	
A_{nv} =	1.08	in ²	
U=	0.78		
A_e =	0.16	in ²	
V_a =	43.05	k	
V_a =	46.73	k	governs
experimental failure load=	52.3	k	
PF=	1.12	CONSERVATIVE	

1953 CONNECTION



Shape: $L4 \times 3\frac{1}{2} \times \frac{3}{8}$

Edge Distance: 1.75 inches

Length of Connection: 7.25 inches

Thickness: 0.38 inches

Bolts in Tension: 0.5

Bolts in Shear: 2.5

F_u : 58 ksi

F_y : 36 ksi

Bolt Diameter: 0.75 inches

\bar{x} = 0.947 inches

L = 6 inches

A_{gt} = 0.66 in²

A_{nt} = 0.49 in²

A_{gv} = 2.72 in²

A_{nv} = 1.90 in²

U = 0.84

A_e = 0.41 in²

V_a = 160.98 k

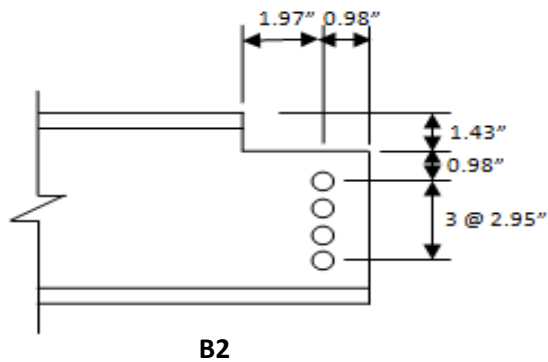
V_a = 180.21 k

governs

experimental failure load = 21.6 k

PF = 0.12 **UNCONSERVATIVE**

Appendix H: 1989 & 1979 ASD Excel Spreadsheets (Coped Beams)



B2

Shape: W16x31

Edge Distance: 0.98 inches

Length of Connection: 9.83 inches

Thickness: 0.275 inches

Bolts in Tension: 3.5

Bolts in Shear: 0.5

F_u : 74.4 ksi

F_y : 53.2 ksi

Bolt Diameter: 0.75 inches

\bar{x} = 0 inches

L = 8.85 inches

A_{gt} = 0.27 in²

A_{nt} = -0.57 in²

A_{gv} = 2.70 in²

A_{nv} = 2.58 in²

U = 1.00

A_e = -0.57 in²

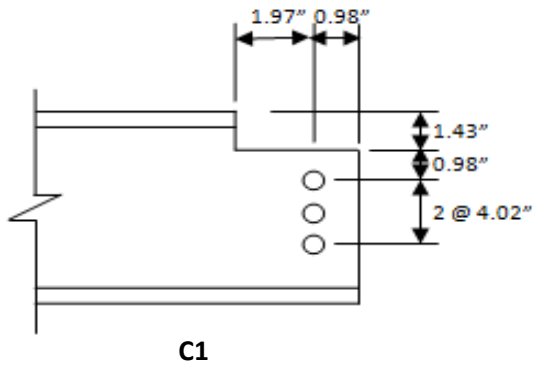
V_a = 89.74 k

governs

V_a = 72.69 k

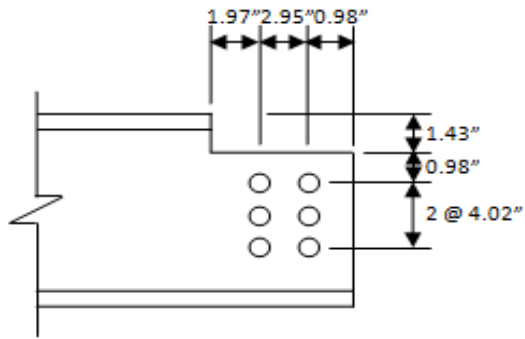
experimental failure load = 106.7842 k

PF = 1.19 **CONSERVATIVE**



Shape: W16x31
 Edge Distance: 0.98 inches
 Length of Connection: 9.02 inches
 Thickness: 0.275 inches
 # Bolts in Tension: 3.5
 # Bolts in Shear: 0.5
 F_u : 74.8 ksi
 F_y : 53.1 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 0 inches
 L = 8.04 inches
 A_{gt} = 0.27 in²
 A_{nt} = -0.57 in²
 A_{gv} = 2.48 in²
 A_{nv} = 2.36 in²
 U = 1.00
 A_e = -0.57 in²
 V_a = 80.22 k **governs**
 V_a = 63.09 k
 experimental failure load = 90.3732 k
 PF = 1.13 **CONSERVATIVE**

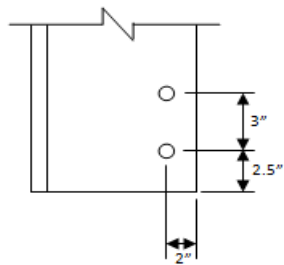


C2

Shape: W16x31
 Edge Distance: 0.98 inches
 Length of C: 9.02 inches
 Thickness: 0.275 inches
 # Bolts in Tension: 3.5
 # Bolts in Shear: 0.5
 F_u : 74.8 ksi
 F_y : 53.1 ksi
 Bolt Diameter: 0.75 inches

e = 0 inches
 L = 8.04 inches
 A_{gt} = 0.27 in²
 A_{nt} = -0.57 in²
 A_{gv} = 2.48 in²
 A_{nv} = 2.36 in²
 U = 1.00
 A_e = -0.57 in²
 V_a = 80.22 k **governs**
 V_a = 63.09 k
 experimental failure load = 120.7224 k
 PF = 1.50 **CONSERVATIVE**

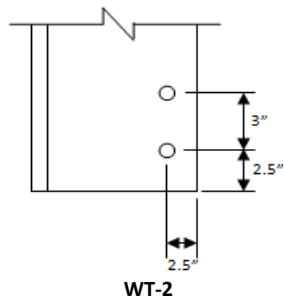
Appendix I: 1989 & 1979 ASD Excel Spreadsheets (Tees)



WT-1

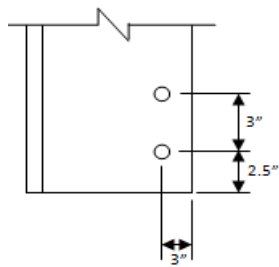
Shape:	WT7x11	
Edge Distance:	2 inches	
Length of Connection:	5.5 inches	
Thickness:	0.23 inches	
# Bolts in Tension:	0.5	
# Bolts in Shear:	1.5	
F_u :	58 ksi	
F_y :	36 ksi	
Bolt Diameter:	1 inches	
e =	0	inches
L =	3	inches
A_{gt} =	0.46	in ²
A_{nt} =	0.33	in ²
A_{gv} =	1.27	in ²
A_{nv} =	0.88	in ²
U =	1.00	
A_e =	0.33	in ²
V_a =	42.02	k
V_n =	49.69	k
experimental failure load =	59.5	k
PF =	1.20	CONSERVATIVE

governs



Shape: WT7x11
 Edge Distance: 2.5 inches
 Length of Connection: 5.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

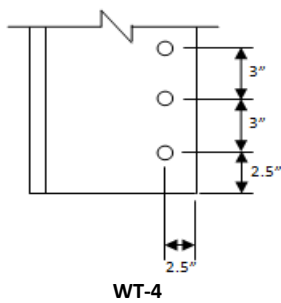
e = 0 inches
 L = 3 inches
 A_{gt} = 0.58 in²
 A_{nt} = 0.45 in²
 A_{gv} = 1.27 in²
 A_{nv} = 0.88 in²
 U = 1.00
 A_e = 0.45 in²
 V_a = 46.02 k
 V_n = 56.36 k **governs**
 experimental failure load = 70.1 k
 PF = 1.24 **CONSERVATIVE**



WT-3

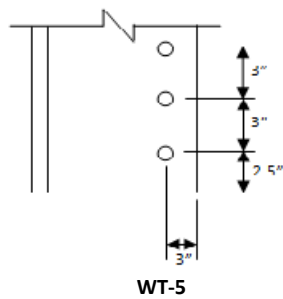
Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

e	=	0	inches	
L	=	3	inches	
A_{gt}	=	0.69	in ²	
A_{nt}	=	0.56	in ²	
A_{gv}	=	1.27	in ²	
A_{nv}	=	0.88	in ²	
U	=	1.00		
A_e	=	0.56	in ²	
V_a	=	50.03	k	
V_s	=	63.03	k	governs
experimental failure load	=	77.6	k	
PF	=	1.23		CONSERVATIVE



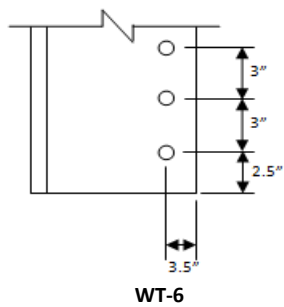
Shape: WT7x11
 Edge Distance: 2.5 inches
 Length of Connection: 8.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

A_e = 0 inches
 L = 6 inches
 A_{gt} = 0.58 in²
 A_{nt} = 0.45 in²
 A_{gv} = 1.96 in²
 A_{nv} = 1.31 in²
 U = 1.00
 A_e = 0.45 in²
 V_d = 61.03 k
 V_a = 71.37 k **governs**
 experimental failure load = 85.2 k
 PF = 1.19 **CONSERVATIVE**



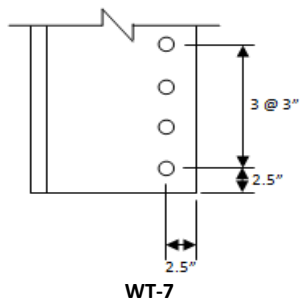
Shape: WT7x11
 Edge Distance: 3 inches
 Length of Connection: 8.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

A_e = 0 inches
 L = 6 inches
 A_{gt} = 0.69 in²
 A_{nt} = 0.56 in²
 A_{gv} = 1.96 in²
 A_{nv} = 1.31 in²
 U = 1.00
 A_e = 0.56 in²
 V_d = 65.03 k
 V_d = 78.04 k **governs**
 experimental failure load = 96.1 k
 PF = 1.23 **CONSERVATIVE**



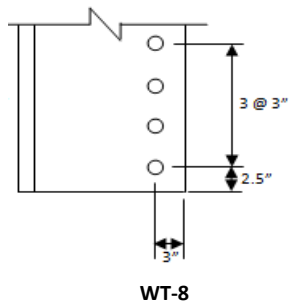
Shape: WT7x11
 Edge Distance: 3.5 inches
 Length of Connection: 8.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

A_e = 0 inches
 L = 6 inches
 A_{gt} = 0.81 in²
 A_{nt} = 0.68 in²
 A_{gv} = 1.96 in²
 A_{nv} = 1.31 in²
 U = 1.00
 A_e = 0.68 in²
 V_d = 69.03 k
 V_d = 84.71 k **governs**
 experimental failure load = 110.4 k
 PF = 1.30 **CONSERVATIVE**



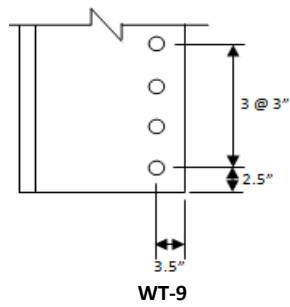
Shape: WT7x11
 Edge Distance: 2.5 inches
 Length of Connection: 11.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 3.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

\bar{x} = 0 inches
 L = 9 inches
 A_{gt} = 0.58 in²
 A_{nt} = 0.45 in²
 A_{gv} = 2.65 in²
 A_{nv} = 1.74 in²
 U = 1.00
 A_e = 0.45 in²
 V_a = 76.04 k
 V_g = 86.38 k **governs**
 experimental failure load = 101.8 k
 PF = 1.18 **CONSERVATIVE**



Shape: WT7x11
 Edge Distance: 3 inches
 Length of Connection: 11.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 3.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

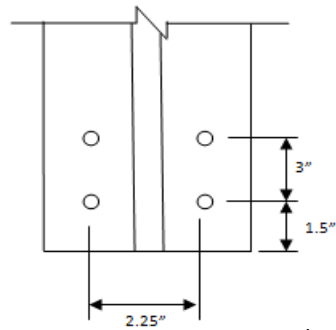
\bar{x} = 0 inches
 L = 9 inches
 A_{gt} = 0.69 in²
 A_{nt} = 0.56 in²
 A_{gv} = 2.65 in²
 A_{nv} = 1.74 in²
 U = 1.00
 A_e = 0.56 in²
 V_a = 80.04 k
 V_g = 93.05 k **governs**
 experimental failure load = 116.99 k
 PF = 1.26 **CONSERVATIVE**



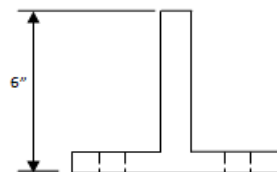
Shape: WT7x11
 Edge Distance: 3.5 inches
 Length of Connection: 11.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 3.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

\bar{x} = 0 inches
 L = 9 inches
 A_{gt} = 0.81 in²
 A_{nt} = 0.68 in²
 A_{gv} = 2.65 in²
 A_{nv} = 1.74 in²
 U = 1.00
 A_e = 0.68 in²
 V_a = 84.04 k
 V_g = 99.72 k **governs**
 experimental failure load = 129.98 k
 PF = 1.30 **CONSERVATIVE**

CHANGES IN LENGTH OF UNBOLTED LEG

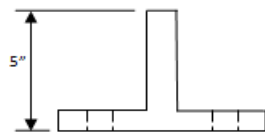


Shape: WT6x8
 Edge Distance: 0.87 inches
 Length of Connection: 4.5 inches
 Thickness: 0.265 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 65 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches



WT6x8

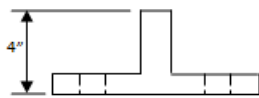
\bar{A}_F = 1.74 inches
 L = 3 inches
 A_{gt} = 0.23 in²
 A_{nt} = 0.11 in²
 A_{gv} = 1.19 in²
 A_{nv} = 0.84 in²
 U = 0.42
 A_e = 0.05 in²
 V_a = 34.82 k
 V_a = 36.07 k **governs**
 experimental failure load = 92.13 k
 PF = 2.55 **CONSERVATIVE**



WT6x8-1

Shape: WT6x8-1
 Edge Distance: 0.87 inches
 Length of Connection: 4.5 inches
 Thickness: 0.265 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 65 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{A}_F = 1.37 inches
 L = 3 inches
 A_{gt} = 0.23 in²
 A_{nt} = 0.11 in²
 A_{gv} = 1.19 in²
 A_{nv} = 0.84 in²
 U = 0.54
 A_e = 0.06 in²
 V_a = 35.37 k
 V_a = 36.99 k **governs**
 experimental failure load = 91.8 k
 PF = 2.48 **CONSERVATIVE**

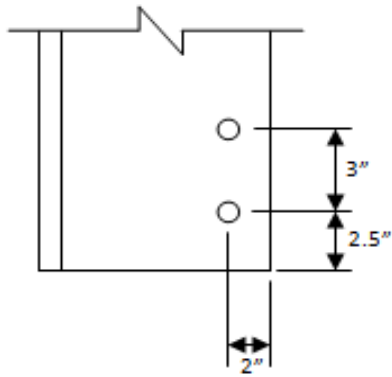


WT6x8-2

Shape: WT6x8-2
 Edge Distance: 0.87 inches
 Length of Connection: 4.5 inches
 Thickness: 0.265 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 65 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{A}_F = 1.007 inches
 L = 3 inches
 A_{gt} = 0.23 in²
 A_{nt} = 0.11 in²
 A_{gv} = 1.19 in²
 A_{nv} = 0.84 in²
 U = 0.66
 A_e = 0.08 in²
 V_a = 35.91 k
 V_a = 37.89 k **governs**
 experimental failure load = 87.6 k
 PF = 2.31 **CONSERVATIVE**

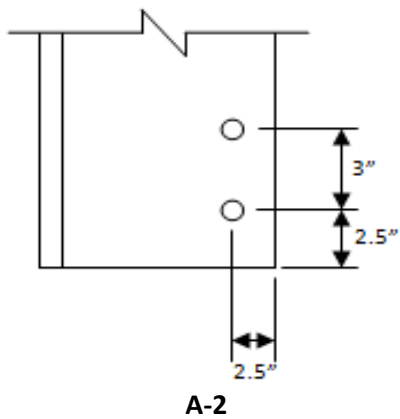
Appendix J: 1993 LRFD Excel Spreadsheets (Angles)



A-1

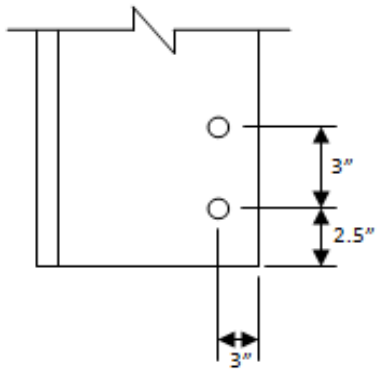
Shape:	L6x4x ⁵ / ₁₆
Edge Distance:	2 inches
Length of Connection:	5.5 inches
Thickness:	0.3125 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F _u :	58 ksi
F _y :	36 ksi
Bolt Diameter:	1 inches

\bar{x}	=	0.908	inches
L	=	3	inches
A _{gt}	=	0.63	in ²
A _{nt}	=	0.45	in ²
A _{gv}	=	1.72	in ²
A _{nv}	=	1.19	in ²
U	=	0.70	
A _e	=	0.31	in ²
F _u A _{nt}	=	18.17	k
0.6F _u A _{nv}	=	41.46	k
R _n	=	63.96	k
experimental failure load	=	81.4	k
PF	=	1.27	CONSERVATIVE



Shape: L6x4x⁵/₁₆
 Edge Distance: 2.5 inches
 Length of Connection: 5.5 inches
 Thickness: 0.3125 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

\bar{x} = 0.908 inches
 L = 3 inches
 A_{gt} = 0.78 in²
 A_{nt} = 0.61 in²
 A_{gv} = 1.72 in²
 A_{nv} = 1.19 in²
 U = 0.70
 A_e = 0.42 in²
 $F_u A_{nt}$ = 24.49 k
 $0.6 F_u A_{nv}$ = 41.46 k
 R_n = 69.59 k
 experimental failure load = 99.99 k
 PF = 1.44 **CONSERVATIVE**

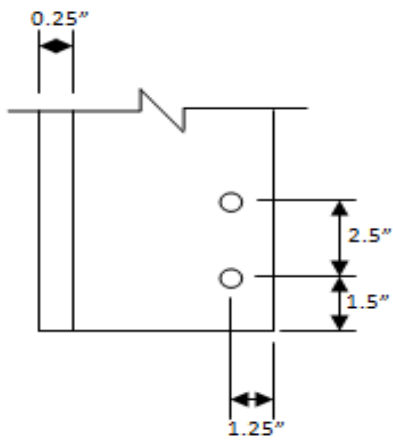


A-3

Shape:	L6x4x ⁵ / ₁₆
Edge Distance:	3 inches
Length of Connection:	5.5 inches
Thickness:	0.3125 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

\bar{x}	=	0.908	inches
L	=	3	inches
A_{gt}	=	0.94	in ²
A_{nt}	=	0.76	in ²
A_{gv}	=	1.72	in ²
A_{nv}	=	1.19	in ²
U	=	0.70	
A_e	=	0.53	in ²
$F_u A_{nt}$	=	30.81	k
$0.6 F_u A_{nv}$	=	41.46	k
R_n	=	75.21	k
experimental failure load	=	112.4	k
PF	=	1.49	CONSERVATIVE

Outstanding Leg= 3"

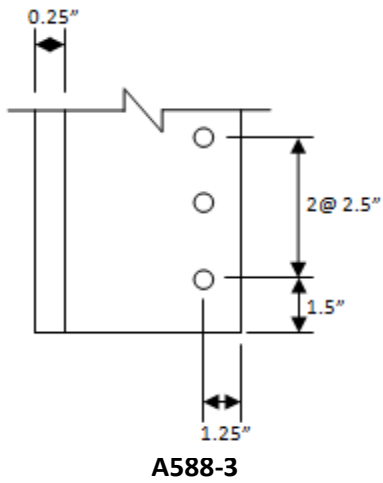


A588-1

Edge Distance:	1.25 inches
Length of Connection:	4 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	70 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

\bar{x}	=	0.736	inches
L	=	2.5	inches
A_{gt}	=	0.31	in ²
A_{nt}	=	0.20	in ²
A_{gv}	=	1.00	in ²
A_{nv}	=	0.67	in ²
U	=	0.71	
A_e	=	0.14	in ²
$F_u A_{nt}$	=	10.03	k
$0.6F_u A_{nv}$	=	28.22	k
R_n	=	43.84	k
experimental failure load	=	52	k
PF	=	1.19	CONSERVATIVE

Outstanding Leg= 3"

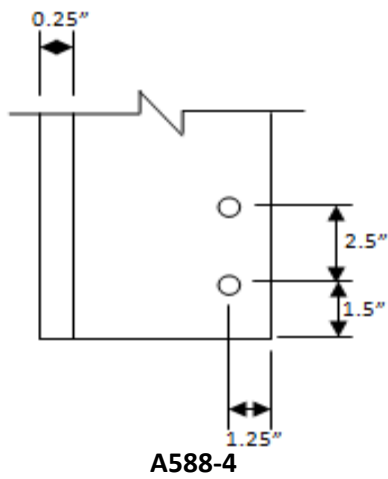


A588-3

Edge Distance:	1.25 inches
Length of Connection:	6.5 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	70 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

\bar{x} =	0.736	inches
L=	5	inches
A_{gt} =	0.31	in ²
A_{nt} =	0.20	in ²
A_{gv} =	1.63	in ²
A_{nv} =	1.08	in ²
U=	0.85	
A_e =	0.17	in ²
$F_u A_{nt}$ =	12.13	k
$0.6F_u A_{nv}$ =	45.28	k
R_n =	60.91	k
experimental failure load=	71.4	k
PF=	1.17	CONSERVATIVE

Outstanding Leg= 4"



Edge Distance: 1.25 inches
 Length of Connection: 4 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 1.09 inches

L = 2.5 inches

A_{gt} = 0.31 in²

A_{nt} = 0.20 in²

A_{gv} = 1.00 in²

A_{nv} = 0.67 in²

U = 0.56

A_e = 0.11 in²

$F_u A_{nt}$ = 8.02 k

$0.6 F_u A_{nv}$ = 28.22 k

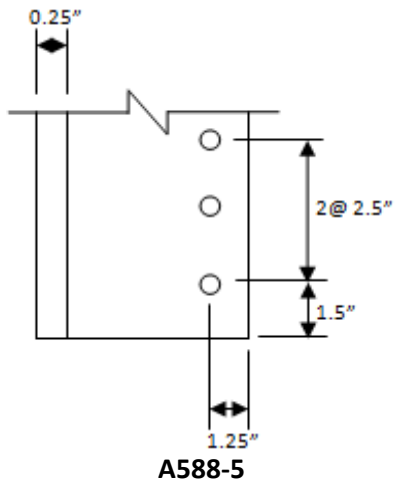
R_n = 43.84 k

experimental failure load =

48.5 k

PF = 1.11 **CONSERVATIVE**

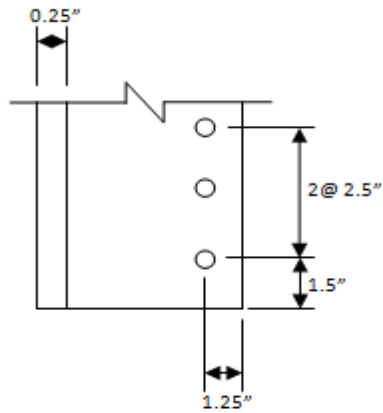
Outstanding Leg= 4"



Edge Distance: 1.25 inches
 Length of Connection: 6.5 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 1.09 inches
 L = 5 inches
 A_{gt} = 0.31 in²
 A_{nt} = 0.20 in²
 A_{gv} = 1.63 in²
 A_{nv} = 1.08 in²
 U = 0.78
 A_e = 0.16 in²
 $F_u A_{nt}$ = 11.12 k
 $0.6F_u A_{nv}$ = 45.28 k
 R_n = 60.91 k
 experimental failure load = 67.2 k
 PF = 1.10 **CONSERVATIVE**

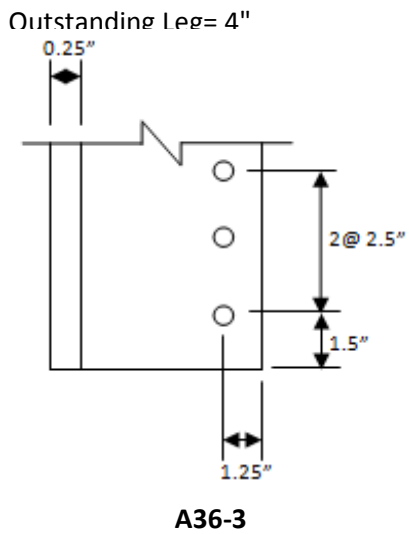
Outstanding Leg= 3"



A36-2

Edge Distance: 1.25 inches
 Length of Connection: 6.5 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 0.75 inches

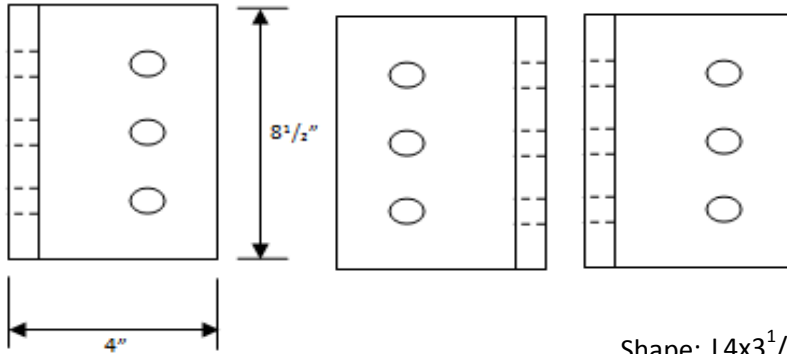
\bar{x} = 0.736 inches
 L = 5 inches
 A_{gt} = 0.31 in²
 A_{nt} = 0.20 in²
 A_{gv} = 1.63 in²
 A_{nv} = 1.08 in²
 U = 0.85
 A_e = 0.17 in²
 $F_u A_{nt}$ = 10.05 k
 $0.6 F_u A_{nv}$ = 37.52 k
 R_n = 48.77 k
 experimental failure load = 57.8 k
 PF = 1.19 **CONSERVATIVE**



Edge Distance:	1.25 inches
Length of Connection:	6.5 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	0.75 inches

\bar{x}	=	1.09	inches
L	=	5	inches
A_{gt}	=	0.31	in ²
A_{nt}	=	0.20	in ²
A_{gv}	=	1.63	in ²
A_{nv}	=	1.08	in ²
U	=	0.78	
A_e	=	0.16	in ²
$F_u A_{nt}$	=	9.21	k
$0.6F_u A_{nv}$	=	37.52	k
R_n	=	48.77	k
experimental failure load	=	52.3	k
PF	=	1.07	CONSERVATIVE

1953 CONNECTION



Shape: $L4 \times 3 \frac{1}{2} \times \frac{3}{8}$

Edge Distance: 1.75 inches

Length of Connection: 7.25 inches

Thickness: 0.38 inches

Bolts in Tension: 0.5

Bolts in Shear: 2.5

F_u : 58 ksi

F_y : 36 ksi

Bolt Diameter: 0.75 inches

\bar{x} = 0.947 inches

L = 6 inches

A_{gt} = 0.66 in²

A_{nt} = 0.49 in²

A_{gv} = 2.72 in²

A_{nv} = 1.90 in²

U = 0.84

A_e = 0.41 in²

$F_u A_{nt}$ = 24.04 k

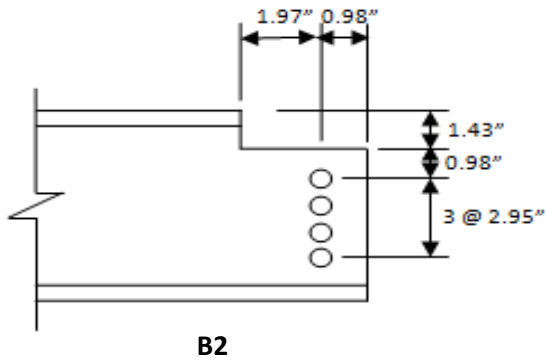
$0.6 F_u A_{nv}$ = 66.07 k

R_n = 179.38 k

experimental failure load = 21.6 k

PF = 0.12 **UNCONSERVATIVE**

Appendix K: 1993 LRFD Excel Spreadsheets (Coped Beams)



B2

Shape: W16x31

Edge Distance: 0.98 inches

Length of Connection: 9.83 inches

Thickness: 0.275 inches

Bolts in Tension: 3.5

Bolts in Shear: 0.5

F_u : 74.4 ksi

F_y : 53.2 ksi

Bolt Diameter: 0.75 inches

\bar{x} = 0 inches

L = 8.85 inches

A_{gt} = 0.27 in²

A_{nt} = -0.57 in²

A_{gv} = 2.70 in²

A_{nv} = 2.58 in²

U = 0.90

A_e = -0.52 in²

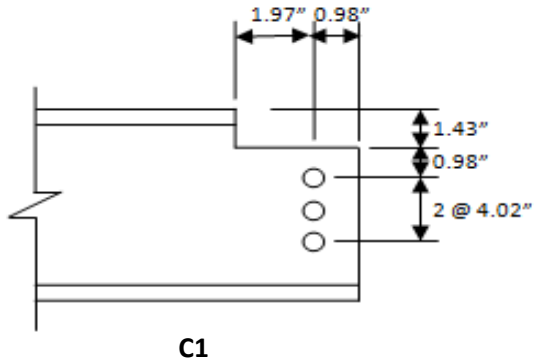
$F_u A_{nt}$ = -38.35 k

$0.6 F_u A_{nv}$ = 115.30 k

R_n = 129.64 k

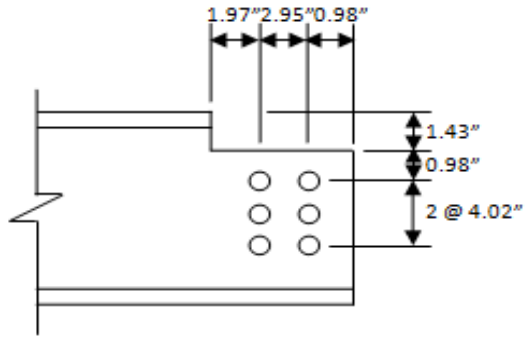
experimental failure load = 106.7842 k

PF = 0.82 **UNCONSERVATIVE**



Shape: W16x31
 Edge Distance: 0.98 inches
 Length of Connection: 9.02 inches
 Thickness: 0.275 inches
 # Bolts in Tension: 3.5
 # Bolts in Shear: 0.5
 F_u : 74.8 ksi
 F_y : 53.1 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 0 inches
 L = 8.04 inches
 A_{gt} = 0.27 in²
 A_{nt} = -0.57 in²
 A_{gv} = 2.48 in²
 A_{nv} = 2.36 in²
 U = 0.90
 A_e = -0.52 in²
 $F_u A_{nt}$ = -38.55 k
 $0.6 F_u A_{nv}$ = 105.93 k
 R_n = 120.24 k
 experimental failure load = 90.3732 k
 PF = 0.75 **UNCONSERVATIVE**

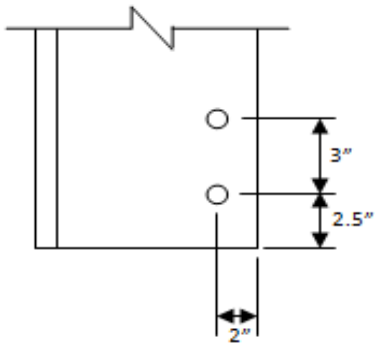


C2

Shape: W16x31
 Edge Distance: 0.98 inches
 Length of C 9.02 inches
 Thickness: 0.275 inches
 # Bolts in Tension: 3.5
 # Bolts in Shear: 0.5
 F_u : 74.8 ksi
 F_y : 53.1 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 0 inches
 L = 8.04 inches
 A_{gt} = 0.27 in²
 A_{nt} = -0.57 in²
 A_{gv} = 2.48 in²
 A_{nv} = 2.36 in²
 U = 0.90
 A_e = -0.52 in²
 $F_u A_{nt}$ = -38.55 k
 $0.6 F_u A_{nv}$ = 105.93 k
 R_n = 120.24 k
 experimental failure load = 120.7224 k
 PF = 1.00 **CONSERVATIVE**

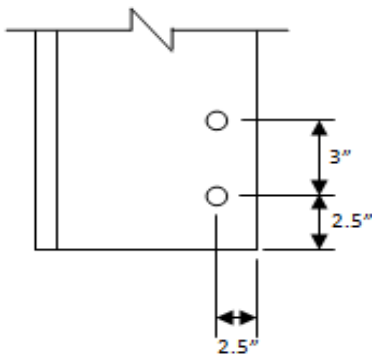
Appendix L: 1993 LRFD Excel Spreadsheets (Tees)



WT-1

Shape:	WT7x11
Edge Distance:	2 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

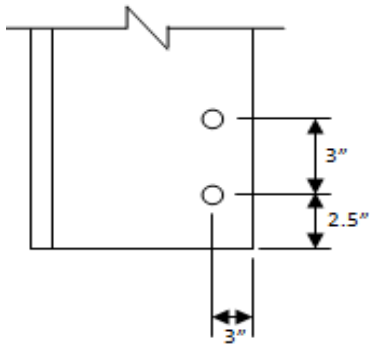
\bar{x} =	0	inches
L =	3	inches
A_{gt} =	0.46	in ²
A_{nt} =	0.33	in ²
A_{gv} =	1.27	in ²
A_{nv} =	0.88	in ²
U =	0.90	
A_e =	0.30	in ²
$F_u A_{nt}$ =	17.26	k
$0.6 F_u A_{nv}$ =	30.52	k
R_n =	47.08	k
experimental failure load =	59.5	k
PF =	1.26	CONSERVATIVE



WT-2

Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

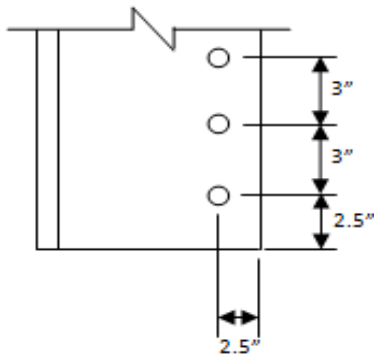
\bar{x}	=	0	inches
L	=	3	inches
A_{gt}	=	0.58	in ²
A_{nt}	=	0.45	in ²
A_{gv}	=	1.27	in ²
A_{nv}	=	0.88	in ²
U	=	0.90	
A_e	=	0.40	in ²
$F_u A_{nt}$	=	23.26	k
$0.6F_u A_{nv}$	=	30.52	k
R_n	=	51.22	k
experimental failure load	=	70.1	k
PF	=	1.37	CONSERVATIVE



WT-3

Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

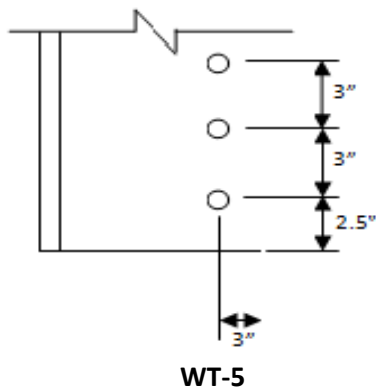
\bar{x}	=	0	inches
L	=	3	inches
A_{gt}	=	0.69	in ²
A_{nt}	=	0.56	in ²
A_{gv}	=	1.27	in ²
A_{nv}	=	0.88	in ²
U	=	0.90	
A_e	=	0.50	in ²
$F_u A_{nt}$	=	29.26	k
$0.6F_u A_{nv}$	=	30.52	k
R_n	=	55.36	k
experimental failure load	=	77.6	k
PF	=	1.40	CONSERVATIVE



WT-4

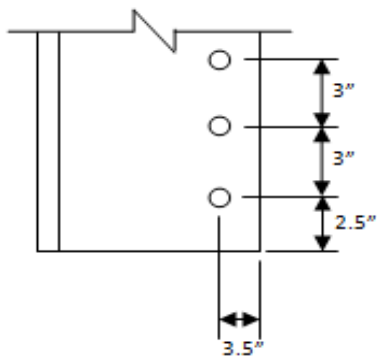
Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

\bar{x}	=	0	inches
L	=	6	inches
A_{gt}	=	0.58	in ²
A_{nt}	=	0.45	in ²
A_{gv}	=	1.96	in ²
A_{nv}	=	1.31	in ²
U	=	0.90	
A_e	=	0.40	in ²
$F_u A_{nt}$	=	23.26	k
$0.6 F_u A_{nv}$	=	45.52	k
R_n	=	66.22	k
experimental failure load	=	85.2	k
PF	=	1.29	CONSERVATIVE



Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

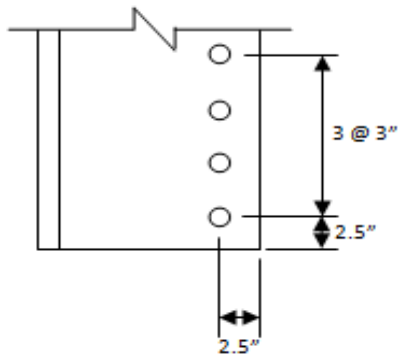
\bar{x}	=	0	inches
L	=	6	inches
A_{gt}	=	0.69	in ²
A_{nt}	=	0.56	in ²
A_{gv}	=	1.96	in ²
A_{nv}	=	1.31	in ²
U	=	0.90	
A_e	=	0.50	in ²
$F_u A_{nt}$	=	29.26	k
$0.6 F_u A_{nv}$	=	45.52	k
R_n	=	70.36	k
experimental failure load	=	96.1	k
PF	=	1.37	CONSERVATIVE



WT-6

Shape:	WT7x11
Edge Distance:	3.5 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

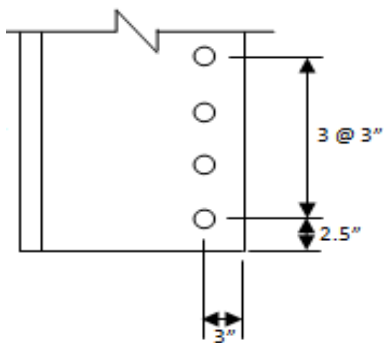
\bar{x}	=	0	inches
L	=	6	inches
A_{gt}	=	0.81	in ²
A_{nt}	=	0.68	in ²
A_{gv}	=	1.96	in ²
A_{nv}	=	1.31	in ²
U	=	0.90	
A_e	=	0.61	in ²
$F_u A_{nt}$	=	35.27	k
$0.6 F_u A_{nv}$	=	45.52	k
R_n	=	74.50	k
experimental failure load	=	110.4	k
PF	=	1.48	CONSERVATIVE



WT-7

Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	11.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	3.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

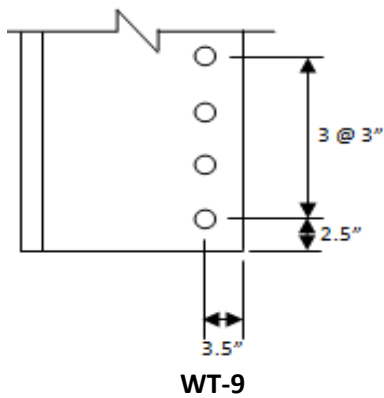
\bar{x}	=	0	inches
L	=	9	inches
A_{gt}	=	0.58	in ²
A_{nt}	=	0.45	in ²
A_{gv}	=	2.65	in ²
A_{nv}	=	1.74	in ²
U	=	0.90	
A_e	=	0.40	in ²
$F_u A_{nt}$	=	23.26	k
$0.6 F_u A_{nv}$	=	60.53	k
R_n	=	81.23	k
experimental failure load	=	101.8	k
PF	=	1.25	CONSERVATIVE



WT-8

Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	11.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	3.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

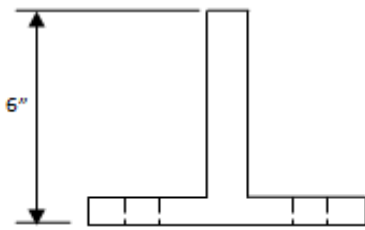
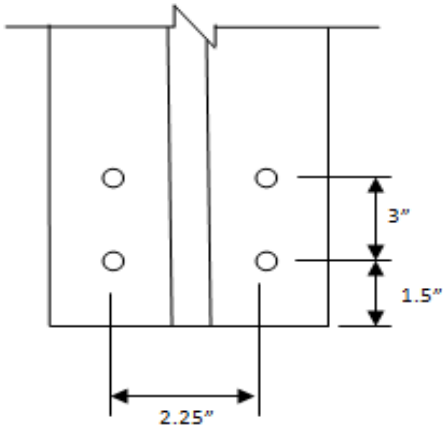
\bar{x}	=	0	inches
L	=	9	inches
A_{gt}	=	0.69	in ²
A_{nt}	=	0.56	in ²
A_{gv}	=	2.65	in ²
A_{nv}	=	1.74	in ²
U	=	0.90	
A_e	=	0.50	in ²
$F_u A_{nt}$	=	29.26	k
$0.6 F_u A_{nv}$	=	60.53	k
R_n	=	85.37	k
experimental failure load	=	116.99	k
PF	=	1.37	CONSERVATIVE



Shape: WT7x11
 Edge Distance: 3.5 inches
 Length of Connection: 11.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 3.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

\bar{x} = 0 inches
 L = 9 inches
 A_{gt} = 0.81 in²
 A_{nt} = 0.68 in²
 A_{gv} = 2.65 in²
 A_{nv} = 1.74 in²
 U = 0.90
 A_e = 0.61 in²
 $F_u A_{nt}$ = 35.27 k
 $0.6 F_u A_{nv}$ = 60.53 k
 R_n = 89.51 k
 experimental failure load = 129.98 k
 PF = 1.45 **CONSERVATIVE**

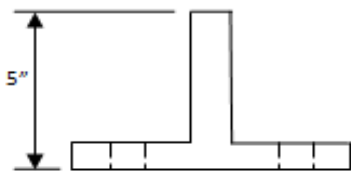
CHANGES IN LENGTH OF UNBOLTED LEG



WT6x8

Shape:	WT6x8
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

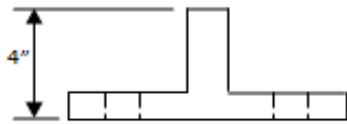
\bar{x}	=	1.74	inches
L	=	3	inches
A_{gt}	=	0.23	in ²
A_{nt}	=	0.11	in ²
A_{gv}	=	1.19	in ²
A_{nv}	=	0.84	in ²
U	=	0.42	
A_e	=	0.05	in ²
$F_u A_{nt}$	=	3.13	k
$0.6 F_u A_{nv}$	=	32.94	k
R_n	=	44.47	k
experimental failure load	=	92.13	k
PF	=	2.07	CONSERVATIVE



WT6x8-1

Shape:	WT6x8-1
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

\bar{x}	=	1.37	inches
L	=	3	inches
A_{gt}	=	0.23	in ²
A_{nt}	=	0.11	in ²
A_{gv}	=	1.19	in ²
A_{nv}	=	0.84	in ²
U	=	0.54	
A_e	=	0.06	in ²
$F_u A_{nt}$	=	4.05	k
$0.6F_u A_{nv}$	=	32.94	k
R_n	=	44.47	k
experimental failure load	=	91.8	k
PF	=	2.06	CONSERVATIVE

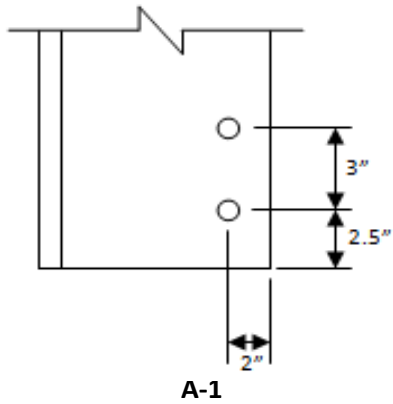


WT6x8-2

Shape:	WT6x8-2
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

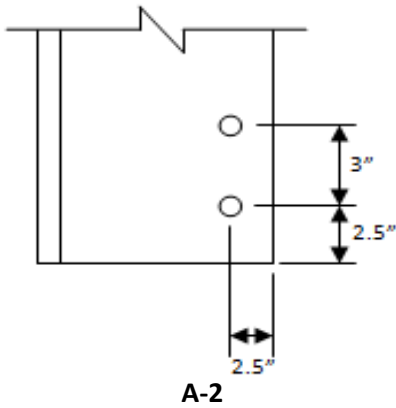
\bar{x}	=	1.007	inches
L	=	3	inches
A_{gt}	=	0.23	in ²
A_{nt}	=	0.11	in ²
A_{gv}	=	1.19	in ²
A_{nv}	=	0.84	in ²
U	=	0.66	
A_e	=	0.08	in ²
$F_u A_{nt}$	=	4.95	k
$0.6F_u A_{nv}$	=	32.94	k
R_n	=	44.47	k
experimental failure load	=	87.6	k
PF	=	1.97	CONSERVATIVE

Appendix M: 1999 LRFD Excel Spreadsheets (Angles)



Shape:	L6x4x ⁵ / ₁₆
Edge Distance:	2 inches
Length of Connection:	5.5 inches
Thickness:	0.3125 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F _u :	58 ksi
F _y :	36 ksi
Bolt Diameter:	1 inches

\bar{x} =	0.908	inches	
L =	3	inches	
A _{gt} =	0.63	in ²	
A _{nt} =	0.45	in ²	
A _{gv} =	1.72	in ²	
A _{nv} =	1.19	in ²	
U =	0.70		
A _e =	0.31	in ²	
F _u A _{nt} =	18.17	k	
0.6F _u A _{nv} =	41.46	k	
R _n =	63.96	k	
R _n =	59.63	k	GOVERNS
experimental failure load =	81.4	k	
PF =	1.37	CONSERVATIVE	



Shape: $L6x4x^{5/16}$
 Edge Distance: 2.5 inches
 Length of Connection: 5.5 inches
 Thickness: 0.3125 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

$$\bar{x} = 0.908 \text{ inches}$$

$$L = 3 \text{ inches}$$

$$A_{gt} = 0.78 \text{ in}^2$$

$$A_{nt} = 0.61 \text{ in}^2$$

$$A_{gv} = 1.72 \text{ in}^2$$

$$A_{nv} = 1.19 \text{ in}^2$$

$$U = 0.70$$

$$A_e = 0.42 \text{ in}^2$$

$$F_u A_{nt} = 24.49 \text{ k}$$

$$0.6 F_u A_{nv} = 41.46 \text{ k}$$

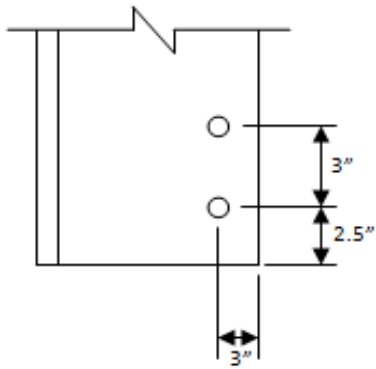
$$R_n = 69.59 \text{ k}$$

$$R_n = 65.95 \text{ k}$$

$$\text{experimental failure load} = 99.99 \text{ k}$$

$$PF = 1.52 \text{ CONSERVATIVE}$$

GOVERNS



A-3

Shape: $L6x4x^{5/16}$

Edge Distance: 3 inches

Length of Connection: 5.5 inches

Thickness: 0.3125 inches

Bolts in Tension: 0.5

Bolts in Shear: 1.5

F_u : 58 ksi

F_y : 36 ksi

Bolt Diameter: 1 inches

\bar{x} = 0.908 inches

L = 3 inches

A_{gt} = 0.94 in²

A_{nt} = 0.76 in²

A_{gv} = 1.72 in²

A_{nv} = 1.19 in²

U = 0.70

A_e = 0.53 in²

$F_u A_{nt}$ = 30.81 k

$0.6F_u A_{nv}$ = 41.46 k

R_n = 75.21 k

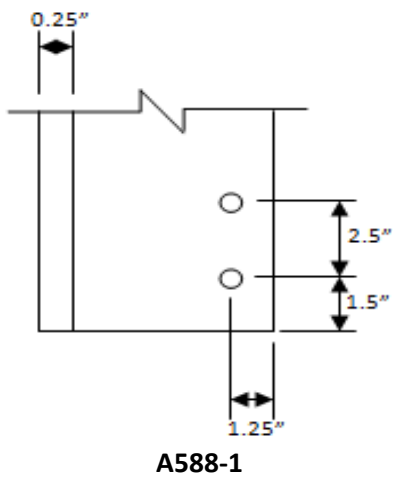
R_n = 72.27 k

GOVERNS

experimental failure load = 112.4 k

PF = 1.56 **CONSERVATIVE**

Outstanding Leg= 3"



Edge Distance:	1.25 inches
Length of Connection:	4 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	70 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

\bar{x}	=	0.736	inches
L	=	2.5	inches
A_{gt}	=	0.31	in ²
A_{nt}	=	0.20	in ²
A_{gv}	=	1.00	in ²
A_{nv}	=	0.67	in ²
U	=	0.71	
A_e	=	0.14	in ²

$F_u A_{nt} = 10.03$ k

$0.6 F_u A_{nv} = 28.22$ k

$R_n = 43.84$ k

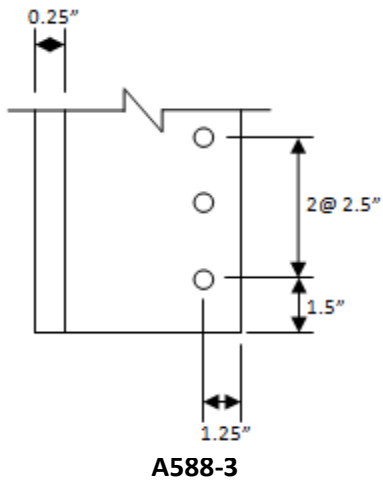
$R_n = 38.25$ k

GOVERNS

experimental failure load = 52 k

PF = 1.36 **CONSERVATIVE**

Outstanding Leg= 3"

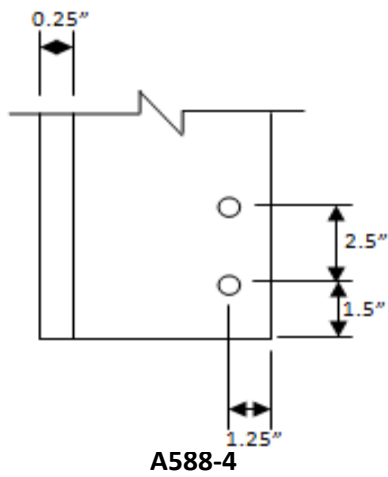


Edge Distance:	1.25 inches
Length of Connection:	6.5 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	70 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

\bar{x} =	0.736	inches	
L=	5	inches	
A_{gt} =	0.31	in ²	
A_{nt} =	0.20	in ²	
A_{gv} =	1.63	in ²	
A_{nv} =	1.08	in ²	
U=	0.85		
A_e =	0.17	in ²	
$F_u A_{nt}$ =	12.13	k	
$0.6F_u A_{nv}$ =	45.28	k	
R_n =	60.91	k	
R_n =	57.41	k	GOVERNS
	71.4	k	
PF=	1.24	CONSERVATIVE	

experimental failure load=

Outstanding Leg= 4"



Edge Distance: 1.25 inches
 Length of Connection: 4 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

\bar{x} = 1.09 inches

L= 2.5 inches

A_{gt} = 0.31 in²

A_{nt} = 0.20 in²

A_{gv} = 1.00 in²

A_{nv} = 0.67 in²

U= 0.56

A_e = 0.11 in²

$F_u A_{nt}$ = 8.02 k

$0.6F_u A_{nv}$ = 28.22 k

R_n = 43.84 k

R_n = 36.24 k

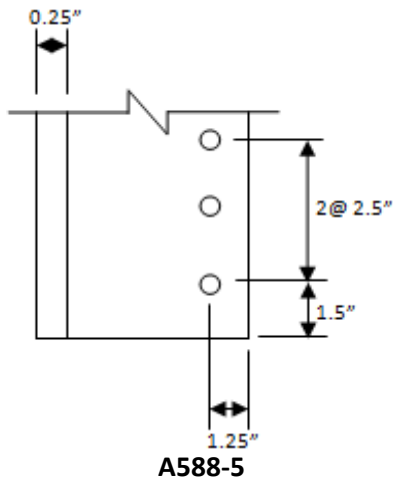
GOVERNS

experimental failure load=

48.5 k

PF= 1.34 **CONSERVATIVE**

Outstanding Leg= 4"



Edge Distance: 1.25 inches
 Length of Connection: 6.5 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 70 ksi
 F_y : 50 ksi
 Bolt Diameter: 0.75 inches

$$\bar{u} = 1.09 \text{ inches}$$

$$L = 5 \text{ inches}$$

$$A_{gt} = 0.31 \text{ in}^2$$

$$A_{nt} = 0.20 \text{ in}^2$$

$$A_{gv} = 1.63 \text{ in}^2$$

$$A_{nv} = 1.08 \text{ in}^2$$

$$U = 0.78$$

$$A_e = 0.16 \text{ in}^2$$

$$F_u A_{nt} = 11.12 \text{ k}$$

$$0.6 F_u A_{nv} = 45.28 \text{ k}$$

$$R_n = 60.91 \text{ k}$$

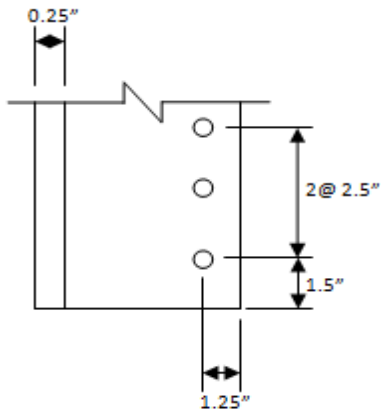
$$R_n = 56.40 \text{ k}$$

experimental failure load = 67.2 k

$$PF = 1.19 \text{ CONSERVATIVE}$$

GOVERNS

Outstanding Leg= 3"



A36-2

Edge Distance: 1.25 inches
 Length of Connection: 6.5 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 0.75 inches

$\bar{x} = 0.736$ inches

$L = 5$ inches

$A_{gt} = 0.31$ in²

$A_{nt} = 0.20$ in²

$A_{gv} = 1.63$ in²

$A_{nv} = 1.08$ in²

$U = 0.85$

$A_e = 0.17$ in²

$F_u A_{nt} = 10.05$ k

$0.6 F_u A_{nv} = 37.52$ k

$R_n = 48.77$ k

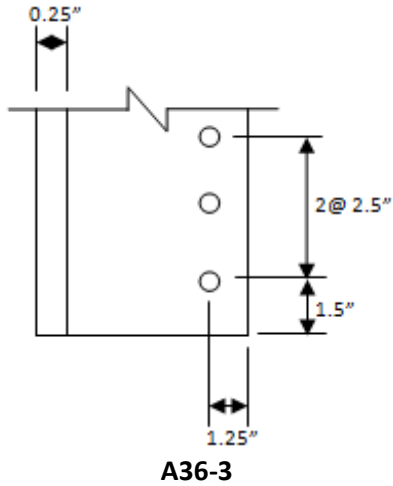
$R_n = 47.57$ k

experimental failure load = 57.8 k

PF = 1.22 **CONSERVATIVE**

GOVERNS

Outstanding Leg= 4"



Edge Distance:	1.25 inches
Length of Connection:	6.5 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	0.75 inches

$$\bar{x} = 1.09 \text{ inches}$$

$$L = 5 \text{ inches}$$

$$A_{gt} = 0.31 \text{ in}^2$$

$$A_{nt} = 0.20 \text{ in}^2$$

$$A_{gv} = 1.63 \text{ in}^2$$

$$A_{nv} = 1.08 \text{ in}^2$$

$$U = 0.78$$

$$A_e = 0.16 \text{ in}^2$$

$$F_u A_{nt} = 9.21 \text{ k}$$

$$0.6F_u A_{nv} = 37.52 \text{ k}$$

$$R_n = 48.77 \text{ k}$$

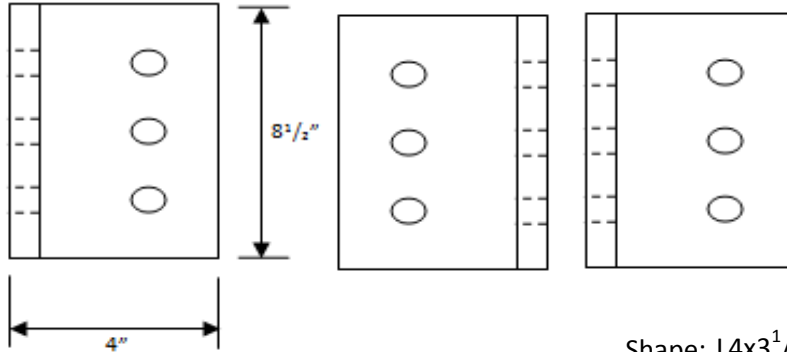
$$R_n = 46.73 \text{ k}$$

GOVERNS

$$\text{experimental failure load} = 52.3 \text{ k}$$

$$PF = 1.12 \text{ CONSERVATIVE}$$

1953 CONNECTION



Shape: L4x3¹/₂x³/₈

Edge Distance: 1.75 inches

Length of Connection: 7.25 inches

Thickness: 0.38 inches

Bolts in Tension: 0.5

Bolts in Shear: 2.5

F_u : 58 ksi

F_y : 36 ksi

Bolt Diameter: 0.75 inches

\bar{x} = 0.947 inches

L = 6 inches

A_{gt} = 0.66 in²

A_{nt} = 0.49 in²

A_{gv} = 2.72 in²

A_{nv} = 1.90 in²

U = 0.84

A_e = 0.41 in²

$F_u A_{nt}$ = 24.04 k

$0.6F_u A_{nv}$ = 66.07 k

R_n = 179.38 k

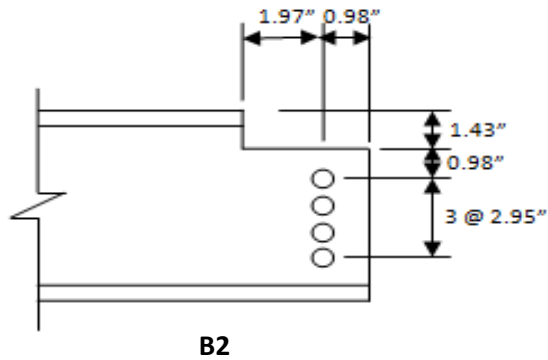
GOVERNS

R_n = 180.21 k

experimental failure load = 21.6 k

PF = 0.12 **UNCONSERVATIVE**

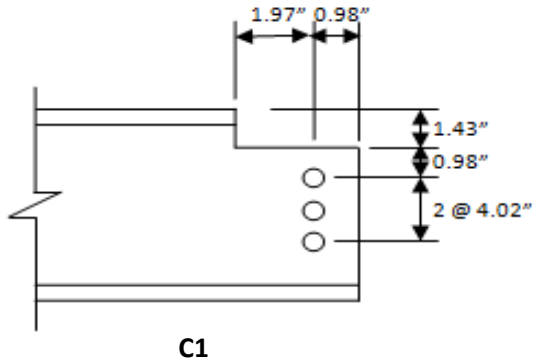
Appendix N: 1999 LRFD Excel Spreadsheets (Coped Beams)



Shape:	W16x31
Edge Distance:	0.98 inches
Length of Connection:	9.83 inches
Thickness:	0.275 inches
# Bolts in Tension:	3.5
# Bolts in Shear:	0.5
F_u :	74.4 ksi
F_y :	53.2 ksi
Bolt Diameter:	0.75 inches

\bar{x}	=	0	inches
L	=	8.85	inches
A_{gt}	=	0.27	in ²
A_{nt}	=	-0.57	in ²
A_{gv}	=	2.70	in ²
A_{nv}	=	2.58	in ²
U	=	0.90	
A_e	=	-0.52	in ²
$F_u A_{nt}$	=	-38.35	k
$0.6F_u A_{nv}$	=	115.30	k
R_n	=	129.64	k
R_n	=	76.96	k
experimental failure load	=	106.7842	k
PF	=	1.39	CONSERVATIVE

GOVERNS



Shape: W16x31
 Edge Distance: 0.98 inches
 Length of Connection: 9.02 inches
 Thickness: 0.275 inches
 # Bolts in Tension: 3.5
 # Bolts in Shear: 0.5
 F_u : 74.8 ksi
 F_y : 53.1 ksi
 Bolt Diameter: 0.75 inches

$\bar{x} = 0$ inches

$L = 8.04$ inches

$A_{gt} = 0.27$ in²

$A_{nt} = -0.57$ in²

$A_{gv} = 2.48$ in²

$A_{nv} = 2.36$ in²

$U = 0.90$

$A_e = -0.52$ in²

$F_u A_{nt} = -38.55$ k

$0.6 F_u A_{nv} = 105.93$ k

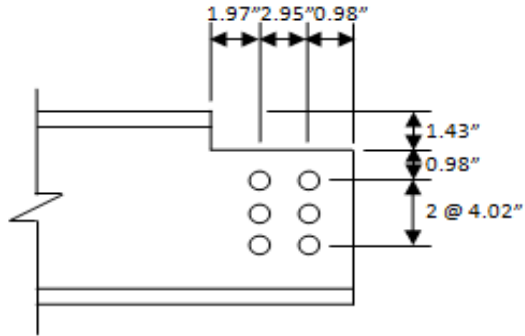
$R_n = 120.24$ k

$R_n = 67.37$ k

GOVERNS

experimental failure load = 90.3732 k

PF = 1.34 **CONSERVATIVE**

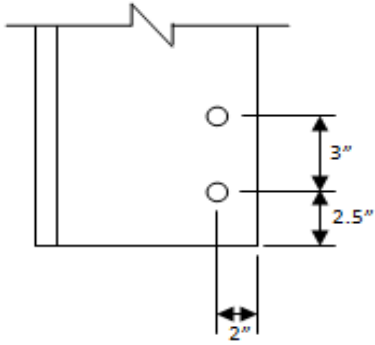


C2

Shape:	W16x31
Edge Distance:	0.98 inches
Length of C	9.02 inches
Thickness:	0.275 inches
# Bolts in Tension:	3.5
# Bolts in Shear:	0.5
F_u :	74.8 ksi
F_y :	53.1 ksi
Bolt Diameter:	0.75 inches

\bar{x}	0	inches
L	8.04	inches
A_{gt}	0.27	in^2
A_{nt}	-0.57	in^2
A_{gv}	2.48	in^2
A_{nv}	2.36	in^2
U	0.90	
A_e	-0.52	in^2
$F_u A_{nt}$	-38.55	k
$0.6 F_u A_{nv}$	105.93	k
R_n	120.24	k
R_n	67.37	k
		GOVERNS
experimental failure load	120.7224	k
PF	1.79	CONSERVATIVE

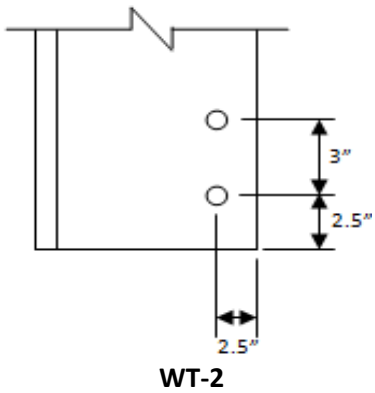
Appendix O: 1999 LRFD Excel Spreadsheets (Tees)



WT-1

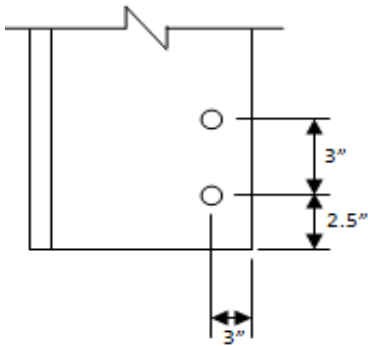
Shape:	WT7x11
Edge Distance:	2 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

\bar{x}	=	0	inches	
L	=	3	inches	
A_{gt}	=	0.46	in ²	
A_{nt}	=	0.33	in ²	
A_{gv}	=	1.27	in ²	
A_{nv}	=	0.88	in ²	
U	=	0.90		
A_e	=	0.30	in ²	
$F_u A_{nt}$	=	17.26	k	
$0.6 F_u A_{nv}$	=	30.52	k	
R_n	=	47.08	k	GOVERNS
R_n	=	47.77	k	
experimental failure load	=	59.5	k	
PF	=	1.26	CONSERVATIVE	



Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

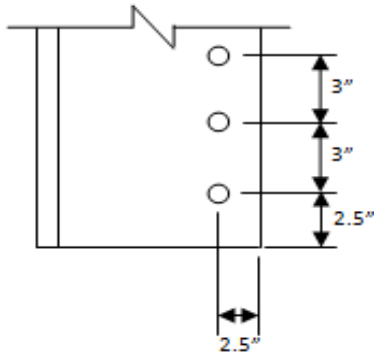
\bar{x}	=	0	inches	
L	=	3	inches	
A_{gt}	=	0.58	in ²	
A_{nt}	=	0.45	in ²	
A_{gv}	=	1.27	in ²	
A_{nv}	=	0.88	in ²	
U	=	0.90		
A_e	=	0.40	in ²	
$F_u A_{nt}$	=	23.26	k	
$0.6 F_u A_{nv}$	=	30.52	k	
R_n	=	51.22	k	GOVERNS
R_n	=	53.78	k	
experimental failure load	=	70.1	k	
PF	=	1.37	CONSERVATIVE	



WT-3

Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

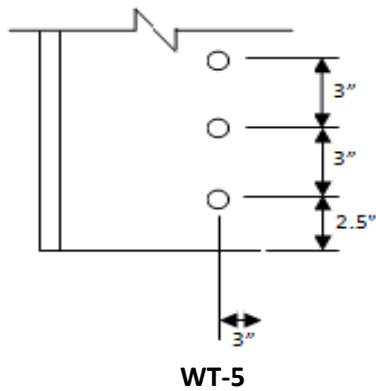
\bar{x}	=	0	inches	
L	=	3	inches	
A_{gt}	=	0.69	in ²	
A_{nt}	=	0.56	in ²	
A_{gv}	=	1.27	in ²	
A_{nv}	=	0.88	in ²	
U	=	0.90		
A_e	=	0.50	in ²	
$F_u A_{nt}$	=	29.26	k	
$0.6 F_u A_{nv}$	=	30.52	k	
R_n	=	55.36	k	GOVERNS
R_n	=	59.78	k	
experimental failure load	=	77.6	k	
PF	=	1.40	CONSERVATIVE	



WT-4

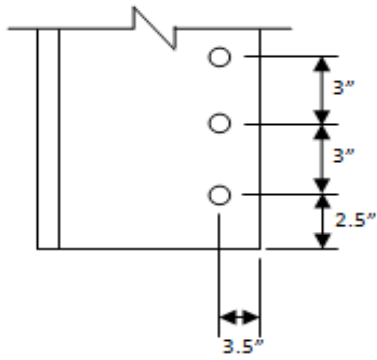
Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

\bar{x}	=	0	inches	
L	=	6	inches	
A_{gt}	=	0.58	in ²	
A_{nt}	=	0.45	in ²	
A_{gv}	=	1.96	in ²	
A_{nv}	=	1.31	in ²	
U	=	0.90		
A_e	=	0.40	in ²	
$F_u A_{nt}$	=	23.26	k	
$0.6F_u A_{nv}$	=	45.52	k	
R_n	=	66.22	k	GOVERNS
R_n	=	68.78	k	
experimental failure load	=	85.2	k	
PF	=	1.29	CONSERVATIVE	



Shape: WT7x11
 Edge Distance: 3 inches
 Length of Connection: 8.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

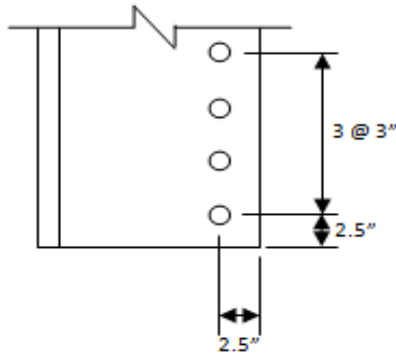
\bar{x} = 0 inches
 L = 6 inches
 A_{gt} = 0.69 in²
 A_{nt} = 0.56 in²
 A_{gv} = 1.96 in²
 A_{nv} = 1.31 in²
 U = 0.90
 A_e = 0.50 in²
 $F_u A_{nt}$ = 29.26 k
 $0.6F_u A_{nv}$ = 45.52 k
 R_n = 70.36 k **GOVERNS**
 R_n = 74.79 k
 experimental failure load = 96.1 k
 PF = 1.37 **CONSERVATIVE**



WT-6

Shape:	WT7x11
Edge Distance:	3.5 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

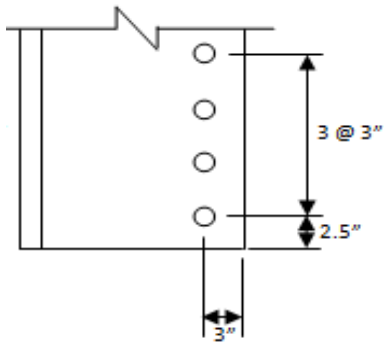
\bar{x}	=	0	inches	
L	=	6	inches	
A_{gt}	=	0.81	in ²	
A_{nt}	=	0.68	in ²	
A_{gv}	=	1.96	in ²	
A_{nv}	=	1.31	in ²	
U	=	0.90		
A_e	=	0.61	in ²	
$F_u A_{nt}$	=	35.27	k	
$0.6F_u A_{nv}$	=	45.52	k	
R_n	=	74.50	k	GOVERNS
R_n	=	80.79	k	
experimental failure load	=	110.4	k	
PF	=	1.48	CONSERVATIVE	



WT-7

Shape: WT7x11
 Edge Distance: 2.5 inches
 Length of Connection: 11.5 inches
 Thickness: 0.23 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 3.5
 F_u : 58 ksi
 F_y : 36 ksi
 Bolt Diameter: 1 inches

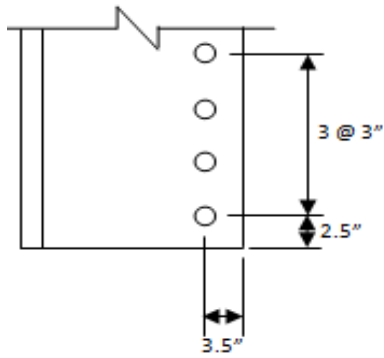
\bar{x} = 0 inches
 L = 9 inches
 A_{gt} = 0.58 in²
 A_{nt} = 0.45 in²
 A_{gv} = 2.65 in²
 A_{nv} = 1.74 in²
 U = 0.90
 A_e = 0.40 in²
 $F_u A_{nt}$ = 23.26 k
 $0.6F_u A_{nv}$ = 60.53 k
 R_n = 81.23 k **GOVERNS**
 R_n = 83.79 k
 experimental failure load = 101.8 k
 PF = 1.25 **CONSERVATIVE**



WT-8

Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	11.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	3.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

\bar{x}	=	0	inches	
L	=	9	inches	
A_{gt}	=	0.69	in ²	
A_{nt}	=	0.56	in ²	
A_{gv}	=	2.65	in ²	
A_{nv}	=	1.74	in ²	
U	=	0.90		
A_e	=	0.50	in ²	
$F_u A_{nt}$	=	29.26	k	
$0.6F_u A_{nv}$	=	60.53	k	
R_n	=	85.37	k	GOVERNS
R_n	=	89.79	k	
experimental failure load	=	116.99	k	
PF	=	1.37	CONSERVATIVE	

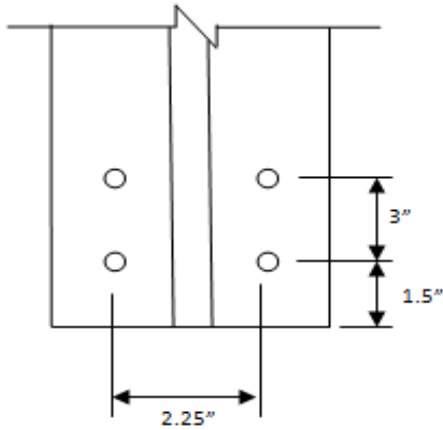


WT-9

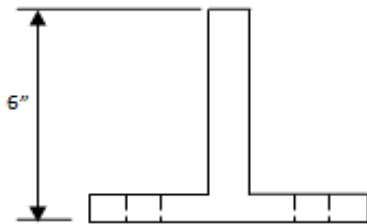
Shape:	WT7x11
Edge Distance:	3.5 inches
Length of Connection:	11.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	3.5
F_u :	58 ksi
F_y :	36 ksi
Bolt Diameter:	1 inches

\bar{x}	=	0	inches	
L	=	9	inches	
A_{gt}	=	0.81	in ²	
A_{nt}	=	0.68	in ²	
A_{gv}	=	2.65	in ²	
A_{nv}	=	1.74	in ²	
U	=	0.90		
A_e	=	0.61	in ²	
$F_u A_{nt}$	=	35.27	k	
$0.6F_u A_{nv}$	=	60.53	k	
R_n	=	89.51	k	GOVERNS
R_n	=	95.80	k	
experimental failure load	=	129.98	k	
PF	=	1.45	CONSERVATIVE	

CHANGES IN LENGTH OF UNBOLTED LEG



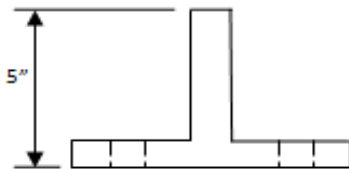
Shape:	WT6x8
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches



WT6x8

\bar{x}	=	1.74	inches
L	=	3	inches
A_{gt}	=	0.23	in ²
A_{nt}	=	0.11	in ²
A_{gv}	=	1.19	in ²
A_{nv}	=	0.84	in ²
U	=	0.42	
A_e	=	0.05	in ²
$F_u A_{nt}$	=	3.13	k
$0.6 F_u A_{nv}$	=	32.94	k
R_n	=	44.47	k
R_n	=	36.07	k
experimental failure load	=	92.13	k
PF	=	2.55	CONSERVATIVE

GOVERNS

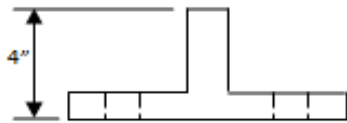


WT6x8-1

Shape:	WT6x8-1
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

\bar{x}	=	1.37	inches
L	=	3	inches
A_{gt}	=	0.23	in ²
A_{nt}	=	0.11	in ²
A_{gv}	=	1.19	in ²
A_{nv}	=	0.84	in ²
U	=	0.54	
A_e	=	0.06	in ²
$F_u A_{nt}$	=	4.05	k
$0.6 F_u A_{nv}$	=	32.94	k
R_n	=	44.47	k
R_n	=	36.99	k
experimental failure load	=	91.8	k
PF	=	2.48	CONSERVATIVE

GOVERNS

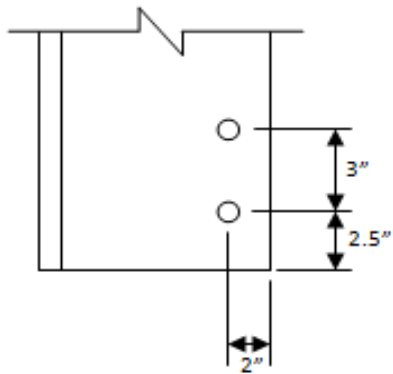


WT6x8-2

Shape:	WT6x8-2
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
Bolt Diameter:	0.75 inches

\bar{x}	=	1.007	inches	
L	=	3	inches	
A_{gt}	=	0.23	in ²	
A_{nt}	=	0.11	in ²	
A_{gv}	=	1.19	in ²	
A_{nv}	=	0.84	in ²	
U	=	0.66		
A_e	=	0.08	in ²	
$F_u A_{nt}$	=	4.95	k	
$0.6 F_u A_{nv}$	=	32.94	k	
R_n	=	44.47	k	
R_n	=	37.89	k	GOVERNS
experimental failure load	=	87.6	k	
PF	=	2.31	CONSERVATIVE	

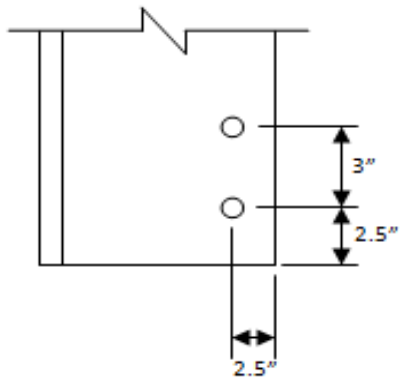
Appendix P: 2005 LRFD/ASD Excel Spreadsheets (Angles)



A-1

Shape:	L6x4x ⁵ / ₁₆
Edge Distance:	2 inches
Length of Connection:	5.5 inches
Thickness:	0.3125 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F _u :	58 ksi
F _y :	36 ksi
U _{bs} :	1
Bolt Diameter:	1 inches

\bar{x}	=	0.908	inches	
L	=	3	inches	
A _{gt}	=	0.63	in ²	
A _{nt}	=	0.45	in ²	
A _{gv}	=	1.72	in ²	
A _{nv}	=	1.19	in ²	
U	=	0.70		
A _e	=	0.31	in ²	
R _n	=	59.63	k	
R _n	=	55.29	k	governs
experimental failure load	=	81.4	k	
PF	=	1.47	CONSERVATIVE	

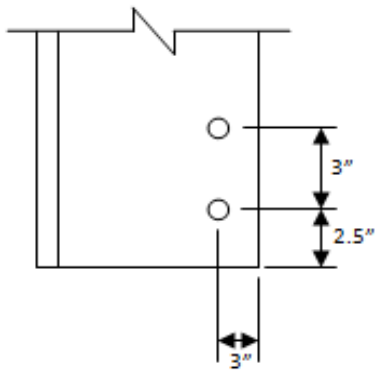


A-2

Shape:	L6x4x ⁵ / ₁₆
Edge Distance:	2.5 inches
Length of Connection:	5.5 inches
Thickness:	0.3125 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

\bar{x}	=	0.908	inches
L	=	3	inches
A_{gt}	=	0.78	in ²
A_{nt}	=	0.61	in ²
A_{gv}	=	1.72	in ²
A_{nv}	=	1.19	in ²
U	=	0.70	
A_e	=	0.42	in ²
R_n	=	65.95	k
R_n	=	61.61	k
experimental failure load	=	99.99	k
PF	=	1.62	CONSERVATIVE

governs

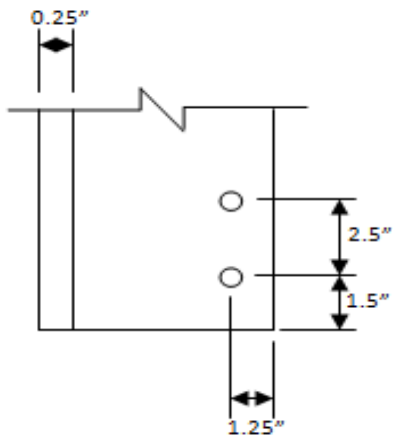


A-3

Shape:	L6x4x ⁵ / ₁₆
Edge Distance:	3 inches
Length of Connection:	5.5 inches
Thickness:	0.3125 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

\bar{x} =	0.908	inches	
L =	3	inches	
A_{gt} =	0.94	in ²	
A_{nt} =	0.76	in ²	
A_{gv} =	1.72	in ²	
A_{nv} =	1.19	in ²	
U =	0.70		
A_e =	0.53	in ²	
R_n =	72.27	k	
R_n =	67.93	k	governs
experimental failure load =	112.4	k	
PF =	1.65	CONSERVATIVE	

Outstanding Leg= 3"

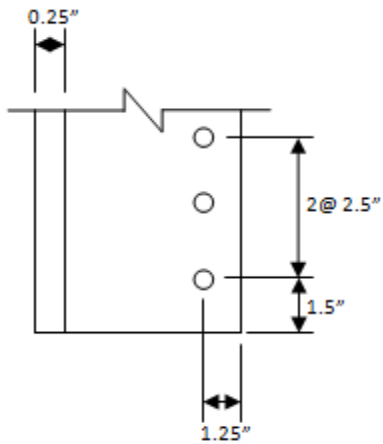


A588-1

Edge Distance: 1.25 inches
 Length of Connection: 4 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 70 ksi
 F_y : 50 ksi
 U_{bs} : 1
 Bolt Diameter: 0.75 inches

\bar{x} = 0.736 inches
 L = 2.5 inches
 A_{gt} = 0.31 in²
 A_{nt} = 0.20 in²
 A_{gv} = 1.00 in²
 A_{nv} = 0.67 in²
 U = 0.71
 A_e = 0.14 in²
 R_n = 38.25 k **governs**
 R_n = 40.03 k
 experimental failure load = 52 k
 PF = 1.36 **CONSERVATIVE**

Outstanding Leg= 3"



A588-3

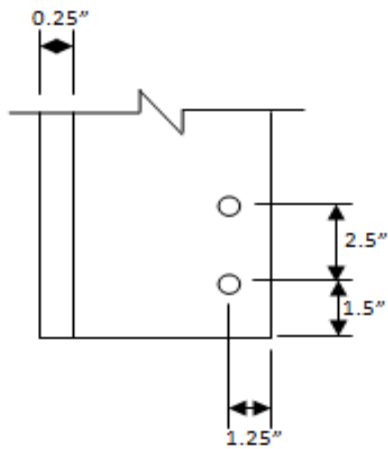
Edge Distance:	1.25 inches
Length of Connection:	6.5 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	70 ksi
F_y :	50 ksi
U_{bs} :	1
Bolt Diameter:	0.75 inches

\bar{x} =	0.736	inches
L=	5	inches
A_{gt} =	0.31	in ²
A_{nt} =	0.20	in ²
A_{gv} =	1.63	in ²
A_{nv} =	1.08	in ²
U=	0.85	
A_e =	0.17	in ²
R_n =	57.41	k
R_n =	60.88	k
R_n =	71.4	k
PF=	1.24	CONSERVATIVE

governs

experimental failure load=

Outstanding Leg= 4"



A588-4

Edge Distance: 1.25 inches
 Length of Connection: 4 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 1.5
 F_u : 70 ksi
 F_y : 50 ksi
 U_{bs} : 1
 Bolt Diameter: 0.75 inches

$\bar{x} = 1.09$ inches

$L = 2.5$ inches

$A_{gt} = 0.31$ in²

$A_{nt} = 0.20$ in²

$A_{gv} = 1.00$ in²

$A_{nv} = 0.67$ in²

$U = 0.60$

$A_e = 0.12$ in²

$R_n = 36.75$ k **governs**

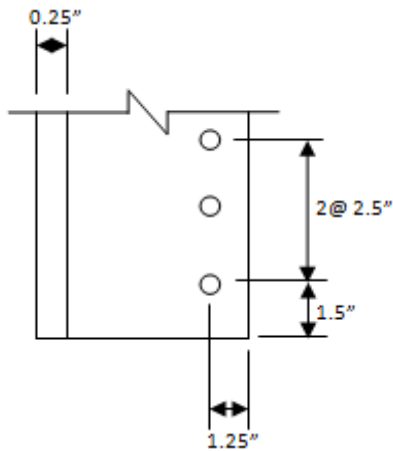
$R_n = 38.53$ k

48.5 k

PF= 1.32 **CONSERVATIVE**

experimental failure load=

Outstanding Leg= 4"

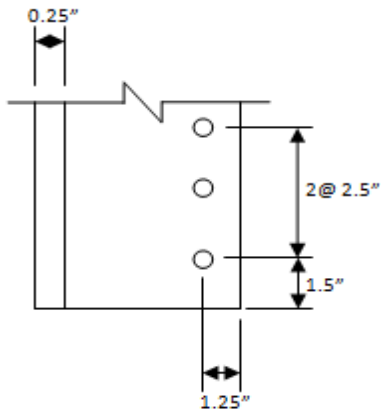


A588-5

Edge Distance:	1.25 inches
Length of Connection:	6.5 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	70 ksi
F_y :	50 ksi
U_{bs} :	1
Bolt Diameter:	0.75 inches

\bar{x}	=	1.09	inches	
L	=	5	inches	
A_{gt}	=	0.31	in ²	
A_{nt}	=	0.20	in ²	
A_{gv}	=	1.63	in ²	
A_{nv}	=	1.08	in ²	
U	=	0.78		
A_e	=	0.16	in ²	
R_n	=	56.40	k	governs
R_n	=	59.87	k	
experimental failure load	=	67.2	k	
PF	=	1.19		CONSERVATIVE

Outstanding Leg= 3"

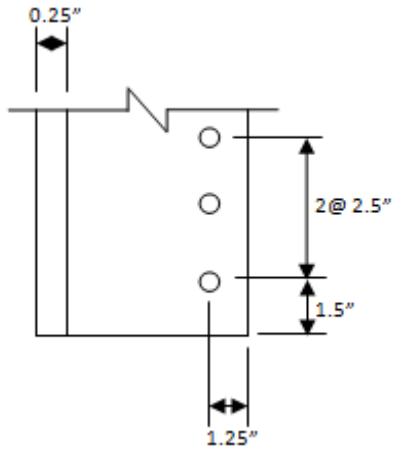


A36-2

Edge Distance: 1.25 inches
 Length of Connection: 6.5 inches
 Thickness: 0.25 inches
 # Bolts in Tension: 0.5
 # Bolts in Shear: 2.5
 F_u : 58 ksi
 F_y : 36 ksi
 U_{bs} : 1
 Bolt Diameter: 0.75 inches

$\bar{x} = 0.736$ inches
 $L = 5$ inches
 $A_{gt} = 0.31$ in²
 $A_{nt} = 0.20$ in²
 $A_{gv} = 1.63$ in²
 $A_{nv} = 1.08$ in²
 $U = 0.85$
 $A_e = 0.17$ in²
 $R_n = 47.57$ k
 $R_n = 45.15$ k **governs**
 experimental failure load= 57.8 k
 $PF = 1.28$ **CONSERVATIVE**

Outstanding Leg= 4"

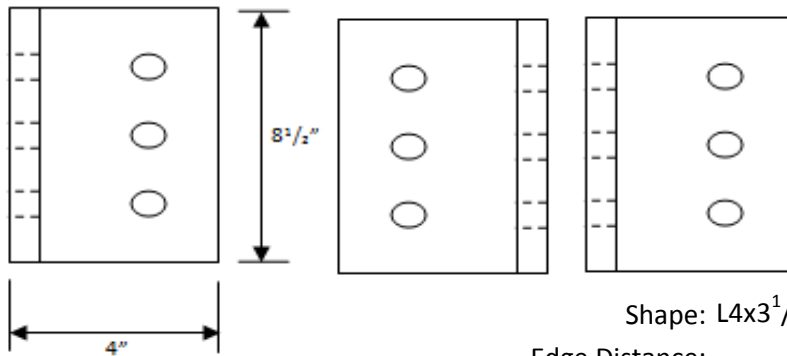


A36-3

Edge Distance:	1.25 inches
Length of Connection:	6.5 inches
Thickness:	0.25 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	0.75 inches

\bar{x} =	1.09	inches	
L=	5	inches	
A_{gt} =	0.31	in ²	
A_{nt} =	0.20	in ²	
A_{gv} =	1.63	in ²	
A_{nv} =	1.08	in ²	
U=	0.78		
A_e =	0.16	in ²	
R_n =	46.73	k	
R_n =	44.31	k	governs
experimental failure load=	52.3	k	
PF=	1.18	CONSERVATIVE	

1953 CONNECTION



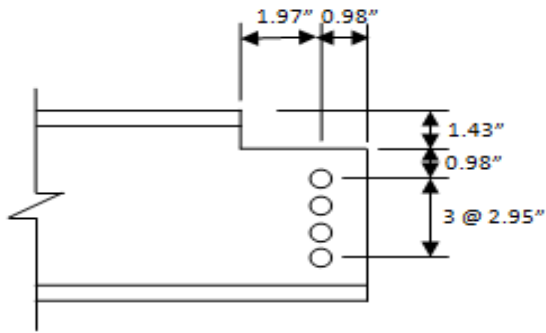
Shape: $L4 \times 3 \frac{1}{2} \times \frac{3}{8}$

Edge Distance:	1.75 inches
Length of Connection:	7.25 inches
Thickness:	0.38 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	0.75 inches

\bar{x}	=	0.947	inches
L	=	6	inches
A_{gt}	=	0.66	in^2
A_{nt}	=	0.49	in^2
A_{gv}	=	2.72	in^2
A_{nv}	=	1.90	in^2
U	=	0.84	
A_e	=	0.41	in^2
R_n	=	180.21	k
R_n	=	165.53	k
experimental failure load	=	21.6	k
PF	=	0.13	UNCONSERVATIVE

governs

Appendix Q: 2005 LRFD/ASD Excel Spreadsheets (Coped Beams)



B2

Shape: W16x31

Edge Distance: 0.98 inches

Length of Connection: 9.83 inches

Thickness: 0.275 inches

Bolts in Tension: 3.5

Bolts in Shear: 0.5

F_u : 74.4 ksi

F_y : 53.2 ksi

U_{bs} : 1

Bolt Diameter: 0.75 inches

\bar{x} = 0 inches

L = 8.85 inches

A_{gt} = 0.27 in²

A_{nt} = -0.57 in²

A_{gv} = 2.70 in²

A_{nv} = 2.58 in²

U = 1.00

A_e = -0.57 in²

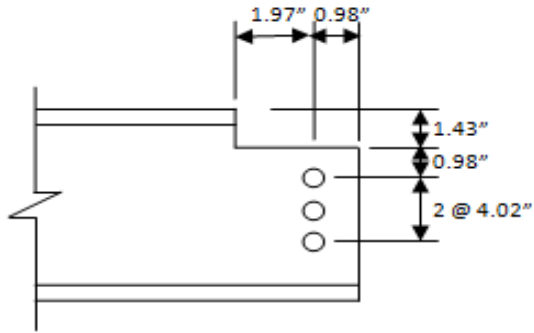
R_n = 72.69 k

R_n = 43.68 k

governs

experimental failure load = 106.7842 k

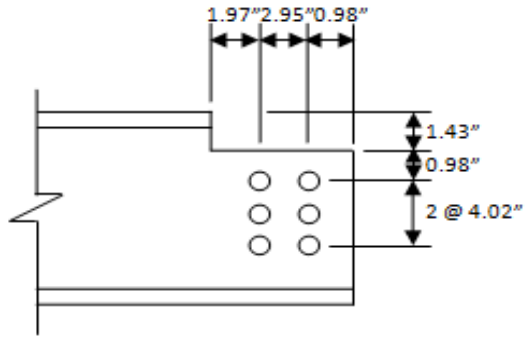
PF = 2.44 **CONSERVATIVE**



C1

Shape: W16x31
 Edge Distance: 0.98 inches
 Length of Connection: 9.02 inches
 Thickness: 0.275 inches
 # Bolts in Tension: 3.5
 # Bolts in Shear: 0.5
 F_u : 74.8 ksi
 F_y : 53.1 ksi
 U_{bs} : 1
 Bolt Diameter: 0.75 inches

$\bar{x} = 0$ inches
 $L = 8.04$ inches
 $A_{gt} = 0.27$ in²
 $A_{nt} = -0.57$ in²
 $A_{gv} = 2.48$ in²
 $A_{nv} = 2.36$ in²
 $U = 1.00$
 $A_e = -0.57$ in²
 $R_n = 63.09$ k
 $R_n = 36.19$ k **governs**
 experimental failure load = 90.3732 k
 PF = 2.50 **CONSERVATIVE**

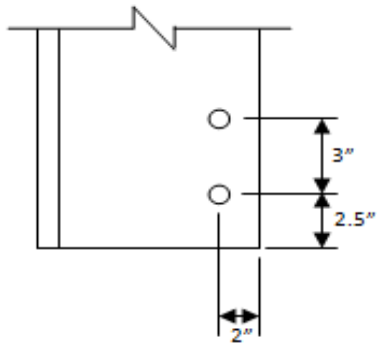


C2

Shape:	W16x31
Edge Distance:	0.98 inches
Length of C	9.02 inches
Thickness:	0.275 inches
# Bolts in Tension:	3.5
# Bolts in Shear:	0.5
F_u :	74.8 ksi
F_y :	53.1 ksi
U_{bs} :	0.5
Bolt Diameter:	0.75 inches

e	= 0	inches
L	= 8.04	inches
A_{gt}	= 0.27	in^2
A_{nt}	= -0.57	in^2
A_{gv}	= 2.48	in^2
A_{nv}	= 2.36	in^2
U	= 1.00	
A_e	= -0.57	in^2
R_n	= 84.51	k
R_n	= 57.61	k
		governs
experimental failure load	= 120.7224	k
PF	= 2.10	CONSERVATIVE

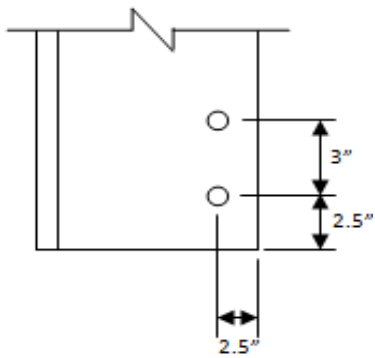
Appendix R: 2005 LRFD/ASD Excel Spreadsheets (Tees)



WT-1

Shape:	WT7x11
Edge Distance:	2 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

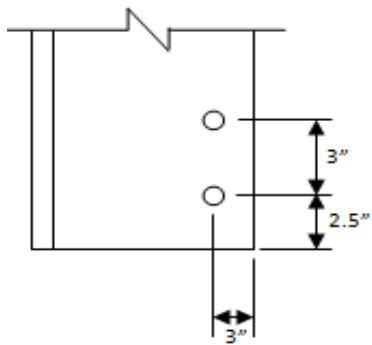
\bar{x} =	0	inches	
L =	3	inches	
A_{gt} =	0.46	in ²	
A_{nt} =	0.33	in ²	
A_{gv} =	1.27	in ²	
A_{nv} =	0.88	in ²	
U =	1.00		
A_e =	0.33	in ²	
R_n =	49.69	k	
R_n =	46.50	k	governs
experimental failure load =	59.5	k	
PF =	1.28	CONSERVATIVE	



WT-2

Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

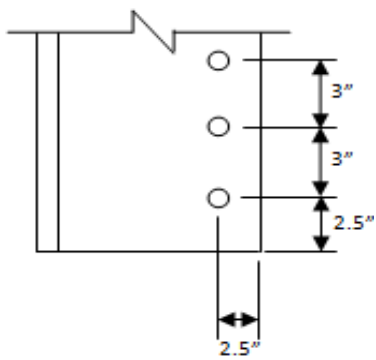
\bar{x}	=	0	inches	
L	=	3	inches	
A_{gt}	=	0.58	in ²	
A_{nt}	=	0.45	in ²	
A_{gv}	=	1.27	in ²	
A_{nv}	=	0.88	in ²	
U	=	1.00		
A_e	=	0.45	in ²	
R_n	=	56.36	k	
R_n	=	53.17	k	governs
experimental failure load	=	70.1	k	
PF	=	1.32		CONSERVATIVE



WT-3

Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	5.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

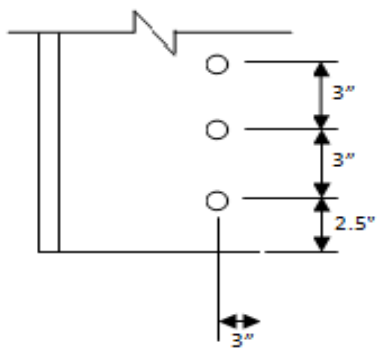
\bar{x} =	0	inches	
L =	3	inches	
A_{gt} =	0.69	in ²	
A_{nt} =	0.56	in ²	
A_{gv} =	1.27	in ²	
A_{nv} =	0.88	in ²	
U =	1.00		
A_e =	0.56	in ²	
R_n =	63.03	k	
R_n =	59.84	k	governs
experimental failure load =	77.6	k	
PF =	1.30	CONSERVATIVE	



WT-4

Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

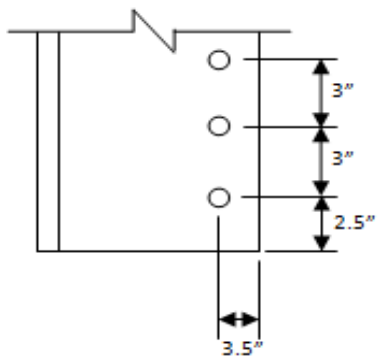
e	=	0	inches	
L	=	6	inches	
A_{gt}	=	0.58	in^2	
A_{nt}	=	0.45	in^2	
A_{gv}	=	1.96	in^2	
A_{nv}	=	1.31	in^2	
U	=	1.00		
A_e	=	0.45	in^2	
R_n	=	71.37	k	
R_n	=	68.07	k	governs
experimental failure load	=	85.2	k	
PF	=	1.25	CONSERVATIVE	



WT-5

Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

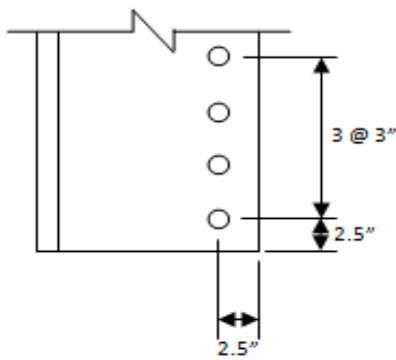
e	=	0	inches	
L	=	6	inches	
A_{gt}	=	0.69	in^2	
A_{nt}	=	0.56	in^2	
A_{gv}	=	1.96	in^2	
A_{nv}	=	1.31	in^2	
U	=	1.00		
A_e	=	0.56	in^2	
R_n	=	78.04	k	
R_n	=	74.74	k	governs
experimental failure load	=	96.1	k	
PF	=	1.29		CONSERVATIVE



WT-6

Shape:	WT7x11
Edge Distance:	3.5 inches
Length of Connection:	8.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	2.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

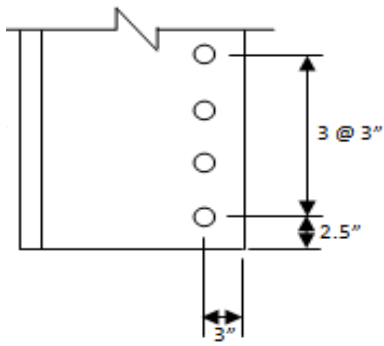
e	=	0	inches	
L	=	6	inches	
A_{gt}	=	0.81	in^2	
A_{nt}	=	0.68	in^2	
A_{gv}	=	1.96	in^2	
A_{nv}	=	1.31	in^2	
U	=	1.00		
A_e	=	0.68	in^2	
R_n	=	84.71	k	
R_n	=	81.41	k	governs
experimental failure load	=	110.4	k	
PF	=	1.36		CONSERVATIVE



WT-7

Shape:	WT7x11
Edge Distance:	2.5 inches
Length of Connection:	11.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	3.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

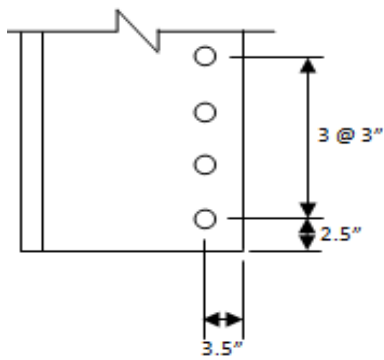
e	=	0	inches	
L	=	9	inches	
A_{gt}	=	0.58	in^2	
A_{nt}	=	0.45	in^2	
A_{gv}	=	2.65	in^2	
A_{nv}	=	1.74	in^2	
U	=	1.00		
A_e	=	0.45	in^2	
R_n	=	86.38	k	
R_n	=	82.98	k	governs
experimental failure load	=	101.8	k	
PF	=	1.23		CONSERVATIVE



WT-8

Shape:	WT7x11
Edge Distance:	3 inches
Length of Connection:	11.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	3.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

e	=	0	inches
L	=	9	inches
A_{gt}	=	0.69	in^2
A_{nt}	=	0.56	in^2
A_{gv}	=	2.65	in^2
A_{nv}	=	1.74	in^2
U	=	1.00	
A_e	=	0.56	in^2
R_n	=	93.05	k
R_n	=	89.65	k
experimental failure load	=	116.99	k
PF	=	1.30	CONSERVATIVE



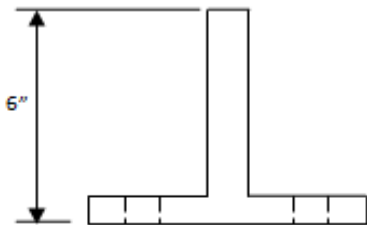
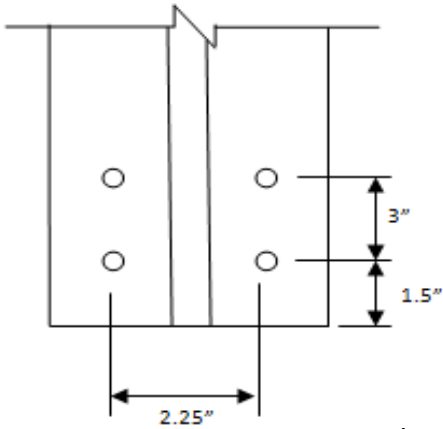
WT-9

Shape:	WT7x11
Edge Distance:	3.5 inches
Length of Connection:	11.5 inches
Thickness:	0.23 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	3.5
F_u :	58 ksi
F_y :	36 ksi
U_{bs} :	1
Bolt Diameter:	1 inches

e	=	0	inches
L	=	9	inches
A_{gt}	=	0.81	in^2
A_{nt}	=	0.68	in^2
A_{gv}	=	2.65	in^2
A_{nv}	=	1.74	in^2
U	=	1.00	
A_e	=	0.68	in^2
R_n	=	99.72	k
R_n	=	96.32	k
experimental failure load	=	129.98	k
PF	=	1.35	CONSERVATIVE

governs

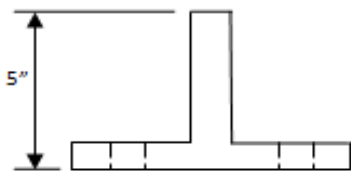
CHANGES IN LENGTH OF UNBOLTED LEG



WT6x8

Shape:	WT6x8
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
U_{bs} :	1
Bolt Diameter:	0.75 inches

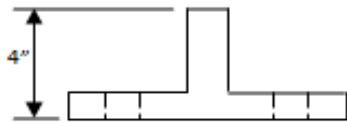
\bar{x} =	1.74	inches	
L =	3	inches	
A_{gt} =	0.23	in ²	
A_{nt} =	0.11	in ²	
A_{gv} =	1.19	in ²	
A_{nv} =	0.84	in ²	
U =	0.60		
A_e =	0.07	in ²	
R_n =	74.83	k	governs
R_n =	80.49	k	
experimental failure load =	92.13	k	
PF =	1.23	CONSERVATIVE	



WT6x8-1

Shape:	WT6x8-1
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
U_{bs} :	1
Bolt Diameter:	0.75 inches

\bar{x} =	1.37	inches	
L =	3	inches	
A_{gt} =	0.23	in ²	
A_{nt} =	0.11	in ²	
A_{gv} =	1.19	in ²	
A_{nv} =	0.84	in ²	
U =	0.60		
A_e =	0.07	in ²	
R_n =	74.83	k	governs
R_n =	80.49	k	
experimental failure load =	91.8	k	
PF =	1.23	CONSERVATIVE	



WT6x8-2

Shape:	WT6x8-2
Edge Distance:	0.87 inches
Length of Connection:	4.5 inches
Thickness:	0.265 inches
# Bolts in Tension:	0.5
# Bolts in Shear:	1.5
F_u :	65 ksi
F_y :	50 ksi
U_{bs} :	1
Bolt Diameter:	0.75 inches

\bar{x}	=	1.007	inches	
L	=	3	inches	
A_{gt}	=	0.23	in ²	
A_{nt}	=	0.11	in ²	
A_{gv}	=	1.19	in ²	
A_{nv}	=	0.84	in ²	
U	=	0.66		
A_e	=	0.08	in ²	
R_n	=	75.78	k	governs
R_n	=	81.45	k	
experimental failure load	=	87.6	k	
PF	=	1.16		CONSERVATIVE