Behavior and Design of

Skewed Extended Shear Tab Connections

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THESIS
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DEDICATION

This dissertation is dedicated to my siblings, Mohannad Al Hijaj, Marah Al Hijaj, Mais Al Hijaj, Maram Al Hijaj, Manar Al Hijaj, and Mahmoud Al Hijaj. And to the most important two people in my life, my Mom (Maha Adham) and Dad (Ahmed Al Hijaj) for their endless support, encouragement, and love.

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AISC	American Institute of Steel Construction
ASTM	American Society for Testing and Materials
FEA	Finite Element Analysis
DOFs	Degrees of Freedom
PEEQ	Equivalent Plastic Strain

SUMMARY

A study on the behavior and design of skewed extended shear tab connections was carried out using the finite element software ABAQUS. The finite element models were validated with experimental results done by other researchers for orthogonal configurations. The validation done by comparing the connection shear-connection vertical displacement curves, connection shear-beam end rotation curves, and comparing the failure modes.

A comparison was carried out between the behavior of skewed and orthogonal configurations for: unstiffened extended shear tab connections with the plate welded to the supporting member web (flexible support), stiffened extended shear tab connections with the plate welded to the supporting member web, and unstiffened extended shear tab connections with the plate welded to the supporting member flange (rigid support).

Parametric study was performed on orthogonal and skewed extended shear tab connections. The interaction of different parameters (connection orientation, plate thickness, a-distance, and number of bolts) and their effect on the connection behavior was investigated.

Equations that relate the plate twist, connection vertical displacement, and connection shear capacity for the skewed and orthogonal extended shear tab connections were proposed. Moreover, design procedure and modifications on the current design procedure to account for the connection orientation effect were proposed.

1. CHAPTER ONE – INTRODUCTION

1.1. Background

The skewed extended shear tab connection is considered as single-plate type connection. These connections are made of a plate welded to the supporting member using fillet weld and bolted to the supported member as shown in Figure 1-1 below. The goal of using the shear tab connection is to transfer shear force from the beam to the supporting member (column or girder). There are two types of single-plate connections; standard and extended configuration; the standard configuration must have a single vertical row of bolts, the number of bolts must be between 2 and 12, and the distance from the bolt line to the weld line (a-distance) must be equal to or less than 3.5 in. This configuration usually used when the plate is welded to the flange of the supporting member. The extended configuration can be used when the previous limitations are not satisfied. Figure 1-1 shows extended shear tab connection with a-distance greater than 3.5 in. and two vertical rows of bolts.

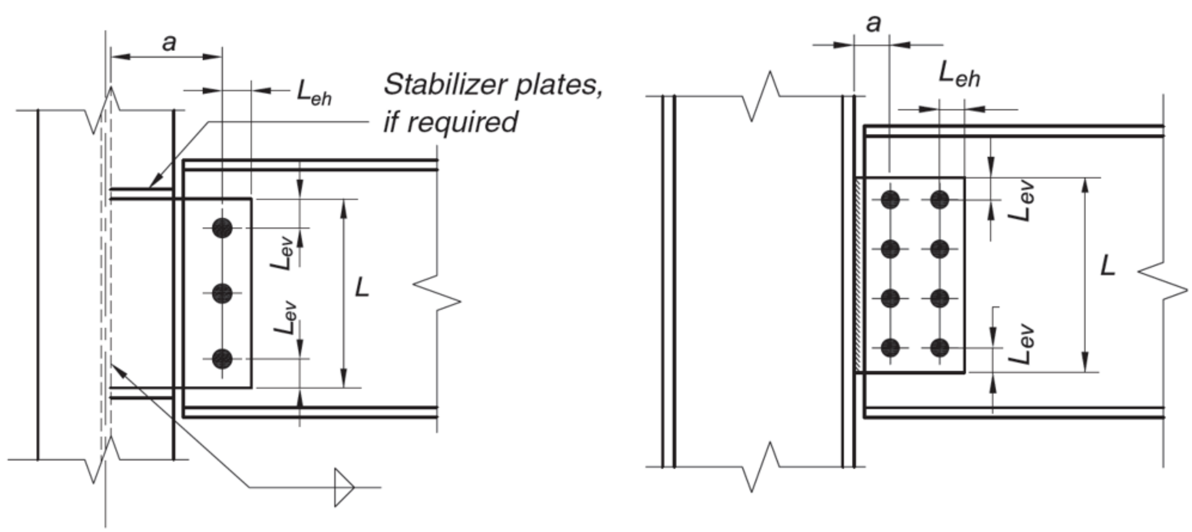


Figure 1-1. Extended shear tab connections

The main advantage of using extended shear tab connections is to prevent coping and excessive cutting of beam flanges and part of the web in the vicinity of the joint. Additionally, this configuration is more practical and more economical for complex geometry applications. The extended shear tab connection has been introduced to the structural engineering practice in the American Institute of Steel Construction (AISC) manual 13th edition (AISC, 2005).

Skewed connection (Figure 1-2) is defined as the connection where the beam flanges lie in a plane perpendicular to the plane of the face of the supporting member, but its web inclined to the face of the supporting member (AISC, 2010). According to the AISC manual 14th edition (AISC, 2010), when the skewed angle (α) is smaller than 5^0, a pair of double bent angles or bent plates can be used to make the connection. On the other hand, when $5^0 < \alpha < 30^0$, skewed single-plate can be used to make the connection due to its simplicity and ease of erection. Based on the AISC manual 14th edition (AISC, 2010), the design of skewed single-plate connections is similar to the design of single-plate connections.

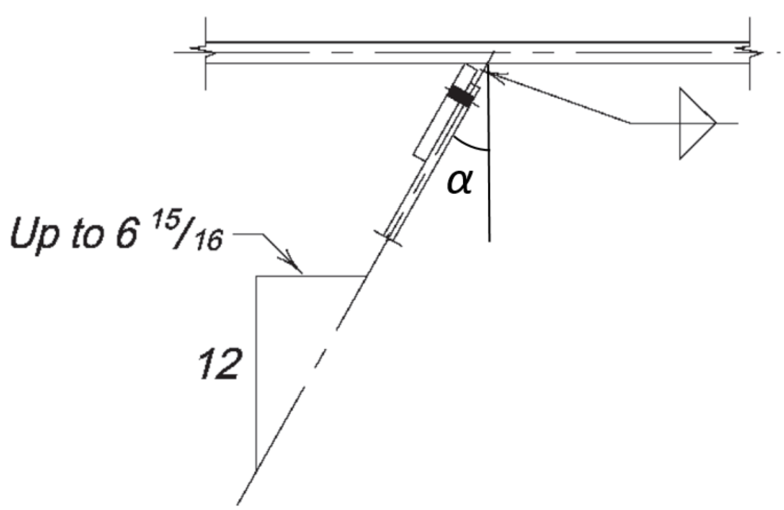


Figure 1-2. Skewed single-plate connection

1.2. Literature Review

Numerous studies related to the single and extended shear tab connections have been carried out. However, few studies related to the skewed connections were conducted. The most relevant studies to the current study are shown below:

Richard et al. (1980) studied the behavior of the single plate framing connections with ASTM A325 and ASTM A490 bolts. The study was divided into four stages: (1) Single bolt-single shear tests, these tests were performed with different bolt diameters and plate thicknesses, it was concluded form these tests that in order to provide sufficient ductility to the framing connection the bolt shear and tension tearing failure modes should be circumvented. (2) Finite element models were created for single plate framing connections to determine the behavior of the single plate connection using the software program (NELAS). (3) Developing a non-dimensional analytical expression for representing the typical framing plate design using moment-rotation curves obtained from the finite element model analyses, the author concluded that the moment-rotation curve is dependent on the shear at the connection when the eccentricity is less than the height of the bolt pattern, and independent from the shear at the connection when the eccentricity is greater or equal to the height of the bolt pattern. (4) In order to validate the numerical results, the author performed a full scale laboratory tests for two-, three-, five-, and seven-bolt connections, and five full scale beam tests. The experimental results were in good agreement with the finite element results and the author showed that all beams were loaded at least 1.5 times the working load and in all cases the connections exhibited no distortion or distress. At the end of this study, the authors proposed a design procedure for the single plate framing connections with ASTM A325 and ASTM A490 bolts based on the numerical and experimental results obtained from his study.

Richard et al. (1982) investigated the behavior of the single plate framing connections with A307 bolts. The study was divided into two stages: In the first stage, A307 single bolt-single shear tests were performed, where the author showed that the bolt diameter-to-plate thickness ratio should be equal to 4 when using A307 bolt in standard hole in order to prevent the bolt shear failure mode, It was also recommended to use slotted hole with A307 bolt where the bolt diameter-to-plate thickness ratio does not apply. In the second stage, fifteen full scale tests were performed using 7/8 inches diameter A307 bolts with slotted holes made in the plate and standard holes made in the beam web, the bolts were tightened to a snug fit with a spud wrench, other tests were performed with bolts simply finger tight for the purpose of comparison. It was noticed that a moment was generated in the connection for the cases where the bolts were installed with a spud wrench and it was generated as result of the clamping force between the beam web and the connection plate. Also, a formula to calculate the eccentricity was derived based on the results obtained from the experimental tests. At the end of this study, the author proposed a detailed procedure to design single plate framing connections with A307 bolts.

Cheng et al. (1984) performed theoretical parametric study to investigate the behavior of coped beams with various coping details using the finite element software BASP and ABAQUS. The author indicated that coped beams can fail in local web buckling at the cope region and lateral-torsional buckling over the span in addition to yielding. Additionally, the authors showed that buckling capacity is highly affected by the cope length, cope depth and span length. A plate buckling and lateral-torsional buckling models were developed to check for web buckling in top flange coped and double flange coped beams, respectively, in order to develop interaction equations for design of lateral-torsional buckling of those beams. Also, the authors showed that the lateral-torsional buckling capacity was reduced by ten percent when coping the tension flange of beams. In order to verify the design recommendations, the author performed sixteen full scale experimental tests; eight tests considered local web buckling and six tests for lateral torsional buckling. The experimental results showed that the suggested design recommendations are acceptable and conservative. Finally, the author investigated different types of stiffeners at the coped region to evaluate their effects on coped beams capacities.

Astaneh et al. (1989) investigated the single plate shear connections behavior, the authors indicated that the single plate shear connection should be designed for shear strength and rotational flexibility. Five full scale beam-to-column connections were tested to obtain the limit states for the single plate shear connections, coupon tests of the plate material were performed to obtain the yield and ultimate strengths. Each connection consisted of a single plate welded to a column flange and bolted from the other side to a wide flange beam, the bot holes were standard holes. The experiments were conducted in two groups, the variable parameters were the bolt type (A325-N or A490-N), beam material (A36 or grade 50) and edge distance ($2d_b$ or $1.5d_b$); where d_b is the bolt diameter. The results revealed that the specimens with A325 bolts had failed due to sudden shear fracture of the bolts and the bolts had developed significant permanent deformations prior to fracture, another observation was the permanent bearing deformations in the plate as well as in the beam web. Specimens with A490 bolts failed by simultaneous fracture of weld lines and bolts, the A490 bolts showed less permanent deformations than the A325 bolts. Also, the author observed that the connections exhibited larger rotational ductility and developed smaller moments as the number of bolts decreased and showed that local buckling can be avoided by using distance between the bolt line and the weld line less than half the length of the plate. The author recommended a design procedure for single plate shear connections based on the analyses of the experimental results. Finally, the author concluded that the limit states associated with single plate connections are: plate yielding, fracture of the net section of plate, bolt fracture, weld fracture, and bearing failure of bolt holes.

Astaneh et al. (1993) studied the behavior of steel single plate shear connections, the author showed that in addition to the adequate shear capacity, shear connections should have sufficient rotational ductility to accommodate simply supported beam end-rotation to prevent development of significant moment in the connection. First, the end support shear-rotation relationship for simply supported beam was studied using computer software that simulate the behavior of simply supported beam up to failure. It was indicated that the shear-rotation relationship is highly affected by the span-to-depth ratio and the beam shape factor. A trilinear shear-rotation curve was proposed to represent the actual behavior of the shear connection including the elastic, inelastic and full plastic-hinge formation behaviors. Second, the author performed experimental tests on six full scale connections in order to develop a design procedure and understand the actual behavior of connections. The author indicated that the proposed test procedure will provide enough shear strength and rotational flexibility to the connection and enable the beam to reach its plastic collapse mechanism. ASTM-A325 and ASTM-A490 bolts, E7018 welding electrodes and typical wide-flange beams connected to short columns were used. It was observed from results that six failure modes might govern the connection behavior: Plate yielding, bearing failure of bolt holes, fracture of net section of plate, fracture of plate edge distance, bolt fracture, and weld fracture. Additionally, it was observed that connections with more bolts exhibit larger moment and develop smaller rotational ductility. Finally, the author proposed design procedure in such a way that the ductile limit states will precede the brittle limit states in order to provide sufficient rotational flexibility to the connection. The proposed design procedure was adapted by the American Institute of Steel Construction.

Bursi and Jaspart (1998) investigated the finite element analysis difficulties in simulating the behavior of extended end plate connections using ABAQUS. The author indicated that the accuracy of the finite element modeling highly dependent on element types, mesh size, number of integration points, kinematic descriptions, time step size, and material properties' relationships. The study was started with modeling tee stubs with non-preloaded and preloaded bolts as benchmarks. Additionally, the author proposed modeling the bolts using solid model or assemblage of beam, viz and spin elements. Finally, the author proposed and validated a 3D non-linear finite element model to simulate the behavior of isolated extended end plate moment resisting connections.

Kloiber and Thornton (2001) discussed some of the design considerations required for the skewed connections and provided some guidance in the choice of these connection configurations based on economy and safety. It was mentioned that in most structures, many members do not meet at right angles, and skewed connections need to be used to join these members. It was shown that single plates, end plates, single bent plates, and eccentric end plates are preferred skewed connections to join beams to wide flange girder webs. However, single plates with snug-tight bolts are the most economical skewed connections with excellent dimensional control for 90^{o} to 30^{o} intersection angles. But the author showed that the carrying capacity of these connections are low; it was shown that this is not a problem since the skewed members carry less loads. The author indicated that the required size of the plate, bolts and weld for the required load can be selected using the AISC tables. Additionally, there is eccentricity related to the distance between the bolt line and the weld line which depends on the hole type, bolt installation and support flexibility. Also, the author mentioned that the minimum weld size should be ¾ of the plate thickness for A36 plate and E70 electrodes to insure that the plate will yield before the weld. It was indicated that skewed connections to wide flange column flanges are preferred instead of skewed connections to column webs, since the latter has very limited access. The author added that end plate or eccentric end plate are preferred for skewed connections to column web, and single plate should not be used unless the bolt line exceed column flanges, and stiffener plates at the top and bottom of the plate might be needed. The authors showed that eccentricity should be included in the connection and column design. Furthermore, skewed connections will have eccentricity when the beam alignment is centered to the column center, however, there will be no eccentricity about the minor axis and the major axis eccentricity will not govern the column design when the beam alignment is centered to the column flange. Kloiber and Thornton stated that there are two ways to modify the standard orthogonal fillet welds when using skewed connections. The initial approach is the AWS D1.1 Structural Welding Code method, which provides equal strength in the obtuse and acute welds by calculating the effective throat for skewed T-joints with varying dihedral angles. The second way is the AISC method which is simpler since it increases the obtuse weld size by the gap between the plate and the supporting member. Additionally, the authors indicated that both methods provides more strength than the required orthogonal weld size. The author mentioned that the leg size is measured as the projection of the contact length of one leg on a line perpendicular to the other leg, which is not the case for the orthogonal fillets where the leg size is the contact length. Finally, the authors indicated that their study provided a reliable limit state approach for the analysis and design of the skewed connections since there is an insufficient amount of guidance for the design of these connections.

Sherman and Ghorbanpoor (2002) studied experimentally the behavior of extended shear tab connections by conducting 31 full scale tests in for groups. In two groups of these tests, the extended shear tab was welded to the column web, in the other two groups, the extended shear tab was welded to the girder web. The study was conducted in three phases: In phase one, seventeen tests were performed to investigate the unstiffened and stiffened shear tabs. The capacity of these connections were studied as a function of lateral bracing of the supported beam, width-to-thickness ratio of the supporting member web, size of the shear tab, number of bolts, type of bolt holes, and span-to-depth ratio of the supported beam. In phase two, four tests were conducted to investigate the effect of using snug tightened bolts in short slotted holes, the effect of using one stiffener plate instead of two, and to investigate the behavior of the stiffener plates. In phase three, the author performed ten tests to study the behavior of deep connections (connections with six and eight bolts). For stiffened connections with a supporting column, the shear tab was stiffened using two stiffener plates welded to the column flanges, top, and bottom of the shear tab. Also, for stiffened connections with supporting girder, the shear tab was stiffened by extending the shear tab to the upper flange of the girder. In some of the connection configurations, the plate was also extended to the girder bottom flange. The author found a correlation between uniformly loaded and point loaded beams so that the location of a point load can be chosen in such a manner as to produce the same reaction and rotation at the beam ends to have a realistic shear-rotation loading. The author found that the shear tab twist was observed in a majority of the unstiffened tests. And the increase of the stiffener plate thickness will not improve the performance of the stiffened connection. Thus, it was recommended to use stiffener plates that have the same thickness as the shear tab. Also, the author identified web mechanism of the supporting member web as a new limit state for the unstiffened extended shear tab connections. Finally, the author proposed a new design procedure for the extended shear tab connections. This procedure was recommended for connections with number of bolts varies between two to ten bolts, with vertical welds to the web of the supporting member, shear tab with distance between the bolt line and the tips of the supporting member flange varies between 2.5 in. and 3.5 in., bolt type ASTM A325 and A490, and with lateral braces applied to the beam near the connection. This study formed the basis of the AISC manual 13th edition (AISC 2005) design procedure for the extended configuration of the simple plate connections.

Rex and Easterling (2003) studied experimentally and analytically the behavior of single bolt single plate connections. The author showed that the behavior of bolt bearing on steel plate is one of the important factors for the moment-rotation behavior for a partially restrained steel connection. In the experimental investigation, single bolt bearing on a single plate tests were conducted at Virginia Tech, the variable parameters in these tests included the bolt diameter, plate thickness, edge distance, plate width, and edge condition. It was observed from these tests that the plate have four different failure modes: plate bearing, curling, tearout, and splitting. For the analytical investigation, twenty models were created using the finite element software ANSYS. Eight node brick elements was used for all the models. Additionally, symmetry was taken into account, and mesh size was varied to determine the most efficient modeling scheme. Results obtained from the finite element analysis were used to verify the experimental results and develop methods for predicting the initial stiffness. The author concluded that the model given in the AISC Specification (AISC 1993) best represents the experimental strength values, and shearing plates negatively affect the nominal strength of the connection.

Ashakul (2004) investigated the parameters affecting the bolt shear rupture strength of the single shear plate connection including the effect of the distance from the weld line to the bolt line(a-distance), plate thickness, plate material, and the position of a connection with respect to a beam neutral axis using the finite element program ABAQUS. The author showed that bolt shear rupture strength of a connection is not a function of the a-distance, and a significant horizontal forces will be generated at the bolts if the plate materials and thicknesses do not satisfy ductility criteria, also the author clarified that these forces will lead to a reduction in the shear strength of the bolt group and must be considered in the design of these connections. Furthermore, the author proposed a relationship for calculating plate shear yielding strength based on shear stress distribution.

Creech (2005) suggested in his study that the AISC design procedure for single-plate shear connections is overly conservative. He performed ten full-scale tests for single-plate shear connections with rigid and flexible configurations. Standard and short-slotted bolt holes were used with various numbers of bolts. Also, the effect of using a slab on top of the beam was considered in his tests. The author concluded that the magnitude of eccentricity for connections with four bolts or more is not significant but for two and three bolts connections, the eccentricity should be considered in the design procedure. Also the author suggested that using snug-tight bolts prevent slippage in the connection generated by connection bolts. Additionally, he mentioned that the single plate should satisfy the thickness requirements specified in AISC Manual (AISC, 2001) and should be manufactured from lower grades of steel. Also, the type of hole has no significant effect on ultimate capacity of the connection. Moreover, he concluded that simulating the slab restraint acts as a fixed point where the connections rotate about affects the rotational behavior of the connection.

Goodrich (2005) studied the behavior of extended shear tabs in stiffened beam-to-column web connections experimentally and theoretically. The author performed six experimental tests at different three phases; each phase included two tests. The first phase connections were designed based on AISC Manual and a design load of 44.7 kips. These connections had four bolts with 3/8" shear tab and 5/16" fillet welds. A load of 27.8 kips was used for the design of phase two connections. These connections were made of four bolts with ¼” shear tab and 3/16” fillet welds. For phase three, the designed load was 27.8 kips for four bolts with ½” shear tab and 5/16” fillet welds in order to force the connection to fail in welds or bolts. The author concluded from the experimental investigation that the connections can carry twice the design load based on the AISC manual (AISC, 2001); the connection generally failed due to the buckling of the shear tab. Furthermore, the author created finite element models using ANSYS to verify the experimental tests and to study more cases without the need for experimental investigation. However, he mentioned that the models had a lot of assumptions and estimations and can only be used for approximation purposes.

Metzger (2006) studied experimentally the single plate shear connections behavior at Virginia Polytechnic Institute and State University. The author performed eight full scale single plate shear connections tests (four conventional and four extended configurations), each test consisted of single plate welded to column flange to support the supported member from one side, the far side of the beam was supported by a roller, the load was applied at three points until the failure of the connection or the supported beam. **Metzger** indicated that the AISC 13th Edition (AISC, 2005) design procedure is very conservative in predicting the ultimate strength of both the conventional and the extended configurations. Also the author concluded that the maximum shear values in the study represent lower bound strength prediction. Two tests failed due to weld rupture, the author could not confirm if the weld size or fabrication issues was the reason of the weld failure. Additionally, it was recommended that a parametric study might be performed to find the maximum allowable plate thickness in terms of bolt diameter-to-plate thickness ratio.

Rahman et al. (2007) presented a three dimensional model to study the behavior of the unstiffened extended shear tab connections and validate the experimental results done by Sherman and Ghorbanpoor (2002). The study focused on two configurations: three bolts unstiffened beam-to-column web configuration, and five bolts unstiffened beam-to-column web configuration. The model was created using the finite element software **ANSYS**. Four element types were used in the model: Eight node brick element to model the shear tab, supporting member and the beam. While ten node tetrahedral element was used to model the bolts. Both elements have the ability to track the elastic and plastic behavior and appropriate for modeling the steel members. Additionally, the pretensioning forces in bolts were simulated using pretensioning elements, these elements were defined on the pretensioning section which divides each bolt into two parts. Finally, contact surfaces were created in order to transfer forces from the beam web to bolts and from bolts to the shear tab using contact elements. Three load steps were used: Applying the pretensioning forces in bolts, transformation of pretensioning stresses into strain, and applying the external load on the beam. The bolt material used was ASTM A325-X, ASTM A36 used for the shear tab material, and finally, ASTM A572 Gr. 50 used for the column and beam material. The weld lines between the shear tab and the column web were modeled as continuous since the weld failure was not a critical failure mode. The author did not consider the shear-twist curves in the finite element analysis to determine the capacity of the connection since only one of the experimental tests has the twist as primary failure mode. The connection capacity was determined using the shear-displacement curves. These curves were obtained from FEA showed good agreement with the results obtained from the experiments. In order to determine ultimate shear capacities, yield points and failure modes of the connections, shear-displacement; shear-twist; and shear-rotation curves were investigated. It was observed that failure modes obtained from the FEA and experiments were the same. Rahman et al. concluded that the presented model in this study is a powerful tool in addressing the failure of the unstiffened extended shear tab connection in the plastic region.

Mahamid et al. (2007) addressed in detail the failure modes and analyses of the stiffened extended shear tab connection. Finite element models were created using ANSYS, these models were compared and verified with experimental study done by Sherman and Ghorbanpoor (2002). The author created models for eight different connection configurations: beam-to-column connections (2, 8, 10 and 12 bolts), and beam-to-girder connections (3, 6, 10 and 12 bolts). The author used four element types to model the connections: Pretensioning elements to model the bolt's pretensioning force, contact elements with surface-to-surface and flexible-to-flexible interaction properties to define the interaction between bolts-beam web holes; bolts-shear tab holes; and beam web- shear tab, eight node brick elements to model the beams; columns; girders; and shear tabs, three dimensional tetrahedral elements to model the bolts. Material properties of ASTM A36 was used for shear tabs, ASTM A572 Grade 50 for beams; columns; and girders, and A325-X for bolts. Additionally, bilinear curves were used to simulate the stress-strain responses by identifying the experimental results of yield strength, ultimate strength, and elongation. The twist of the connection plate, vertical displacement along the connection bolt line, shear load eccentricity relative to the connection bolt line, failure modes and nonlinearity were studied for each model so as to analyze the behavior of the stiffened extended shear tab connections. Moreover, the author observed good agreement between the finite element and experimental results, nevertheless, the zero strain locations were slightly off because of the use of the linear regression approach which is considered as approximate but acceptable method. Also, it was observed that the connection response becomes close to the rigid connections behavior as the number of bolts increases. Additionally, the twist was identified as secondary failure mode and detected at the bottom of the shear tab for the case where the beam framed into girder because of the fact that the tab is stiffened by the girder's top flange and not stiffened to the bottom flange.

Twist was also detected along the shear tab for the case of deep connections when the beam is framed into column. The author considered the nonlinearity as a crucial aspect in the extended shear tab connections' behavior since it helps in identifying the failure modes of these connections. Nonlinearity was implemented in the models using several parameters: the use of contact elements, material properties nonlinearity, and geometric nonlinearity. The author studied the nonlinear behavior by monitoring the plastic deformation and stresses at different locations in the connections, also by analyzing the shear-displacement curves. The author observed five different failure modes in these connections: bolt shear, shear yielding of the plate, bolt bearing, twist in the shear tab, and web mechanism in the girder web. Mahamid et al. concluded that his model is accurate and unique in examining the behavior of the stiffened extended shear tab connections.

Muir and Hewitt (2009) studied the behavior of unstiffened extended shear tab connections. They showed that these connections generate additional moment on the supporting member. This moment is not included in the design of the supporting members and should be considered in their design. Additionally, the background and development of the design of these connections in the AISC 13th edition (ASIC, 2005) was outlined. The authors highlighted that the plate in these connections must act as fuse and must yield before the bolts or weld failure. Additionally, they mentioned that plate buckling was not a primary failure mode. However, when extending the plate to the supporting girder bottom flange, the plate was more likely to buckle. Finally, the authors presented a general design procedure for the extended single-plate shear connections when the dimensional and other limitations of the conventional single-plate shear connection design method cannot be applied.

Marosi (2011) proposed a new design procedure applicable to single and double row bolted shear tab connections up to ten bolts per row for use in Canada. The author conducted the study since the design procedure for shear tab connections in the Canadian Institute of Steel Construction (CISC) handbook is limited in its applicability to one row of bolts up to seven bolts, and based on research performed in the 1980s. Sixteen full-scale tests were performed with different beam sizes, one and two rows of bolts were used and the number of bolts ranged between three to ten bolts per row. Ten retrofit weld tests and six bolted connections were conducted. In addition to the new proposed design procedure, the author concluded that the weld retrofit connections reached the target rotations and resist at least the same loads as their corresponding bolted connections were predicted to resist. The behavior at the onset of flexural and shear yielding for the welded and bolted connections was different; weld retrofit connections performed in the same manner as the bolted counterparts in single row tests but tend to outperform their bolted counterparts in double row tests.

Muir and Thornton (2011) outlined the revised design procedure for single-plate shear connections in the 14th edition AISC Manual (AISC, 2010). The author showed that before the manual (AISC, 2010) was released, an additional factor of safety was added for all bolted connections designed in accordance with the specification by reducing the nominal bolt shear values by 20% to account for uneven force distribution among the bolts in end-loaded connections. The authors mentioned that the design procedure for conventional single-plate shear connections contained in the AISC 13th edition Manual (AISC, 2010) relied on this reduction to justify the practice of neglecting eccentricity in the bolt group for most configurations. Muir and Thornton indicated that the design procedure in the AISC Manual 13th edition (AISC, 2010) was needed to be re-evaluated since neglecting the eccentricity is no longer appropriate because of the increase in the nominal bolt shear values provided in AISC's specification (AISC, 2010) for structural steel buildings. The author concluded that the new procedure represents an economical and safe method to the single-plate shear connections based on rational design methods and confirmed by testing.

Thornton and Fortney (2011) studied the lateral-torsional stability of the extended single plate connections and the effect of the small eccentricity caused by the overlap of the plate with the beam web, the authors suggested that the AISC Manual 13th edition (AISC, 2005) requires many design checks to guarantee adequate performance, but it does not include a check on the effect of the extended tab on lateral-torsional stability. In order to develop a new check to control the twisting of the plate limit state, the author investigated double coped beam to estimate the lateral torsional stability of beams with extended tabs, it was postulated that the uncoped portion of the beam can be treated as a rigid body and that lateral-torsional buckling is dependent solely on the coped section. Thornton and Fortney indicated that the need for stiffeners can be checked by evaluating the ratio of available shear to required shear, the author showed that the stiffeners are not required if the ratio is equal or greater than one. Moreover, they showed several examples using the proposed method to check for the need of stiffeners and to compare the proposed theory with other theories from literature. The authors suggested that the proposed theory is much simpler approach in that the only assumption required is that the ucoped portion of the beam acts like a rigid body. Additionally, the effect of lap splice eccentricity were studied, the authors indicated that the torsional moment produced by the lap splice eccentricity is resisted by two parts of the connection system: the torsional strength of the shear tab itself and the local torsional strength of the beam due to the floor slab or roof deck. Finally, a new theory was proposed to check the satisfactory of the connection against the small eccentricity caused by the overlap between the plate and the beam web.

Cui et al. (2012) investigated the ultimate strength of interface fillet weld connection between frame elements and gusset plate. The authors performed four tests on the gusset plate connections. Then, finite element models were generated using ABAQUS, these models were verified with the experimental results. After that, the authors performed a parametric study on these connections to investigate the effect of the brace angle, brace eccentricity, and gusset plate size on the gusset plate force distribution. The authors confirmed that the brace tensile force is transferred to the interface weld through the effective region. Additionally, it was proven that the effective interface length is affected by the aforementioned parameters. Moreover, the authors proposed a revised method to evaluate the ultimate strength of the gusset plate based on his modifications on the effective interface length.

Yim and Krauthammer (2012) studied the behavior of steel single plate shear connections subjected to monotonic, cyclic, and blast loadings by presenting mechanical models to define structural properties of these connections. The models were created using the finite element software ABAQUS. Results obtained from these models were verified with results obtained from experimental investigation and provided connection's moment-connection end rotation curves within 15 percent of the experimental results. The authors showed that the proposed models can be used for frame analyses are highly dependent on the connection properties using geometrical and material properties of individual components and configurations, and can predict the characterization curves for the single plate shear connections accurately without highly expensive cost and time efforts. Additionally, they showed that these models are useful for characterizing a large number of high-rise framed structure shear connections under various loading conditions.

Wen et al. (2013) proposed simplified connection models to facilitate structural analysis including steel gravity frames based on detailed finite element analysis and assumptions supported by experimental investigation observations and fundamental engineering judgment. The authors showed that the moment-rotation behavior is the same for bare steel shear tab connections under positive and negative moments. However, this behavior is different for composite steel shear tab connections under positive and negative moments. Additionally, the authors investigated the effect of steel gravity frames on the behavior of non-ductile concentrically braced frames. It was proven that the steel gravity frames using the proposed connection models increase the stability and delay the failure for non-ductile concentrically braced frames. The authors concluded that the shear tab in the bare steel shear tab connection is the major contributor to the flexural strength for positive and negative moment. On the other hand, the contributions from the concrete slab is insignificant and can be neglected in the case of negative moment for composite steel shear tab connections. The authors showed that the proposed model is able to capture the initial stiffness, yielding strength and ultimate strength for bare connection. But for the composite connections, there is a deviations in the initial stiffness. However, the overall behavior is in acceptable ranges.

Suleiman et al. (2013) studied the plate twisting in extended shear tab connections. The authors mentioned that some of unstiffened connections with five or more bolts failed due to plate twisting in an experimental study done by Sherman and Ghorpanboor (2002). However, the effect of the floor slab was not taken into consideration. Additionally, the authors suggested that including this effect might change the failure modes of these connections. The authors used the finite element software ABAQUS to create three dimensional nonlinear models for these connections in order to predict the failure modes and determine the importance of including the floor slab effect. An eight node, reduced integration brick elements were used for the entire model except the shear tab where twenty node, reduced integration brick elements were used. General contact with surface-to-surface option and 0.3 friction coefficient was used to define the interaction between bolts, bolt holes, the plate, and beam web. Additionally, tie connection was used to simulate the welding between the plate and column flange. Geometric and materials nonlinearities were included in the analysis to capture the failure modes for these connections. Also, bolts pretensioning forces were simulated by imposing temperature gradient to bolts shanks. Then, the model meshing was checked to ensure that the model does not have any poor elements with high aspect ratios. The authors compared the results obtained from the Finite Element Analysis (FEA) with results from the experimental study in order to verify the models. FEA results showed good agreement with experimental results. Then for same models, the lateral translation movement of the supported beam compression flange was restricted to simulate the effect of the floor slab. It was concluded that the plate twist was reduced significantly when including the effect of the floor slab and the plate twist was not a primary failure mode any more. Additionally, the connection shear capacity was not affected by including the floor slab.

Wen et al. (2014) investigated the inelastic behavior of shear tab connections analytically using the finite element software ABAQUS. Models were created for shear tab connections with and without concrete slab. Different solution strategies (newton method and explicit dynamic method) were used to optimize the solution accuracy. The finite element results were verified with a reliable experimental investigation. The authors showed that the proposed models are helpful in understanding the behavior of shear tab connections at a micro level including normal stress distribution along the concrete slabs and shear tabs, and neutral axis development along the beam sections. The author concluded that shear tab connections with composite beams have unsymmetrical behavior under positive and negative bending moments. On the other hand, bare shear tab connections have the same behavior under negative and positive bending moment, and the shear tab contributes significantly in resisting bending moments applied to these connections.

Abou-zidan and Liu (2015) investigated analytically the behavior of the unstiffened extended shear tab connections using the finite element analysis software ANSYS. A parametric study was performed to study the effect of distance between the weld line and the center of the bolt lines, number of bolts, plate thickness, and the supporting member web slenderness ratio. The author used three dimensional eight-node structural solid element for the beam, column, shear tab and bolts, and 4-node quadrilateral surface to surface contact elements to define the interaction between bodies in contact. The models had fine mesh at areas that are expected to have a high level of stress and coarse mesh at areas that are expected to have a low level of stress. Additionally, elasto-perfect plastic material model was used for the column, beam and shear tab. On the other hand, the real stress-strain curve obtained experimentally was used to model the bolts material properties. The analysis done in two steps, in the first step, pretensioning force was applied for each bolt to simulate the snug tightened condition. In the second step, the actuator load was applied as distributed pressure load on the top flange of the beam at a location that will give connection shear and beam end rotation similar to the case of uniformly loaded beam. The author concluded that the AISC design procedure is overly conservative in predicting the bolt shear fracture strength for extended shear tab connections with single line of bolts and number of bolts varies between 2 and 6 bolts. On the other hand, the AISC procedure reasonably estimates the bolt shear fracture strength for connections with bolts more than 6 bolts. Also, the authors showed that the estimation of bolt shear strength was improved when the AISC design procedure was used in combination with the finite element analysis.

1.3. Problem Statement

The use of extended shear tab connections for skewed members (Figure 1-3) eliminates the need for modifications and excessive cutting of beam ends which makes these connections attractive to engineers and fabricators. There have been high demands on using these connections to accommodate the complex geometries in modern structures. However, there is no provisions for such connections yet. Skewed connections have been in the manual for many years and their design is based on practical recommendations only. Thus, the behavior of these connections is still not fully understood and more research should be conducted to develop a specific design procedure for these connections.

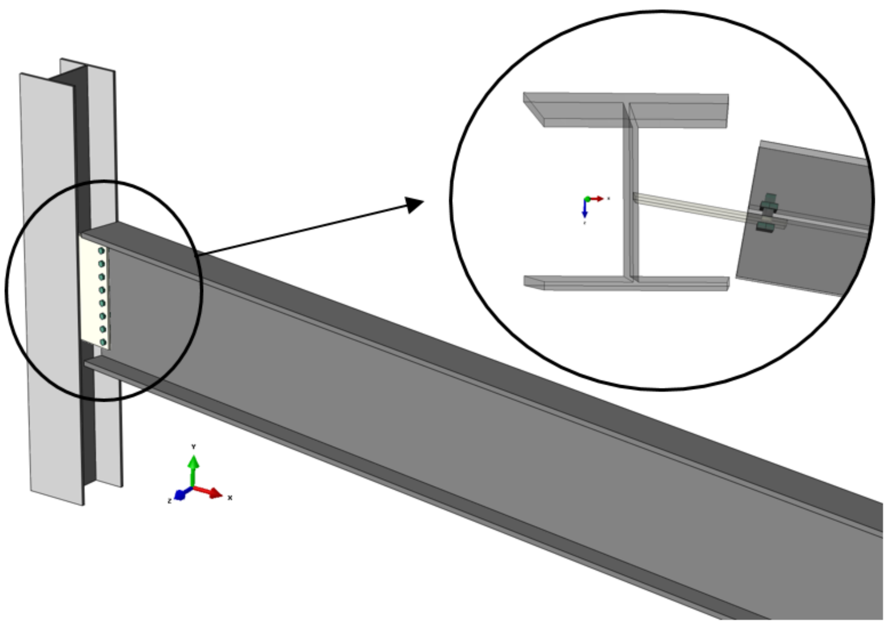


Figure 1-3. Skewed extended shear tab connection

1.4. Research Objectives and scope

This research has been performed in order to achieve the following objectives:

1. Achieve better understanding of the skewed extended shear tab connections behavior to identify all the possible limit states that govern the failure of these connections.

2. Develop a finite element analysis procedure that is capable of predicting the shear capacity and capturing the failure modes of these connections.
3. Verify the experimental results obtained by other researches and investigate the effect of any additional failure modes.
4. Study the effect of the connection orientation on the connection bending and torsional capacities.
5. Investigate the effect of the supporting member rigidity (flexible or rigid) on the connection performance.
6. Investigate the effect of adding stiffeners on the connection performance.
7. Study the interaction between the connection orientation with other parameters (shear tab thickness, number of bolts, and a-distance) in terms of connection shear capacity, shear tab twist along the weld line, shear tab twist along the bolt line, and connection vertical displacement.
8. Derive equations that relate the shear tab twist and connection vertical displacement for skewed and orthogonal extended shear tab connections.
9. Propose modifications to the current design procedure to consider the effect of the connection orientation.

1.5. Thesis Organization

The thesis is divided into eight chapters. Chapter two summarizes the experimental investigation done by other researches on the orthogonal extended shear tab connections in which their results were used as reference to validate the Finite Element Analysis (FEA) results. The two studies included in this chapter are: AISC final report done by Sherman and Ghorpanboor (2002) under the title of “Extended Shear Tabs”, and a master thesis submitted by Kirsten Metzger (2006) under the title of “Experimental Verification of a New Single Plate Shear Connection Design Model”.

Chapter three describes in details the process of creating the finite element models for the orthogonal configuration using the FEA software ABAQUS. This chapter includes a description of: Element selection, material properties, contact properties, boundary conditions, loading, meshing, and processing.

Chapter four includes the validation of the FEA results with the experimental investigation results for the orthogonal extended shear tab connections. This chapter is divided into three sections: comparing the connection shear-connection vertical displacement curves, connection shear-beam end rotation curves, and comparing the failure modes.

Chapter five is divided into two sections. The first section describes in details the proposed skewed extended shear tab connections parameters and the modeling process of these connections. The second section shows a comparison between the behavior of skewed and orthogonal configurations for: unstiffened extended shear tab connections with the plate welded to the supporting member web (flexible support), stiffened extended shear tab connections with the plate welded to the supporting member web, and unstiffened extended shear tab connections with the plate welded to the supporting member flange (rigid support).

Chapter six shows the procedure of the parametric study that is performed on orthogonal and skewed extended shear tab connections. The interaction of different parameters (connection orientation, plate thickness, a-distance, and number of bolts) and their effect on the connection behavior is investigated. This chapter includes a description of the constant and variable parameters that have been used in the parametric study, discussion, and analysis of the obtained results.

Chapter seven includes the derivation process for the proposed equations that relate the plate twist, connection vertical displacement, and connection shear capacity for the skewed and orthogonal extended shear tab connections. Moreover, this chapter shows the proposed design procedure and modifications on the current design procedure to account for the connection orientation effect.

Chapter eight shows summary of results, and outlines the conclusions and observations were found in this research. Also, future research that can be done by other researchers to have better understanding of the behavior of these connections is included in this chapter.

2. CHAPTER TWO – PRIOR EXPERIMENTAL WORK

The methodology used in the research is finite element analysis. Finite element models were created for the skewed extended shear tab connections. The finite element results were compared with experimental results made available by other researchers to validate the models. Two experimental studies investigated the behavior of orthogonal extended shear tab connections were selected as a reference to generate and validate the finite element models. This chapter provide a brief discerption of the experimental work performed by other researchers:

2.1. Sherman and Ghorpanboor (2002)

Sherman and Ghorpanboor (2002) conducted this research to study the behavior of the orthogonal extended shear tab connections. The purpose of this study was to develop a specific design procedure for these connections. Thirty-one (31) full-scale tests were performed in three phases. The results of this research formed the basis of the design procedure for the extended configuration of the single-plate connections in AISC Manual 13th edition (AISC, 2005).

2.1.1. Beams, Girders, and Columns Selection

The author selected beams based on the span-to-depth (L/d) ratio. A formula was derived to determine the location of the loading point on the beam to give the same end reaction and rotation as the uniformly distributed load case. For Phases one and two, two beams were selected, W12x86 (L/d = 23) as a flexible beam, and W18x71 (L/d = 10) as a stiff beam. For Phase three, W24x146 (L/d = 14) and W30x148 (L/d = 11) were selected as stiff beams. The supporting members were selected based on the width-to-thickness ratio (h/t_w). Four girders and two columns were selected with h/t_w ratios vary between 22 and 54. Additionally, all columns and girders were 8 ft. and 10 ft.

long, respectively. Figure 2-1 shows the location of the supports applied on the supporting members. Also, all supported beams and supporting members were ASTM A572 Grade 50 steel.

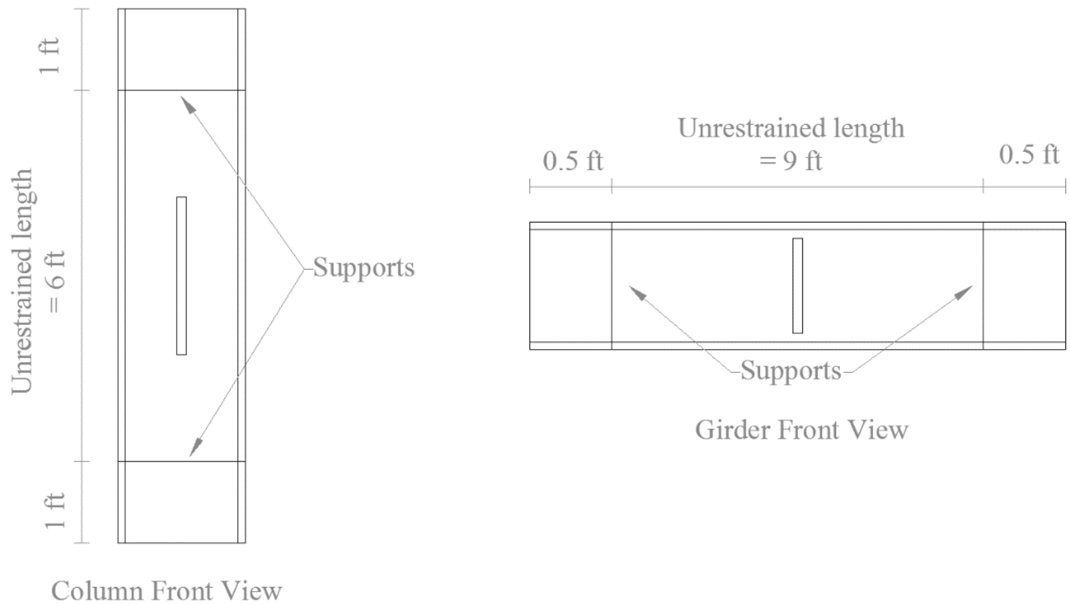


Figure 2-1. Supports location for columns and girders

2.1.2. Shear Tab Dimensions

For the unstiffened connections tests, the shear tab thickness was chosen based on the stability criteria in the AISC Manual 2nd edition for single plate shear connections and the stability limit states under the anticipated maximum shear where the plate thickness is selected to prevent local buckling of the plate. Additionally, short slotted holes were used and the weld size was equal to ¾ the plate thickness to make sure that the weld will not fail prior the shear tab yielding. Based on this criterion, 3/8 in. and 1/2 in. shear tab thicknesses with 5/16 in. fillet weld were used for the unstiffened connections tests. For stiffened connections, the shear tab thickness was selected based on the stability criteria in the AISC Manual 2nd edition for single plate shear connections only, since the distance between the end of the horizontal weld lines and the bolt line was only 3 inches. Three thicknesses were used for the stiffened connections: 1/4, 3/16, and 3/8 inches.

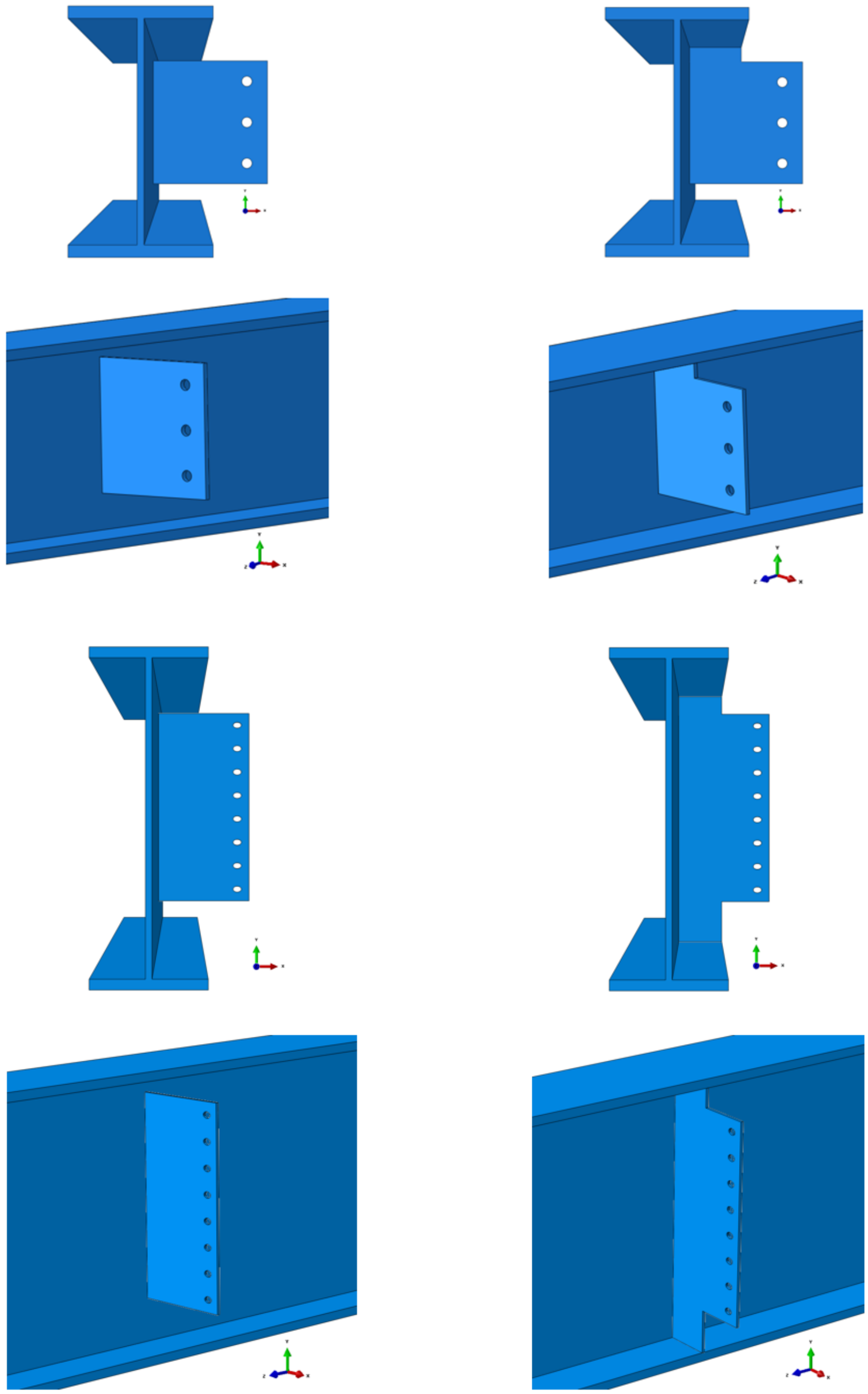
2.1.3. Experiment Phases

In phase one, seventeen tests were conducted; four of these tests were unstiffened connections where two connections had the shear tab welded to a supporting column web and the other two connections had the shear tab welded to a supporting girder web. For the stiffened connections, thirteen connections were tested, eight tests had the shear tab welded to a supporting column web and to the stiffener plates, and the other five connections had the shear tab welded to a supporting girder's web and to the top flange. The objective of this phase is to investigate the behavior and capacity of unstiffened and stiffened orthogonal extended shear tab connections as a function of the number of bolts (3 and 5 bolts), type of bolt holes (Standard or Short Slotted Holes), size of the shear tab, width-to-thickness ratio of the supporting member web, span-to-depth ratio of the supported beam, and lateral bracing of the supported beam. Moreover, special investigations were performed to determine the effect of other parameters on the connection behavior. These parameters are: Removing the vertical weld between the shear tab and the supporting column web, large a-distance, welding the stiffener plates to the supporting column web, and welding the shear tab to the girder's web from one side.

In Phase two, four tests were conducted, in all of these tests the shear tab was welded to a supporting column web. There was only one unstiffened shear tab in this category. The goal of this phase was to investigate the effect of using snug tight bolts instead of fully tightened bolts in short slotted holes, the effect of using one stiffener plate on the top of the shear tab instead of two, and to study the behavior of the stiffener plates. Figures 2-2 and 2-3 show the unstiffened and stiffened orthogonal extended shear tab connections with the shear tab welded to the supporting girder and supporting column web, respectively.

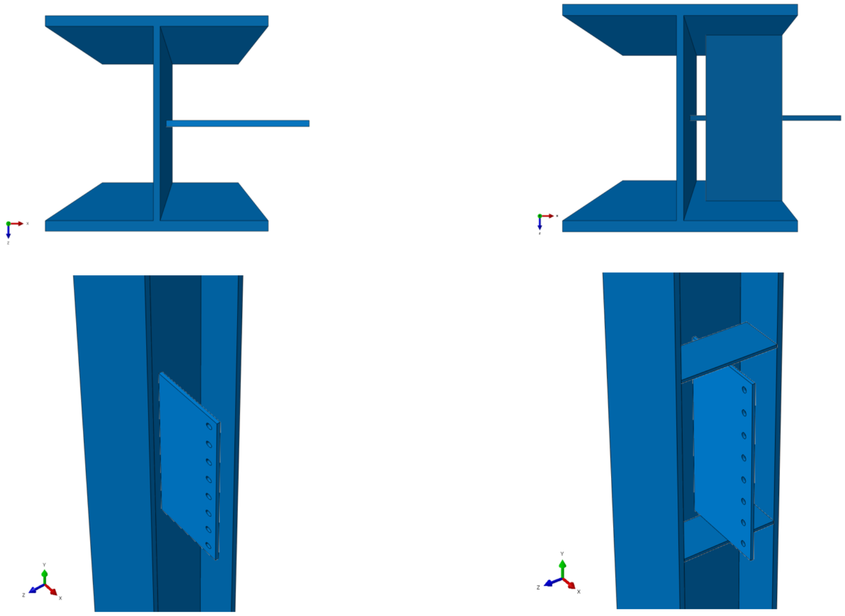
In Phase three, the researcher investigated the behavior of deep connections with six and eight bolts. Ten tests were conducted, six of these connections had the shear tab welded to a supporting column web with and without stiffener plates. The other four connections had the shear tab welded to a girder web. Two of these tests had the shear tab extended and welded to the top flange of the girder, the shear tab for the other two connections were extended and welded to the top and bottom girder flanges.

Table 2-1 shows a summary of the experimental program including the three phases that was investigated by Sherman and Ghorbanpoor. Additionally, Tables 2-2 and 2-3 show the test configurations and geometries of the selected unstiffened and stiffened connections, respectively, that have been used for as a reference for this thesis.



Unstiffened Configuration Stiffened Configuration

Figure 2-2. Unstiffened and stiffened configurations with supporting girder



Unstiffened Configuration Stiffened Configuration

Figure 2-3. Unstiffened and stiffened configurations with supporting column

Table 2-1. Summary of Sherman-Ghornbanpoor Experimental Tests

Phase 1 (17 tests)				Phase 2 (4 tests)		Phase 3 (10 tests)			
Unstiffened (4 tests)		Stiffened (13 tests)		Unstiffened (1 tests)	Stiffened (3 tests)	Unstiffened (3 tests)		Stiffened (7 tests)	
Column (2 tests)	Girder (2 tests)	Column (8 tests)	Girder (5 tests)	Column (1 test)	Column (3 tests)	Column (3 tests)	Girder (0 test)	Column (8 tests)	Girder (5 tests)
3-U (3 Bolts)	1-U (3 Bolts)	3-A (3 Bolts)	1-A (3 Bolts)	3-UM (3 Bolts)	3-F (3 Bolts)	6-U (6 Bolts)		6-B (6 Bolts)	5-A (6 Bolts)
4-U (5 Bolts)	2-U (5 Bolts)	3-B (3 Bolts)	1-B (3 Bolts)		3-G (3 Bolts)	6-UB (6 Bolts)		8-A (8 Bolts)	5-B (6 Bolts)
		3-C (3 Bolts)	2-A (5 Bolts)		3-H (3 Bolts)	8-U (8 Bolts)		8-B (8 Bolts)	7-B (8 Bolts)
		3-D (3 Bolts)	2-B (5 Bolts)						7-C (8 Bolts)
		3-E (3 Bolts)	2-C (5 Bolts)						
		4-A (5 Bolts)							
		4-B (5 Bolts)							
		4-C (5 Bolts)							

Table 2-2. Sherman-Ghornbanpoor unstiffened tests configurations and geometries

Test Designation	L_p (in)	t_p (in)	# of Bolts	d_b (in)	a (in)	L_{eh}, L_{ev} (in)	Supporting Member	Test Beam	Span (ft)	Lateral Bracing
1U	9	3/8	3	3/4	6.85	1.5	W14X53 Girder	W12X87	30	NO
3U	9	3/8	3	3/4	6.86	1.5	W8X31 Column	W12X87	30	NO
4U	15	1/2	5	3/4	1004	1.5	W14X90 Column	W18X71	20	NO
6U	18	1/2	6	3/4	10.04	1.5	W14X90 Column	W24X14	33	YES
6UB	18	1/2	6	3/4	10.04	1.5	W14X90 Column	W24X14	33	NO
8U	24	1/2	8	3/4	10.04	1.5	W14X90 Column	W30X14	33	YES

Where:

L_p = length of the plate

t_p = thickness of the plate

a = distance between the weld line and bolt line

L_{eh} = the horizontal edge distance

L_{ev} = the vertical edge distance

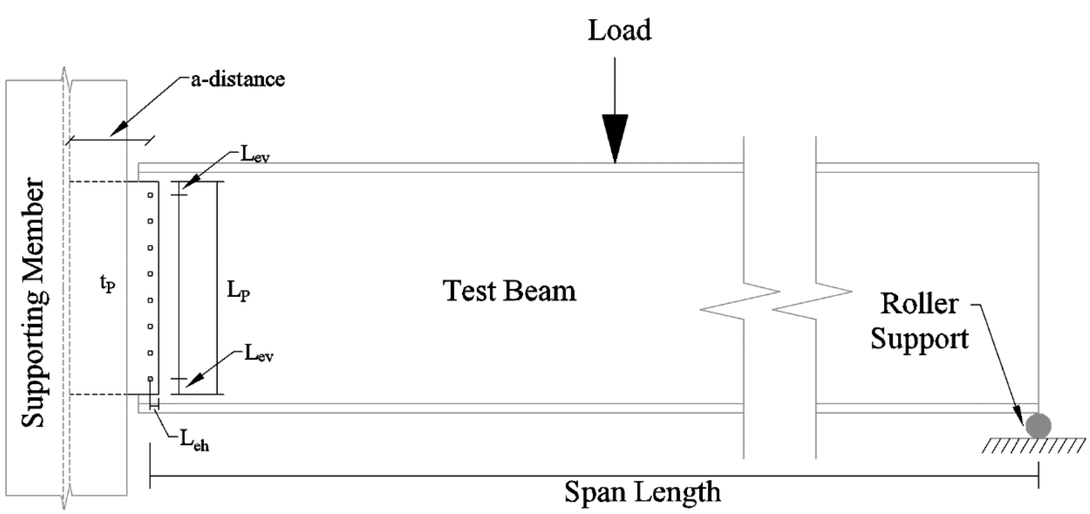


Figure 2-4. Geometric properties of Sherma-Ghorpanboor connections

Table 2-3. Sherman-Ghornbanpoor stiffened tests configurations and geometries

Test Designation	L_p (in)	t_p (in)	# of Bolts	Bolt Hole Type	a (in)	Weld Configuration*	Supporting Member	Test Beam	Span (ft)	Bracing
3A	9	1/4	3	STD***	5.91	W-T-B	W8x31 Column	W12x87	30	NO
3B	9	1/4	3	SSL***	5.91	W-T-B	W8x31 Column	W12x87	30	NO
3D	9	1/4	3	STD	5.91	W-T-B**	W8x31 Column	W12x87	30	NO
4A	15	1/4	5	STD	8.25	W-T-B	W14x90 Column	W18x71	20	NO
4B	15	1/4	5	SSL	8.25	W-T-B	W14x90 Column	W18x71	20	YES
5A	18	5/16	6	SSL	9.27	W-T-B	W30x173 Girder	W24x146	33	Lateral & Rotational
6B	18	5/16	6	SSL	8.66	W-T-B	W14x90 Column	W24x146	33	Lateral & Rotational
7B	24	5/16	8	SSL	8.01	W-T	W33x152 Girder	W33x148	33	Lateral & Rotational
7C	24	5/16	8	SSL	7.76	W-T-B	W33x152 Girder	W33x148	33	Lateral & Rotational
8B	24	3/8	8	SSL	8.93	W-T-B	W14x90 Column	W30x148	33	Lateral & Rotational

* Weld Configurations: W = Web; T = Top; B = Bottom

** Stiffening Plates Welded to Column Web

*** STD = Standard Holes, SSL = Short Slotted Holes

2.2. Metzger (2006)

Metzger performed eight experiments, four conventional shear tab connections and four extended shear tab connections. Since the purpose of this research is to investigate the behavior of the extended shear tab connections, only the four extended connections were studied and modeled.

2.2.1. Tests Configurations and Geometries

The requirements of the AISC 2005 specification and design procedure in the AISC 13th edition manual (AISC, 2005) were used to design these connections. All bolt holes were standard holes with 1.25 in. and 1.5 in. vertical (L_{ev}) and horizontal (L_{eh}) edge distances, respectively. In order to prevent brittle failure of the connections, the shear tabs were designed to a moment capacity less than the moment capacity of the bolt group. Additionally, the weld size used in these connections was equal to half the thickness of the shear tab. Table 2-4 and Figure 2-5 show the details of the extended connections tested by Metzger (2006).

2.2.2. Tests Setup

Each test consisted of an extended plate welded from one side to the column flange in such a way that the plate's longitudinal axis and column's weak axis align; the other side of the plate was bolted to the supported beam. The supported beam was supported on the far side by a simple roller support. Two hydraulic rams were placed on top of the beam flange to control the shear and rotation imposed on the connection. Additionally, braces were placed along the beam test to prevent lateral torsional buckling by using angles bolted to the beam web and extend between the beam flanges. Moreover, the four extended plates were welded to the same column, two on each side (Figure 2-5). In order to provide sufficient bracing for the column, a channel was bolted to the testing frame columns and to the test column (Figure 2-6). The goal of the tests was to drive the connection up to failure and reach a beam end rotation of 0.03 radians at the same time by imposing a combination of shear and rotation on the connection. Figures 2-5 and 2-6 demonstrate the test setup of Metzger's extended connections.

Table 2-4. Metzger tests configurations and geometries

Test	Bolt Columns	Bolts Rows	t_p (in)	a-distance (in)	Beam Section	Beam Length (in)	Column Section
6B2C - 4.5 -1/2	2	3	1/2	4.5	W18x55	223	W21x62
10B2C - 4.5 - 1/2	2	5	1/2	4.5	W30x108	295	W21x62
7B1C - 9 - 3/8	1	7	3/8	9	W24x62	275	W21x62
10B2C - 10.5 - 1/2	2	5	1/2	10.5	W24x62	275	W21x62

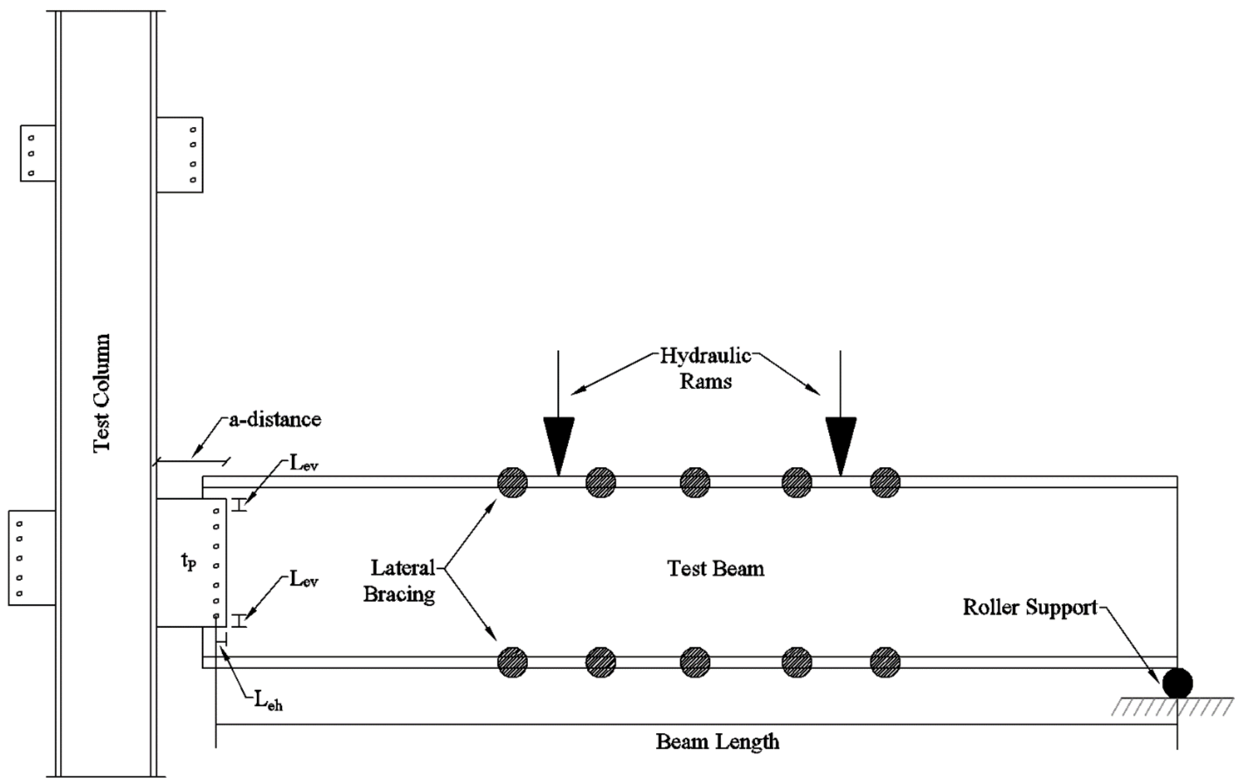


Figure 2-5. Geometric properties of Metzger connections

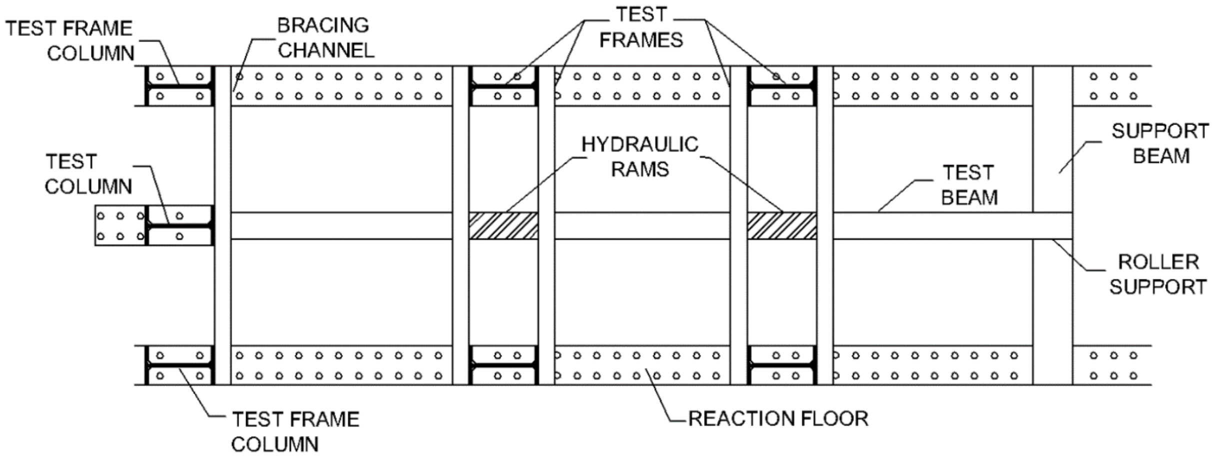


Figure 2-6. Plan view of Metzger test setup (Metzger, 2006)

3. CHAPTER THREE – MODELING

Three-Dimensional finite element models were created for extended shear tab connections tested experimentally by Sherman-Ghorbanpoor (2002) and Metzger (2006) using the finite element software ABAQUS 6.13-2 (ABAQUS, 2013). Results obtained from the finite element analysis were compared with the experimental results mentioned (Sherman-Ghorbanpoor, 2002) and (Metzger, 2006) for validation. The content of this chapter was previously published as "Behavior of Skewed Extended Shear Tab Connections" in the 2015 Structures Congress (2015), and published as "Behavior of Stiffened Skewed Extended Shear Tab Connections" in the 2016 Geotechnical and Structural Engineering Congress (2016).

3.1. Element Selection

A three-dimensional eight-node-brick element reduced integration with hourglass control (named as C3D8R in ABAQUS library) was used for the entire model, the active Degrees Of Freedom (DOFs) for this element are the translational DOFs. Furthermore, the integration points were reduced to one point at the center of the element in order to prevent elements locking at boundaries and to make the model more flexible in order to capture the inelastic behavior of the connection.

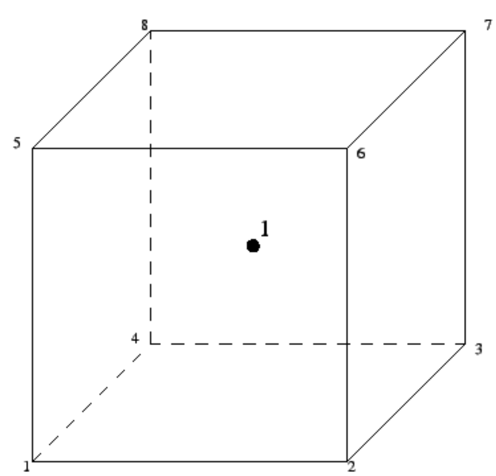


Figure 3-1. Three-dimensional eight-node brick element, reduced integration (C3D8R)

3.2. Material Properties

3.2.1. Bolts Material Properties

To simulate the materials nonlinearity, elastic perfectly-plastic constitutive relationship was used to model the bolts. The same bolts (A325-X) were used as the experiments done by Sherman-Ghorpanboor (2002) and Metzger (2006). The modulus of elasticity (E), Poisson's ratio (ν) and ultimate strength (σ_u) used are 29,000 ksi, 0.3 and 120 ksi, respectively. The ultimate strength (σ_u) for A325-X bolts used was taken from a report by Kulak (2001) that studied the mechanical properties of different bolts and rivets types. Figure 3-2 shows coupon stress versus strain relationships for different fastener materials done by Kulak (2001).

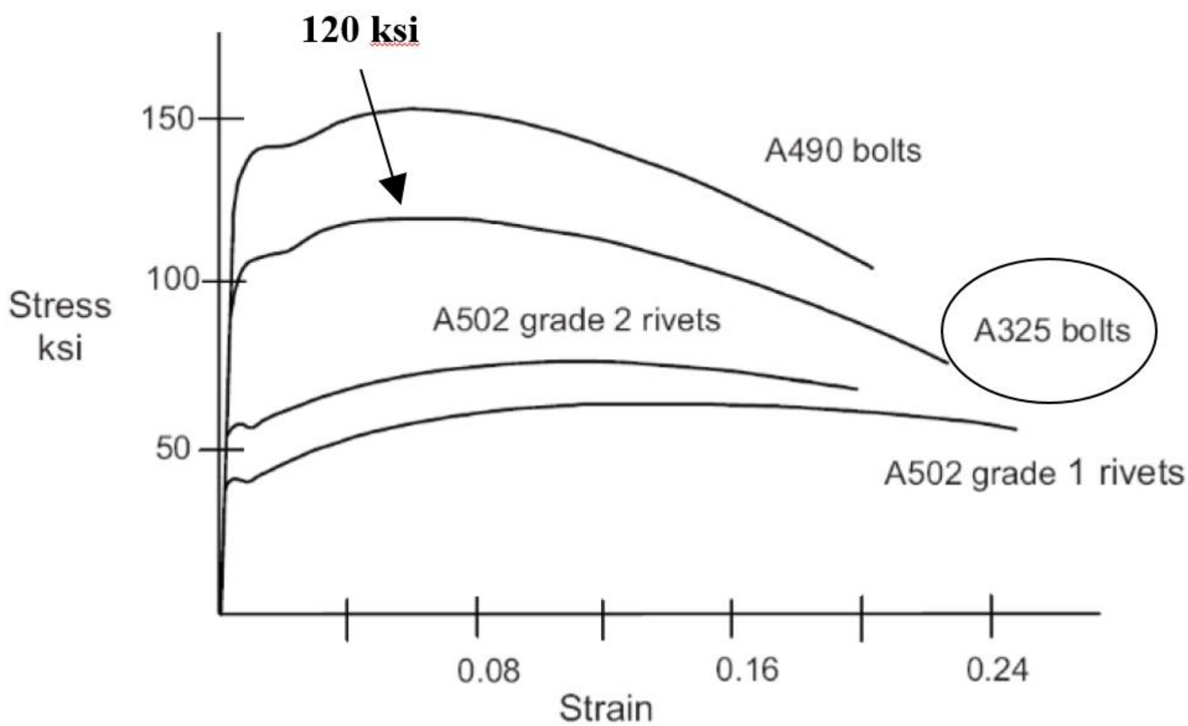


Figure 3-2. Stress-Strain diagrams of coupon tests for rivets and bolts (Kulak 2001)

3.2.2. Shear Tabs, Beams, and Columns Material Properties

The elastic-plastic with linear hardening (bilinear) model was used to model shear tabs, supported beams and supporting members. Table 3-1 shows the material properties obtained by Sherman-Ghorbanpoor (2002) and Metzger (2006) for the shear tabs and members, the same properties were used in the finite element models.

Table 3-1. Material Properties

Member	E, ksi	ν	σ_y, ksi	σ_u, ksi	% Elongation
Sherman and Ghorbanpoor					
3/8in. TAB	29,000	0.26	42.6	66.5	34
1/2in. TAB	29,000	0.26	40.5	63.6	36
W14X53	29,000	0.3	54.2	70.8	38
W24X55	29,000	0.3	55.1	70.1	38
W8X31	29,000	0.3	55.2	75.3	31
W14X90	29,000	0.3	56.7	71.7	37
Metzger					
3/8" TAB	29,000	0.26	69.3	96.3	20
1/2" TAB	29,000	0.26	68.2	97.7	22
W18X55	29,000	0.3	58.9	77.6	27
W24X62	29,000	0.3	58	77.1	27
W30X108	29,000	0.3	61.5	79.3	31

3.3. Contact Properties

Defining interaction between different model parts is an important step in the analysis and is considered vital for solution convergence problems and results; improper use of contact surfaces and properties may lead to unrealistic results. Two contact types were used in the model:

3.3.1. Surface-to-Surface Interaction

This Contact type was used between bolt shank and bolt hole, bolt head and shear tab, and bolt nut and beam web. A *friction coefficient* of 0.3 (commonly used in steel-on-steel contact) is used to define the contact behavior. *Hard contact* that *allow separation after contact* options were used to simulate contact between bolts and bolt holes, and beam web and plate surfaces. Figure 3-3 shows surface-to-surface interaction between bolts and bolt holes. The bolt head, shank and nut were defined as master surfaces. While slave surfaces were assigned to the beam and plate since the bolt is stiffer than the plate and beam.

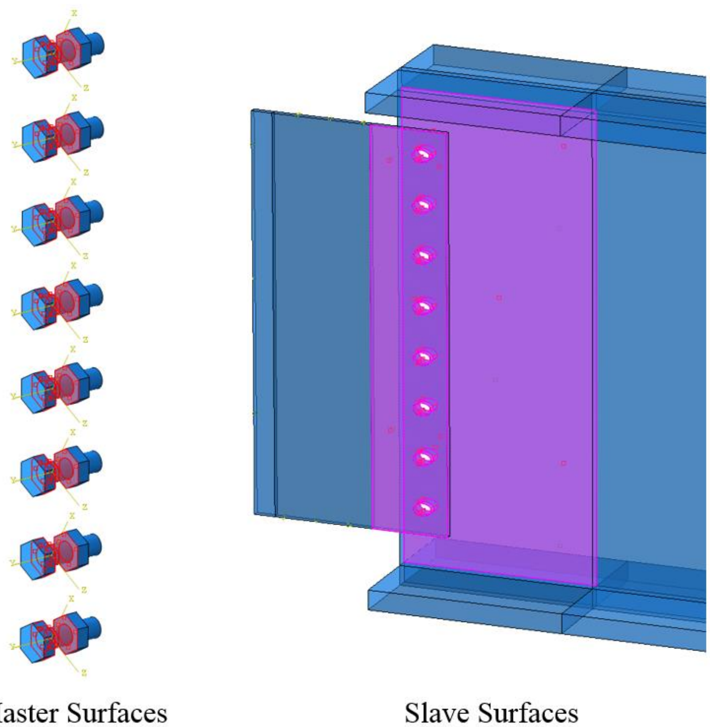


Figure 3-3. Surface-to-surface interaction between the bolts, plate and beam web

3.3.2. Tie Constraint

The second contact type used was tie constraint to simulate welding between the supporting member and plate. The supporting member considered as the master surface, while the plate side was considered as the slave surface. Thus, the degrees of freedom on slave surface nodes will follow the degrees of freedom on master surface nodes. Figure 3-4 shows the tie constraint between the plate and supporting member web.

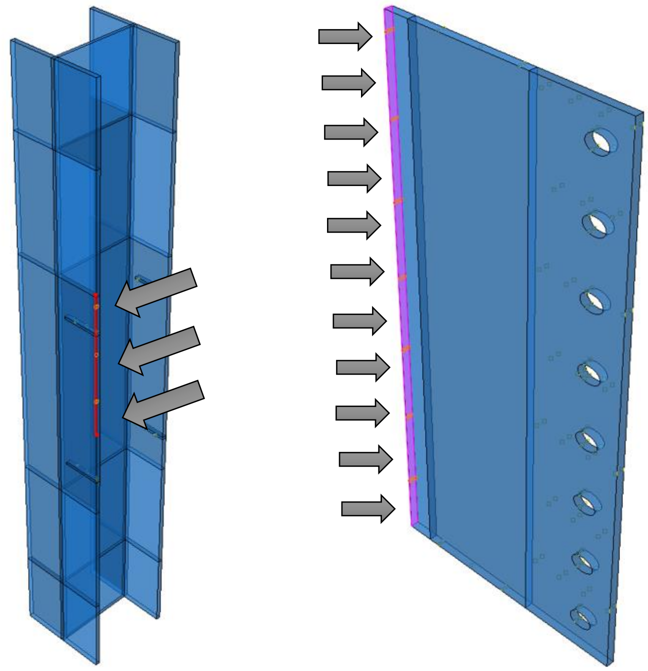


Figure 3-4. Tie constraint between the supporting member web and plate

3.4. Boundary Conditions

Applying accurate boundary conditions in the model is a key to achieve accurate results. Three types of boundary conditions were applied to the model: pin supports at the supporting members, roller support at the beam far end, and lateral bracing at specified locations along the beam length. These boundary conditions are the same as used in the experimental tests done by Sherman-Ghorpanboor (2002) and Metzger (2006).

3.4.1. Pin Supports

Pin supports were applied to the supporting columns and girders at the same location as the experimental tests. Figure 3-5 shows the supports location for the connections supporting members. For the pin support, the translational degrees of freedom about x, y, and z axes are restrained.

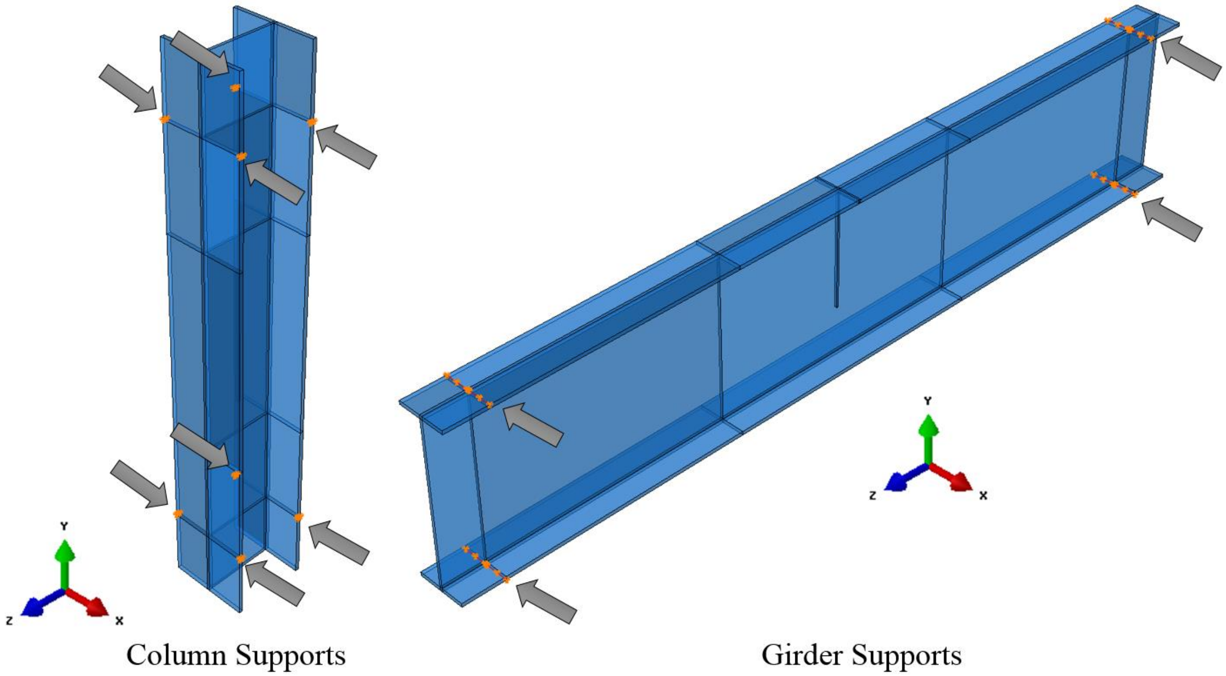


Figure 3-5. Pin supports location on the supporting members

3.4.2. Roller Supports

For all the models, roller support was applied on the beam far end to simulate the behavior of simply supported beam (Figure 3-6). For the beam far end, the lateral (z) and vertical (y) DOFs of the beam bottom flange nodes were restrained.

3.4.3. Lateral Bracing

The lateral DOFs along the beam flanges were applied to simulate the lateral bracing systems used in the experiments by Sherman-Ghorpanboor (2002) and Metzger (2006) to prevent lateral torsional buckling of the beam during the tests. The lateral bracing was applied by restraining the lateral (z) DOFs at the same location as the experiments. Figure 3-6 shows the location of the lateral bracing applied on one of the models.

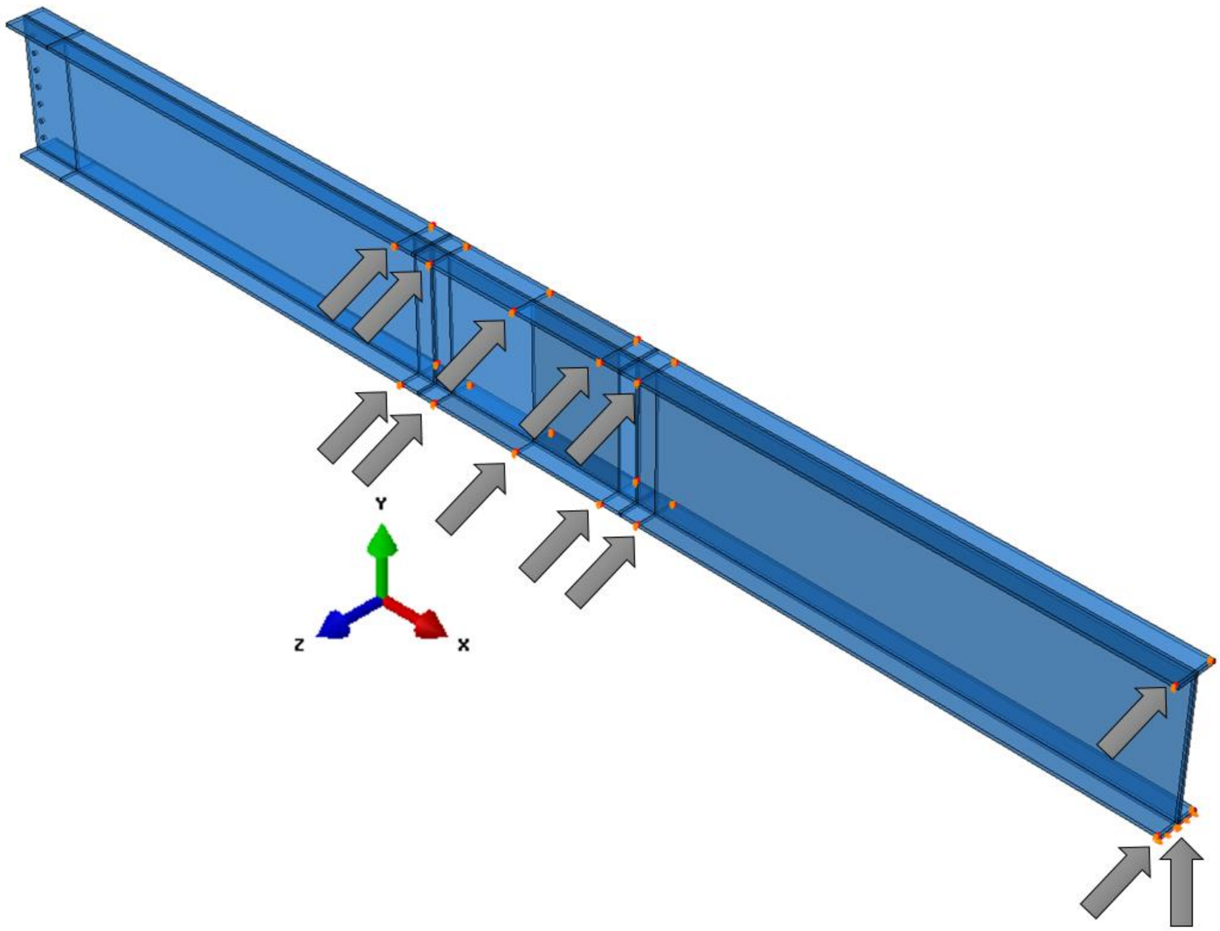


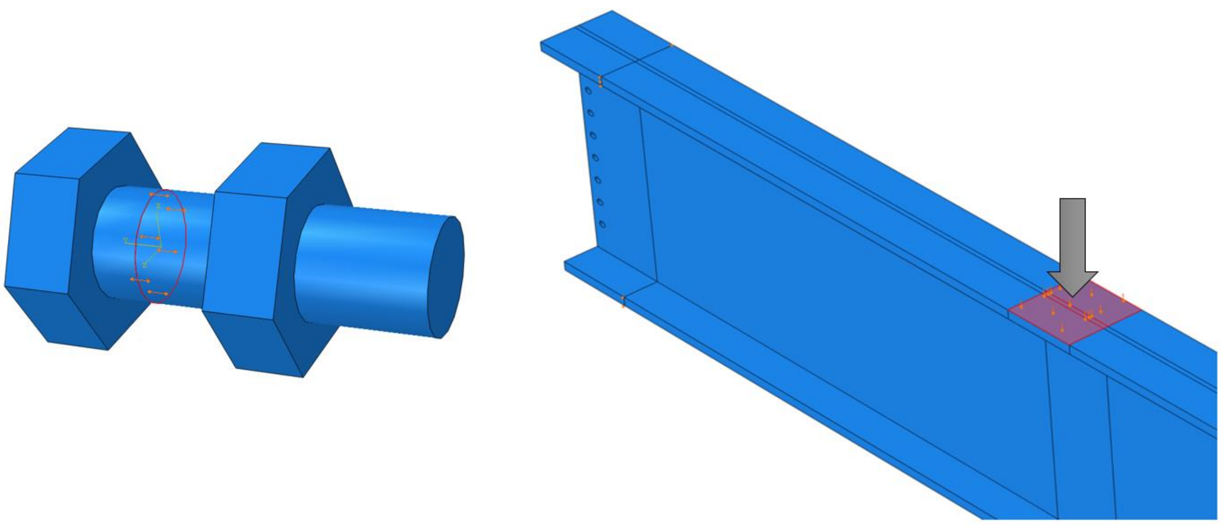
Figure 3-6. Roller support and lateral braces location

3.5. Loading

The next step in the modeling process was defining and applying loads on the beams, two types of loading were applied: Bolt pretensioning force, and applied external load (Figure 3-7). Bolt pretensioning force is important to keep the beam web and plate in contact under the applied external loads, this load was applied at each bolt using *bolt force option*. The minimum bolt pretensioning force from the AISC manual is 28 kips for ¾ in. diameter A325 bolts (group A) which is the same value that was used in the experiments done by Sherman-Ghorpanboor (2002) and Metzger (2006). The external forces applied on the connection using actuators were applied to the models as equivalent pressure load on the top flange at the same loading point as the experiments. The location of these loads were selected to produce the same reaction and rotation at the beam ends as the case were the beam is loaded with uniformly distributed load, which represents the actual loading case in beams (Sherman-Ghorpanboor, 2002).

3.6. Loading Steps

Boundary conditions, bolts pretensioning forces, and applied external loads were applied in three deferent time steps to simulate the experiments and for convergence purposes. The initial time step used is to define the boundary conditions in the model; the second time step used is to apply bolt pretensioning force to each bolt to initiate the contact between the beam web and plate. Finally, the external load is applied after establishing the contact surfaces and boundary conditions in the models. Figure 3-7 shows the pretensioning force in the bolts and the applied external load. The initial step is predefined by ABAQUS, while general static step was used for the second and third steps. Additionally, the time used is 1 second with $0.01, 1\times10^{-9}$ and 0.1 seconds as initial, minimum and maximum increment size, respectively.



Bolt Pretentioning Force   Applied External Load

Figure 3-7. Bolt pretensioning force and the applied external load

3.7. Meshing

All parts were chosen as dependent parts in order to perform meshing at a part level. Part partitioning in ABAQUS was used to discretize the model into small elements. Fine mesh was used at regions that are expected to have a high stress levels such as the plate, contact area between the plate and supporting member, and contact area between bolts and beam web and bolts and plate. Coarse mesh was used at regions that are expected to have a low stress levels such as beam far end, the top and bottom of columns, and ends of girders. Finally, the mesh quality was checked using *Verifying Mesh* option in ABAQUS, this option detects if there is any poorly meshed elements and gives warnings/errors for elements with poor aspect ratios. All models passed this check with no warnings or errors.

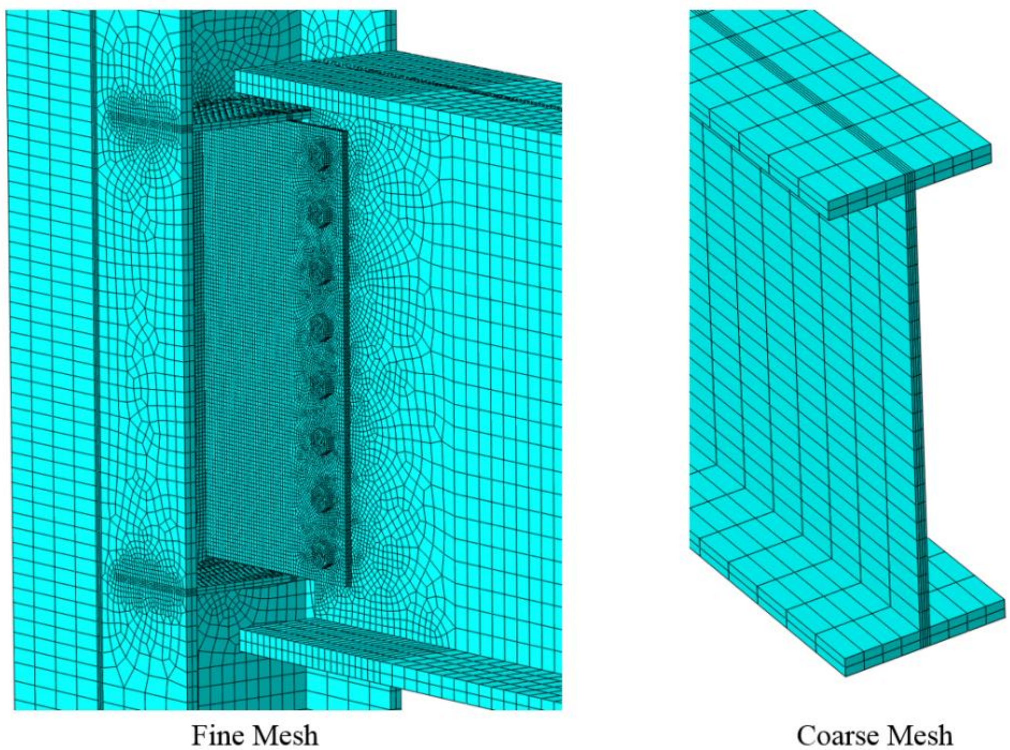


Figure 3-8. Fine and coarse meshing

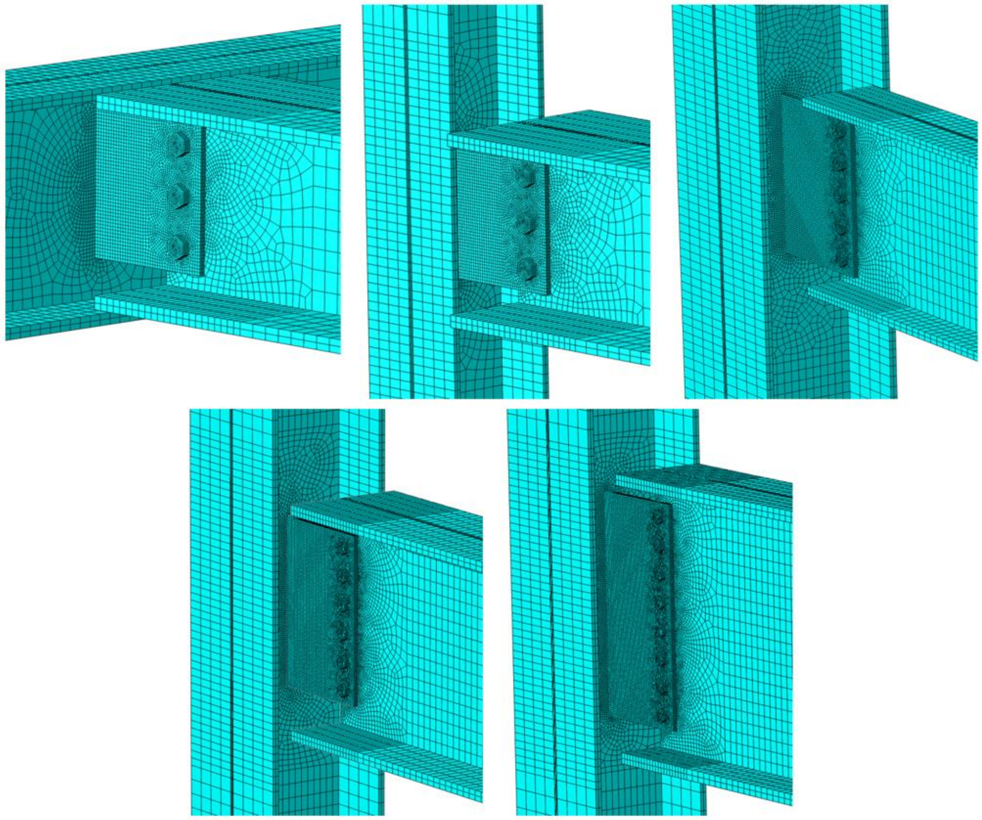


Figure 3-9. Unstiffened configurations models meshing (Sherman-Ghorpanboor)

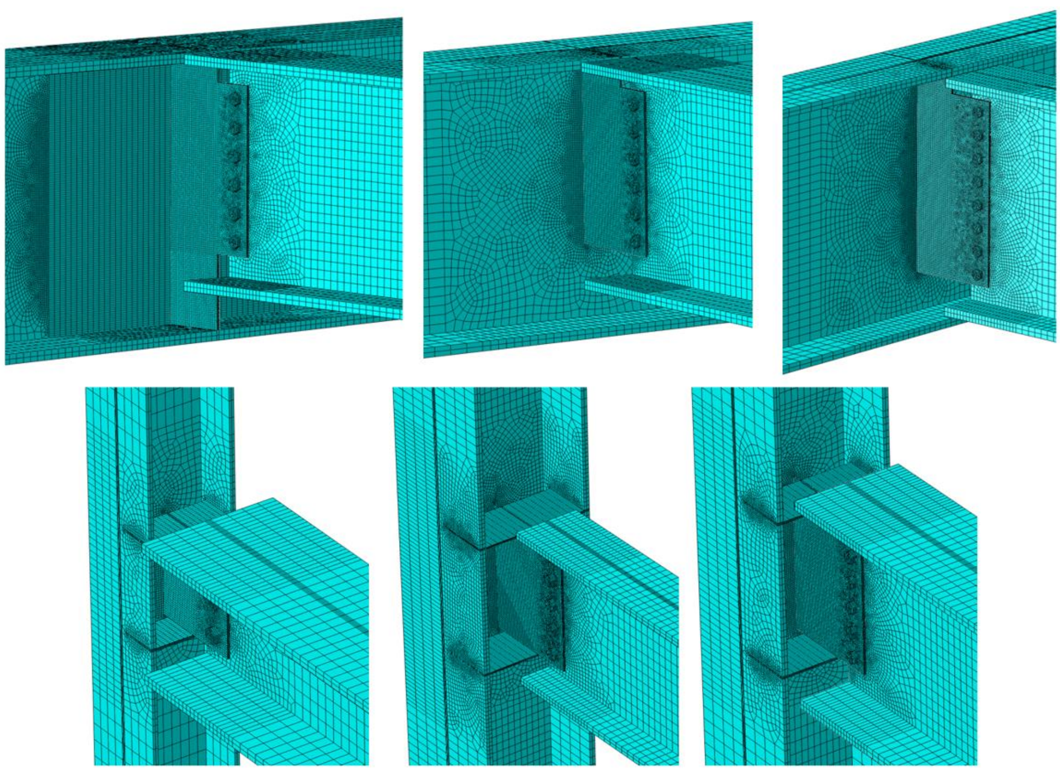


Figure 3-10. Stiffened configurations models meshing (Sherman-Ghorpanboor)

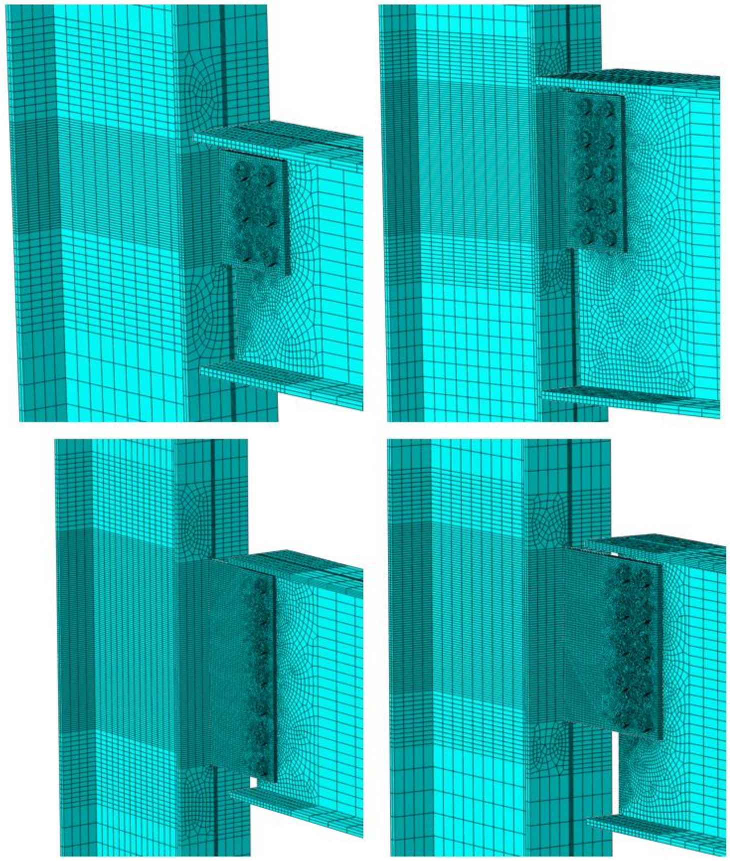


Figure 3-11. Unstiffened configurations models meshing (Metzger)

3.8. Solution

During the processing stage in ABAQUS (2013), the run will be terminated when excessive distortion presented at the elements with the highest strains. At this point, the model will automatically stop. When the ultimate loads is reached, the following is displayed:

1. The strain increment has exceeded fifty times the strain to cause first yield
2. Excessive distortion at some integration points in solid (continuum) elements.

The unloading path can be modelled using ABAQUS (2013). However, investigating the post-failure stage will not serve the objective of this study. Also, modeling the post-failure (unloading) stage will increase significantly the running time of the models. Additionally, the failure load was comparable and close to the failure loads reached during the experiments.

4. CHAPTER FOUR – FINITE ELEMENT VALIDATION

In order to check the validity of the models, the FEA results were compared with the experimental results. The validation of the models using Sherman-Ghorbanpoor experiments was done by comparing connection's shear-vertical displacement curves and by failure modes. Where for Metzger experiments, the models were validated by comparing connection's shear-beam end rotation curves and by failure modes.

4.1. FEA Models Using Sherman-Ghorpanboor Experiments

4.1.1. Unstiffened Connections Supported by Member's Web (Flexible Support)

Table 4-1 shows a comparison between the ultimate shear forces and failure modes obtained from FEA and experiments for unstiffened orthogonal shear tab connections where the shear tab is only welded to the web of the supporting member. This table lists primary failure modes that are followed by secondary failure modes in parentheses.

Table 4-1. Ultimate shear forces and failure modes (unstiffened with flexible supports)

Test	Experimental		FEA	
	V_{exp} (kips)	Failure Modes*	V_{FEA} (kips)	Failure Modes*
1-U	58.7	B (A,C)	56.4	B (A,D,E)
3-U	54.8	D (A)	48.7	D (A,B)
4-U	98.7	E (A,D)	87.0	E (A,D,B)
6-U	138.0	D,F (B,E,G,H,I)	125.4	D,F (B,H,I)

* Failure modes:

A = bolt shear
B = bolt bearing of the plate
C = shear rupture
D = web mechanism
E = twist
F = bolt fracture
G = shear yield
H = web shear
I = weld

4.1.1.1. Connection shear-connection vertical displacement curves

Figures 4-1 to 4-4 show the connections' shear-vertical displacement curves of the FEA and experiments. As shown in the figures below, there is good agreement between the experiment and FEA results. However, some of the experiment curves showed unrealistic and irregular behavior due to several difficulties in controlling the test environment. The FEA showed a more realistic behavior (in the linear and non-linear range) as shown in the figures below. In general, the difference between the two curves are related to several factors: uncertainty in the material properties, difficulties and uncertainty of the effectiveness of the lateral bracing during the experiments, uncertainty of the actual bolt forces, the use of tie constraint instead of modeling the welds in order to have time-efficient models, and excluding the effect of residual stresses due to welding and rolling process. However, the difference between the ultimate shear forces was within an acceptable range (equal to or less than 11%).

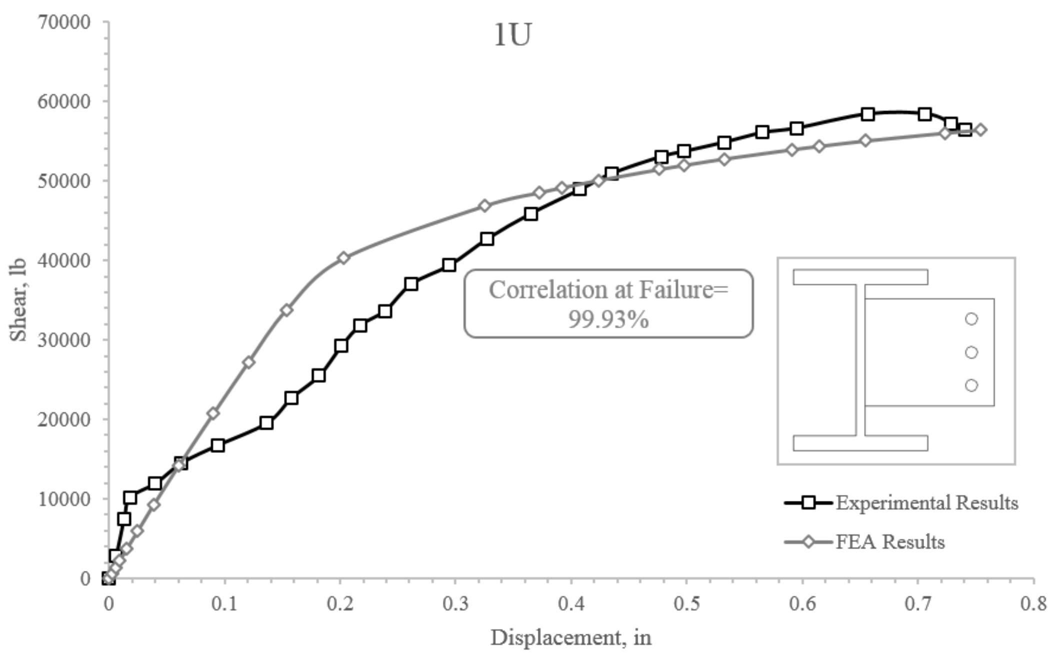


Figure 4-1. Shear-Displacement curves for test 1U

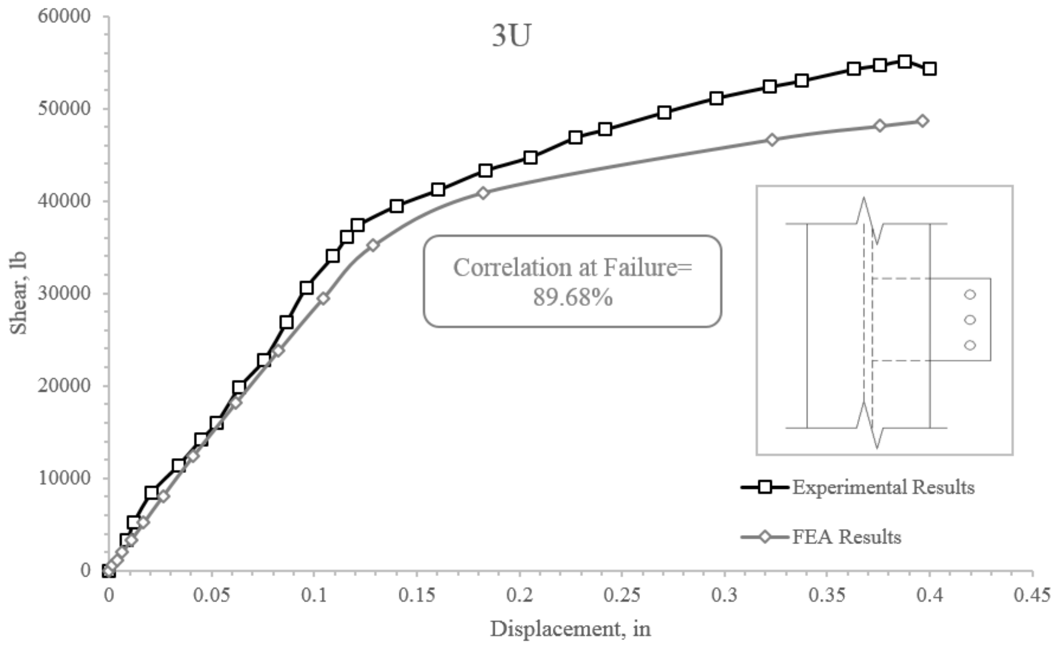


Figure 4-2. Shear-Displacement curves for test 3U

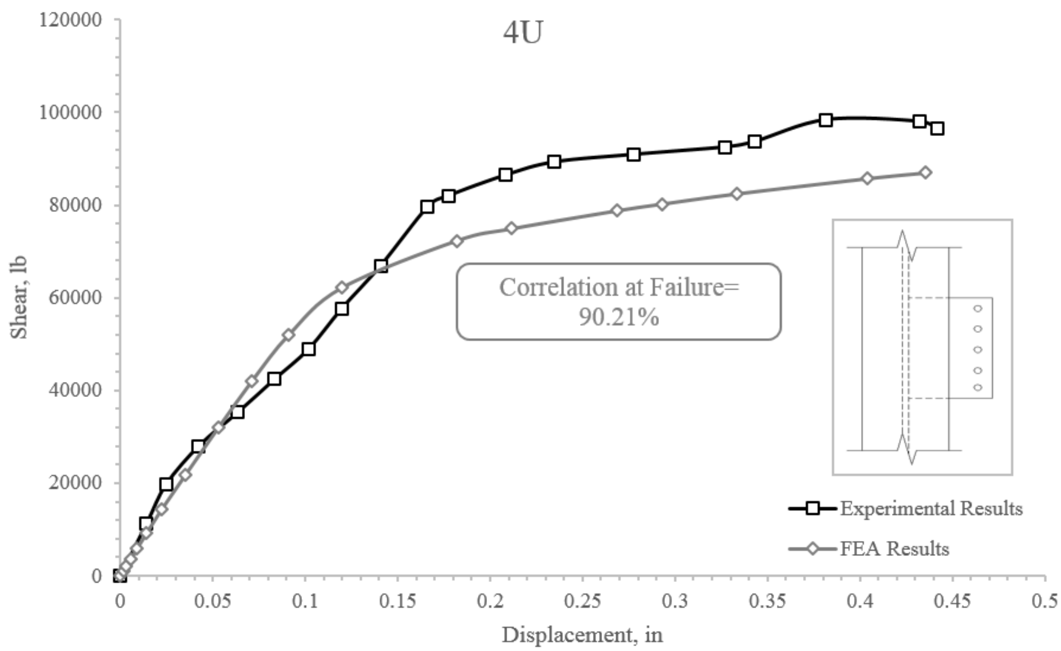


Figure 4-3. Shear-Displacement curves for test 4U

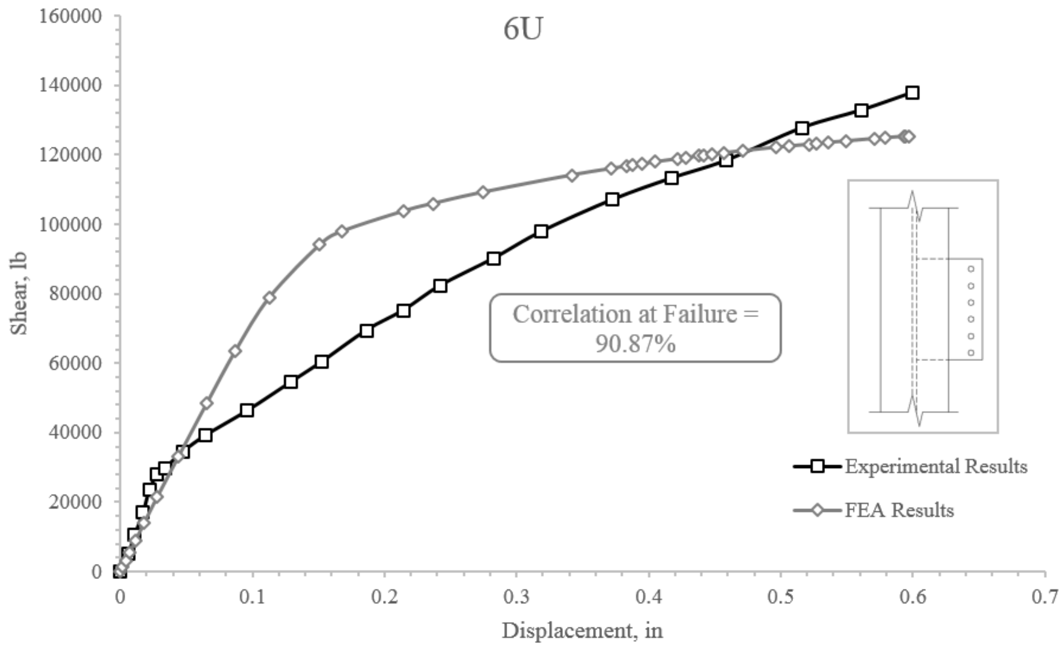


Figure 4-4. Shear-Displacement curves for test 6U

4.1.1.2. Failure modes

The behavior of the FEA was investigated along the loading steps, all failure modes were investigated and compared with the failure modes observed during the experiments. In general, it was observed that the FEA and experiments have the same primary failure modes. However, additional secondary failure modes (failure modes in parentheses in Table 4-1) were captured in the FEA only (plate twist for test 1-U and bolt bearing for tests 3-U and 4-U). These failure modes were not observed in the experiments and should be considered in the design of the extended shear tab connections to provide sufficient strength and ductility. The failure modes were addressed in the experimental investigation using visual inspection for the tested connections. The failure modes were observed in the finite element models through the presence of the equivalent plastic strain. It was observed that the equivalent plastic strain for the additional secondary failure modes are small and hard to be observed using visual inspection. Moreover, these additional secondary failure modes might become a primary failure modes if different plate thicknesses were used. This finding will be investigated later in the parametric study results section. Figures 4-5 – 4-8 show some of the failure modes for the connections mentioned in this section. Note that "PEEQ" is the equivalent plastic strain, and "U3" is the lateral displacement which is the displacement about the z-axis.

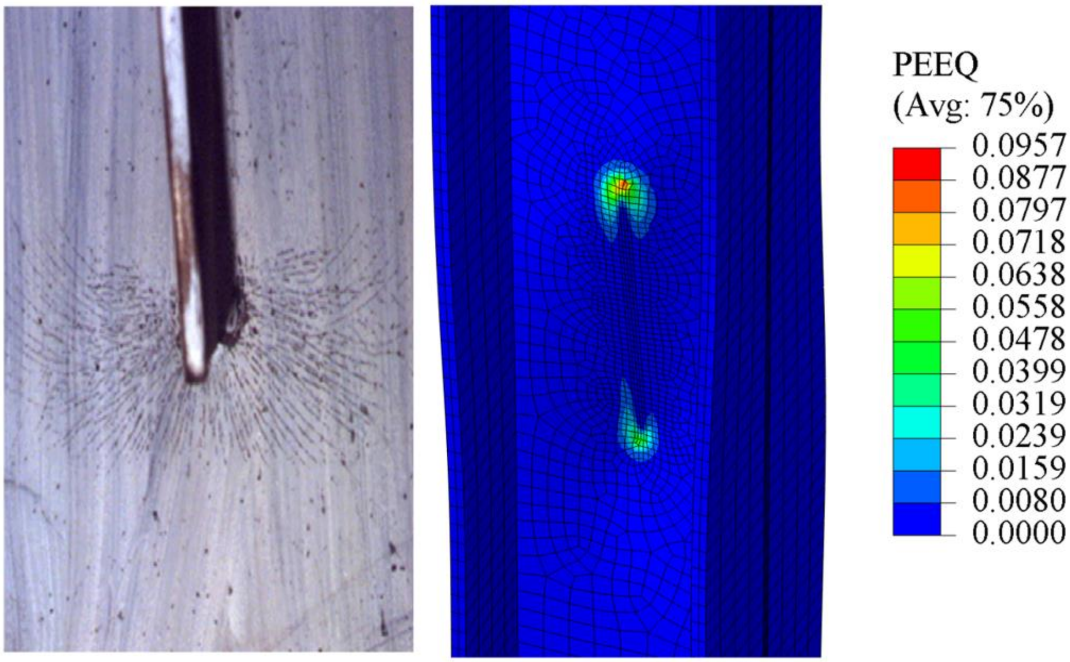


Figure 4-5. Web mechanism

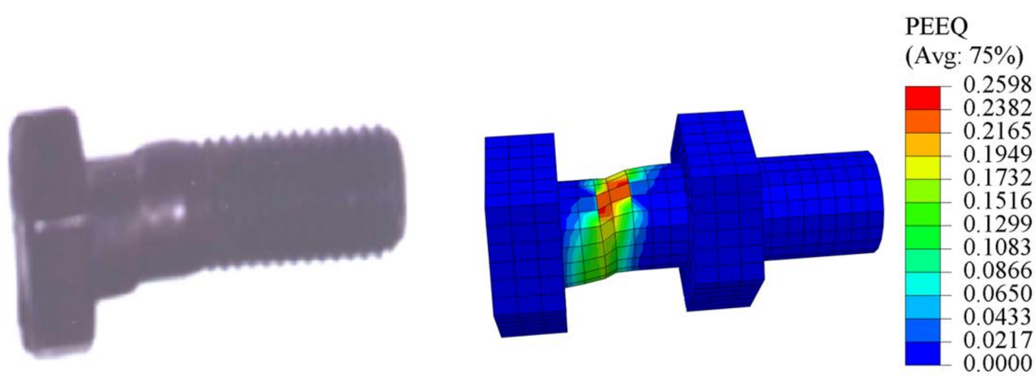


Figure 4-6. Bolt shear

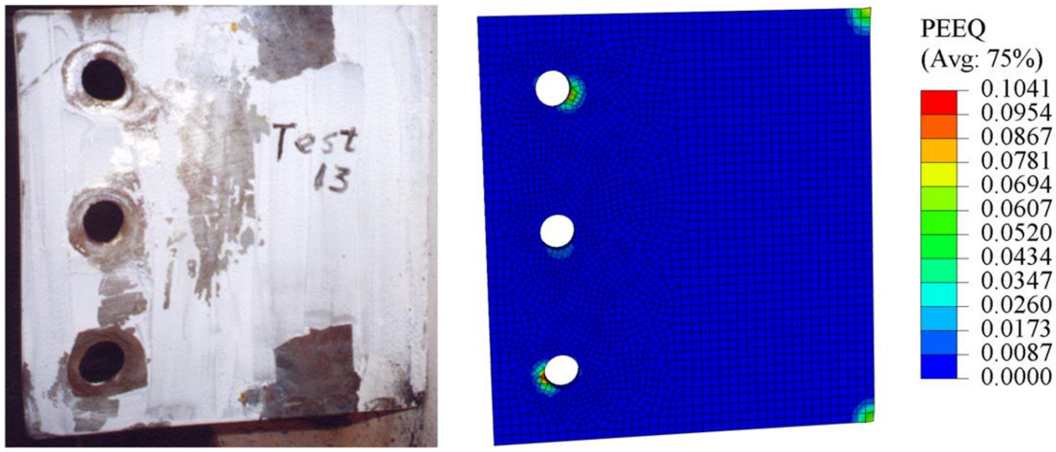


Figure 4-7. Bolt bearing

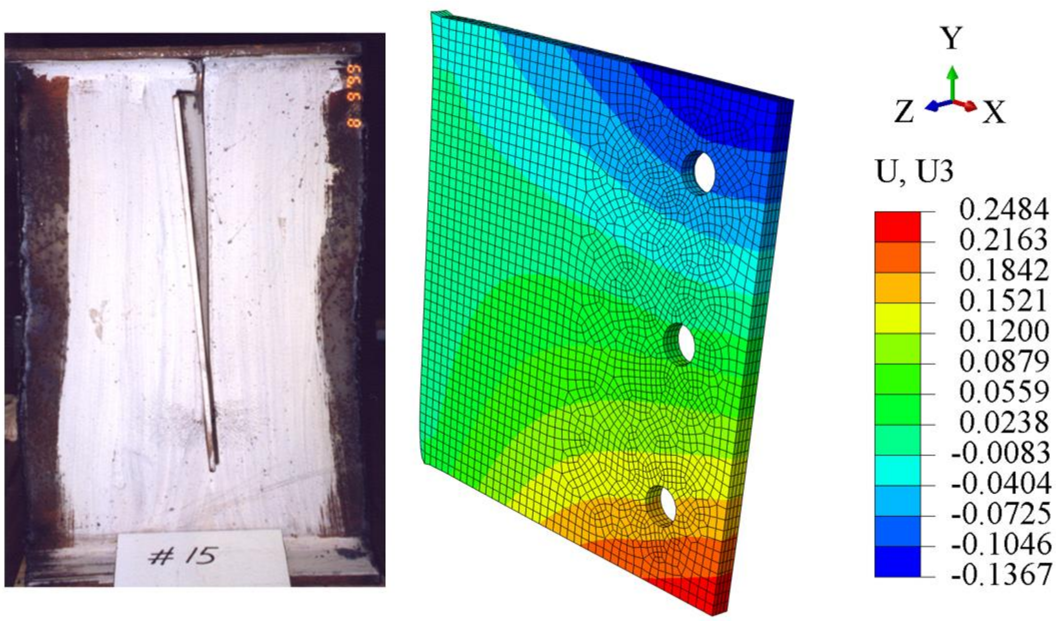


Figure 4-8. Shear tab twist

4.1.2. Stiffened Connections with Flexible Supports

Table 4-2 shows a comparison between the ultimate shear forces and failure modes obtained from the FEA and the experiments for stiffened orthogonal shear tab connections where the shear tab is welded to the stiffener plates and to the web of the supporting column in the case of a column support; and welded to the web, top flange of the supporting girder in the case of girder support. Some of the connections the shear tab was also extended and welded to the girder bottom flange.

As shown in Table 4-2, for most of the cases, the primary failure modes are the same in the experiments and FEA. However, additional secondary failure modes were obtained from the FEA. As was mentioned in the previous section, the effect of these failure modes will be investigated more in the parametric study section. This table lists primary failure modes that are followed by secondary failure modes in parentheses.

Table 4-2. Ultimate shear forces and failure modes (Stiffened with flexible supports)

Test	Experimental		FEA	
	V_{exp} (kips)	Failure Modes*	V_{FEA} (kips)	Failure Modes*
3A	53.2	B, D	53.3	B, D (E)
3B	53.1	B, D	49.6	B, D (E, A, D)
3D	51.1	B, D	50.1	B, D (E)
4A	103	B, D	98.9	B, D, E
4B	107	B, D	99.2	B, D (A, E)
5A	122.9	E (A,B)	140.2	E (A, B)
6B	124.5	D, A	139.1	B, A (F)
7B	224.2	D, B (A, C, G)	225.7	B (A, C, F)
7C	204.2	E, D (A, B, F, G)	181.2	E, B (A, F)
8B	227.4	D (B, A, G)	244.9	B (A)

* Failure Modes:

A = bolt bearing B = shear yield C = web mechanism
D = twist E = plate buckling F = tearing
G = web shear

4.1.2.1. Connection shear-connection vertical displacement curves

Figures 4-9 – 4-18 show the connections' shear-vertical displacement curves obtained from the FEA and experiments. As shown in the figures, there is good agreement between the results and the difference did not exceed 15%.

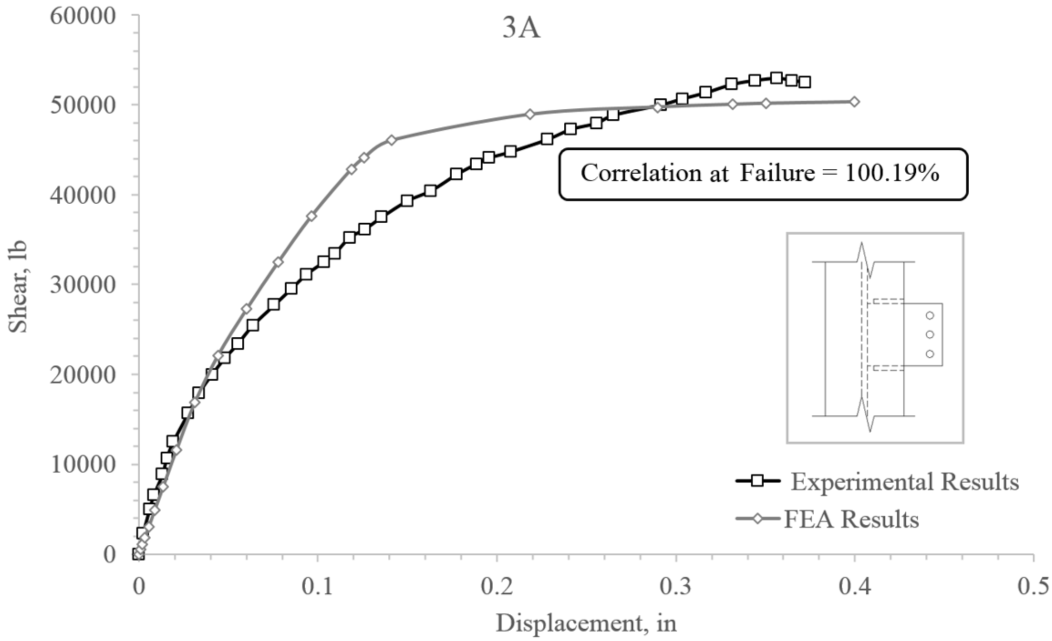


Figure 4-9. Shear-Displacement curves for test 3A

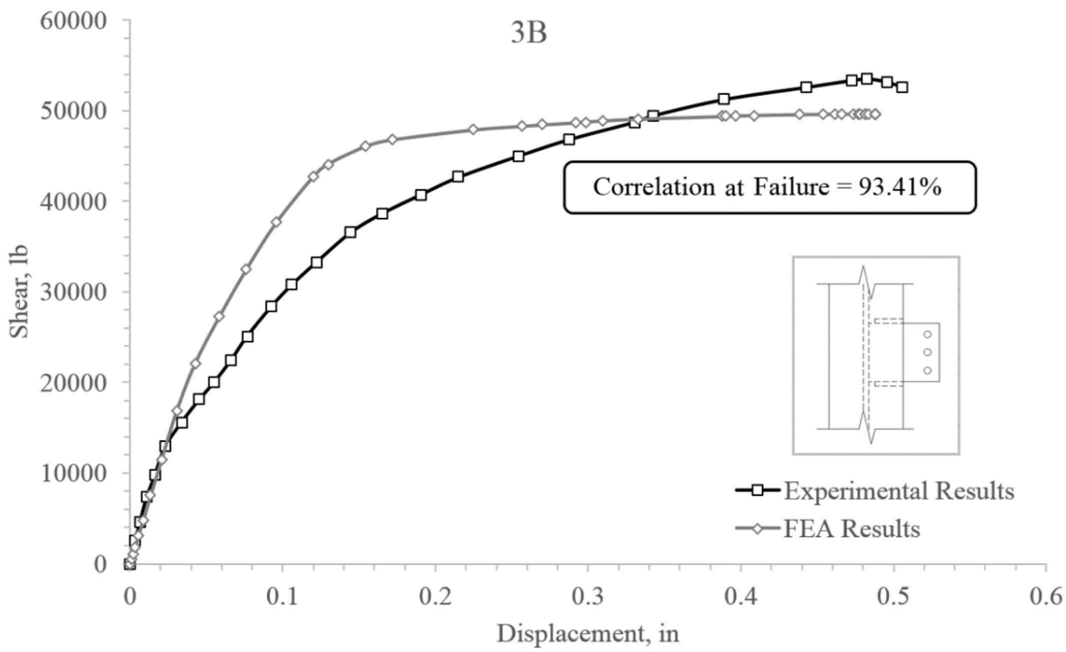


Figure 4-10. Shear-Displacement curves for test 3B

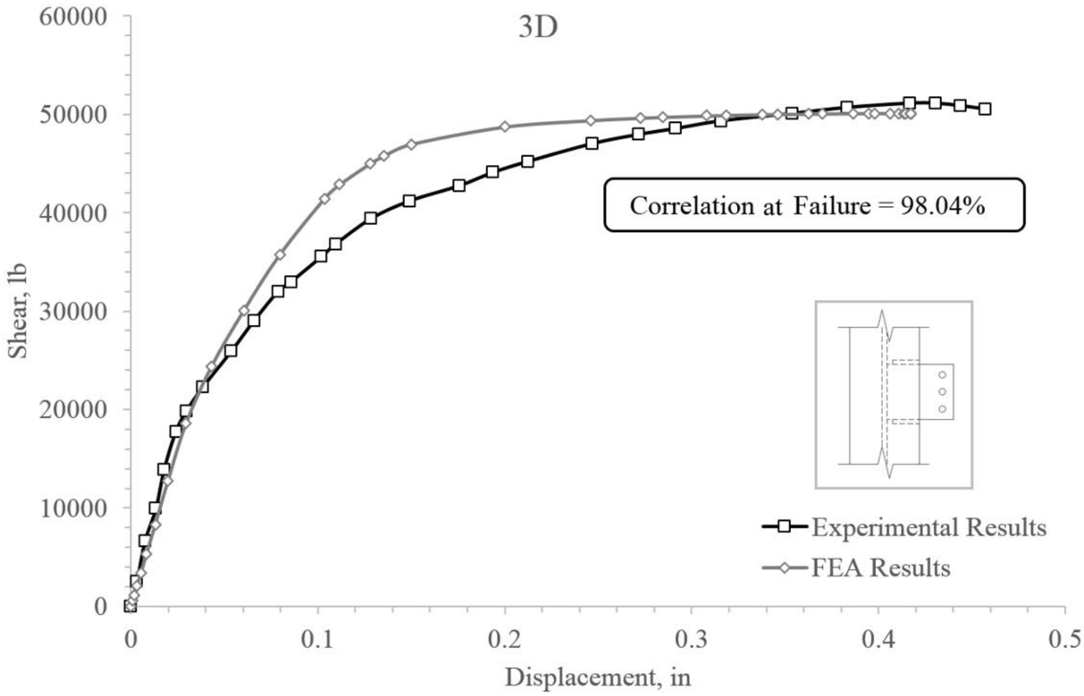


Figure 4-11. Shear-Displacement curves for test 3D

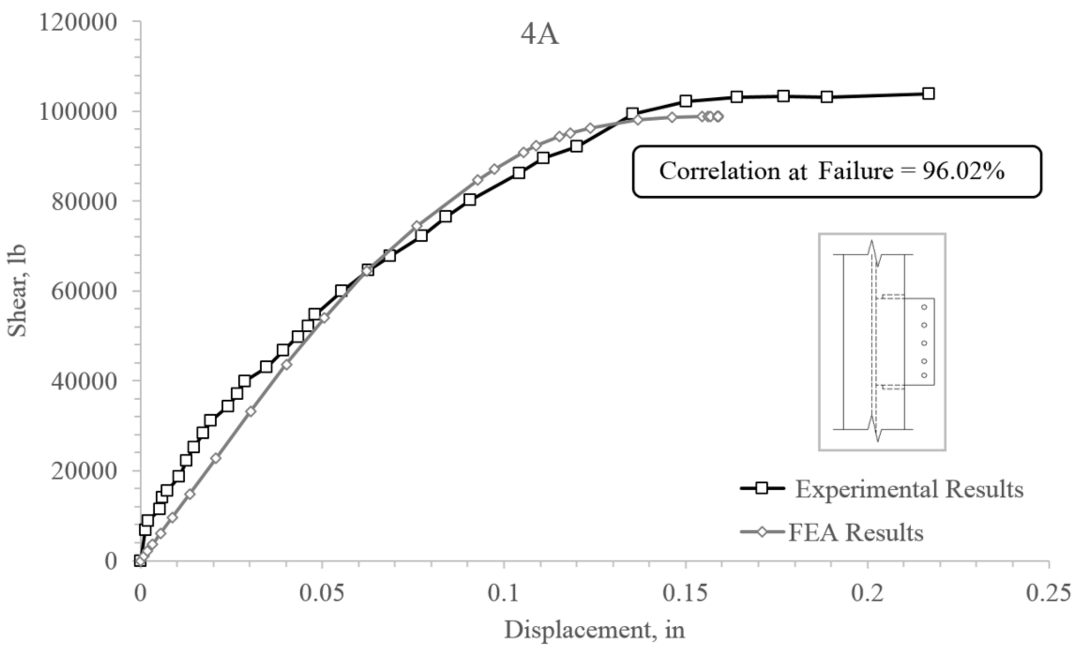


Figure 4-12. Shear-Displacement curves for test 4A

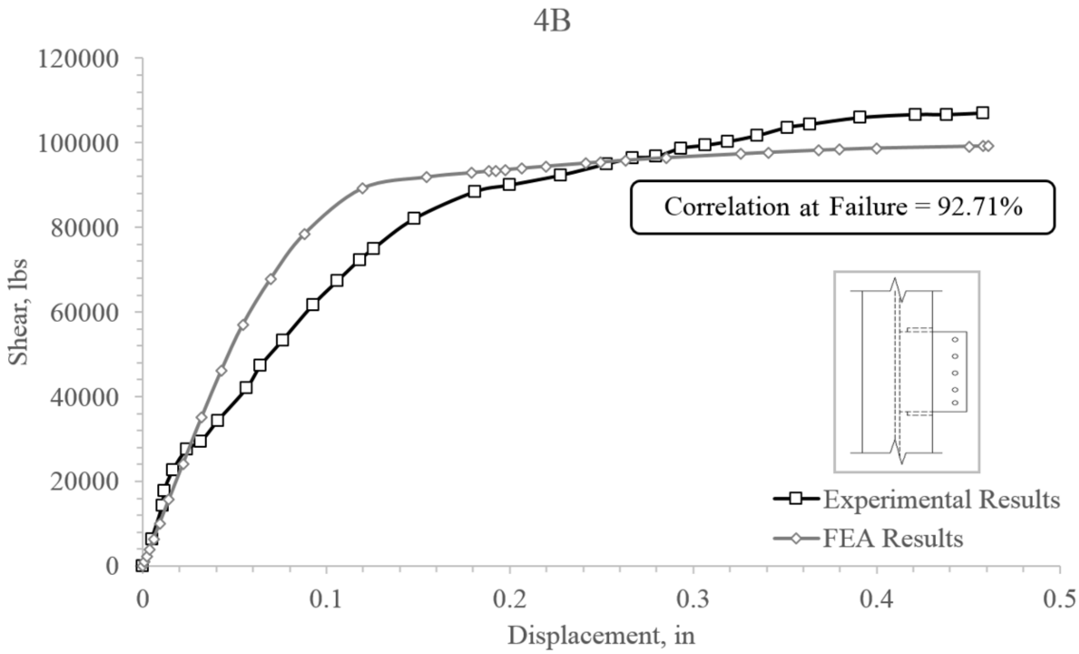


Figure 4-13. Shear-Displacement curves for test 4B

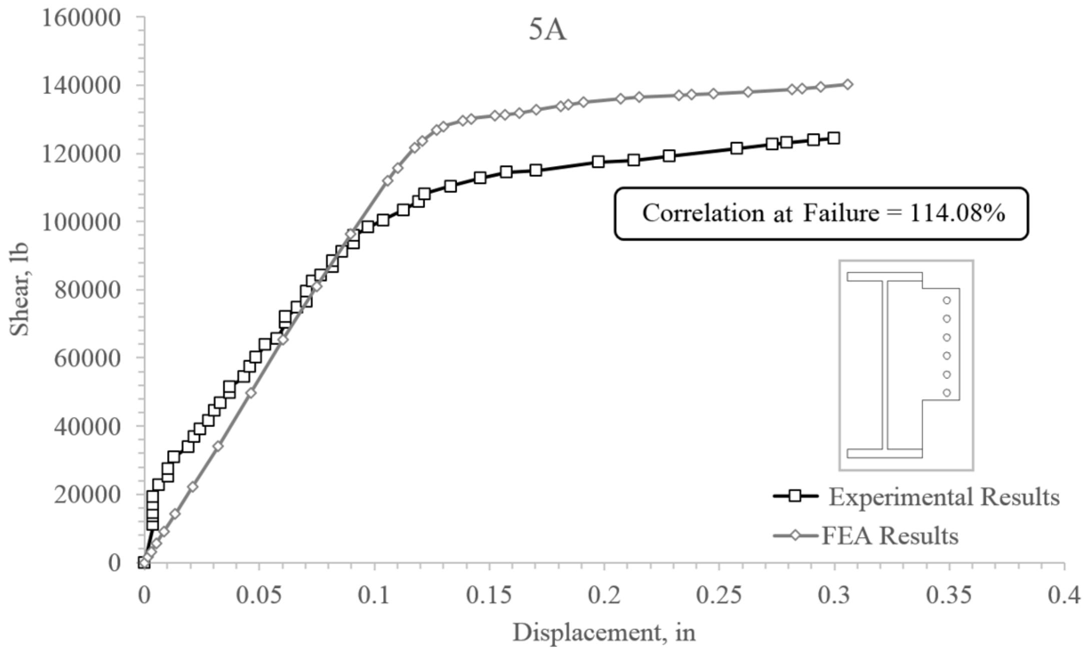


Figure 4-14. Shear-Displacement curves for test 5A

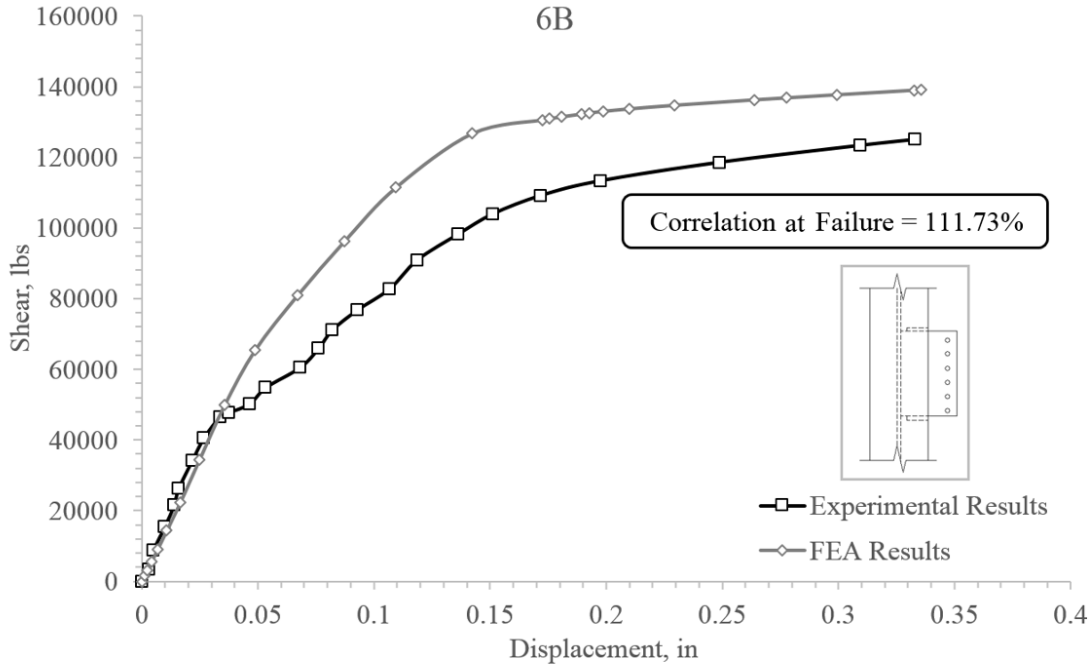


Figure 4-15. Shear-Displacement curves for test 6B

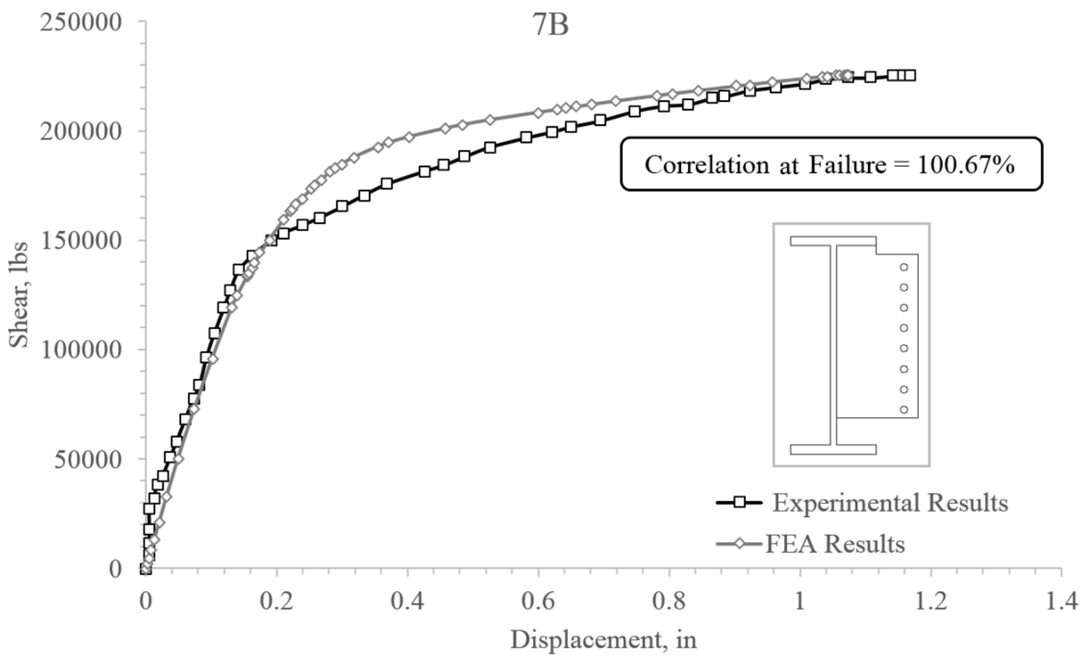


Figure 4-16. Shear-Displacement curves for test 7B

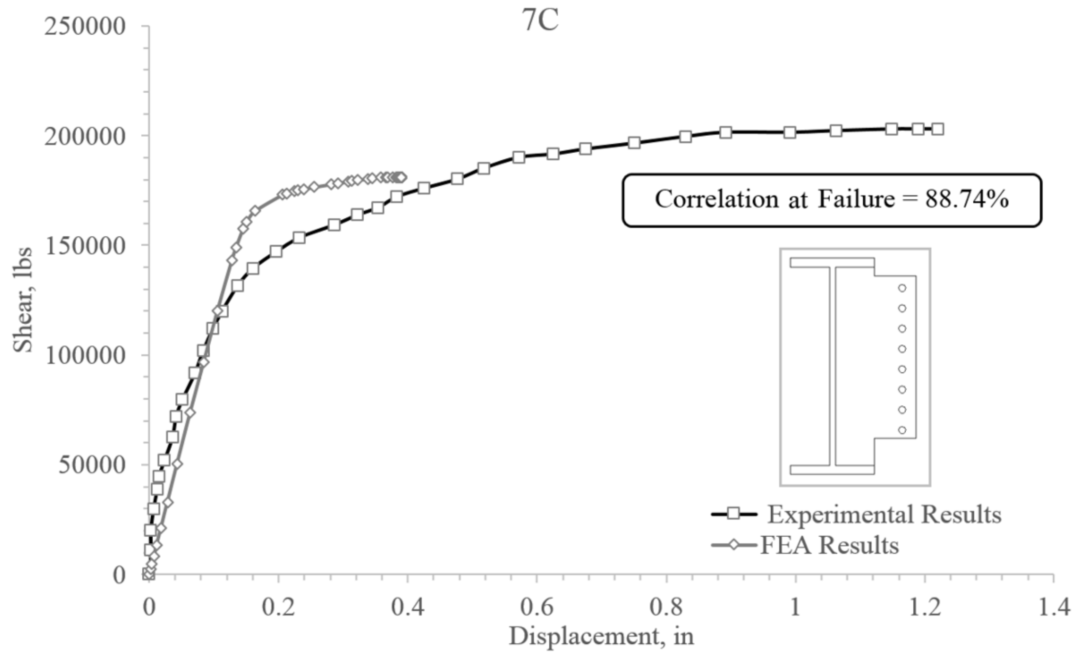


Figure 4-17. Shear-Displacement curves for test 7C

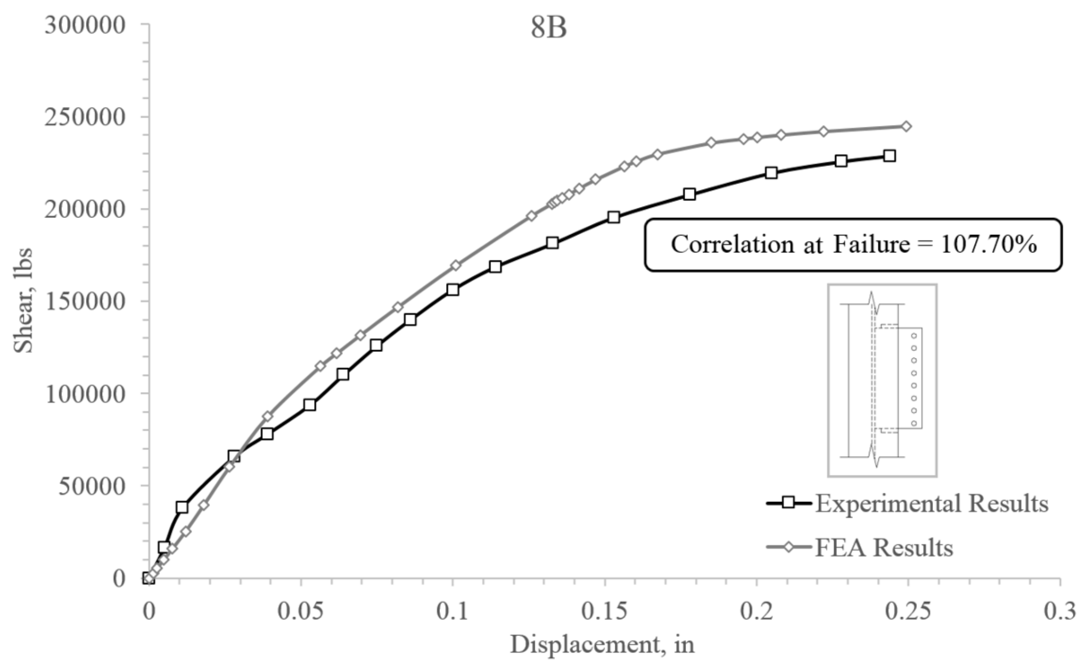


Figure 4-18. Shear-Displacement curves for test 8B

4.1.2.2. Failure modes

In general, same primary failure modes were obtained from the FEA and experiments. Figures 4-19 – 4-21 show some of the failure modes obtained from the FEA and experiments for the stiffened shear tab connections. However, additional secondary failure modes (failure modes in parentheses in Table 4-2) were observed in the FEA only. These failure modes should be considered in the design of the stiffened extended shear tab connections to assure sufficient strength and ductility for these connections. Figure 4-22 shows failure modes obtained by the FEA for orthogonal connections with different configurations. Note that “PEEQ” is the equivalent plastic strain, and “U1” is the lateral displacement on the plate local coordinates.

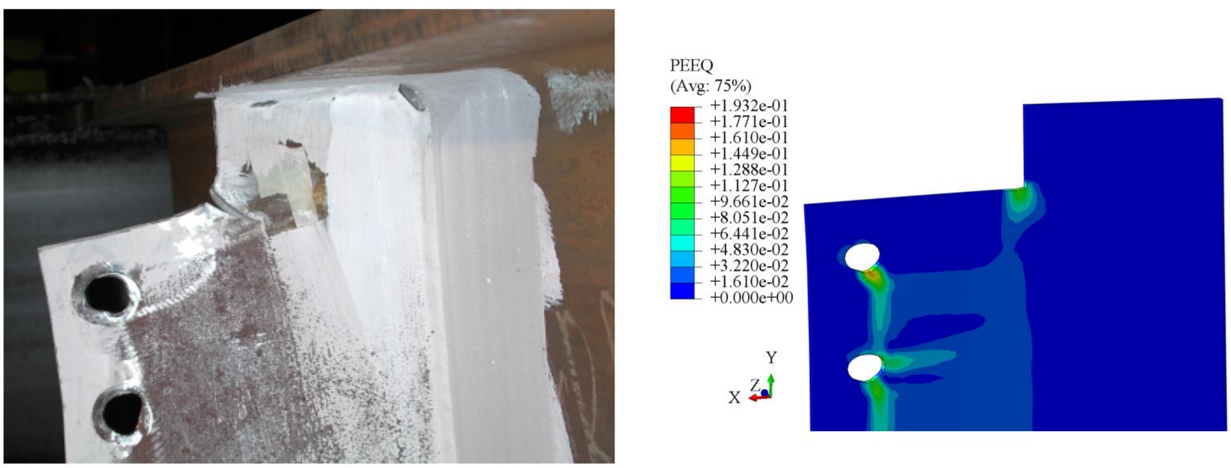


Figure 4-19. Shear tab tearing and bolt bearing

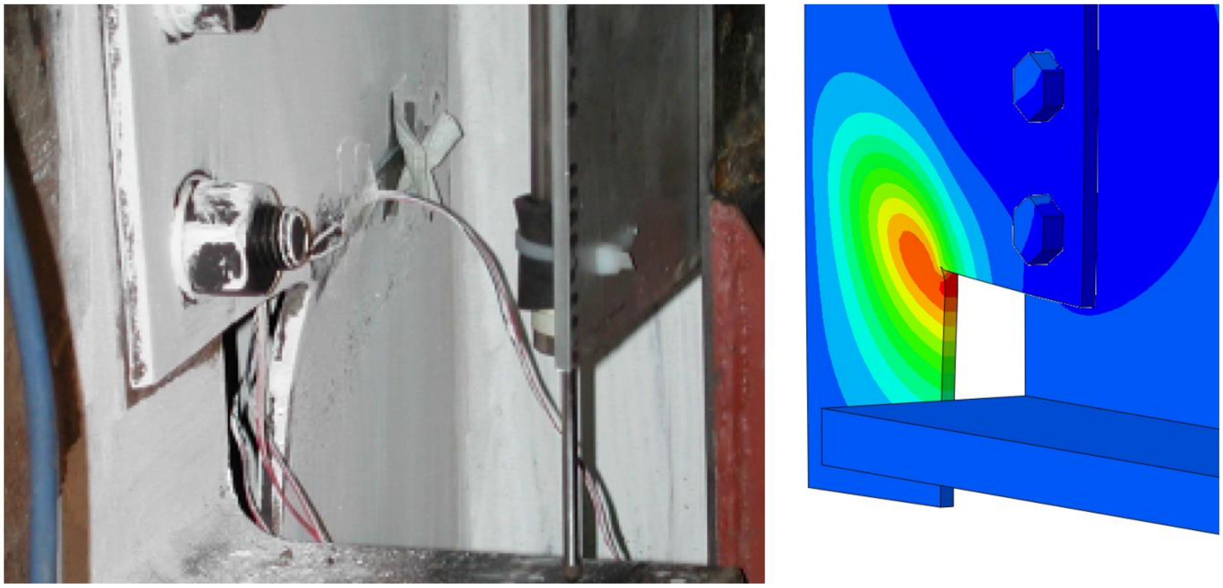


Figure 4-20. Shear tab buckling

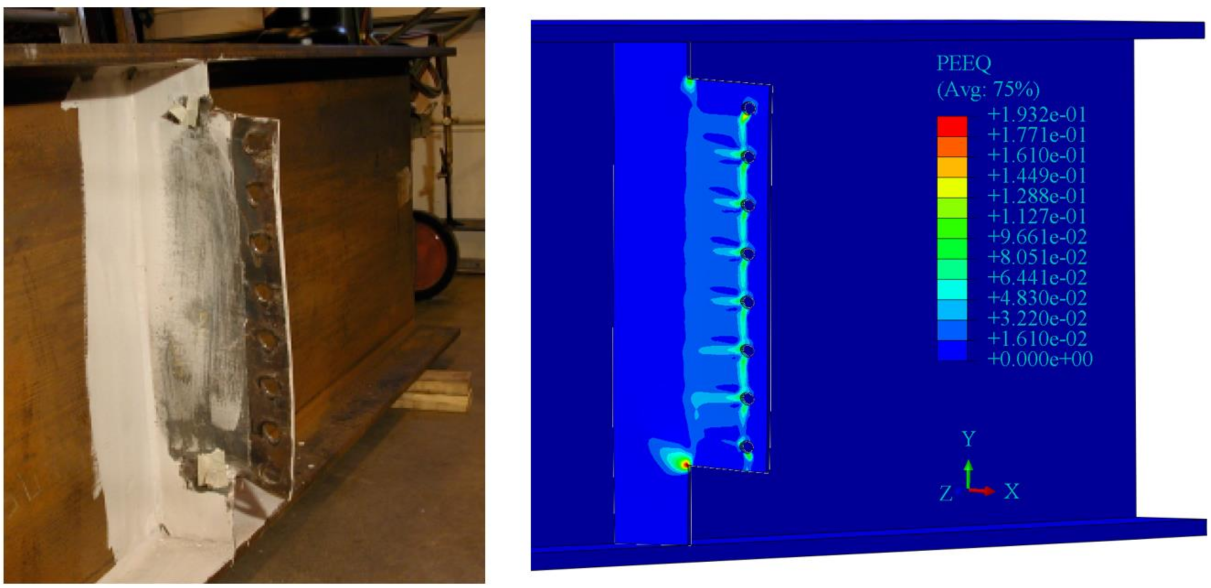


Figure 4-21. Failure modes of Test 7C

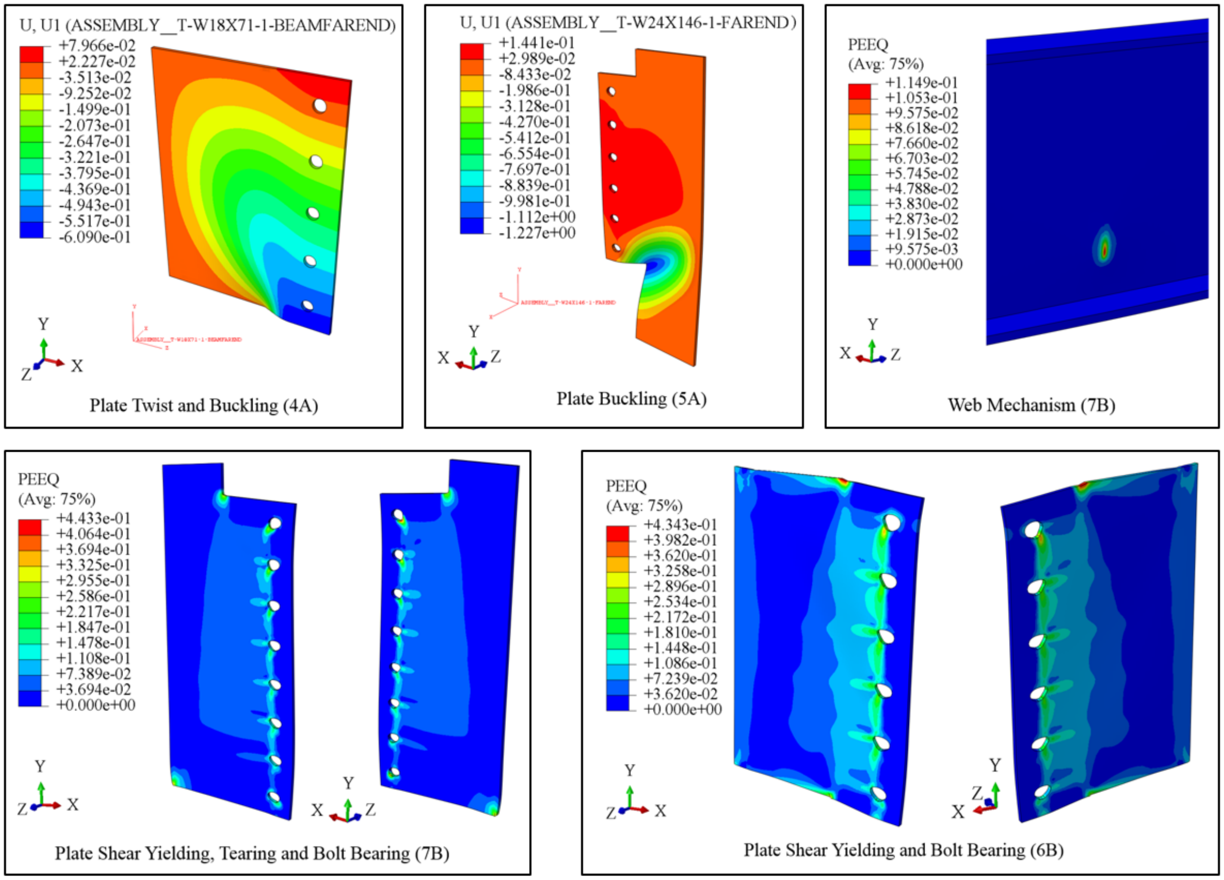


Figure 4-22. FEA failure modes for different configurations

4.2. FEA Models Using Metzger Experiments

4.2.1. Unstiffened Connections Supported by Column Flange (Rigid Support)

For connections tested by Metzger (2006), the shear tab was welded to the flange of the supporting column. The FEA results were validated in two ways: Comparing the connection's shear-beam end rotation curves and by comparing failure modes. Table 4-3 shows a comparison between the ultimate shear forces and failure modes obtained from the FEA and experiments. This table lists primary failure modes that are followed by secondary failure modes in parentheses.

Table 4-3. Ultimate shear forces and failure modes (Unstiffened with rigid supports)

Test	Experimental		FEA	
	V_{exp} (kips)	Failure Modes*	V_{FEA} (kips)	Failure Modes*
6B2C - 4.5 - 1/2	89.7	G	91.9	G, (J)
7B1C - 9 - 3/8	98.0	I, B, H	105.0	I, B, H, (C, A, F)
10B2C - 10.5 - 1/2	94.6	L,C	103.3	L, (I, F, J, G)

* Failure modes:

A = bolt shear
B = bolt bearing of the beam web
C = shear yield
F = twist
G = weld
H = plate buckling
I = LTB of the beam flanges at midspan
J = yielding of plate corners
L = local buckling of the beam web at midspan

4.2.1.1. Shear-beam end rotation curves

In the experiments, the beam end rotation was measured using two linear potentiometers, the first one was placed over the center of gravity of the bolt groups, and the second potentiometer was placed 6 in. away from the first one. In the FEA, the vertical displacement was obtained at the same locations as measured and recorded in the experiments to measure the beam end rotation.

Figures 4-23 – 4-25 demonstrate the shear-beam end rotation curves of the FEA and experiments. As shown in the figures, there is good agreement between the experimental results and FEA results. In general, the results show that the FEA curves are stiffer than the experimental curves. This was expected due to the difficulties in applying the lateral bracings along the beam and in controlling the lateral torsional buckling of the beam's top flange during the experiments. Additionally, based on the column deflected shape, the FEA shows that there is an expected local buckling in the column's web and flange; note that in the experimental work, all the connections were welded to the same column which would affect the column capacity. The local buckling in the column web and flange affect the connections' capacity.

However, the lateral bracing can be more controlled in the FEA and unlike the experimental study, no initial stress and possible plastic deformations existed in the column. Nevertheless, the error did not exceeded 10% in all the models.

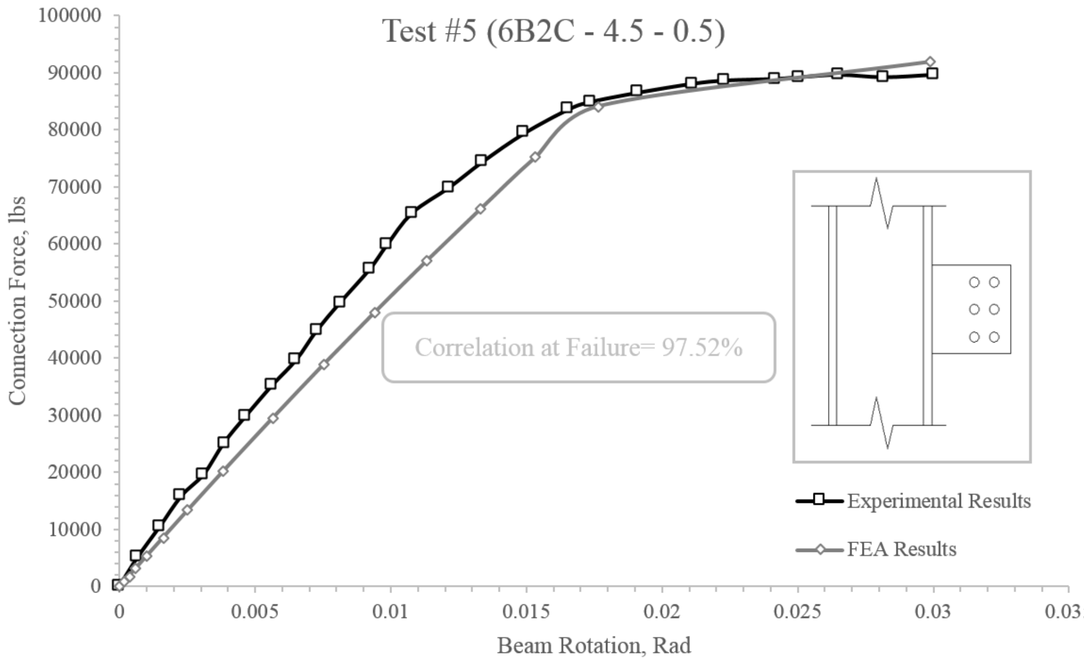


Figure 4-23. Shear-beam end rotation curves for test #5

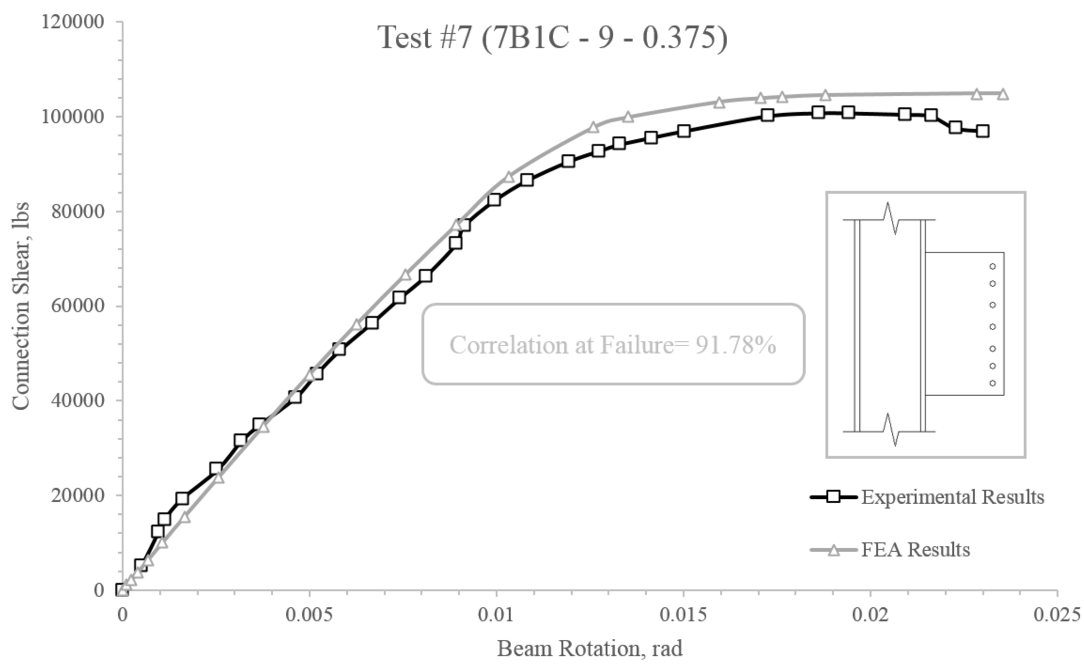


Figure 4-24. Shear-beam end rotation curves for test #7

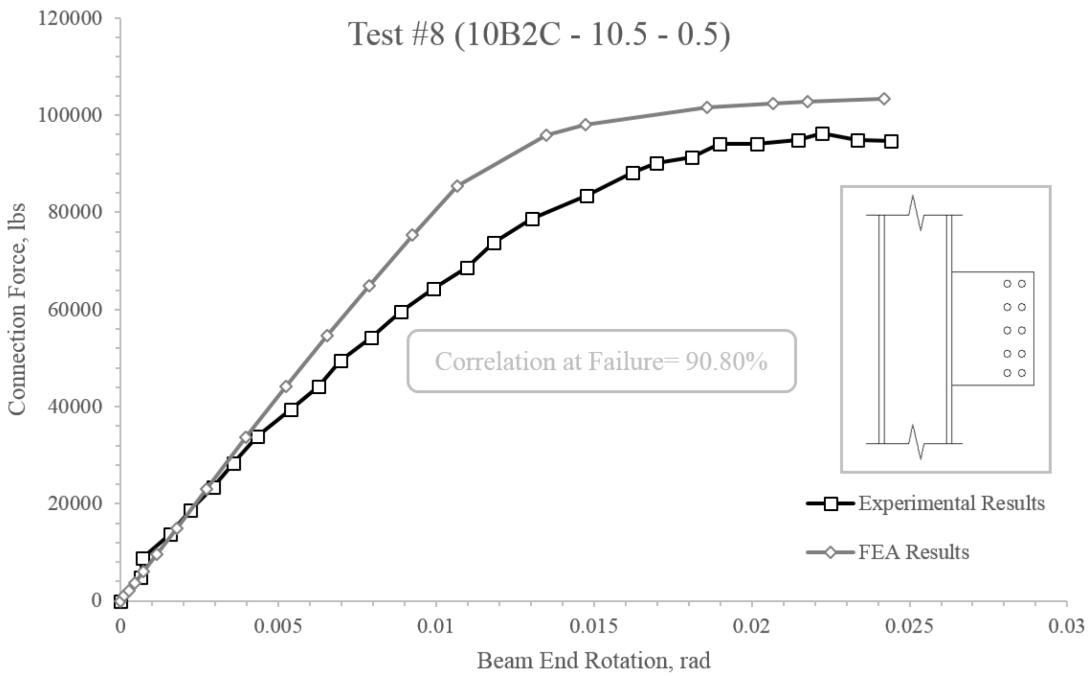


Figure 4-25. Shear-beam end rotation curves for test #8

4.2.1.2. Failure modes

Figures 4-26 – 4-29 show the failure modes obtained from the FEA and experiments for the connections in this section. As shown in the figures below and Table 4-3, same primary failure modes were obtained from the FEA and the experiments. However, the FEA showed additional secondary failure modes (failure modes in parentheses in Table 4-3). For example, bolt shear failure mode was detected in test #7, this failure mode was not observed in the experiments. Also, shear tab twisting in tests with an a-distance (the distance between the weld line and the bolt line) more than 4.5 in. was significant. Figure 4-30 shows the expected buckling of the column web and flange for test #7 based on the column deformed shape. Note that "PEEQ" is the equivalent plastic strain, "U3" is the lateral displacement for the column web which is the displacement about the z-axis, and "U1" is the lateral displacement for the column flange which is the displacement about the x-axis.

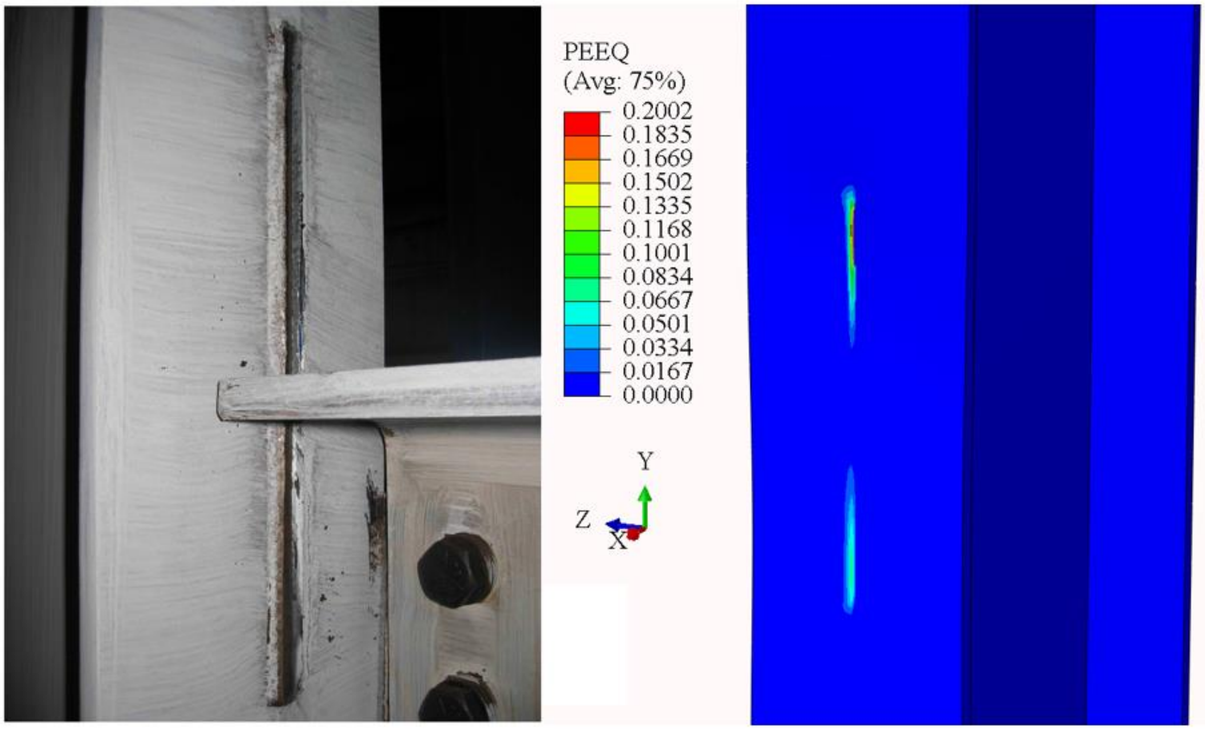


Figure 4-26. Weld rupture

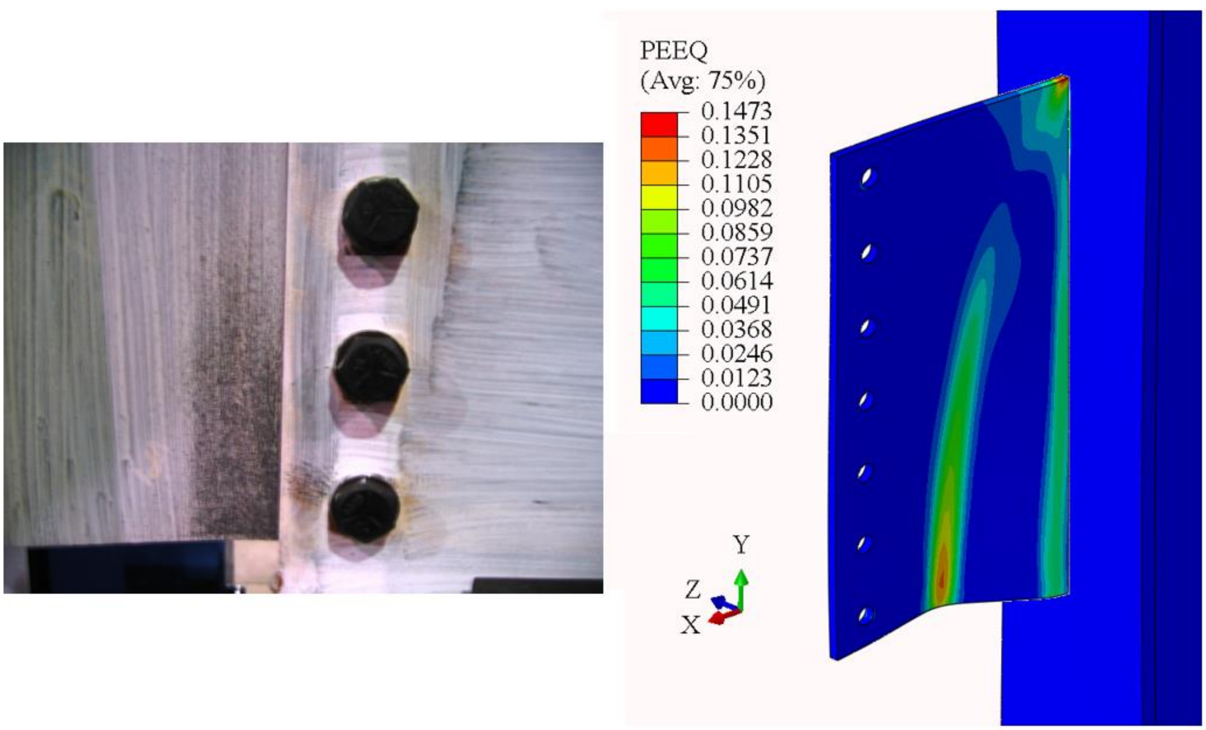


Figure 4-27. Shear tab buckling

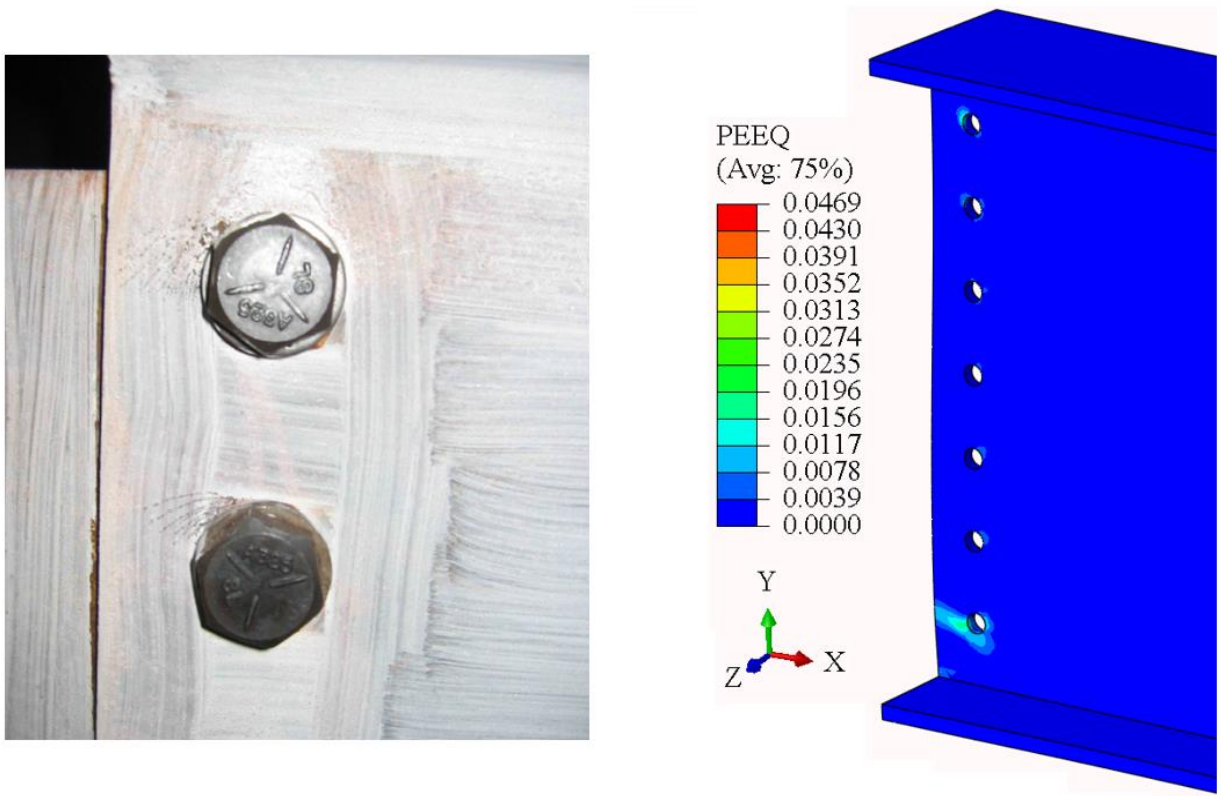


Figure 4-28. Bolt bearing of the beam web

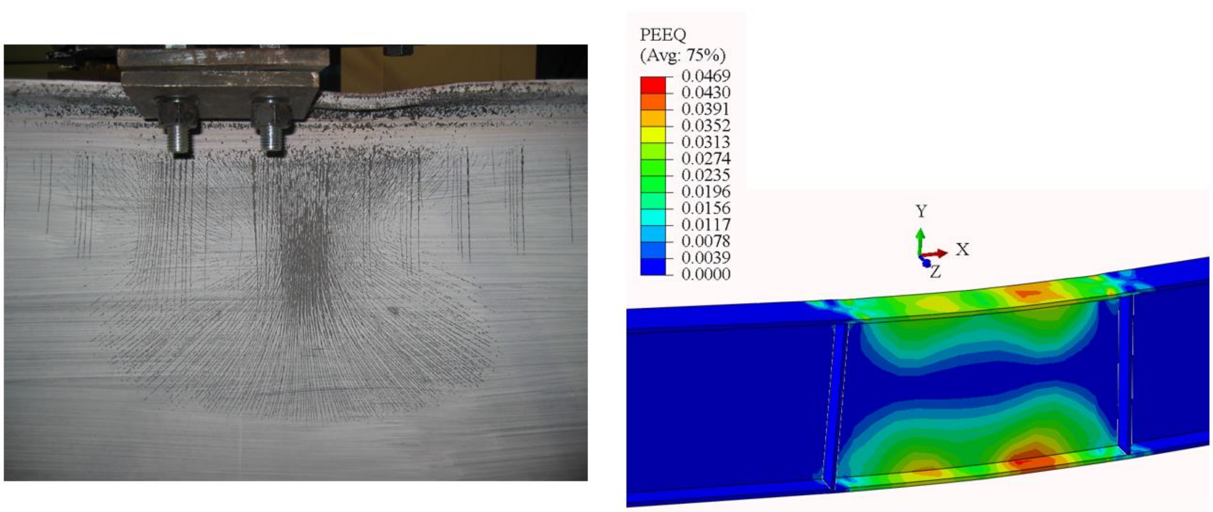


Figure 4-29. Plastic hinge formation at the beam midspan

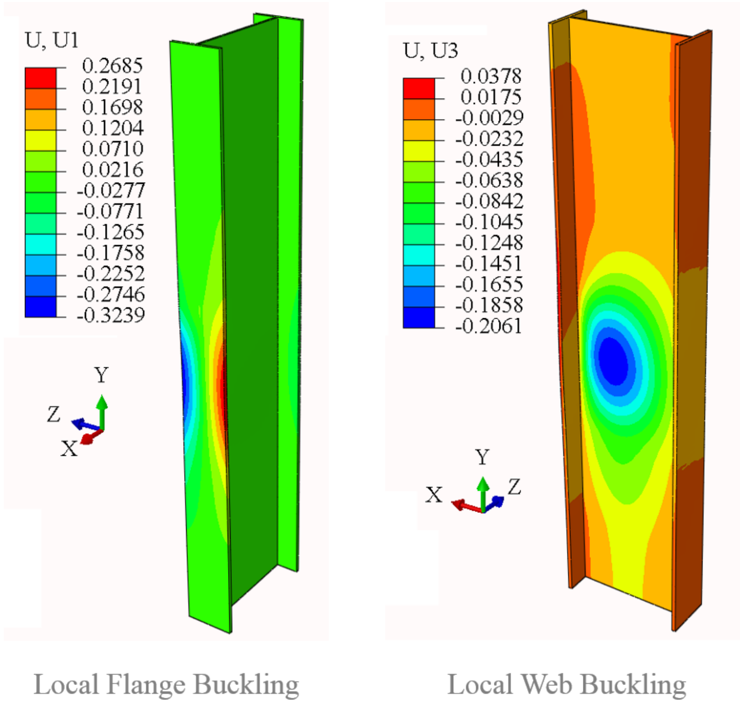


Figure 4-30. Local buckling of test #7 column

4.3. Validation Conclusion

As a conclusion of this chapter, the proposed FEA procedure is a powerful tool to simulate the behavior of the orthogonal extended shear tab connections with different configurations and support types. The FEA allowed the prediction of the shear capacity of the orthogonal extended shear tab connections with an error of less than 15%. The FEA allowed to capture the primary failure modes as the experiments. Moreover, it allowed the capture of additional secondary failure modes that were not observed during experiments. These failure modes might become critical and affect the behavior of the orthogonal extended shear tab connections when changing any of the connection parameters. Thus, the proposed FEA is used to model skewed extended shear tab connections to investigate their behavior, and to perform a parametric study on these connections to understand the relationship between the connection orientation and other parameters.

5. CHAPTER FIVE – SKEWED CONFIGURATION

5.1. Unstiffened Connections Supported by Supporting Member's Web (Flexible Supports)

In the orthogonal configuration, webs of the supported beam and the supporting member are framed at a right angle. In the skewed configurations, each extended connection was modified by rotating the supported beam and shear tab in such a way that the beam longitudinal axis has an angle α with the axis perpendicular to the supporting member's web as shown in Figure 5-1. Four angles were used 5, 10, 15 and 20 degrees. When using angles larger than 20 degrees, the flanges of beam intersect with the flanges of the supporting member, in this case, the flanges of the beam can be coped. However, this will affect the connection behavior and make the process of deriving relationships between the skewed angle and different parameters harder since the flange coping will be different as the skewed angle changes. Local coordinate systems were assigned to the shear tab, supported beam and each bolt in order to adjust the plate orientation, bolts pre-tensioning forces and far end reactions with the beam orientation. Furthermore, the plate geometry was modified to ensure full contact between the plate and supporting member. Figure 5-1 shows the extended shear tab connection with the shear tab welded to the supporting member web for the orthogonal and skewed configurations. Note that the skewed connections geometries, dimensions, material properties, boundary conditions, and the location of the applied loads used in this research are the same as for the orthogonal connections discussed earlier, the only difference is the connection orientation.

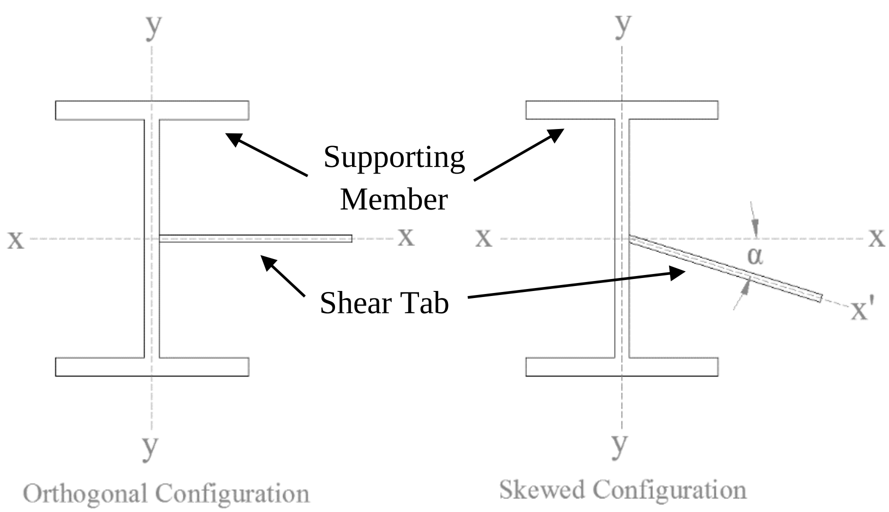


Figure 5-1. Extended shear tab connections with shear tab welded to the supporting member web

Shear-displacement curves, shear-twist curves and failure modes for each connection at different skewed angles (α) were obtained and investigated. Figures 5-2 and 5-3 show the shear-displacement and shear-twist curves for three-bolted beam-to-column connection, respectively. These figures for the other connections can be found in Appendix A. It was observed that the vertical displacement of the connection is not affected by the connection orientation in the elastic range. However, the vertical displacement starts to decrease slightly in the inelastic range as the connection orientation increases. On the other hand, it was observed that the plate twist is increased significantly as the connection orientation increases in the elastic and inelastic ranges.

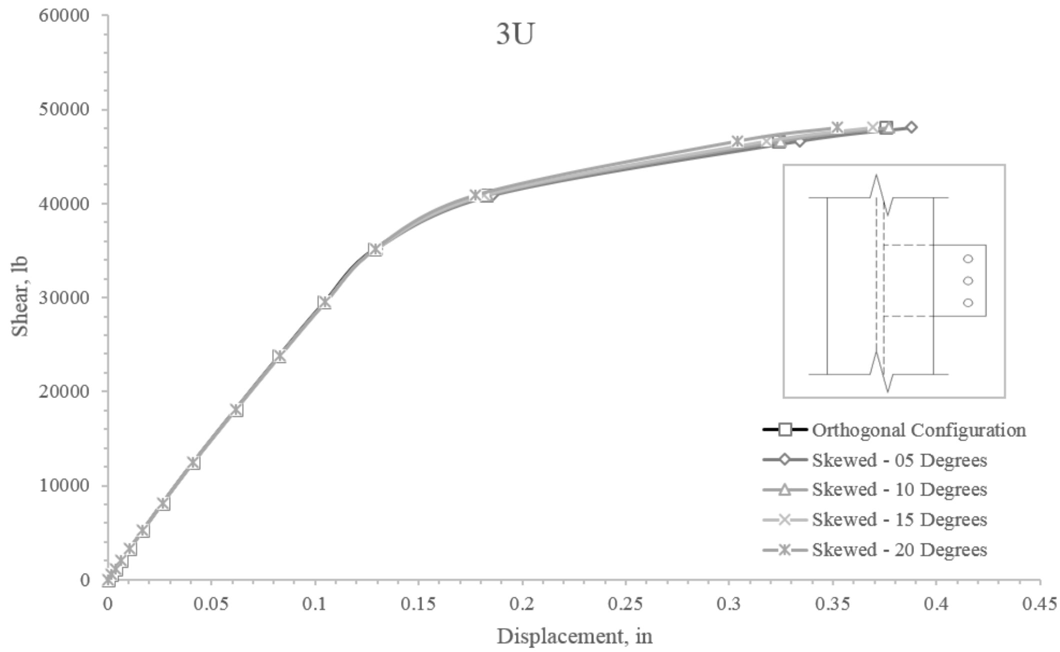


Figure 5-2. Connection shear-displacement curve (3U)

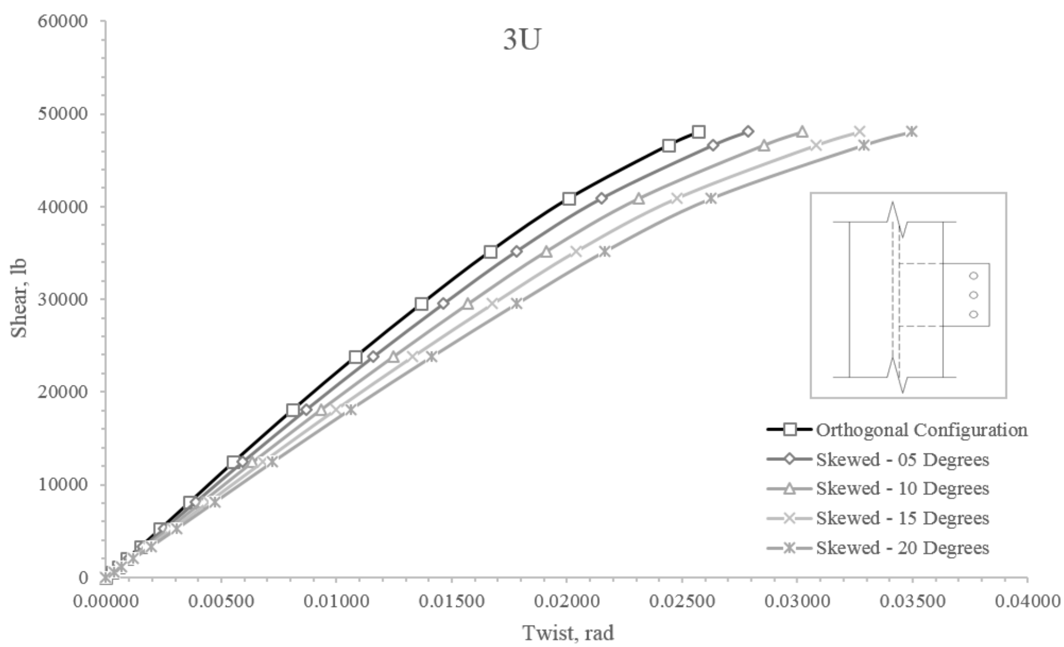


Figure 5-3. Connection shear-twist curve (3U)

In the orthogonal configuration, the shear force transferred to the supporting member generates a moment on the supporting member web, this moment results in increasing the connection vertical displacement for the orthogonal configuration. The shear tab twist for orthogonal configuration is only a function of the small eccentricity which is due to the overlap between the longitudinal axes of the beam and shear tab (Figure 5-4). Figure 5-5 shows the additional moment on the supporting column web for the orthogonal beam-to-column web connections.

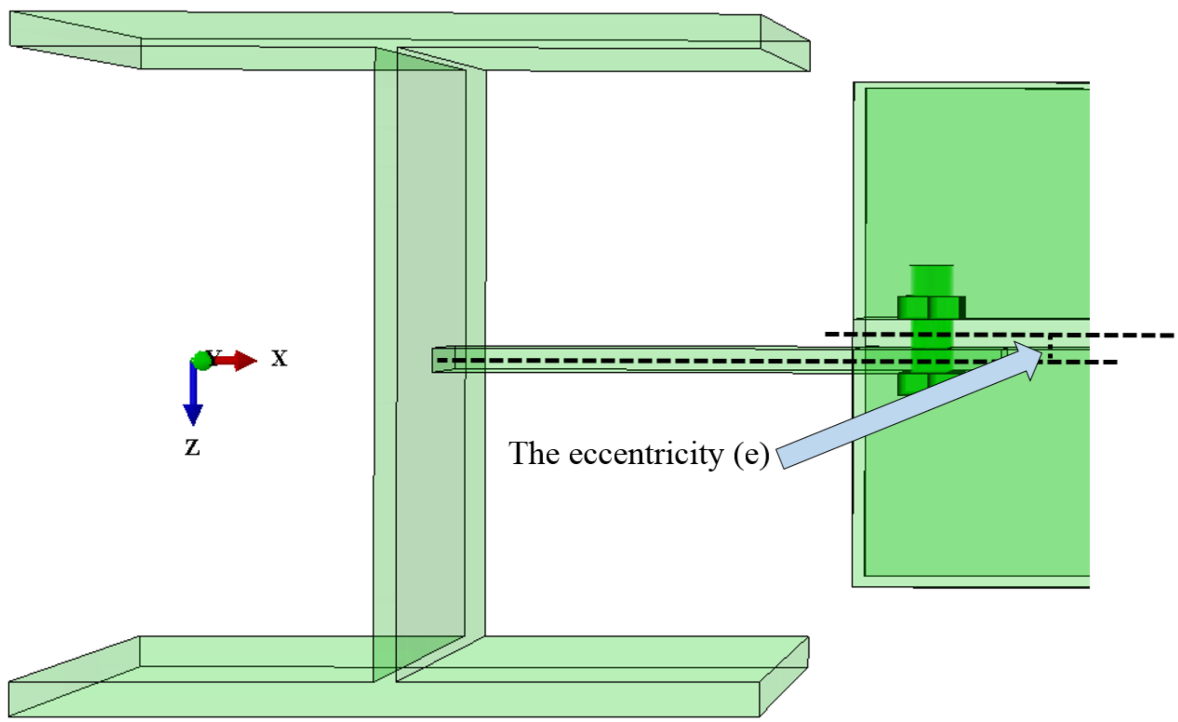


Figure 5-4. The overlap between the longitudinal axes of the beam and shear tab

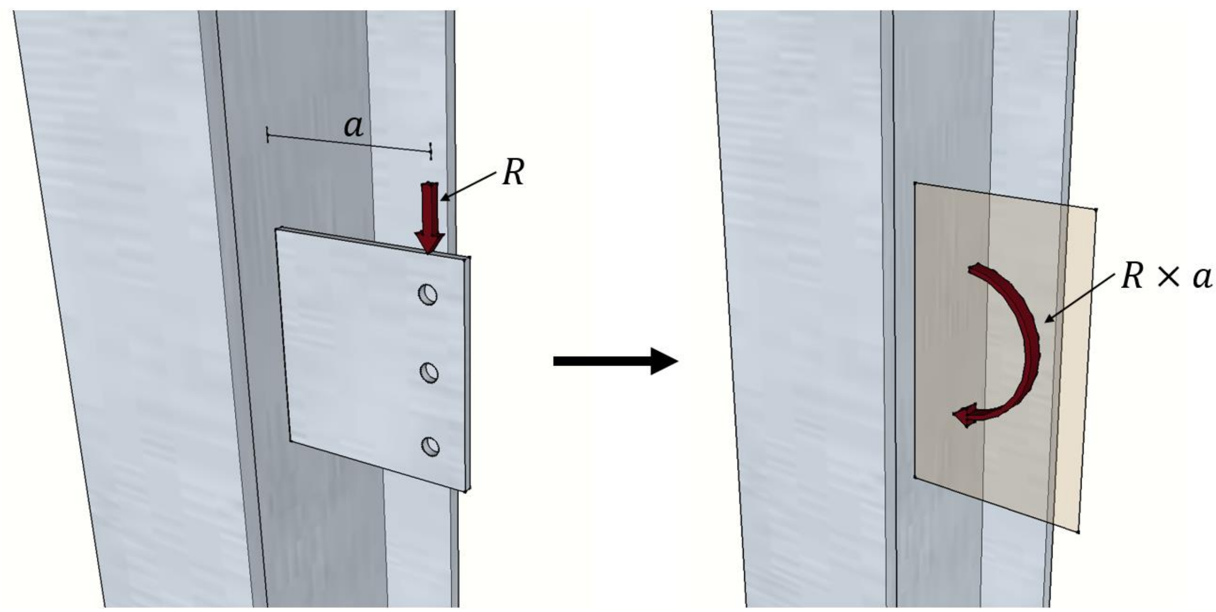


Figure 5-5. Applied moment on the supporting member web for orthogonal connections

For skewed connections, the shear force generates two moment components on the supporting member web ($R \times a \times \cos\alpha$ and $R \times a \times \sin\alpha$). The shear tab twist for these connections is a function of the overlap between the longitudinal axes of the beam and shear tab, and the moment applied at the weld line of the connection. Figure 5-6 shows the moment components on the supporting column web for skewed beam-to-column connections. The same failure modes were obtained from orthogonal and skewed connections at different connection orientations. Table 5-1 shows the ultimate shear, failure modes, maximum vertical displacement, and maximum twist for orthogonal and skewed connections discussed in this section. This table lists primary failure modes that are followed by secondary failure modes in parentheses. As a conclusion, the connection orientation affect primarily the connection torsional stability. Note that the welds have to resist these moment components for skewed configuration.

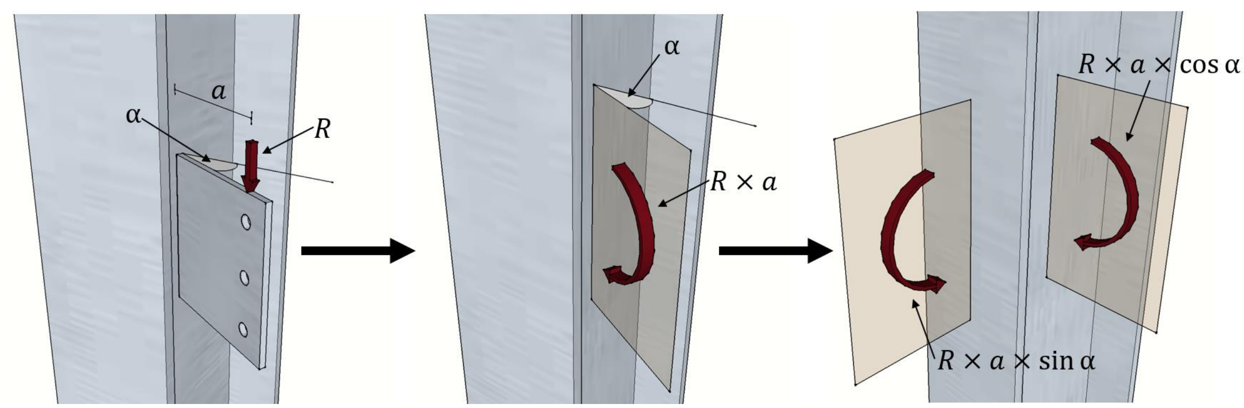


Figure 5-6. The moment components along the weld line for skewed connections

Table 5-1. FEA results for orthogonal and skewed configurations where the plate is welded to the column web

Test	α	V_{FEA} (kips)	Failure Modes	Displacement (in)	Twist (rad)
1-U	0	56.42	B (A,D,E)	0.75	0.04253
	5	56.5		0.74	0.04493
	10	56.49		0.77	0.04601
	15	56.35		0.7	0.048
	20	56.35		0.69	0.05138
3-U	0	48.66	D (A,B)	0.4	0.02625
	5	48.11		0.39	0.02785
	10	48.12		0.38	0.03021
	15	48.12		0.37	0.03271
	20	48.12		0.35	0.03497
4-U	0	86.97	E (A,D,B)	0.44	0.03554
	5	86.97		0.44	0.03751
	10	86.97		0.44	0.03974
	15	86.97		0.43	0.04201
	20	86.97		0.41	0.04406
6-U	0	125.39	D,A (B)	0.6	0.00131
	5	125.44		0.6	0.00143
	10	125.31		0.57	0.00153
	15	125.57		0.54	0.00163
	20	125.61		0.51	0.0017
6-UB	0	124.65	D,A (B,E)	0.62	0.04764
	5	124.57		0.62	0.05002
	10	124.72		0.62	0.05261
	15	124.75		0.58	0.05514
	20	124.42		0.54	0.05665
8-U	0	192.93	D,A (B)	0.43	0.00132
	5	192.92		0.43	0.00142
	10	192.86		0.42	0.00151
	15	192.96		0.4	0.00159
	20	192.58		0.37	0.00167

* Failure modes:

A = bolt shear

B = bolt bearing of the plate

C = shear rupture

D = web mechanism

E = twist

5.2. Unstiffened Connections Supported by Supporting Member's Flange (Rigid Supports)

Each orthogonal model was modified by changing the orientation of the supported beam and shear tab in such a way that the skewed beam's longitudinal axis has an angle α with the axis perpendicular to the column flange. The shear tab geometry was modified by extending the plate to the surface of the column flange to insure full contact between the plate and column flange. Local coordinate systems were assigned to the shear tab, supported beam and each bolt in order to adjust the plate orientation, bolts pre-tensioning forces and far end reactions with the beam orientation. Figure 5-7 shows the orthogonal and skewed extended shear tab connection.

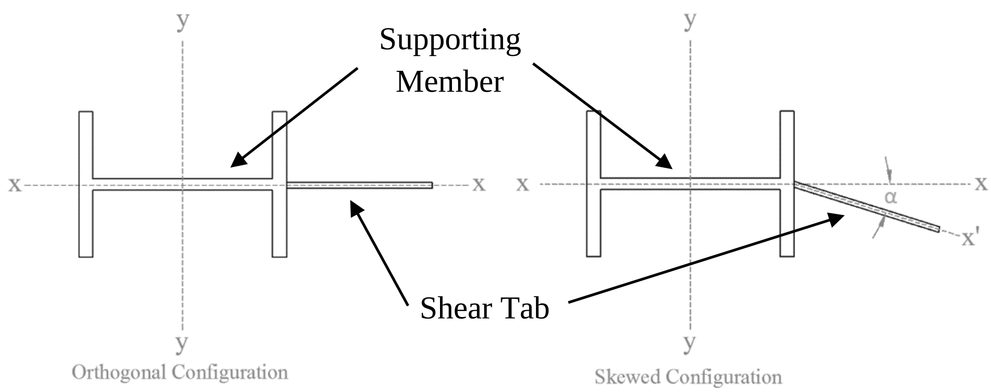


Figure 5-7. Extended shear tab connections with shear tab welded to the supporting member flange

Connection shear-displacement curves, shear-twist curves, and failure modes for each connection at different skewed angles were obtained and investigated. Figures 5-8 and 5-9 show connection shear-vertical displacement and connection shear-twist curves for orthogonal and skewed configurations of six bolted beam to column flange connection. These figures for the other connections can be found in Appendix A. Figure 5-8 and Figure 5-9 show that the vertical displacement and shear tab twist slightly affected by the connection orientation.

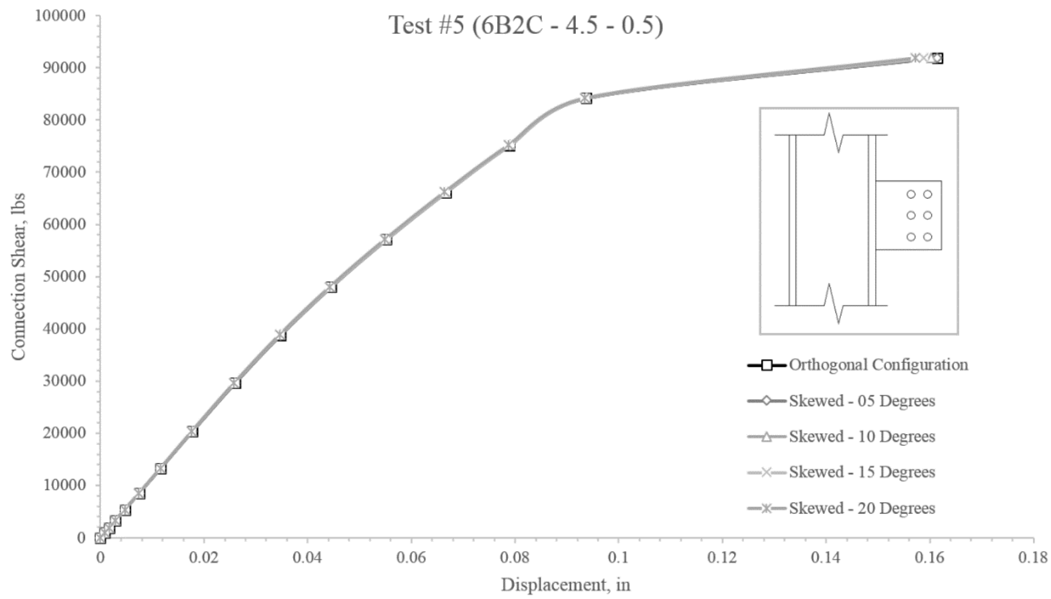


Figure 5-8. Connection shear-displacement curve for six bolted, beam-to-column flange connection

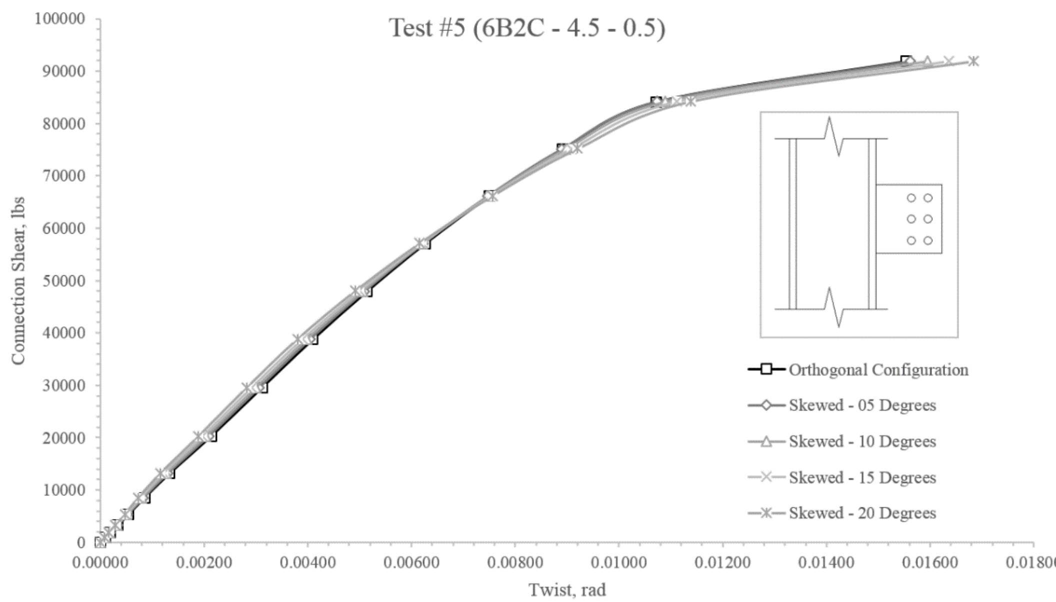


Figure 5-9. Connection shear-twist curve for six bolted, beam-to-column flange connection

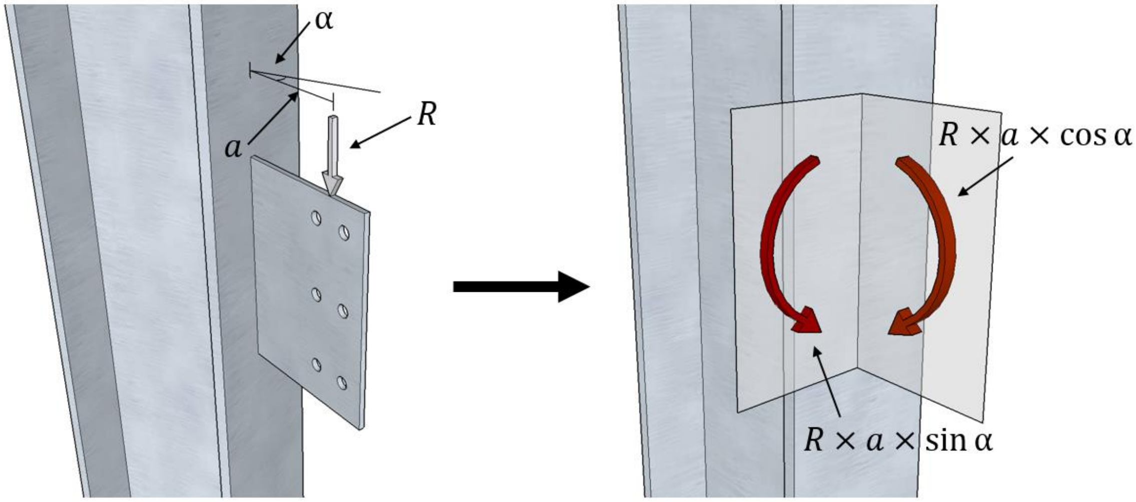


Figure 5-10. Additional moment components on the supporting column

Bending moment ($R \times a \times \cos\alpha$) about the column strong axis, and bending moment ($R \times a \times \sin\alpha$) about the column flange strong axis shown in Figure 5-10 are created as a result of the shear force, a-distance and shear tab orientation. It was observed from the FEA that the column contributes significantly in resisting the moment components applied at the weld line since the moment component ($R \times a \times \cos\alpha$) and moment component ($R \times a \times \sin\alpha$) are applied along the column strong axis and column flange strong axis, respectively. Figure 5-11 shows the stress distribution along the column at the ultimate capacity of the six bolted beam-to-column flange connection. This explains the slight effect of the connection orientation on the bending and torsional behaviors of the skewed extended shear tab connections with the shear tab welded to the flange of the supporting column. Note that in Figure 5-11, "S, Misses" is the von misses stresses.

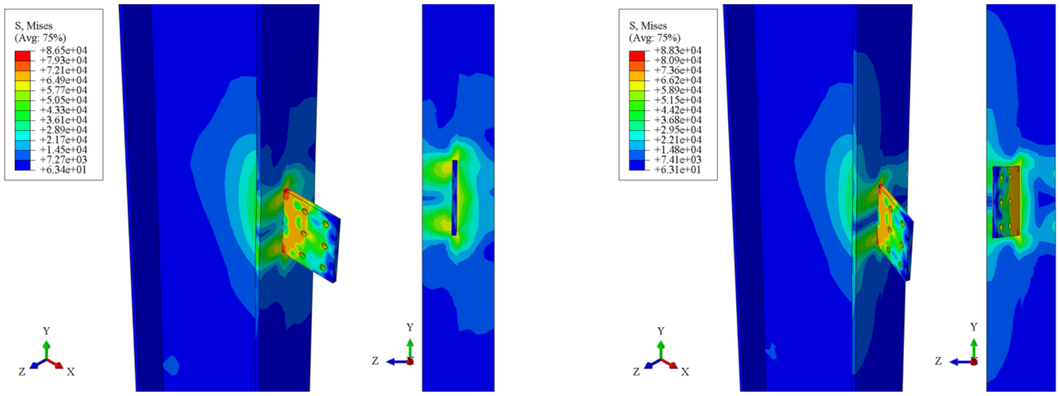


Figure 5-11. Stress distribution of orthogonal and skewed connections with rigid support

Table 5-2 shows the ultimate shear, failure modes, maximum vertical displacement, and maximum twist for connections with shear tab welded to the supporting column flange. This table lists primary failure modes that are followed by secondary failure modes in parentheses. The same failure modes were obtained from the orthogonal and skewed configurations at different connection orientation. In conclusion, for skewed extended shear tab connections with the shear tab welded to the column flange, the column contributes significantly in resisting the moment components since the moment component ($R \times a \times \cos\alpha$) and moment component ($R \times a \times \sin\alpha$) are applied along the column strong axis and column flange strong axis, respectively. Thus, the effect of the connection orientation on the connection bending and torsional behavior is insignificant and can be neglected.

Table 5-2. FEA results for orthogonal and skewed configurations where the plate is welded to the column flange

Test	α	V_{FEA} (kips)	Failure Modes	Displacement (in)	Twist (rad)
6B2C - 4.5 - 1/2	0	91.93	E, (H)	0.161	0.01553
	5	91.93		0.161	0.01562
	10	91.92		0.16	0.01595
	15	91.92		0.159	0.01636
	20	91.92		0.157	0.01684
7B2C - 9 - 3/8	0	104.98	G, B, F, (C, A, D)	0.17	0.04695
	5	104.85		0.171	0.04857
	10	104.71		0.158	0.04652
	15	104.6		0.164	0.05006
	20	104.49		0.17	0.05377
10B2C - 10.5 - 1/2	0	103.32	I, (G, F, H, E)	0.28	0.04881
	5	103.33		0.278	0.04981
	10	103.34		0.278	0.05169
	15	103.33		0.279	0.05459
	20	103.35		0.292	0.0613

* Failure modes:

A = bolt shear

B = bolt bearing of the beam web

C = shear yield

D = twist

E = weld

F = plate buckling

G = LTB of the beam flanges at midspan

H = yielding of plate corners

I = local buckling of the beam web at midspan

5.3. Stiffened Connections with flexible Supports

In the orthogonal configuration, webs of the beam and supporting member are framed at a right angle. Each extended connection was modified by rotating the supported beam and the shear tab in such a way that the beam longitudinal axis has an angle α with the axis perpendicular to the supporting member web. Four angles were used (5, 10, 15 and 20 degrees). When using angles larger than 20 degrees, the flanges of beam intersect with the flanges of the supporting member, in this case, the flanges of the beam can be coped. However, this will affect the connection behavior and make the process of deriving relationships between the skewed angle and different parameters harder since the flange coping will be different as the skewed angle changes. The shear tab geometry and the interaction between the plate and the stiffeners were modified to insure full contact between the shear tab, supporting member, and stiffeners. Local coordinate systems were assigned to the shear tab, supported beam and each bolt in order to adjust the shear tab orientation, bolts pre-tensioning forces and far end reactions with the beam orientation. Figure 5-12 shows the stiffened extended shear tab connection for the orthogonal and skewed configurations.

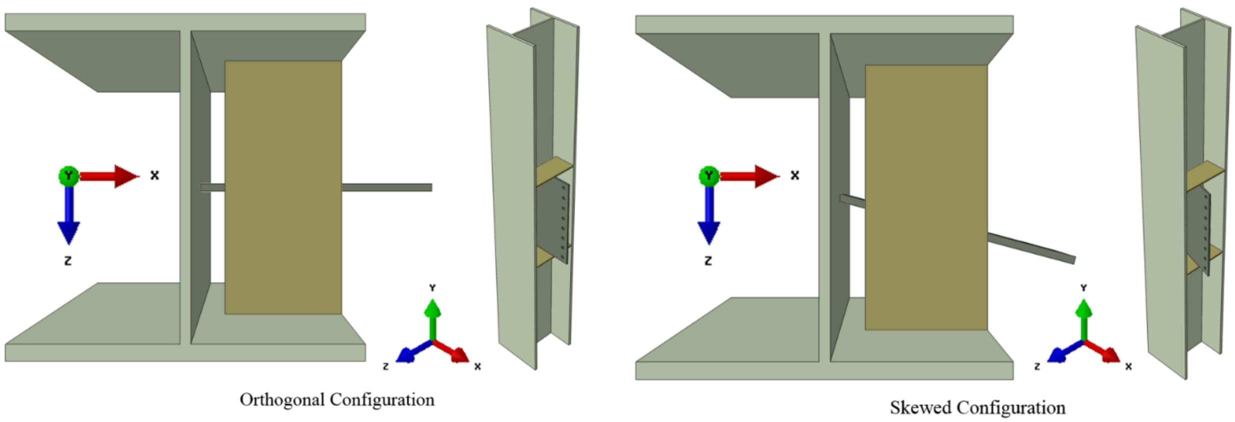


Figure 5-12. Orthogonal and skewed stiffened extended shear tab connections

The connection shear-vertical displacement curves, shear-shear tab twist curves, and failure modes for each model at different skewed angles (α) were obtained and investigated. It was observed that the connection vertical displacement and plate twist are slightly affected by the connection orientation. This is because the shear tab unsupported length for stiffened connections is short due to the presence of the stiffeners which makes the connection act like the conventional configuration with an a-distance of 3.5 in. maximum. Although the torsional moment component increases with the increase of the connection orientation, the welded area between the plate and the stiffeners increases with the increase of the connection orientation. This will increase the length of the horizontal weld group leading to shifting the center of gravity of the total weld group closer to the bolt line, leading to the reduction of the a-distance. As a conclusion, the effect of the connection orientation on the connection vertical displacement and shear tab twist for stiffened configuration is insignificant and can be neglected. Figures 5-13 and 5-14 show the connection's shear-vertical displacement and the connection's shear-twist curves for orthogonal and skewed configurations of stiffened three bolted beam to column web connection, respectively. These figures for the other connections can be found in Appendix A.

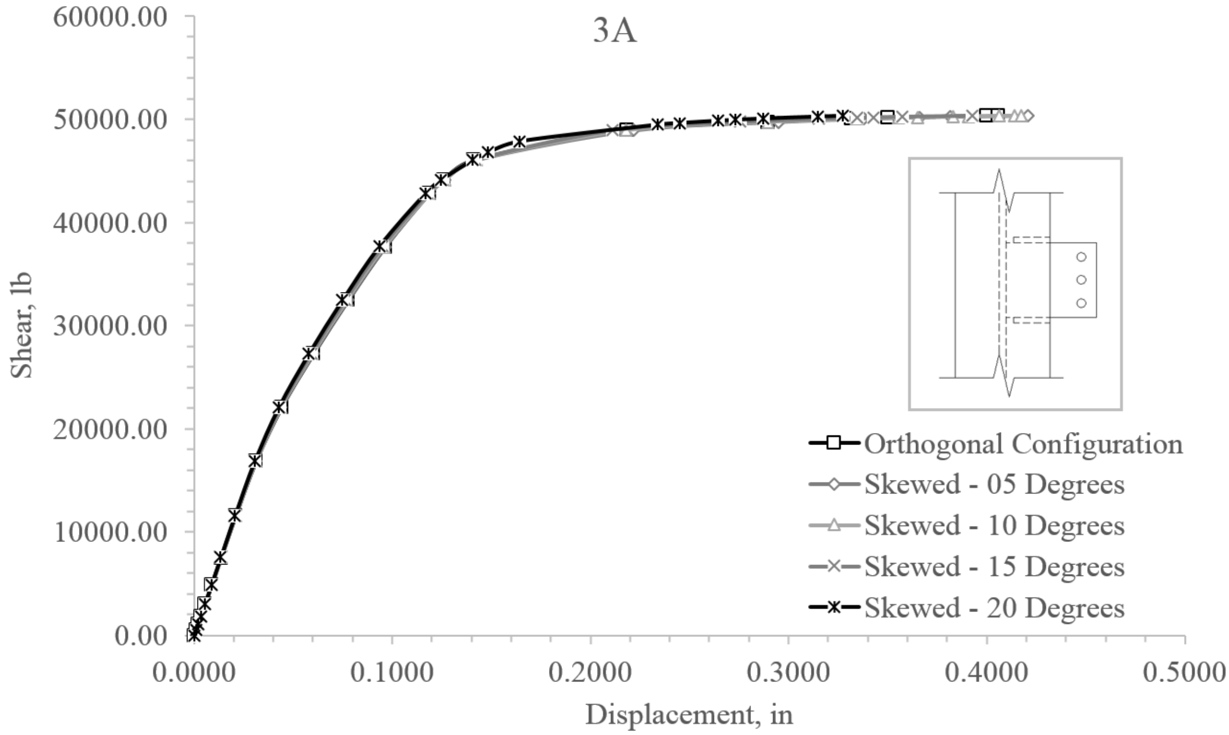


Figure 5-13. Connection shear-displacement curve for stiffened three bolted, beam-to-column web connection

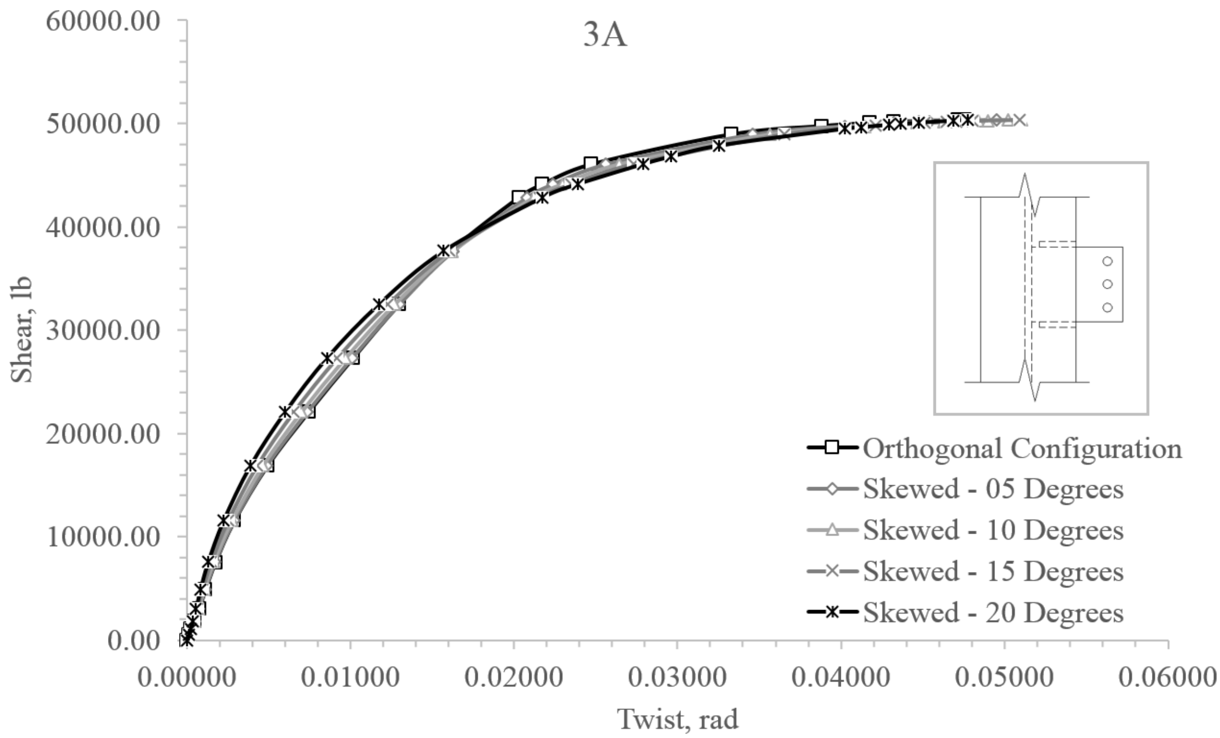


Figure 5-14. Connection shear-twist curve for stiffened three bolted beam-to-column web connection

Table 5-3 shows the ultimate shear, failure modes, maximum vertical displacement, and maximum twist for stiffened connections with shear tab welded to the supporting member web. This table lists primary failure modes that are followed by secondary failure modes in parentheses. The same failure modes were obtained from the orthogonal and skewed configurations at different connection orientation. As a conclusion, for stiffened skewed extended shear tab connections with the shear tab welded to the supporting member web, the stiffener plates contribute significantly in resisting the additional moment components and by reducing the unbraced length of the shear tab. Thus, the effect of the connection orientation on the connection bending and torsional behavior is insignificant and can be neglected.

Table 5-3. FEA results for orthogonal and skewed configurations where the plate is welded to the column flange

Test	α	V_{FEA} (kips)	Failure Modes	Displacement (in)	Twist (rad)
3A	0	50350	B, D (E)	0.4059	0.04760
	5	50362		0.4206	0.04949
	10	50345		0.4172	0.05102
	15	50368		0.3929	0.05095
	20	50348		0.3272	0.04773
4A	0	98889	B, D, E	0.1588	0.04534
	5	99395		0.1626	0.04508
	10	99910		0.1631	0.04440
	15	100726		0.1625	0.04286
	20	102629		0.1684	0.04101
4B	0	99206	B, D (A, E)	0.4610	0.01581
	5	98996		0.4878	0.01670
	10	99418		0.4834	0.01635
	15	100102		0.4886	0.01591
	20	100809		0.4838	0.01529
5A	0	145281	E (A, B)	0.4487	0.00605
	5	153968		0.8139	0.01304
	10	153978		0.7887	0.01195
	15	153993		0.7584	0.01144
	20	154015		0.7061	0.01030
7B	0	225669	B (A, C, F)	1.0737	0.00116
	5	213901		1.2066	0.00206
	10	213045		1.2423	0.00234
	15	212549		1.2137	0.00265
	20	211858		1.1471	0.00262
7C	0	181178	E, B (A, F)	0.3896	0.00121
	5	171530		0.2086	0.00100
	10	171133		0.2009	0.00102
	15	170388		0.1928	0.00103
	20	170038		0.1893	0.00108
8B	0	260227	B (A)	0.5276	0.00249
	5	260396		0.5437	0.00227
	10	260585		0.5159	0.00205
	15	261138		0.5049	0.00197
	20	265861		0.4827	0.00186

* Limit States and other Failure Modes:
A = bolt bearing   B = shear yield   C = web mechanism   D = twist
E = plate buckling   F = tearing   G = web shear

6. CHAPTER SIX - PARAMETRIC STUDY

The objective of this chapter is to investigate the interaction between different parameters on the connection behavior. These parameters are: connection orientation, plate thickness, and the a-distance. These parameters were investigated as a function of the connection vertical displacement, shear tab twist along the bolt line, shear tab twist along the weld line, and connection shear force. In order to achieve the goal of this chapter, 300 models were created using ABAQUS. The study was performed on three different connections: three bolted, five bolted, and eight bolted extended shear tab connections with five different plate thicknesses, four a-distances, and five different skewed angles. In this chapter, selected curves were used to explain the results, the rest of the parametric study curves are shown in Appendix B. Table 6-1 shows the selected values for each parameter for the three different connections.

Table 6-1. Variable Parameters

Connection	3 bolted-Connection	5 bolted-Connection	8 bolted-Connection
Plate Thickness (in)	1/4	5/16	5/16
	5/16	3/8	3/8
	3/8	7/16	7/16
	1/2	1/2	1/2
	5/8	5/8	5/8
a-distance (in)	7.0	10.5	10.5
	7.5	11.0	11.0
	8.0	11.5	11.5
	8.5	12.0	12.0
Skewed Angle (degrees)	0	0	0
	5	5	5
	10	10	10
	15	15	15
	20	20	20

Model identification codes were assigned to each connection. The identification code includes the number of bolts, the plate thickness, the a-distance, and the skewed angle. For example, the connection 3B-1/4-7-00 indicates a three-bolted connection with a plate thickness of 1/4 in., a-distance of 7 in., and a skewed angle of 0 degrees.

6.1. Three-Bolted Connection

Figure 6-1 shows the constant parameters for the three bolted connection. In this group, 100 models were created with five plate thicknesses (1/4", 5/16", 3/8", 1/2", 5/8"), four a-distances (7", 7.5", 8", 8.5"), and five skewed angles (0^0, 5^0, 10^0, 15^0, 20^0).

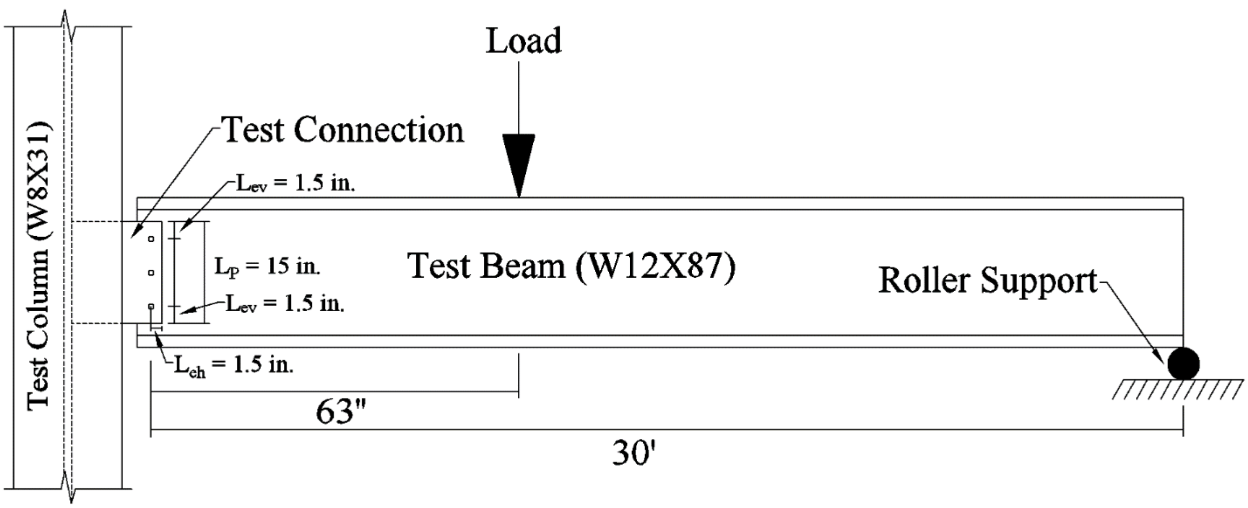


Figure 6-1. Constant parameters for the three bolted connection

Figure 6-2 shows the relationship between skewed angle and a-distance with the connection vertical displacement for twenty connections with the same plate thickness (t = 1/4") and three bolts. It was observed from this figure that for a certain a-distance, as the skewed angle increases, the connection vertical displacement slightly decreases.

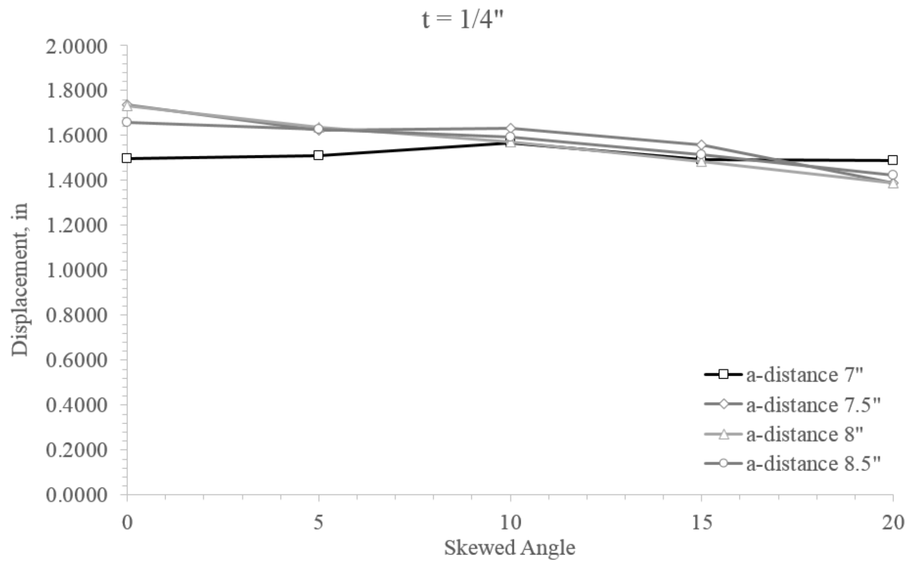


Figure 6-2. Vertical displacement-skewed angle curves (t = 1/4'')

Figures 6-3 and 6-4 show the relationship between skewed angle and a-distance with the plate twist along the weld line and relative twist. It was observed from these figures that for a certain a-distance, as the skewed angle increases, the plate twist along weld line increases significantly. But the relative twist (twist caused by the torsion due to the overlap between the beam web and shear tab longitudinal axes) is almost constant and independent from the connection orientation. However, the shear tab twist due to the overlap between the shear tab and beam web longitudinal axes (relative twist) decreases with the increase of the a-distance because of the reduction in the connection shear.

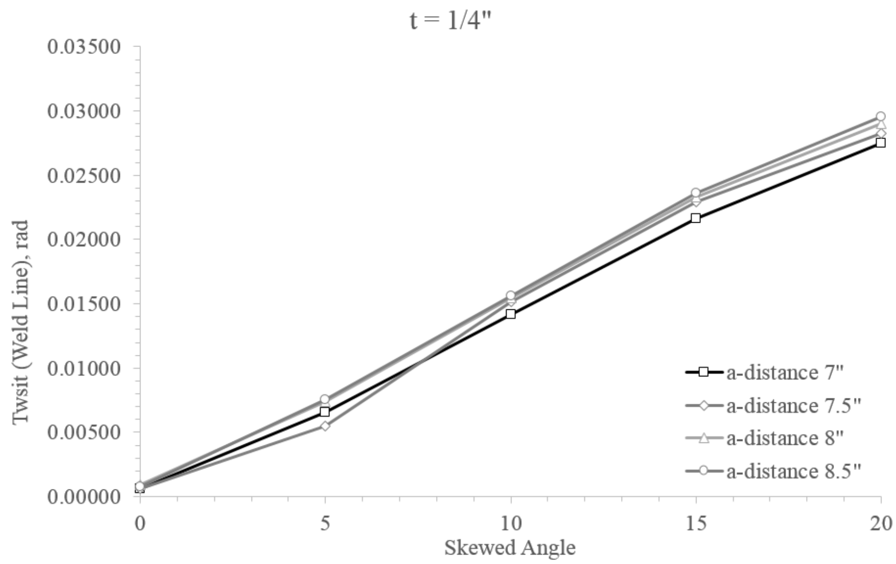


Figure 6-3. Plate twist along the weld line-skewed angle curves (t = 1/4’’)

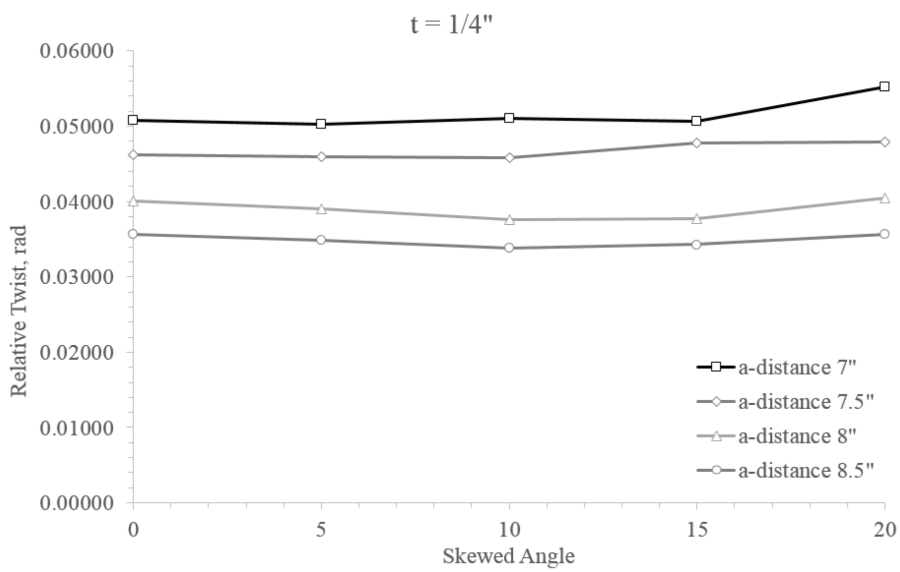


Figure 6-4. Relative twist-skewed angle curves (t = 1/4’’)

Figures 6-5 shows the relationship between skewed angle and a-distance with the connection shear capacity. It is observed that the connection shear capacity is almost constant and independent from the connection orientation, and decreases with the increase of the a-distance due to the increase of the applied bending moment on the connection. Figures 6-6 and 6-7 show the interaction between the plate thickness and skewed angle with the connection vertical displacement, and relative twist for twenty connections with the same a-distance (a = 7”) and three bolts, respectively.

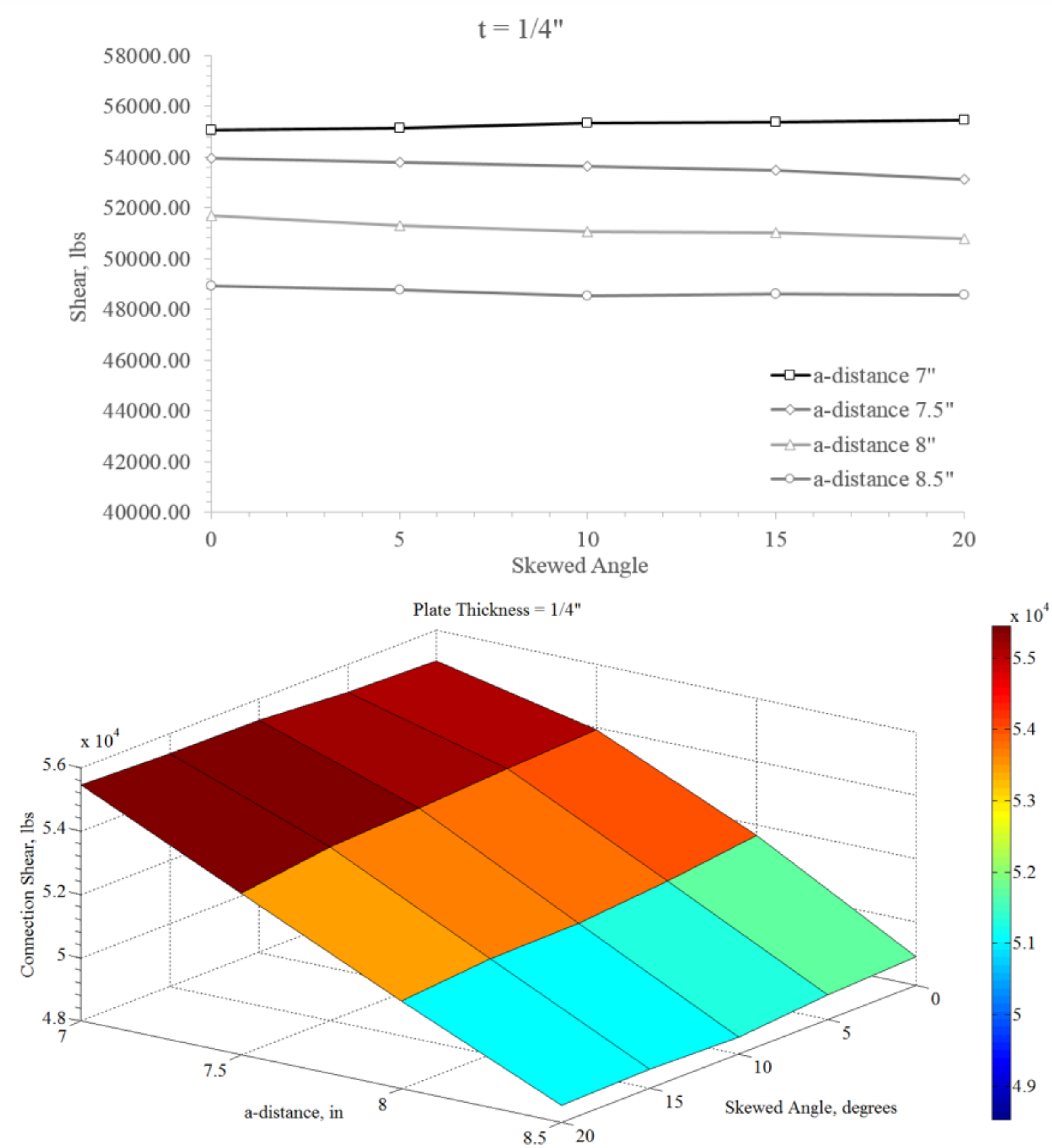


Figure 6-5. Connection shear-skewed angle curves (t = 1/4'')

It is observed from Figures 6-6 and 6-7 that for a certain skewed angle, the connection vertical displacement and the plate relative twist dropped significantly when the plate thickness was increased from 1/4" to 5/16". Then, these parameters were decreased slightly when using plate thicker than 5/16".

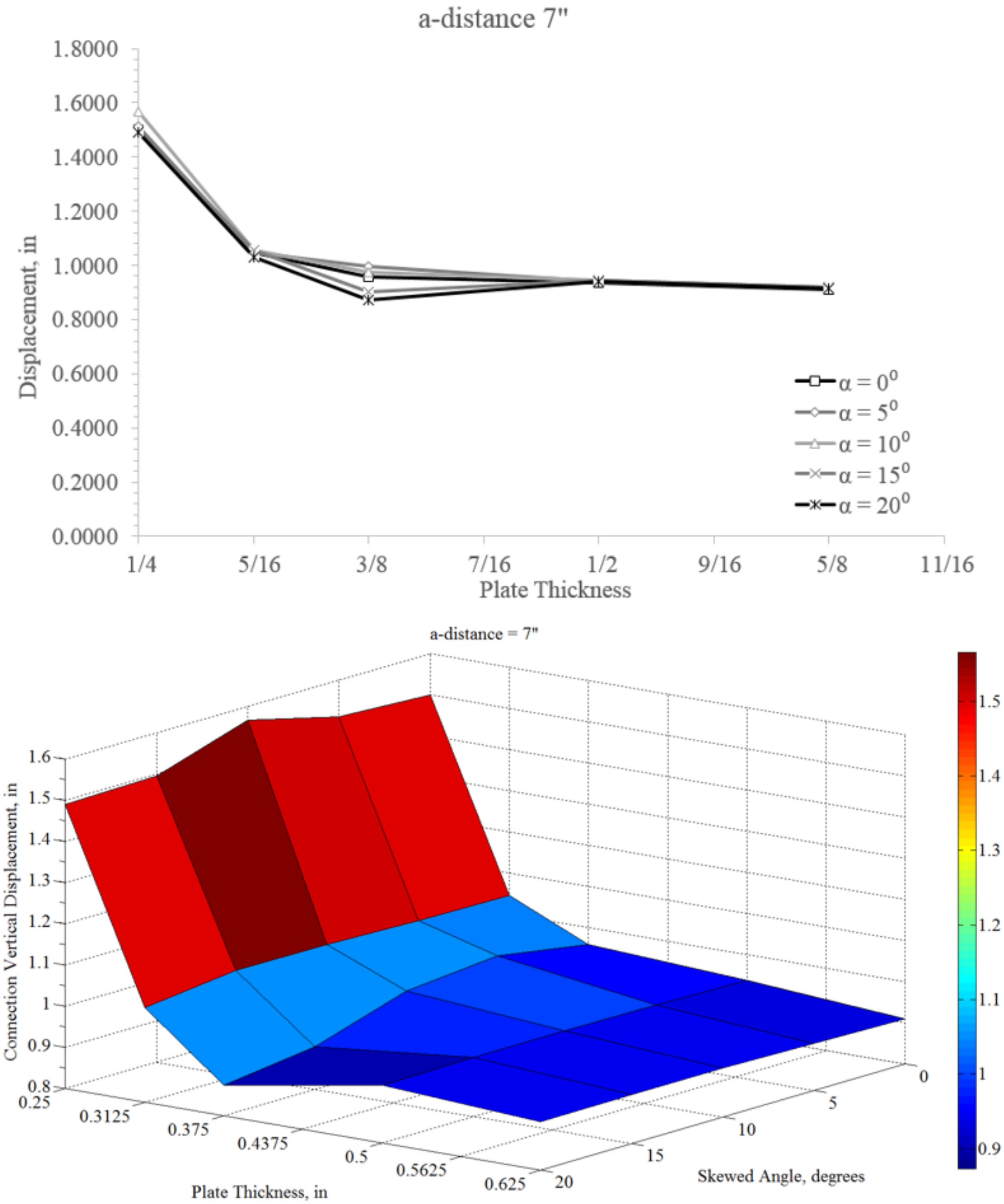


Figure 6-6. Vertical displacement-plate thickness curves (a-distance = 7")

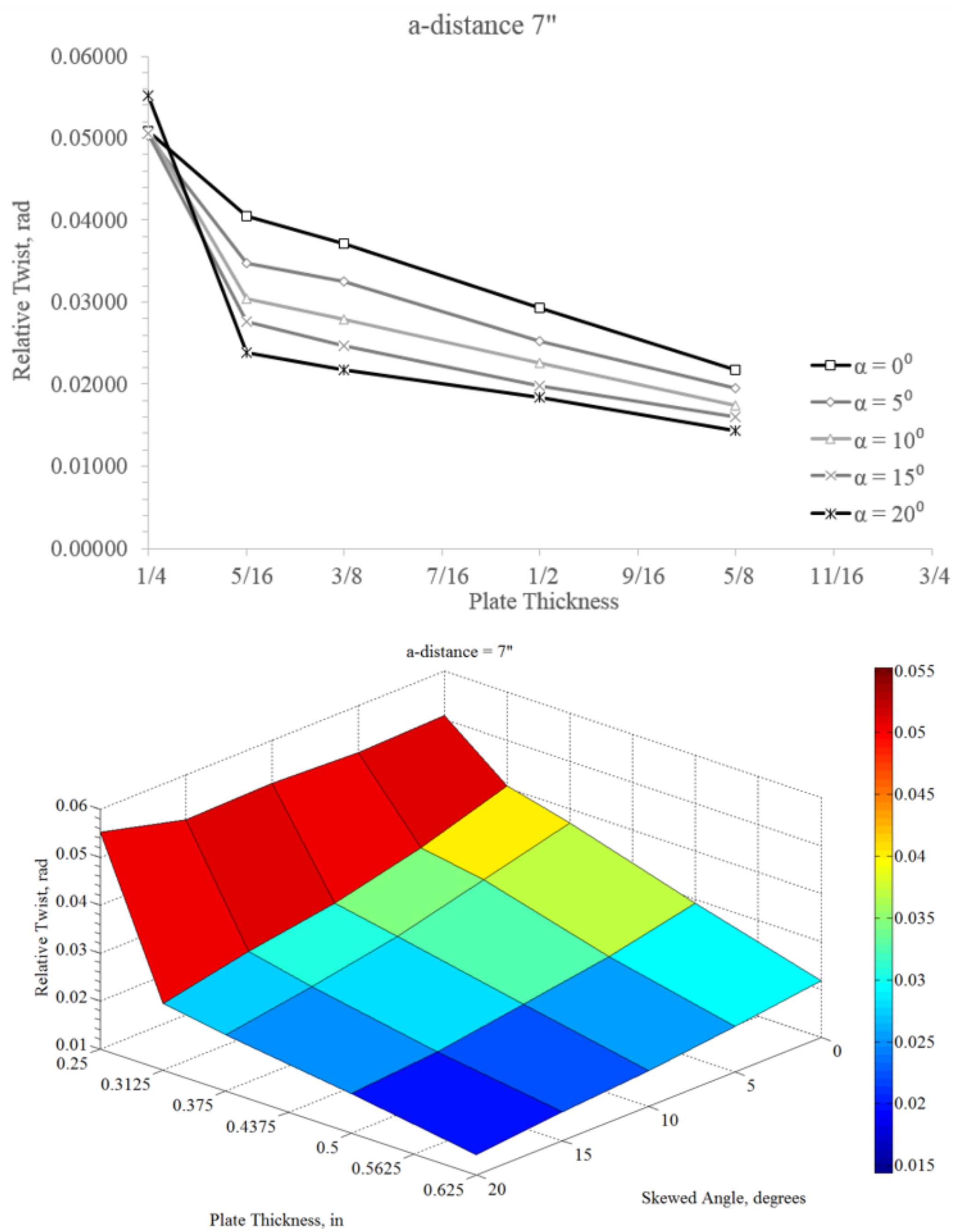


Figure 6-7. Relative twist- plate thickness curves (a-distance = 7")

Failure modes were investigated for each case. It was observed that the failure modes for connections with a plate thickness equals to 1/4" and an a-distance = 7" were: bolt bearing on the plate and bolt shear as a primary failure modes. Plate shear rupture, shear yielding, plate tearing, web mechanism, and bolt bearing on the beam web were secondary failure modes (Figure 6-8), these modes are ductile failure modes.

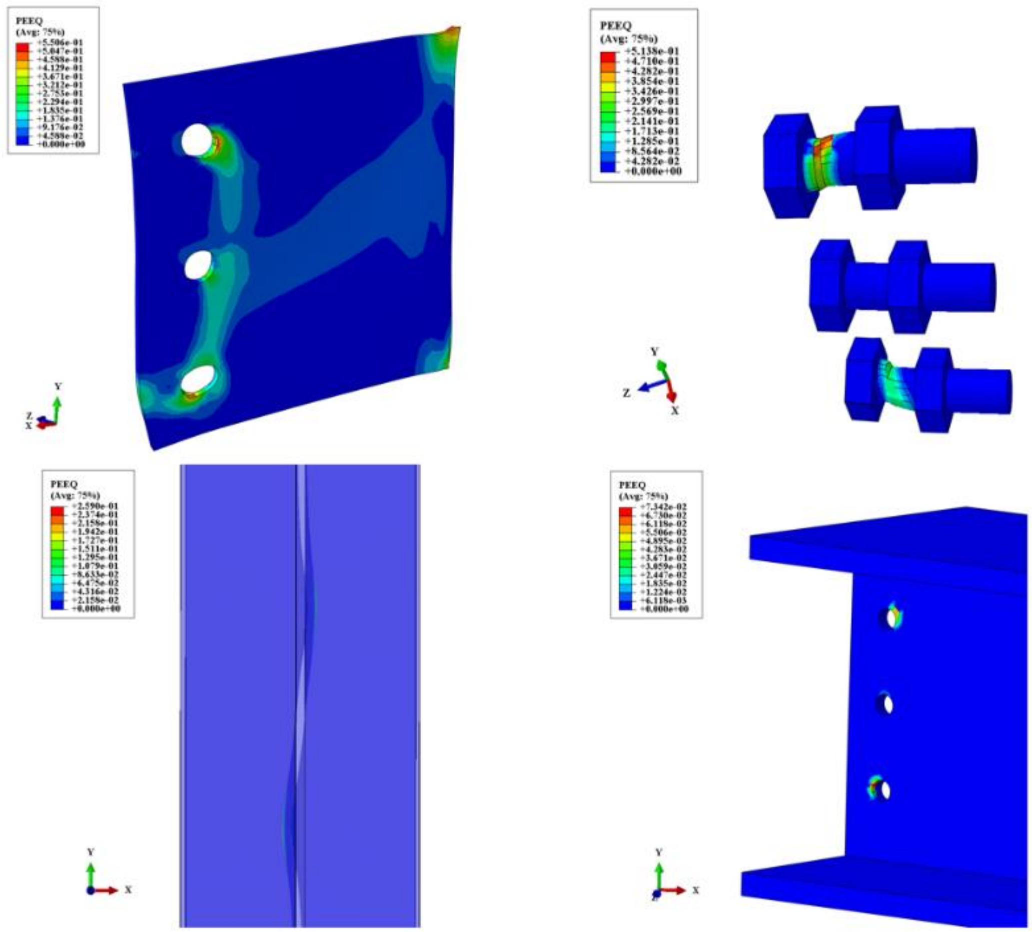


Figure 6-8. Failure modes of connection 3B-1/4-7-20

However, it was noticed that the primary failure mode for connections with thicker plates is bolt shear, and the secondary failure modes were bolt bearing on the plate and beam web, and web mechanism (Figure 6-9). But the plastic strain at these regions are very small compared with the plastic strain developed in the bolts. Thus, the effect of these secondary failure modes on the connection ductility is minimal. As the plate thickness increases, the primary failure mode remains the same (bolt shear). Thus, the connection behavior is primarily controlled by the bolt shear.

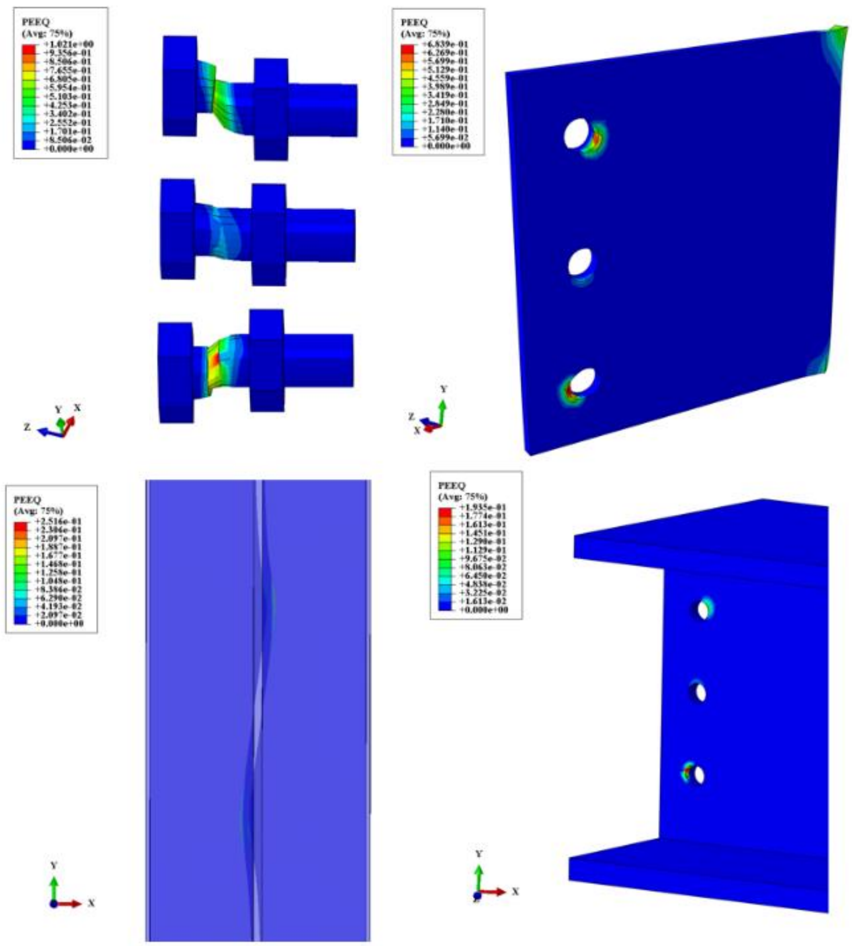


Figure 6-9. Failure modes of connections 3B-(5/16, 3/8, 1/2, and 5/8)-7-20

It is observed that the plate twist along the weld line is slightly affected with change of the plate thickness as shown in Figure 6-10. This might be related to the significant contribution of the supporting member in resisting the torsional moment applied along the weld line.

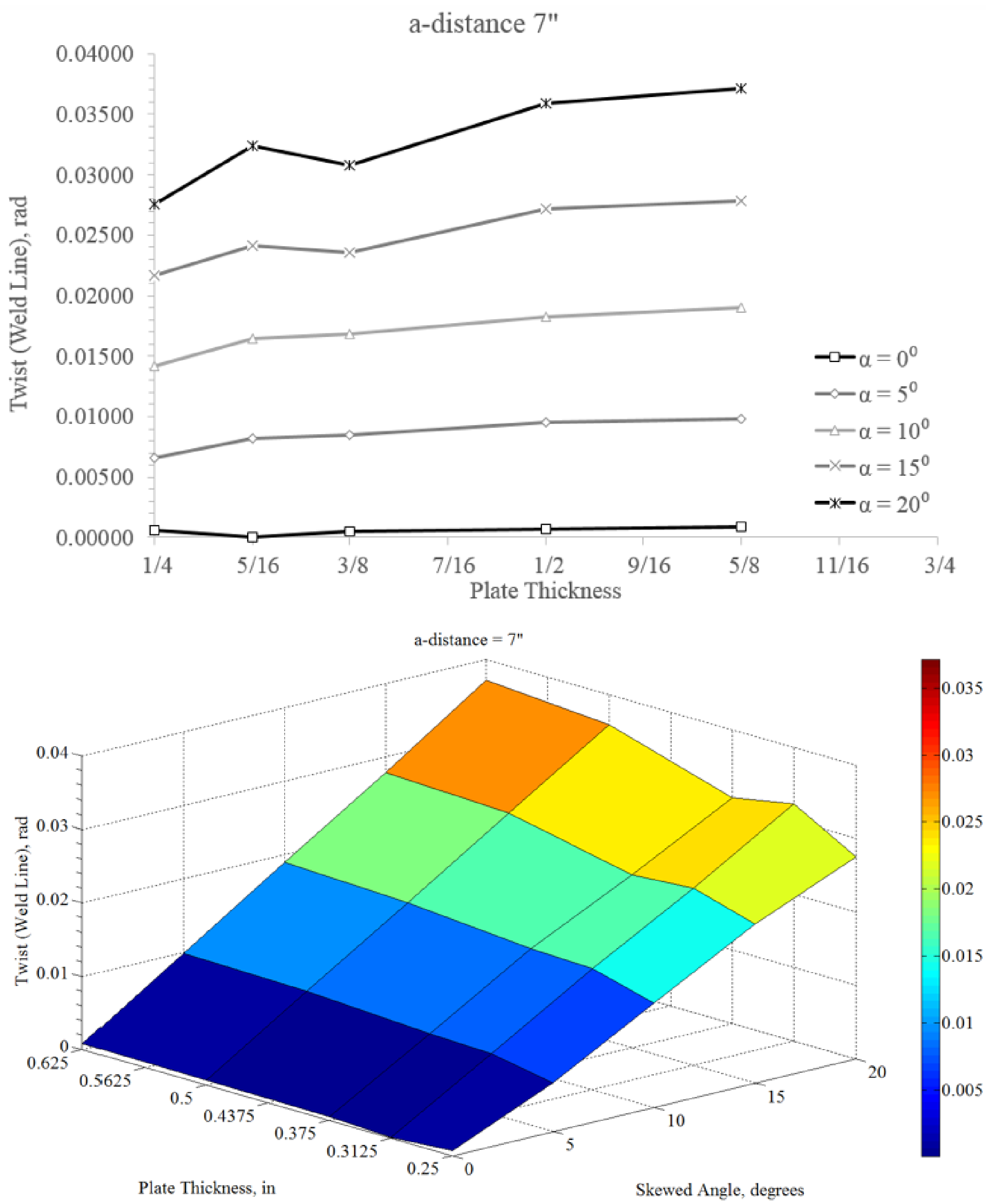


Figure 6-10. Plate twist along the weld line- plate thickness curves (a-distance = 7")

It is observed that the connection shear capacity is not affected by the change of the plate thickness as can be seen from Figures 6-11 and 6-12. The primary failure modes for these connections regardless of the skewed angle are bolt shear and/or bolt bearing on the plate; depending on the plate thickness; thus, any change in the plate thickness will only affect the secondary failure modes of the connections which slightly affect the shear capacity of the connection. Also, for a one shear tab thickness, the connection's shear capacity decreases with the increase of the a-distance due to the increase of the applied bending moment component on the connection.

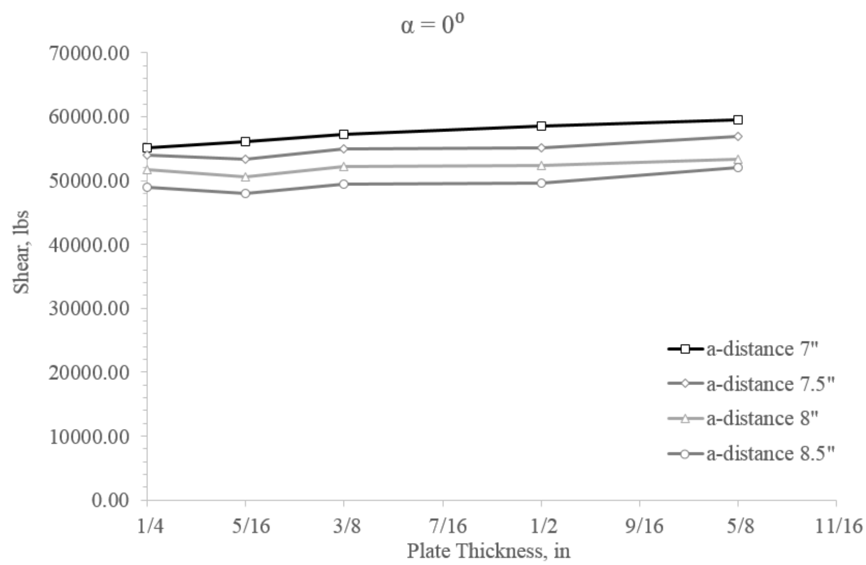


Figure 6-11. Connection shear- plate thickness curves ($\alpha = 0^0$) for 3 bolted connection

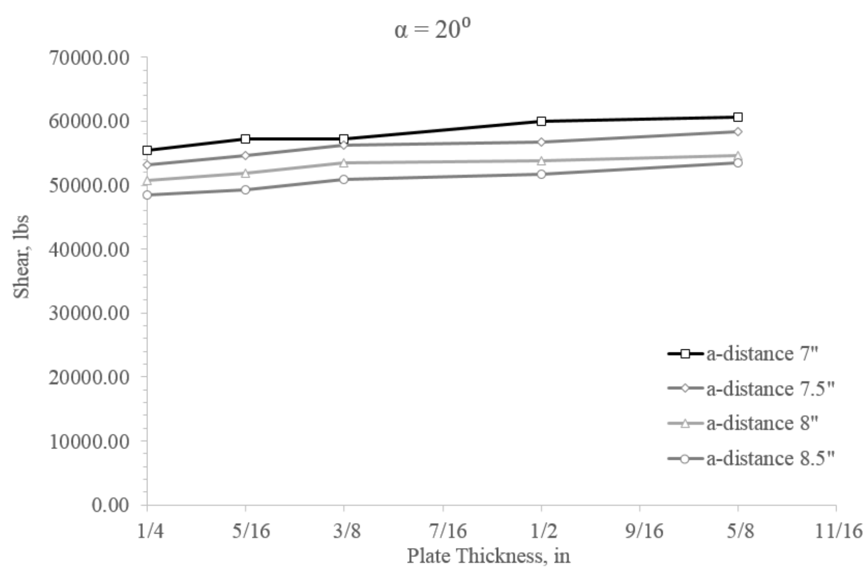


Figure 6-12. Connection shear- plate thickness curves ($\alpha = 20^0$) for 3 bolted connection

6.2. Five-Bolted Connection

Figure 6-13 shows the constant parameters for the five-bolted connection. In this group, 100 models were created with five plate thicknesses (5/16”, 3/8”, 7/16”, 1/2”, 5/8”), four a-distances (10.5”, 11”, 11.5”, 12”), and five skewed angles (0^0, 5^0, 10^0, 15^0, 20^0).

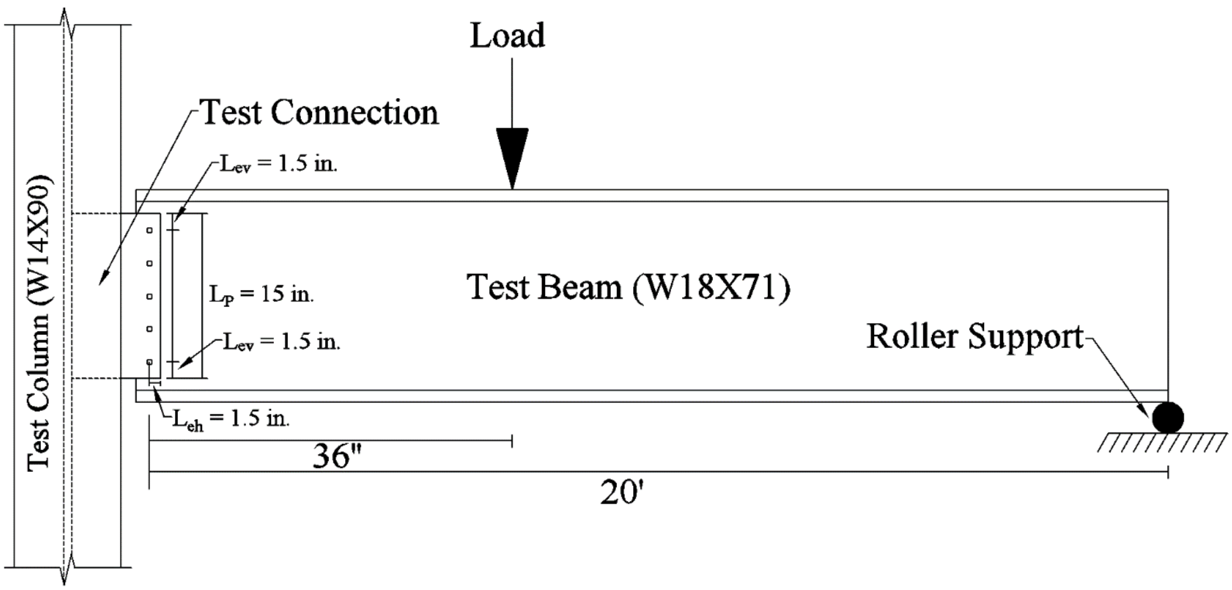


Figure 6-13. Constant parameters for the five bolted connection

Figure 6-14 shows the relationship between skewed angle and a-distance with the connection vertical displacement for twenty connections with the same plate thickness (t = 3/8”) and five bolts. It was observed from this figure that for a one a-distance, the connection orientation has an insignificant effect on the connection vertical displacement.

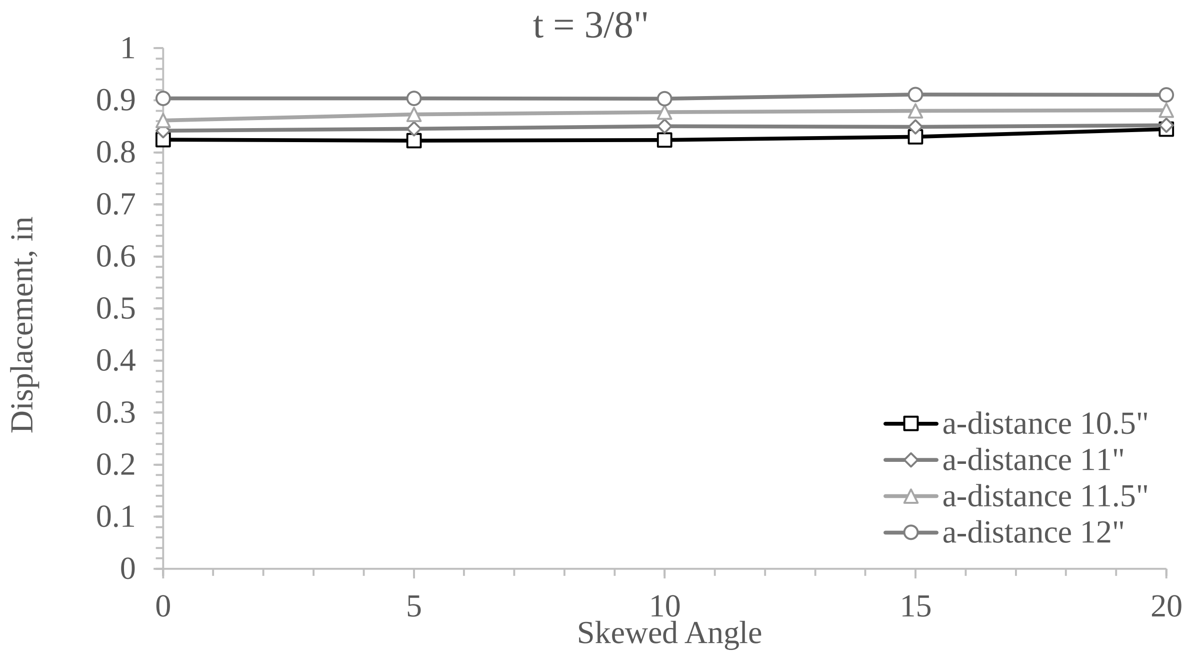


Figure 6-14. Vertical displacement-skewed angle curves (t = 3/8")

Figures 6-15 and 6-16 show the relationship between skewed angle and a-distance with the plate twist along the weld line and relative twist. It was observed from these figures that for a certain a-distance, as the skewed angle increases, the plate twist along weld line increases significantly. But the relative twist (twist caused by the torsion due to the overlap between the beam web and shear tab longitudinal axes) is almost constant and independent from the connection orientation. However, the shear tab twist due to the overlap between the shear tab and beam web longitudinal axes (relative twist) decreases with the increase of the a-distance because of the reduction in the connection shear.

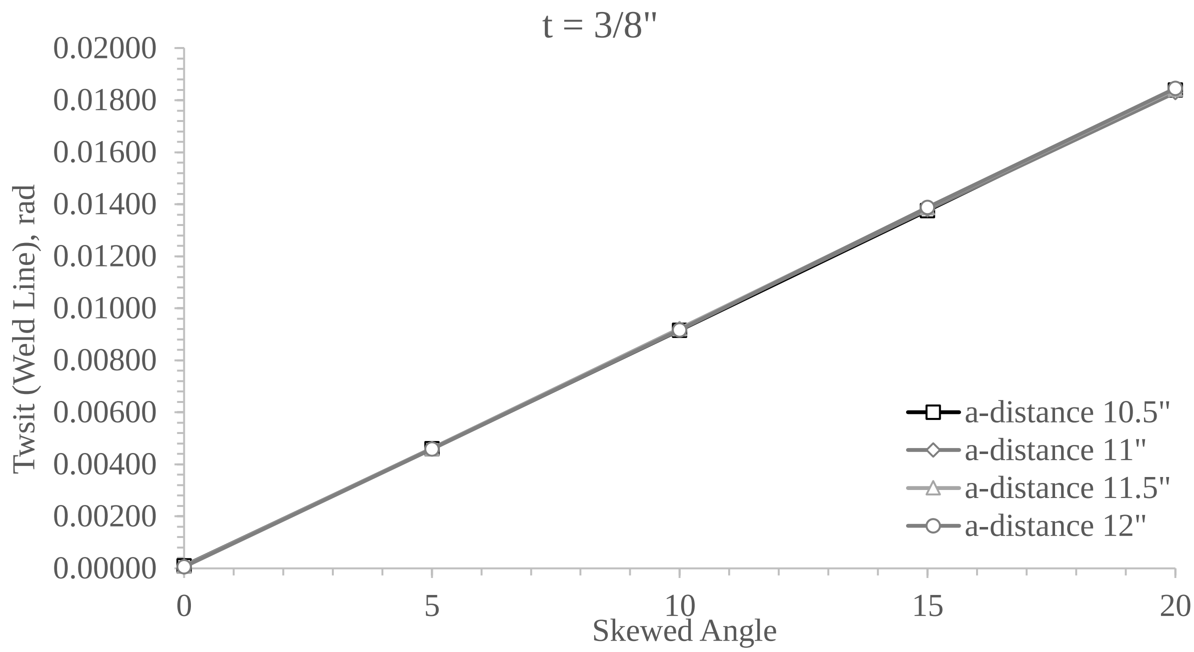


Figure 6-15. Plate twist along the weld line-skewed angle curves (t = 3/8'')

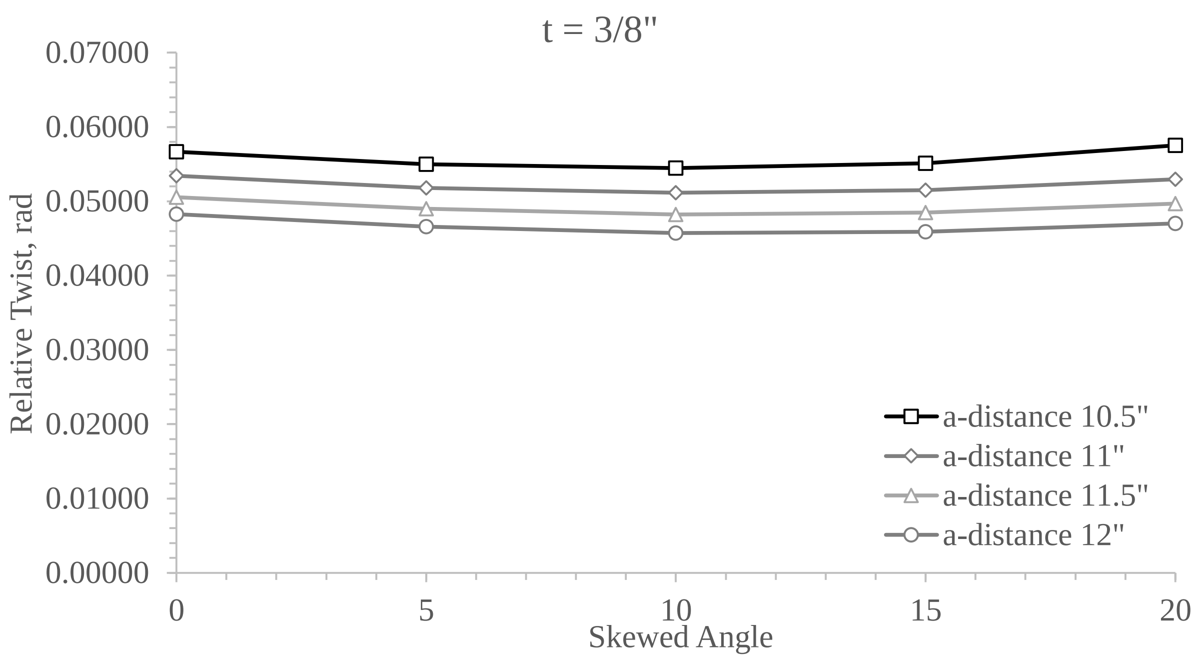
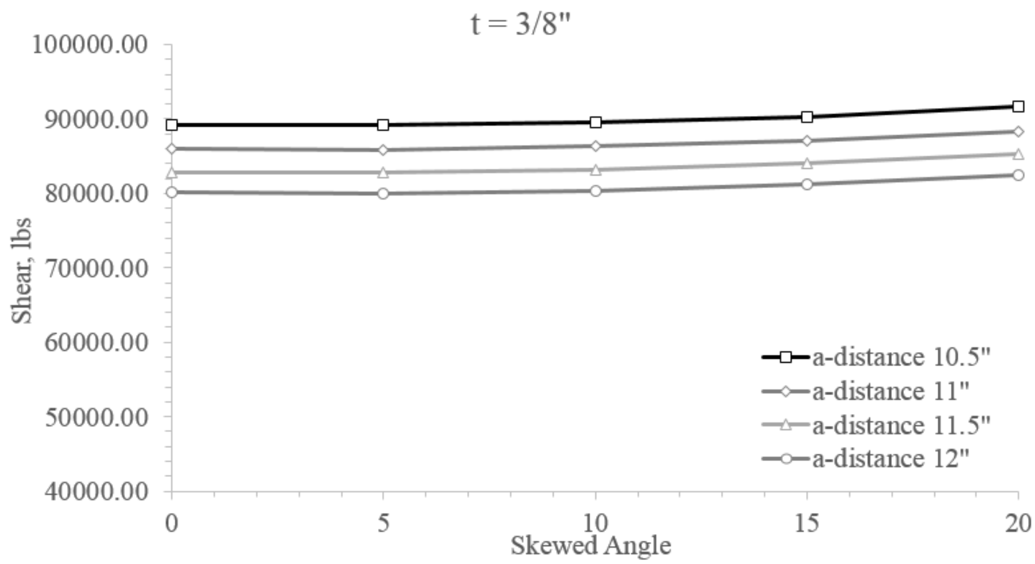


Figure 6-16. Relative twist-skewed angle curves (t = 3/8")

Figures 6-17 shows the relationship between skewed angle and a-distance with the connection shear capacity. For a certain a-distance, it is observed that the connection shear capacity is almost constant and independent from the connection orientation. For a one skewed angle, as the a-distance increases, the connection shear capacity decreases.



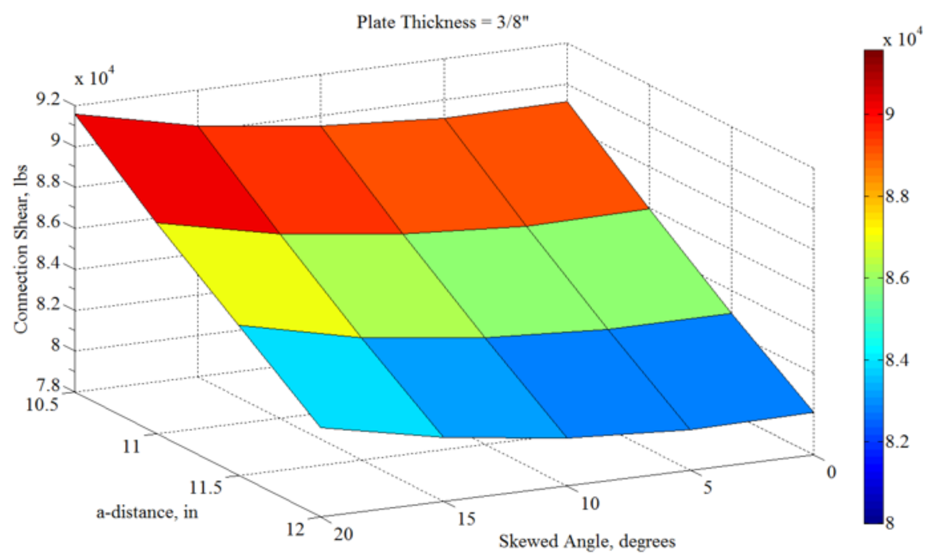


Figure 6-17. Connection shear-skewed angle curves (t = 3/8")

Figures 6-18 and 6-19 show the relationship between a-distance and plate thickness with the connection shear capacity for five-bolted connections with α = 0 and α = 20, respectively. It was observed that the connection shear capacity increases significantly with when using 3/8" plate instead of 5/16". For connections with 3/8" plate thick or greater, the connection shear capacity is not affected by the change of the plate thickness. Also, for a one shear tab thickness, the connection's shear capacity decreases with the increase of the a-distance due to the increase of the applied bending moment component on the connection.

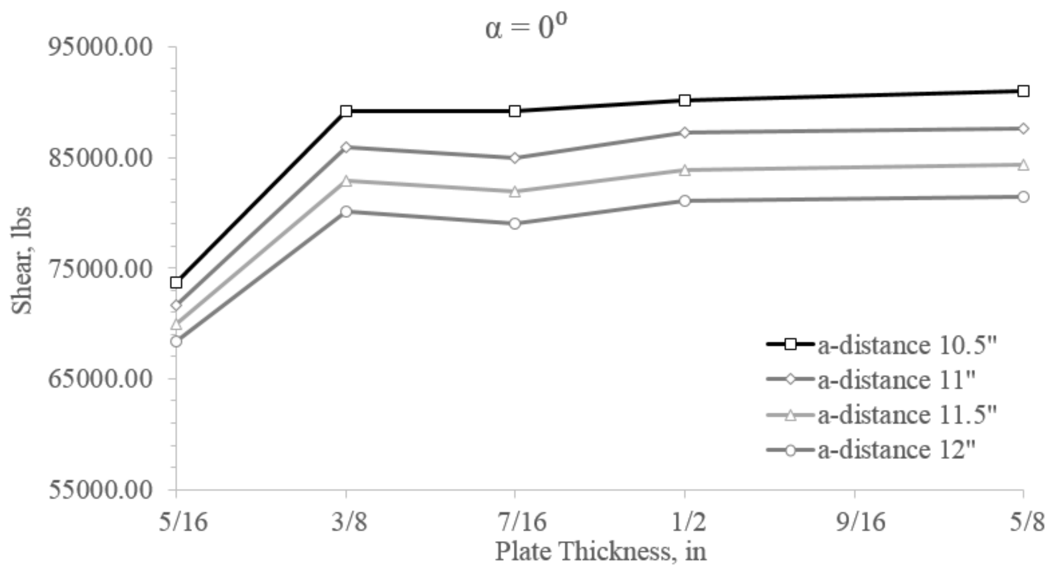


Figure 6-18. Connection shear-plate thickness curves ($\alpha = 0^0$) for 5 bolted connection

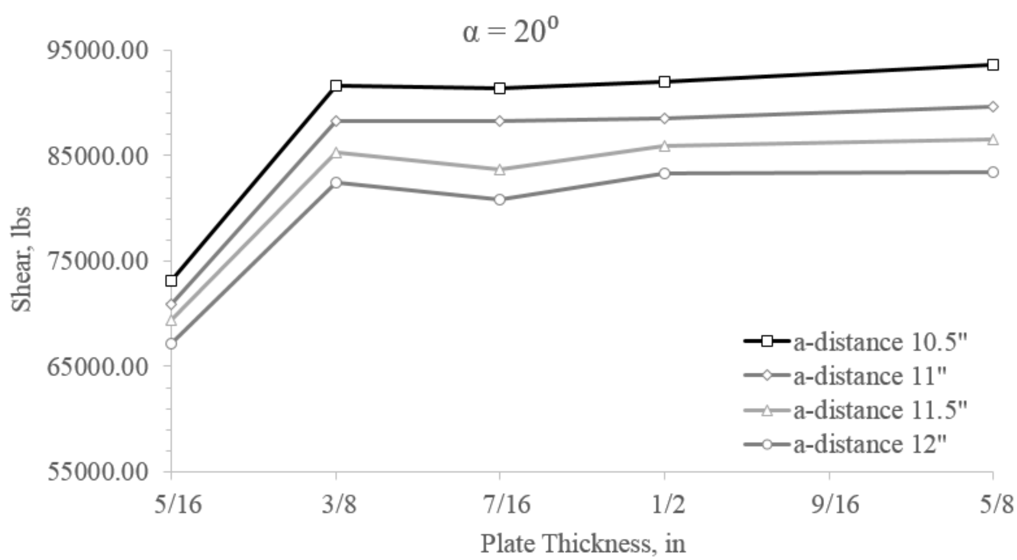


Figure 6-19. Connection shear-plate thickness curves ($\alpha = 20^0$) for 5 bolted connection

Figure 6-20 shows the failure modes of five bolted connection with 5/16" plate thickness. The primary failure mode for this connection is plate twist caused by the eccentricity between the plate and beam web longitudinal axes, while the secondary failure modes are bolt shear, plate shear rapture, and plate tearing. However, when using thicker plates (3/8”, 7/16”, 1/2”, 5/8), the plate torsional stiffness increases, and the primary failure mode become bolt shear, and the secondary failure mode is the plate twist (Figure 6-21), thus, the connection shear capacity increases significantly.

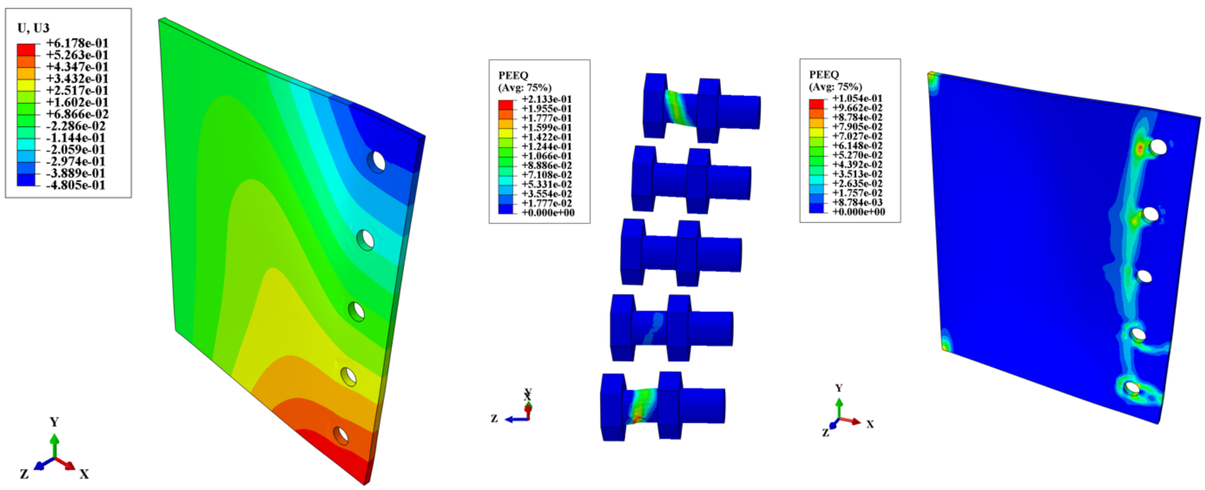
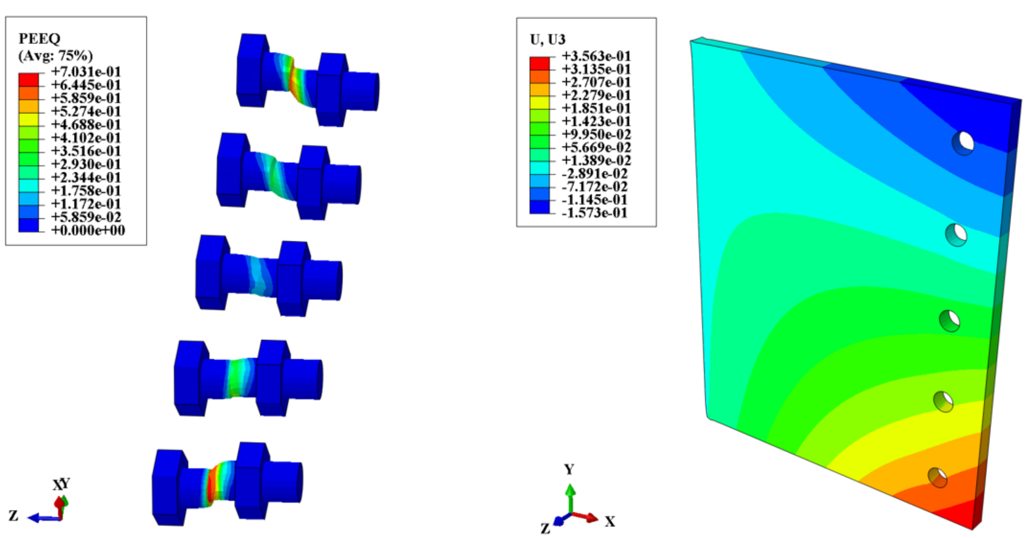


Plate Twist (Primary)

Bolt Shear, Plate Shear Rupture, Plate Tearing (Secondary)

Figure 6-20. Failure modes of connection 5B-5/16-10.5-20



Bolt Shear (Primary)

Plate Twist (Secondary)

Figure 6-21. Failure modes of connections 5B-(3/8, 7/16, 1/2, and 5/8)-10.5-20

For connections with 3/8” and thicker plates, as the plate thickness increases, the primary failure mode remains the same (bolt shear). Thus, the connection behavior is primarily controlled by this failure mode. This explains the slight change in the connection shear capacity as the plate thickness increases to more than 3/8”. The plate thickness plays a crucial rule in resisting the plate relative twist (plate twist caused by the eccentricity between the plate and beam web longitudinal axes). Figures 6-22 and 6-23 show the interaction between the plate thickness and skewed angle with the connection vertical displacement and plate twist along the weld line for twenty connections with the same a-distance (a = 10.5”) and five bolts, respectively. Note that the vertical displacement is obtained at the ultimate connection shear capacity.

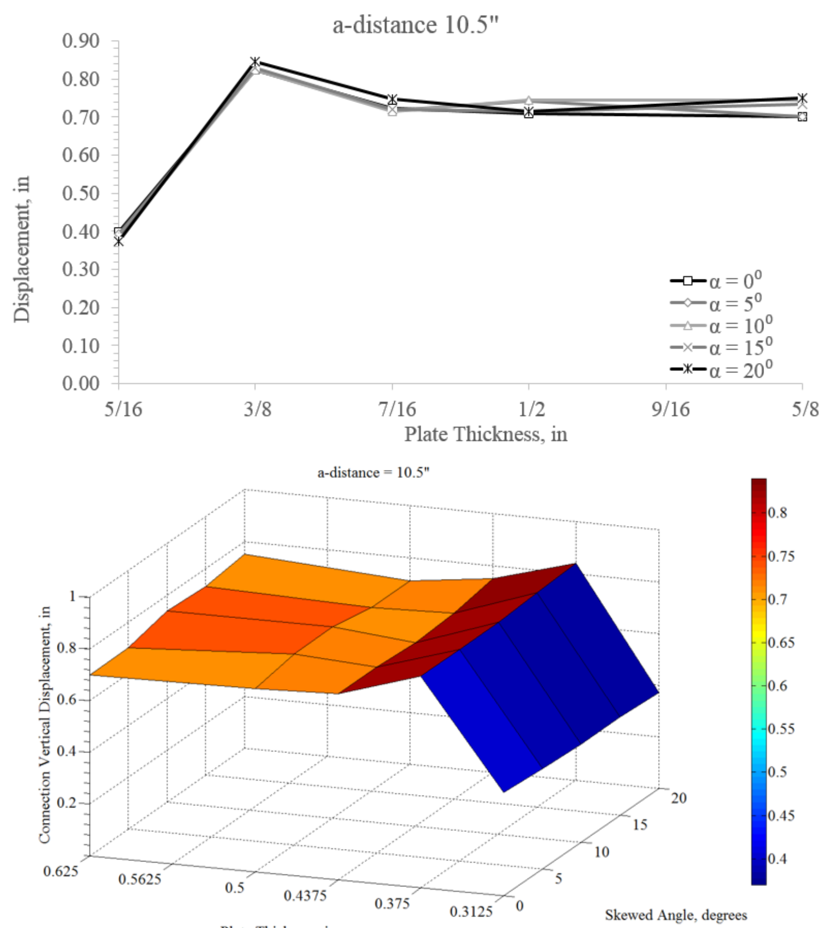


Figure 6-22. Connection vertical displacement-plate thickness curves (a-distance = 10.5”)

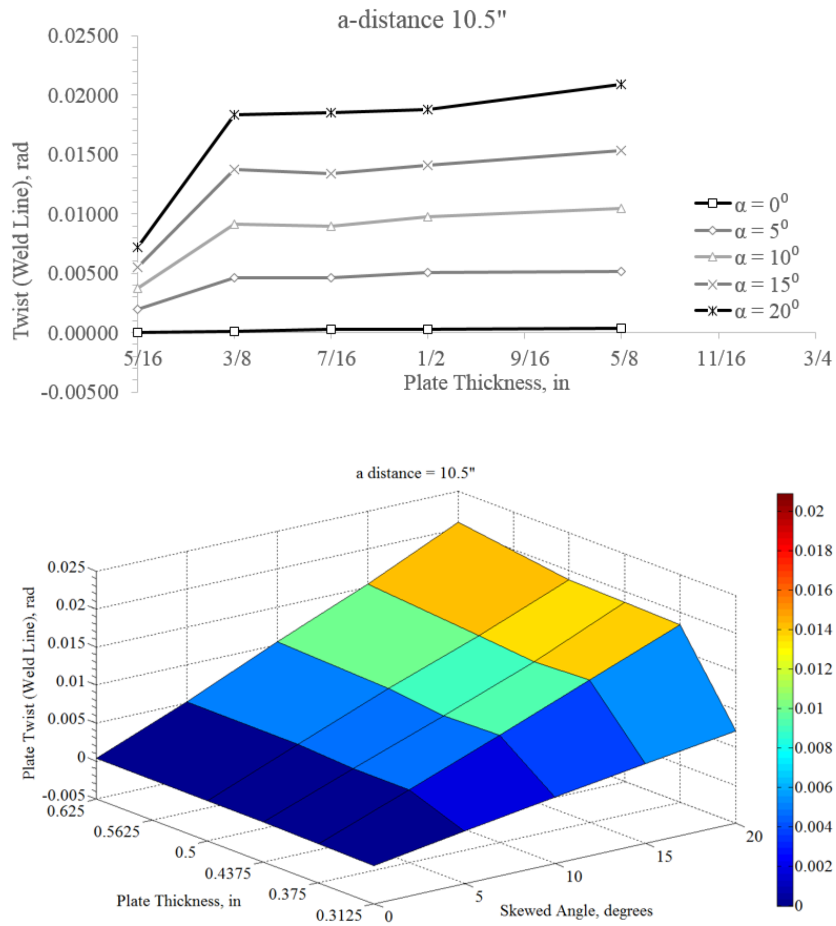
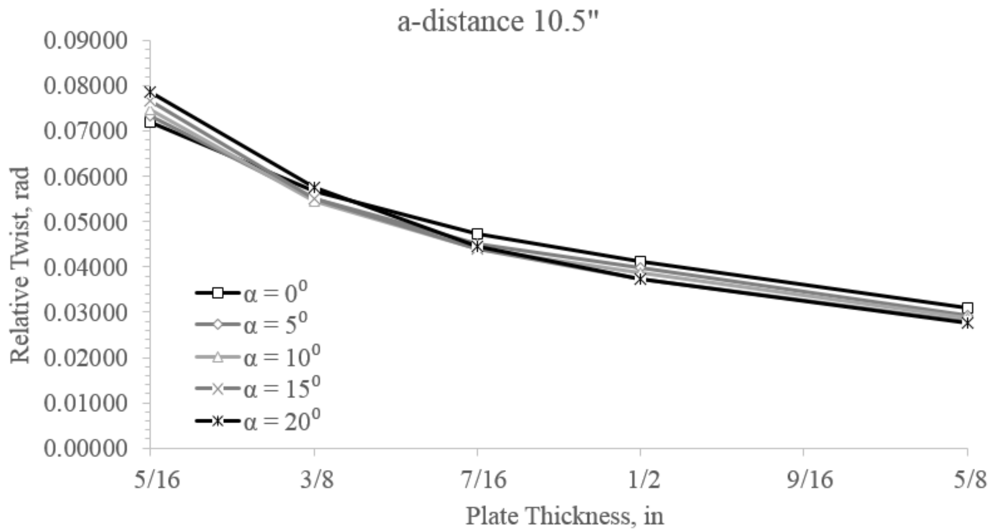


Figure 6-23. Plate twist along the weld line-plate thickness curves (a-distance = 10.5")

It is observed from these figures that for a certain skewed angle, the connection vertical displacement and the plate twist along the weld line dropped significantly when the plate thickness was increased from 5/16" to 3/8". Then, these parameters were slightly changed when using plate thicker than 3/8". This behavior is a result of the change in the connection rigidity and failure modes as was explained earlier in the previous paragraph. On the other hand, it was observed that the relative twist decreases significantly with the increase of the plate thickness for a one skewed angle due to the increase of the plate torsional resistance (Figure 6-24).

As a conclusion, the plate thickness and connection orientation affect significantly the torsional behavior of deep skewed extended shear tab connections.



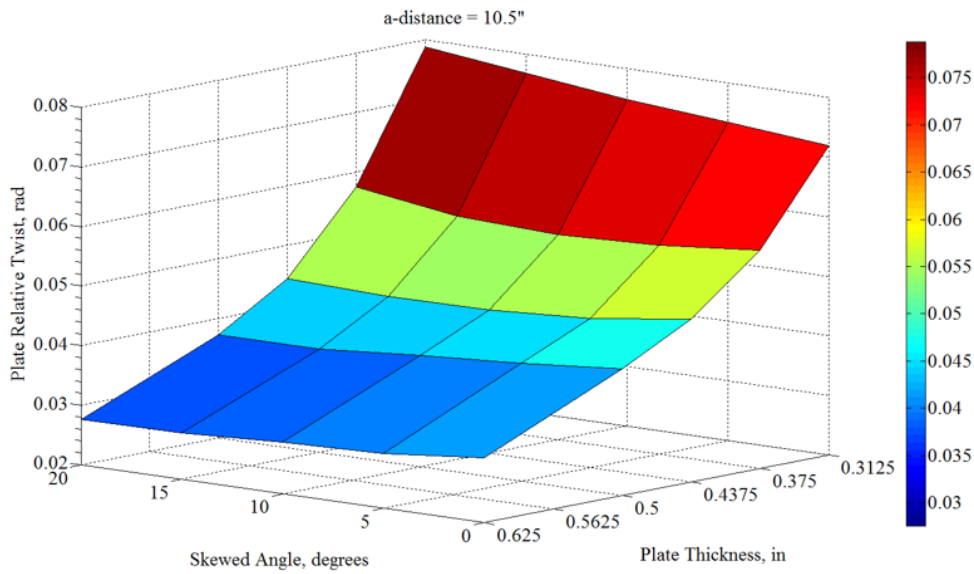


Figure 6-24. Relative twist-plate thickness curves (a-distance = 10.5”)

6.3. Eight-Bolted Connection

Figure 6-25 shows the constant parameters for the eight bolted connection. In this group, 100 models were created with five plate thicknesses (5/16”, 3/8”, 7/16”, 1/2”, 5/8”), four a-distances (10.5”, 11”, 11.5”, 12”), and five skewed angles (0^0, 5^0, 10^0, 15^0, 20^0). In this connection, the beam was braced against lateral movement to check the effect on using a slab in top of the supported beam.

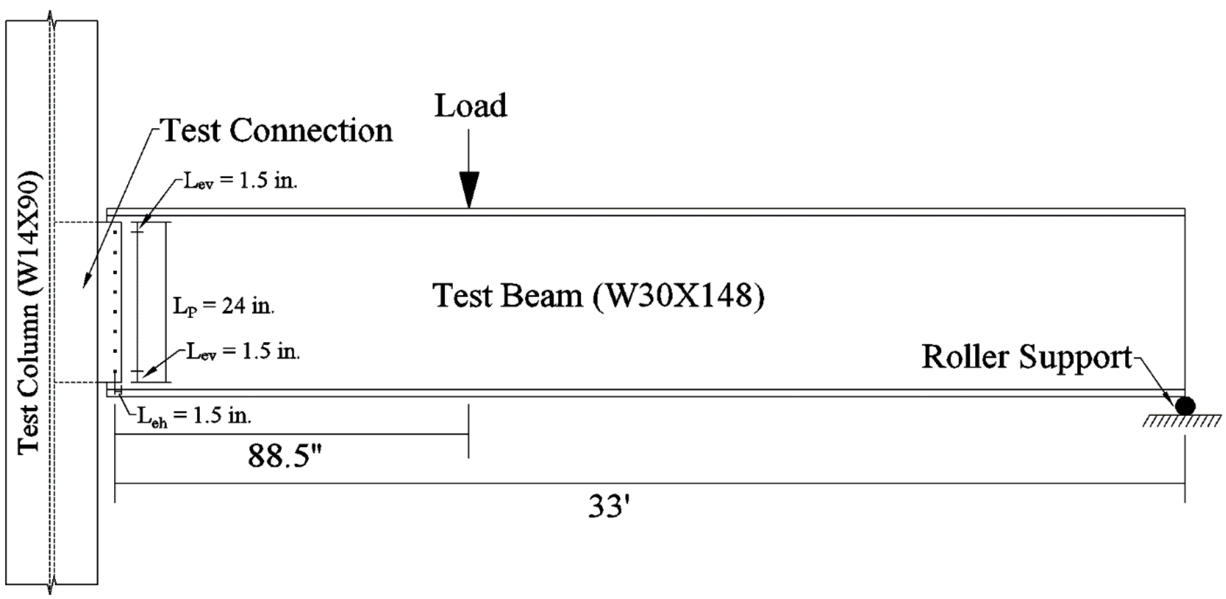


Figure 6-25. Constant parameters for the eight bolted connection

Adding the lateral braces along the beam affect significantly the torsional behavior of the beam and connection, especially the plate twist caused by the eccentricity due to the overlap between the beam web and plate longitudinal axes. Figure 6-26 shows the effect of the skewed angle and a-distance on the plate relative twist for connection with 5/16” plate thick. As was defined earlier, the relative twist represents the plate twist caused by the eccentricity due to the overlap between the plate and beam web longitudinal axes, also it is the difference between the plate twist along bolt line and weld line. For orthogonal configuration (α = 0), the relative twist equals to zero for all connections with different a-distance values. This proves that the addition of the lateral bracing was effective in eliminating the plate twist due to the eccentricity. However, for skewed connections (α = 5, 10, 15, 20), the relative twist have negative values and represent the plate twist along the weld line due to the connection orientation.

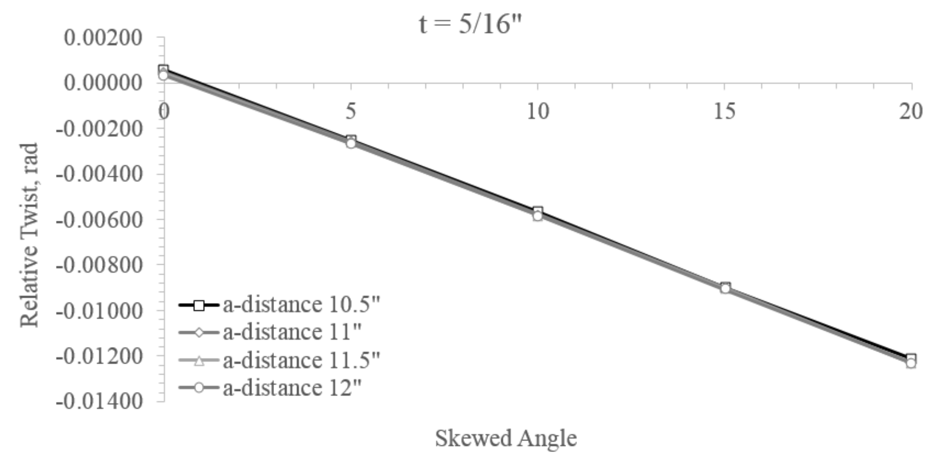


Figure 6-26. Relative twist-skewed angle curves (t = 5/16”)

Figure 6-27 shows the effect of the skewed angle and a-distance on the plate twist along the weld line. Similar to the three- and five-bolted connections, the plate twist along the weld line increases with increase of the connection orientation linearly. Also it was observed from this figure that the a-distance has no effect on the connection torsional behavior.

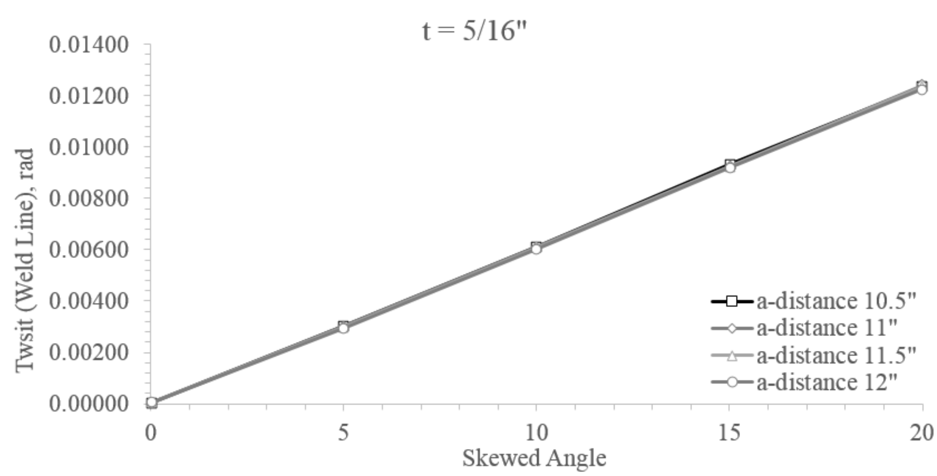


Figure 6-27. Plate twist along the weld line-skewed angle curves (t = 5/16”)

It was observed from Figures 6-28 and 6-29 that the skewed angle has an insignificant effect on both, the connection vertical displacement and connection shear capacity. Also, as the a-distance increases, the connection shear capacity and connection vertical displacement slightly decrease. The same behavior was shown for connections with thicker shear tabs. The figures for these connections are shown in Appendix B.

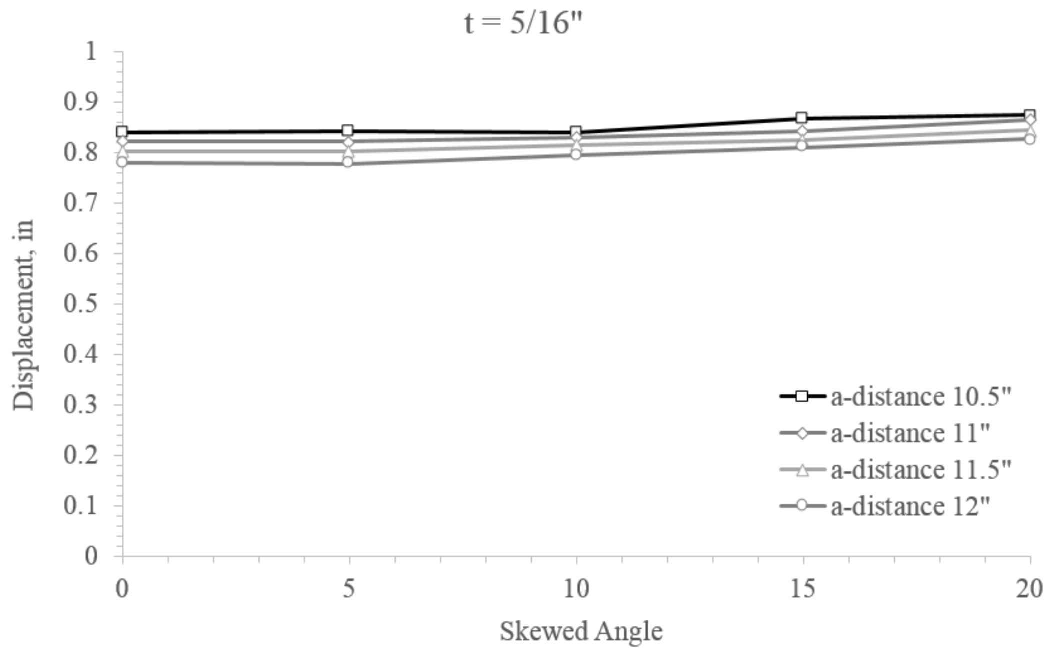
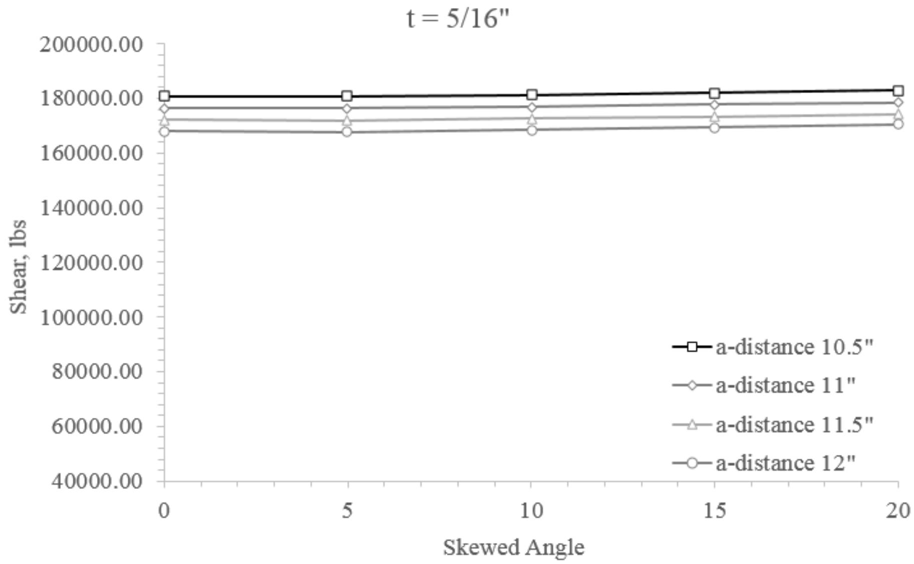


Figure 6-28. Connection vertical displacement-skewed angle curves (t = 5/16”)



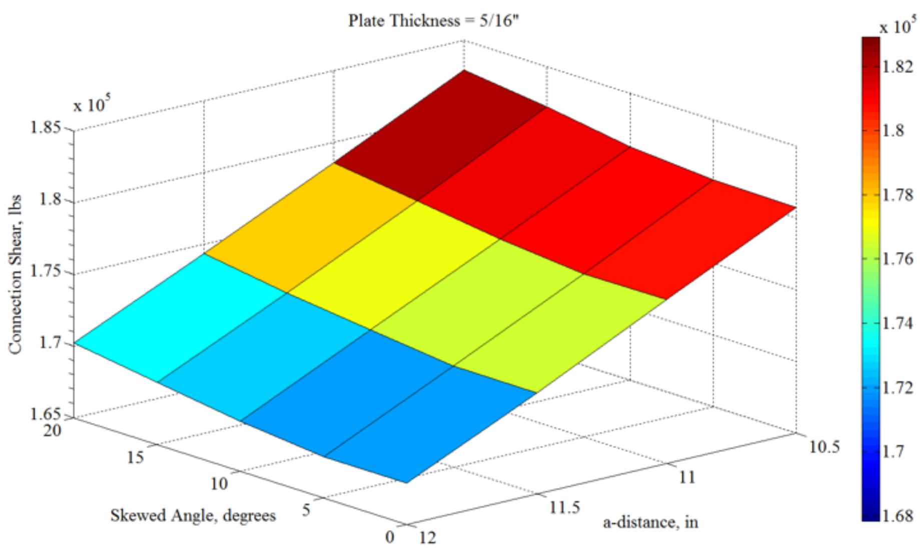
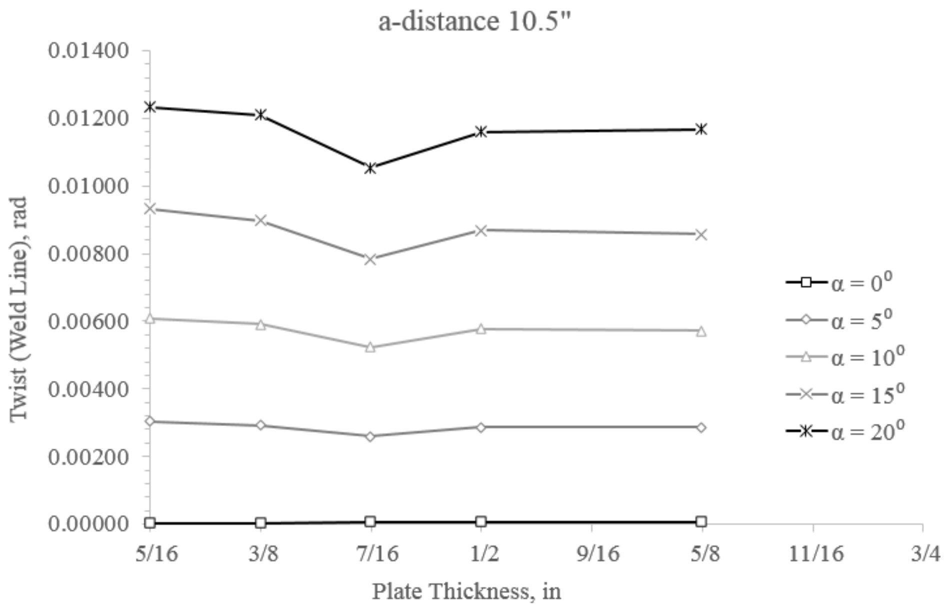


Figure 6-29. Connection shear capacity-skewed angle curves (t = 5/16")

Additionally, it was observed that the plate thickness slightly affects the connection torsional behavior (Figures 6-30 and 6-31) and the connection shear capacity (Figure 6-32).



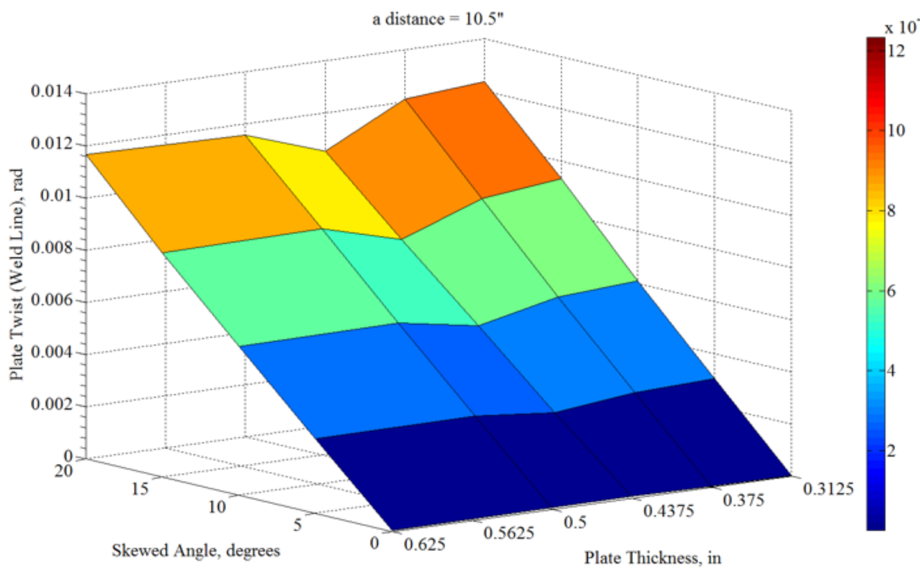


Figure 6-30. Plate twist along the weld line-plate thickness curves (a-distance = 10.5")

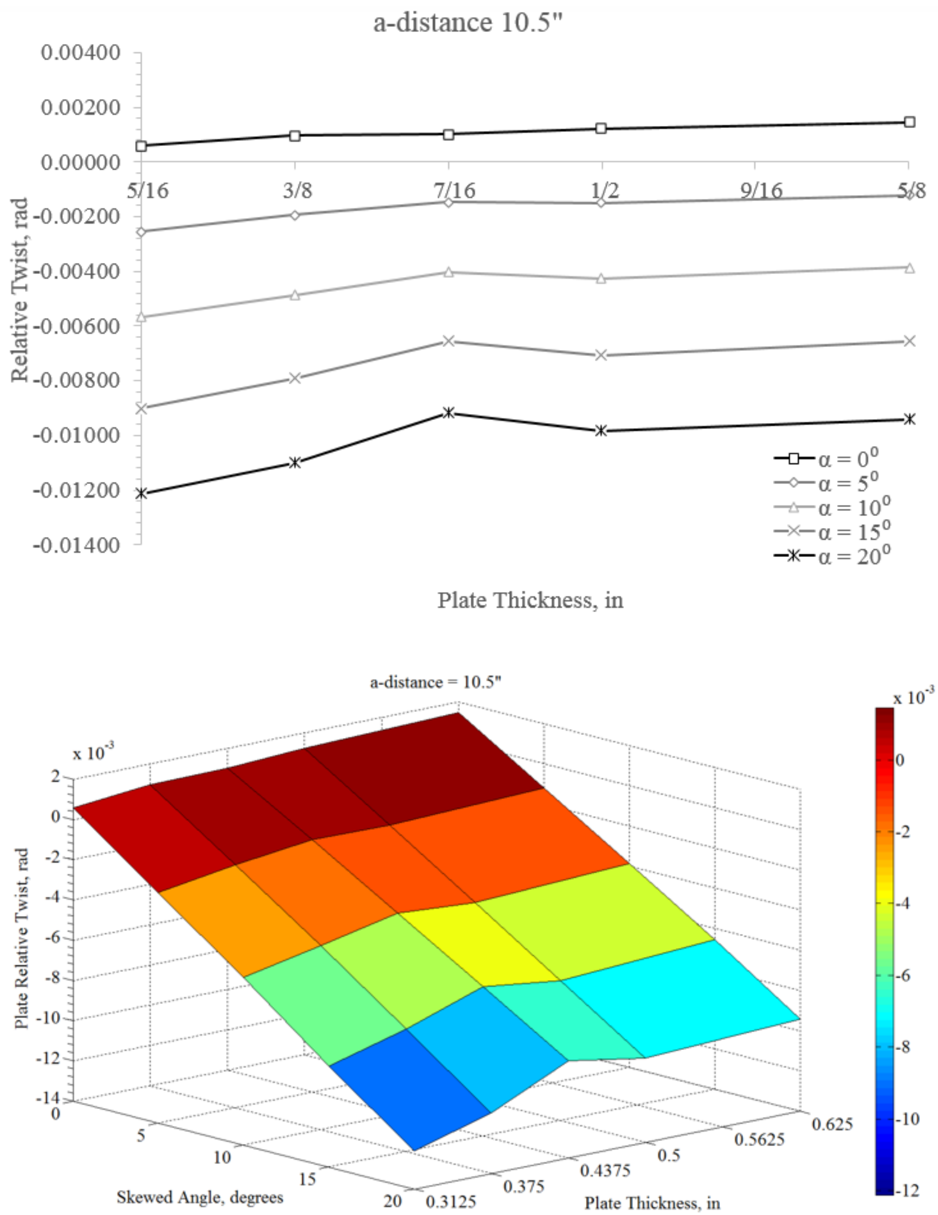


Figure 6-31. Plate relative twist-plate thickness curves (a-distance = 10.5")

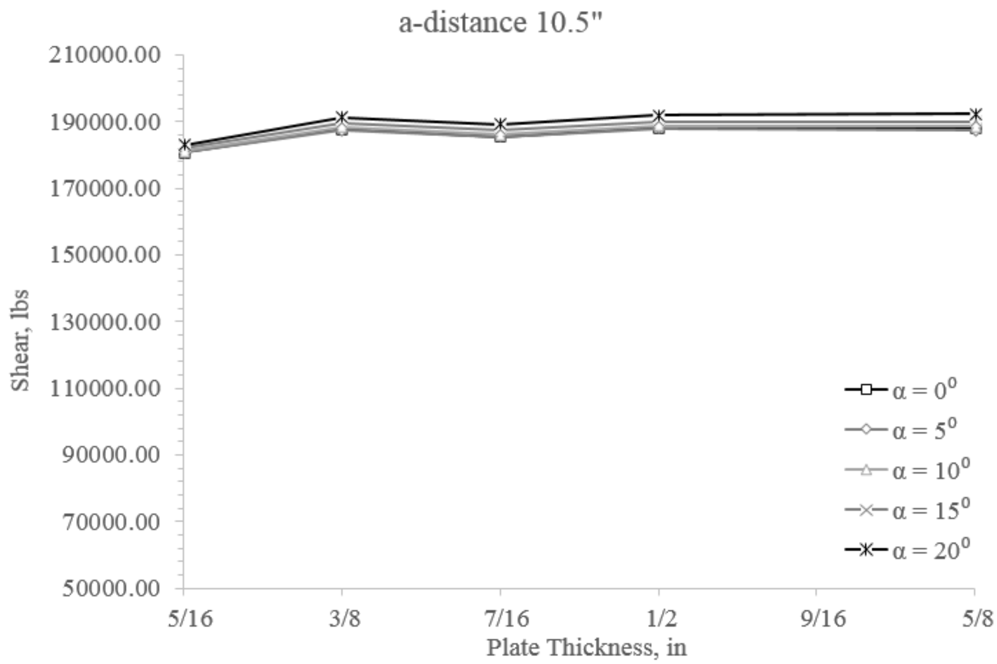


Figure 6-32. Connection shear-plate thickness curves (a-distance = 10.5”)

Figures 6-33 and 6-34 show that as the plate thickness increases, the connection vertical displacement decreases. The reduction is significant when the plate thickness was increased from 5/16” to 3/8” due to the change of the primary failure modes. Figure 6-35 and 6-36 show failure modes for connections with 5/16” plate thick and connections with 3/8” plate thick, respectively. For connections with 5/16” plate thickness, the primary failure modes are bolt shear and bolt bearing, and the secondary failure mode is web mechanism. For connections with thicker plates (3/8”, 7/16”, 1/2”, 5/8”) the plate bending stiffness increases, and the bolt bearing becomes a secondary failure mode.

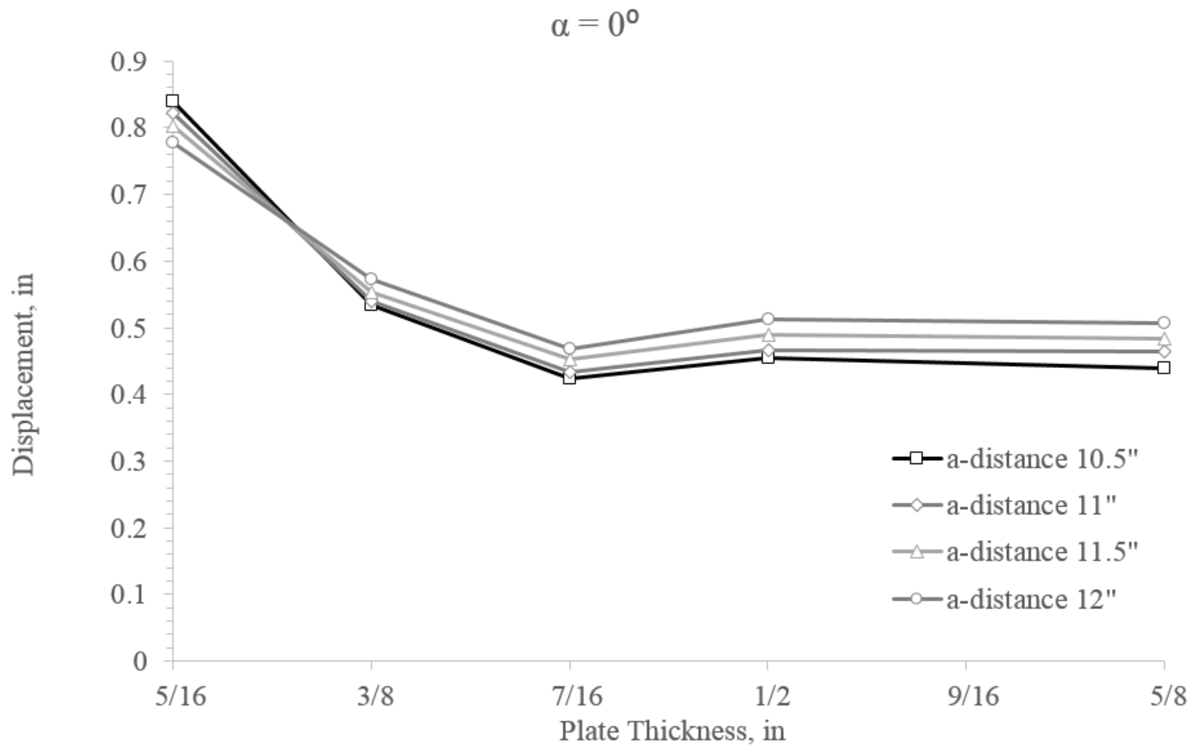


Figure 6-33. Connection shear-plate thickness curves ($\alpha = 0^0$) for 8 bolted connection

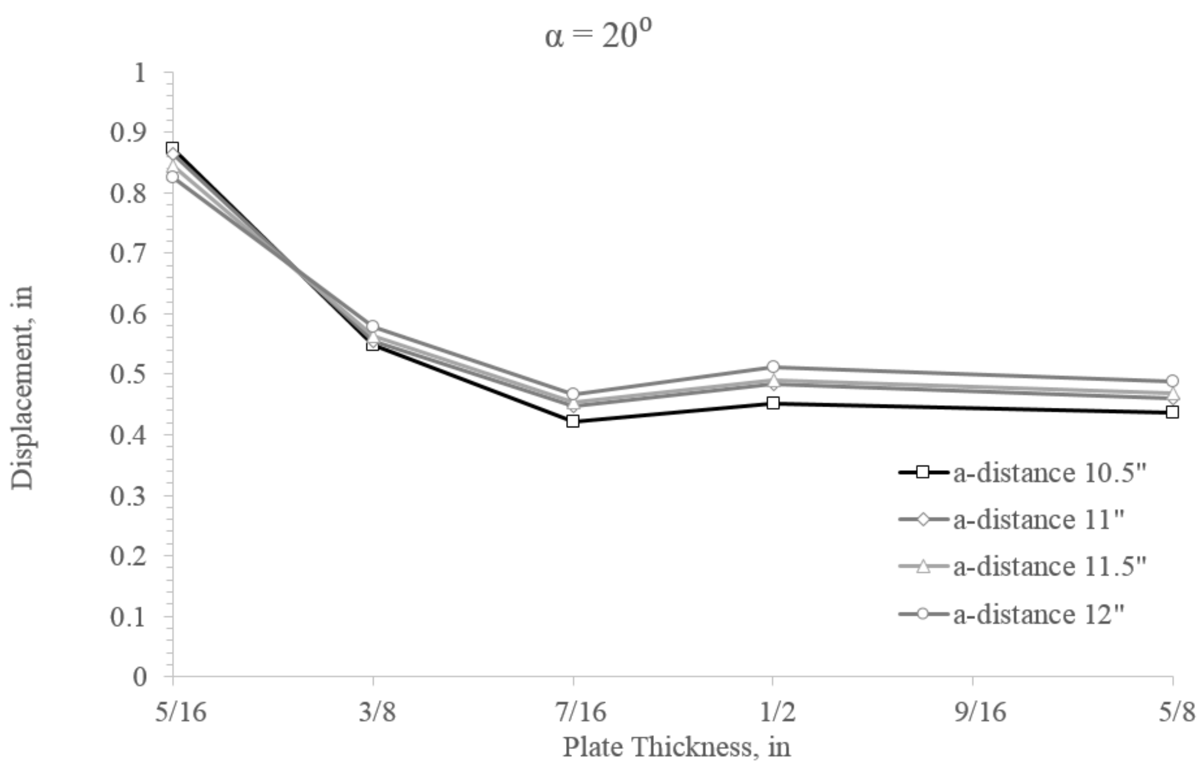


Figure 6-34. Connection shear-plate thickness curves ($\alpha = 20^o$) for 8 bolted connection

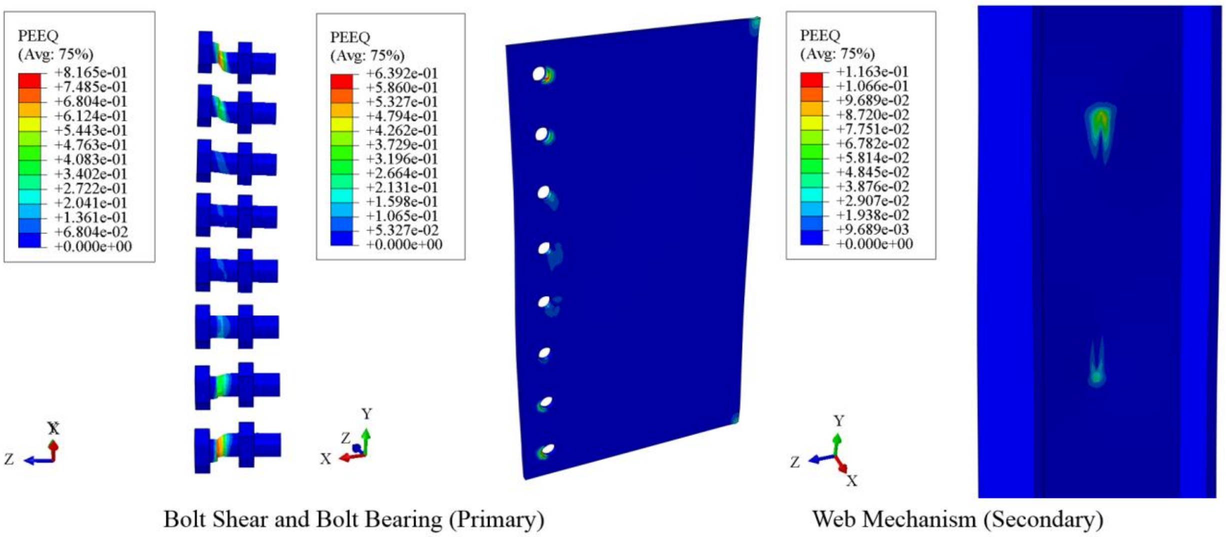


Figure 6-35. Failure modes of connection 8B-5/16-12-20

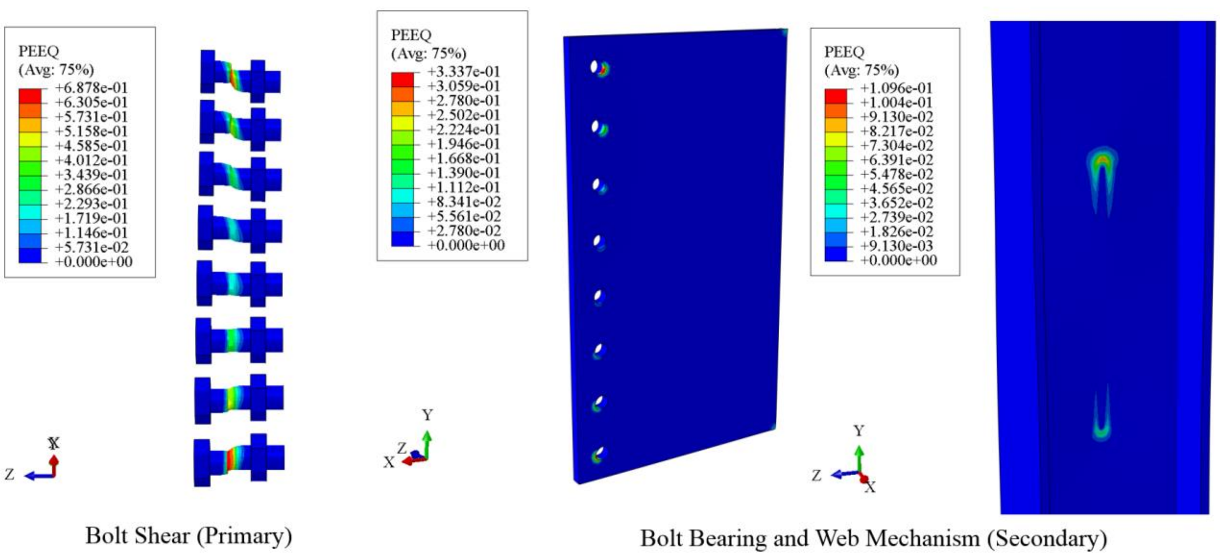


Figure 6-36. Failure modes of connection 8B-5/8-12-20

7. CHAPTER SEVEN - DERIVATIONS AND MODIFICATIONS TO CURRENT DESIGN PROCEDURE

7.1. Derivations

In this section, equations of the skewed and orthogonal extended shear tab connections as a function of different parameters will be derived or modified and represented in tables. The proposed equations can be used to predict the connection's shear capacity, connection's vertical displacement, plate twist caused by the eccentricity due to the overlap between the plate and beam web longitudinal axes, and plate twist due to the connection orientation. These equations will be a function of the connection orientation, plate thickness, a-distance, and bolt numbers.

Figure 7-1 shows the generalized proposed equation that can be used to determine the connection behavior based on the plate thickness, a-distance, number of bolts, and connection orientation. The connection behavior can be found using the following equations:

$$V = a_V \alpha + b_V \tag{1}$$

$$D = a_D \alpha + b_D \tag{2}$$

$$T_w = a_{Tw} \alpha + b_{Tw} \tag{3}$$

$$T_R = a_{TR} \alpha + b_{TR} \tag{4}$$

Where V is the connection shear, D is the connection vertical displacement, T_w is plate twist along weld line, T_R is the relative plate twist, α is the skewed angle in degrees (ranges between 0^0 and 20^0), and a_V , b_V , a_D , b_D , a_{Tw} , b_{Tw} , a_{TR} , b_{TR} are constants depend on the plate thickness, a-distance, and number of bolts.

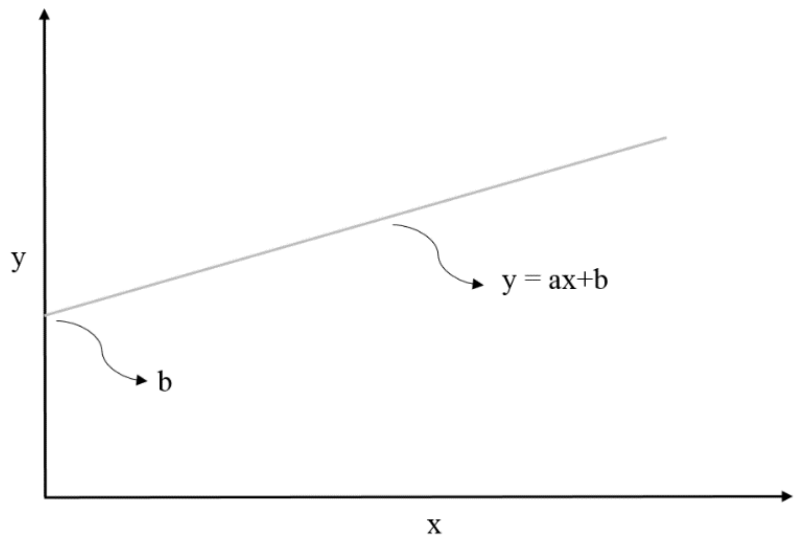


Figure 7-1. Proposed Equation

These values are shown in tables 7-1, 7-2, and 7-3 for three-bolted, five-bolted, and eight-bolted connections, respectively. The maximum error of using the proposed relations to find the connection vertical displacement, plate twist along the weld line, plate relative twist, and connection shear capacity is (6.22%), (7.65%), (5.82%), and (1.48%), respectively. However, the plate twist along the weld line at zero and five degrees for some of the connections were very small, these values were excluded. Also, the plate relative twist values for eight bolted connections were excluded since these values are close to zero due to the lateral bracing applied along the supported beam. Tables that show the error of using the proposed relations for each configuration can be found in Appendix B.

Table 7-1. Values a_V, b_V, a_D, b_D, a_{Tw}, b_{Tw}, a_{TR}, b_{TR} constants for three-bolted connections

t, in	a-distance, in	a_D	a_{Tw}	a_{TR}	a_V	b_D	b_{Tw}	b_{TR}	b_V
1/4	**7**	-0.00071	0.00138	0.00018	20.558	1.519	0	0.05	55063
	7.5	-0.0153	0.00145	0.00011	-39.852	1.741	0	0.046	53990
	8	-0.01677	0.00144	-0.00001	-41.732	1.731	0.001	0.039	51589
	8.5	-0.01162	0.00147	-0.00001	-17.868	1.679	0.001	0.035	48851
5/16	**7**	-0.0005	0.00161	-0.00081	68.258	1.052	0	0.04	55814
	7.5	0.00036	0.00166	-0.00083	72.49	1.134	0	0.037	53007
	8	0.00024	0.00166	-0.00083	80.822	1.205	0	0.035	50124
	8.5	0	0.00166	-0.00085	74.996	1.278	0	0.033	47665
3/8	**7**	-0.00527	0.00151	-0.00077	-0.364	0.994	0.001	0.037	57237
	7.5	-0.0003	0.00173	-0.0008	72.338	1.112	0	0.036	54547
	8	0.00022	0.00175	-0.00081	75.366	1.189	0	0.034	51807
	8.5	0.00046	0.00175	-0.00081	74.706	1.249	0	0.032	49163
1/2	**7**	0.00012	0.00176	-0.00055	67.656	0.941	0.001	0.029	58257
	7.5	0.00056	0.00174	-0.00054	78.616	0.971	0.001	0.027	54698
	8	0.00007	0.00175	-0.00056	71.732	1.053	0.001	0.027	52103
	8.5	0.00052	0.00177	-0.00059	79.968	1.115	0	0.026	49569
5/8	**7**	0.00034	0.00181	-0.00037	54.852	0.913	0.001	0.021	59244
	7.5	-0.00004	0.00189	-0.0004	63.654	1.038	0.001	0.022	56910
	8	0.00118	0.00183	-0.00041	77.8	1.041	0.001	0.021	53213
	8.5	0.002	0.002	-0.00047	86.895	1.178	0	0.022	51463

Table 7-2. Values a_V, b_V, a_D, b_D, a_{Tw}, b_{Tw}, a_{TR}, b_{TR} constants for five-bolted connections

t, in	*a-distance, in*	a_D	a_{Tw}	a_{TR}	a_V	b_D	b_{Tw}	b_{TR}	b_V
5/16	**10.5**	-0.001	0.00036	0.00034	-26.235	0.398	0	0.072	73562
	11	-0.002	0.00036	0.00032	-34.457	0.406	0	0.068	71525
	11.5	-0.002	0.00037	0.00034	-31.963	0.428	0	0.064	69899
	12	-0.003	0.00038	0.00036	-59.624	0.452	0	0.06	68294
3/8	**10.5**	0.001	0.00091	0.00004	119.337	0.819	0	0.055	88725
	11	0.001	0.00091	-0.00003	118.13	0.843	0	0.052	85498
	11.5	0.001	0.00092	-0.00004	121.424	0.865	0	0.05	82442
	12	0	0.00092	-0.00006	114.673	0.902	0	0.047	79642
7/16	**10.5**	0.001	0.00091	-0.00014	114.813	0.713	0	0.046	88582
	11	0.004	0.00091	-0.0001	159.9	0.683	0	0.044	84331
	11.5	0	0.00083	-0.00015	98.58	0.724	0	0.042	81492
	12	0.001	0.00084	-0.00014	109.02	0.749	0	0.041	78553
1/2	**10.5**	0	0.00092	-0.00021	86.641	0.728	0	0.041	89988
	11	-0.002	0.00091	-0.00024	57.304	0.777	0	0.04	86955
	11.5	0	0.00095	-0.0002	100.623	0.789	0	0.039	83506
	12	0.001	0.00098	-0.0002	108.465	0.833	0	0.038	80731
5/8	**10.5**	0.003	0.00102	-0.00016	135.02	0.699	0	0.03	90534
	11	0.001	0.00098	-0.00016	113.989	0.728	0	0.03	87193
	11.5	0.001	0.00098	-0.00015	122.827	0.759	0	0.029	83985
	12	0	0.00096	-0.00016	96.199	0.798	0	0.029	81179

Table 7-3. Values a_V, b_V, a_D, b_D, a_{Tw}, b_{Tw}, a_{TR}, b_{TR} constants for eight-bolted connections

t, in	*a-distance, in*	a_D	a_{Tw}	a_{TR}	a_V	b_D	b_{Tw}	b_{TR}	b_V
5/16	**10.5**	0.002	0.00062	-0.00064	114.322	0.834	0	0.001	180445
	11	0.002	0.00062	-0.00064	112.169	0.814	0	0.001	176095
	11.5	0.002	0.00062	-0.00064	120.018	0.797	0	0	171724
	12	0.003	0.00061	-0.00063	124.598	0.772	0	0	167490
3/8	**10.5**	0.001	0.0006	-0.0006	187.22	0.531	0	0.001	186995
	11	0.001	0.0006	-0.0006	202.621	0.538	0	0.001	181610
	11.5	0.001	0.0006	-0.0006	198.457	0.552	0	0.001	176544
	12	0	0.0006	-0.0006	191.578	0.57	0	0.001	171633
7/16	**10.5**	0	0.00052	-0.00051	192.85	0.424	0	0.001	184767
	11	0.001	0.00053	-0.00052	240.916	0.431	0	0.001	178590
	11.5	0	0.00052	-0.00051	205.944	0.452	0	0.001	173601
	12	0	0.00052	-0.00051	206.002	0.468	0	0.001	168401
1/2	**10.5**	0	0.00058	-0.00055	184.089	0.455	0	0.001	187512
	11	0.001	0.0006	-0.00057	251.947	0.465	0	0.001	181600
	11.5	0	0.00058	-0.00056	195.769	0.49	0	0.001	176678
	12	0	0.00058	-0.00056	189.641	0.514	0	0.001	171823
5/8	**10.5**	0	0.00058	-0.00054	225.874	0.435	0	0.001	186953
	11	0	0.00059	-0.00055	226.336	0.465	0	0.001	182282
	11.5	-0.001	0.00058	-0.00055	205.475	0.481	0	0.001	176718
	12	-0.001	0.00058	-0.00055	199.473	0.511	0	0.001	172232

7.2. Modifications to the Current Design Procedure

The design of orthogonal single-plate connections can be used to design skewed single plate connections when the skewed angle is between 5 and 30 degrees (AISC, 2010). This study shows that the shear capacities and failure modes are similar for orthogonal and skewed extended shear tab connections with the same parameters. However, it was shown that the connection behavior is highly affected by the connection orientation in the case where the plate is welded to the web of the supporting column. Also, additional moment components are developed along the weld line between the plate and the column web due the connection orientation. In this section, modifications to the current design procedure for the extended configuration of the single plate connections will be discussed. First, the effect of the connection orientation on the weld group will be checked.

Then, the current checks in the AISC Manual (AISC, 2010) for determining the need of using stabilizer plates will be examined and modified to account for the connection orientation. Finally, the effect of the connection orientation on the column web will be investigated.

7.2.1. Strength of the weld group

The capacity of the weld need to be checked against the applied moment components along the weld line. The fillet weld size used on both sides of the plate to connect the plate with the supporting member web is equal to 5/8 the plate thickness according to (chapter 10 in the AISC manual (AISC, 2010)) to ensure that the weld will not fail prior to the failure of the connection. For skewed extended shear tab connections, the shear force and moment components (($R \times a \times \sin\alpha$) and ($R \times a \times \cos\alpha$)) shown in Figure 5-6 in Chapter 5 are applied along the weld line need to be checked.

The available strength of the eccentrically loaded weld group is determined according to part 8 in the AISC manual 14th edition (AISC, 2010) which is based on the instantaneous center of rotation method (Butler et al., 1972). This method considers the rotation and translation of one connection element with respect to the other is equivalent to a rotation about a point (instantaneous center of rotation) as shown in Figure 7-2; the location of this point depends on the direction and location of the applied load and the geometry of the weld group (AISC, 2010).

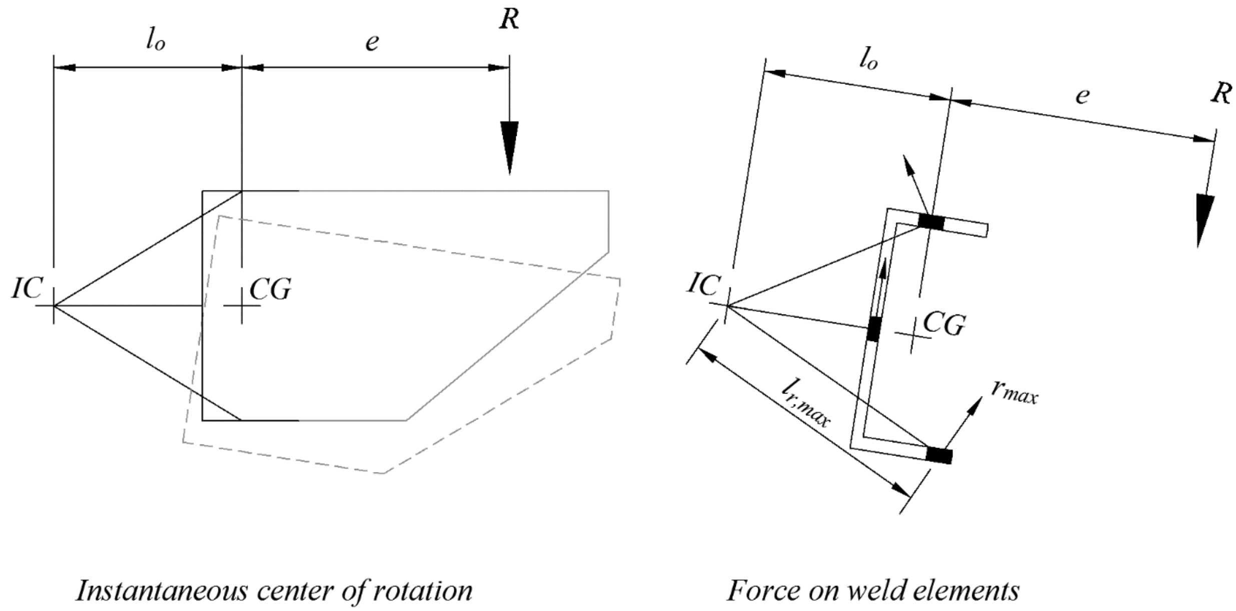


Figure 7-2. Instantaneous center of rotation method (AISC Manual 14th Edition, 2010)

According to this method, the nominal shear strength of the weld group can be calculated as follows:

$$F_{nw} = \sum_i F_{nwi} = \sum_i \left(0.6F_{EXX}\,(1.0 + 0.5\sin^{1.5}\theta_i)\left[p_i(1.9 - 0.9p_i)\right]^{0.3}\right) \tag{5}$$

Where:

F_{nwi} = nominal shear strength of the weld segment at a deformation Δ, ksi

F_{EXX} = weld electrode strength, ksi

θ_i = load angle measured relative to the weld longitudinal axis, degrees

p_i = ratio of element deformation to its deformation at the maximum stress

$\Delta_i = r_i \dfrac{\Delta_u}{r_{crit}}$ = deformation of element i, in

r_{crit} = distance from instantaneous center of rotation to weld element having minimum ratio

$$\frac{\Delta_u}{r_i}$$

$\Delta_m = 0.209(\theta+2)^{-0.32}a$ = deformation of element at maximum strength, in.

$\Delta_u = 1.087(\theta+6)^{-0.65}a \leq 0.17a$ = deformation of element when fracture is imminent, usually in element farthest from instantaneous center of rotation, in.

a = leg size of fillet weld, in.

The additional moment component ($R \times a \times \sin\alpha$) represents the case where the eccentricity is in the plane of the weld group. The weld for this case must be designed to resist the combined effect of the direct shear (R), and the additional shear from the moment component ($R \times a \times \sin\alpha$). On the other hand. The moment component ($R \times a \times \cos\alpha$) represent the case where the eccentricity is normal to the plane of the weld group. The weld for this case must be designed to resist the combined effect of the direct shear, and the additional moment ($R \times a \times \cos\alpha$) that is resisted by tension in welds above the neutral axis and compression below the neutral axis.

The use of the instantaneous center of rotation method is accurate. However, using Equation 5 is tedious since it is an iterative solution. Table 8-4 in the AISC Manual (ASIC, 2010) provides a tabulated nondimensional coefficient (C), this coefficient represents the effective strength of the weld group in resisting the eccentric shear force. Figure 7-3 shows all the parameters needed to find this coefficient for in-plane and out-of-plane loading cases. This coefficient can be found for the moment component ($R \times a \times \sin\alpha$) by considering the eccentricity $(a \times \sin\alpha)$ is in plane of weld group with $P = R$, $e_x = a \times \sin\alpha$, and $k = t_p / l$. It also can be found for the moment component ($R \times a \times \cos\alpha$) by considering the eccentricity $(a \times \cos\alpha)$ is normal to the plane of weld group with $P = R$, $e_x = a \times \cos\alpha$, and $k = 0$. The nominal strength of a weld group is shown in Equation 6. Equations 7 and 8 show the minimum weld size that can be used to resist the moment components ($R \times a \times \sin\alpha$) and ($R \times a \times \cos\alpha$), respectively.

$$R_n = CC_1 Dl \quad (6)$$

$$D_{1,\min} = \frac{R}{C_I C_1 l} \quad (7)$$

$$D_{2,\min} = \frac{R}{C_O C_1 l}$$

(8)

In specification notation:

$$D_{1,\min} = \frac{R_u}{C_I C_1 l} \quad \textit{(LRFD)} \quad (9a\text{-}a)$$

$$D_{1,\min} = \frac{R_a}{C_I C_1 l} \quad \textit{(ASD)} \quad (9a\text{-}b)$$

$$D_{2,\min} = \frac{R_u}{C_O C_1 l} \quad \textit{(LRFD)} \quad (9b\text{-}a)$$

$$D_{2,\min} = \frac{R_a}{C_O C_1 l} \quad \textit{(ASD)} \quad (9b\text{-}b)$$

Where:

R = Required connection shear, R_a or R_a, kips

l = Plate length, in.

C_1 = Electrode strength coefficient from (AISC, 2010) Table 8-3, (1.0 for E70XX electrodes)

C_I = Non-dimensional coefficient associated with the moment component ($R \times a \times \sin\alpha$) tabulated in (AISC, 2010) Table 8-4

C_O = Non-dimensional coefficient associated with the moment component ($R \times a \times \cos\alpha$) tabulated in (AISC, 2010) Table 8-4

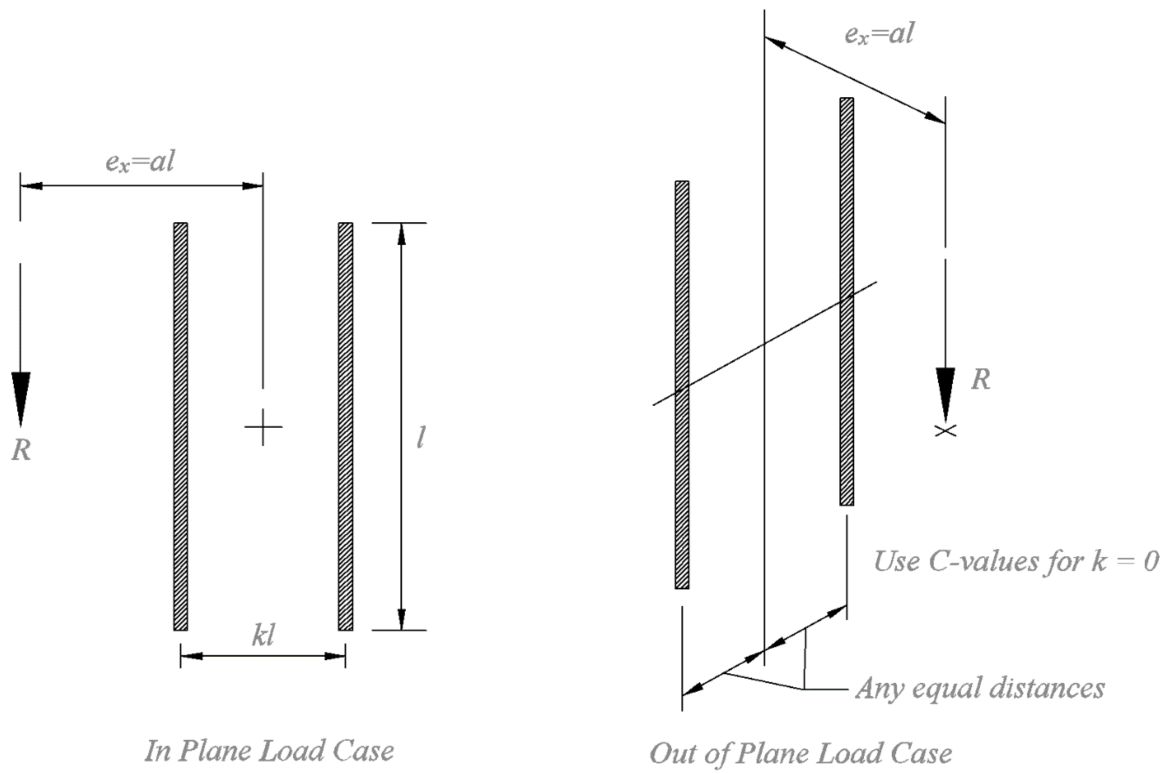


Figure 7-3. Load cases applied on the weld group (AISC, 2010)

7.2.2. Requirement for stabilizer plates

According to the AISC Manual (AISC, 2010), stabilizer plates need to be used if the shear strength of the plate is less than the applied connection shear (Equation 10). This equation applies for skewed connections since the connection shear strength is independent from the connection orientation.

$$R \leq R_n = 1500\,\pi \frac{Lt_p{}^3}{a^2} \tag{10}$$

Where:

a = distance between the weld line and bolt line, in.

t_p = thickness of plate, in.

L = depth of plate, in.

R = connection shear, kips

Equations (10-7a) and (10-7b) in the AISC Manual (AISC, 2010) are used to determine the need for stabilizer plates based on the torsional strength of the connection. These equations were derived by Thornton and Fortney (2011) based on a study done by Cheng et al. (1984) on coped beams behavior. The applied torsional moment on the connection is resisted by the plate and the supported beam due to the floor slab (Equation 11). The first term of the right hand side of the equation represents the lateral shear strength of the shear tab ($M_{t,tab}$), the second term represents the lateral bending resistance of the beam due to the floor slab ($M_{t,beam}$).

$$M_t \leq M_{t,tab} + M_{t,beam} = \left(0.6F_{yp} - \frac{R}{lt_p}\right)\frac{lt_p{}^2}{2} + \frac{2R^2(t_w + t_p)b_f}{F_{yb}Lt_w{}^2} \tag{11}$$

Where:

F_{yp} = specified minimum yield stress of the plate, ksi

F_{yb} = specified minimum yield stress of beam web, ksi

$M_t = R \times \left(\frac{t_w + t_p}{2}\right)$

t_p = thickness of plate, in.

l = depth of plate, in.

L = span length of beam, in.

R = shear on the connection, kips

b_f = width of beam flange, in.

t_w = thickness of beam web, in.

As shown in the previous sections, there is moment applied on the connection along the weld line due to the connection orientation. The connection torsional stability equations in the AISC manual 14th edition (AISC, 2010) check the capacity of the connection to resist the torsional moment applied on the connection due to the overlap between the plate and beam web longitudinal axes. These equations are derived based on the assumption that the plate end (the region where the plate and column web are connected) is fixed. When the connection is skewed at a certain angle, the connection shear will generate moment components ($R \times a \times \sin\alpha$ and $R \times a \times \cos\alpha$) at the end of the plate along the weld line. These moment components will be resisted by the plate, weld and the supporting column web. The capacity of the weld was discussed previously.

M_t on the left hand side of Equation 11 is defined as the torsional moment due to the overlap between the beam and shear tab longitudinal axes (Thornton and Fortney, 2011). However, there is torsional moment component due to the connection orientation as explained before. Thus, M_t becomes:

$$M_t = R \times \left(\frac{t_w + t_p}{2} \right) + R \times a \times \sin \alpha \tag{12}$$

In specification notation,

$$M_{tu} = R_u \times \left[\left(\frac{t_w + t_p}{2} \right) + a \times \sin \alpha \right] \qquad \text{(LRFD)} \tag{13a}$$

$$M_{ta} = R_a \times \left[\left(\frac{t_w + t_p}{2} \right) + a \times \sin \alpha \right] \qquad \text{(ASD)} \tag{13b}$$

Where:

$\phi_v = 1.00$

$\phi_b = 0.90$

$\Omega_v = 1.50$

$\Omega_b = 1.67$

Equations (13a) and (13b) are proposed to check the need of stabilizer plates for the extended shear tab connections, the orthogonal and skewed configurations. Note that these equations for orthogonal connections will be the same as equations (10-8a) and (10-8b) in the AISC manual since the skewed angle is zero.

7.2.3. Strength of the column web

The connection shear produces moment components about the column web, the first moment component ($R \times a \times \sin\alpha$) is applied along the strong axis of the column web and has an insignificant effect on the column web as was shown in the finite element analysis. On the other hand, the second moment component ($R \times a \times \cos\alpha$) overstresses the column web since the column web is relatively thin. The later component does not affect the torsional stability of the connection with orthogonal configuration as was shown in the finite element results for the plate twist along the weld line. However, it affects significantly the torsional stability of the connection for the skewed configuration. Abolitz and Warner (1965) investigated the supporting column web bending under brackets (the shear tab in our case) using the yield line method. This method is used to determine the ultimate loads for two-dimensional members based on the equilibrium between the internal energy (amount of work which is dissipated by the column web under a special failure mode) and external energy (single bracket moment). Equations 14 and 15 were derived by Abolitz and Warner where they show the coefficient (*k*) expression for the case where the plate is welded to the column web, and the ultimate bracket moment (*X*) to which the plate may be subjected, respectively. Equation 14 is used based on the assumption that the ends of the column web are fixed since the column flanges are much stiffer than the column web. All the parameters involved in these equations are shown in Figure 7-4. Note that if (*X*) is greater than or equal the moment component ($R \times a \times \cos\alpha$), the column web will not be overstressed. If the capacity is exceeded, column section with thicker web can be used, Stabilizer plates can be used to reduce the applied moment at the web, or doubler plate can be used to increase the thickness of the column web as shown in Figure 7-5.

$$k = \frac{2T}{l} + \frac{2l}{T} + 8 \tag{14}$$

$$X = k\,\phi\,m\,l \tag{15}$$

In specification notation,

$$\phi_b X = \phi_b k\,\phi\,m\,l \geq X_u = R_u \times a \times \cos\alpha \qquad \text{(LRFD)} \tag{16a}$$

$$\frac{X}{\Omega_b} = \frac{k\,\phi\,m\,l}{\Omega_b} \geq X_a = R_a \times a \times \cos\alpha \qquad \text{(ASD)} \tag{16b}$$

Where:

X = Ultimate bracket moment to which the column web maybe subjected, $kips - in.$

k = coefficient depends on L/T ratio

ϕ = reduction factor (0.6 – 0.9)

m = $f\ S$ = nominal moment in the column web, in.-kips per inch

f = $0.75F_y$ = nominal bending stress of the column web, *ksi*

F_y = yield stress of the column web, *ksi*

S = $\frac{t_c^2}{4}$ = plastic section modulus, in.^{2} per unit width

T = horizontal width of the column web, in.

$\phi_b = 0.90$

$\Omega_b = 1.67$

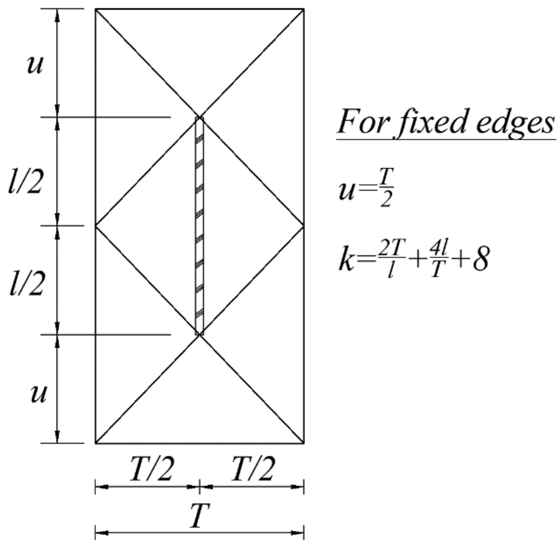


Figure 7-4. Yield lines configuration and coefficient k expression for the case where the plate is welded to the column web (Abolutz and Warner – 1965)

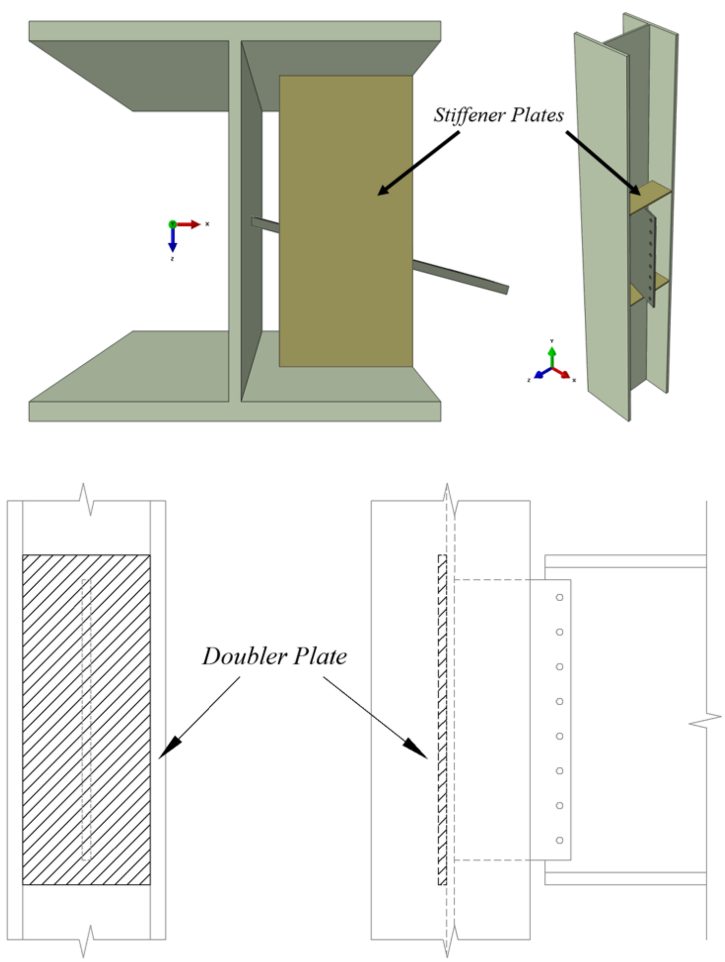


Figure 7-5. Stiffener plates and doubler plate

7.3. Proposed Design Procedure

It was concluded from this research that the connection shear capacity is slightly affected by the connection orientation. However, the connection torsional capacity reduces significantly as connection orientation increases, regardless of the presence of the lateral bracing along the supported beam. The current design procedure in the AISC manual 14th edition (AISC, 2010) for the extended shear tab connections was found to account for orthogonal configurations only. The following procedure is the current design procedure followed by the AISC manual 14th edition (AISC, 2010) to design the orthogonal extended shear tab connections:

1. Estimate the required number of bolts: using the shear strength of one bolt (using AISC manual table 7-1); the required connection shear; distance between weld line and the center of gravity of the bolt group (e); and AISC Manual* Table 7-7, the number of bolts can be estimated.
2. Determine the maximum plate thickness using Equation 10-3 in the AISC Manual* to guarantee that the plate will yield before the bolt shear. Then select a plate thickness that is less than the maximum plate thickness.
3. Check the shear capacity based on the following limit states
    a. Bolt shear using AISC Manual* Table 7-7.
    b. Bolt bearing using AISC Specification** Equations J3-6.
    c. Shear yielding of the plate using AISC Specification** Equation J4-3.
    d. Shear rupture of the plate using AISC Specification** Equation J4-4.
    e. Block shear rupture of the plate using AISC Specification** Equation J4-5.
    f. Local buckling of the plate using AISC Manual* Equation 9-18.

g. Shear yielding, shear buckling, and flexural yielding of plate using AISC Manual* Equation 10-4.
h. Flexural rupture of the plate using AISC Manual* Table 15-3 and Equation 15-3.
i. Weld between the plate and column web using AISC Manual* part 10.
j. Strength of column web at weld using AISC Manual* Equation 9-2.

4. Check the need of using stabilizer plates
   a. Using AISC Manual* Equation 10-6 (Available strength to resist lateral displacement).
   b. Using AISC Manual* Equations 10-7a, 10-7b, 10-8a, and 10-8b (Torsional strength of single-plate shear connection).

This section will show the proposed modified procedure to design the extended shear tab connections including the effect of the connection orientation based on the modifications proposed in this chapter:

1. Estimate the required number of bolts: using the shear strength of one bolt (using AISC Manual* Table 7-1); the required connection shear; distance between weld line and the center of gravity of the bolt group (e); and AISC Manual* Table 7-7, the number of bolts can be estimated.
2. Determine the maximum plate thickness using Equation 10-3 in the AISC Manual* to guarantee that the plate will yield before the bolt shear. Then select a plate thickness that is less than the maximum plate thickness.
3. Check the shear capacity based on the following limit states
   a. Bolt shear using AISC Manual* Table 7-7.
   b. Bolt bearing using AISC Specification** Equations J3-6.

c. Shear yielding of the plate using AISC Specification** Equation J4-3.

d. Shear rupture of the plate using AISC Specification** Equation J4-4.

e. Block shear rupture of the plate using AISC Specification** Equation J4-5.

f. Local buckling of the plate using AISC Manual* Equation 9-18.

g. Shear yielding, shear buckling, and flexural yielding of plate using AISC Manual* Equation 10-4.

h. Flexural rupture of the plate using AISC Manual* Table 15-3 and Equation 15-3.

i. Strength of column web at weld using AISC Manual* Equation 9-2.

4. Check the weld capacity using AISC Manual* part 10 (for orthogonal and skewed configurations), and based on the limit states (Equations 9a-a – 9b-b) derived in this research (for skewed configuration only).

5. Check the need of using stabilizer plates

    a. Using AISC Manual* Equation 10-6 (Available strength to resist lateral displacement).

    b. Using Equations 10-7a and 10-7b in the AISC Manual*, and Equations 13a and 13b in this study (Torsional strength of single-plate shear connection).

6. Check bending strength of the column web using Equations 14, 16a, and 16b in this study (for orthogonal and skewed configurations).

* AISC Manual, 14th edition, 2010

** ANSI/AISC 360-10, 2010

7.4. Examples

Two examples will be solved using the AISC 14th edition procedure and the proposed design procedure to check the adequacy of skewed extended shear tab connections to show the differences between the two procedures.

Example 1 (AISC procedure):

Design the connection between an ASTM A992 W30x148 beam and the web of an ASTM A992 W14x90 to support the following beam end reactions: R_D = 25 kips and R_L = 50 kips. Using ¾ in. diameter ATSM A325-X bolts in standard holes and an ASTM A36 plate. The beam is braced by the floor diaphragm (using LRFD method).

Solution:

The material properties (AISC Manual tables 2-4 and 2-5) are:

	Beam	Column	Plate
F_y, ksi	50	50	36
F_u, ksi	65	65	58

The geometric properties (AISC Manual table 1-1) are:

	Beam	Column
d, in	30.7	14
b_f, in	10.5	14.5
t_f, in	1.18	0.71
t_w, in	0.65	0.44

The required strength (ASCE/SEI 7) is:

$$R_u = 1.2(25) + 1.6(50) = 110 \text{ kips}$$

Using a-distance = 10.5 in. assuming single vertical line of bolts → e = 10.5 in.

1. Estimate the required number of bolts:

$\phi r_n = 22.5\, kips$ (AISC Manual table 7-1)

$$C_{\min} = \frac{P_u}{\phi r_n} = \frac{110}{22.5} = 4.9$$

Using AISC Manual table 7-7 with e= 10.5 in. and S = 3 in. for 8 bolts:

$C = 7.83 > C_{\min} = 4.9$ → Use 8 bolts in single vertical line of bolts

2. Determine the maximum plate thickness:

$F_{nv} = 68\, ksi$ (AISC Specification table J3.2)

$C' = 93.1\, in.$ (AISC Manual table 7-7)

$$M_{\max} = \frac{F_{nv}}{0.90}(A_b C')$$ (AISC Manual Eq. 10-3)

$$M_{\max} = \frac{68}{0.90}(0.442)(93.1) = 3109\, kip-in.$$

$$t_{\max} = \frac{M_{\max}}{F_y d^2}$$ (AISC Manual Eq. 10-2)

$t_{\max} = \dfrac{3109}{36\,(24)^2} = 0.900\, in.$ → Try a plate thickness of ½ in.

3. Check the shear capacity based on the following limit states

a. Bolt shear:

Using AISC Manual table 7-7 with e= 10.5 in. and S = 3 in. for 8 bolts:

$C = 7.83$

$\phi R_n = C\phi r_n = (7.83)(22.5) = 176\, kips > 110\, kips$ ***(ok)***

b. Bolt bearing:

<u>For hole nearest the edge:</u>

$$l_c = 1.5 - \frac{\left(\frac{3}{4} + \frac{1}{16}\right)}{2} = 1.09\,in.$$

$$R_n = 1.2 l_c t F_u \le 2.4 d t F_u$$ (AISC Specification Eq. J3.6a)

$$R_n = 1.2(1.09)(0.5)(58) \le 2.4(0.75)(0.5)(58)$$

$$R_n = 37.9\,kips/bolt \le 52.2\,kips/bolt$$

$$R_n = 37.9\,kips/bolt$$

<u>For the intermediate hole:</u>

$$l_c = 3 - \left(\frac{3}{4} + \frac{1}{16}\right) = 2.49\,in.$$

$$R_n = 1.2 l_c t F_u \le 2.4 d t F_u$$ (AISC Specification Eq. J3.6a)

$$R_n = 1.2(2.49)(0.5)(58) \le 2.4(0.75)(0.5)(58)$$

$$R_n = 86.7\,kips/bolt \ge 52.2\,kips/bolt$$

$$R_n = 52.2\,kips/bolt$$

<u>Bolt Bearing on Plate:</u>

$$\phi R_n = (0.75)\left[2(37.9) + 7(52.2)\right] = 330.9\,kips > 110\,kips$$ ***(ok)***

c. Shear yielding of the plate:

$$\phi R_n = \phi 0.60 F_y A_{gv}$$ (AISC Specification Eq. J3.6a)

$$\phi R_n = (1)(0.60)(36)(24)(0.5) = 260\,kips > 110\,kips$$ ***(ok)***

d. Shear rupture of the plate:

$$A_{nv} = t_p\left[d - n\left(d_b + \frac{1}{8}\right)\right] = 0.5\left[24 - 8\left(\frac{3}{4} + \frac{1}{8}\right)\right] = 8.5\,in.^2$$

$$\phi R_n = \phi 0.60 F_u A_{nv} \qquad \text{(AISC Specification Eq. J4-4)}$$

$$\phi R_n = (0.75)(0.60)(58)(8.5) = 222\,kips > 110\,kips \qquad \textbf{\textit{(ok)}}$$

e. Block shear rupture of the plate:

$$\phi R_n = \phi U_{bs} F_u A_{nt} + \min(\phi 0.60 F_y A_{gv}, \phi 0.60 F_u A_{nv}) \qquad \text{(AISC Specification Eq. J4-5)}$$

Tension rupture component:

$$A_{nt} = 0.5\left[1.5 - \frac{1}{2}\left(\frac{3}{4} + \frac{1}{8}\right)\right] = 0.53\,in.^2$$

$$\phi U_{bs} F_u A_{nt} = 0.75(0.5)(58)(0.53) = 12\,kips$$

Shear yielding component:

$$\phi 0.60 F_y A_{gv} = 0.75(0.6)(36)(24)(0.5) = 182.2\,kips$$

Shear rupture component:

$$A_{nv} = \left[24 - 1.5 - (8 - 0.5)\left(\frac{3}{4} + \frac{1}{8}\right)\right](0.5) = 7.97\,in.^2$$

$$\phi 0.60 F_u A_{nv} = 0.75(0.6)(58)(7.97) = 208.0\,kips$$

Block shear rupture of the plate

$$\phi R_n = \phi U_{bs} F_u A_{nt} + \min(\phi 0.60 F_y A_{gv}, \phi 0.60 F_u A_{nv})$$

$$\phi R_n = 12 + \min(182.2\,, 208.2) = 194\,kips > 110\,kips \qquad \textbf{\textit{(ok)}}$$

f. Local buckling of the plate:

$$\lambda = \frac{h_o \sqrt{F_y}}{10 t_w \sqrt{475 + 280\left(\frac{h_o}{c}\right)^2}} \qquad \text{(AISC Manual Eq. 9-18)}$$

$$\lambda = \frac{24\sqrt{36}}{10(0.5)\sqrt{475 + 280\left(\frac{24}{10.5}\right)^2}} = 0.655 < 0.7$$, therefore, Q = 1.0

$QF_y = F_y$ → Therefore, plate buckling is not a controlling limit state.

g. Shear yielding, shear buckling, and flexural yielding of plate:

$$V_u = 110\, kips,\ \phi_v V_n = 260\, kips$$

$$M_u = V_u e = 110(10.5) = 1155\, kip - in.$$

$$\phi_b M_n = \phi_b Q F_y Z_{pl} = 0.9(1.0)(36)\left[\frac{0.5(24)^2}{4}\right] = 2332\, kip - in.$$

$$\left(\frac{V_u}{\phi_v V_n}\right)^2 + \left(\frac{M_u}{\phi_b M_n}\right)^2 \le 1.0$$ (AISC Manual Eq. 10-4)

$$\left(\frac{110}{260}\right)^2 + \left(\frac{1155}{2332}\right)^2 = 0.425 \le 1.0$$ ***(ok)***

h. Flexural rupture of the plate:

$$Z_{net} = 51\, in.^3$$ (AISC Manual table 15-3)

$$\phi M_n = \phi\, F_u Z_{net}$$ (AISC Manual Eq. 9-4)

$$\phi M_n = 0.75(58)(51) = 2219\, kip - in. > M_u = 1155\, kip - in.$$

(ok)

i. Weld between the plate and column web:

$$w = \frac{5}{8} t_p = \frac{5}{8}(0.5) = 0.3125\, in.$$ (AISC Manual Part 10)

Therefore use a 5/16 in. fillet weld on both sides of the plate.

j. Strength of column web at weld:

$$t_{\min} = \frac{3.09D}{F_u}$$ (AISC Manual Eq. 9-2)

$$t_{\min} = \frac{3.09(5)}{65} = 0.238\,in. < 0.44\,in.$$ ***(ok)***

4. Check the need of using stabilizer plates

   a. Available strength to resist lateral displacement:

$$\phi\, R_n = \phi(1500)(\pi)\frac{(Lt_p{}^3)}{a^2}$$ (AISC Manual Eq. 10-6)

$$\phi\, R_n = 0.9(1500)(\pi)\frac{(24(0.5)^3)}{(10.5)^2} = 115\,kips > 110\,kips$$ ***(ok)***

   b. Torsional strength of single-plate shear connection:

$$M_{tu} \le \phi M_n = \left(\phi_v(0.6F_{yp}) - \frac{R_u}{lt_p}\right)\frac{lt_p{}^2}{2} + \frac{2R_u{}^2(t_w+t_p)b_f}{(\phi_b F_{yb})L_s t_w{}^2}$$ (AISC Manual Eq. 10-7a)

$$M_{tu} = R_u\left(\frac{t_w + t_p}{2}\right)$$ (AISC Manual Eq. 10-8a)

$$M_{tu} = 110\left(\frac{0.65+0.5}{2}\right) = 63.25\,kips - in.$$

$$\phi M_n = \left((1.0)(0.6(36)) - \frac{110}{(24)(0.5)}\right)\frac{(24)(0.5)^2}{2} + \frac{2(110)^2(0.65+0.5)(10.5)}{(0.9(50))(360)(0.65)^2}$$

$$\phi M_n = 80\,kips - in. > 63.25\,kips - in.$$ ***(ok)***

$\therefore$ No stabilizer plates are required

Note that the design of the same connection with any skewed angle will be exactly the same as the previous example as mentioned in the AISC manual (the design of skewed single plate connection is similar to the design of orthogonal single plate connection).

<u>Example 2 (Proposed procedure – Skewed Connection):</u>

Design the connection between an ASTM A992 W30x148 beam and the web of an ASTM A992 W14x90 to support the following beam end reactions: R_D = 25 kips and R_L = 50 kips. Using ¾ in. diameter ATSM A325-X bolts in standard holes and an ASTM A36 plate. The beam is braced by the floor diaphragm (using the proposed procedure). The connection is skewed by 10 degrees.

<u>Solution:</u>

The material properties (AISC Manual tables 2-4 and 2-5) are:

	Beam	Column	Plate
F_y, ksi	50	50	36
F_u, ksi	65	65	58

The geometric properties (AISC Manual table 1-1) are:

	Beam	Column
d, in	30.7	14
b_f, in	10.5	14.5
t_f, in	1.18	0.71
t_w, in	0.65	0.44

The required strength (ASCE/SEI 7) is:

R_u = 1.2(25) + 1.6(50) = 110 kips

Using a-distance = 10.5 in. assuming single vertical line of bolts → e = 10.5 in.

1. Estimate the required number of bolts:

   $\phi r_n = 22.5\, kips$ (AISC Manual table 7-1)

   $$C_{\min} = \frac{P_u}{\phi r_n} = \frac{110}{22.5} = 4.9$$

   Using AISC Manual table 7-7 with e= 10.5 in. and S = 3 in. for 8 bolts:

   $C = 7.83 > C_{\min} = 4.9$ → Use 8 bolts in single vertical line of bolts

2. Determine the maximum plate thickness:

$F_{nv} = 68\,ksi$ (AISC Specification table J3.2)

$C' = 93.1\,in.$ (AISC Manual table 7-7)

$$M_{\max} = \frac{F_{nv}}{0.90}(A_b C')$$ (AISC Manual Eq. 10-3)

$$M_{\max} = \frac{68}{0.90}(0.442)(93.1) = 3109\,kip - in.$$

$$t_{\max} = \frac{M_{\max}}{F_y d^2}$$ (AISC Manual Eq. 10-2)

$t_{\max} = \dfrac{3109}{36\,(24)^2} = 0.900\,in.$ → Try a plate thickness of ½ in.

3. Check the shear capacity based on the following limit states

   a. Bolt shear:

      Using AISC Manual table 7-7 with e= 10.5 in. and S = 3 in. for 8 bolts:

      $C = 7.83$

      $\phi R_n = C\phi\, r_n = (7.83)(22.5) = 176\,kips > 110\,kips$ ***(ok)***

   b. Bolt bearing:

      <u>For hole nearest the edge:</u>

$$l_c = 1.5 - \frac{\left(\frac{3}{4} + \frac{1}{16}\right)}{2} = 1.09\,in.$$

$R_n = 1.2 l_c t F_u \le 2.4 d t F_u$ (AISC Specification Eq. J3.6a)

$$R_n = 1.2(1.09)(0.5)(58) \le 2.4(0.75)(0.5)(58)$$

$$R_n = 37.9\,kips/bolt \le 52.2\,kips/bolt$$

$$R_n = 37.9\,kips/bolt$$

For the intermediate hole:

$$l_c = 3 - \left(\frac{3}{4} + \frac{1}{16}\right) = 2.49\,in.$$

$$R_n = 1.2 l_c t F_u \leq 2.4 d t F_u \qquad \text{(AISC Specification Eq. J3.6a)}$$

$$R_n = 1.2(2.49)(0.5)(58) \leq 2.4(0.75)(0.5)(58)$$

$$R_n = 86.7\,kips/bolt \geq 52.2\,kips/bolt$$

$$R_n = 52.2\,kips/bolt$$

Bolt Bearing on Plate:

$$\phi R_n = (0.75)\left[2(37.9) + 7(52.2)\right] = 330.9\,kips > 110\,kips \qquad \textbf{\textit{(ok)}}$$

c. Shear yielding of the plate:

$$\phi R_n = \phi 0.60 F_y A_{gv} \qquad \text{(AISC Specification Eq. J3.6a)}$$

$$\phi R_n = (1)(0.60)(36)(24)(0.5) = 260\,kips > 110\,kips \qquad \textbf{\textit{(ok)}}$$

d. Shear rupture of the plate:

$$A_{nv} = t_p\left[d - n\left(d_b + \frac{1}{8}\right)\right] = 0.5\left[24 - 8\left(\frac{3}{4} + \frac{1}{8}\right)\right] = 8.5\,in.^2$$

$$\phi R_n = \phi 0.60 F_u A_{nv} \qquad \text{(AISC Specification Eq. J4-4)}$$

$$\phi R_n = (0.75)(0.60)(58)(8.5) = 222\,kips > 110\,kips \qquad \textbf{\textit{(ok)}}$$

e. Block shear rupture of the plate:

$$\phi R_n = \phi U_{bs} F_u A_{nt} + \min(\phi 0.60 F_y A_{gv}, \phi 0.60 F_u A_{nv}) \qquad \text{(AISC Specification Eq. J4-5)}$$

Tension rupture component:

$$A_{nt} = 0.5\left[1.5 - \frac{1}{2}\left(\frac{3}{4} + \frac{1}{8}\right)\right] = 0.53\,in.^2$$

$$\phi U_{bs} F_u A_{nt} = 0.75(0.5)(58)(0.53) = 12\, kips$$

<u>Shear yielding component:</u>

$$\phi 0.60 F_y A_{gv} = 0.75(0.6)(36)(24)(0.5) = 182.2\, kips$$

<u>Shear rupture component:</u>

$$A_{nv} = \left[24 - 1.5 - (8 - 0.5)\left(\frac{3}{4} + \frac{1}{8}\right)\right](0.5) = 7.97\, in.^2$$

$$\phi 0.60 F_u A_{nv} = 0.75(0.6)(58)(7.97) = 208.0\, kips$$

<u>Block shear rupture of the plate</u>

$$\phi R_n = \phi U_{bs} F_u A_{nt} + \min(\phi 0.60 F_y A_{gv}, \phi 0.60 F_u A_{nv})$$

$$\phi R_n = 12 + \min(182.2\,, 208.2) = 194\, kips > 110\, kips$$ ***(ok)***

f. Local buckling of the plate:

$$\lambda = \frac{h_o \sqrt{F_y}}{10 t_w \sqrt{475 + 280\left(\frac{h_o}{c}\right)^2}}$$ (AISC Manual Eq. 9-18)

$$\lambda = \frac{24\sqrt{36}}{10(0.5)\sqrt{475 + 280\left(\frac{24}{10.5}\right)^2}} = 0.655 < 0.7$$, therefore, $Q = 1.0$

$QF_y = F_y$ → Therefore, plate buckling is not a controlling limit state.

g. Shear yielding, shear buckling, and flexural yielding of plate:

$$V_u = 110\, kips,\ \phi_v V_n = 260\, kips$$

$$M_u = V_u e = 110(10.5) = 1155\, kip - in.$$

$$\phi_b M_n = \phi_b Q F_y Z_{pl} = 0.9(1.0)(36)\left[\frac{0.5(24)^2}{4}\right] = 2332\, kip - in.$$

$$\left(\frac{V_u}{\phi_v V_n}\right)^2 + \left(\frac{M_u}{\phi_b M_n}\right)^2 \leq 1.0$$ (AISC Manual Eq. 10-4)

$$\left(\frac{110}{260}\right)^2 + \left(\frac{1155}{2332}\right)^2 = 0.425 \leq 1.0$$ ***(ok)***

h. Flexural rupture of the plate:

$$Z_{net} = 51\, in.^3$$ (AISC Manual table 15-3)

$$\phi M_n = \phi\, F_u Z_{net}$$ (AISC Manual Eq. 9-4)

$$\phi M_n = 0.75(58)(51) = 2219\, kip-in. > M_u = 1155\, kip-in.$$

(ok)

i. Strength of column web at weld:

$$t_{\min} = \frac{3.09D}{F_u}$$ (AISC Manual Eq. 9-2)

$$t_{\min} = \frac{3.09(5)}{65} = 0.238\, in. < 0.44\, in.$$ ***(ok)***

4. Weld between the plate and column web:

$$w = \frac{5}{8}t_p = \frac{5}{8}(0.5) = 0.3125\, in.$$ (AISC Manual Part 10)

$$D_{1,\min} = \frac{R_u}{C_I C_1 l} \quad \textit{(LRFD)}$$ (9a-a)

Using AISC Manual table 8-4 with $e_x = 10.5 \times \sin 20 = 3.59$ in. and

$k = 0.5/24 = 0.021$:

$C_I = 3.67$

$$D_{1,\min} = \frac{110}{(3.67)(1.0)(24)} = 1.25 < 5$$ ***(ok)***

$$D_{2,\min} = \frac{R_u}{C_O C_1 l} \quad \textit{(LRFD)} \tag{9b-a}$$

Using AISC Manual table 8-4 with $e_x = 10.5 \times \cos 20 = 9.87$ in. and $k = 0$:

$$C_O = 2.62$$

$$D_{2,\min} = \frac{110}{(2.62)(1.0)(24)} = 1.75 < 5 \qquad \textbf{\textit{(ok)}}$$

Therefore use a 5/16 in. fillet weld on both sides of the plate.

5. Check the need of using stabilizer plates

   a. Available strength to resist lateral displacement:

$$\phi\, R_n = \phi(1500)(\pi)\frac{(Lt_p{}^3)}{a^2} \qquad \text{(AISC Manual Eq. 10-6)}$$

$$\phi\, R_n = 0.9(1500)(\pi)\frac{(24(0.5)^3)}{(10.5)^2} = 115\, kips > 110\, kips \qquad \textbf{\textit{(ok)}}$$

   b. Torsional strength of single-plate shear connection:

$$M_{tu} \leq \phi M_n = \left(\phi_v(0.6F_{yp}) - \frac{R_u}{lt_p}\right)\frac{lt_p{}^2}{2} + \frac{2R_u{}^2(t_w + t_p)b_f}{(\phi_b F_{yb})L_s t_w{}^2} \qquad \text{(AISC Manual Eq. 10-7a)}$$

$$M_{tu} = R_u \times \left[\left(\frac{t_w + t_p}{2}\right) + a \times \sin\alpha\right] \qquad \text{(LRFD)} \tag{13a}$$

$$M_{tu} = 110 \times \left[\left(\frac{0.65 + 0.5}{2}\right) + 10.5 \times \sin 10\right] = 263.8\, kips - in.$$

$$\phi M_n = \left((1.0)(0.6(36)) - \frac{110}{(24)(0.5)}\right)\frac{(24)(0.5)^2}{2} + \frac{2(110)^2(0.65 + 0.5)(10.5)}{(0.9(50))(360)(0.65)^2}$$

$$\phi M_n = 80\, kips - in. < 263.8\, kips - in. \qquad \textbf{\textit{(no good)}}$$

$\therefore$ Stabilizer plates are required

6. Bending strength of the column web:

$$k = \frac{2T}{l} + \frac{2l}{T} + 8 \tag{14}$$

$$k = \frac{2(14 - 2 \times 0.6875)}{24} + \frac{4(24)}{(14 - 2 \times 0.6875)} + 8 = 16.66$$

$$m = 0.75\, F_y\, \frac{t_c^2}{4} = 0.75\,(50)\,\frac{0.4375^2}{4} = 1.79\, kips - in.\ per\ inch$$

$$\phi_b X = \phi_b k\, \phi\, m\, l \geq X_u = R_u \times a \times \cos\alpha \qquad \text{(LRFD)} \tag{16a}$$

$$X_u = 110 \times 10.5 \times \cos 10 = 1137.5\, kips - in.$$

$$\phi_b X = 0.9(16.66)(0.75)(1.79)(24) = 484.2\, kips - in.$$

$$\phi_b X = 484.2\, kips - in. < 1137.5\, kips - in. \qquad \textbf{\textit{(no good)}}$$

$\therefore$ Stiffener plates or doubler plate are required.

8. CHAPTER EIGHT – CONCLUSIONS, OBSERVATIONS AND FUTURE RESEARCH

8.1. Conclusions and Observations

A detailed nonlinear finite element model was developed to predict the behavior and performance of skewed extended shear tab connections. The models have been validated against data from experiments done by Sherman-Ghorpanboor (2002) and Metzger (2006) for orthogonal extended shear tab connections. In this research, a highly detailed model was developed where special attention was given to element selection, material properties, contact properties, loading, loading steps, boundary conditions, meshing, and model solution. The following observations and conclusions can be made from this investigation based on the finite element results. It should be noted that Sherman-Ghorpanboor (2002) and Metzger (2006) experimental results were used in this research only for validation purposes:

1. The finite element results were in good agreement with the experimental results. The proposed FEA can be used to determine the shear capacity, capture the plastic behavior and failure modes of the orthogonal and skewed extended shear tab connections with flexible and rigid supports, for unstiffened and stiffened configurations.
2. Additional secondary failure modes were obtained from the FEA for the orthogonal configurations, these failure modes are hard to be detected using visual inspection during experiments since the plastic deformation is low for these failure modes.
3. For the unstiffened skewed connections with flexible supports, the shear tab twist increases

with the increase of the connection orientation due to the additional torsional moment applied at the shear tab end along the weld line.

4. The torsional moment applied at the skewed connection is the sum of two components: The torsional moment due to the small eccentricity because of the overlap between the shear tab and beam longitudinal axes, and the torsional moment due to the connection orientation.
5. The connection vertical displacement slightly decreases with the increase of the connection orientation.
6. For unstiffened skewed connections with rigid support, the column contributes significantly in resisting the moment components since the moment component ($R \times a \times \cos\alpha$) and moment component ($R \times a \times \sin\alpha$) are applied along the column strong axis and column flange strong axis, respectively. Thus, the effect of the connection orientation on the connection bending and torsional behavior is insignificant and can be neglected.
7. For stiffened skewed connections, It was observed that the connection vertical displacement and shear tab twist are slightly affected by the connection orientation since the shear tab unsupported length is short due to the presence of the stiffeners which makes the connection act like the conventional configuration with an a-distance of 3.5 in. maximum.
8. Also, the welded area between the shear tab and the stiffeners increases with the increase of the connection orientation. This will increase the length of the horizontal weld group leading to shifting the center of gravity of the total weld group closer to the bolt line, leading to the reduction of the a-distance.

9. Thus, the effect of the connection orientation on the connection bending and torsional behavior for stiffened skewed connections is insignificant and can be neglected.
10. A design procedure for skewed and orthogonal extended shear tab connections has been proposed in this research. This procedure is based on results obtained from 300 finite element models for beam-to-column web connections with different configurations.
11. For a one a-distance, as the skewed angle increases, the connection vertical displacement slightly decreases and the plate twist along weld line increases significantly.
12. The relative twist and connection shear capacity are almost constant and independent from the connection orientation.
13. The connection vertical displacement and the plate relative twist significantly decrease with the increase of the plate thickness.
14. The plate twist along the weld line and the connection shear capacity are slightly affected by the change of the plate thickness.
15. The connection’s shear capacity decreases with the increase of the a-distance due to the increase of the applied moment components on the connection.
16. The shear tab twist due to the overlap between the shear tab and beam web longitudinal axes (relative twist) decreases with the increase of the a-distance because of the reduction in the connection shear.

8.2. Future Work

1. The proposed design procedure is based on 300 different configurations for the skewed extended shear tab connections that includes the interaction between the connection orientation, shear tab thickness, a-distance, and number of bolts. The range of these parameters can be extended and more configurations can be investigated to check the

adequacy of the proposed design procedure in this research.

2. Currently, the authors have been working on studying the behavior of skewed extended shear tab connections with two and three vertical rows of bolts to investigate the connection end rotation with different configurations and compare the results with AISC Manual (AISC, 2010) criterion.
3. Also, the effect of the residual stresses due to welding and fabrication process on the behavior of the skewed extended shear tab connections is being investigated analytically to check the effectiveness and practicality of modeling the weld group in the FEA models.

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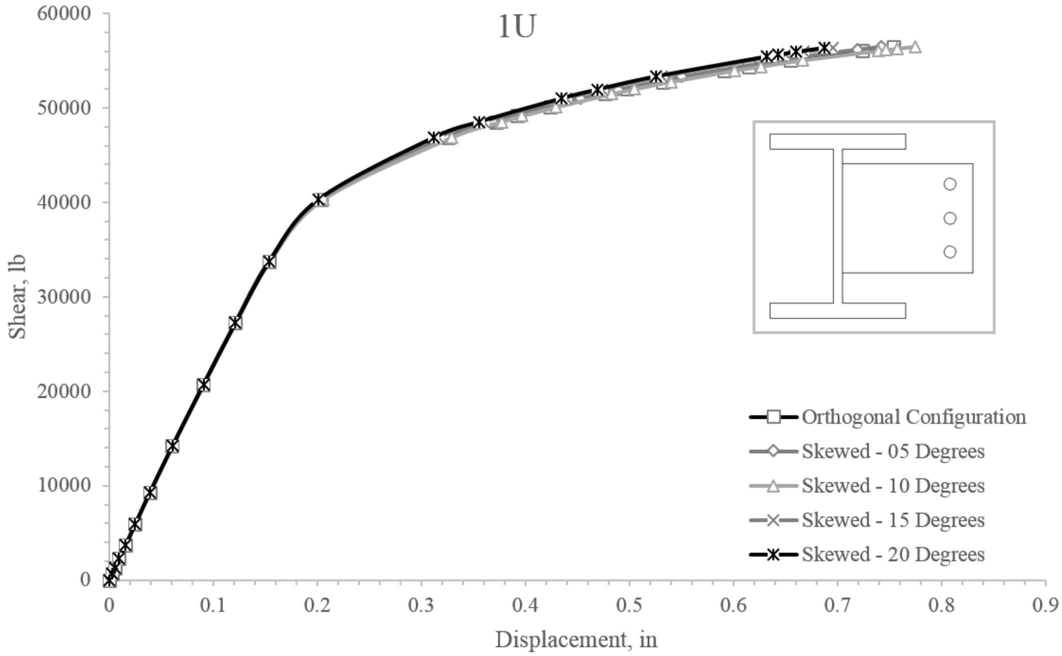
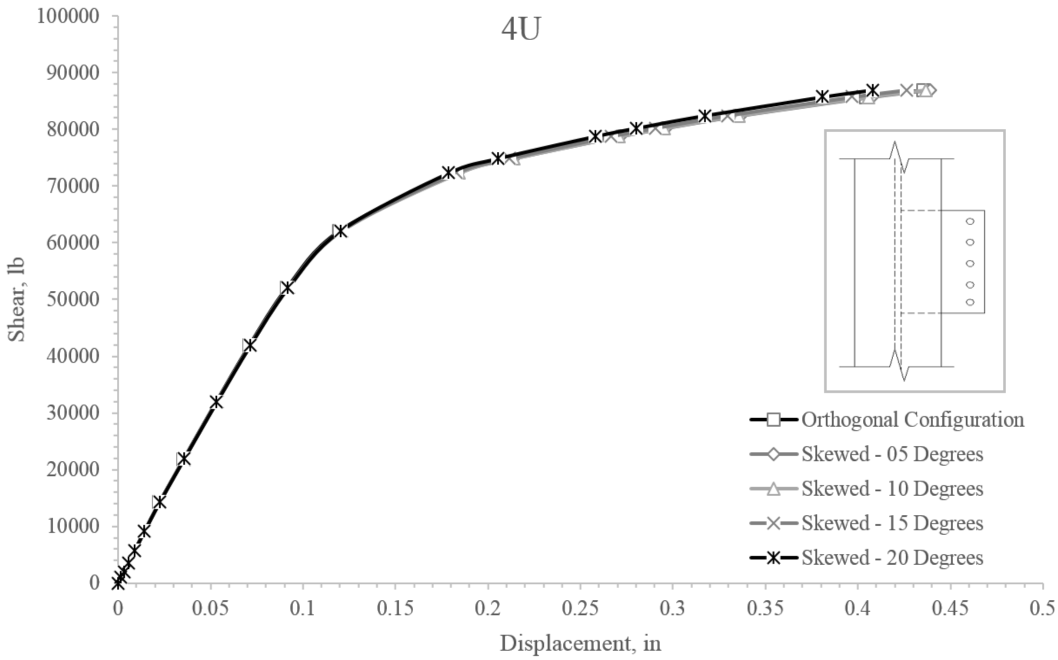
APPENDICES

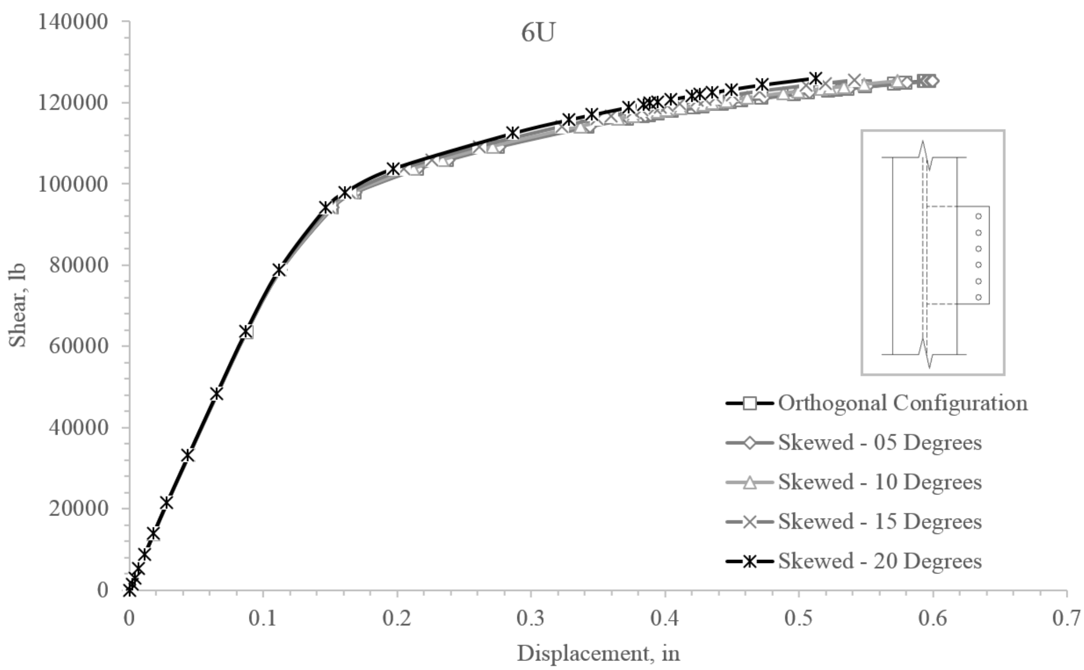
Appendix A (Skewed Connections Curves – Chapter 5)

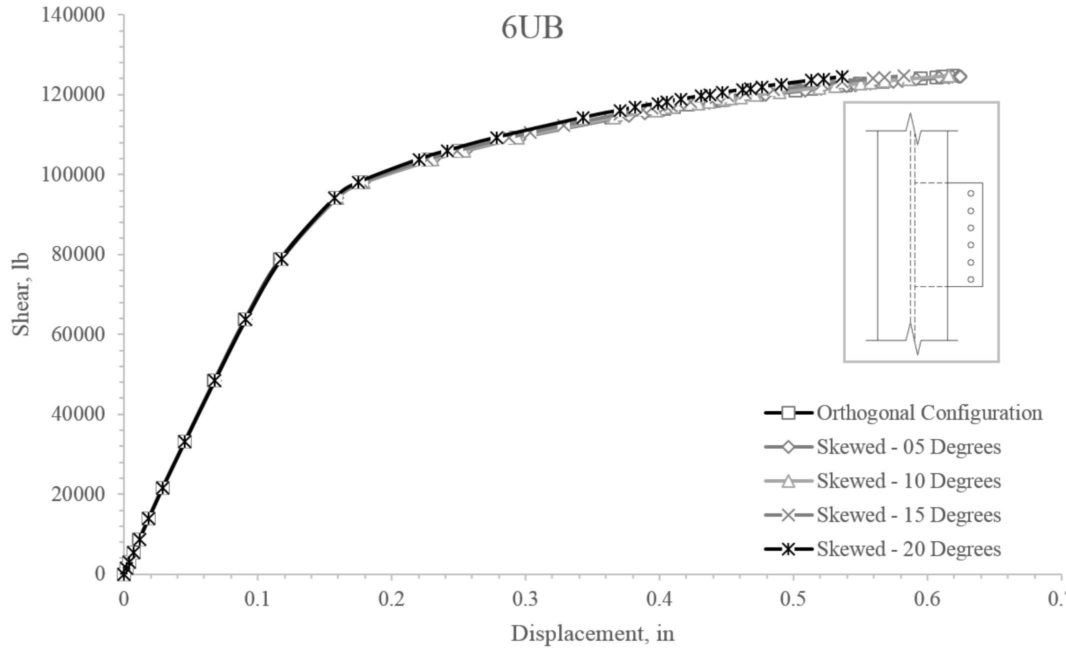
A1. Unstiffened Connections with Flexible Supports

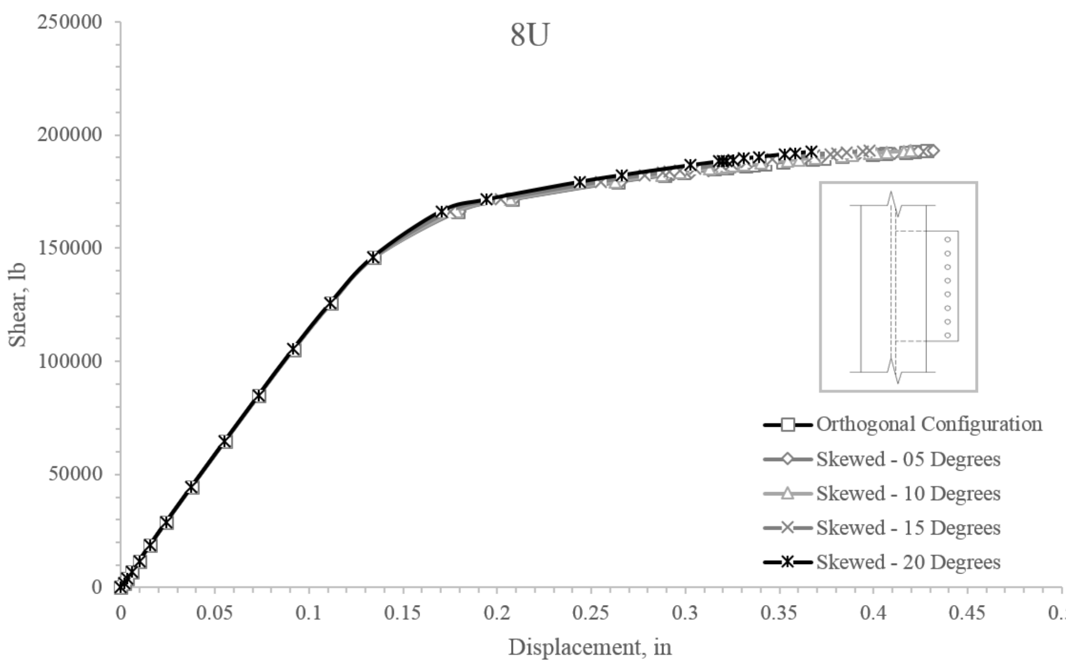
A1.1. Shear-Vertical Displacement Curves



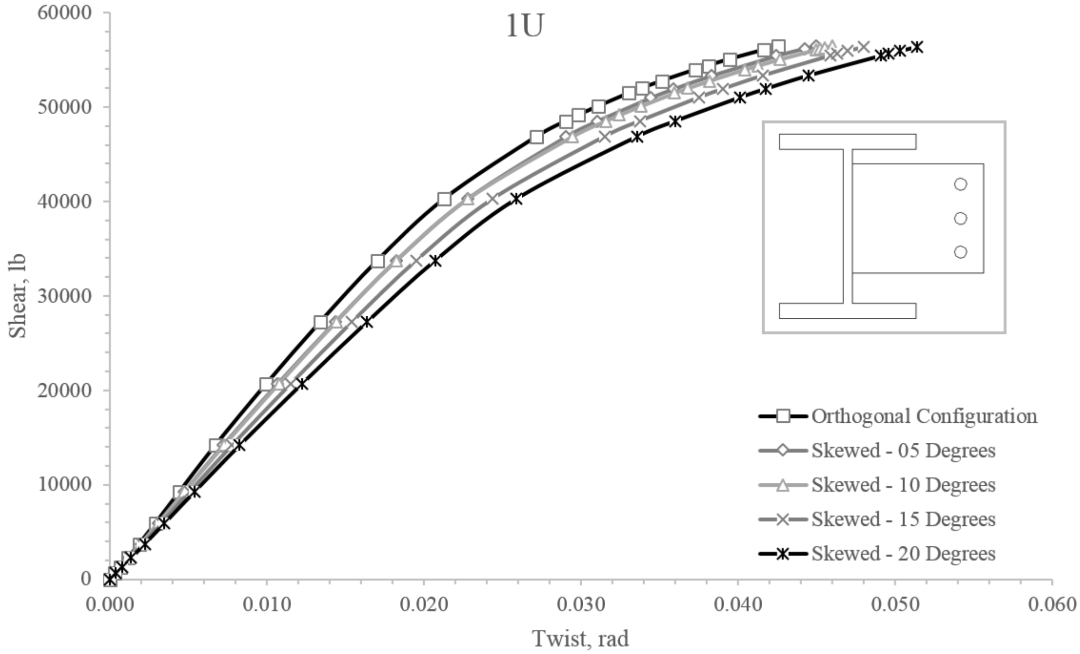

4U
Shear, lb
Displacement, in
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees
100000
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5

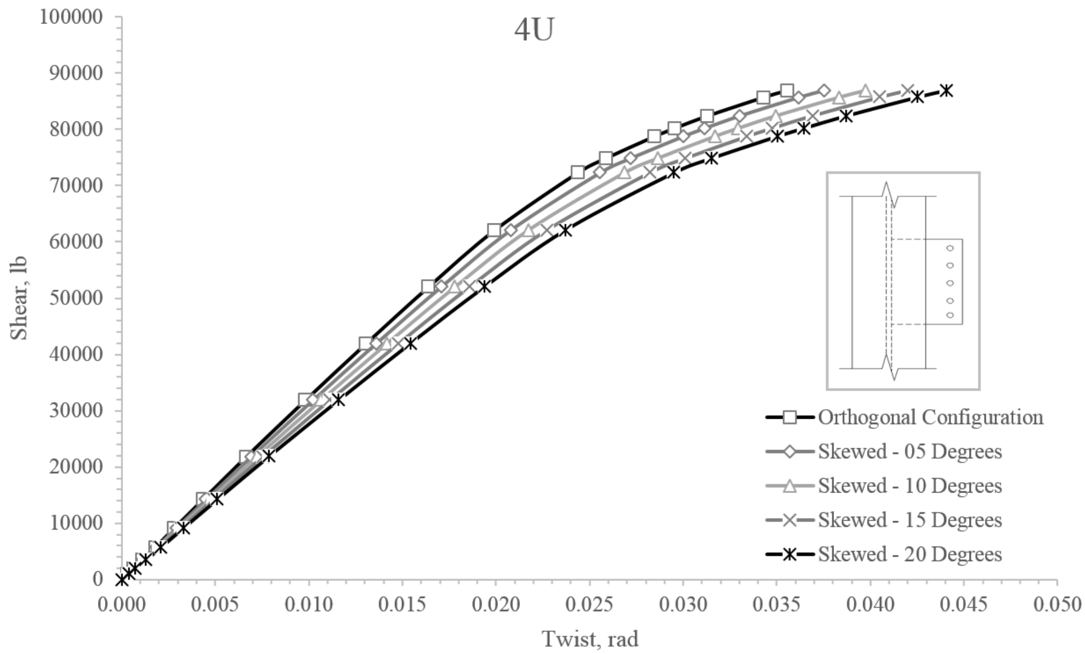

6U
Shear, lb
Displacement, in
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees
140000
120000
100000
80000
60000
40000
20000
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7

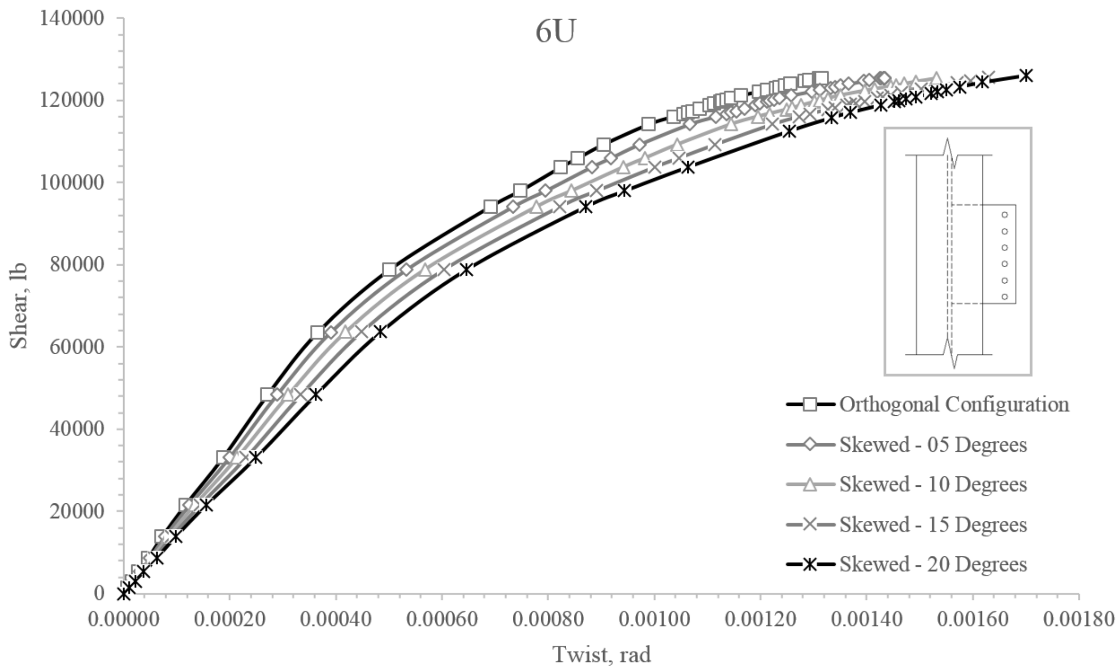

6UB
Shear, lb
Displacement, in
140000
120000
100000
80000
60000
40000
20000
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

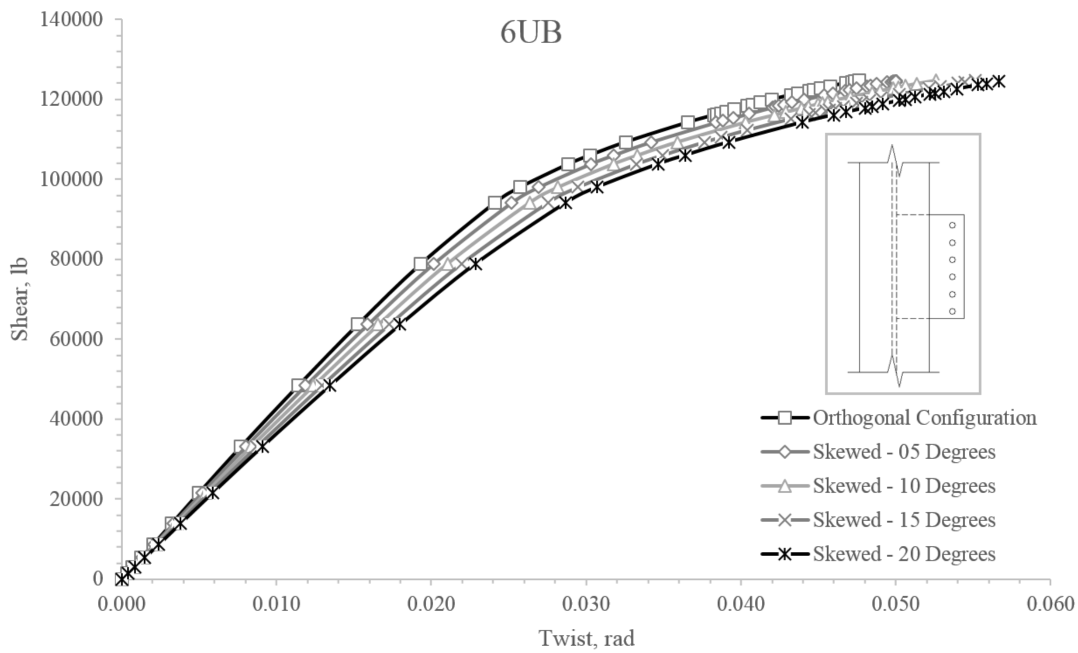

8U
Shear, lb
Displacement, in
250000
200000
150000
100000
50000
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

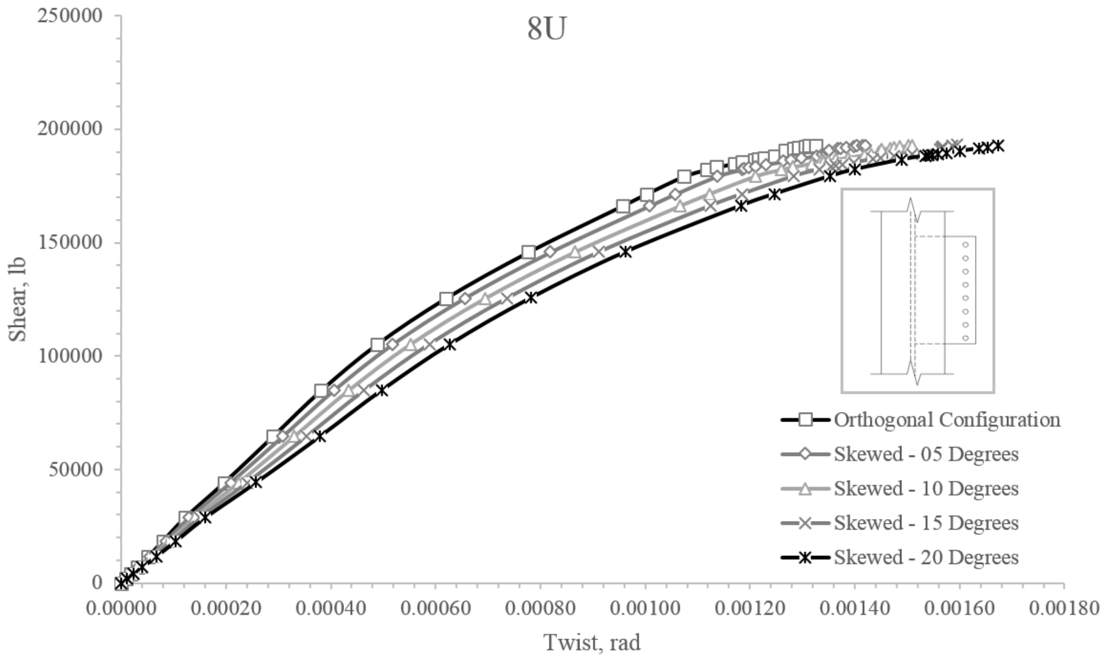
A1.2. Shear-Shear Tab Twist Curves





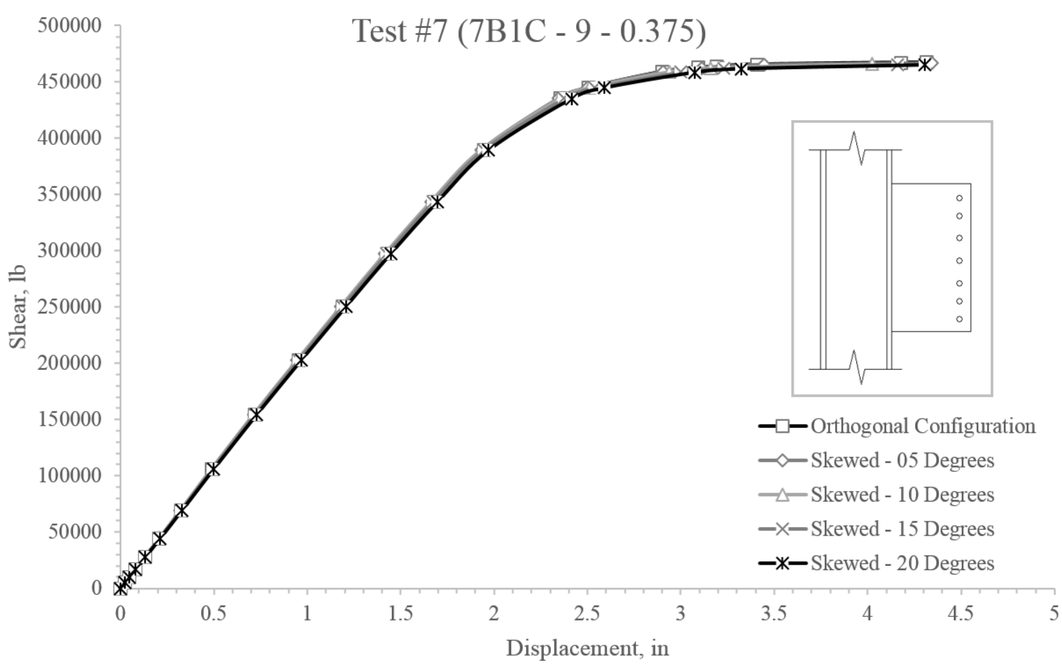

6U
Shear, lb
Twist, rad
140000
120000
100000
80000
60000
40000
20000
0
0.00000
0.00020
0.00040
0.00060
0.00080
0.00100
0.00120
0.00140
0.00160
0.00180
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

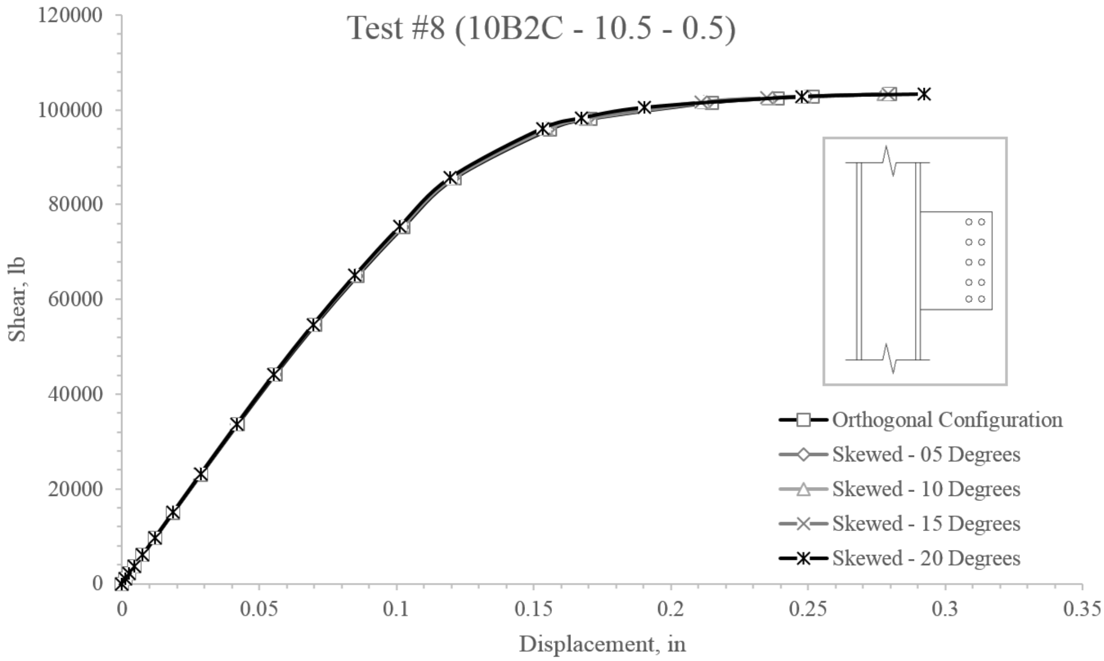

6UB
Shear, lb
Twist, rad
140000
120000
100000
80000
60000
40000
20000
0
0.000
0.010
0.020
0.030
0.040
0.050
0.060
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees



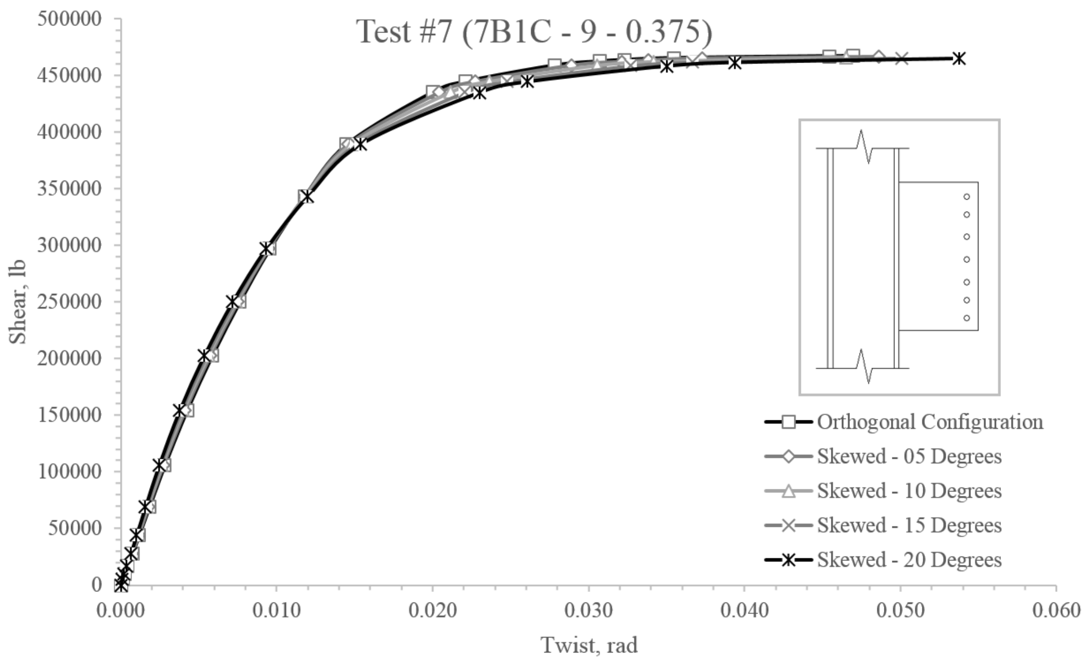
A2. Unstiffened Connections with Rigid Supports

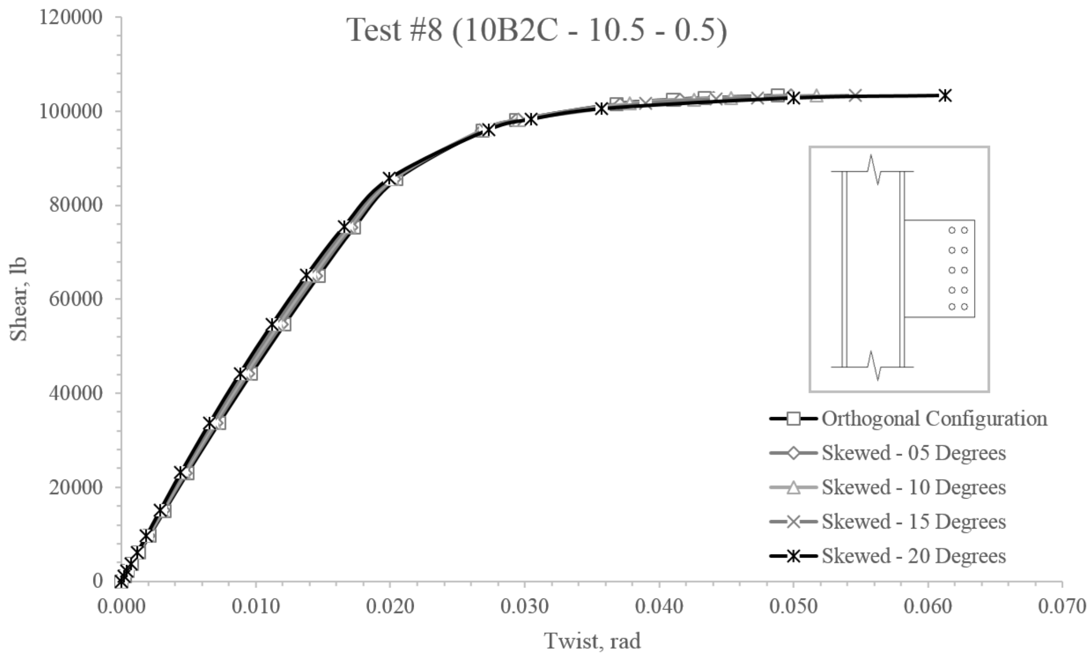
A2.1. Shear-Vertical Displacement Curves





A2.2. Shear-Shear Tab Twist Curves

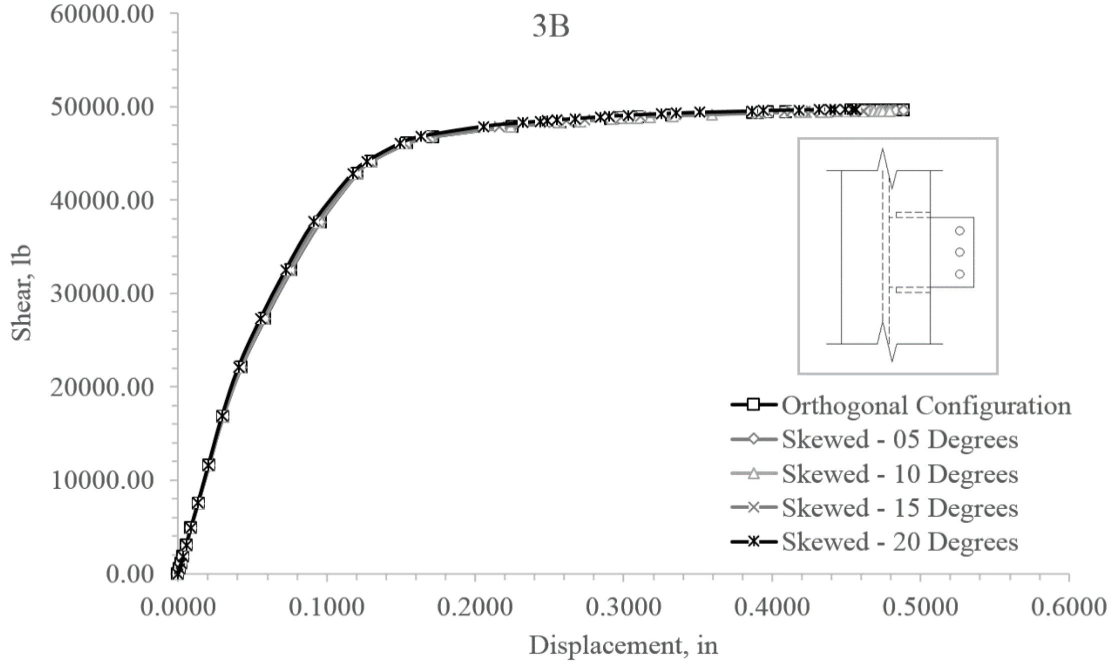


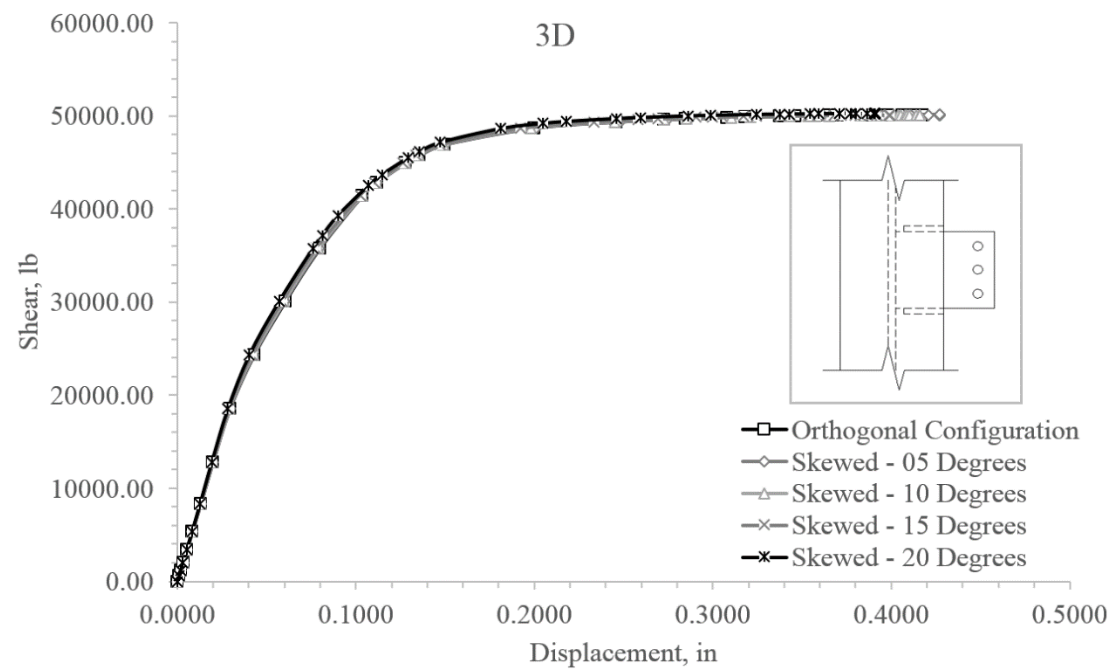


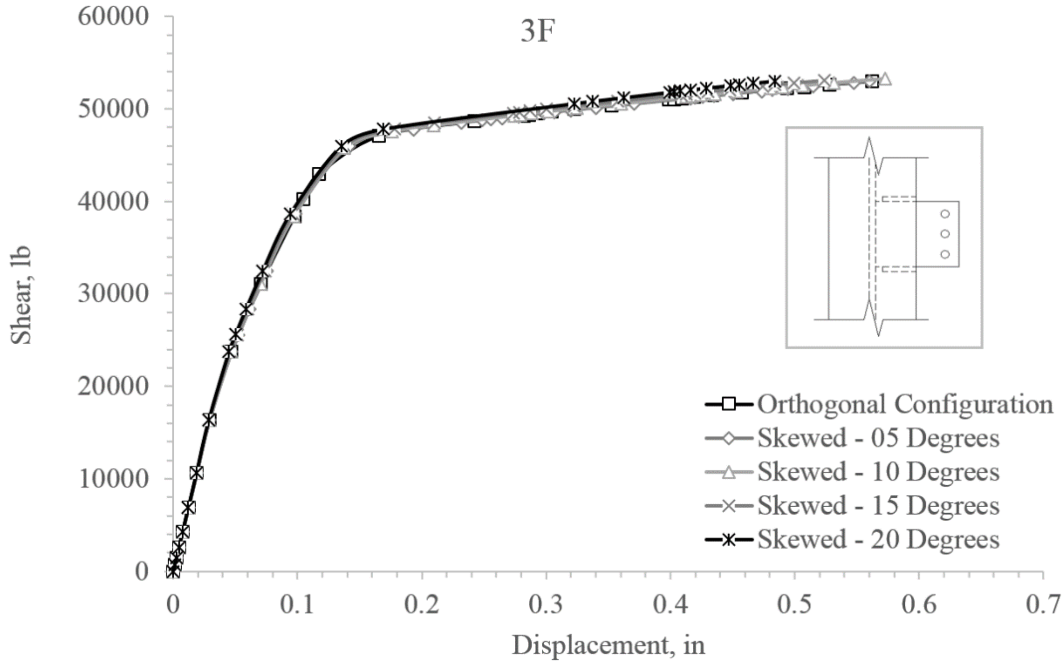
Test #8 (10B2C - 10.5 - 0.5)
Shear, lb
Twist, rad
120000
100000
80000
60000
40000
20000
0
0.000
0.010
0.020
0.030
0.040
0.050
0.060
0.070
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

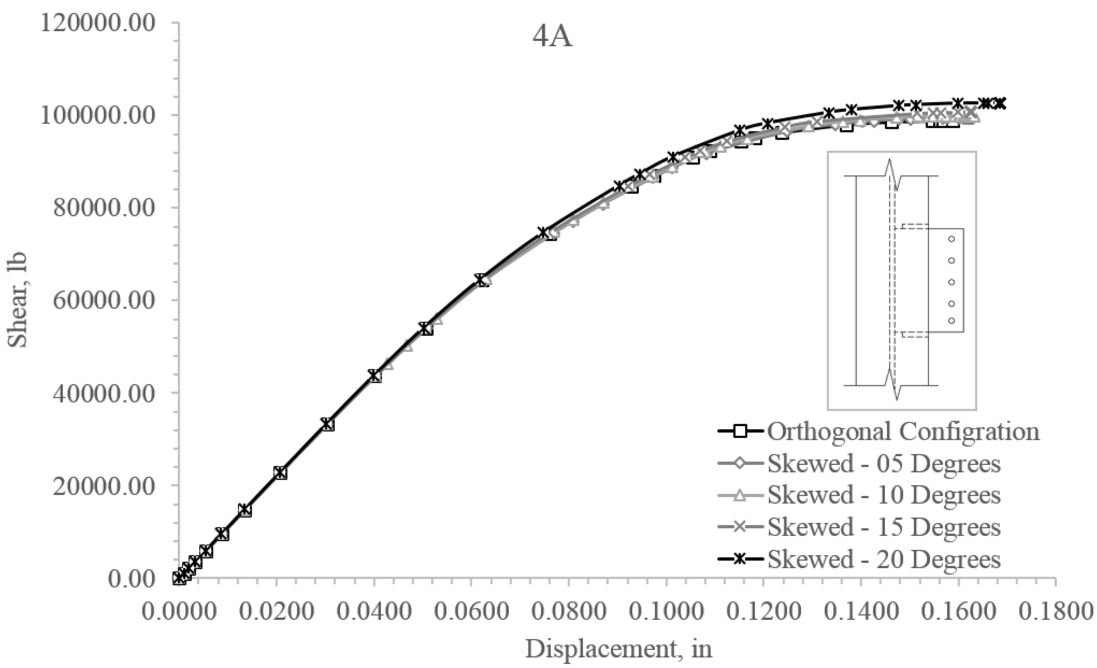
A3. Stiffened Connections

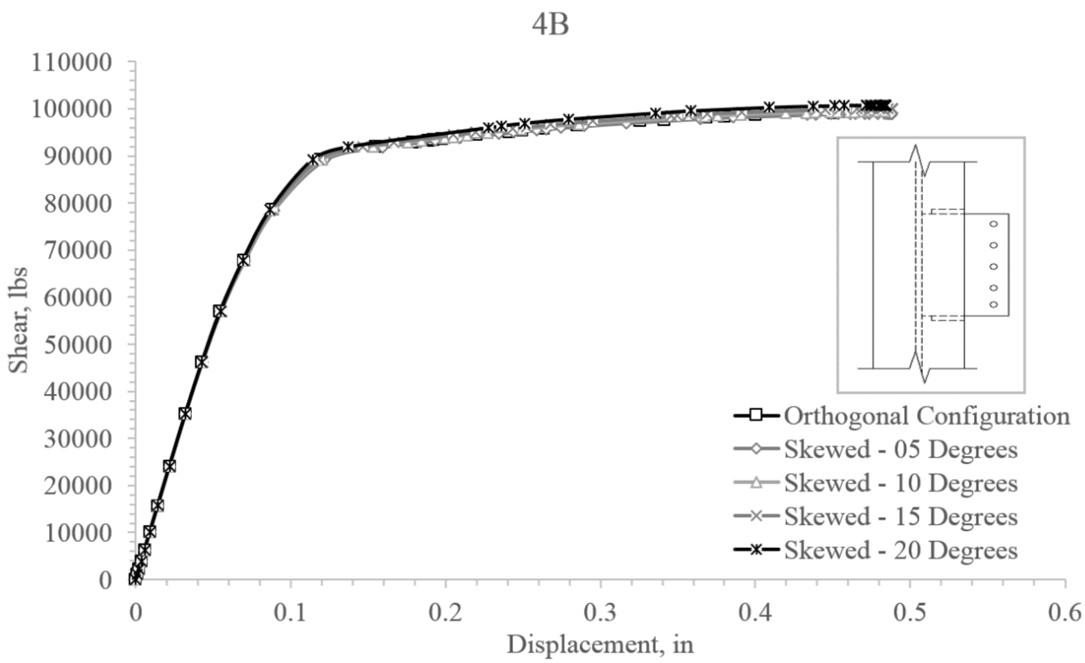
A3.1. Shear-Vertical Displacement Curves

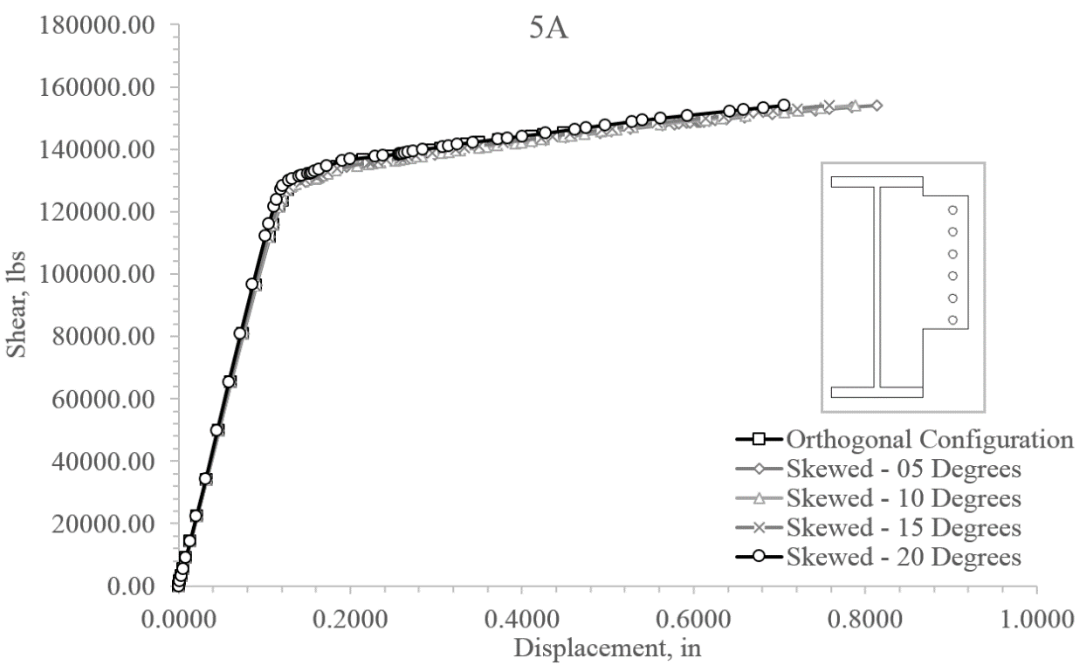


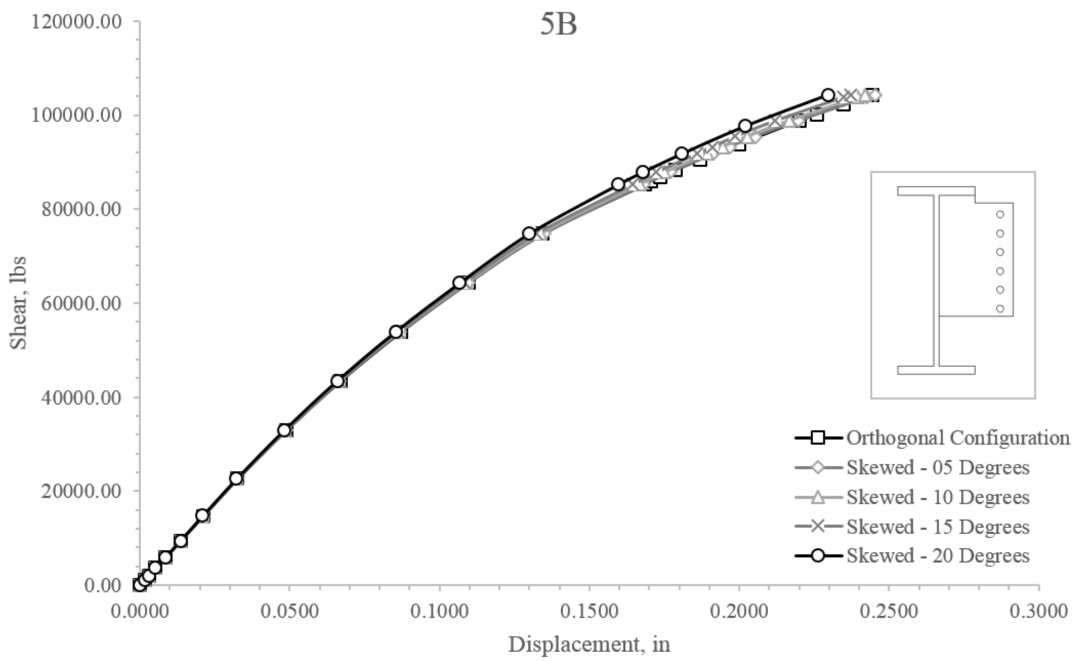



3F
Shear, lb
Displacement, in
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

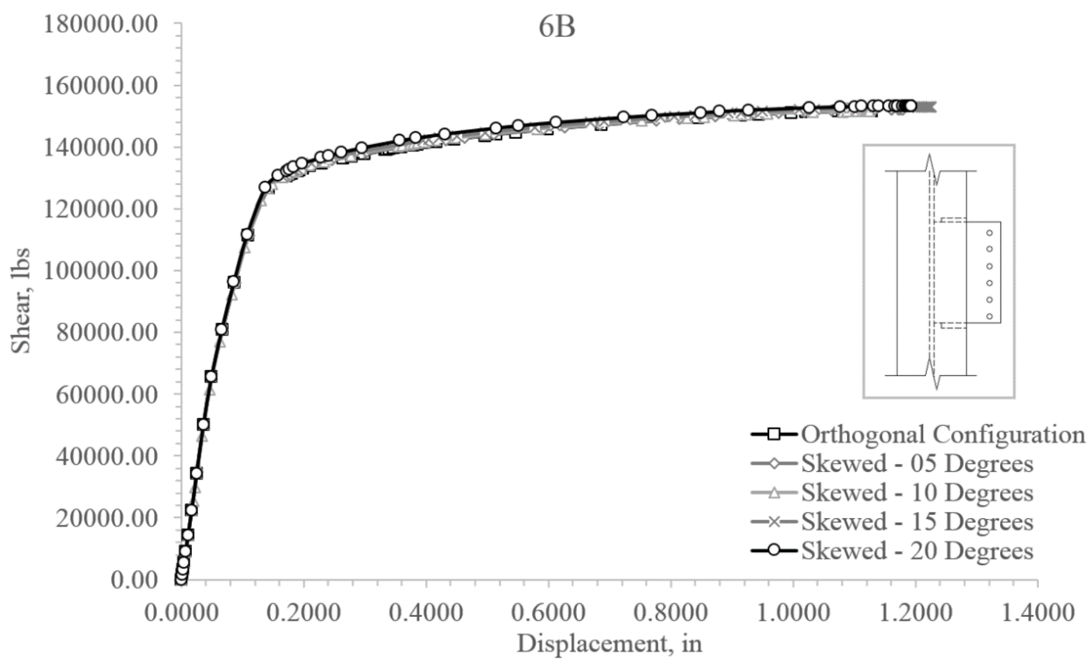

4A
Shear, lb
Displacement, in
Orthogonal Configration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees


4B
Shear, lbs
Displacement, in
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

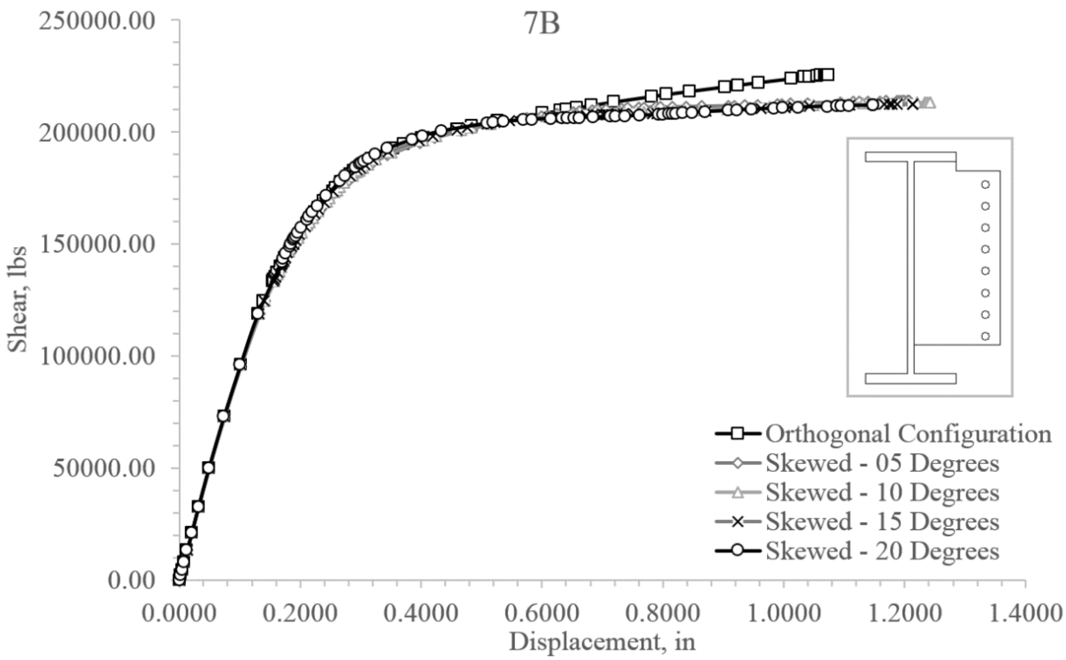

5A
Shear, lbs
Displacement, in
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

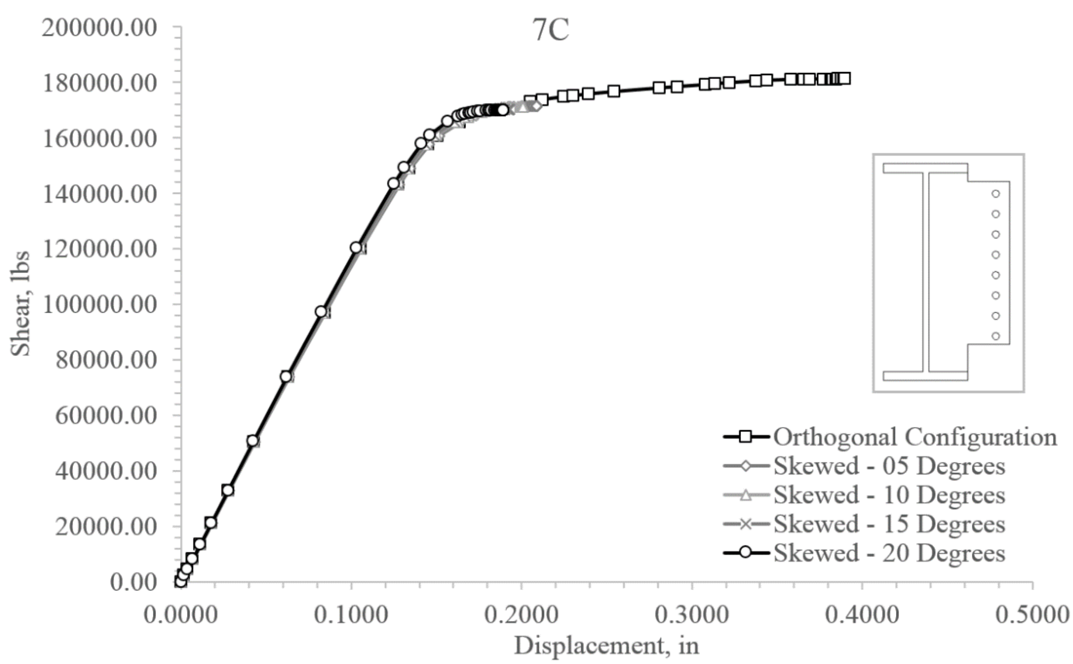


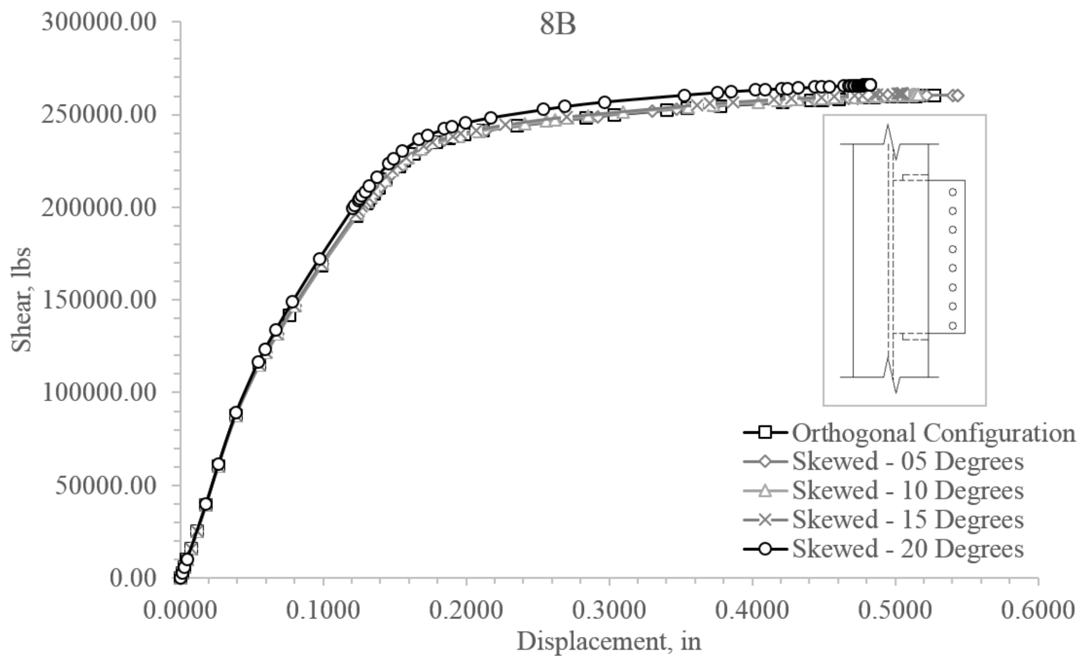
5B
Shear, lbs
Displacement, in
120000.00
100000.00
80000.00
60000.00
40000.00
20000.00
0.00
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees



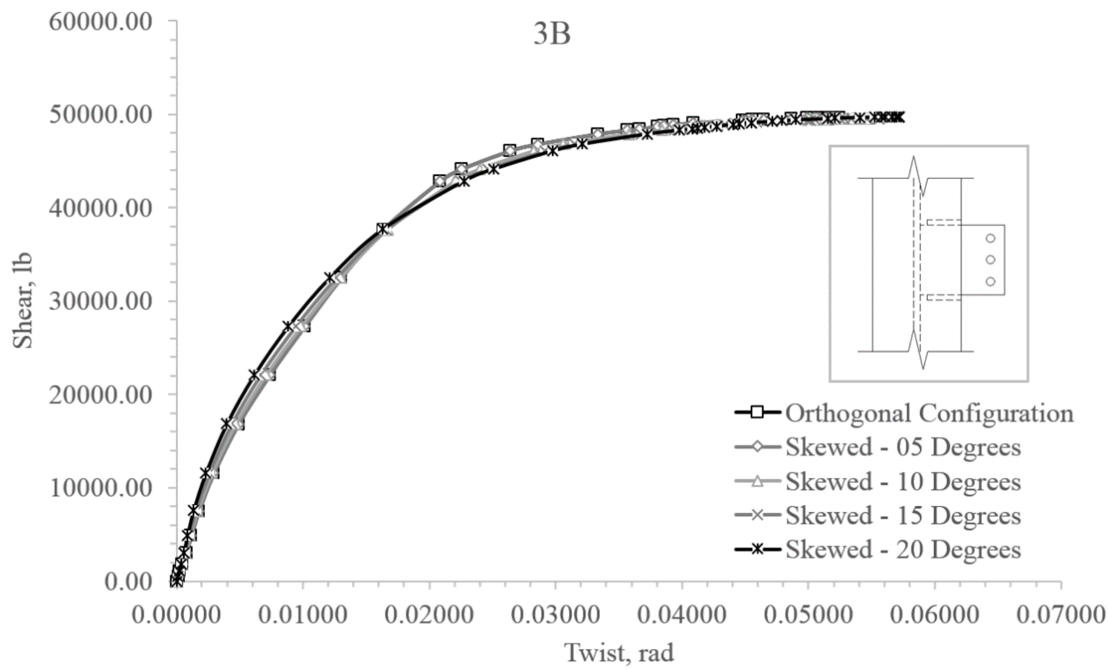
6B
Shear, lbs
Displacement, in
180000.00
160000.00
140000.00
120000.00
100000.00
80000.00
60000.00
40000.00
20000.00
0.00
0.0000
0.2000
0.4000
0.6000
0.8000
1.0000
1.2000
1.4000
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

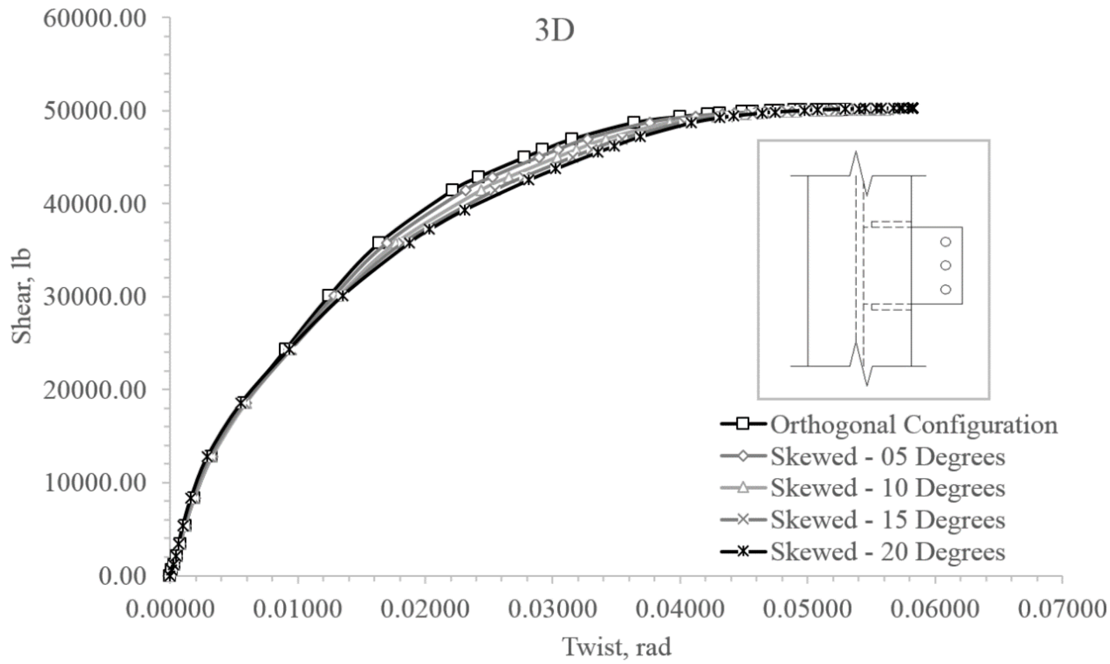

7B
250000.00
200000.00
150000.00
100000.00
50000.00
0.00
Shear, lbs
0.0000
0.2000
0.4000
0.6000
0.8000
1.0000
1.2000
1.4000
Displacement, in
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

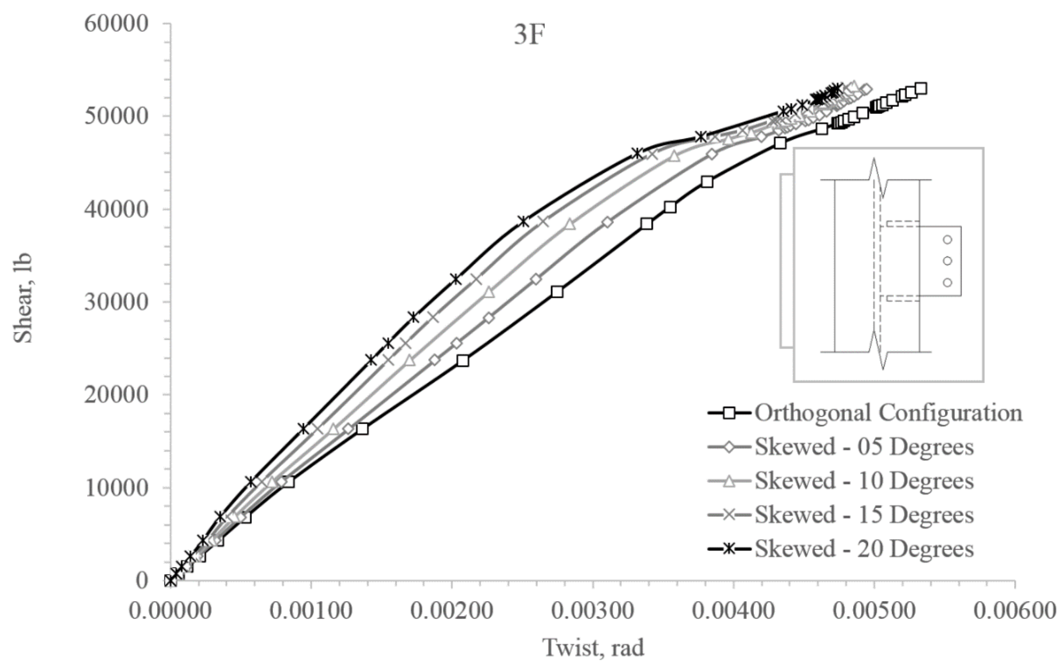

7C
200000.00
180000.00
160000.00
140000.00
120000.00
100000.00
80000.00
60000.00
40000.00
20000.00
0.00
Shear, lbs
0.0000
0.1000
0.2000
0.3000
0.4000
0.5000
Displacement, in
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

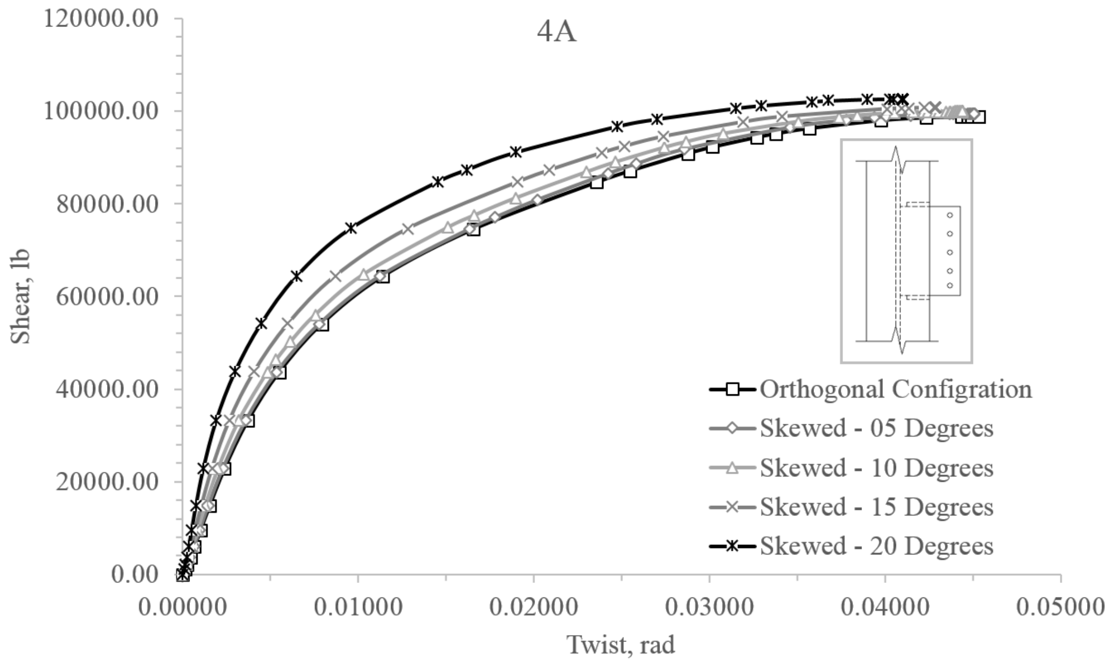


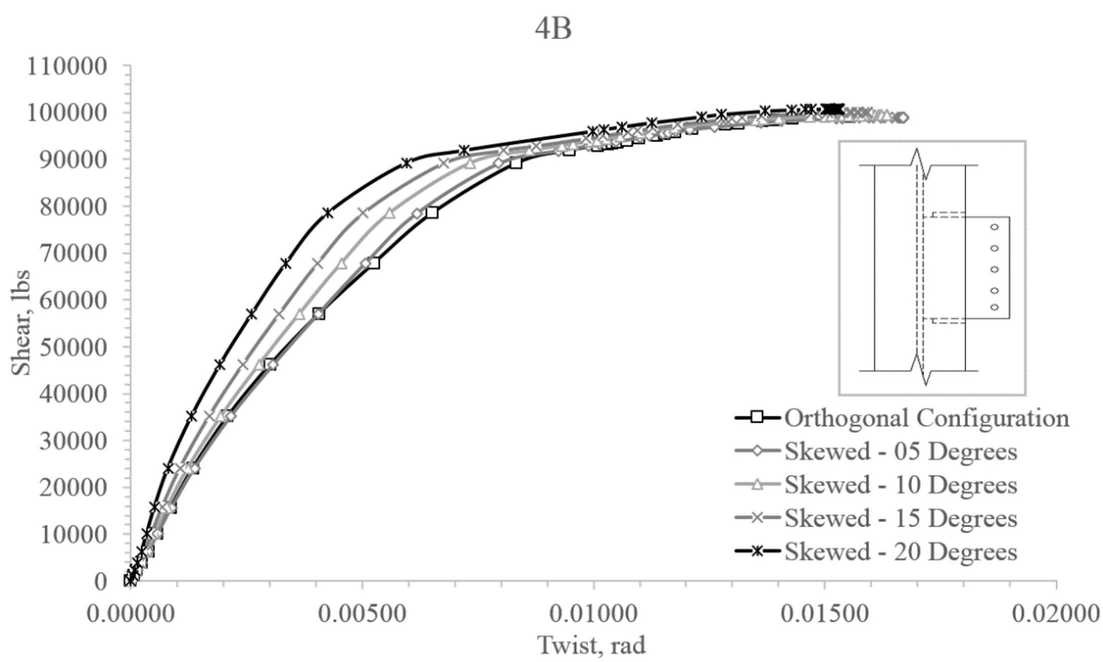
A3.2. Shear-Shear Tab Twist Curves

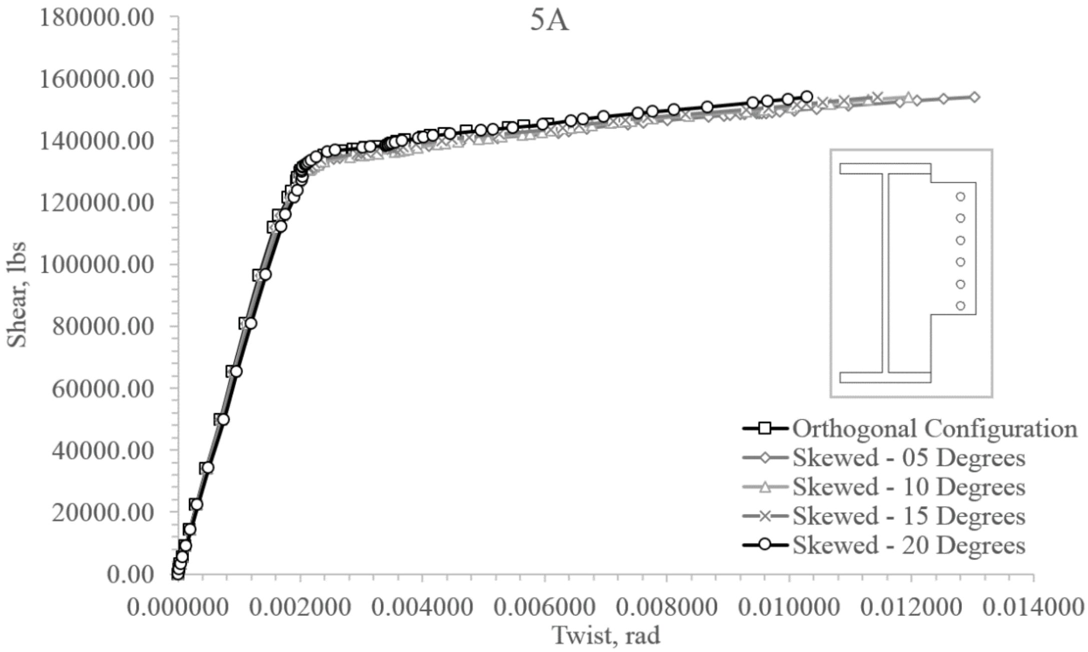


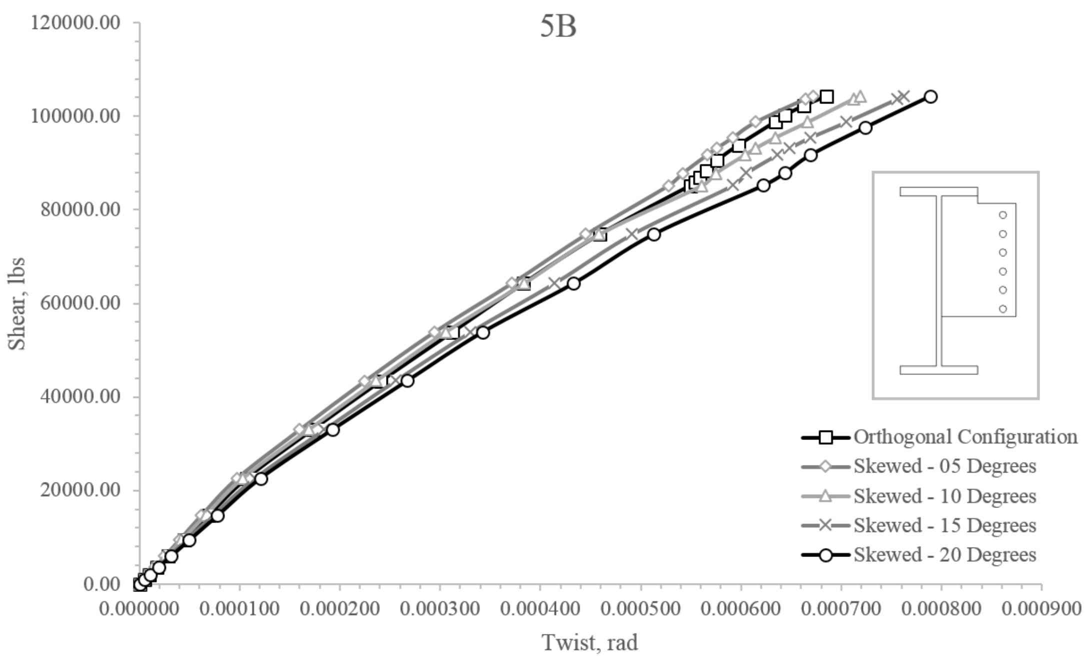

3D
Shear, lb
Twist, rad
60000.00
50000.00
40000.00
30000.00
20000.00
10000.00
0.00
0.00000
0.01000
0.02000
0.03000
0.04000
0.05000
0.06000
0.07000
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

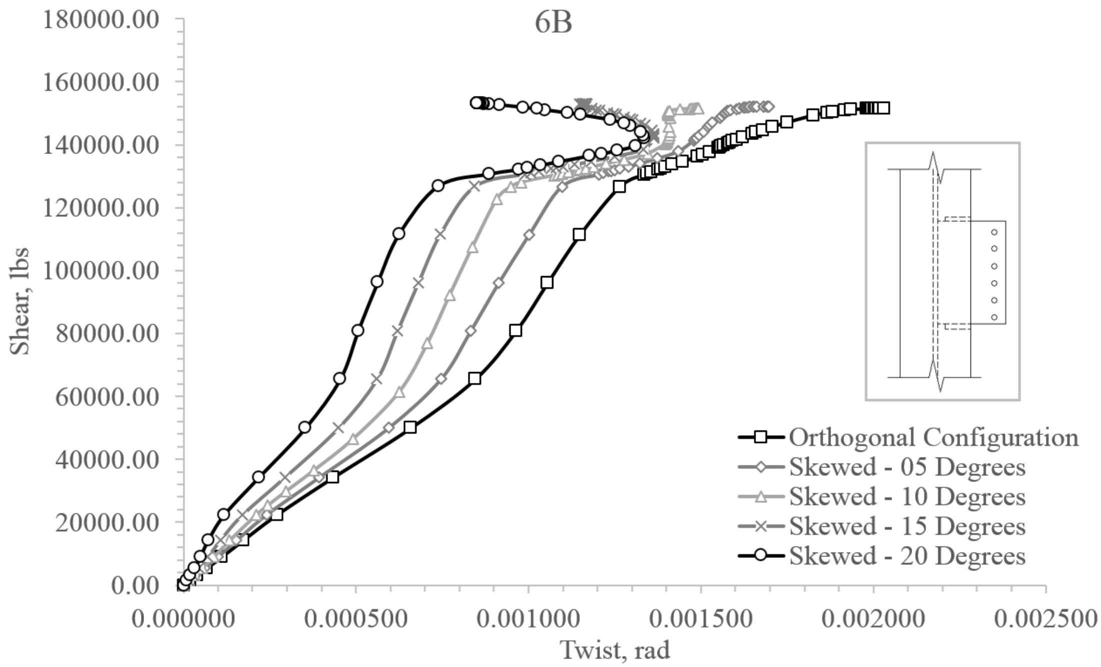

3F
Shear, lb
Twist, rad
60000
50000
40000
30000
20000
10000
0
0.00000
0.00100
0.00200
0.00300
0.00400
0.00500
0.00600
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

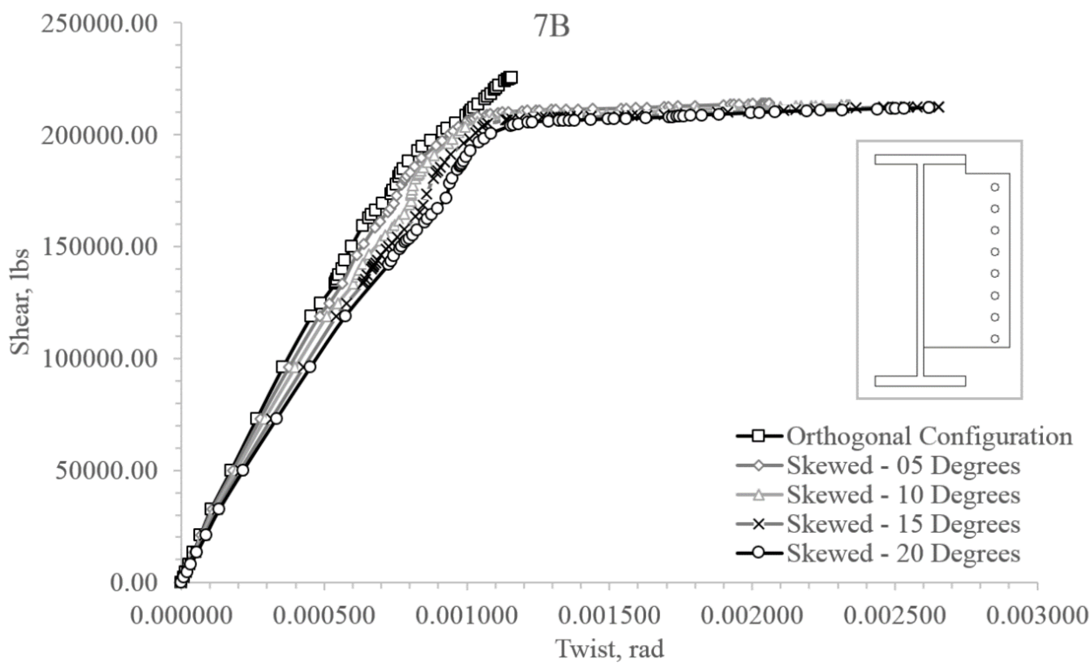

4A
Shear, lb
Twist, rad
120000.00
100000.00
80000.00
60000.00
40000.00
20000.00
0.00
0.00000
0.01000
0.02000
0.03000
0.04000
0.05000
Orthogonal Configration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

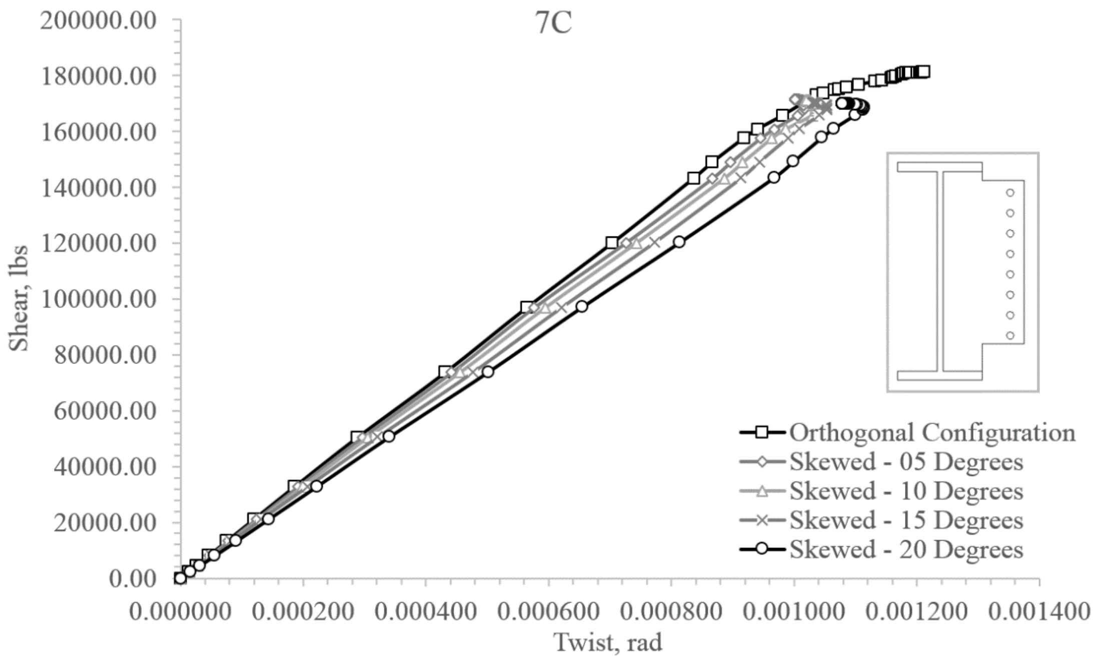

4B
Shear, lbs
Twist, rad
110000
100000
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
0.00000
0.00500
0.01000
0.01500
0.02000
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

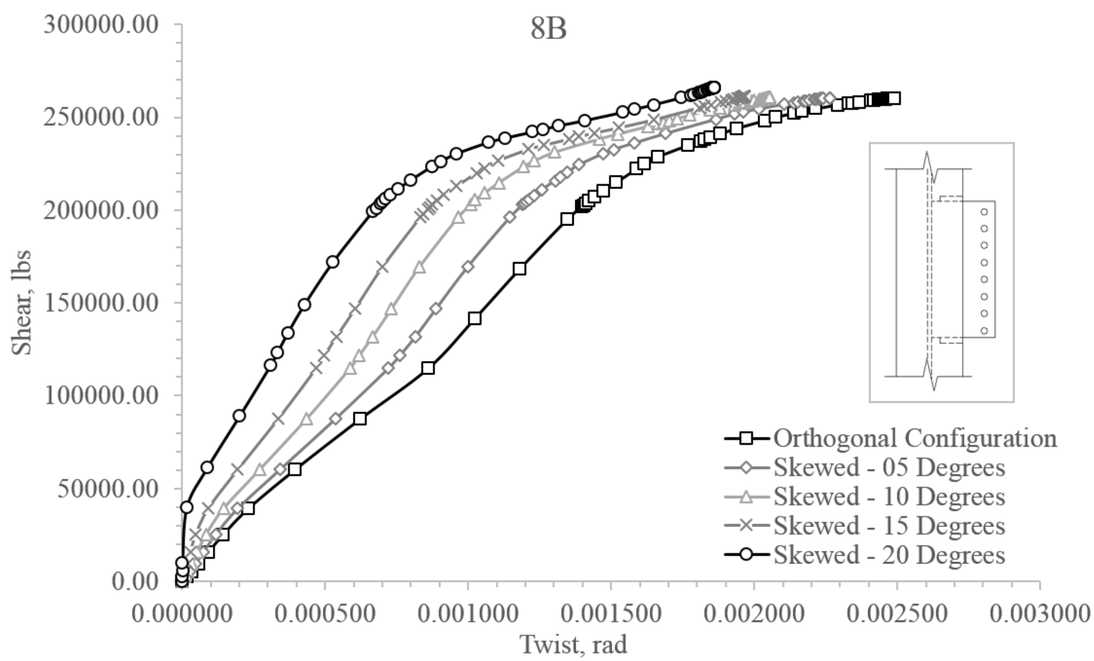

5A
Shear, lbs
Twist, rad
180000.00
160000.00
140000.00
120000.00
100000.00
80000.00
60000.00
40000.00
20000.00
0.00
0.000000
0.002000
0.004000
0.006000
0.008000
0.010000
0.012000
0.014000
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees


5B
Shear, lbs
Twist, rad
120000.00
100000.00
80000.00
60000.00
40000.00
20000.00
0.00
0.000000
0.000100
0.000200
0.000300
0.000400
0.000500
0.000600
0.000700
0.000800
0.000900
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees


6B
Shear, lbs
Twist, rad
180000.00
160000.00
140000.00
120000.00
100000.00
80000.00
60000.00
40000.00
20000.00
0.00
0.000000
0.000500
0.001000
0.001500
0.002000
0.002500
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees


7B
Shear, lbs
Twist, rad
250000.00
200000.00
150000.00
100000.00
50000.00
0.00
0.000000
0.000500
0.001000
0.001500
0.002000
0.002500
0.003000
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

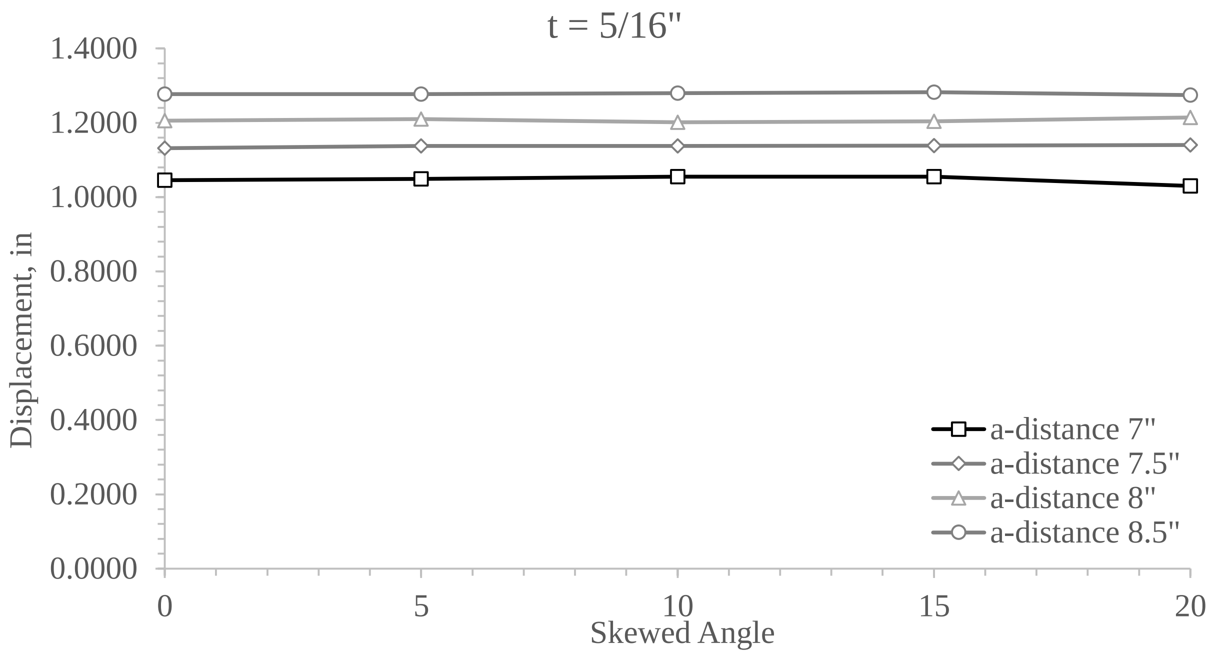

7C
Shear, lbs
200000.00
180000.00
160000.00
140000.00
120000.00
100000.00
80000.00
60000.00
40000.00
20000.00
0.00
0.000000
0.000200
0.000400
0.000600
0.000800
0.001000
0.001200
0.001400
Twist, rad
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

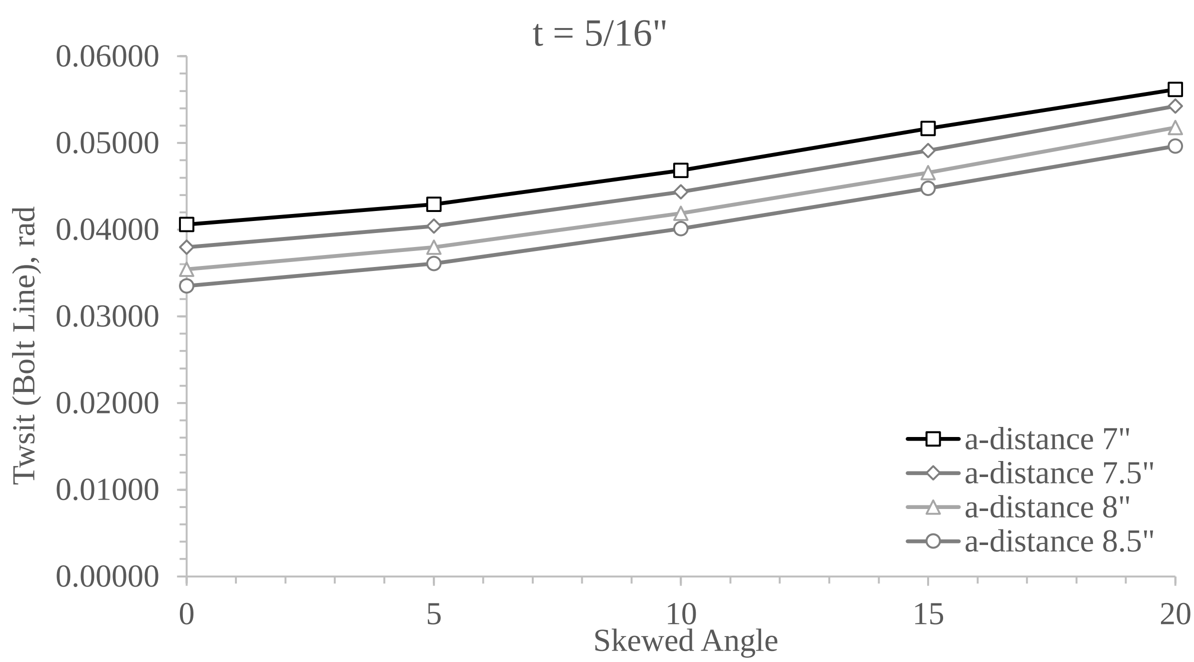

8B
Shear, lbs
300000.00
250000.00
200000.00
150000.00
100000.00
50000.00
0.00
0.000000
0.000500
0.001000
0.001500
0.002000
0.002500
0.003000
Twist, rad
Orthogonal Configuration
Skewed - 05 Degrees
Skewed - 10 Degrees
Skewed - 15 Degrees
Skewed - 20 Degrees

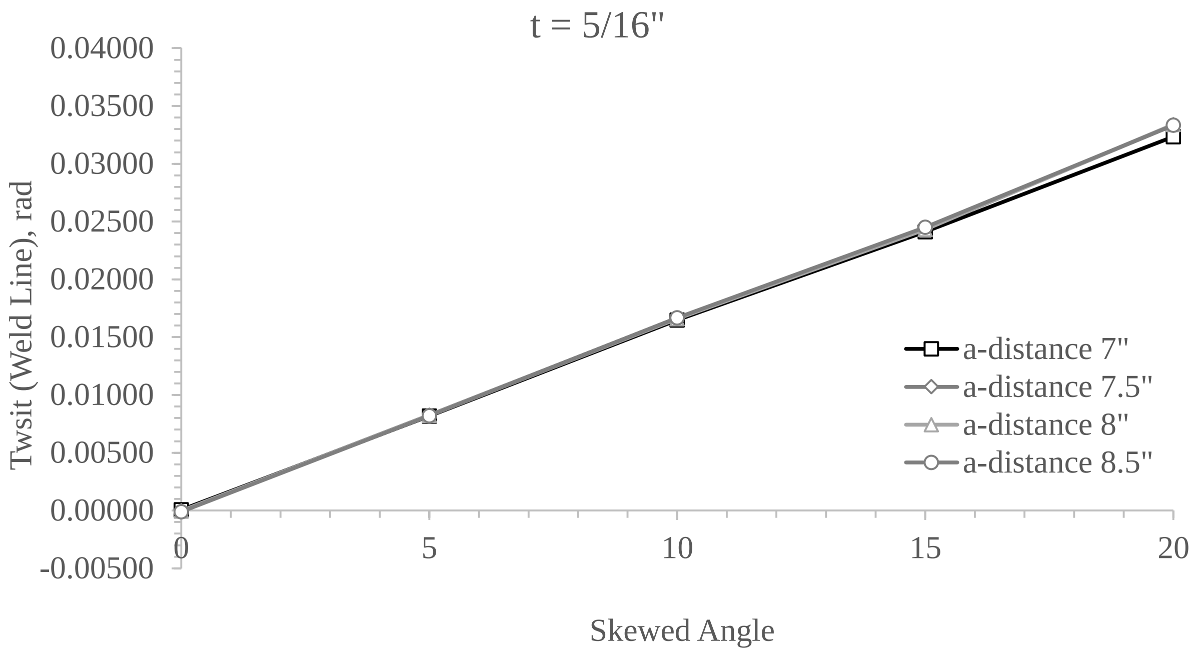
Appendix B (Parametric Study Curves – Chapter 6)

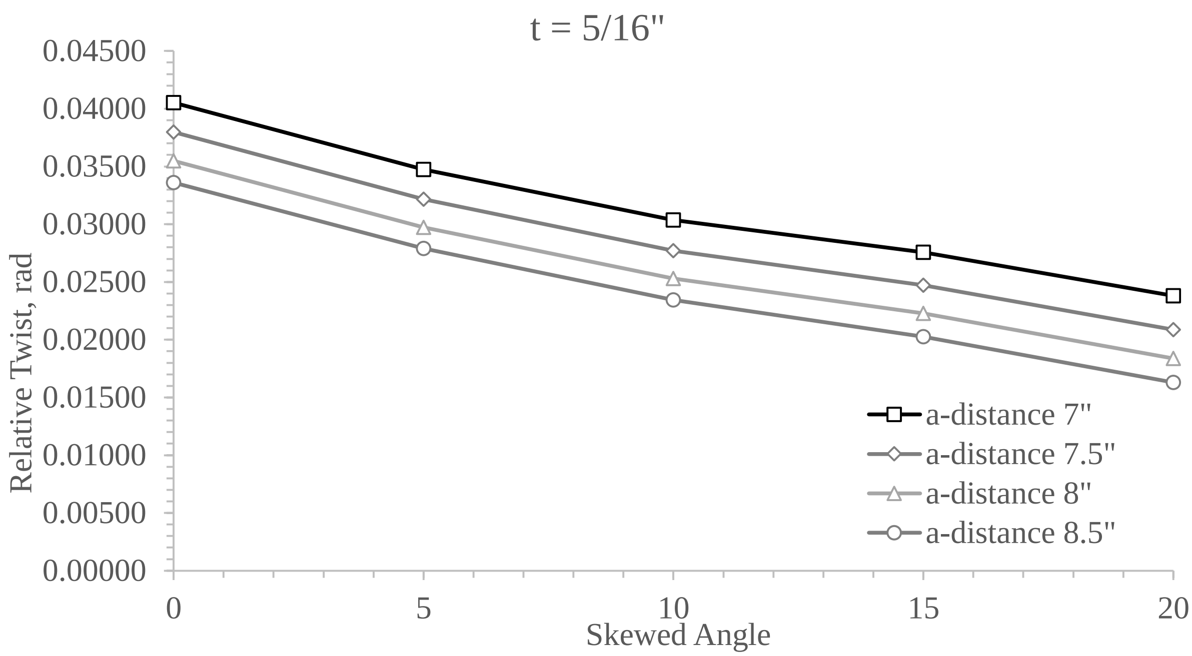
B1. Three Bolted Connections

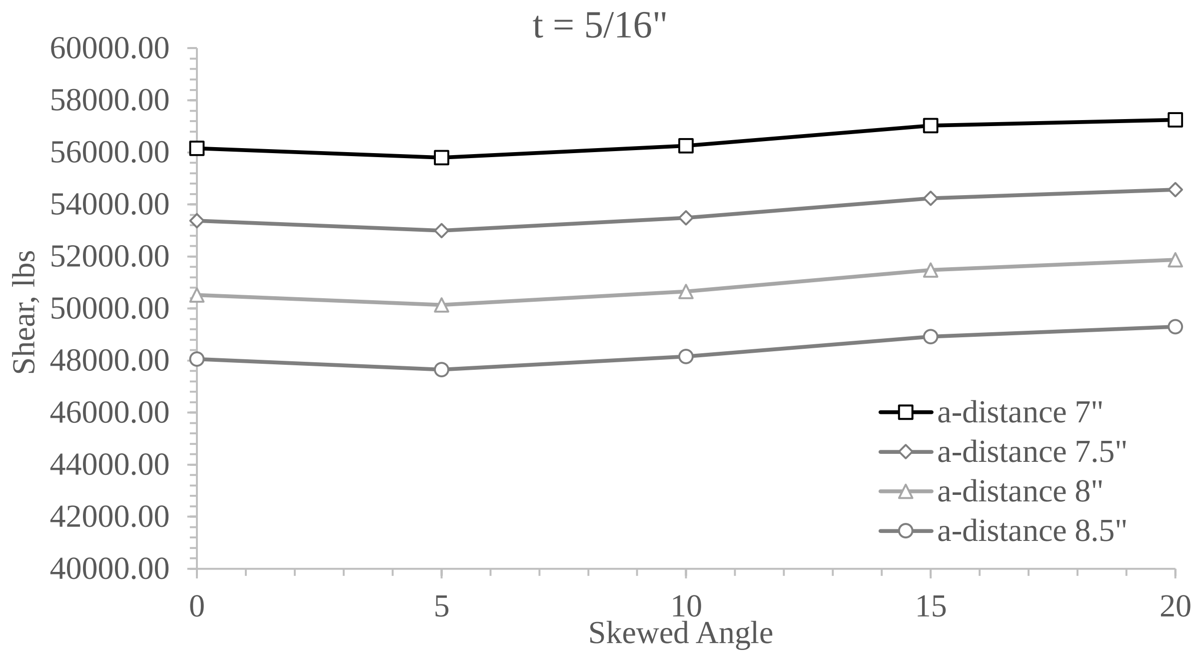
<u>Three Bolted Connections (t = 5/16") Curves</u>






t = 5/16"
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
-0.00500
Twsit (Weld Line), rad
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"


t = 5/16"
0.04500
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
Relative Twist, rad
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"



t = 5/16"
60000.00
58000.00
56000.00
54000.00
52000.00
50000.00
48000.00
46000.00
44000.00
42000.00
40000.00
Shear, lbs
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"

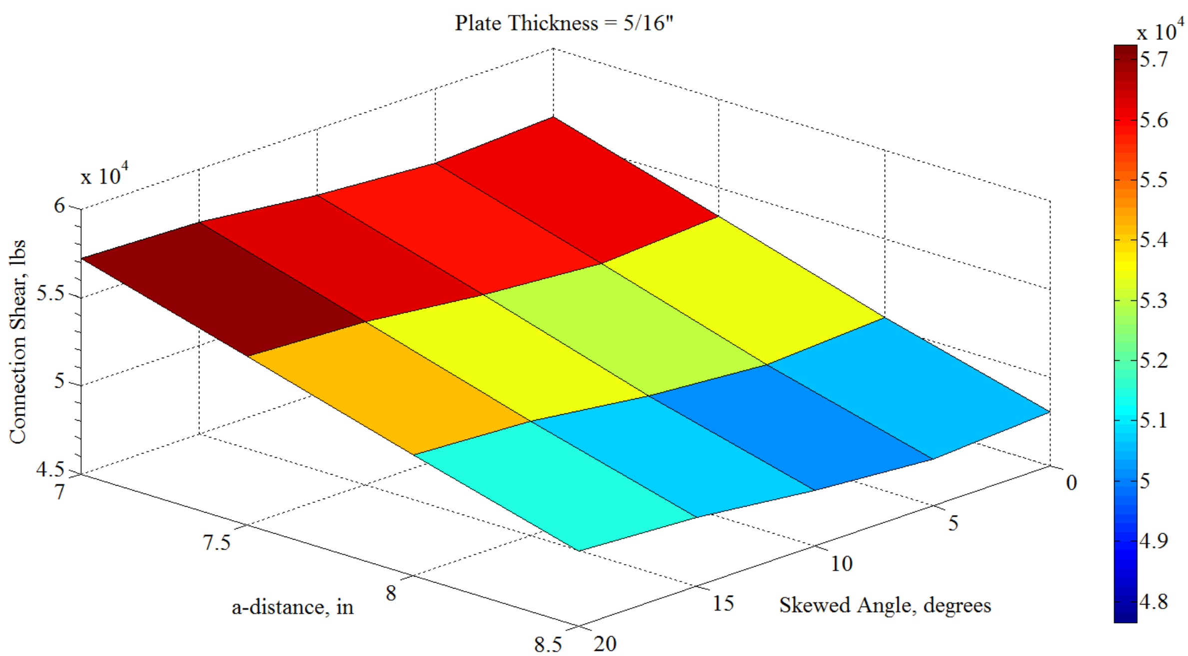
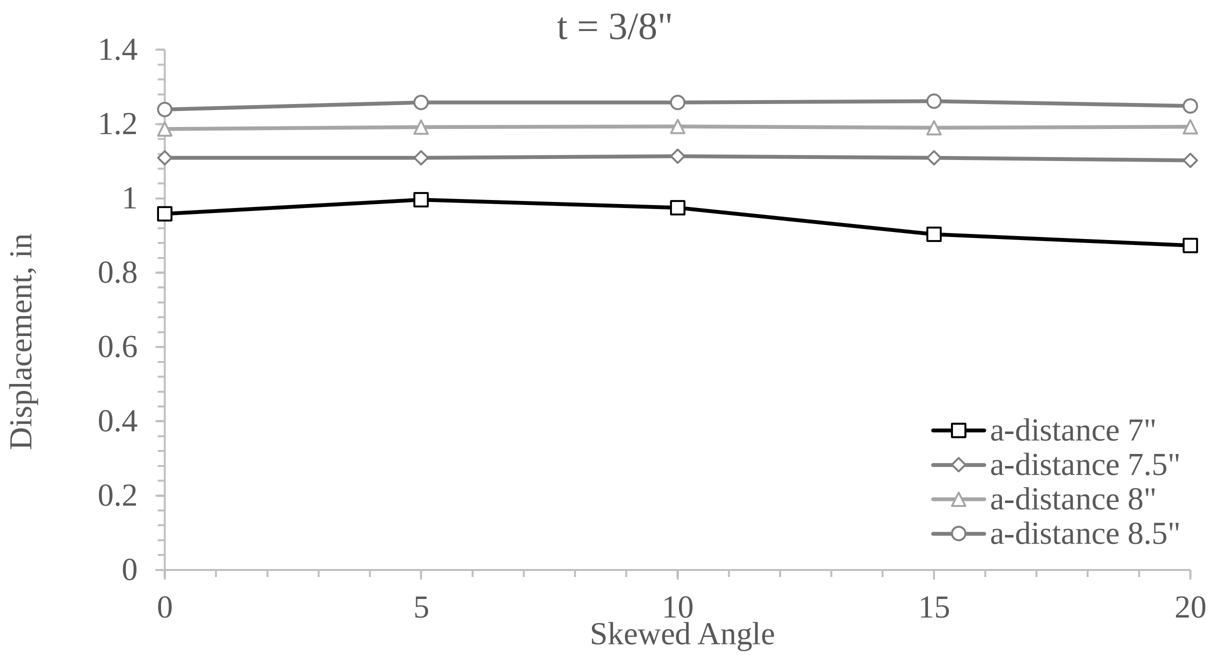
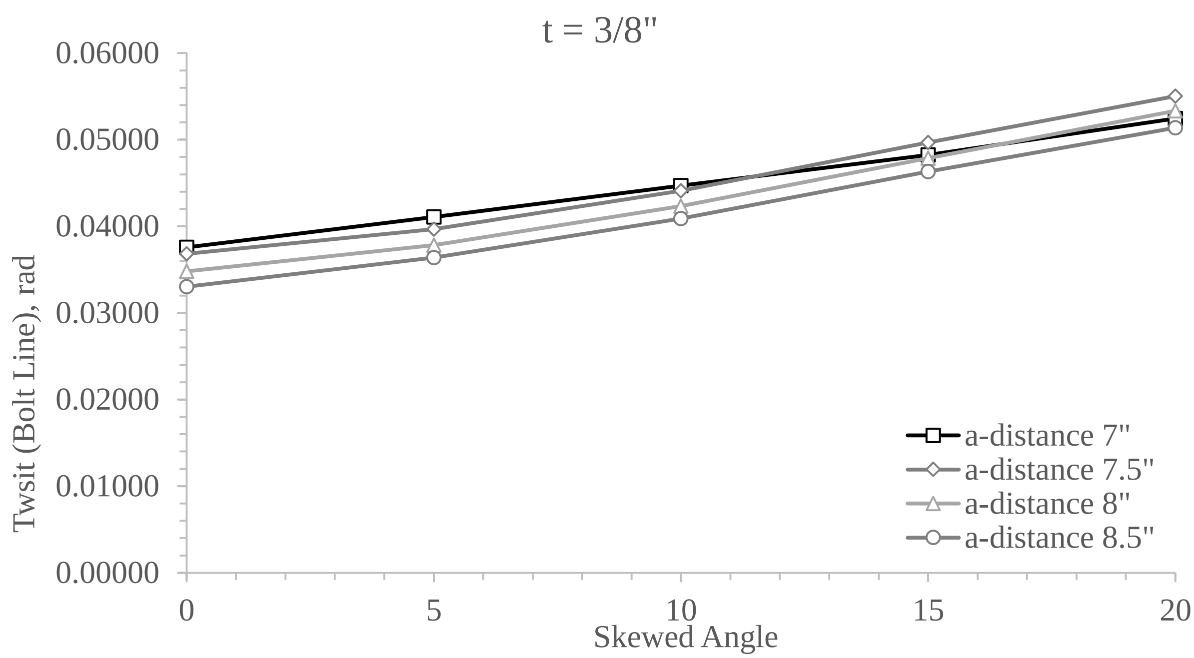
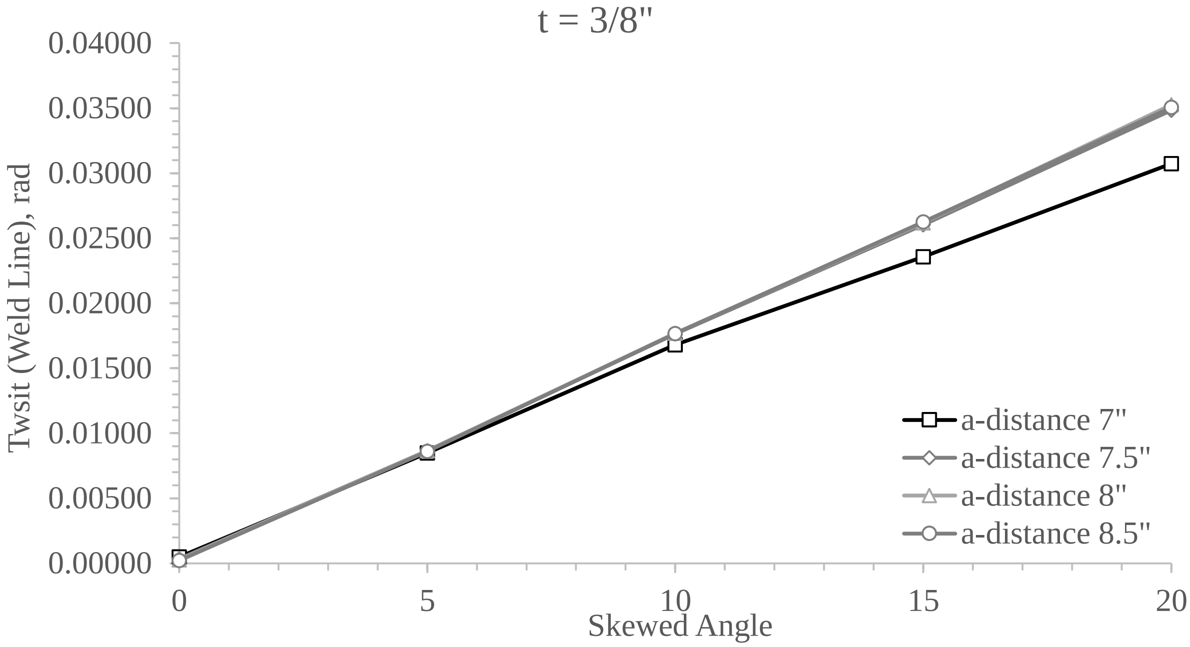


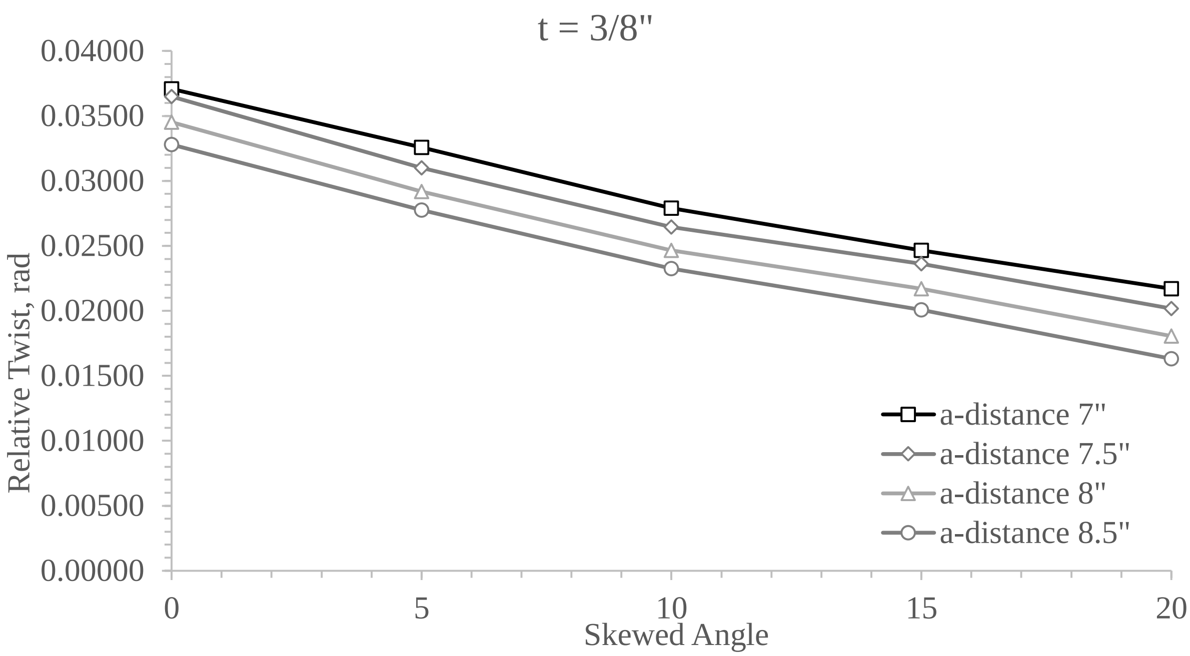
Plate Thickness = 5/16"
x 10^4
6
5.5
5
4.5
Connection Shear, lbs
7
7.5
8
8.5
a-distance, in
0
5
10
15
20
Skewed Angle, degrees
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5
4.9
4.8

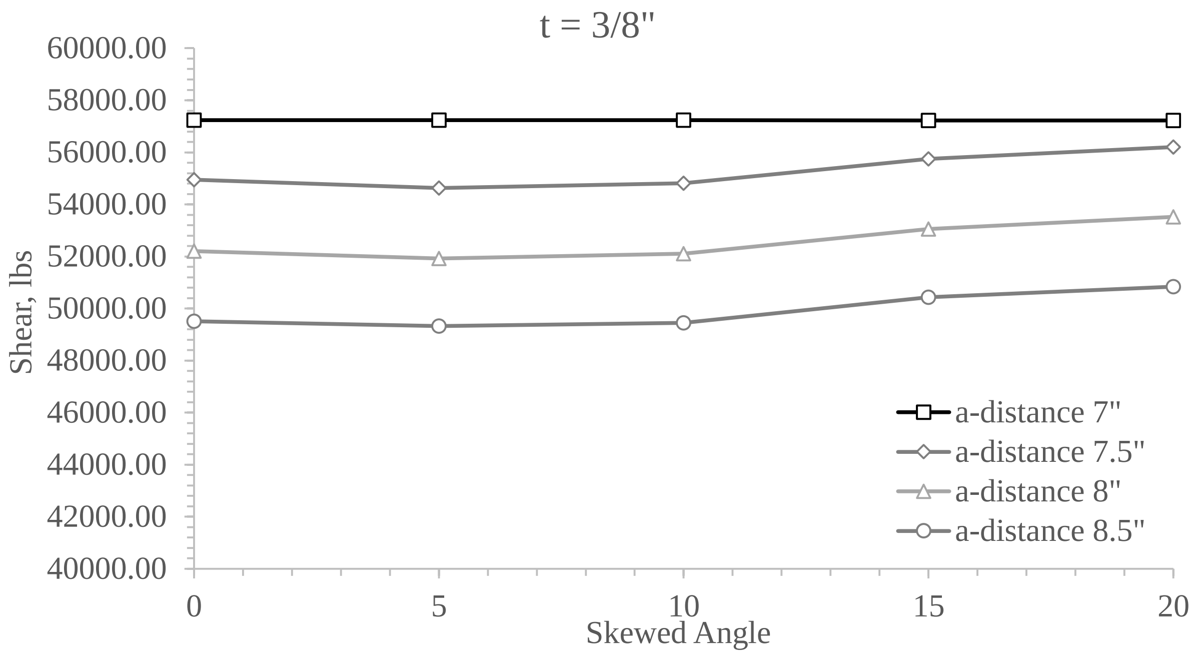
Three Bolted Connections (t = 3/8”) Curves






t = 3/8"
Twsit (Weld Line), rad
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"


t = 3/8"
Relative Twist, rad
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"


t = 3/8"
Shear, lbs
60000.00
58000.00
56000.00
54000.00
52000.00
50000.00
48000.00
46000.00
44000.00
42000.00
40000.00
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"

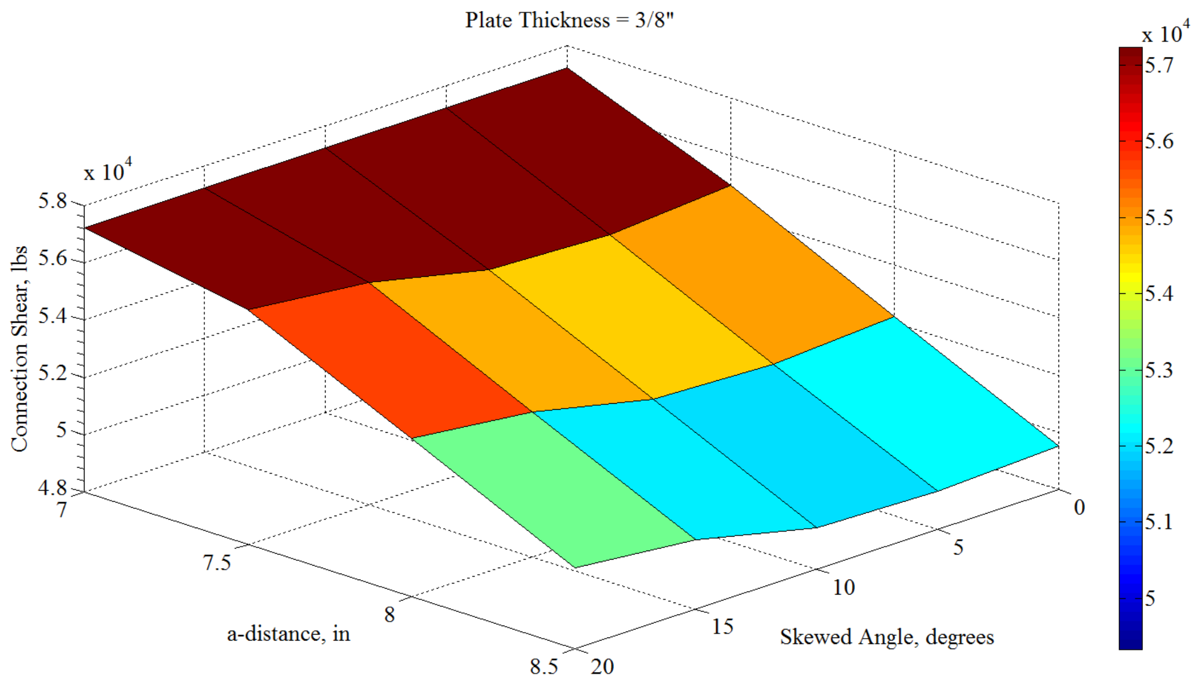
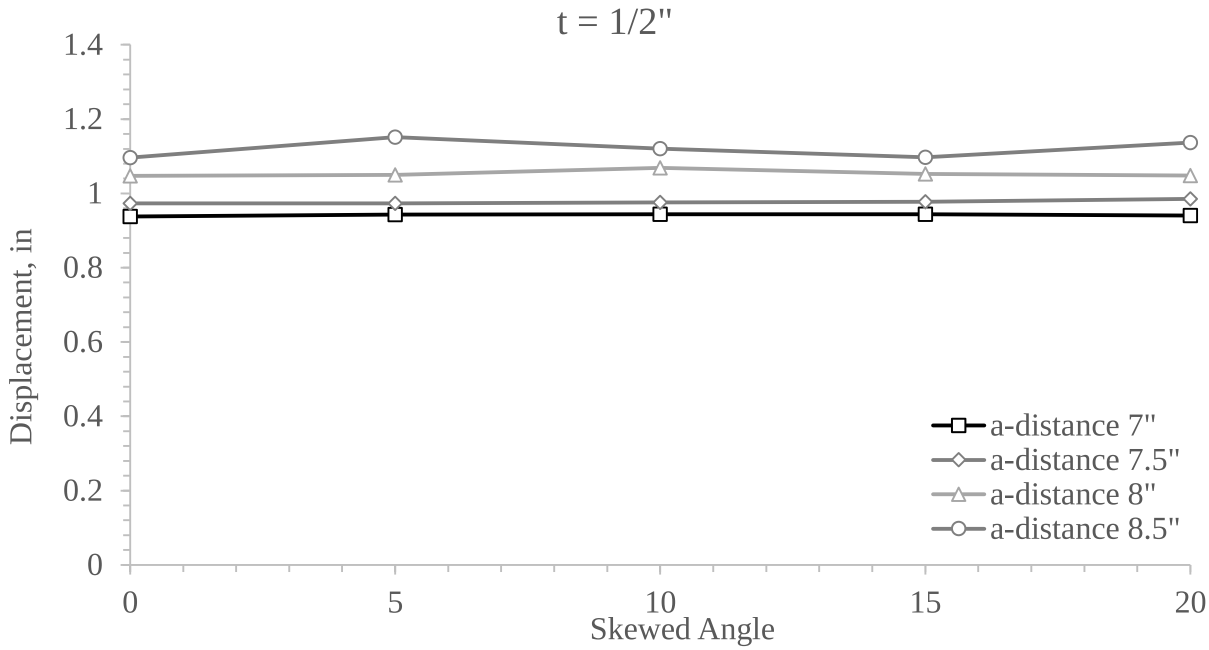
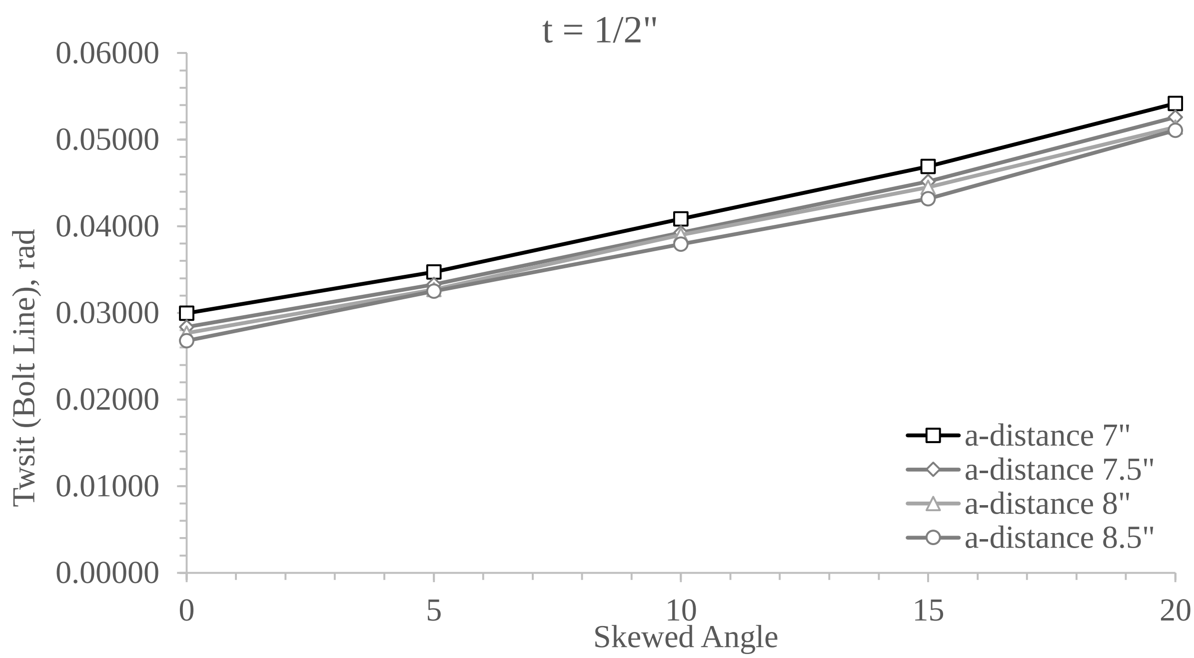
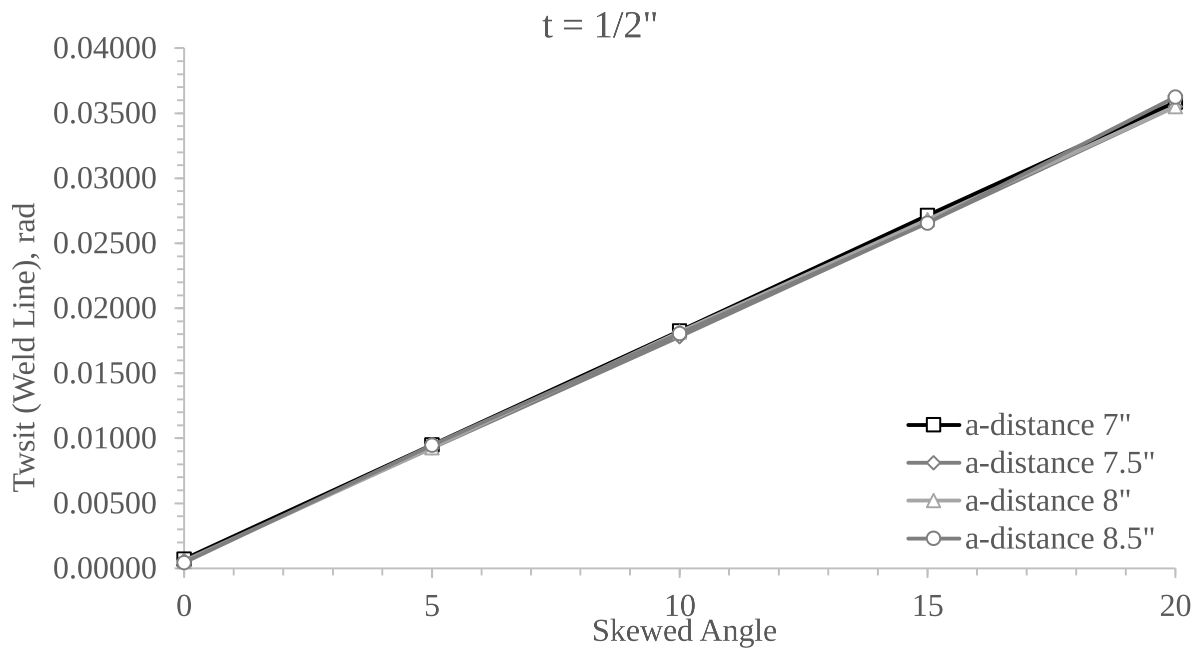


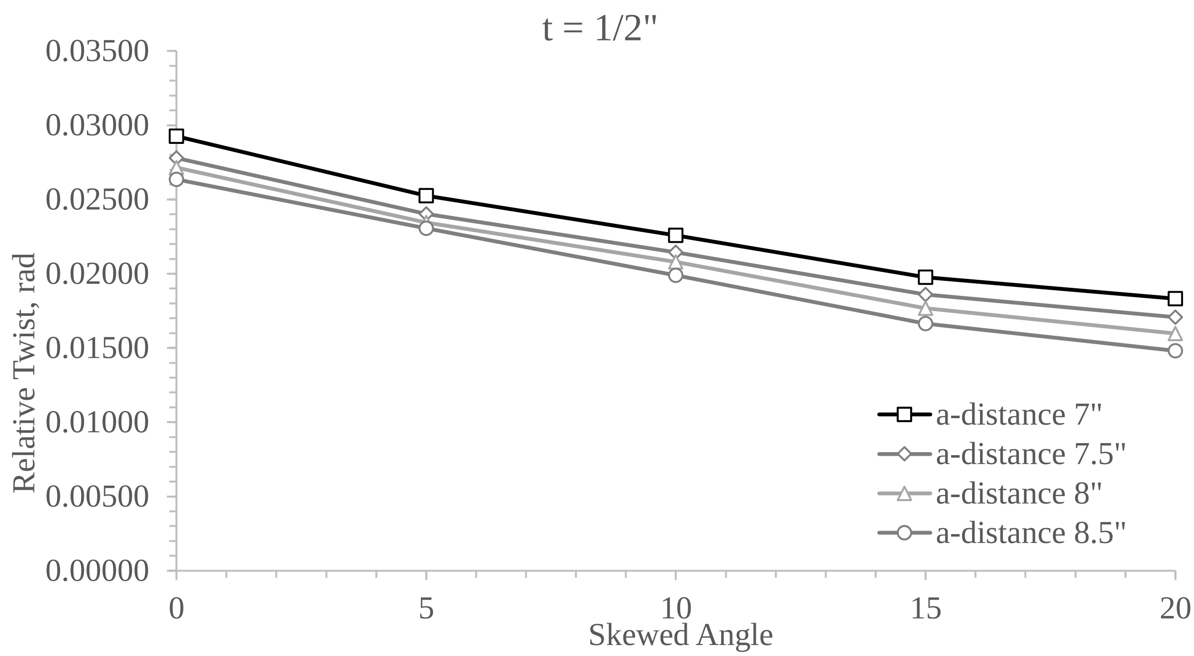
Plate Thickness = 3/8"
x 10^4
Connection Shear, lbs
5.8
5.6
5.4
5.2
5
4.8
7
7.5
8
8.5
a-distance, in
0
5
10
15
20
Skewed Angle, degrees
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5

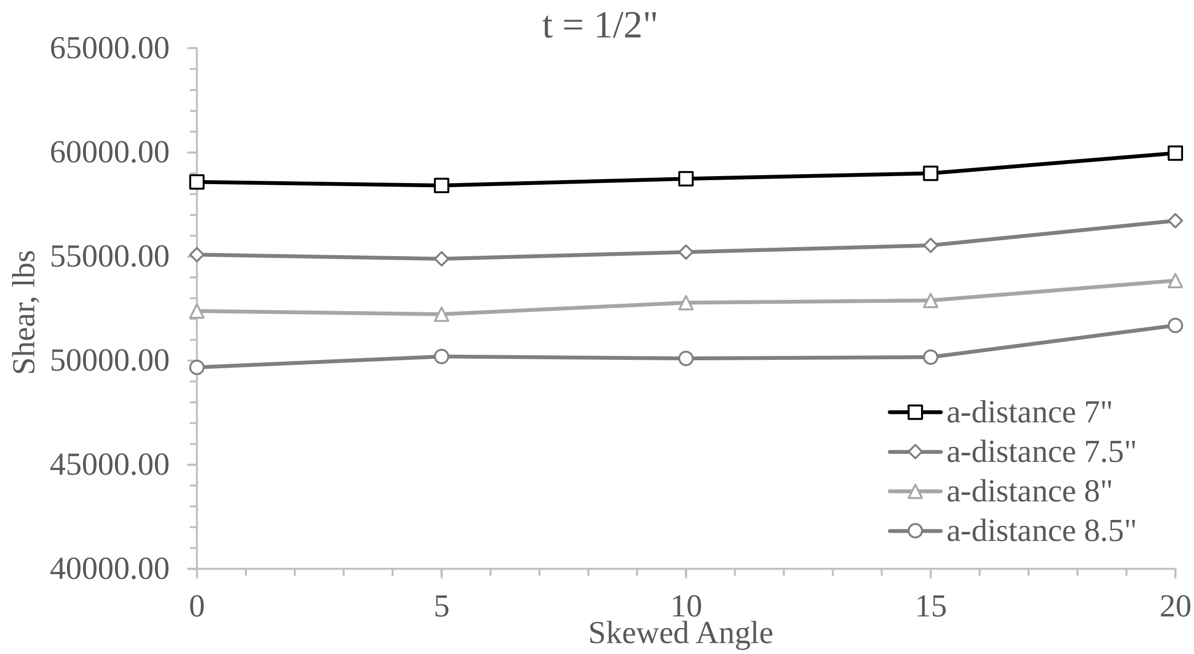
Three Bolted Connections (t = 1/2”) Curves






t = 1/2"
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
Twsit (Weld Line), rad
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"


t = 1/2"
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
Relative Twist, rad
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"


t = 1/2"
65000.00
60000.00
55000.00
50000.00
45000.00
40000.00
Shear, lbs
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"

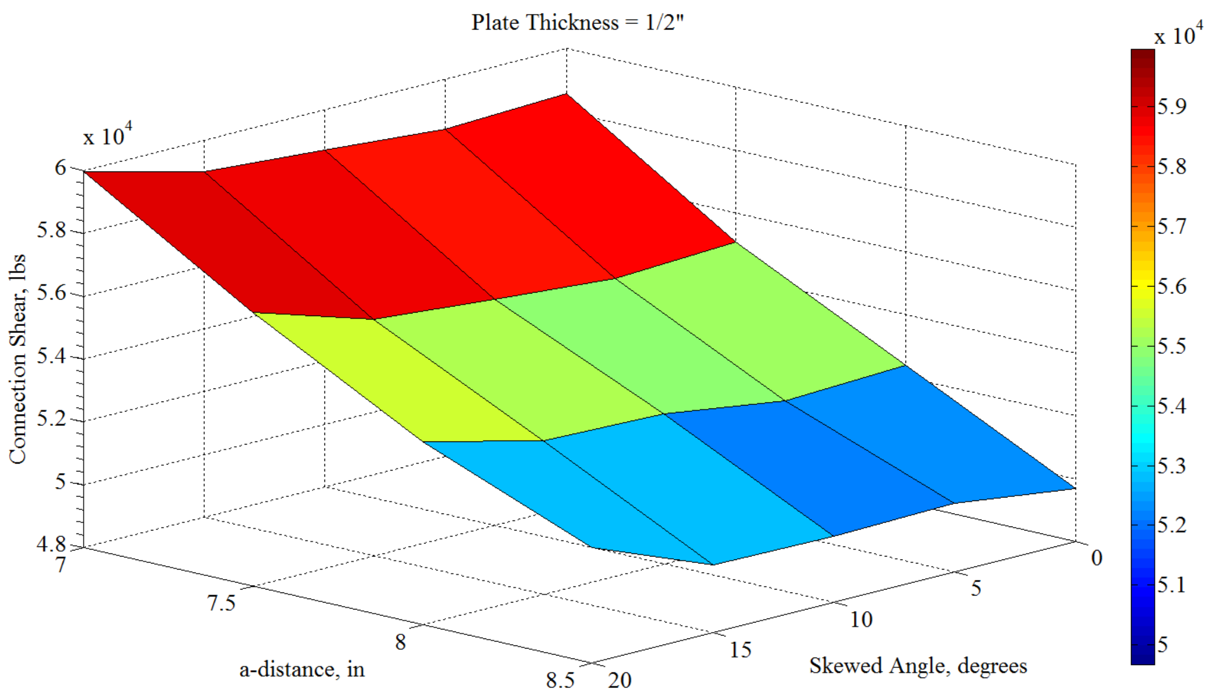
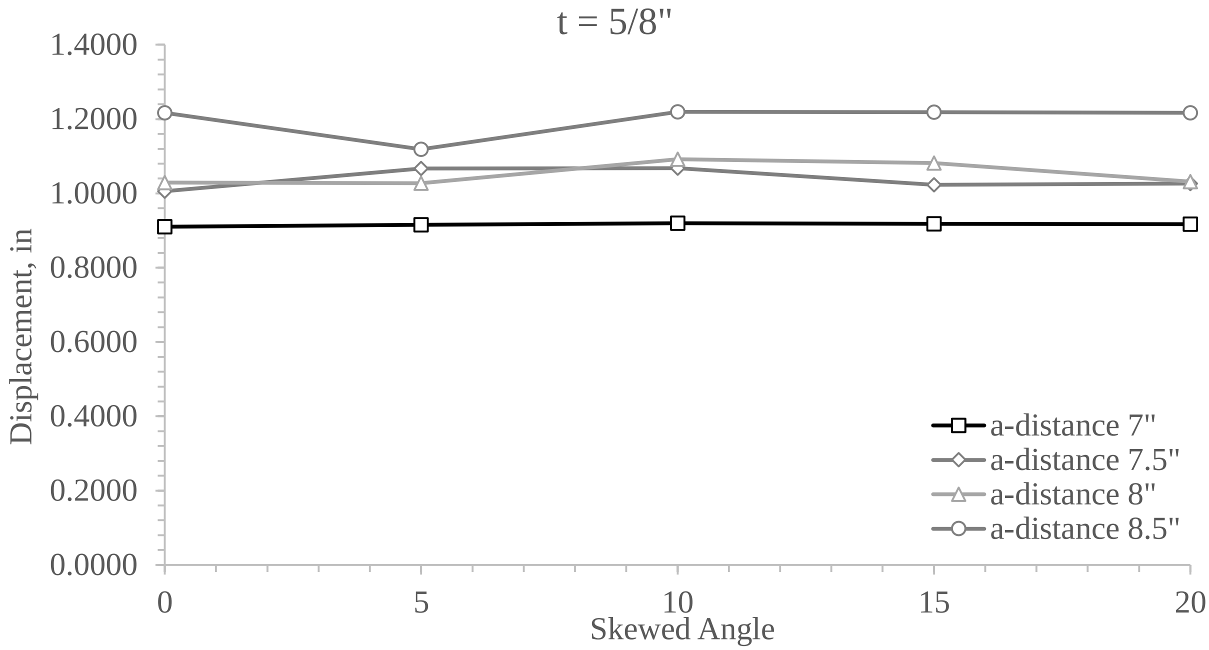
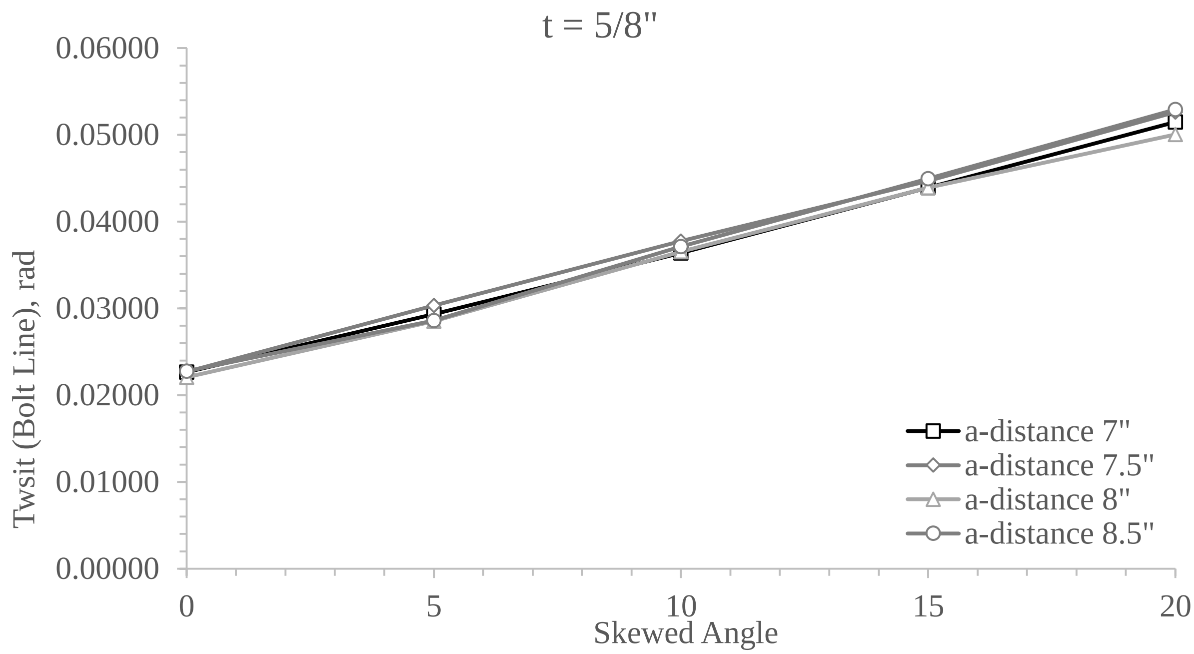
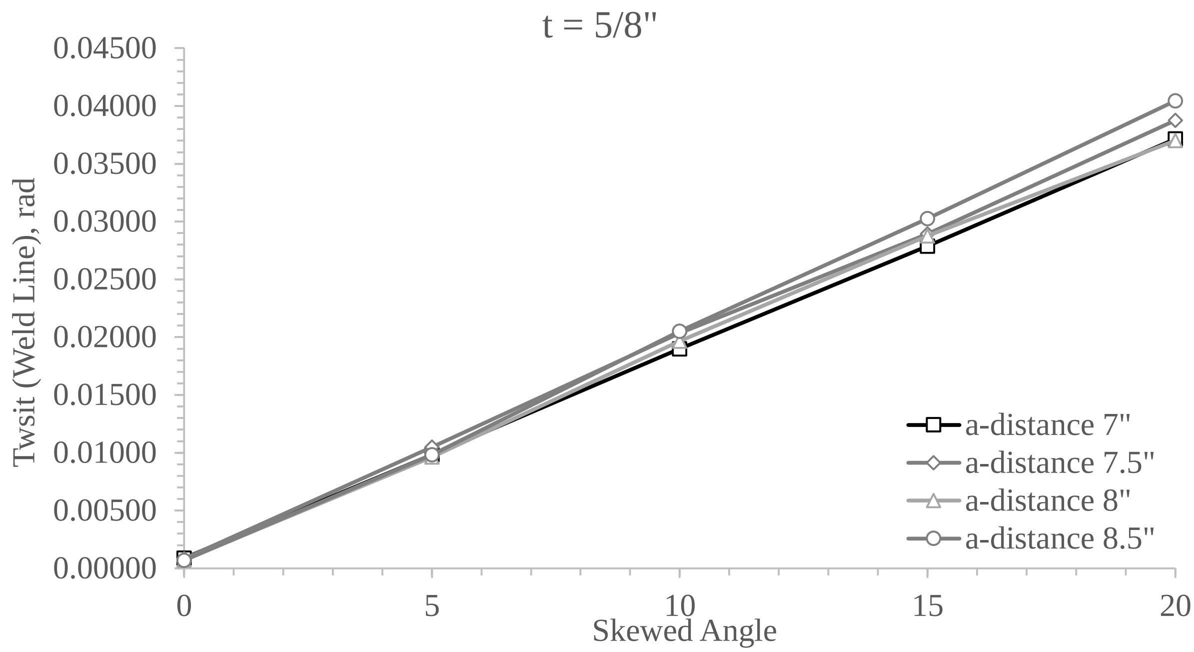


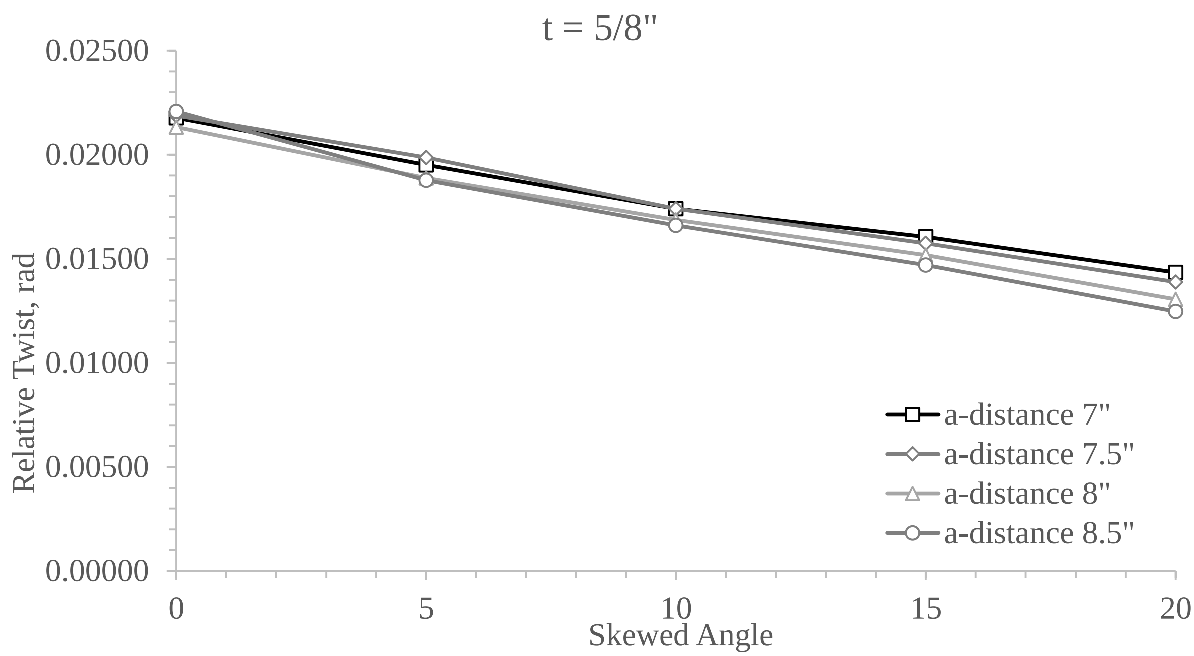
Plate Thickness = 1/2"
x 10^4
6
5.8
5.6
5.4
5.2
5
4.8
Connection Shear, lbs
7
7.5
8
8.5
a-distance, in
0
5
10
15
20
Skewed Angle, degrees
x 10^4
5.9
5.8
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5

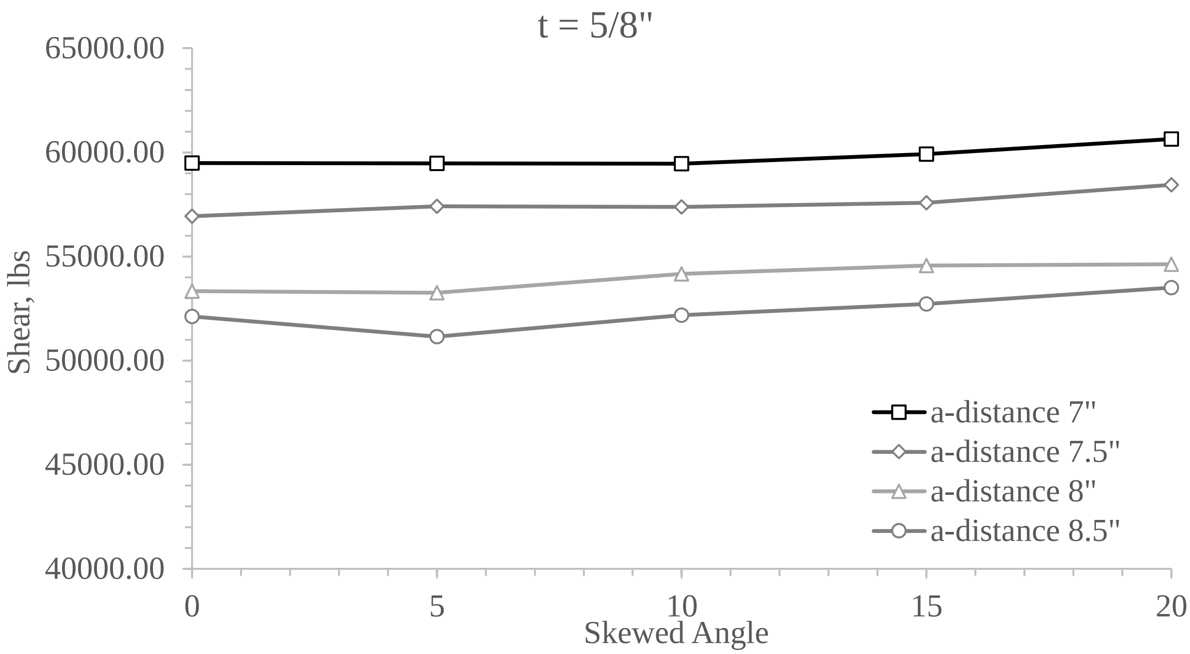
Three Bolted Connections (t = 5/8”) Curves






t = 5/8"
0.04500
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
Twsit (Weld Line), rad
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"


t = 5/8"
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
Relative Twist, rad
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"


t = 5/8"
65000.00
60000.00
55000.00
50000.00
45000.00
40000.00
Shear, lbs
0
5
10
15
20
Skewed Angle
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"

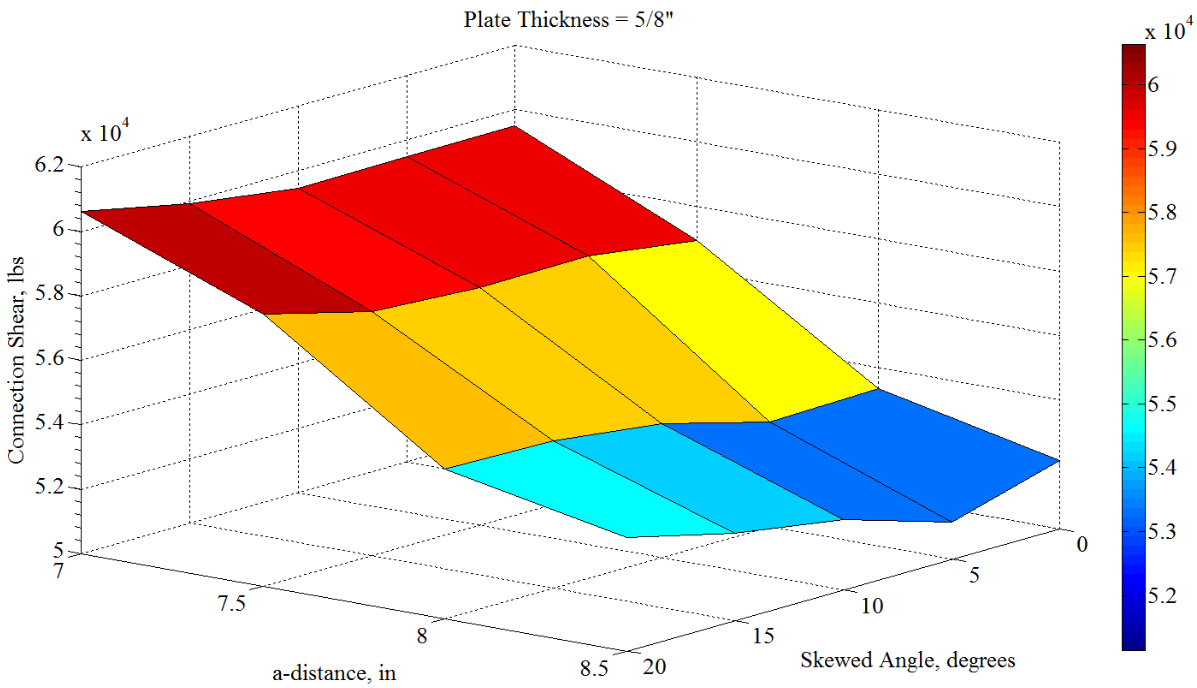
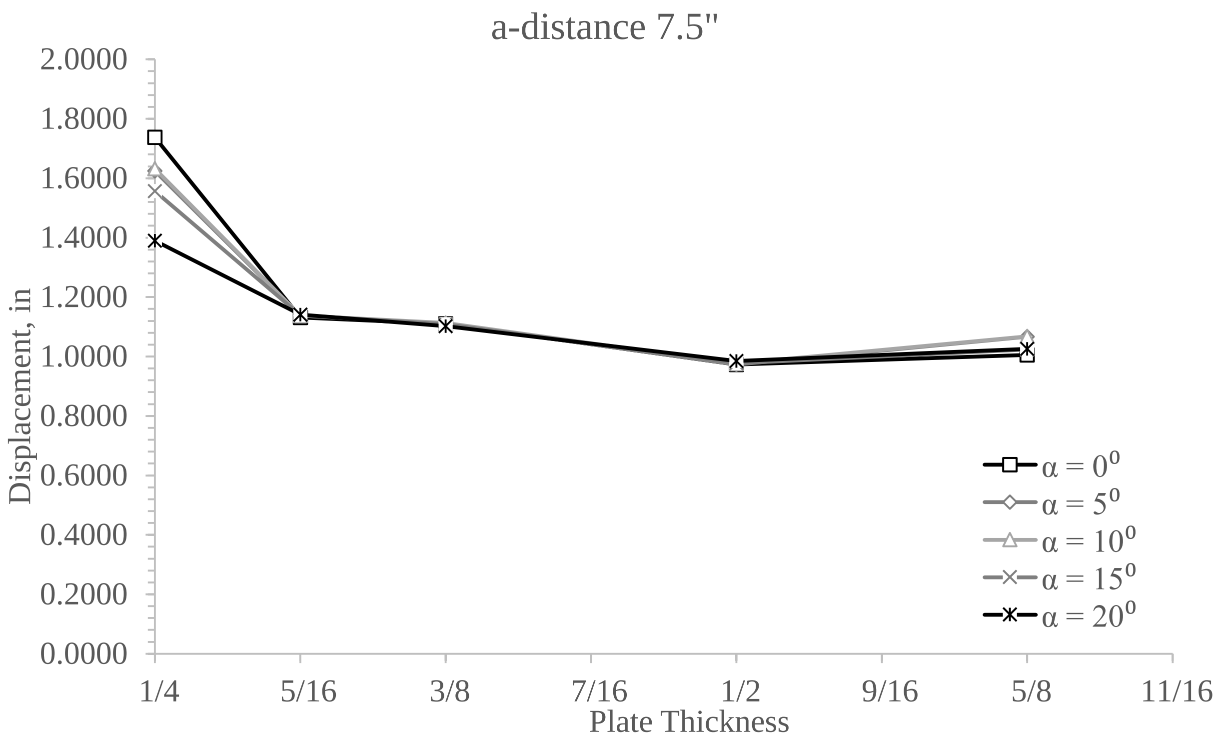
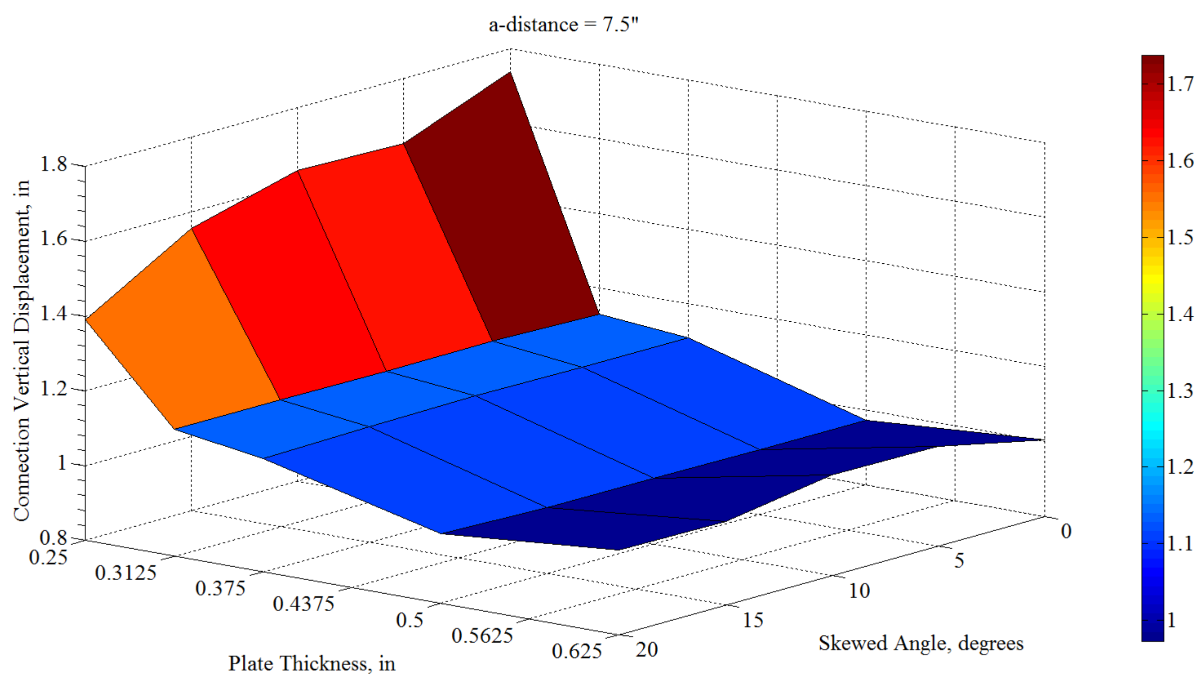
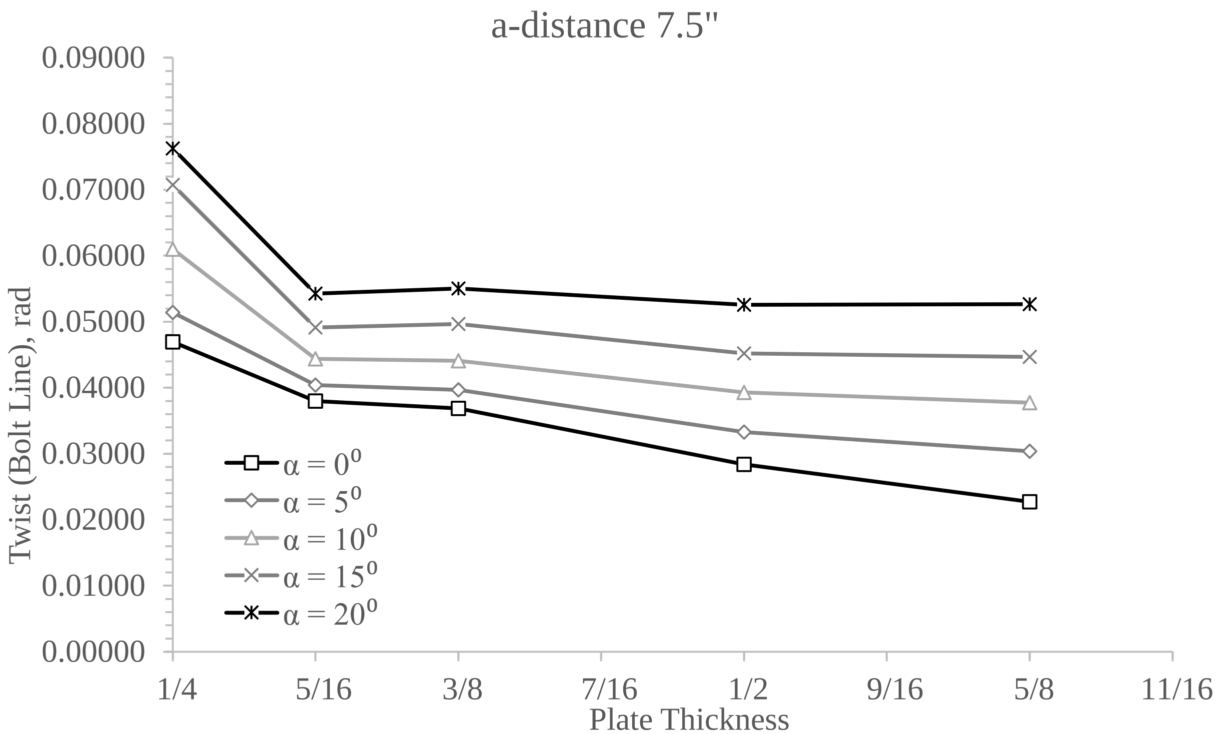


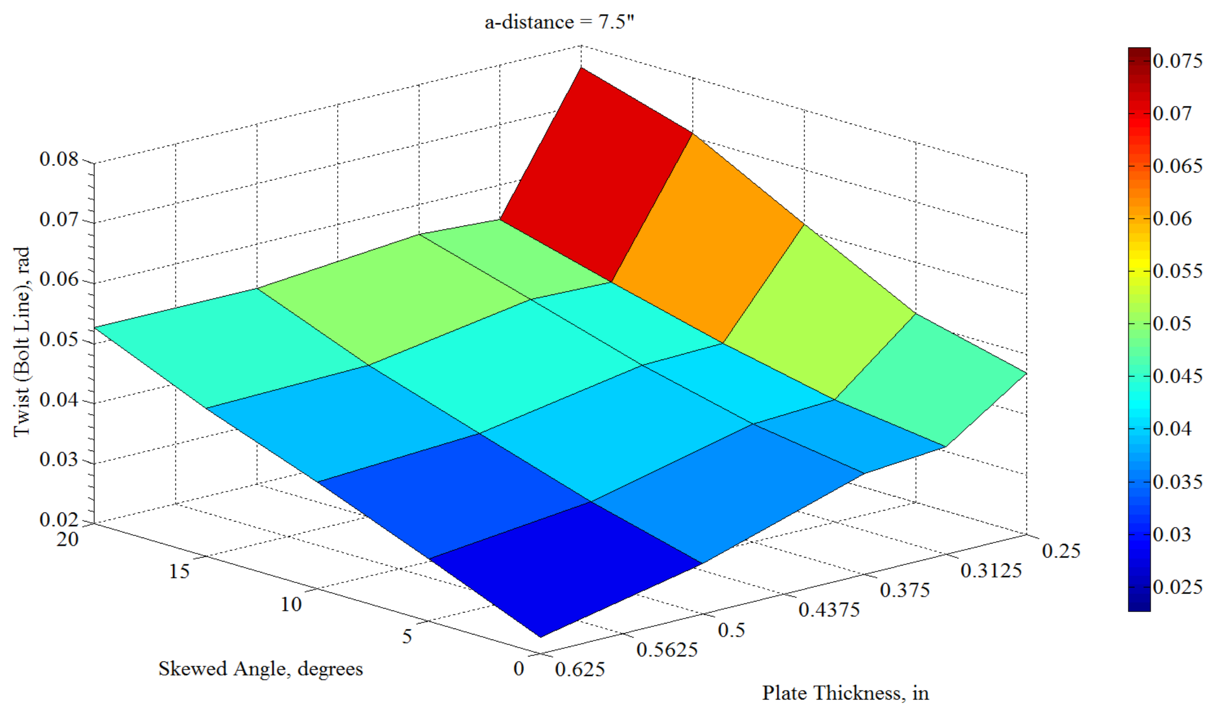
Plate Thickness = 5/8"
x 10^4
Connection Shear, lbs
a-distance, in
Skewed Angle, degrees

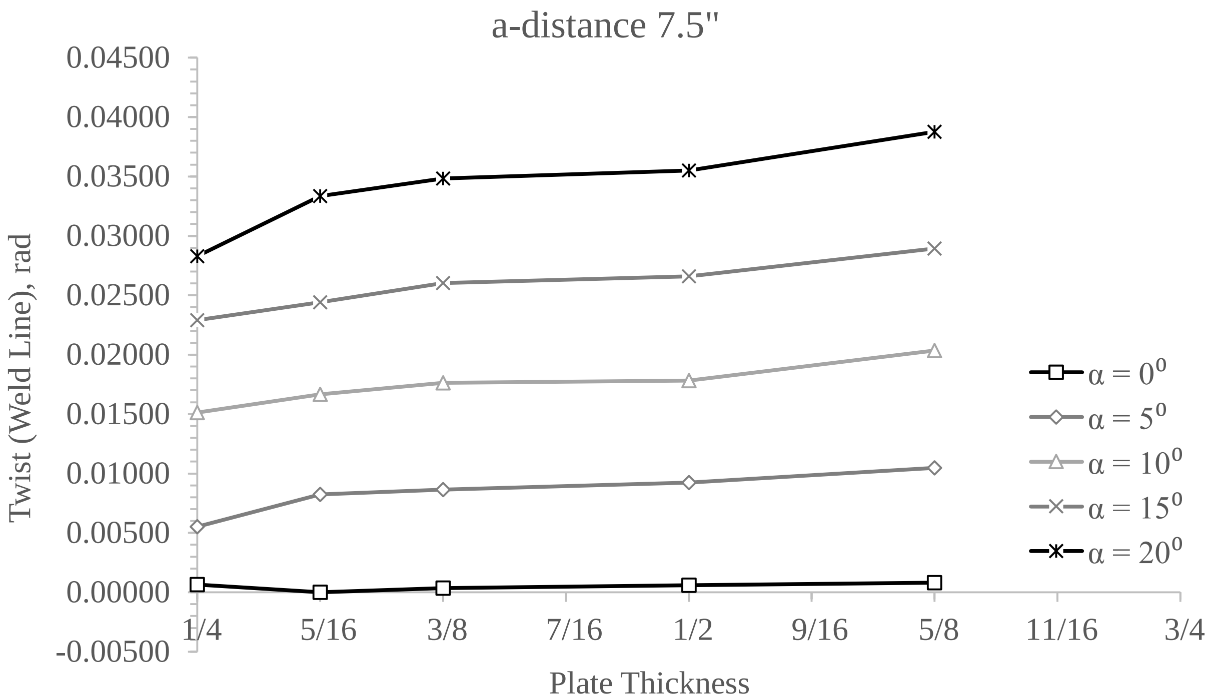
Three Bolted Connections (a-distance = 7.5”) Curves

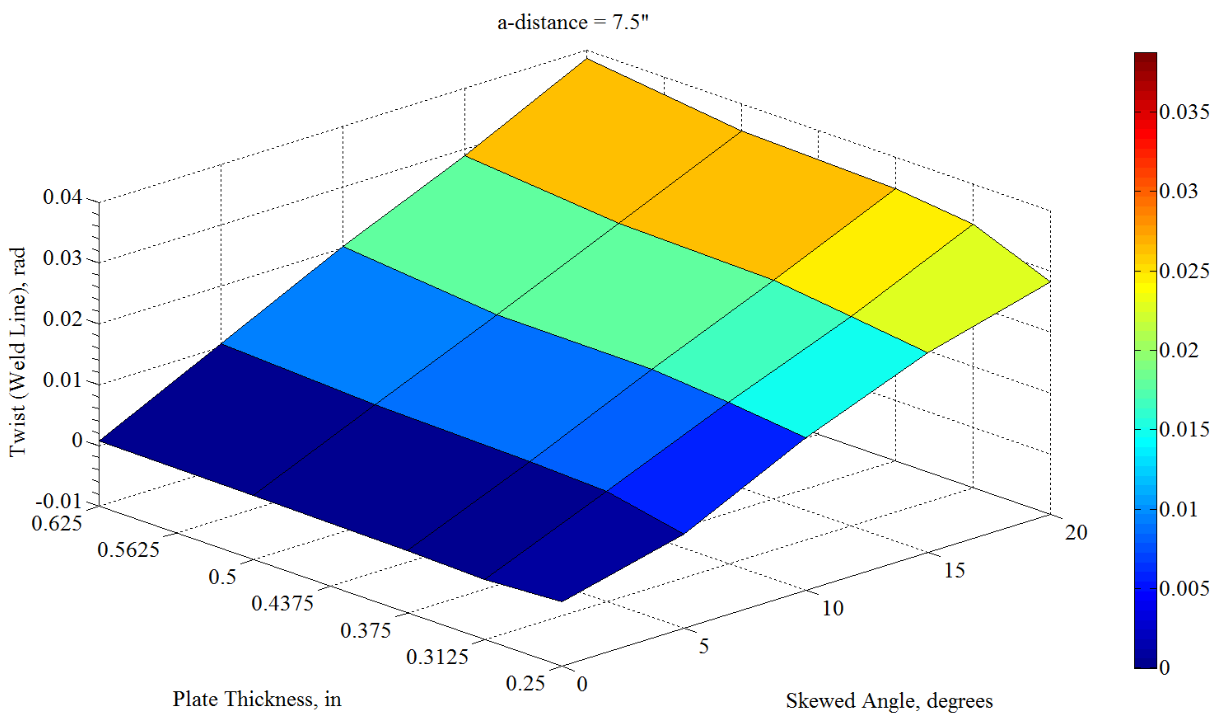


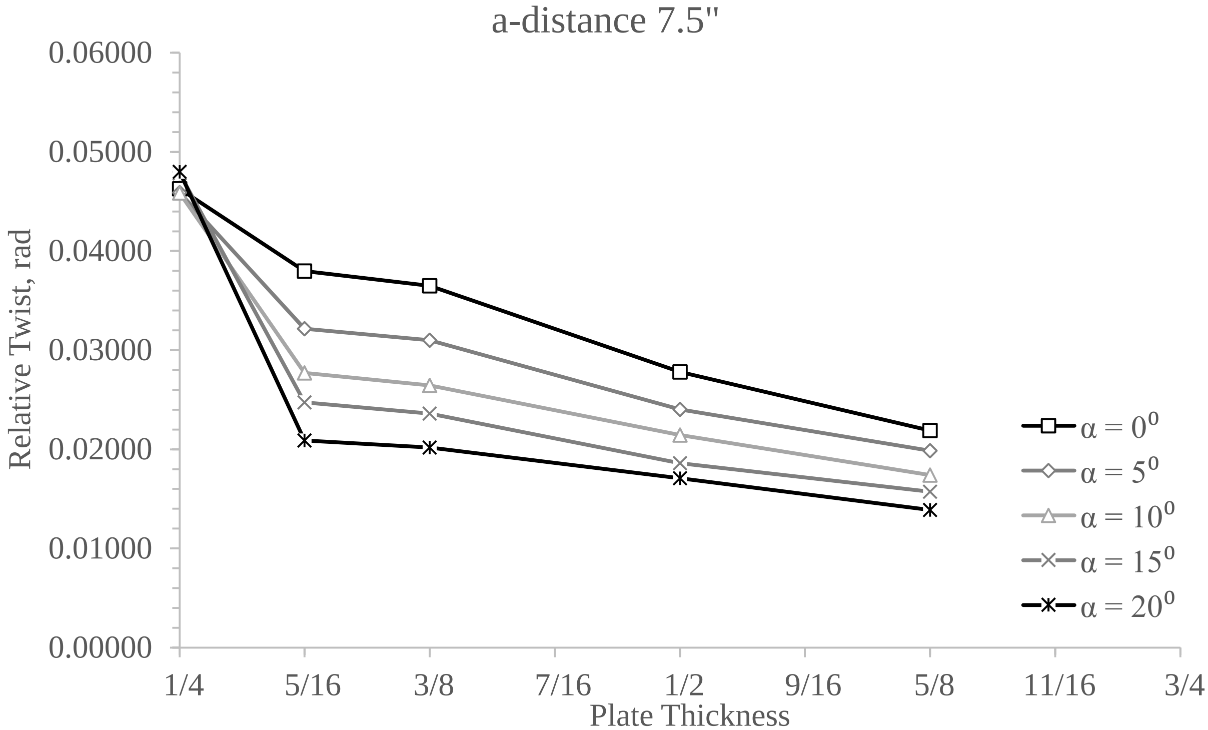



a-distance 7.5"
0.09000
0.08000
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
Twist (Bolt Line), rad
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
Plate Thickness

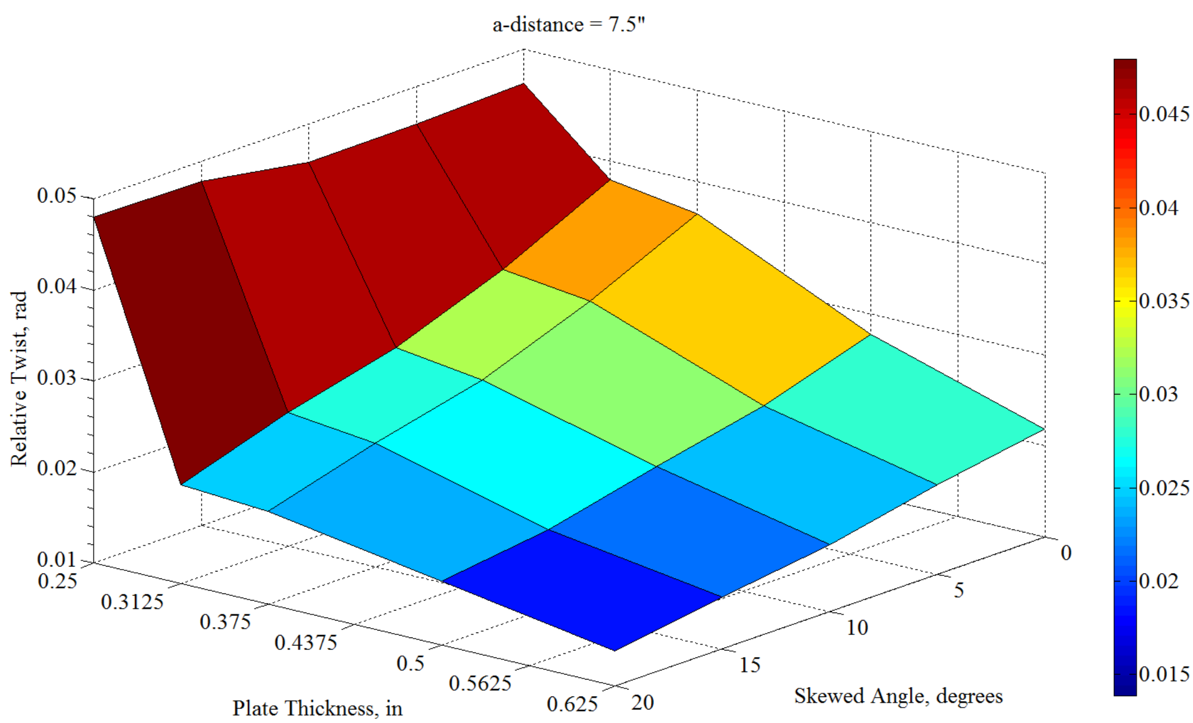

a-distance = 7.5"
0.08
0.07
0.06
0.05
0.04
0.03
0.02
Twist (Bolt Line), rad
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
0.25
Plate Thickness, in
0.075
0.07
0.065
0.06
0.055
0.05
0.045
0.04
0.035
0.03
0.025


a-distance 7.5"
0.04500
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
-0.00500
Twist (Weld Line), rad
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰


a-distance = 7.5"
Twist (Weld Line), rad
Plate Thickness, in
Skewed Angle, degrees
0.04
0.03
0.02
0.01
0
-0.01
0.625
0.5625
0.5
0.4375
0.375
0.3125
0.25
0
5
10
15
20
0.035
0.03
0.025
0.02
0.015
0.01
0.005
0

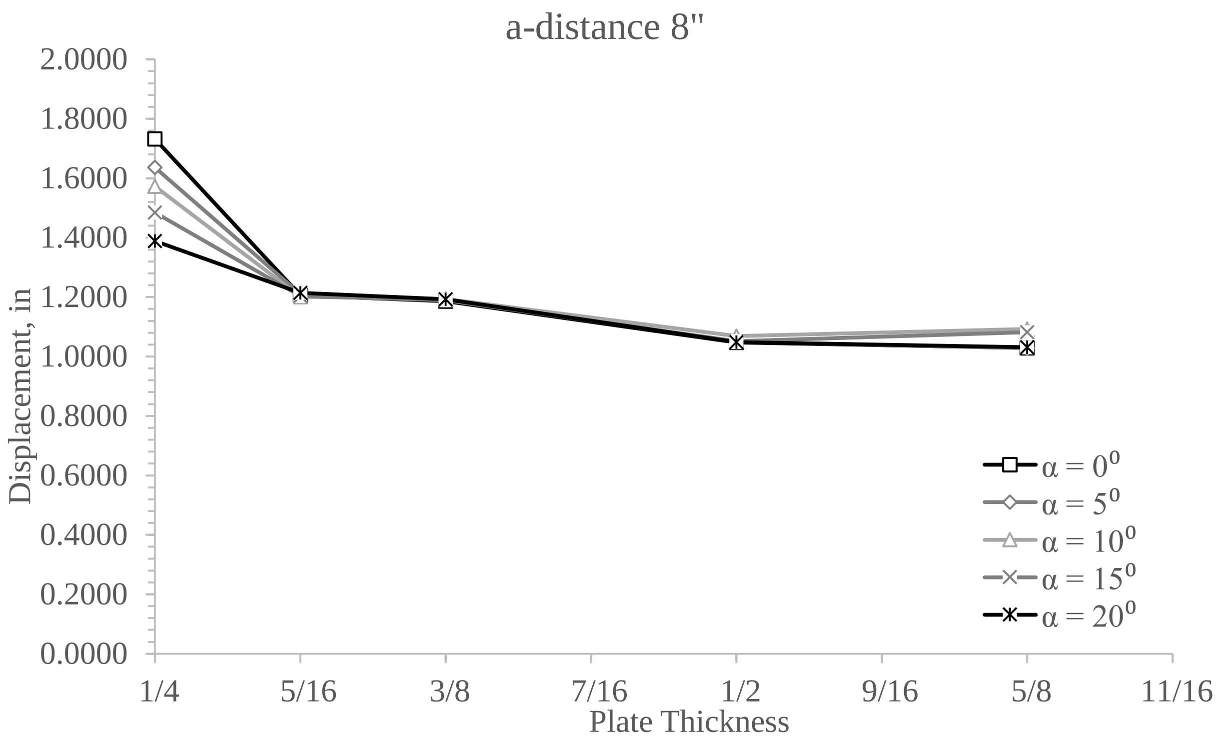


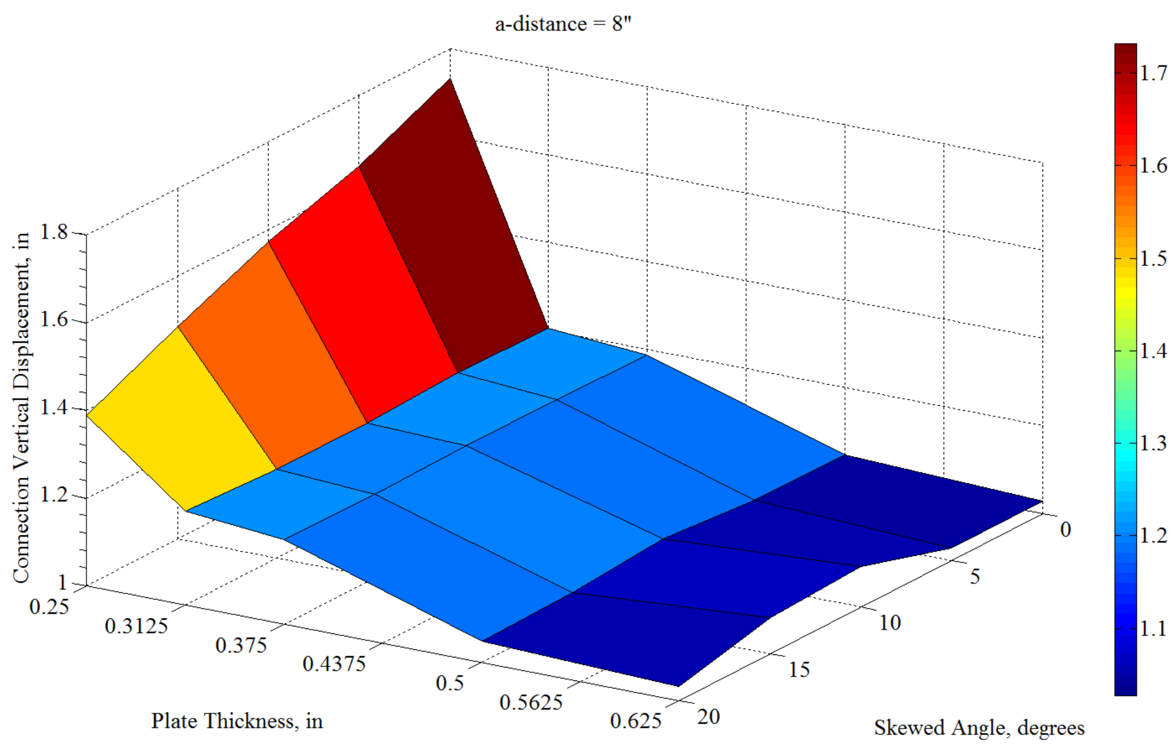
a-distance 7.5"
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
Relative Twist, rad
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

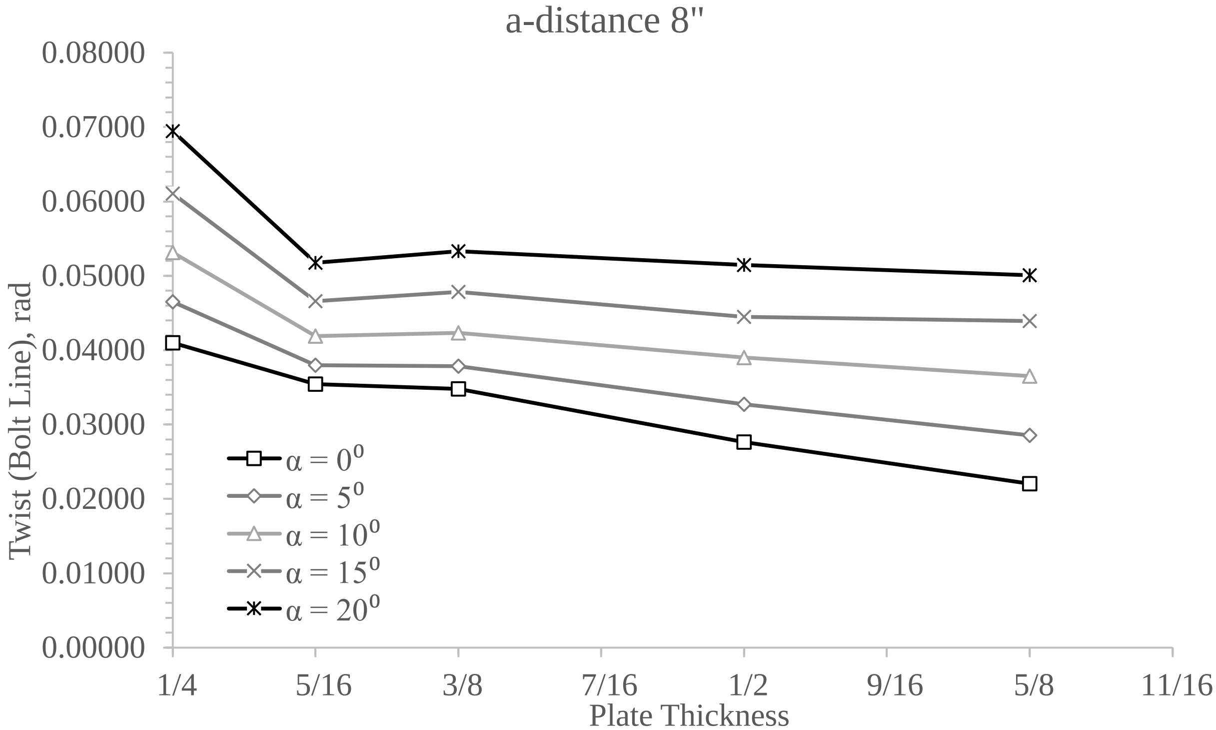


a-distance = 7.5"
0.05
0.04
0.03
0.02
0.01
Relative Twist, rad
0.25
0.3125
0.375
0.4375
0.5
0.5625
0.625
Plate Thickness, in
0
5
10
15
20
Skewed Angle, degrees
0.045
0.04
0.035
0.03
0.025
0.02
0.015

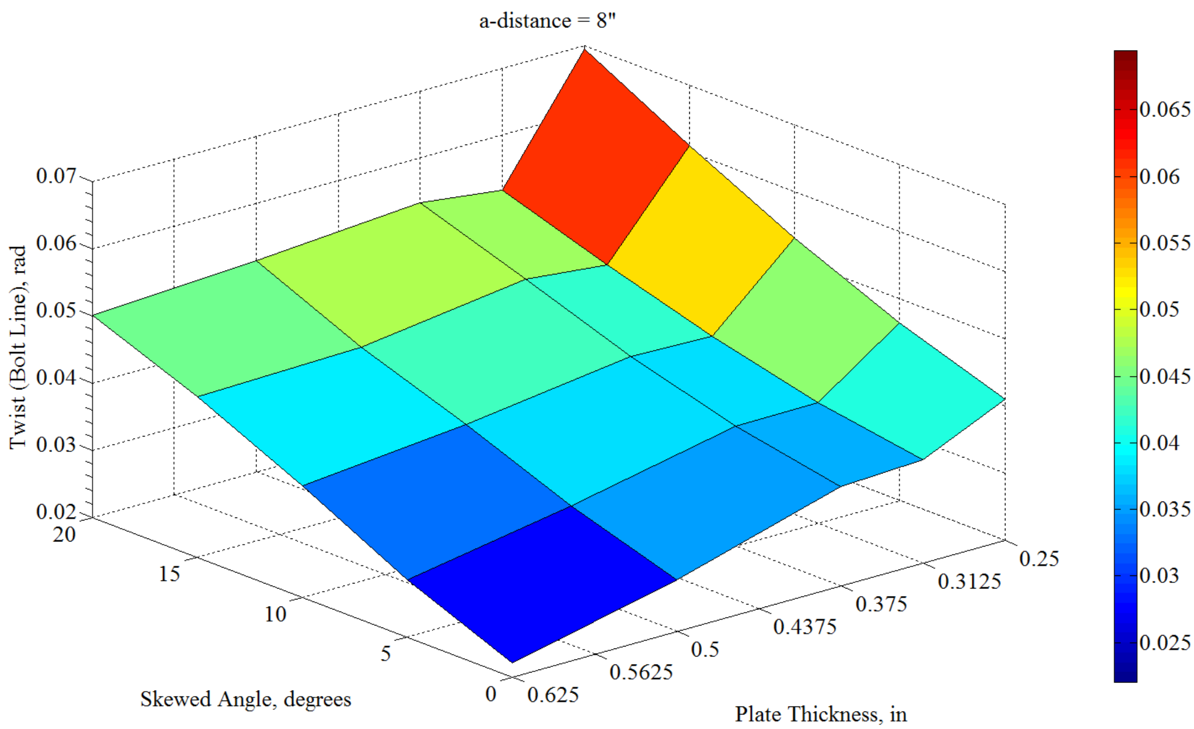
Three Bolted Connections (a-distance = 8”) Curves



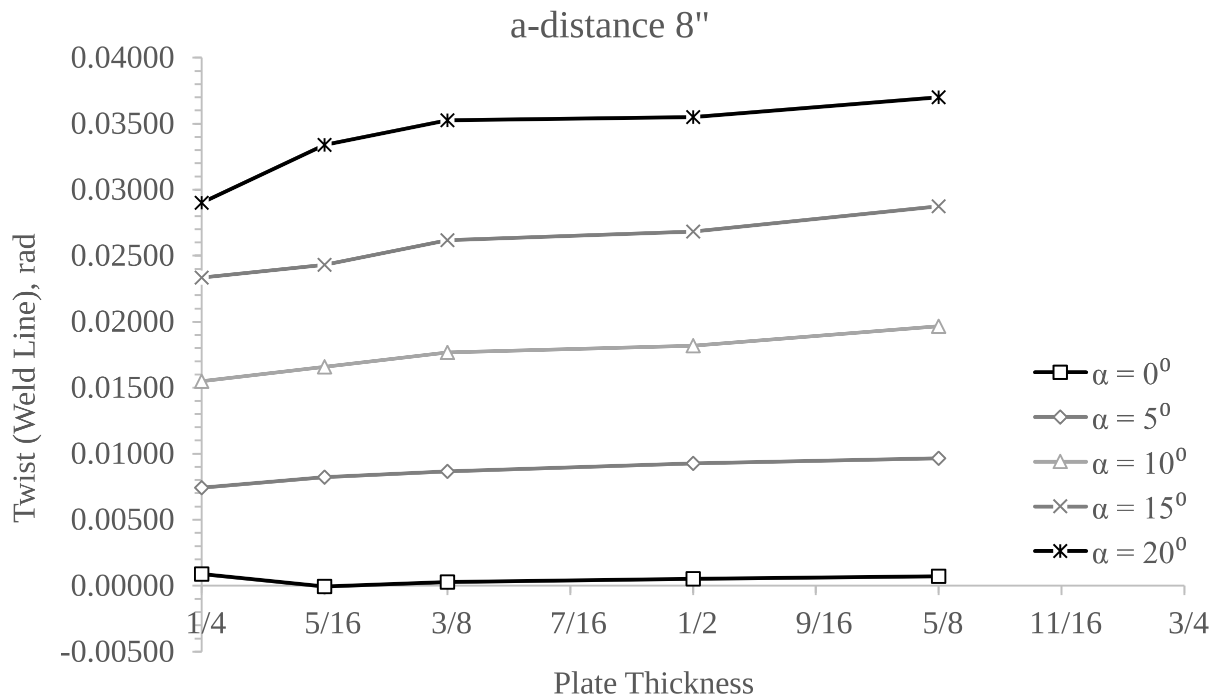




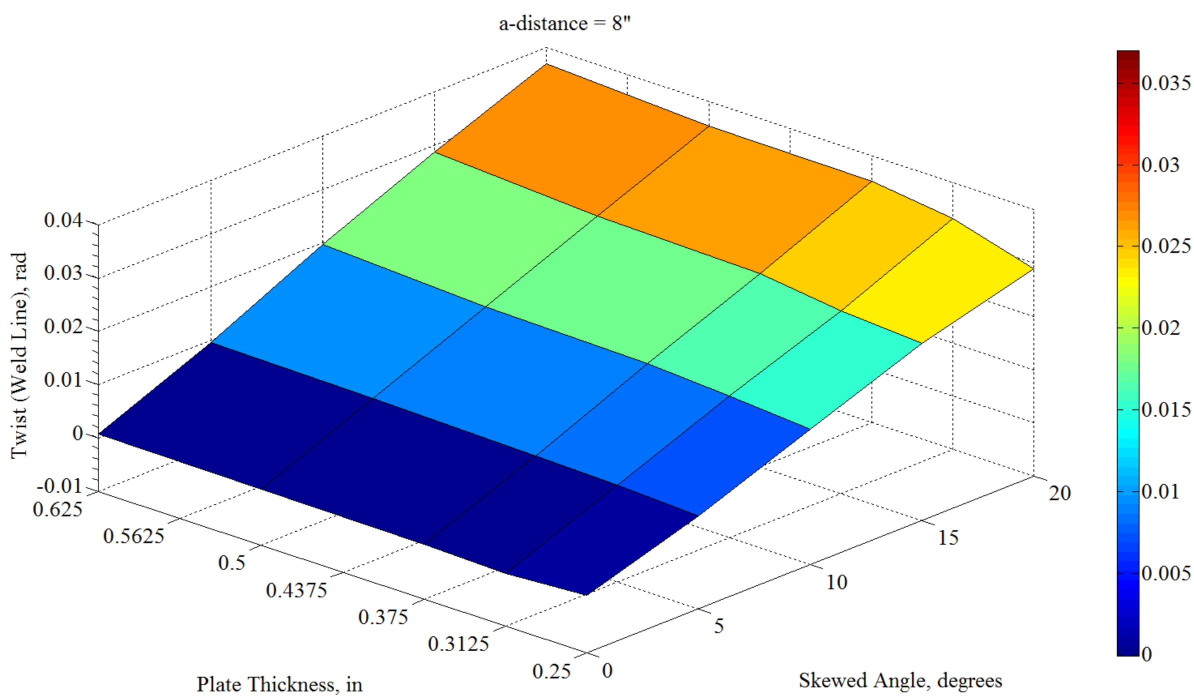
a-distance 8"
0.08000
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
Twist (Bolt Line), rad
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
Plate Thickness



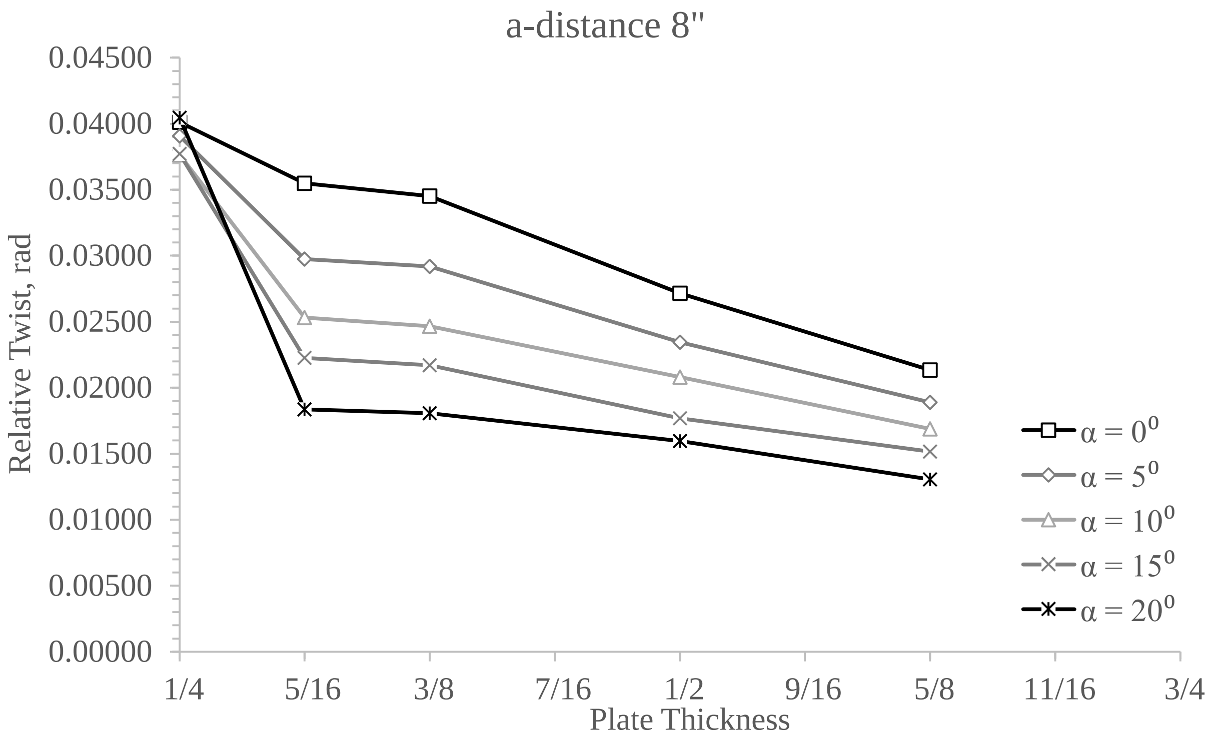
a-distance = 8"
Twist (Bolt Line), rad
0.07
0.06
0.05
0.04
0.03
0.02
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
0.25
Plate Thickness, in
0.065
0.06
0.055
0.05
0.045
0.04
0.035
0.03
0.025

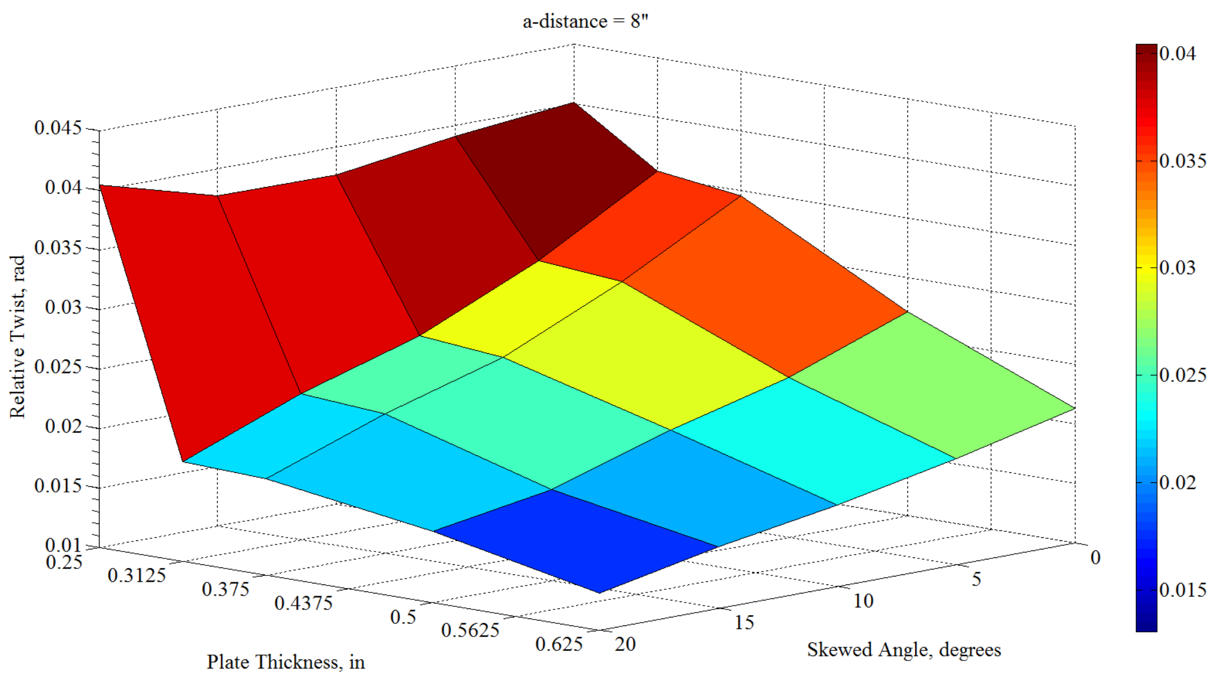


a-distance 8"
Twist (Weld Line), rad
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
-0.00500
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

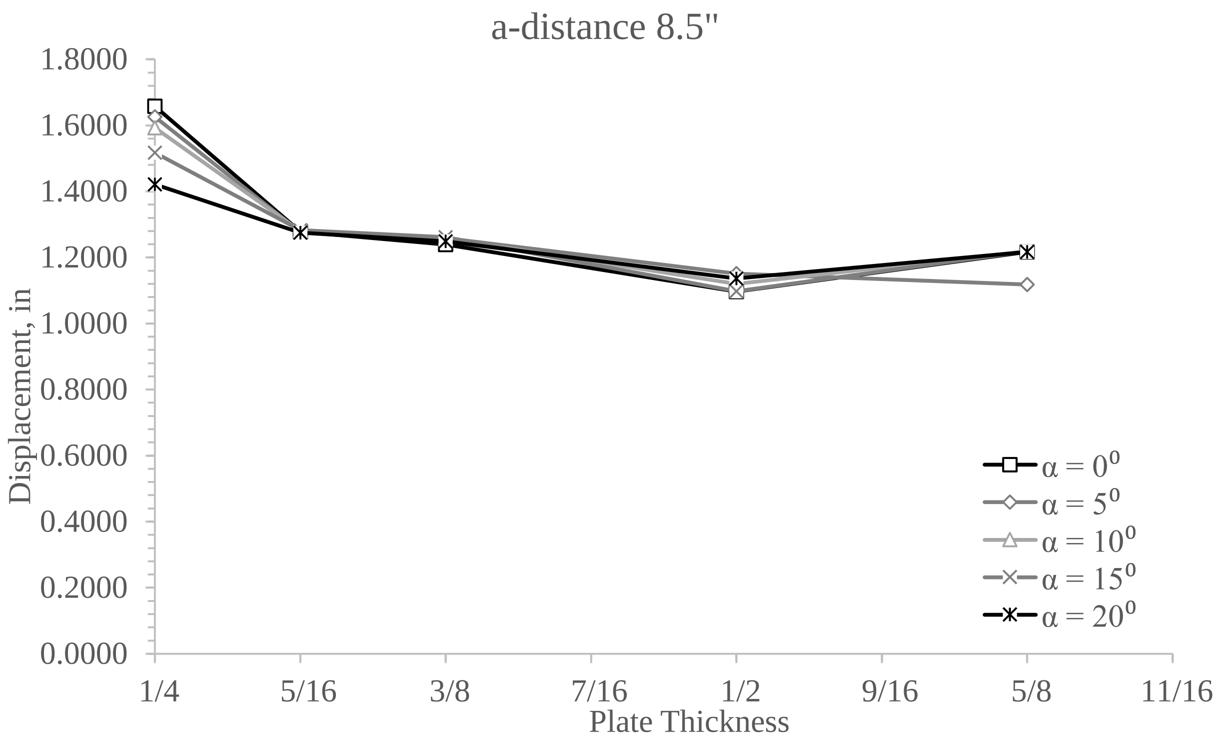


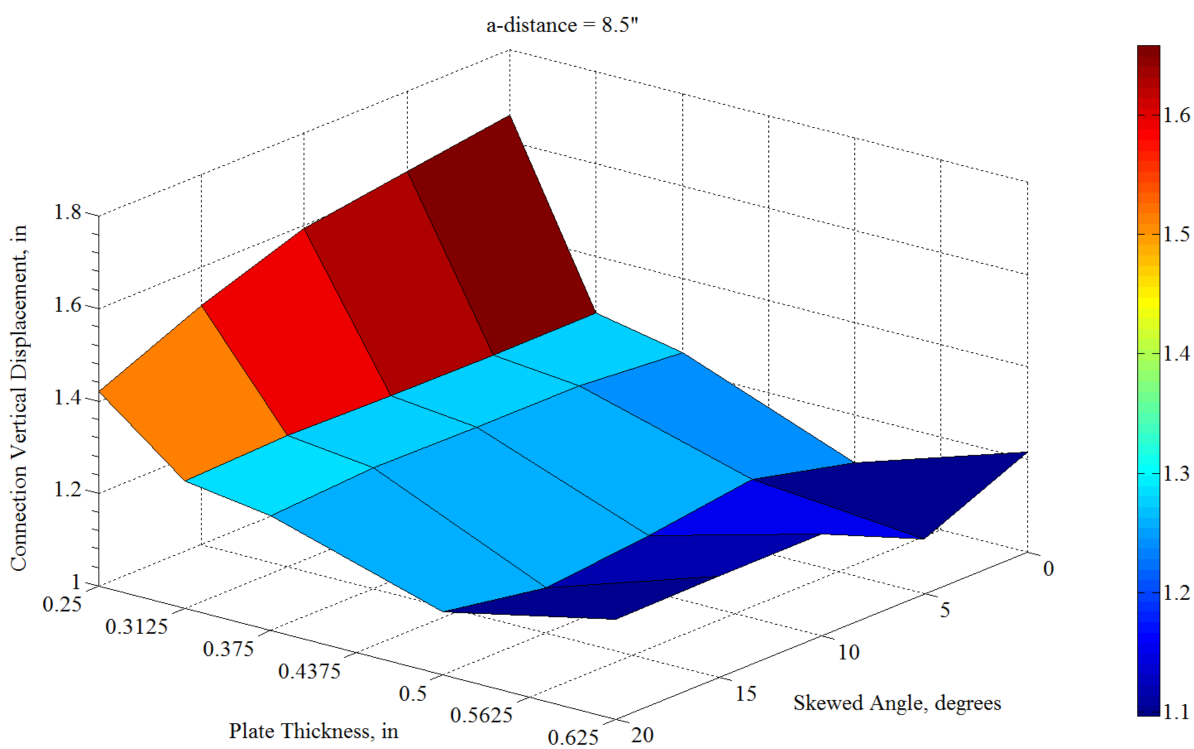
a-distance = 8"
Twist (Weld Line), rad
0.04
0.03
0.02
0.01
0
-0.01
0.625
0.5625
0.5
0.4375
0.375
0.3125
0.25
Plate Thickness, in
0
5
10
15
20
Skewed Angle, degrees
0.035
0.03
0.025
0.02
0.015
0.01
0.005
0

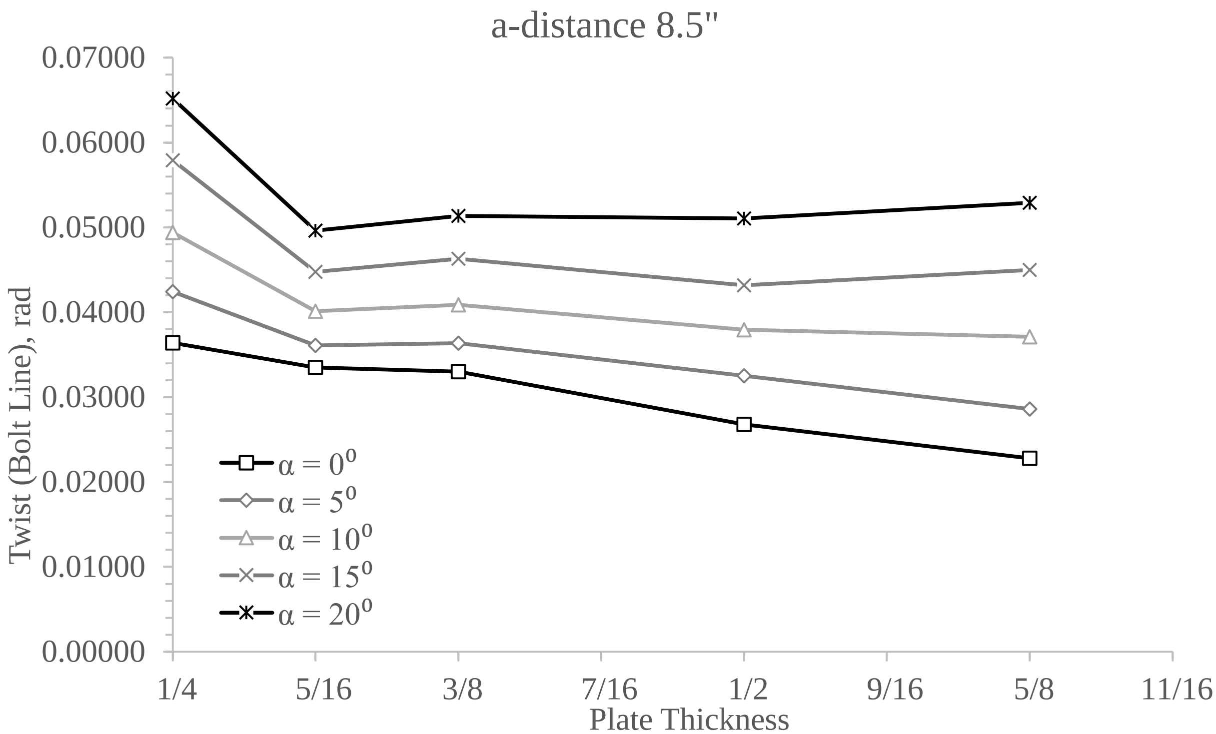

a-distance 8"
0.04500
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
Relative Twist, rad
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness
α = 0°
α = 5°
α = 10°
α = 15°
α = 20°

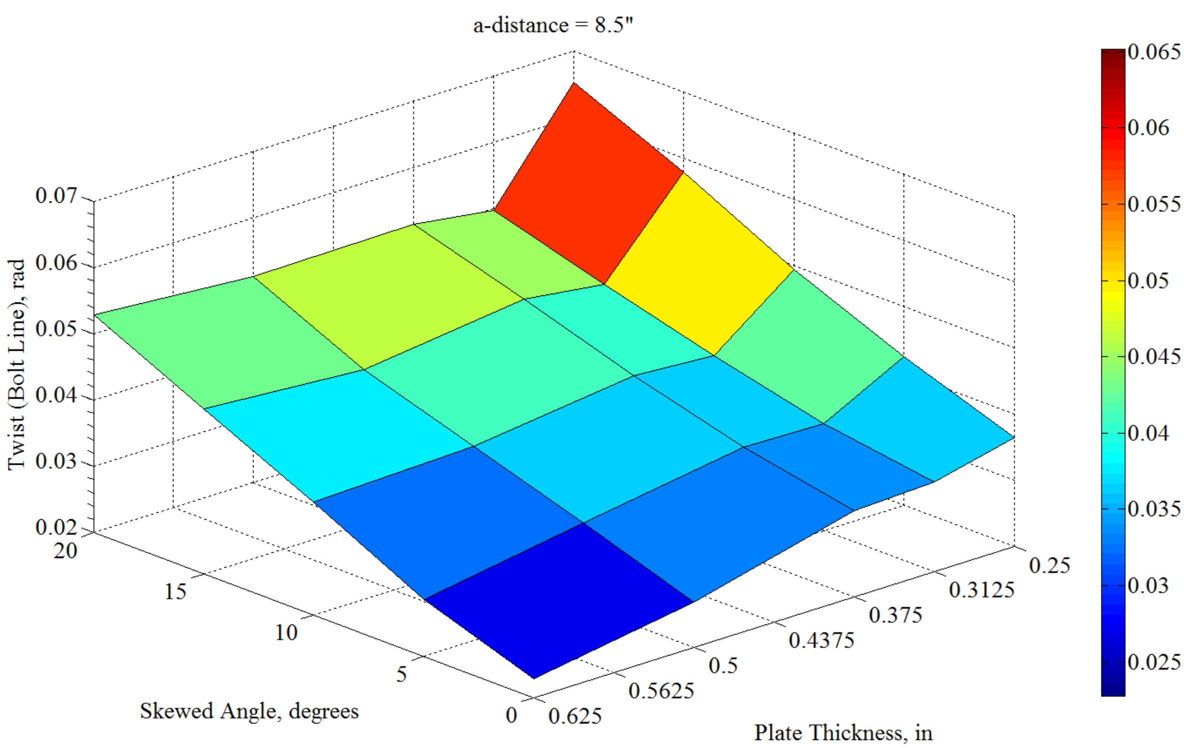

a-distance = 8"
Relative Twist, rad
0.045
0.04
0.035
0.03
0.025
0.02
0.015
0.01
0.25
0.3125
0.375
0.4375
0.5
0.5625
0.625
Plate Thickness, in
0
5
10
15
20
Skewed Angle, degrees
0.04
0.035
0.03
0.025
0.02
0.015

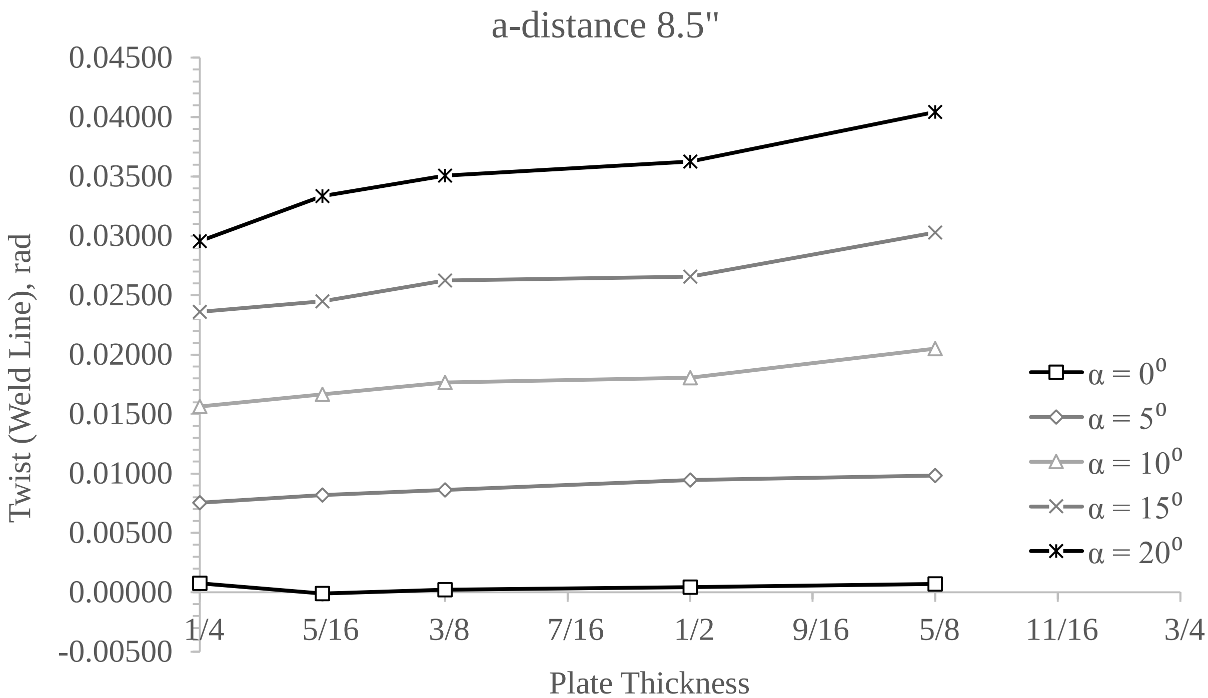
<u>Three Bolted Connections (a-distance = 8.5") Curves</u>



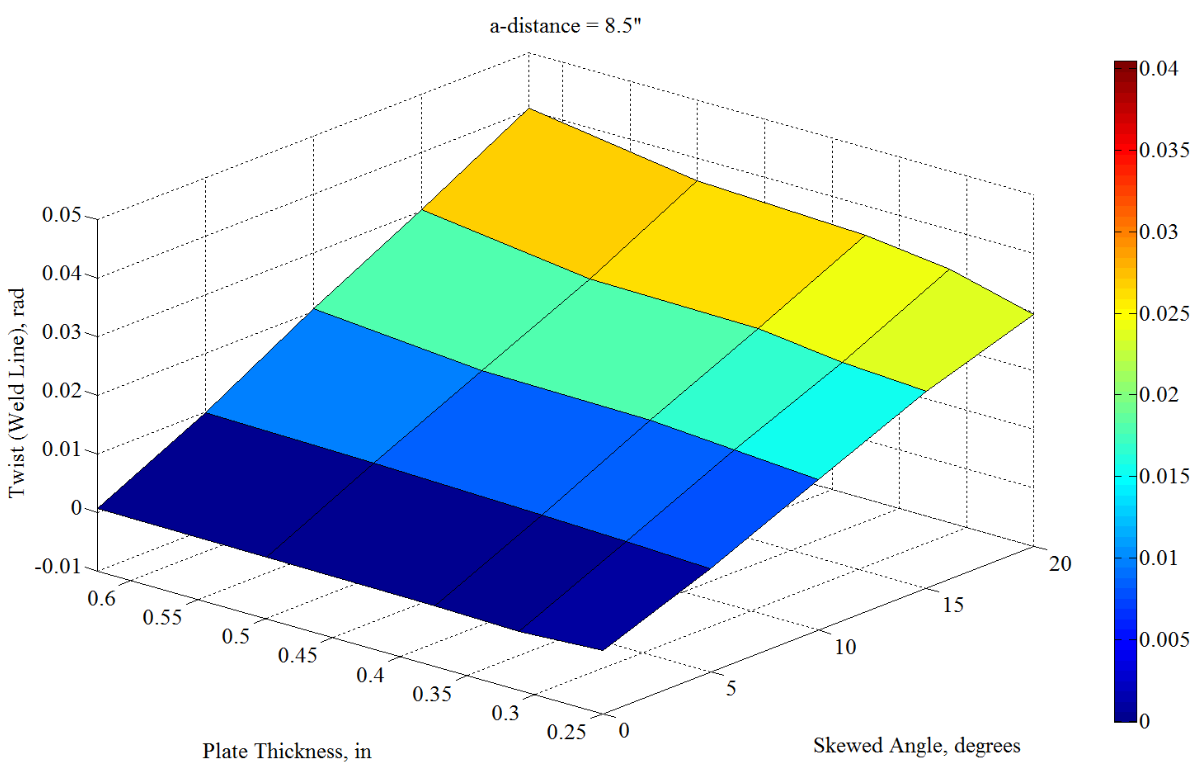



a-distance 8.5"
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
Twist (Bolt Line), rad
$\alpha = 0^0$
$\alpha = 5^0$
$\alpha = 10^0$
$\alpha = 15^0$
$\alpha = 20^0$
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
Plate Thickness

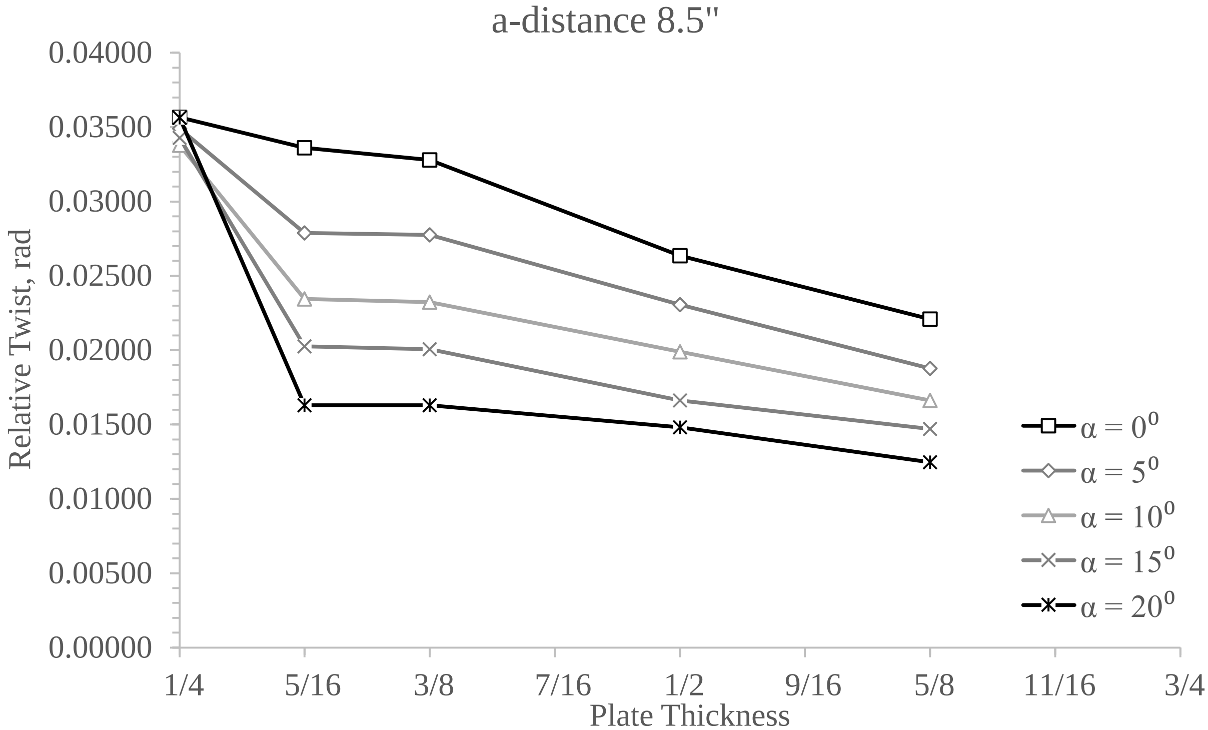

a-distance = 8.5"
0.07
0.06
0.05
0.04
0.03
0.02
Twist (Bolt Line), rad
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
0.25
Plate Thickness, in
0.065
0.06
0.055
0.05
0.045
0.04
0.035
0.03
0.025

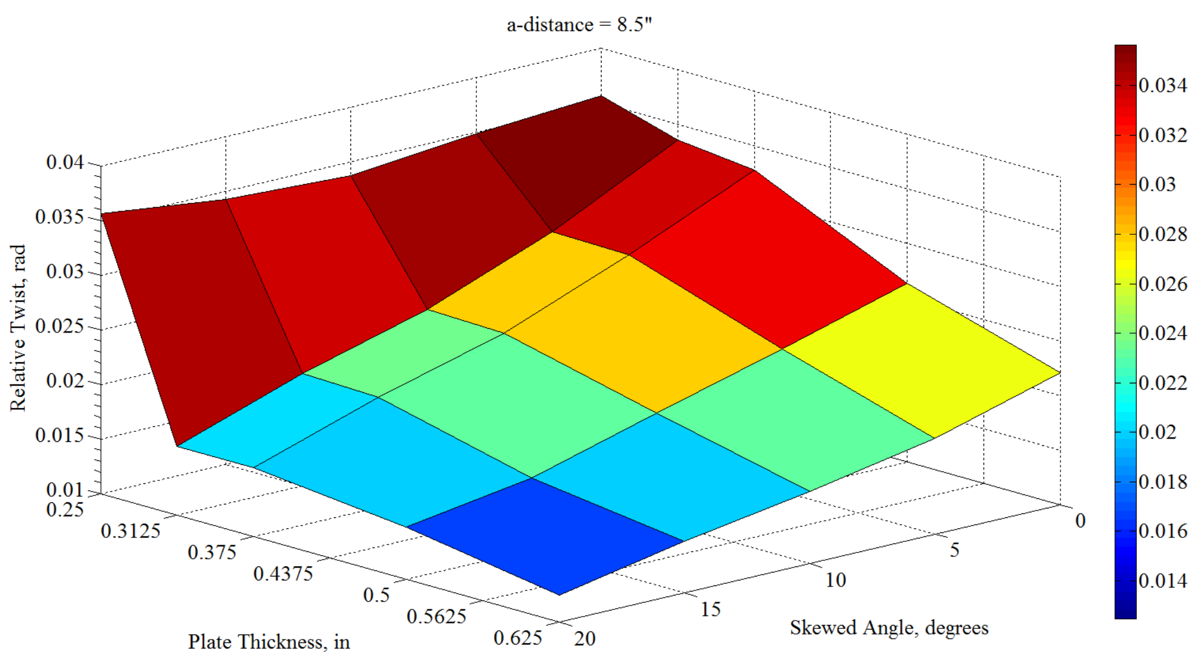


a-distance 8.5"
Twist (Weld Line), rad
0.04500
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
-0.00500
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

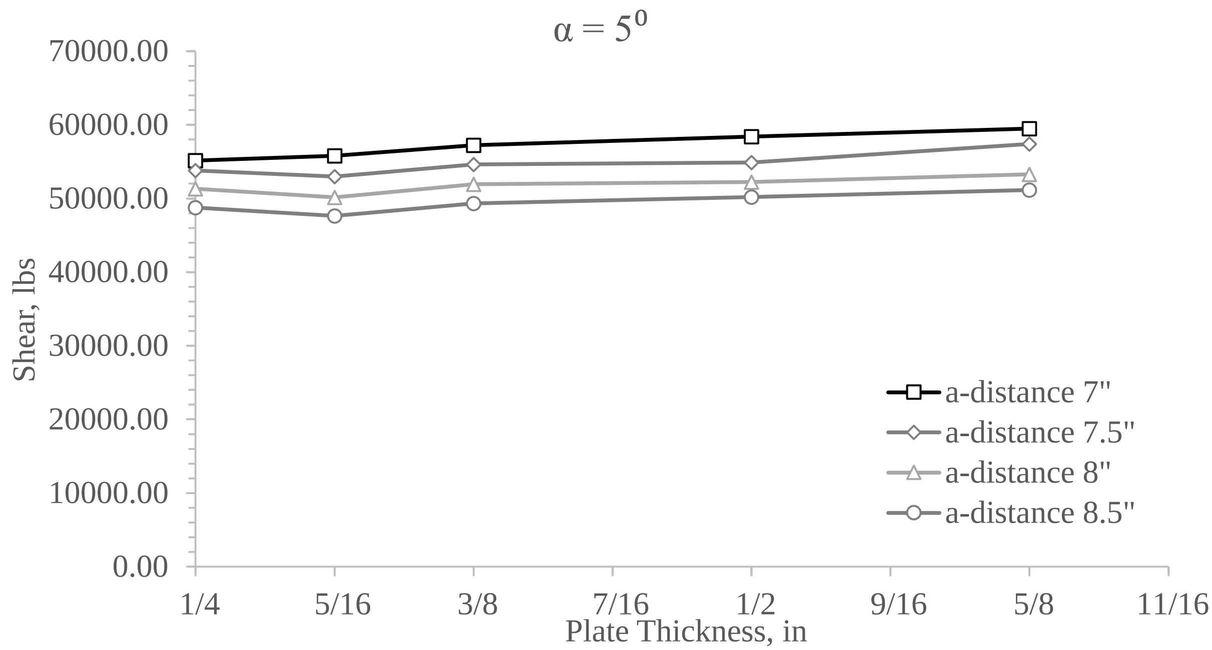


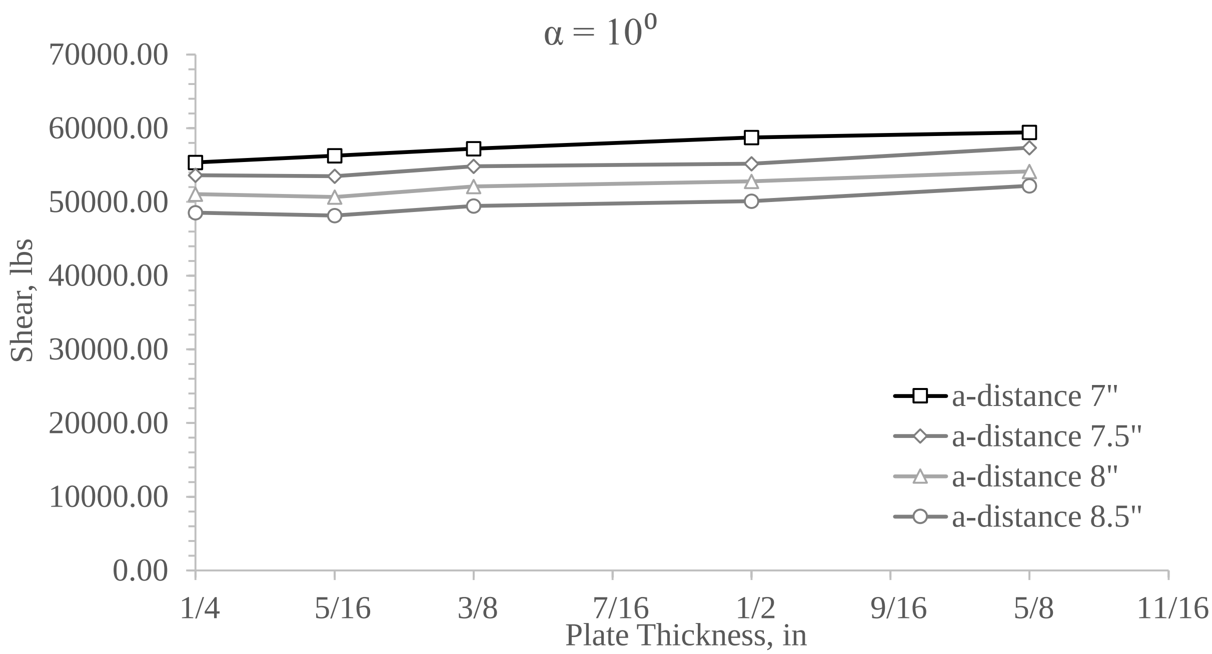
a-distance = 8.5"
Twist (Weld Line), rad
Plate Thickness, in
Skewed Angle, degrees

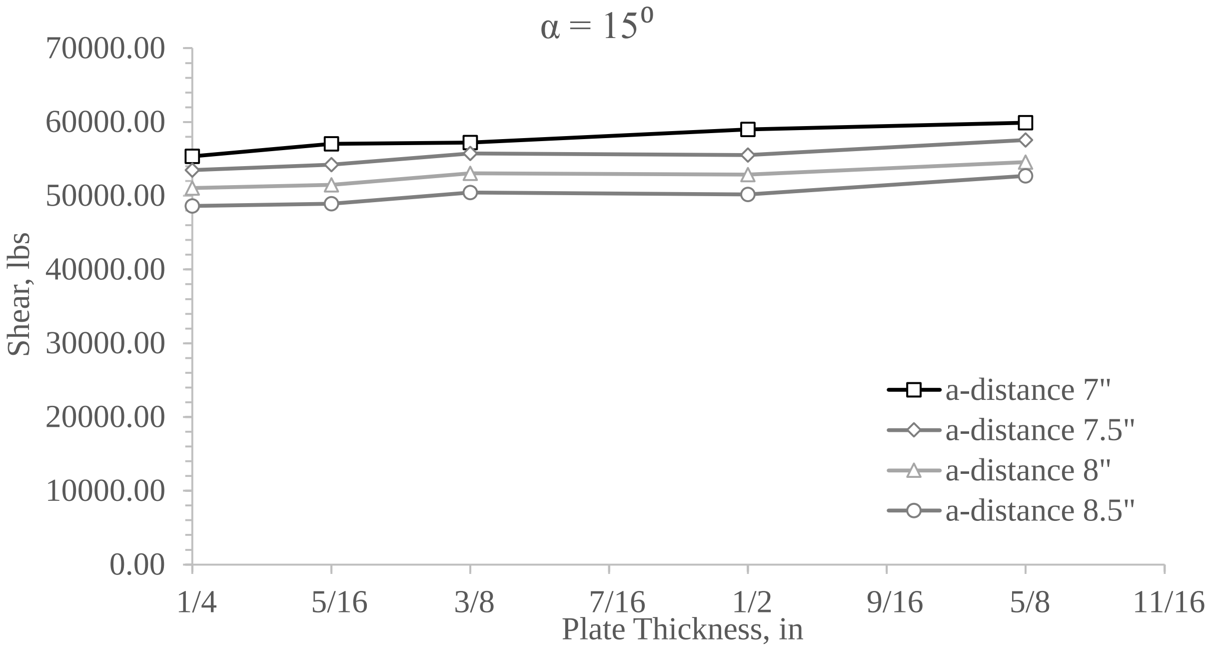

a-distance 8.5"
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
Relative Twist, rad
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰


a-distance = 8.5"
0.04
0.035
0.03
0.025
0.02
0.015
0.01
Relative Twist, rad
0.25
0.3125
0.375
0.4375
0.5
0.5625
0.625
Plate Thickness, in
0
5
10
15
20
Skewed Angle, degrees
0.034
0.032
0.03
0.028
0.026
0.024
0.022
0.02
0.018
0.016
0.014

Three Bolted Connections (5^0, 10^0, and 15^0) Curves

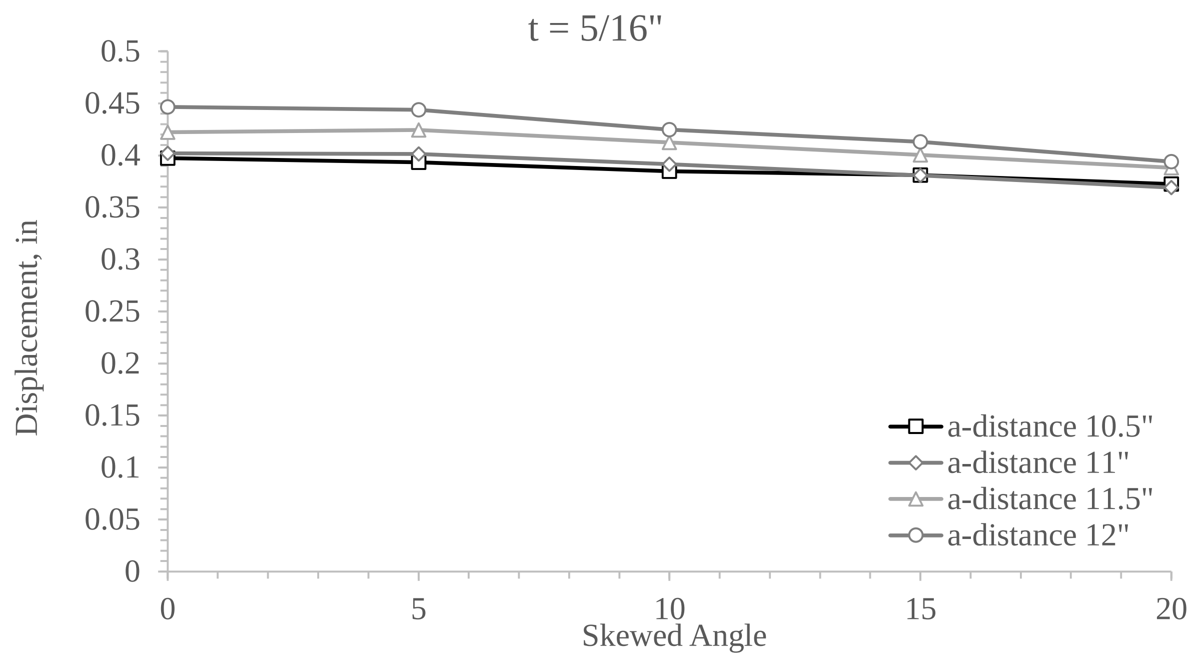


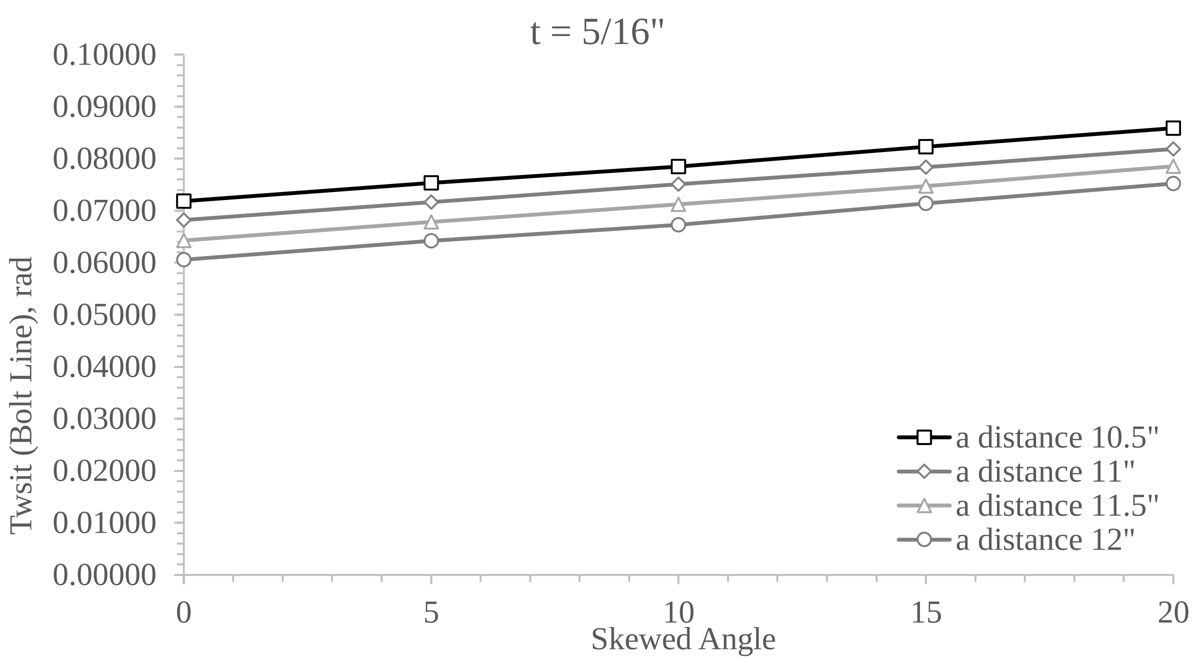


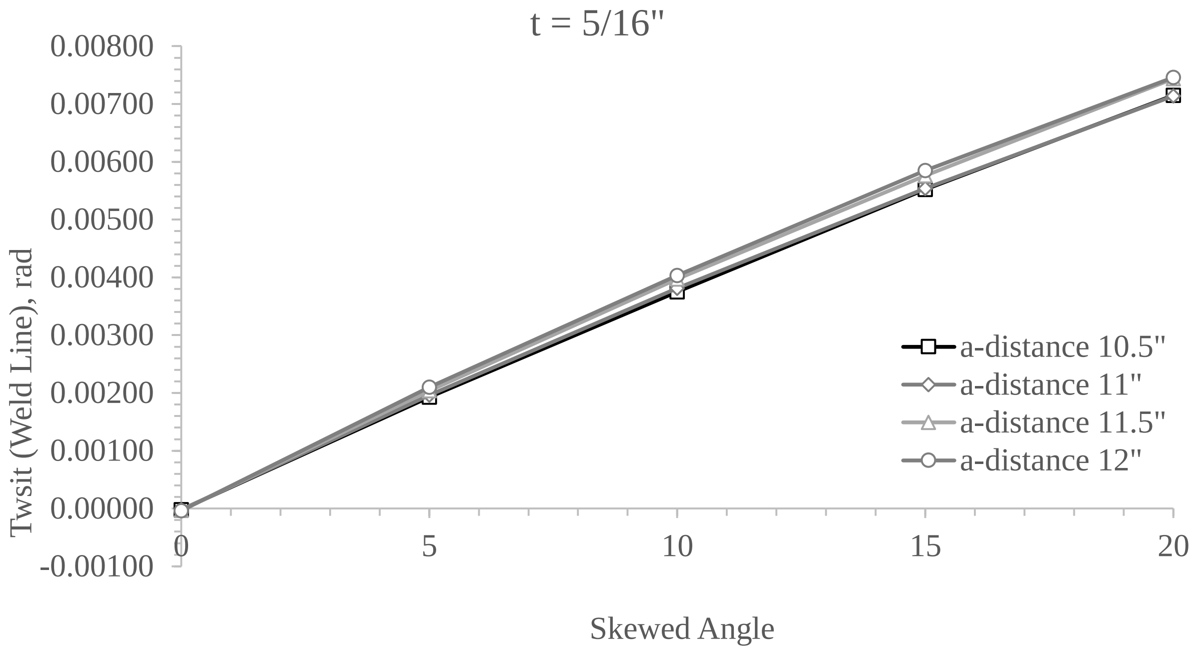

α = 15⁰
70000.00
60000.00
50000.00
40000.00
30000.00
20000.00
10000.00
0.00
Shear, lbs
1/4
5/16
3/8
7/16
1/2
9/16
5/8
11/16
Plate Thickness, in
a-distance 7"
a-distance 7.5"
a-distance 8"
a-distance 8.5"

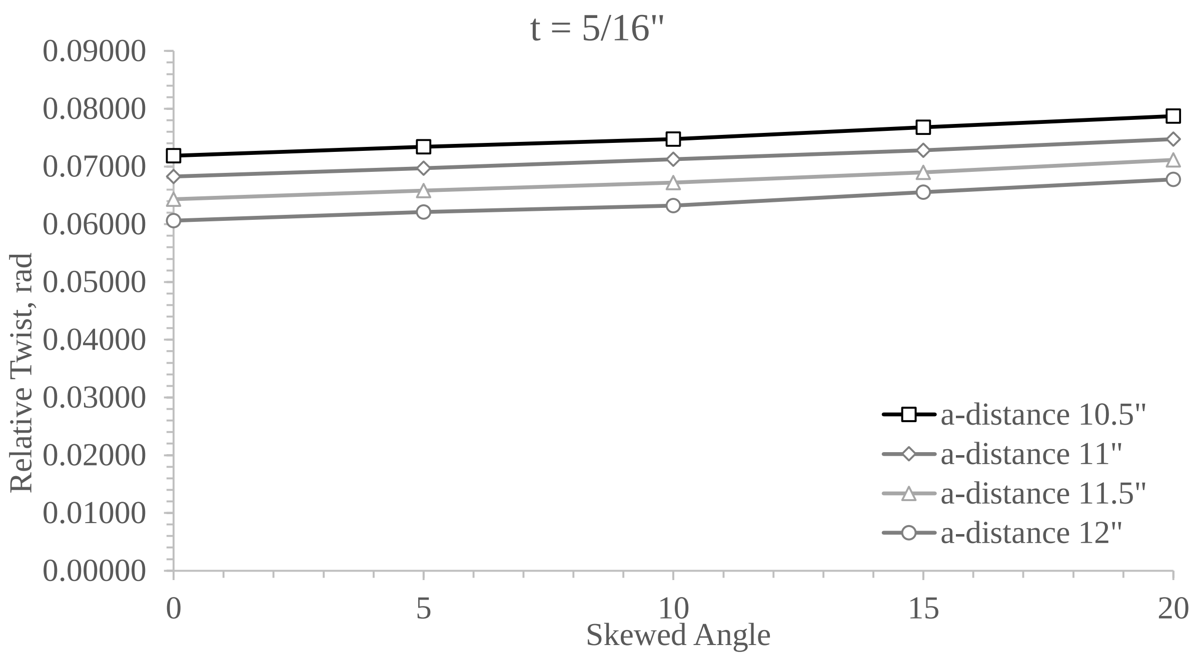
B2. Five Bolted Connections

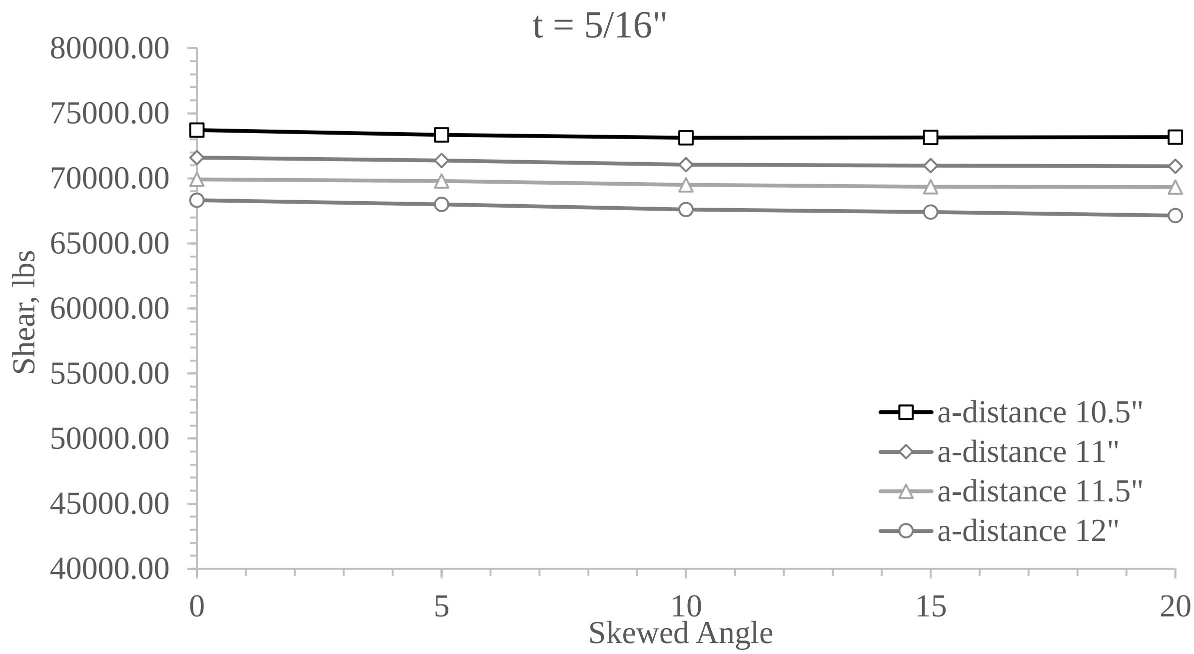
Five Bolted Connections (t = 5/16”) Curves






t = 5/16"
Twsit (Weld Line), rad
0.00800
0.00700
0.00600
0.00500
0.00400
0.00300
0.00200
0.00100
0.00000
-0.00100
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 5/16"
Relative Twist, rad
0.09000
0.08000
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 5/16"
80000.00
75000.00
70000.00
65000.00
60000.00
55000.00
50000.00
45000.00
40000.00
Shear, lbs
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"

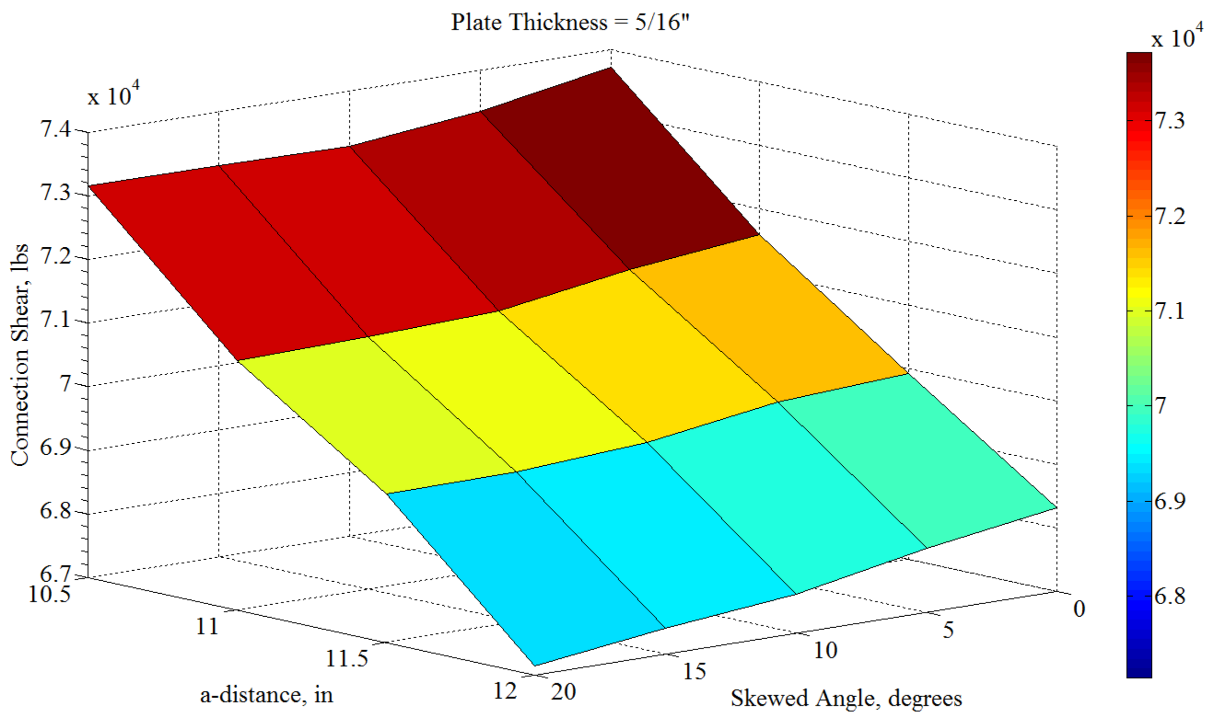
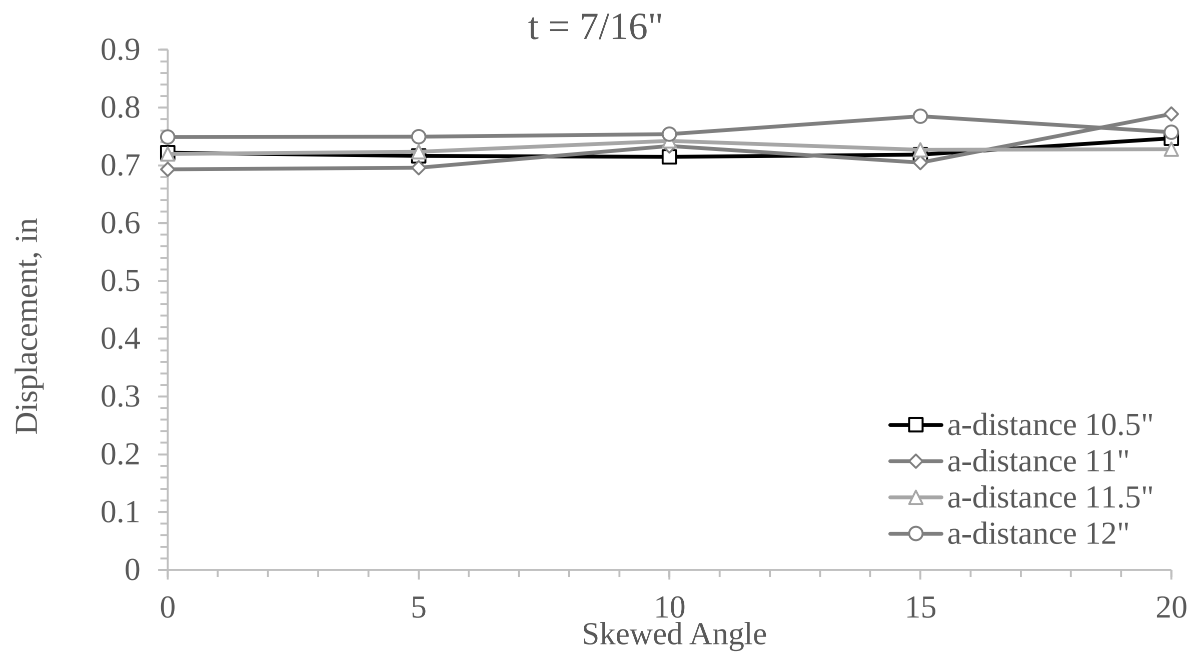
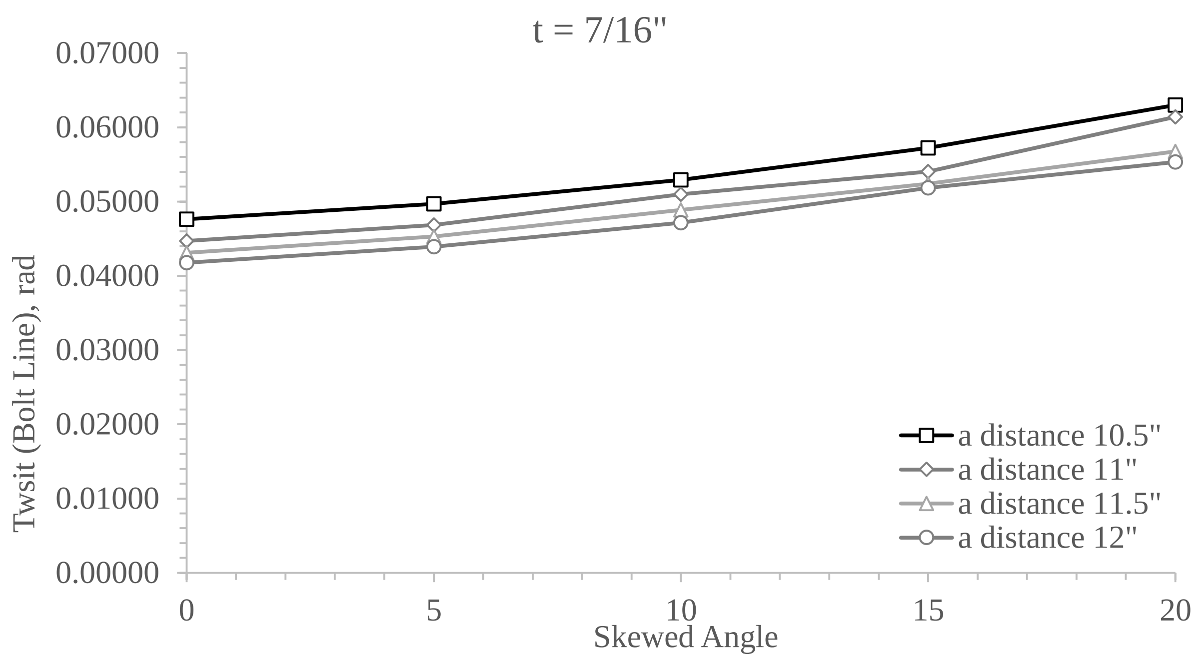
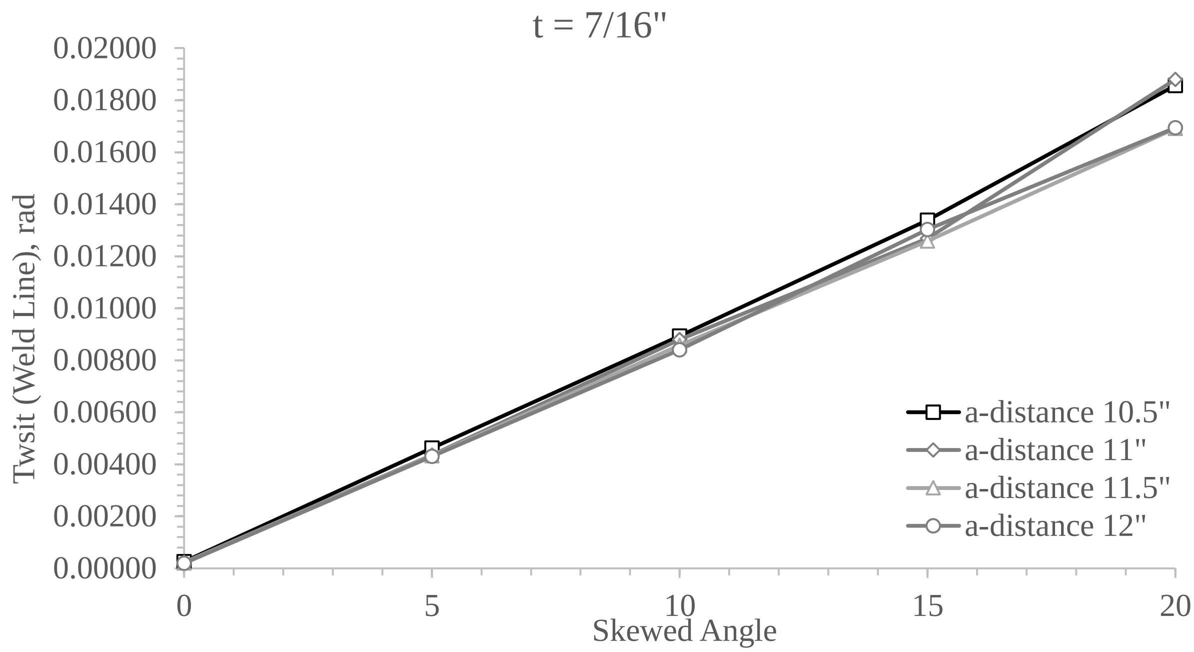


Plate Thickness = 5/16"
x 10^4
7.4
7.3
7.2
7.1
7
6.9
6.8
6.7
Connection Shear, lbs
10.5
11
11.5
12
a-distance, in
20
15
10
5
0
Skewed Angle, degrees
x 10^4
7.3
7.2
7.1
7
6.9
6.8

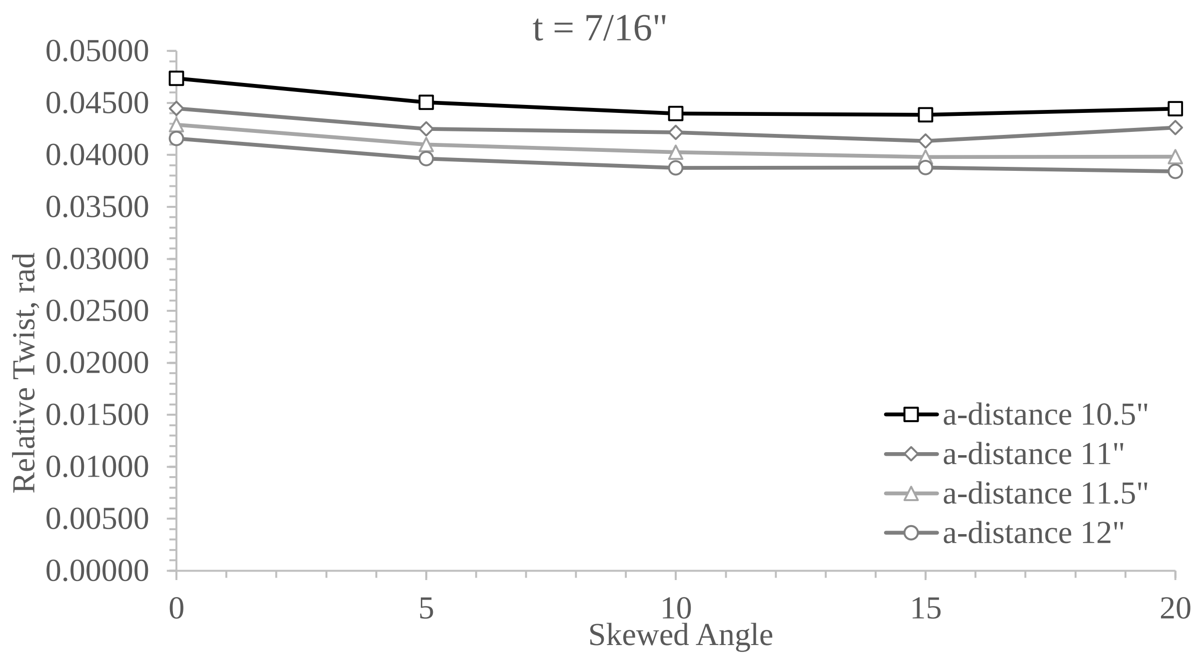
Five Bolted Connections (t = 7/16”) Curves



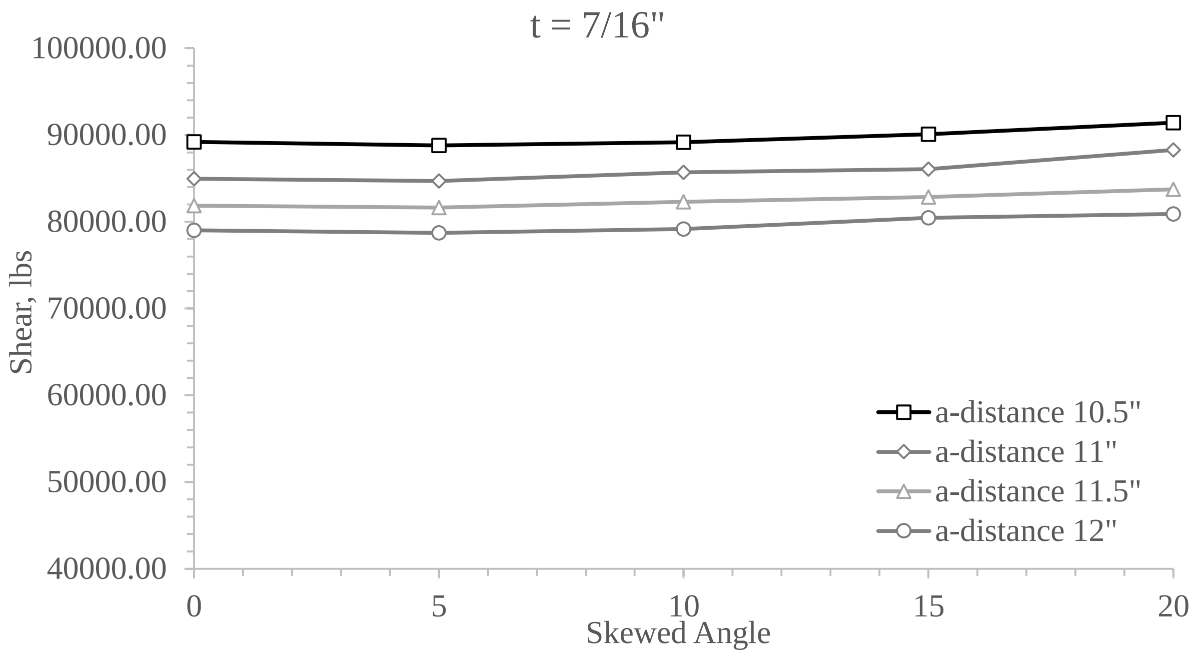




t = 7/16"
0.02000
0.01800
0.01600
0.01400
0.01200
0.01000
0.00800
0.00600
0.00400
0.00200
0.00000
Twsit (Weld Line), rad
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"



t = 7/16"
0.05000
0.04500
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
Relative Twist, rad
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 7/16"
Shear, lbs
100000.00
90000.00
80000.00
70000.00
60000.00
50000.00
40000.00
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"

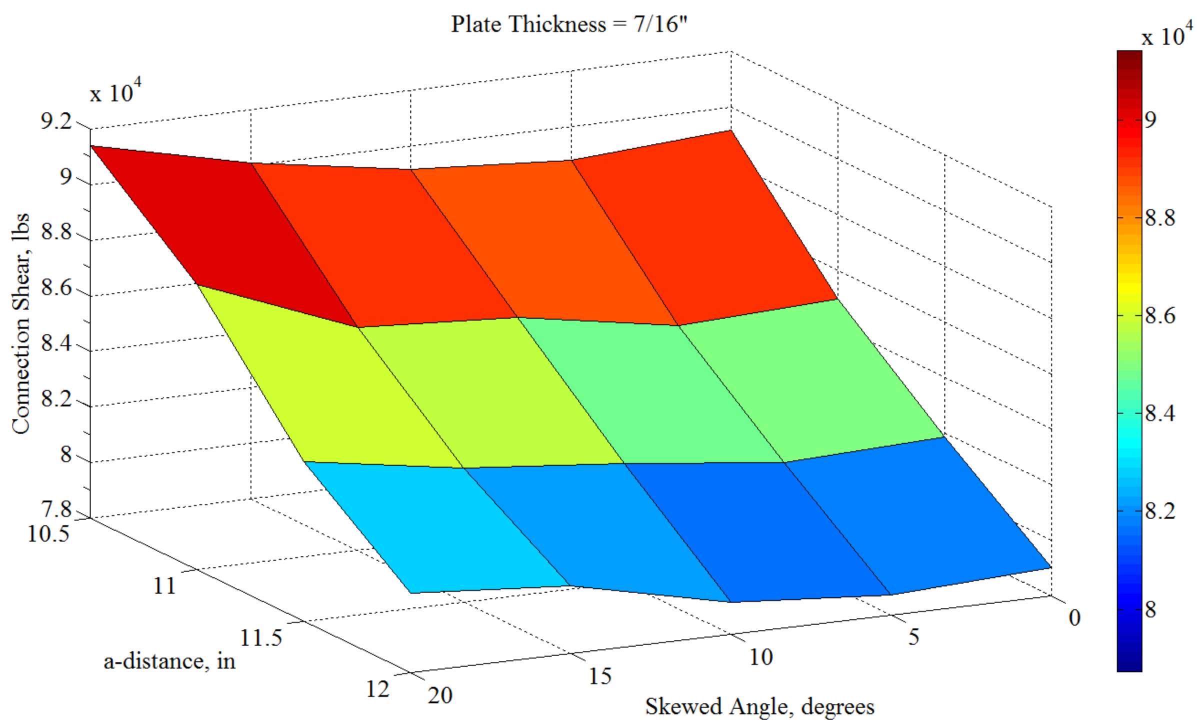
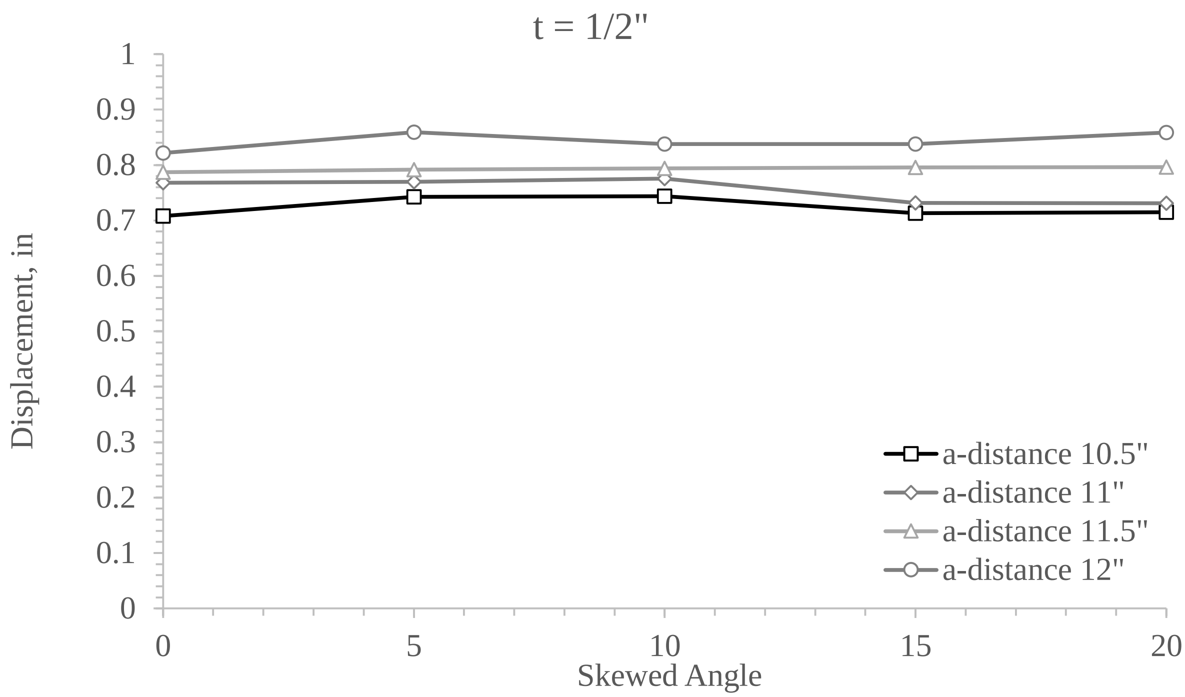
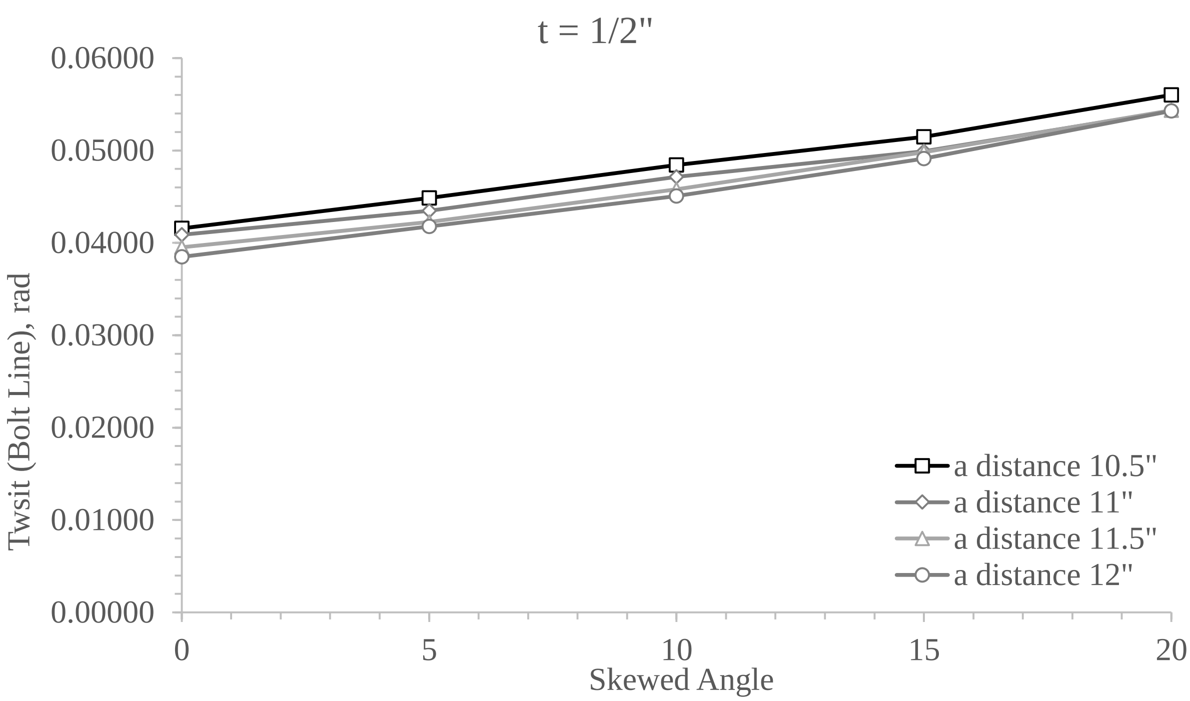
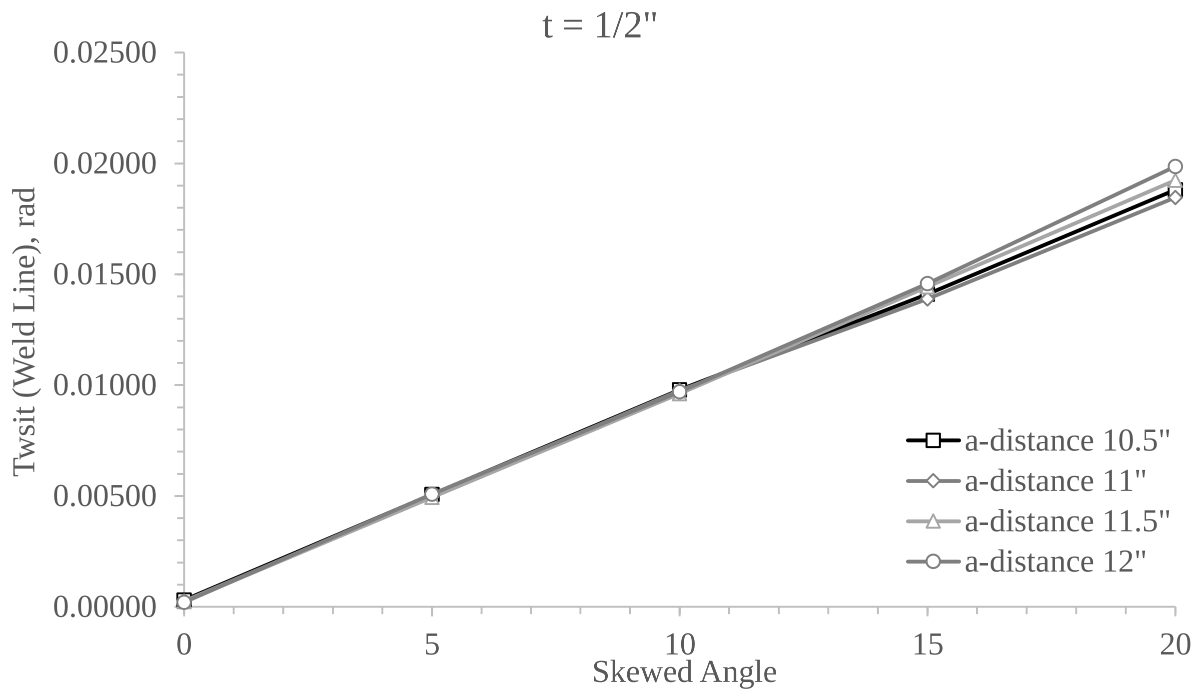


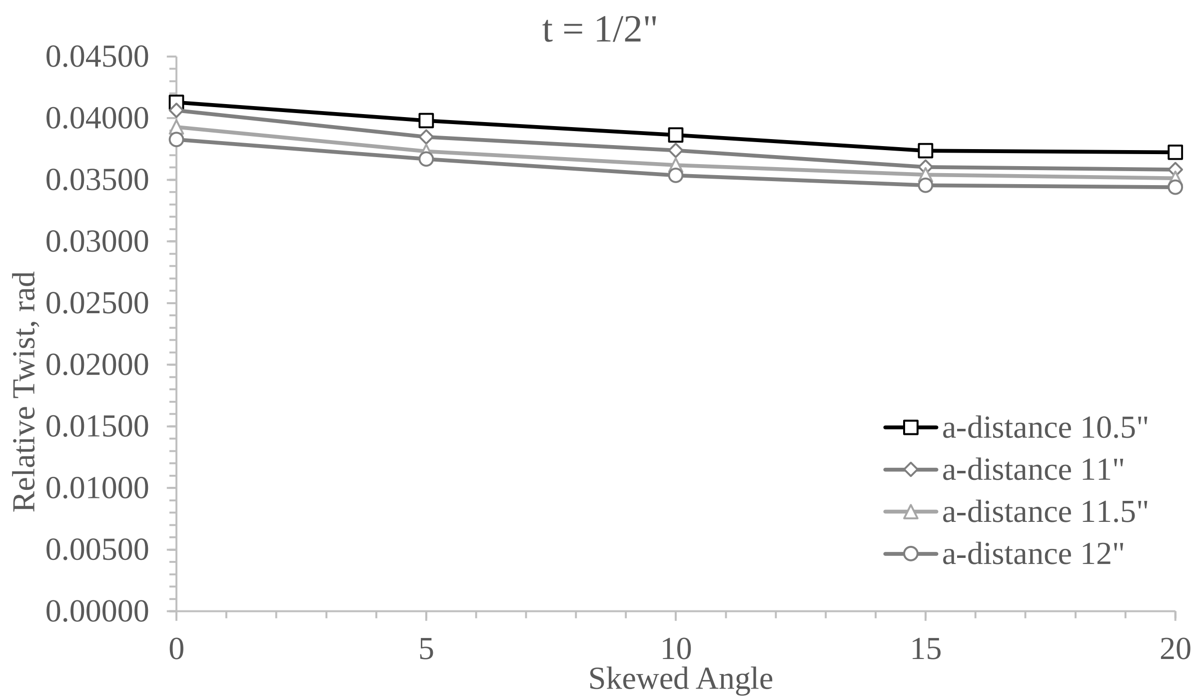
Plate Thickness = 7/16"
x 10^4
Connection Shear, lbs
9.2
9
8.8
8.6
8.4
8.2
8
7.8
a-distance, in
10.5
11
11.5
12
Skewed Angle, degrees
20
15
10
5
0
x 10^4
9
8.8
8.6
8.4
8.2
8

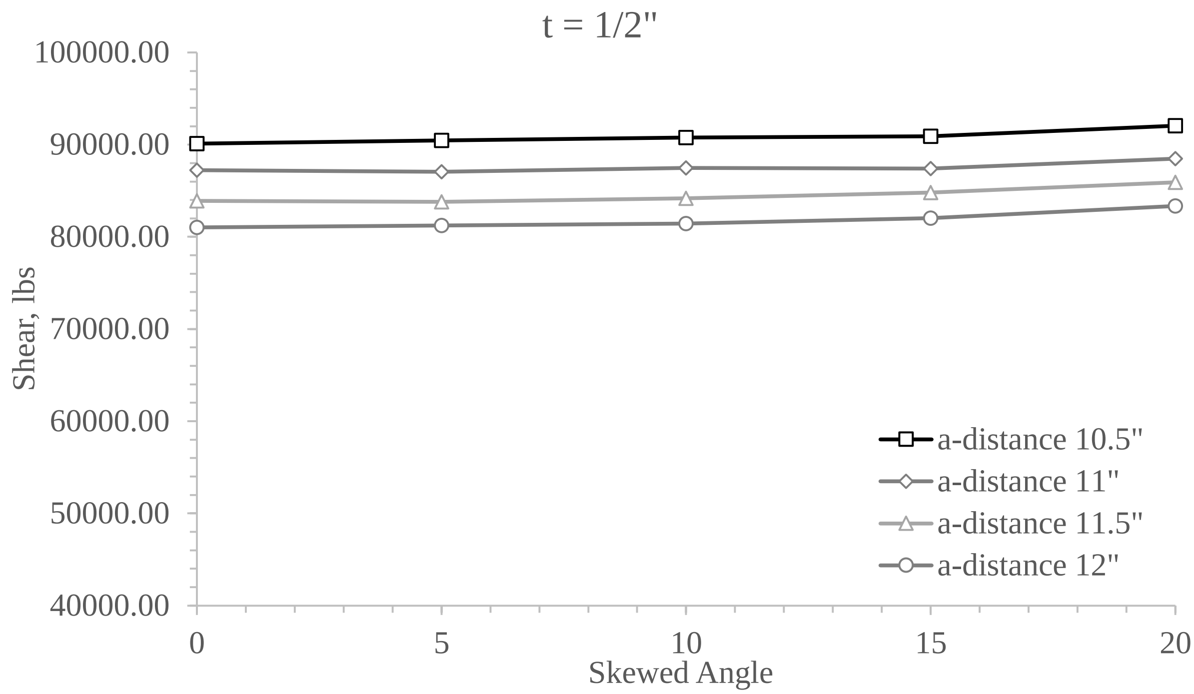
Five Bolted Connections (t = 1/2”) Curves






t = 1/2"
Twsit (Weld Line), rad
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 1/2"
Relative Twist, rad
0.04500
0.04000
0.03500
0.03000
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 1/2"
100000.00
90000.00
80000.00
70000.00
60000.00
50000.00
40000.00
Shear, lbs
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"

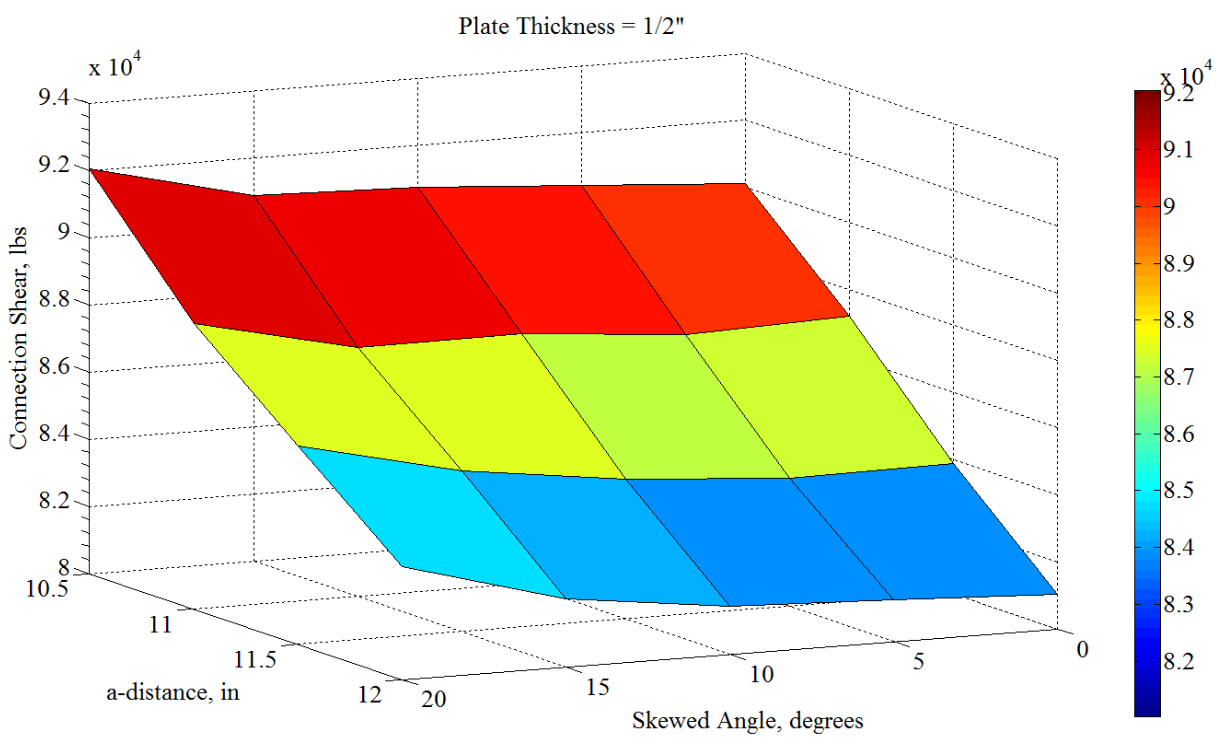
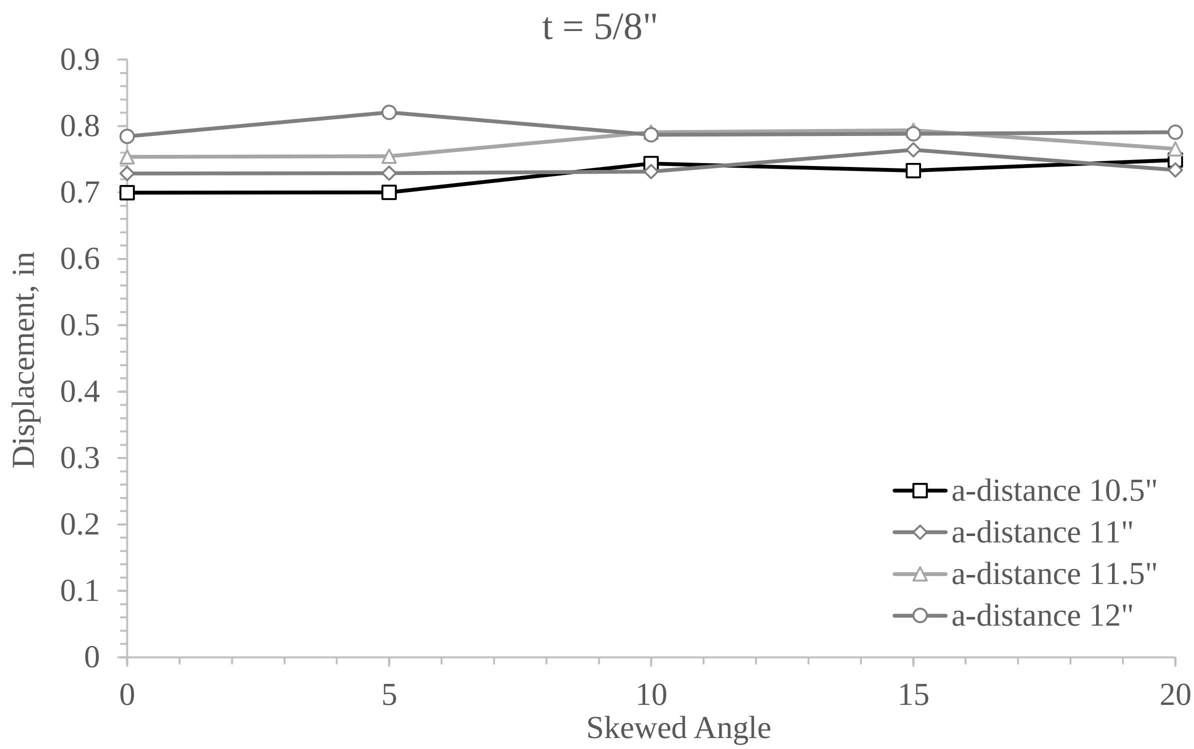
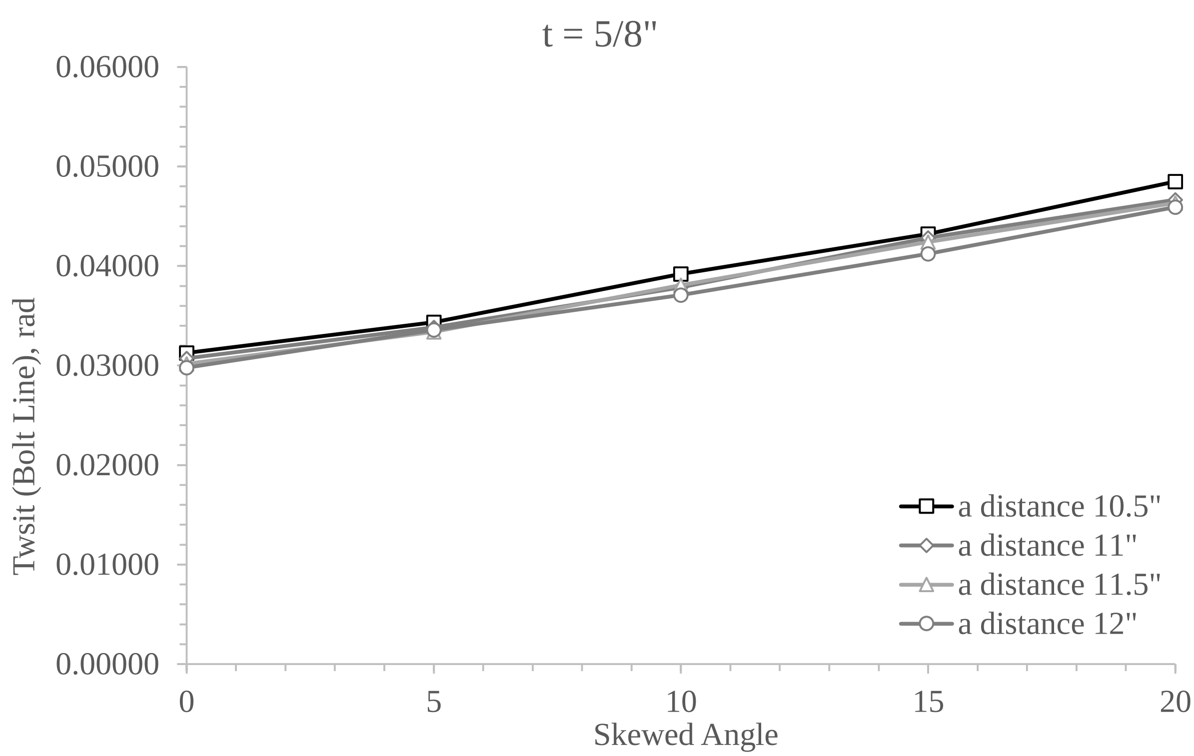
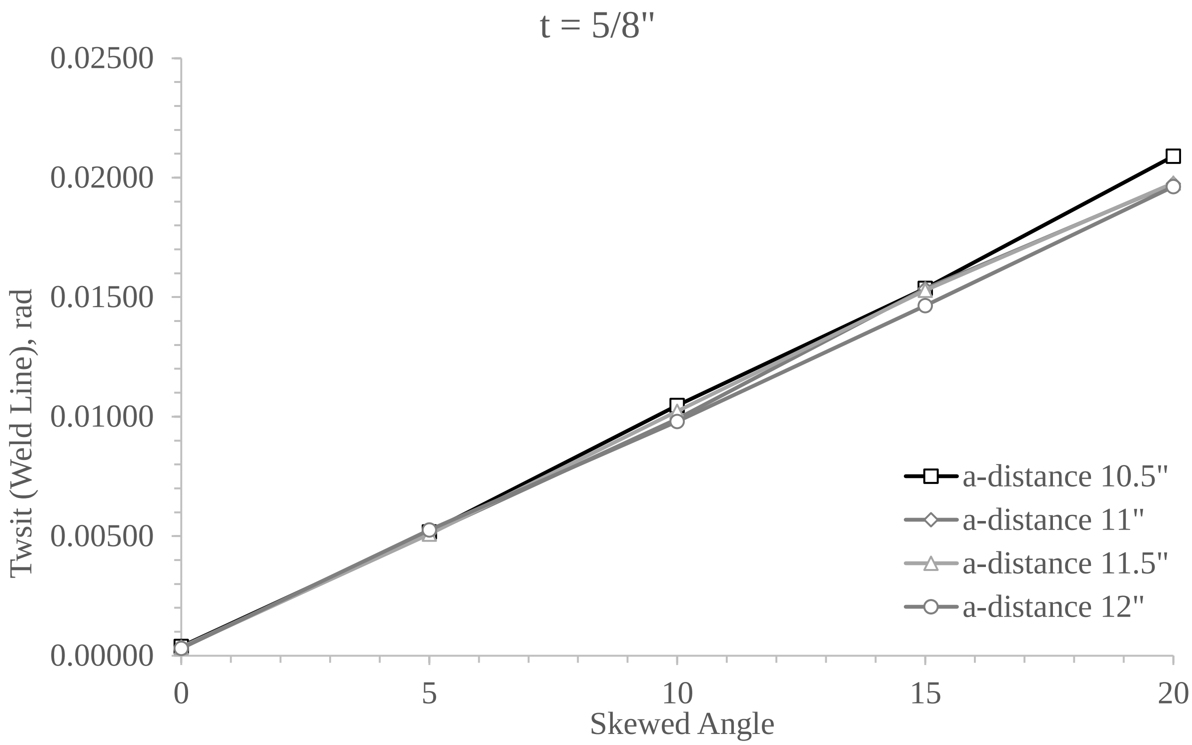


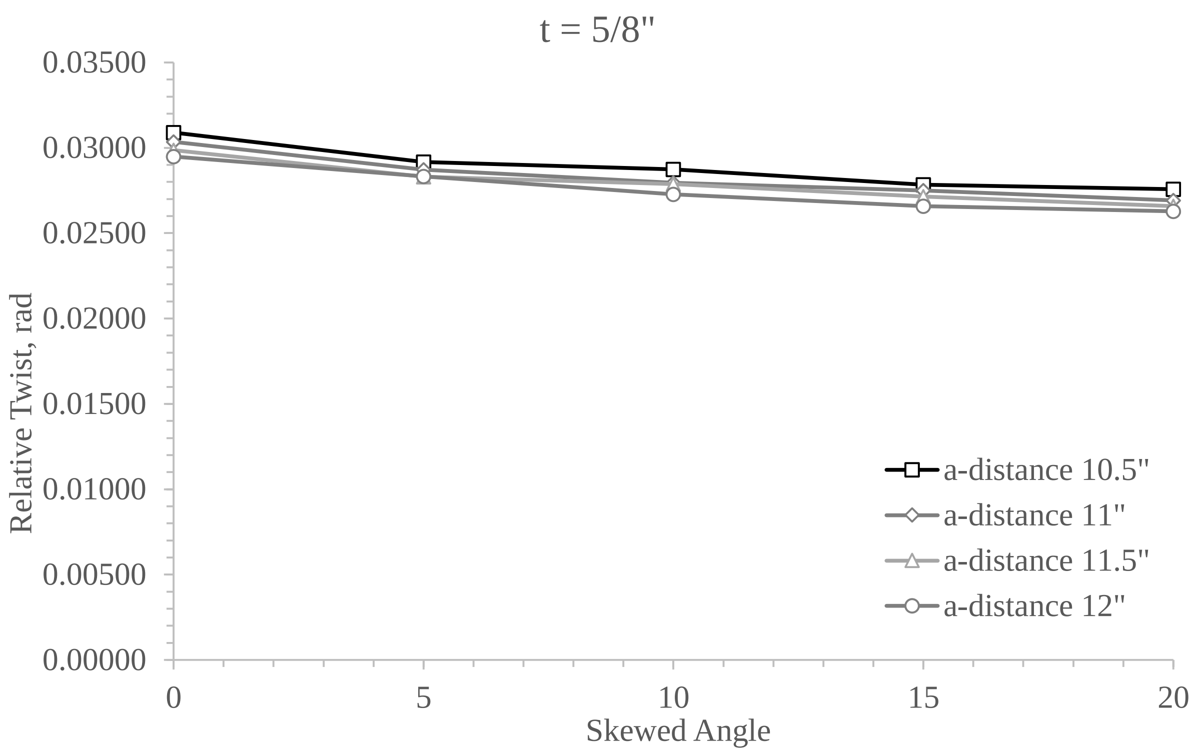
Plate Thickness = 1/2"
x 10^4
9.4
9.2
9
8.8
8.6
8.4
8.2
8
Connection Shear, lbs
10.5
11
11.5
12
a-distance, in
20
15
10
5
0
Skewed Angle, degrees
x 10^4
9.2
9.1
9
8.9
8.8
8.7
8.6
8.5
8.4
8.3
8.2

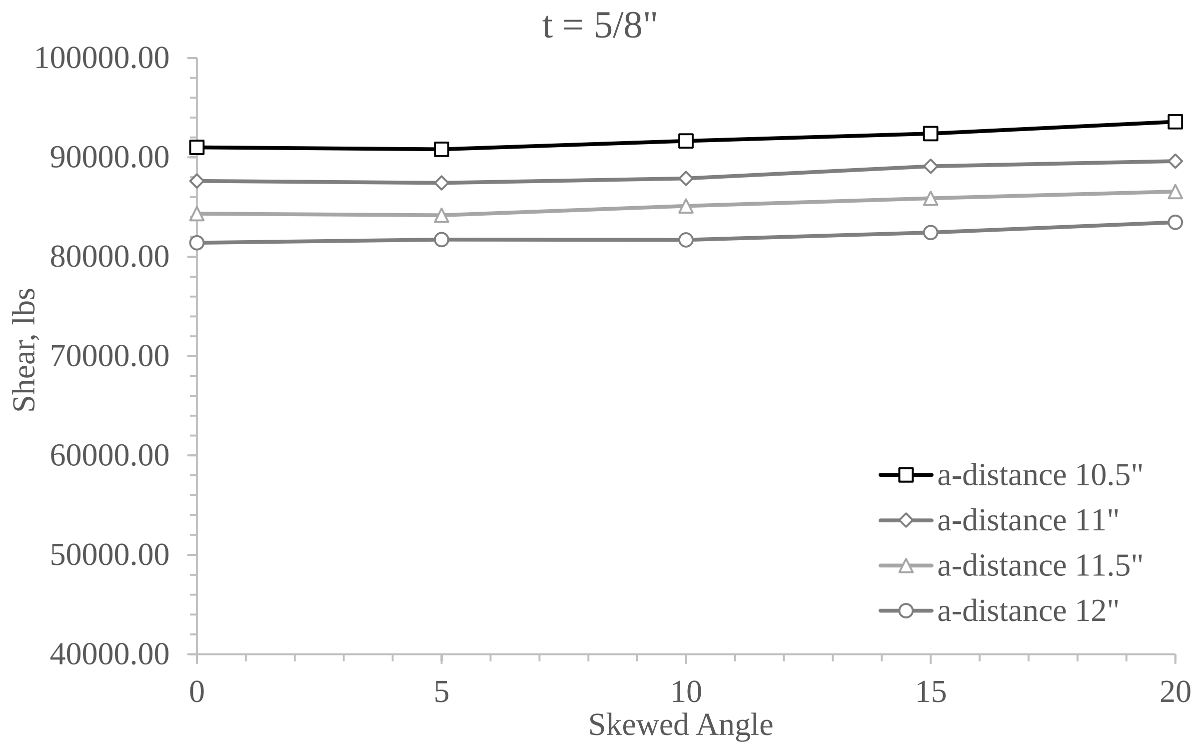
Five Bolted Connections (t = 5/8”) Curves






t = 5/8"
Twsit (Weld Line), rad
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 5/8"
Relative Twist, rad
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 5/8"
100000.00
90000.00
80000.00
70000.00
60000.00
50000.00
40000.00
Shear, lbs
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"

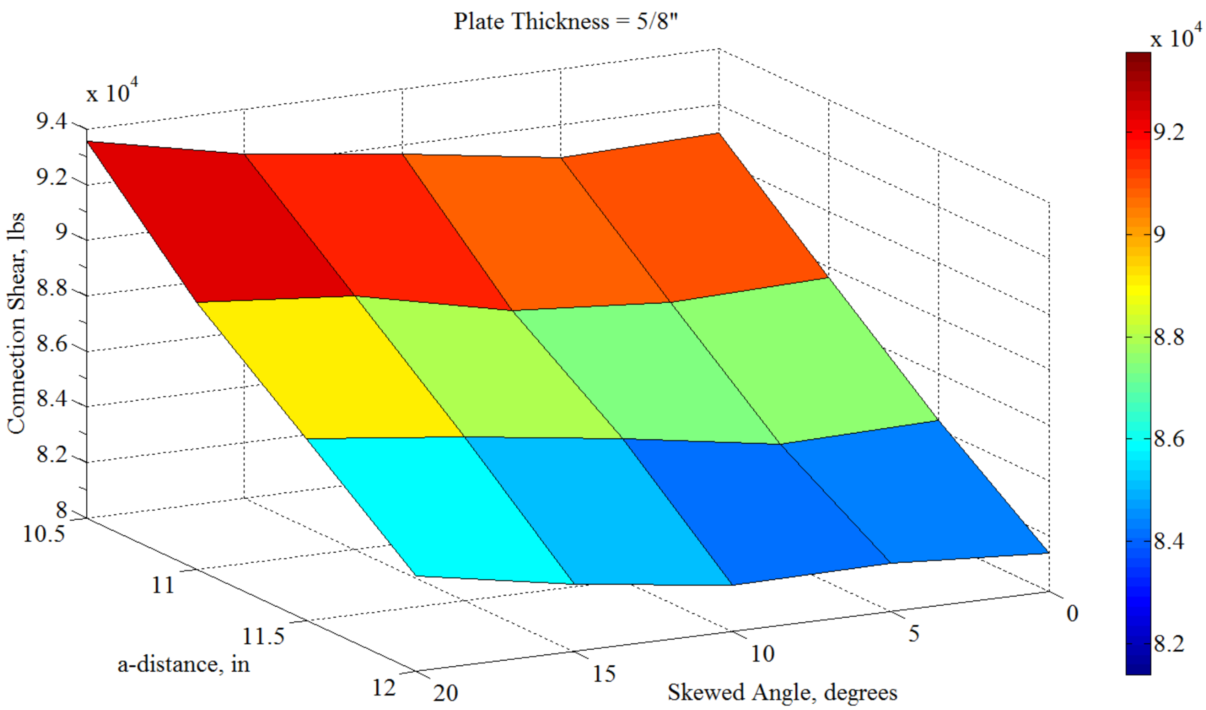
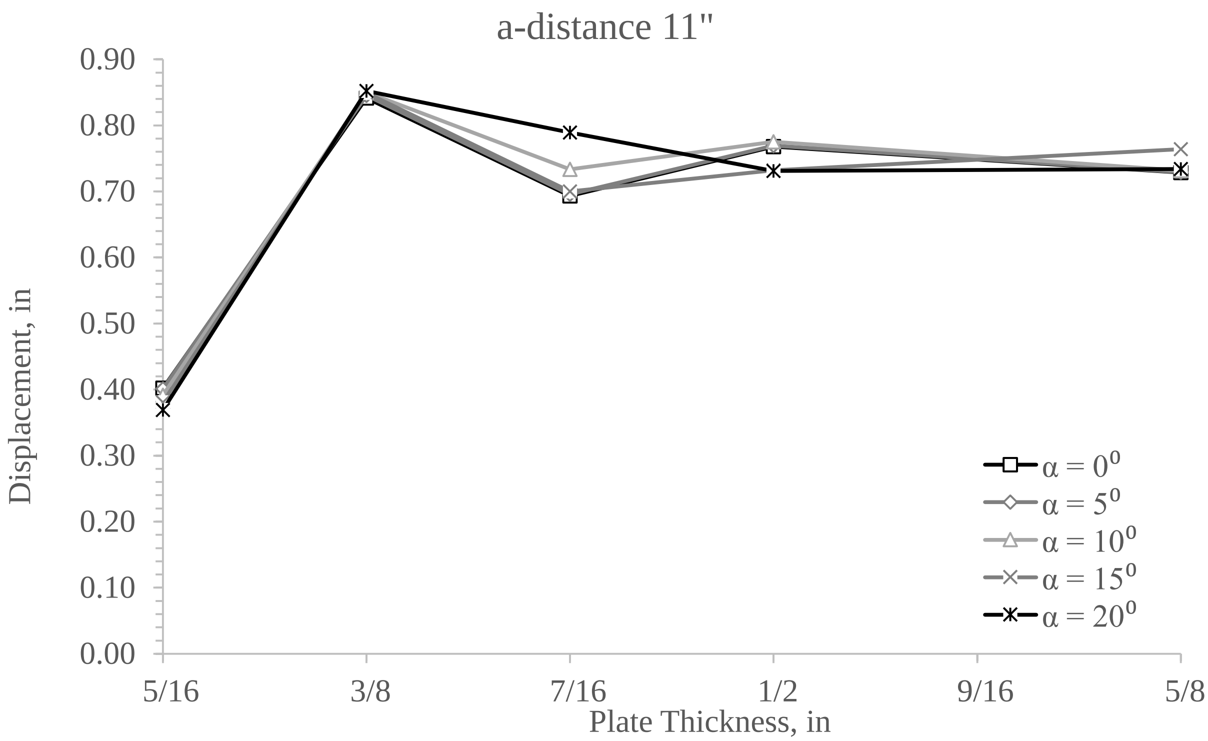
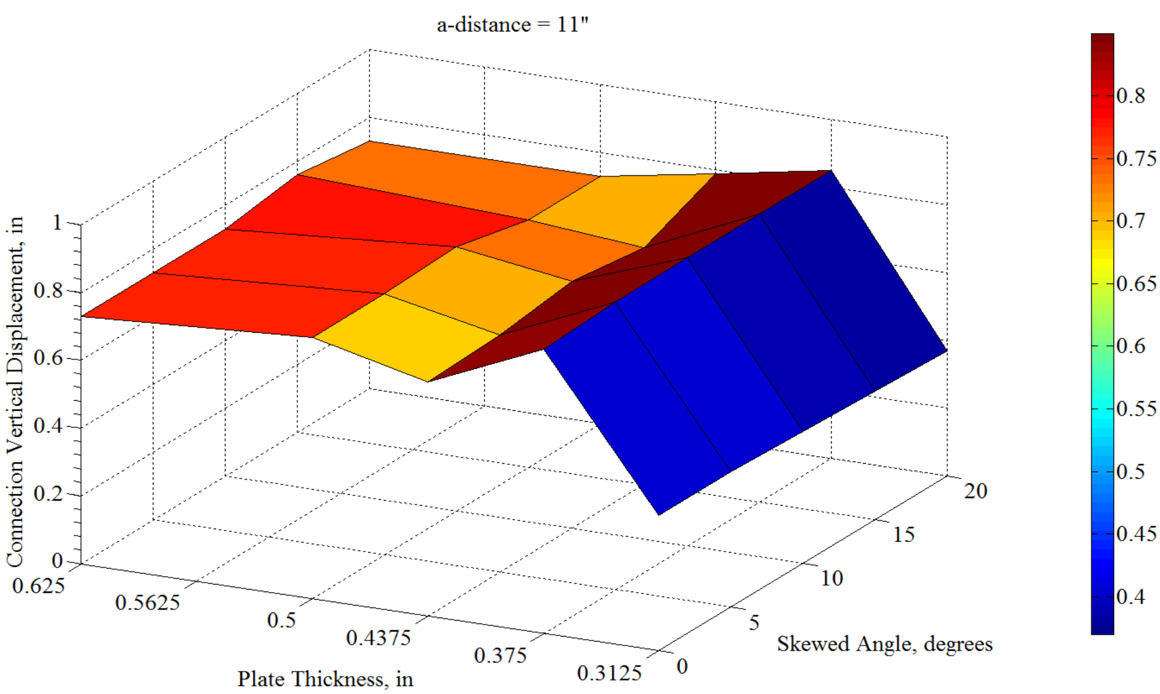
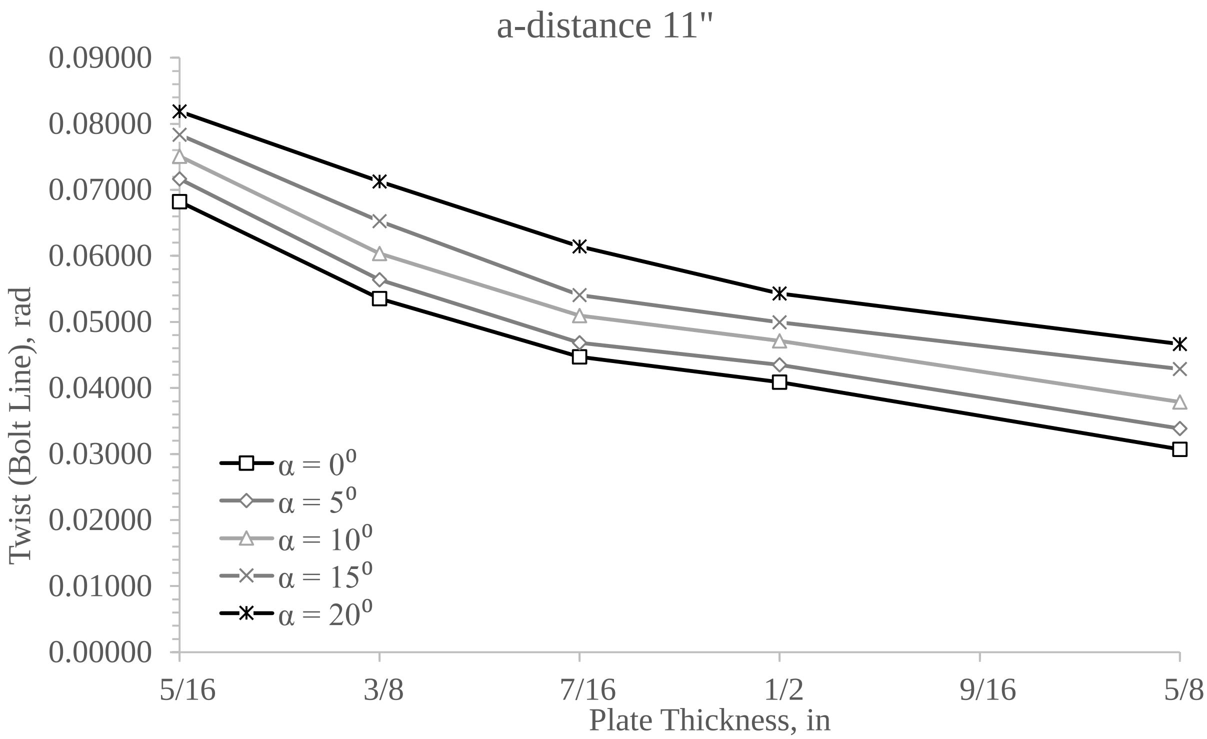


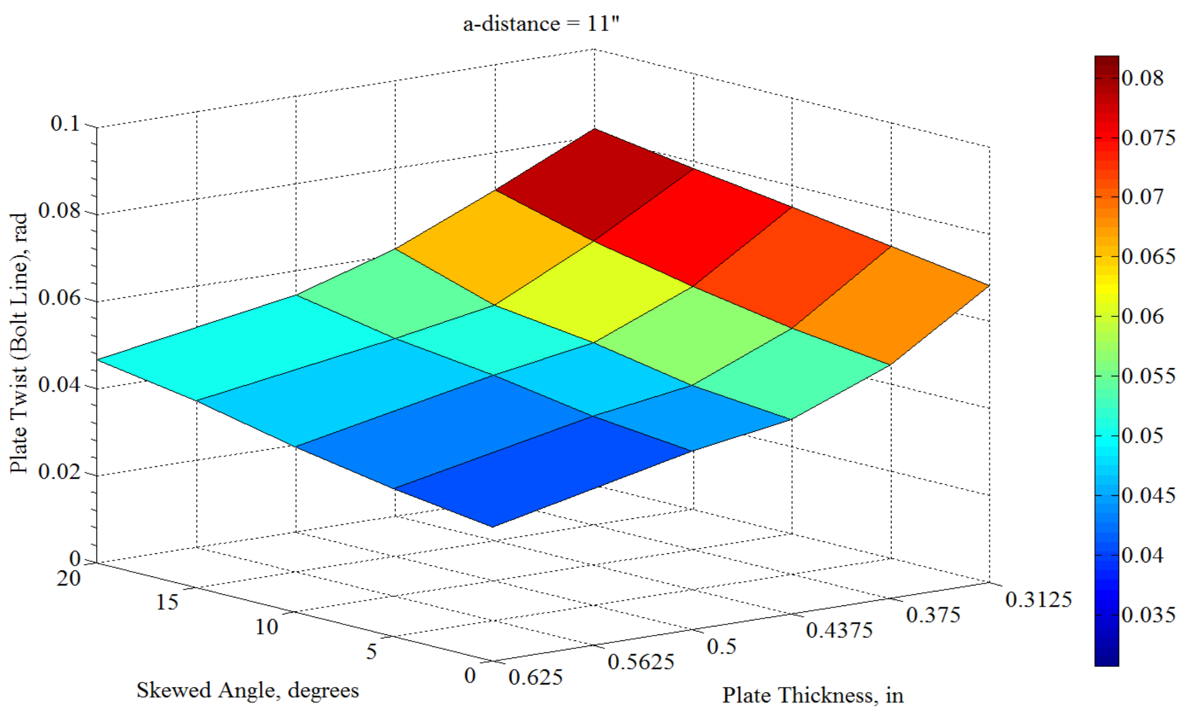
Plate Thickness = 5/8"
x 10^4
9.4
9.2
9
8.8
8.6
8.4
8.2
8
Connection Shear, lbs
10.5
11
11.5
12
a-distance, in
20
15
10
5
0
Skewed Angle, degrees
x 10^4
9.2
9
8.8
8.6
8.4
8.2

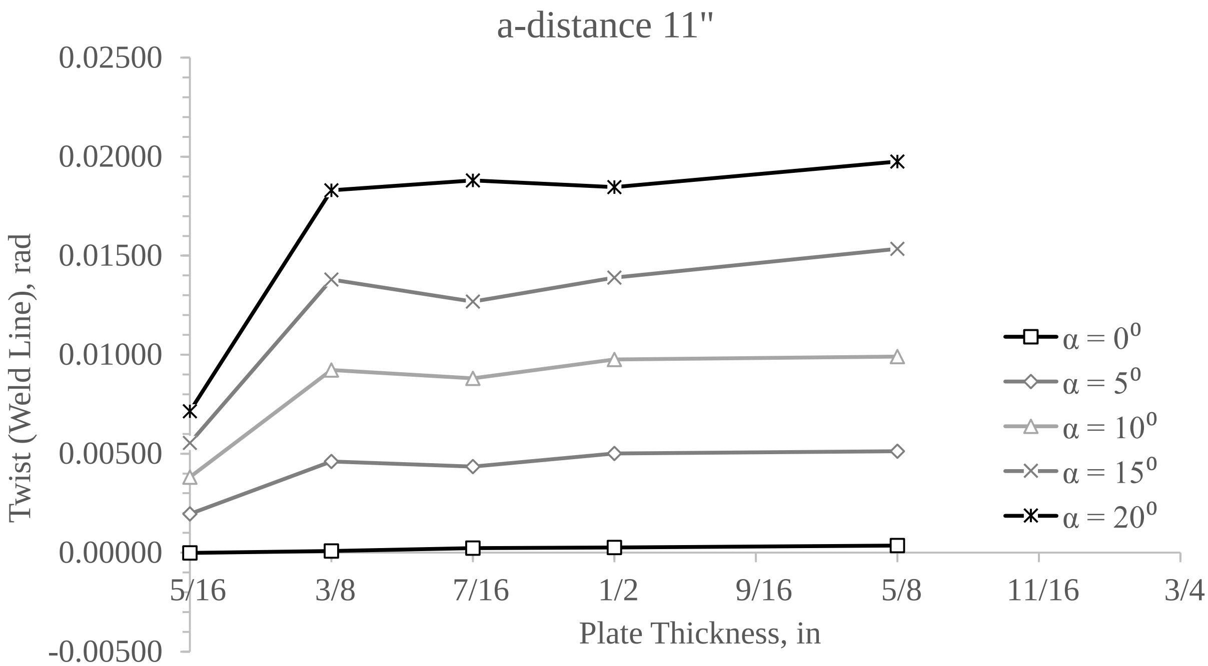
Five Bolted Connections (a-distance = 11”) Curves

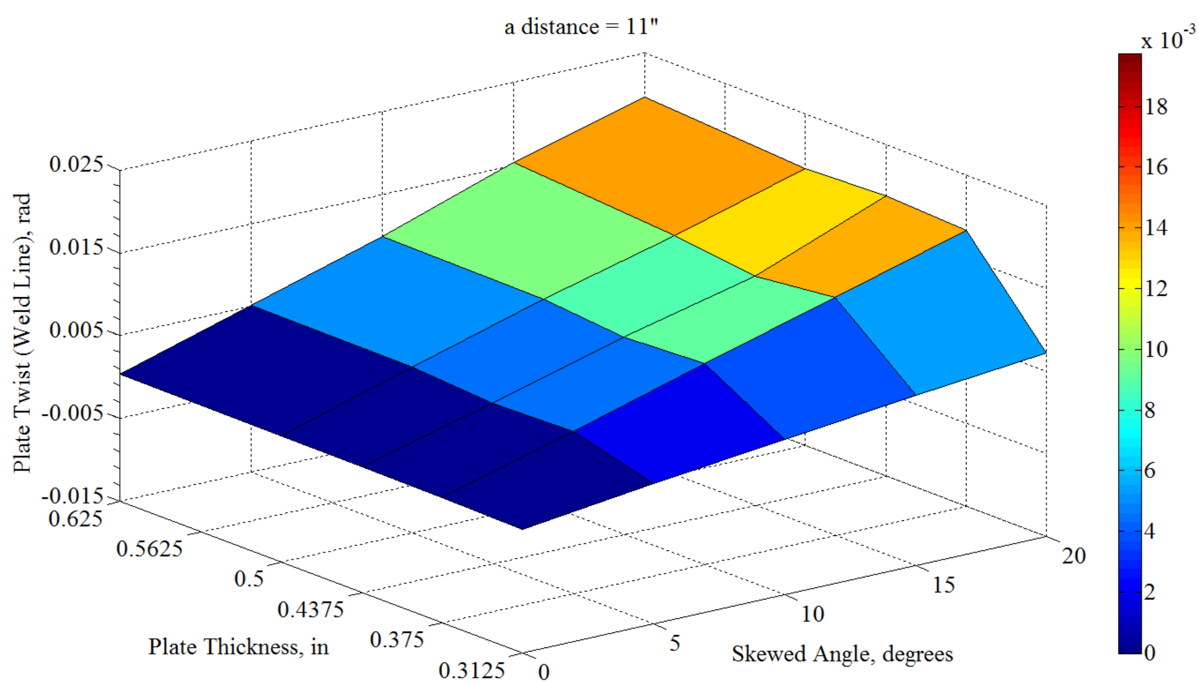


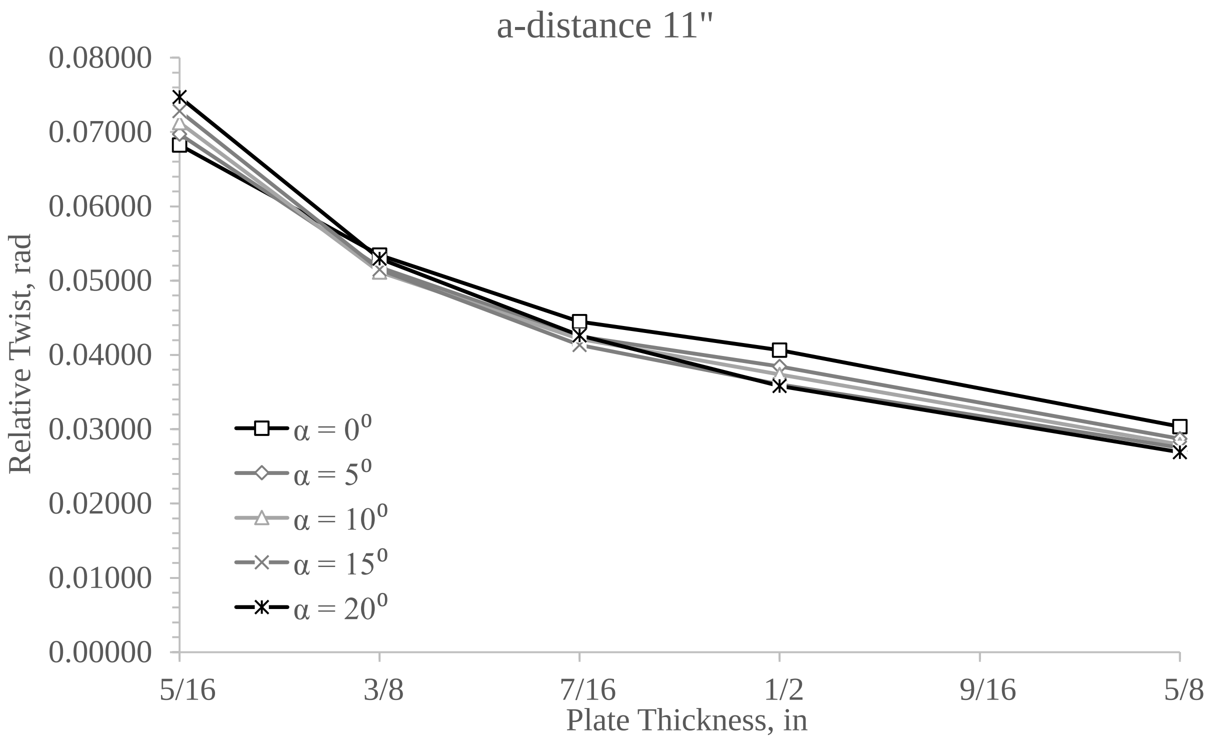



a-distance 11"
0.09000
0.08000
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
Twist (Bolt Line), rad
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in

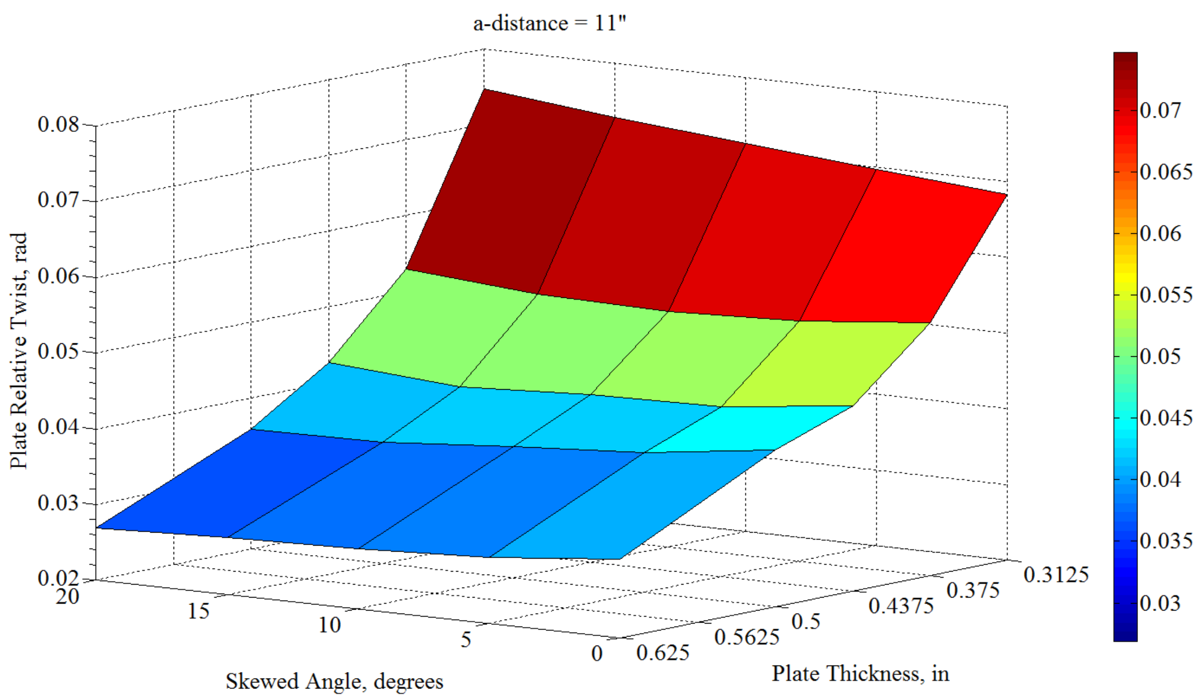

a-distance = 11"
0.1
0.08
0.06
0.04
0.02
0
Plate Twist (Bolt Line), rad
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in
0.08
0.075
0.07
0.065
0.06
0.055
0.05
0.045
0.04
0.035


a-distance 11"
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
-0.00500
Twist (Weld Line), rad
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness, in
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰


a distance = 11"
0.025
0.015
0.005
-0.005
-0.015
Plate Twist (Weld Line), rad
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in
0
5
10
15
20
Skewed Angle, degrees
x 10-3
18
16
14
12
10
8
6
4
2
0

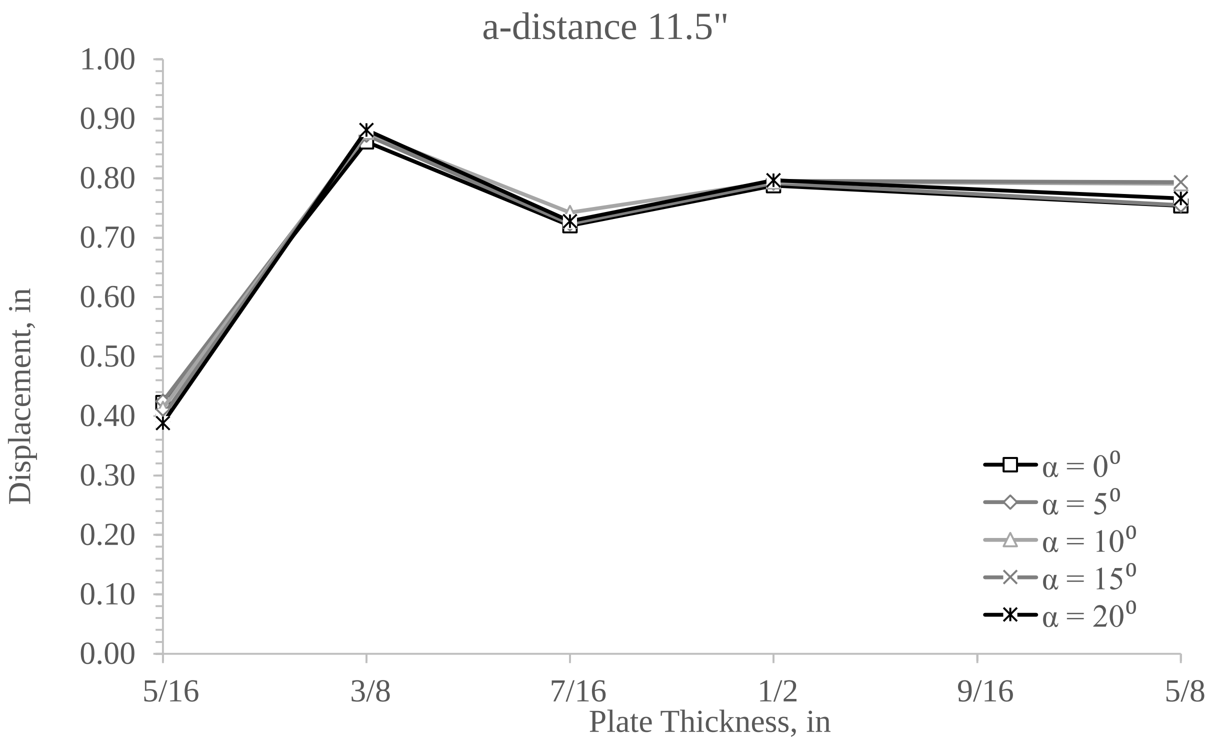


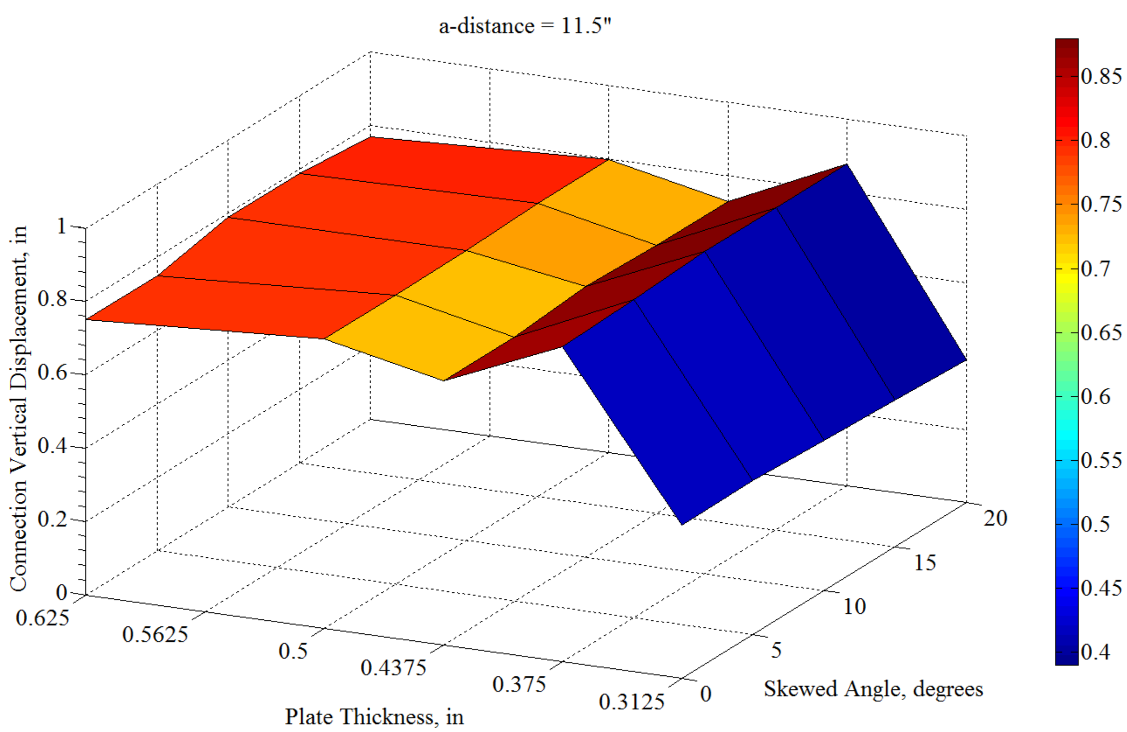
a-distance 11"
0.08000
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
Relative Twist, rad
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in

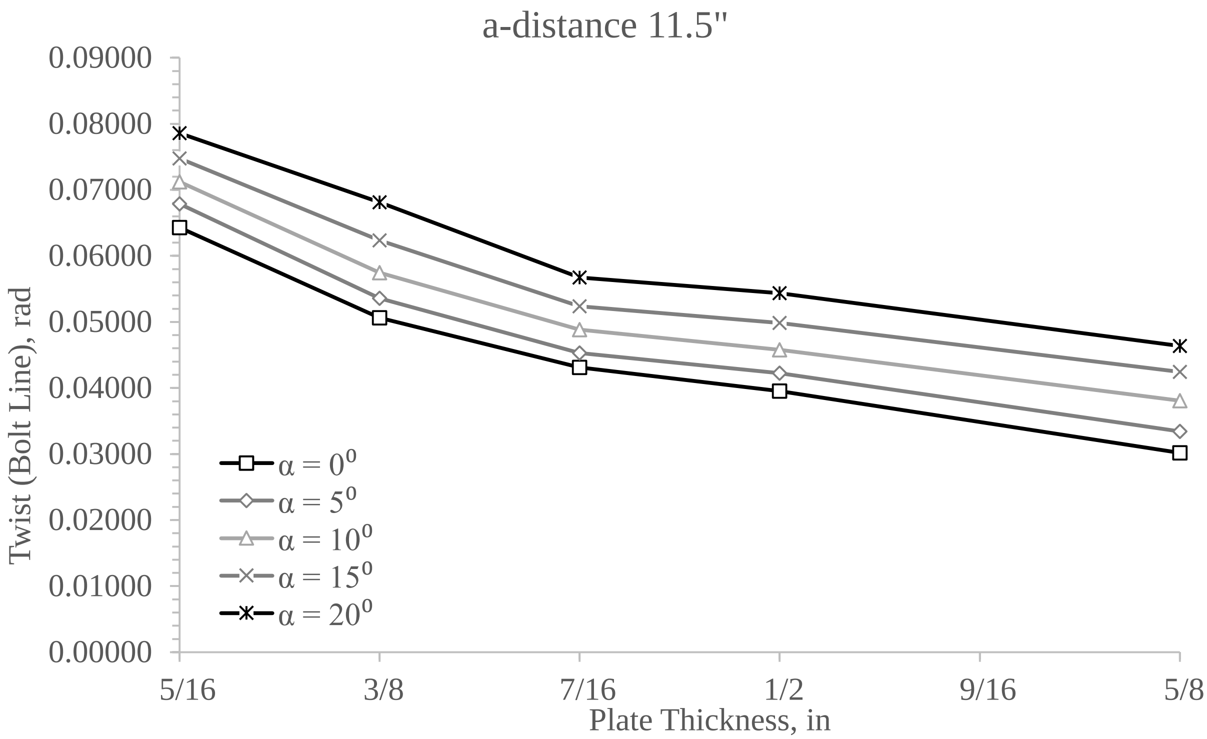


a-distance = 11"
0.08
0.07
0.06
0.05
0.04
0.03
0.02
Plate Relative Twist, rad
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in
0.07
0.065
0.06
0.055
0.05
0.045
0.04
0.035
0.03

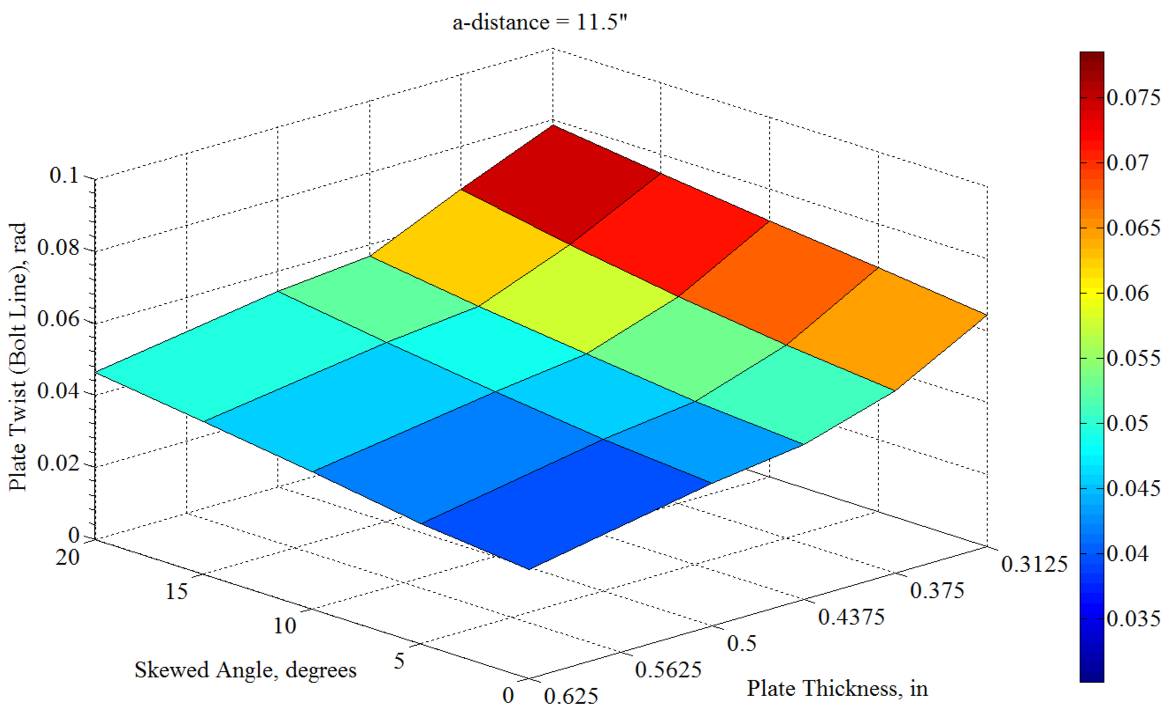
Five Bolted Connections (a-distance = 11.5”) Curves



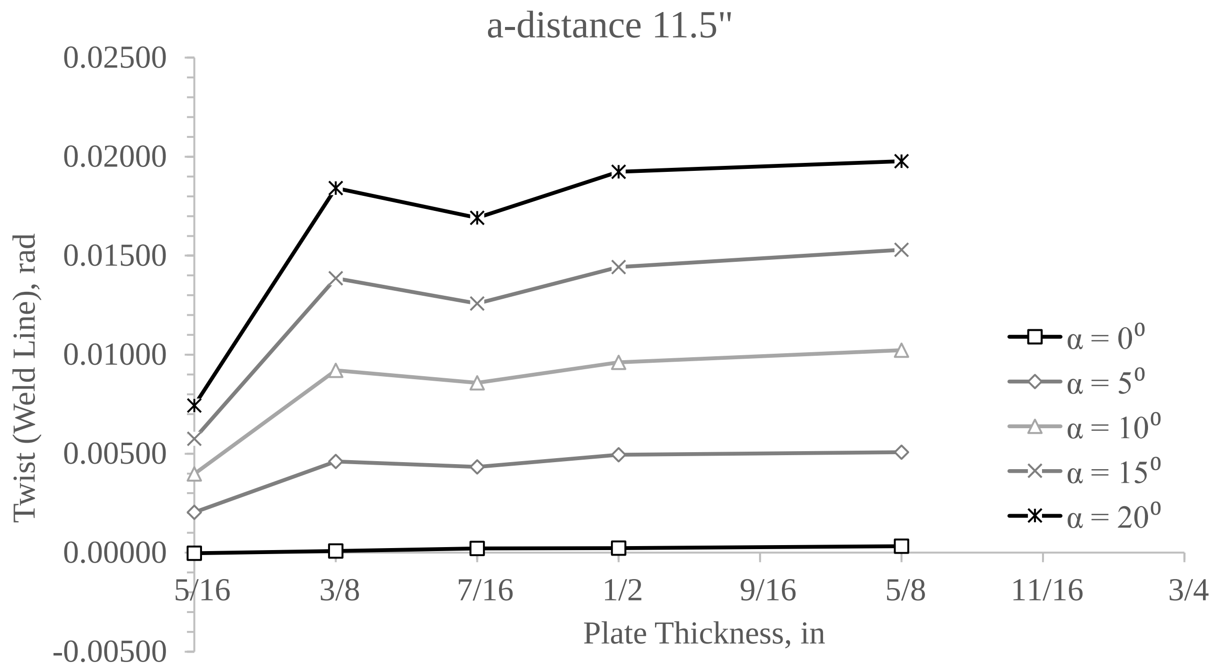


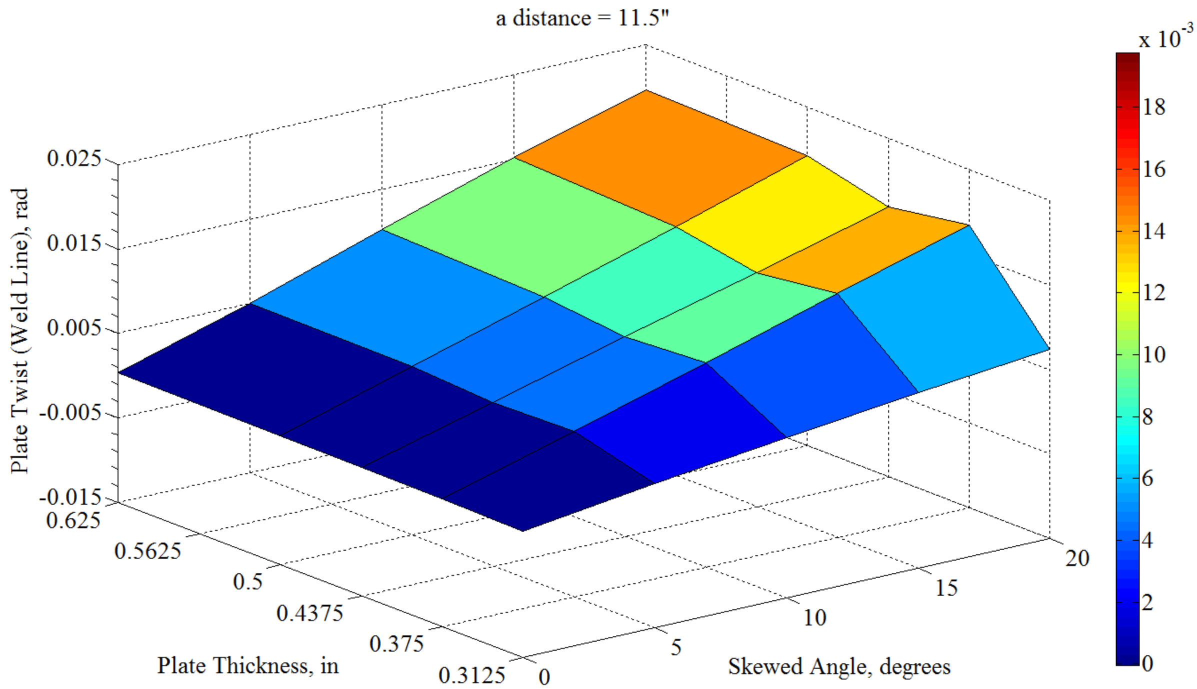


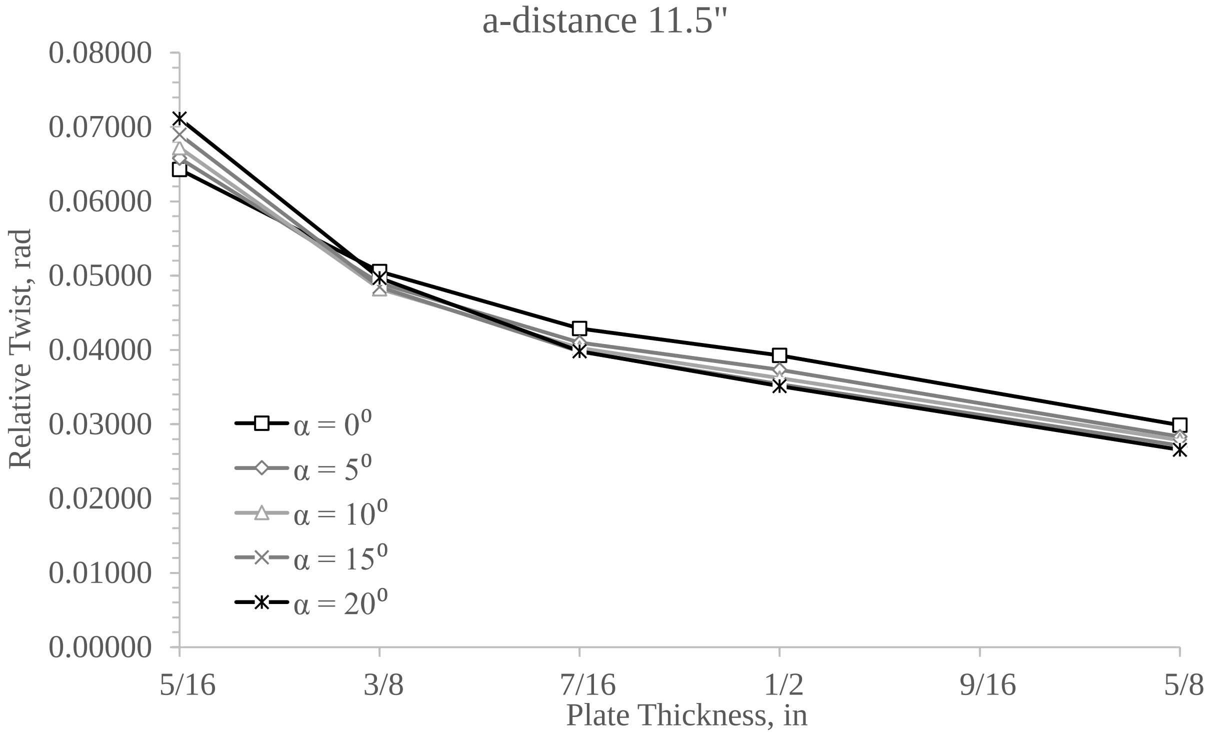
a-distance 11.5"
Twist (Bolt Line), rad
0.09000
0.08000
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

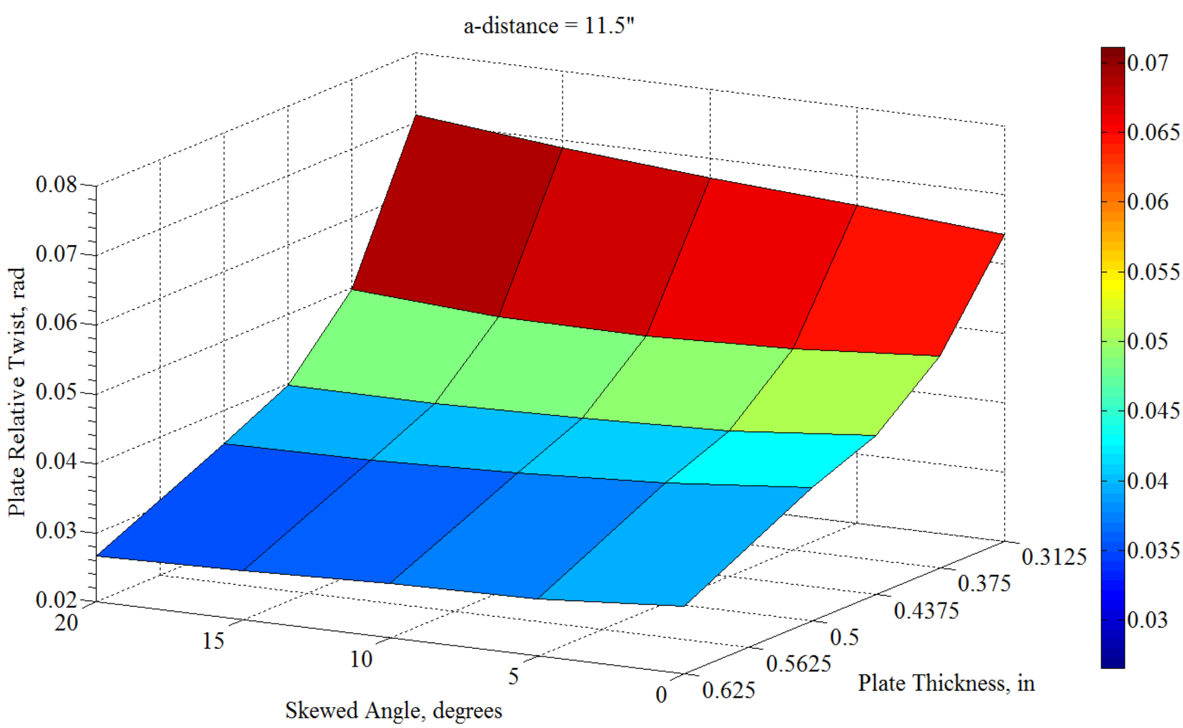


a-distance = 11.5"
Plate Twist (Bolt Line), rad
0.1
0.08
0.06
0.04
0.02
0
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in
0.075
0.07
0.065
0.06
0.055
0.05
0.045
0.04
0.035

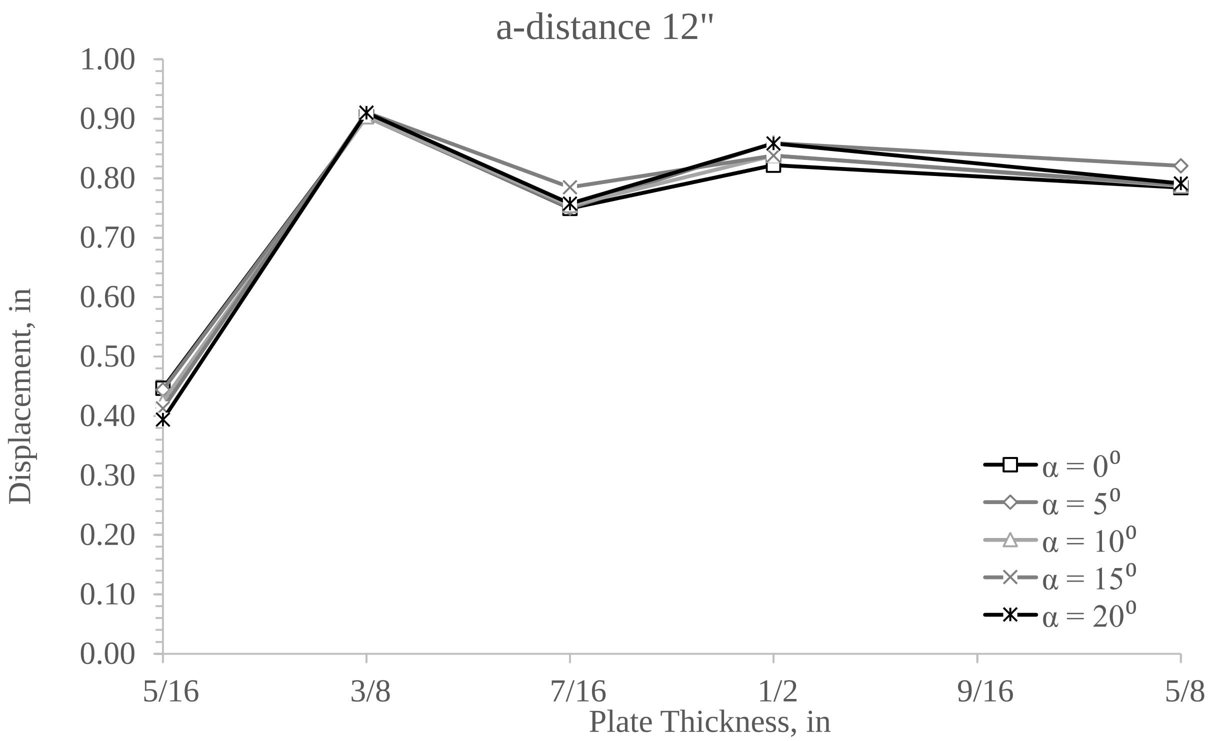

a-distance 11.5"
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
-0.00500
Twist (Weld Line), rad
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness, in
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

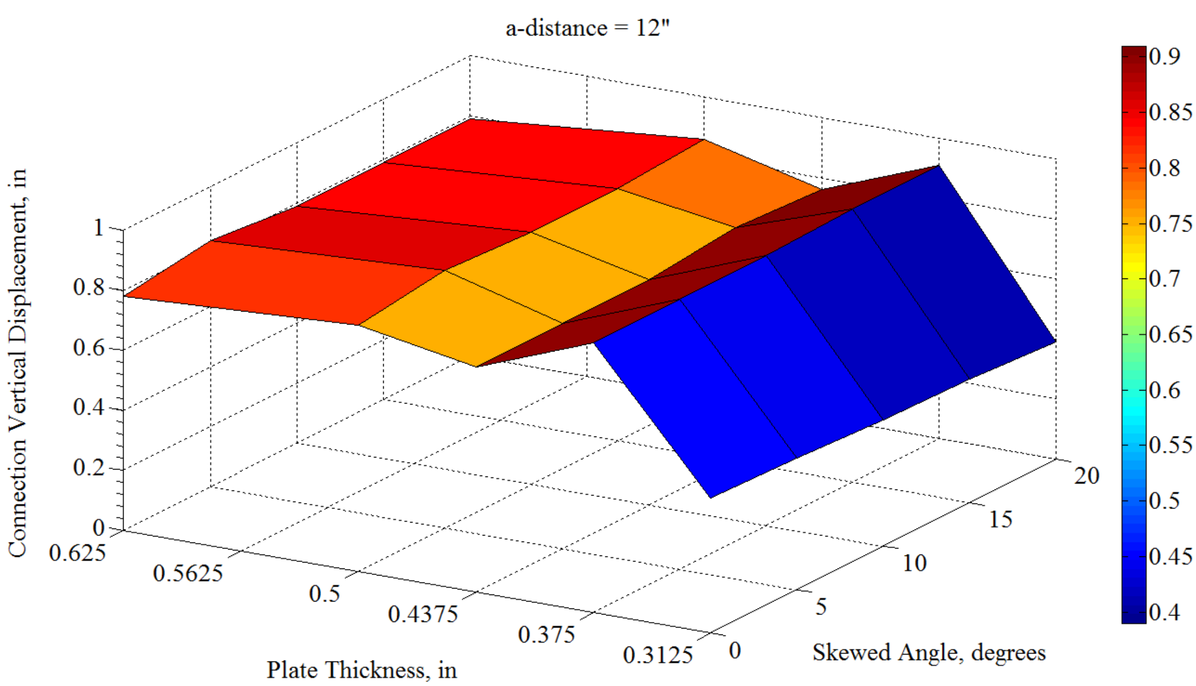

a distance = 11.5"
Plate Twist (Weld Line), rad
0.025
0.015
0.005
-0.005
-0.015
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in
0
5
10
15
20
Skewed Angle, degrees
x 10-3
18
16
14
12
10
8
6
4
2
0

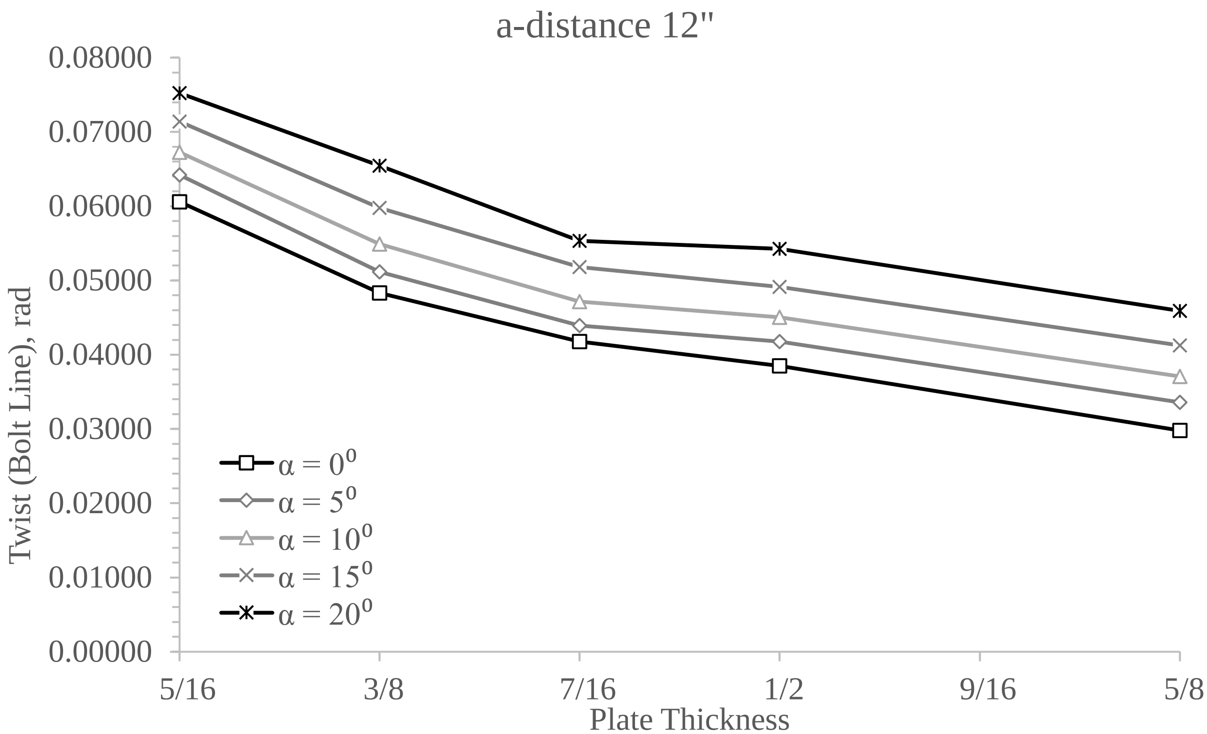

a-distance 11.5"
0.08000
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
Relative Twist, rad
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in

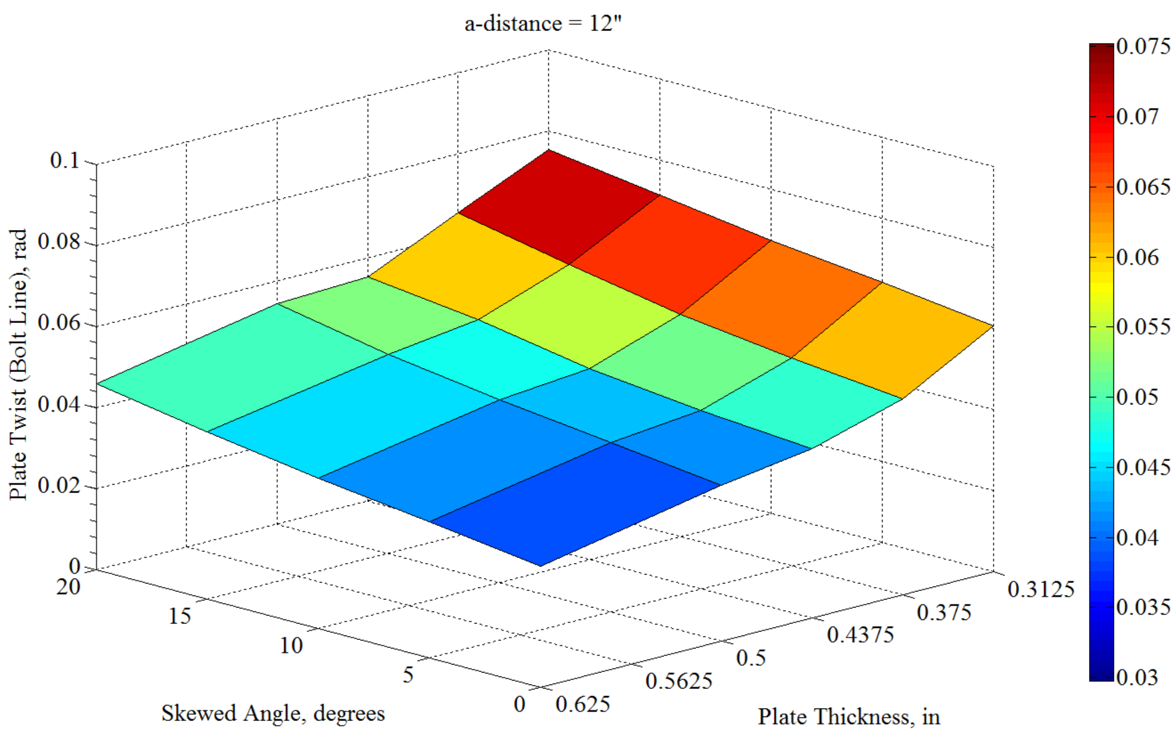

a-distance = 11.5"
Plate Relative Twist, rad
0.08
0.07
0.06
0.05
0.04
0.03
0.02
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in
0.07
0.065
0.06
0.055
0.05
0.045
0.04
0.035
0.03

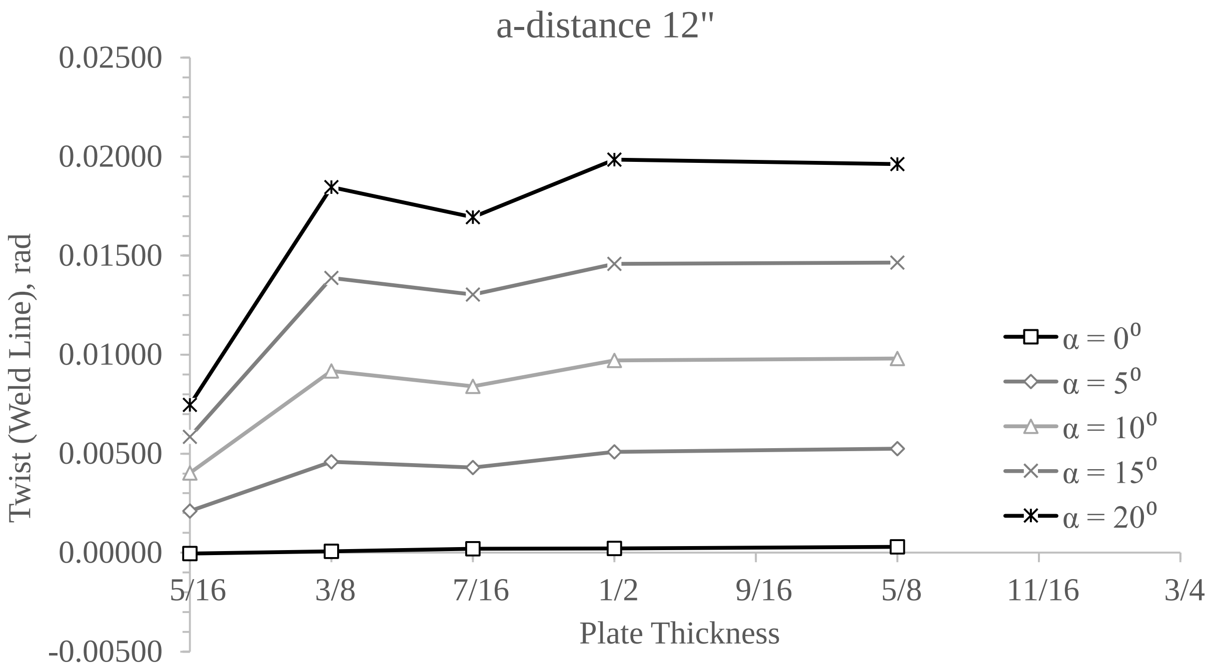
Five Bolted Connections (a-distance = 12”) Curves

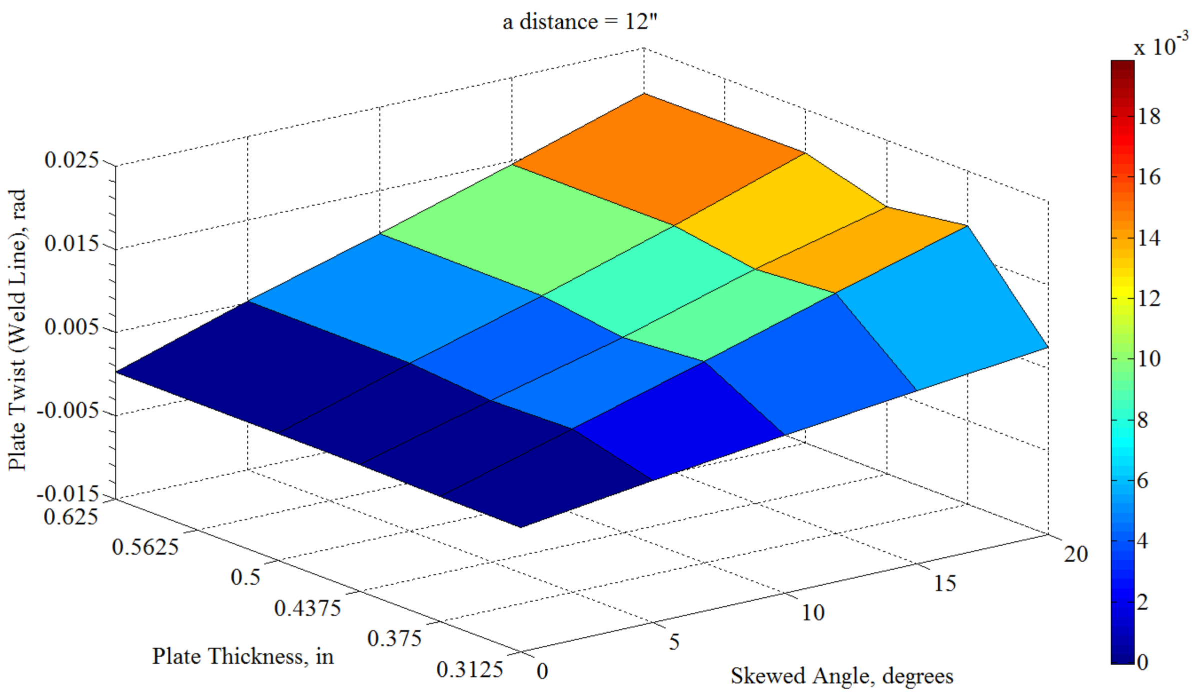


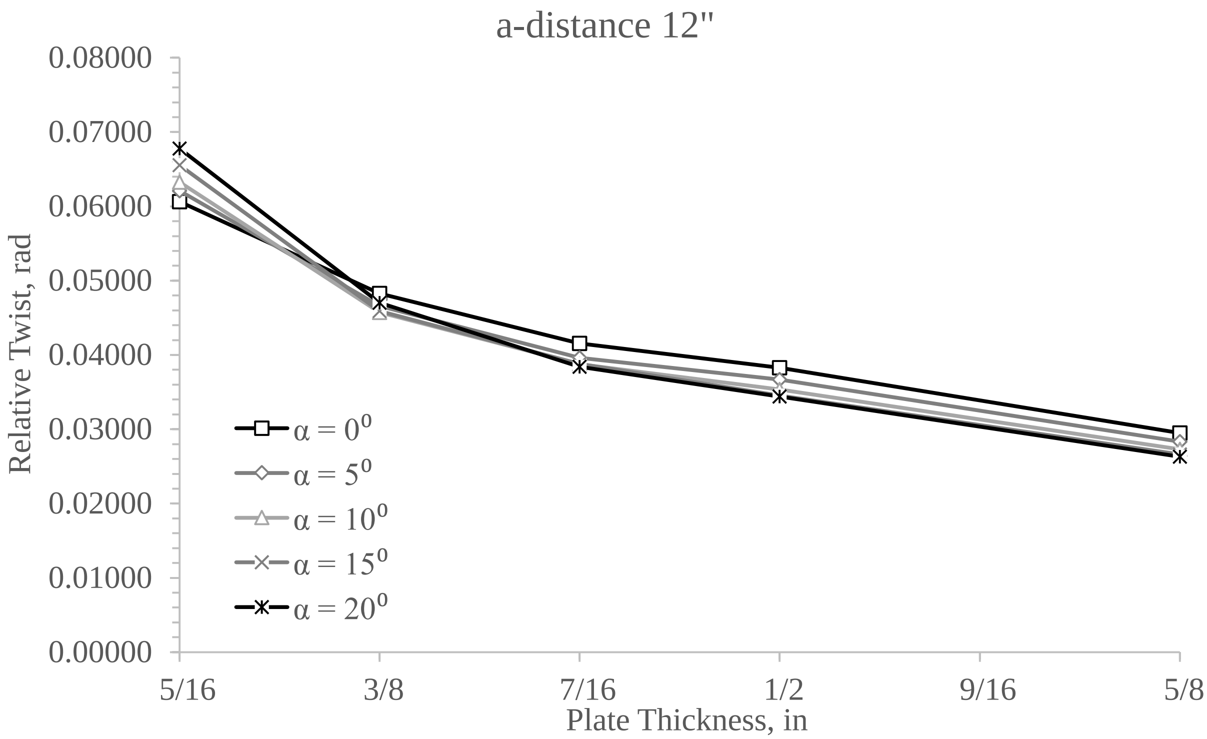



a-distance 12"
0.08000
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
Twist (Bolt Line), rad
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness

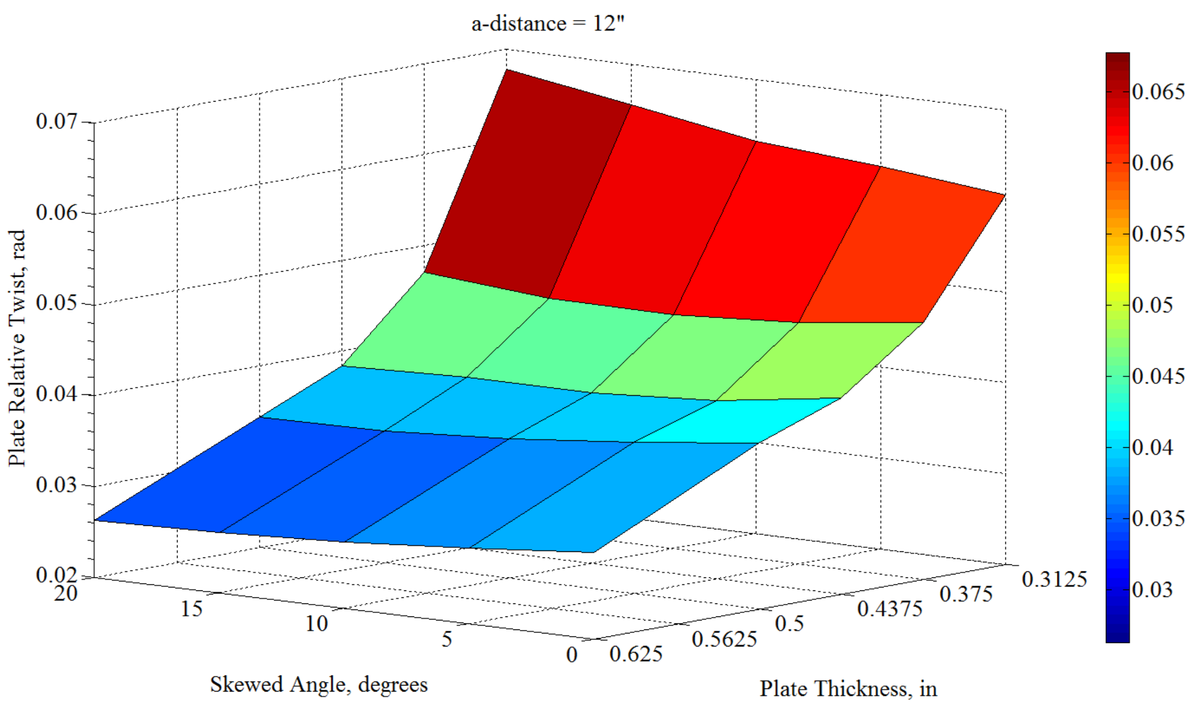

a-distance = 12"
Plate Twist (Bolt Line), rad
Skewed Angle, degrees
Plate Thickness, in
0.3125
0.375
0.4375
0.5
0.5625
0.625


a-distance 12"
Twist (Weld Line), rad
0.02500
0.02000
0.01500
0.01000
0.00500
0.00000
-0.00500
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰


a distance = 12"
Plate Twist (Weld Line), rad
0.025
0.015
0.005
-0.005
-0.015
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in
0
5
10
15
20
Skewed Angle, degrees
x 10-3
18
16
14
12
10
8
6
4
2
0

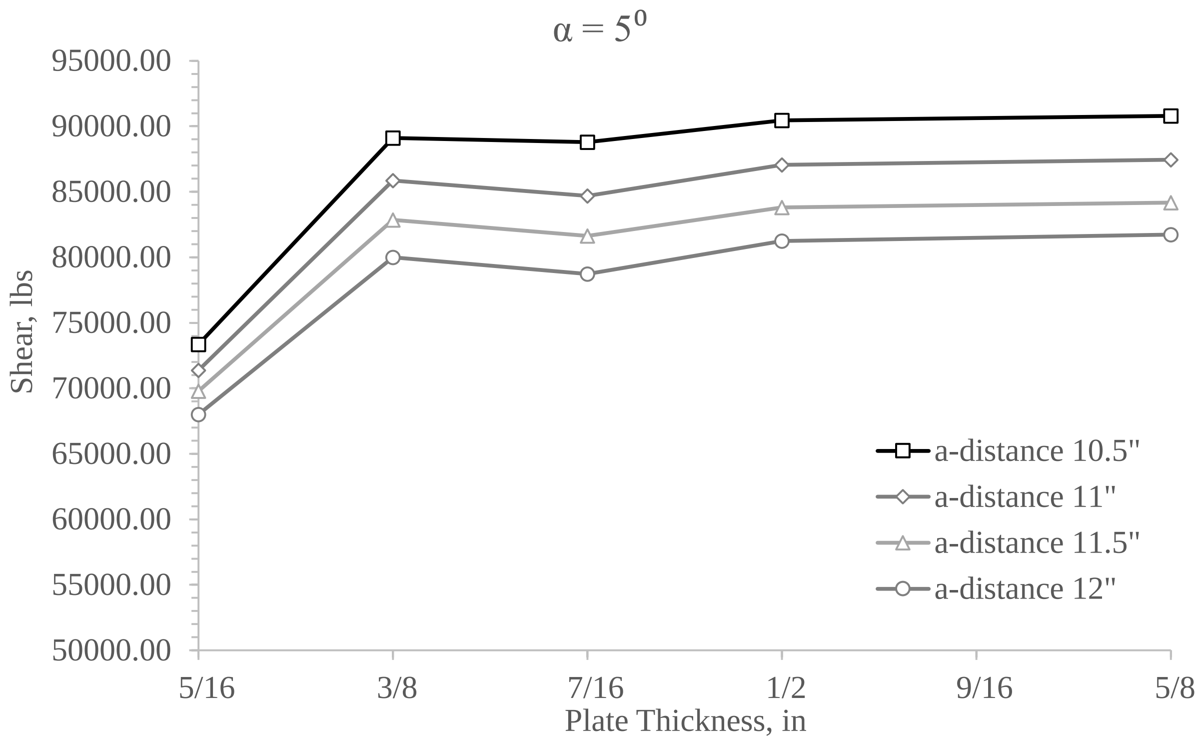


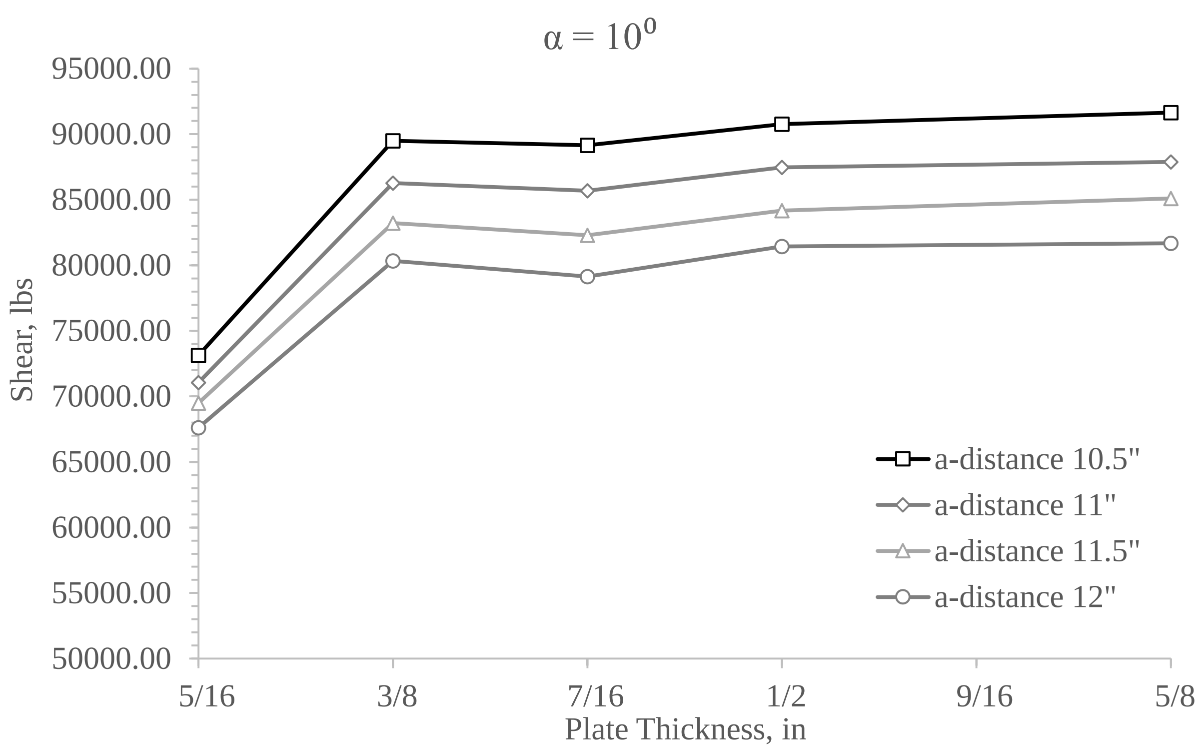
a-distance 12"
0.08000
0.07000
0.06000
0.05000
0.04000
0.03000
0.02000
0.01000
0.00000
Relative Twist, rad
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in

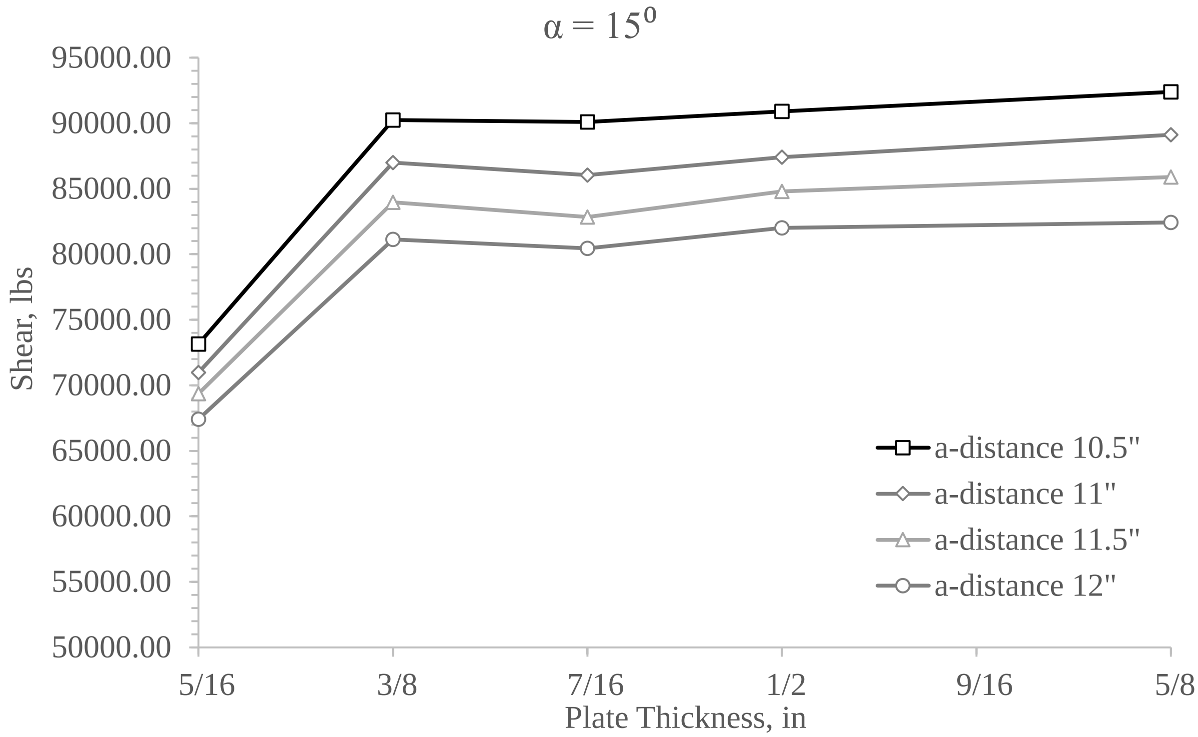


a-distance = 12"
Plate Relative Twist, rad
0.07
0.06
0.05
0.04
0.03
0.02
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in
0.065
0.06
0.055
0.05
0.045
0.04
0.035
0.03

Five Bolted Connections (5^0, 10^0, and 15^0) Curves

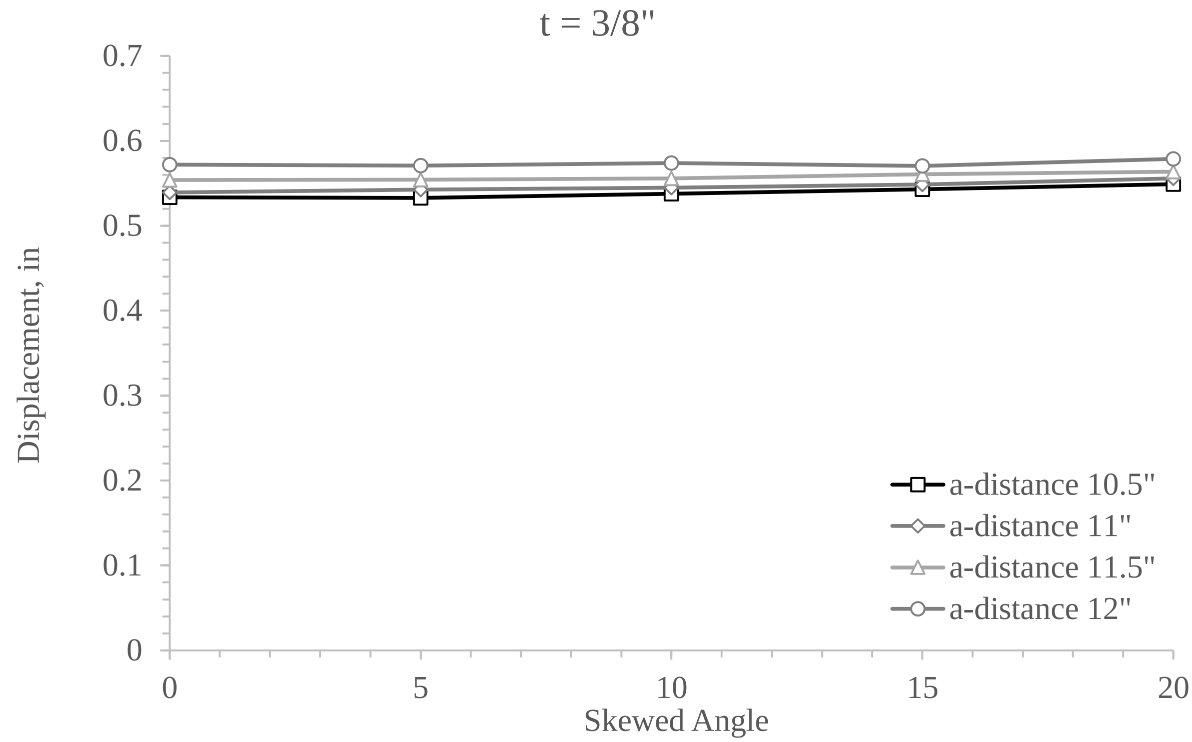


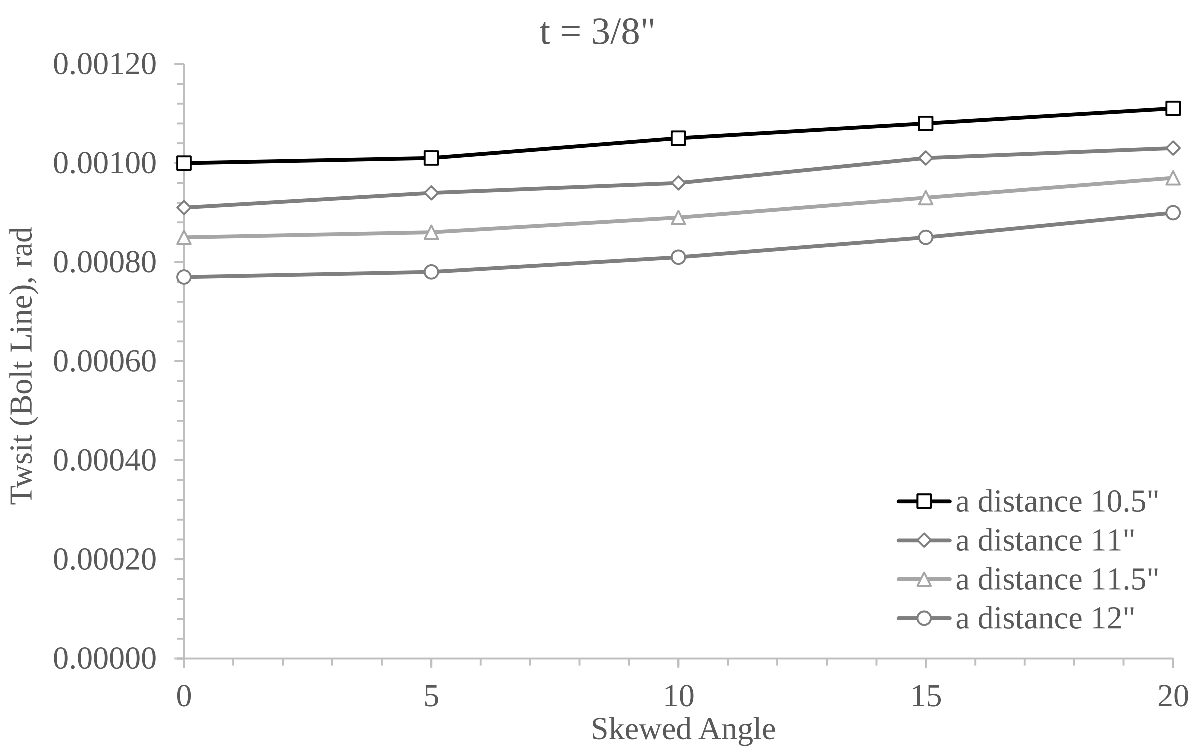


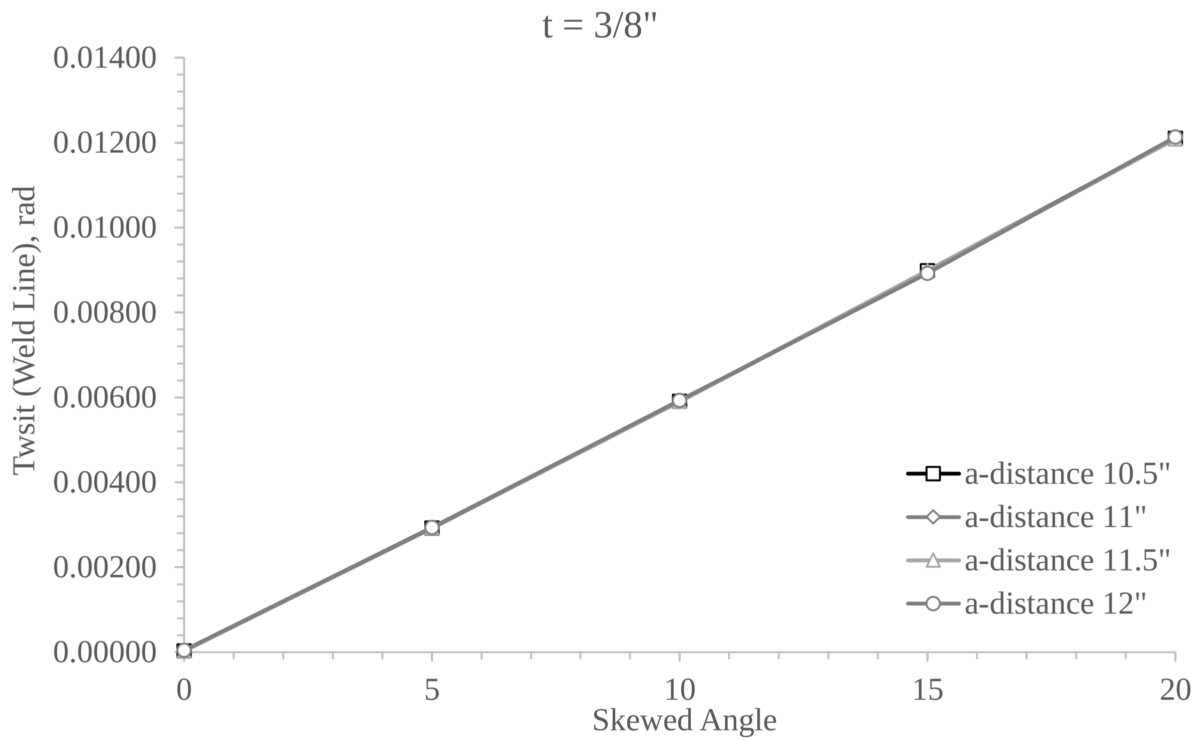

α = 15⁰
95000.00
90000.00
85000.00
80000.00
75000.00
70000.00
65000.00
60000.00
55000.00
50000.00
Shear, lbs
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in

B3. Eight Bolted Connections

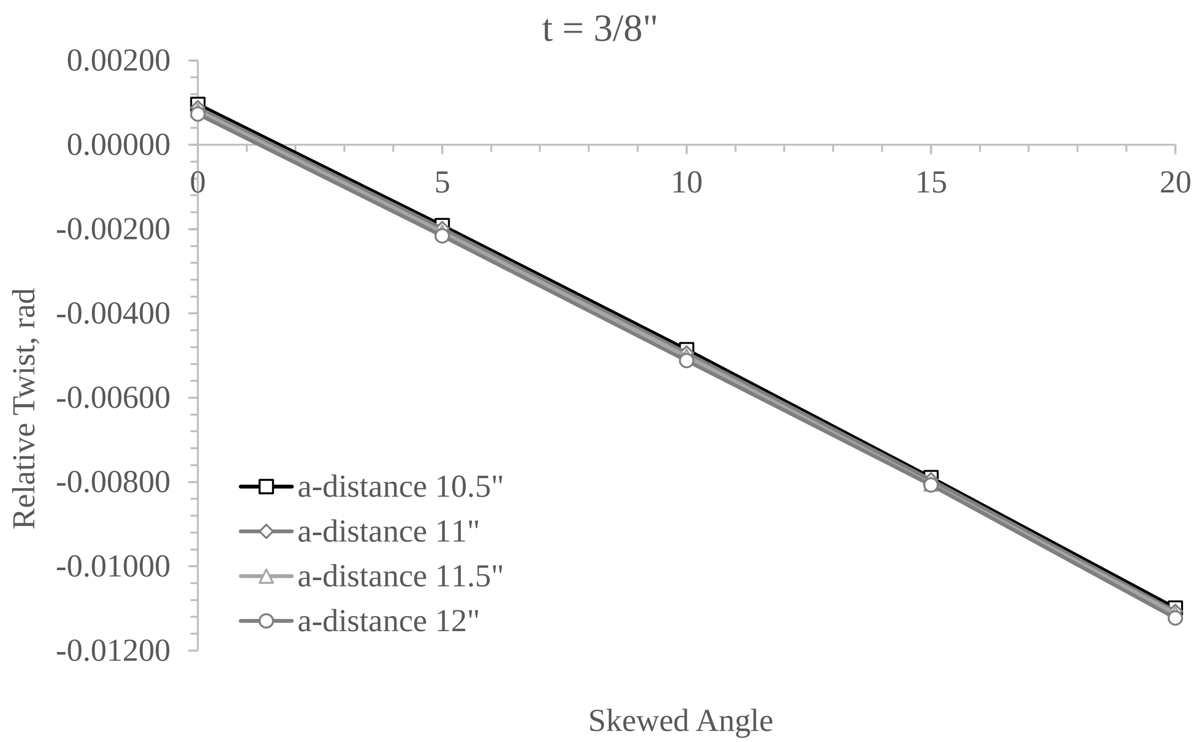
<u>Eight Bolted Connections (t = 3/8”) Curves</u>



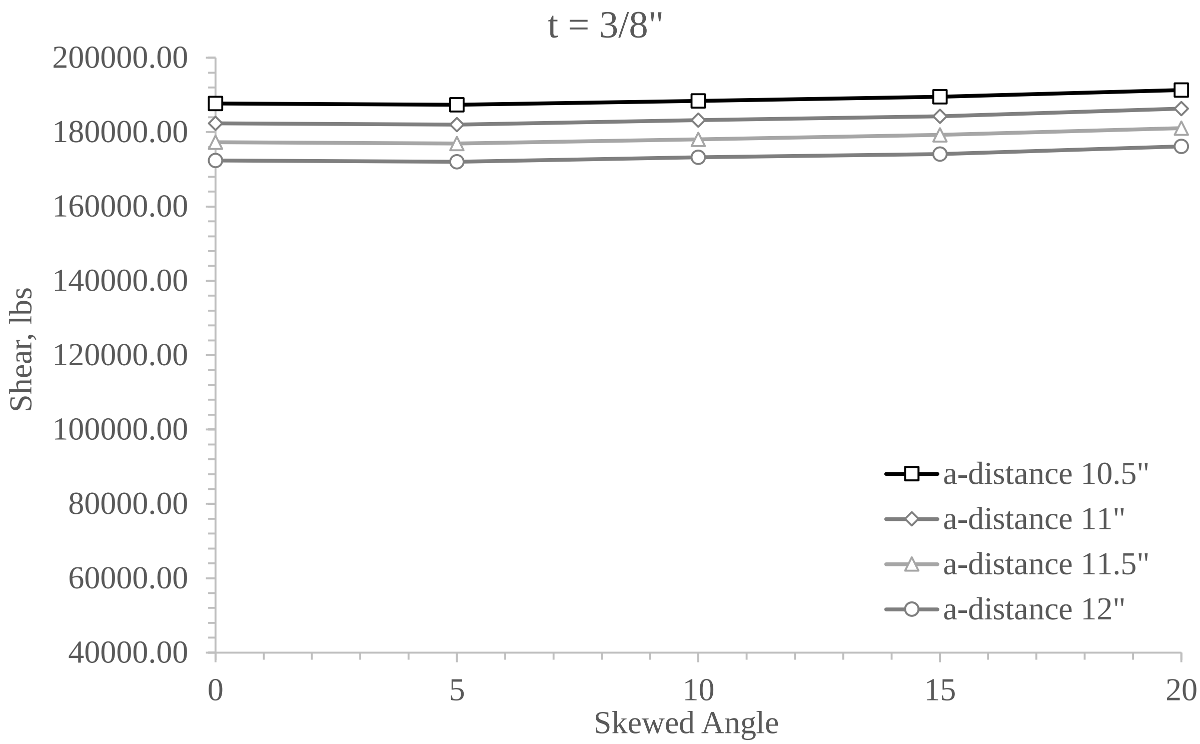




t = 3/8"
0.01400
0.01200
0.01000
0.00800
0.00600
0.00400
0.00200
0.00000
Twsit (Weld Line), rad
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"



t = 3/8"
0.00200
0.00000
-0.00200
-0.00400
-0.00600
-0.00800
-0.01000
-0.01200
Relative Twist, rad
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"



t = 3/8"
Shear, lbs
200000.00
180000.00
160000.00
140000.00
120000.00
100000.00
80000.00
60000.00
40000.00
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"

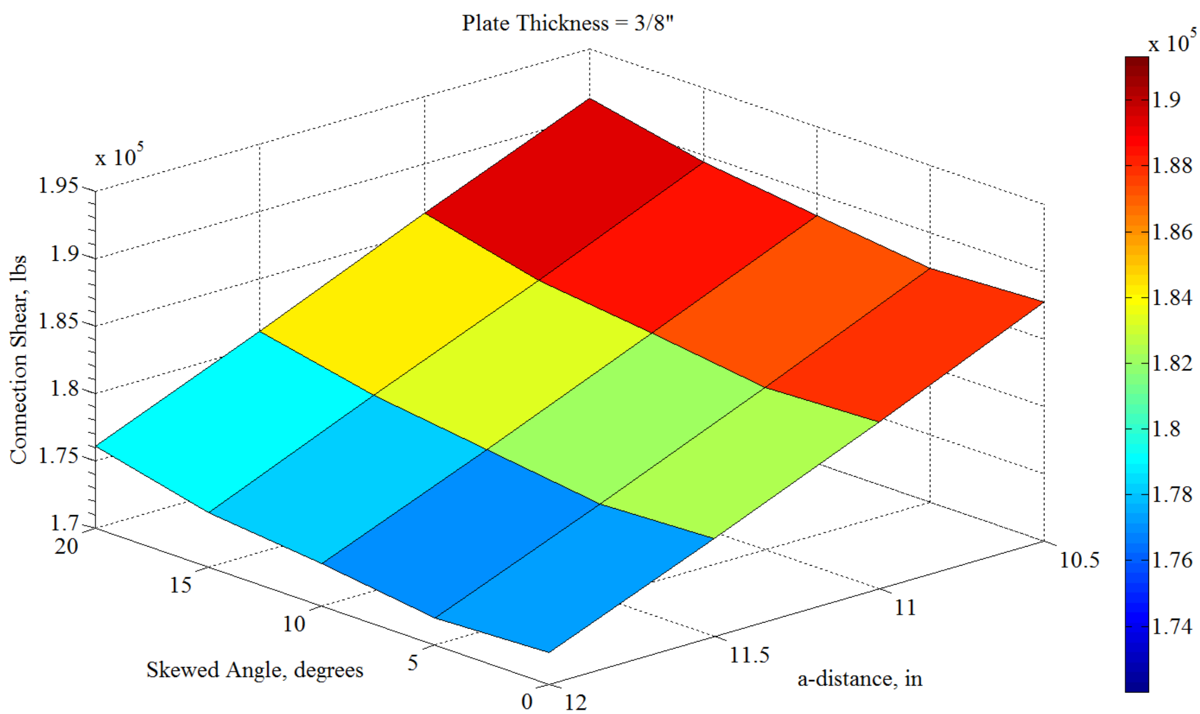
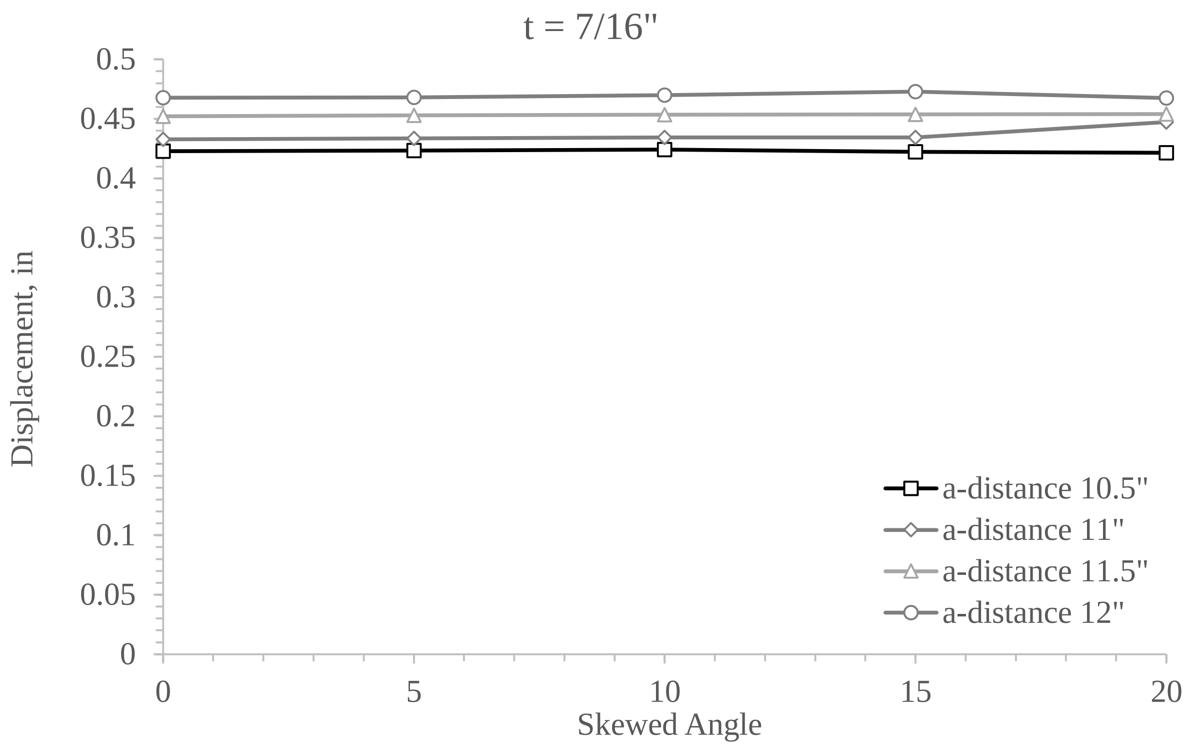
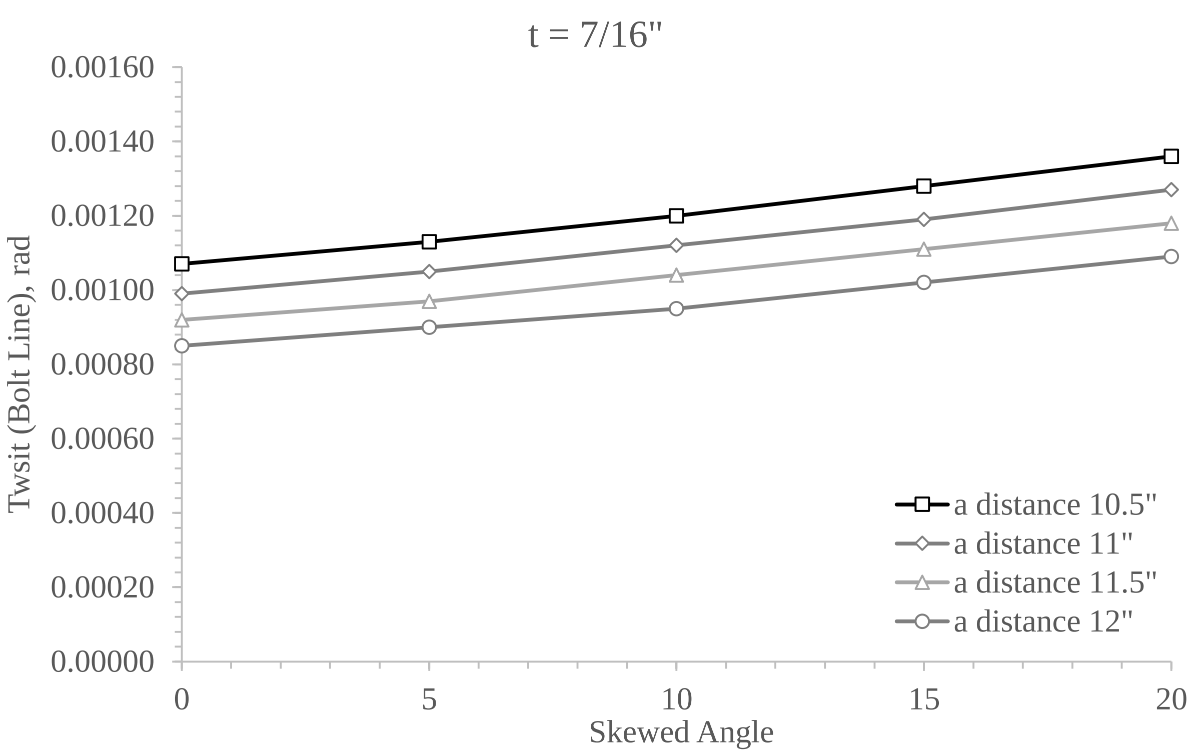
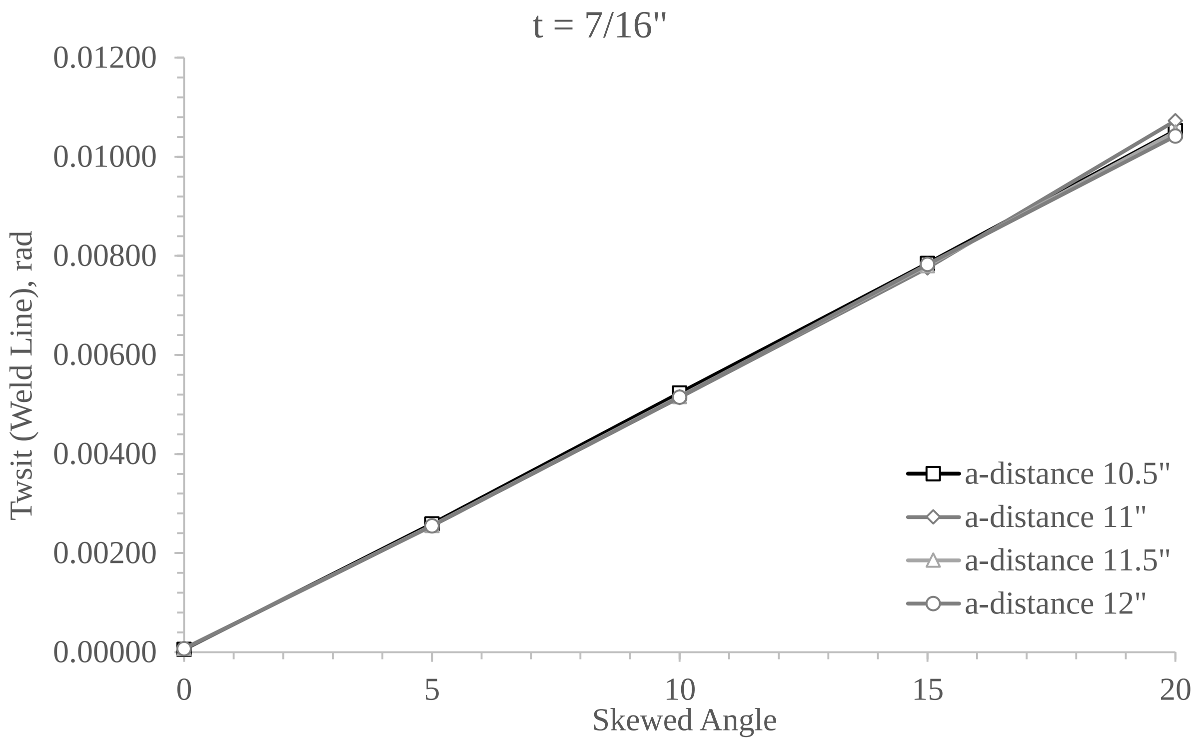


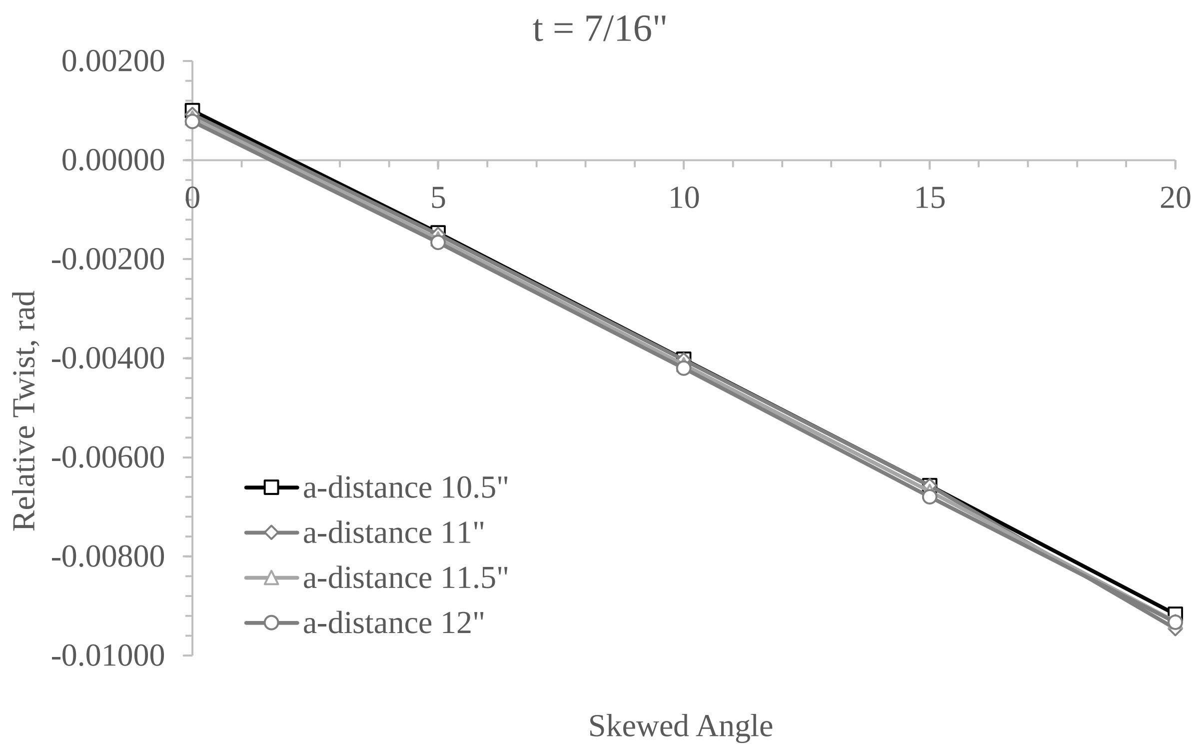
Plate Thickness = 3/8"
x 10^5
Connection Shear, lbs
1.95
1.9
1.85
1.8
1.75
1.7
Skewed Angle, degrees
20
15
10
5
0
a-distance, in
12
11.5
11
10.5
x 10^5
1.9
1.88
1.86
1.84
1.82
1.8
1.78
1.76
1.74

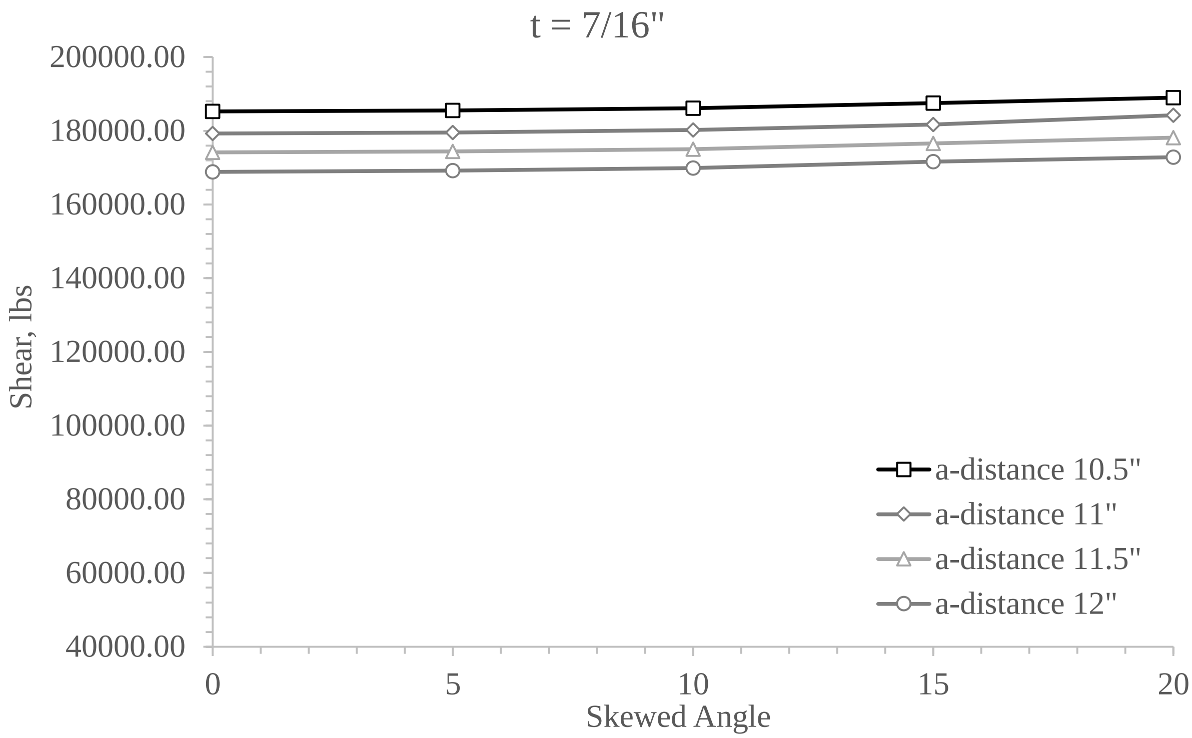
Eight Bolted Connections (t = 7/16”) Curves






t = 7/16"
Twsit (Weld Line), rad
0.01200
0.01000
0.00800
0.00600
0.00400
0.00200
0.00000
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 7/16"
Relative Twist, rad
0.00200
0.00000
-0.00200
-0.00400
-0.00600
-0.00800
-0.01000
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"



t = 7/16"
200000.00
180000.00
160000.00
140000.00
120000.00
100000.00
80000.00
60000.00
40000.00
Shear, lbs
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"

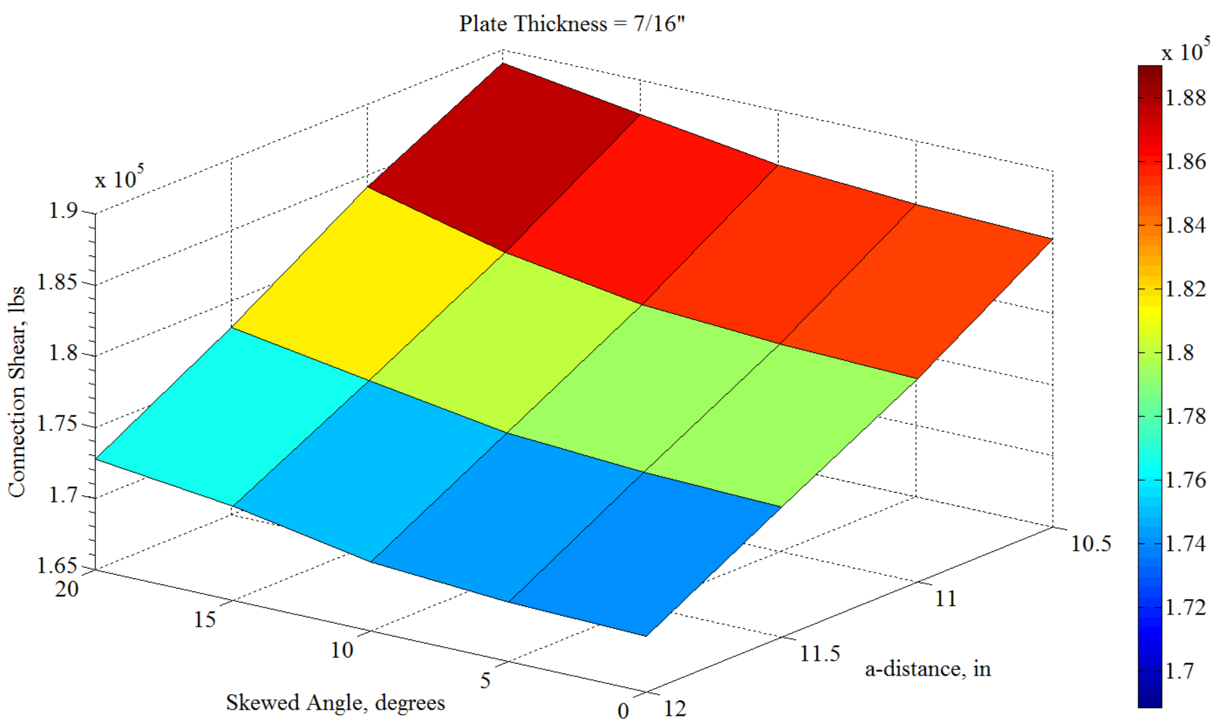
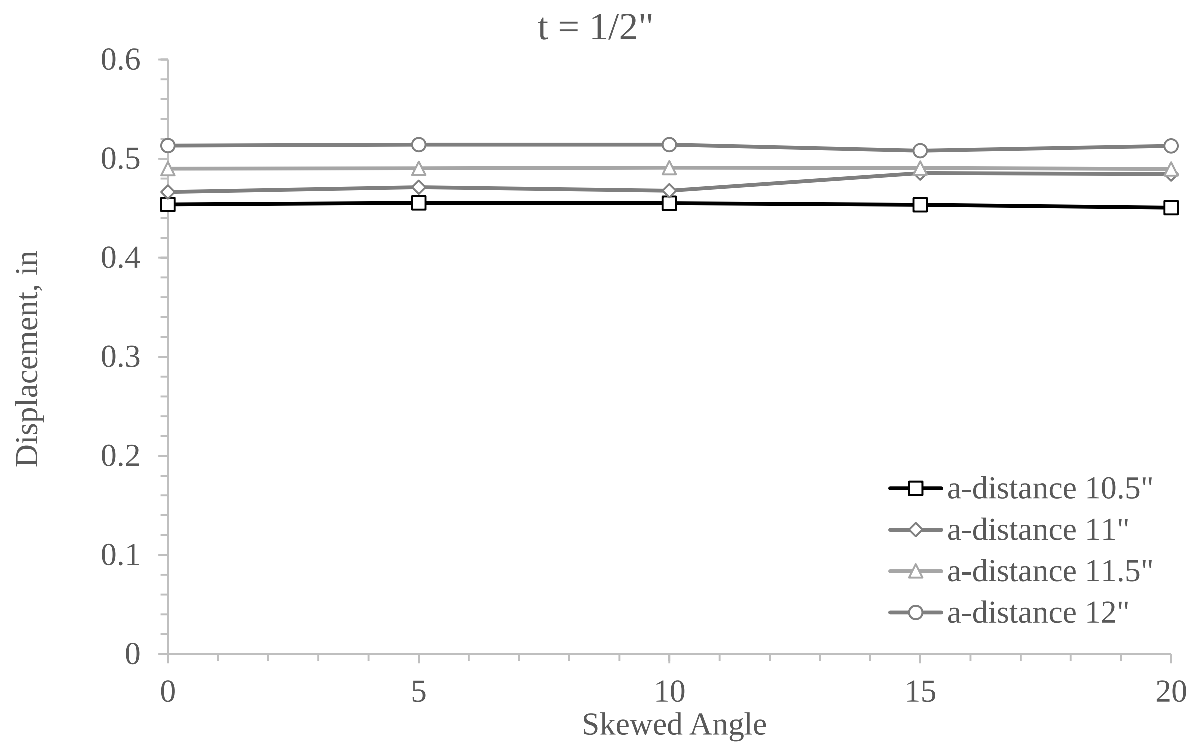
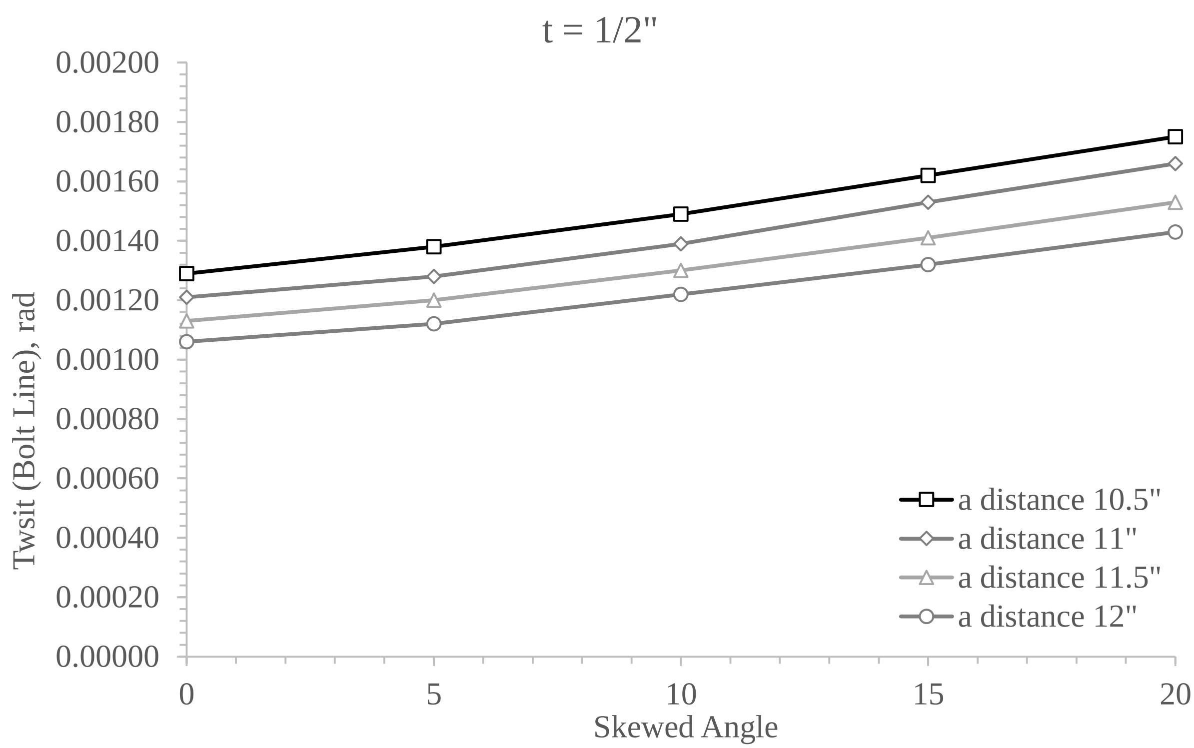
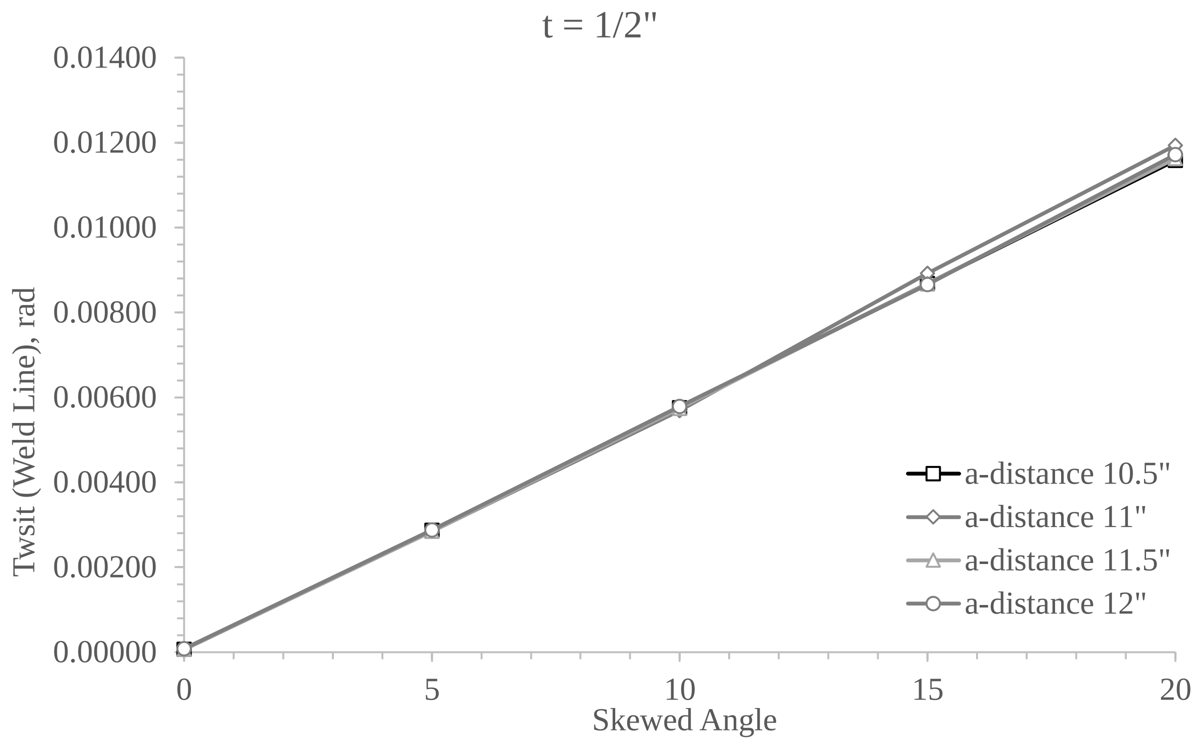


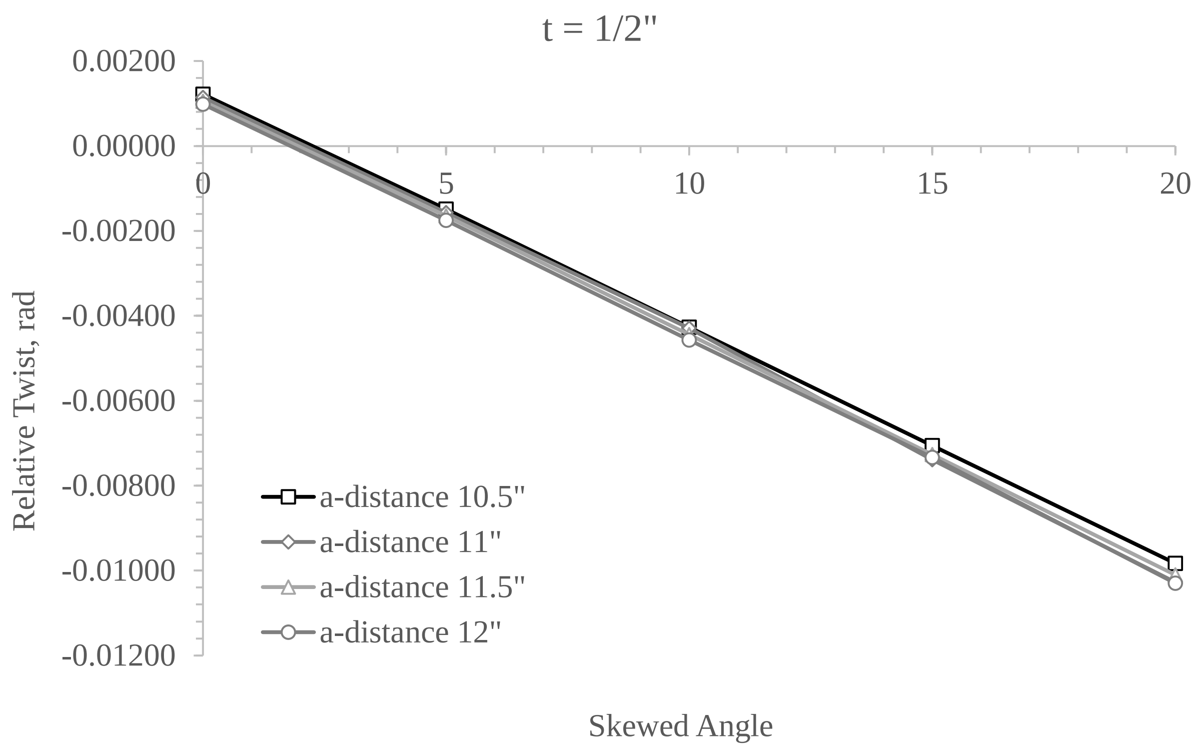
Plate Thickness = 7/16"
x 10^5
1.9
1.85
1.8
1.75
1.7
1.65
Connection Shear, lbs
20
15
10
5
0
Skewed Angle, degrees
10.5
11
11.5
12
a-distance, in
x 10^5
1.88
1.86
1.84
1.82
1.8
1.78
1.76
1.74
1.72
1.7

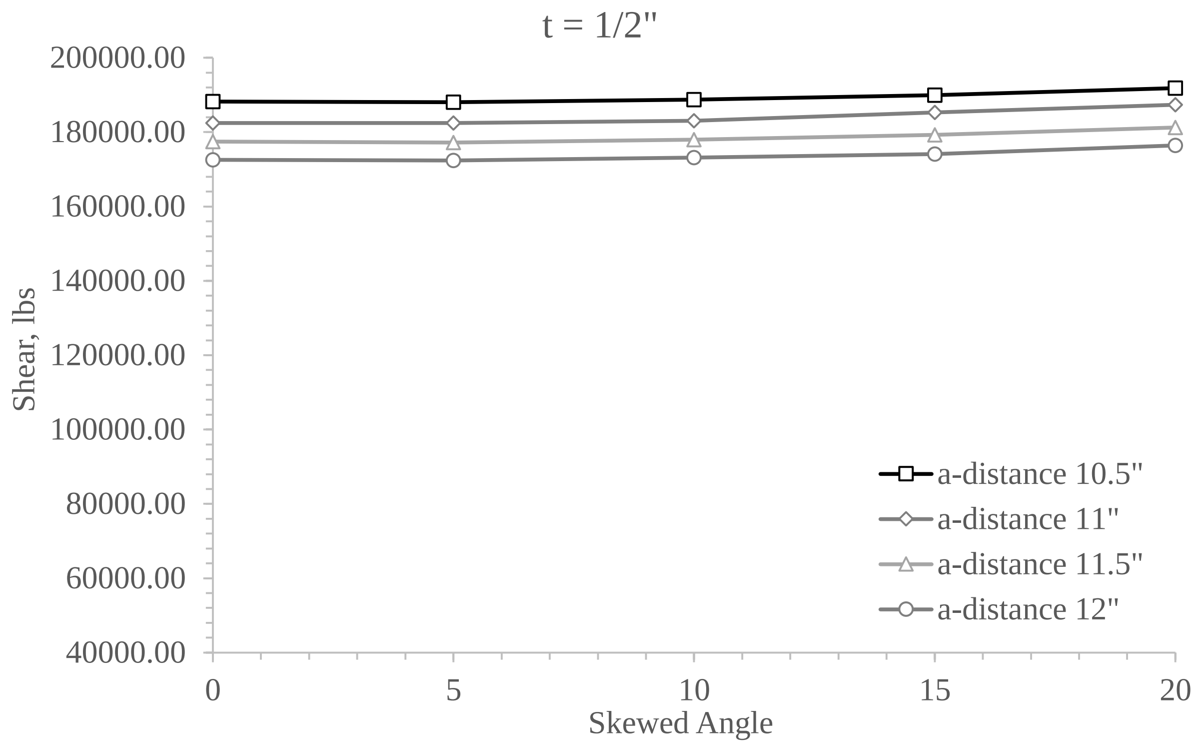
Eight Bolted Connections (t = 1/2”) Curves






t = 1/2"
0.01400
0.01200
0.01000
0.00800
0.00600
0.00400
0.00200
0.00000
Twsit (Weld Line), rad
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 1/2"
0.00200
0.00000
-0.00200
-0.00400
-0.00600
-0.00800
-0.01000
-0.01200
Relative Twist, rad
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 1/2"
200000.00
180000.00
160000.00
140000.00
120000.00
100000.00
80000.00
60000.00
40000.00
Shear, lbs
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"

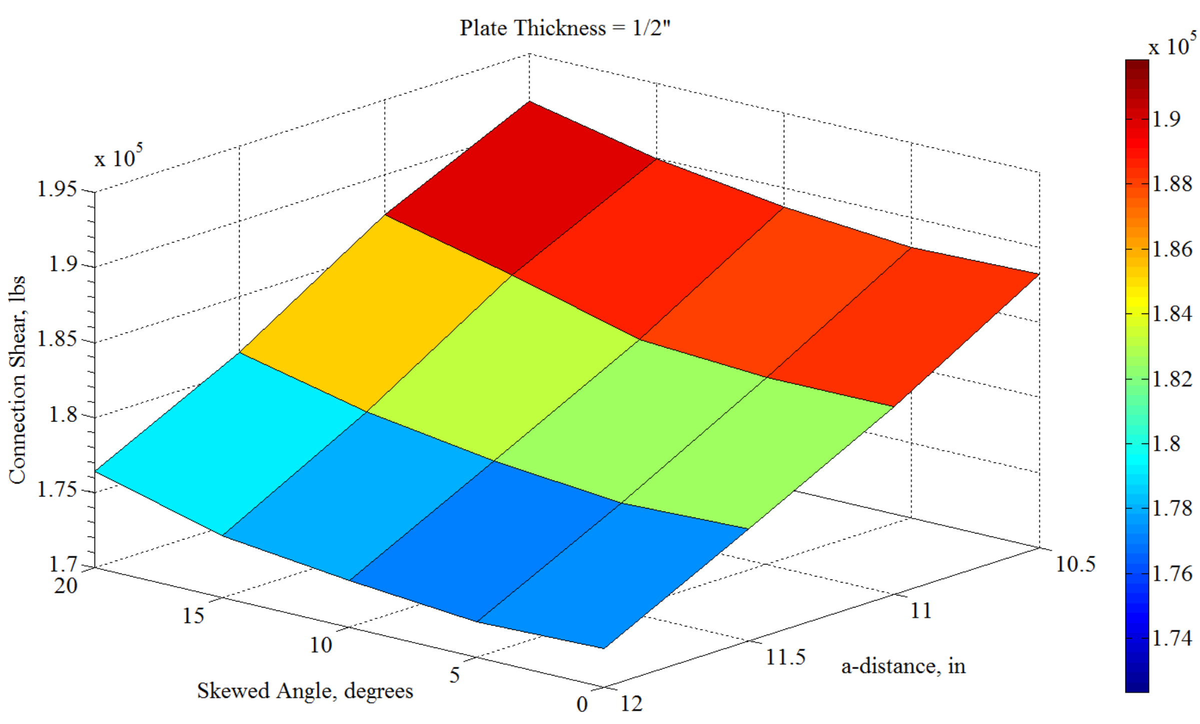
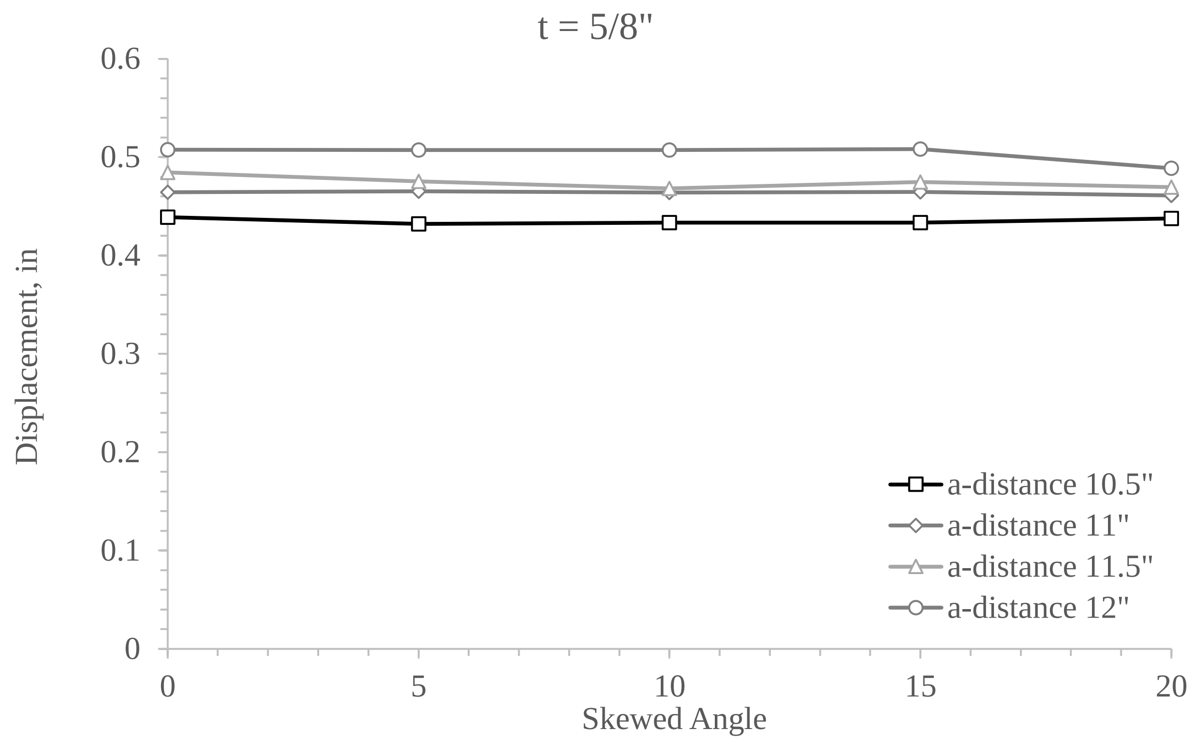
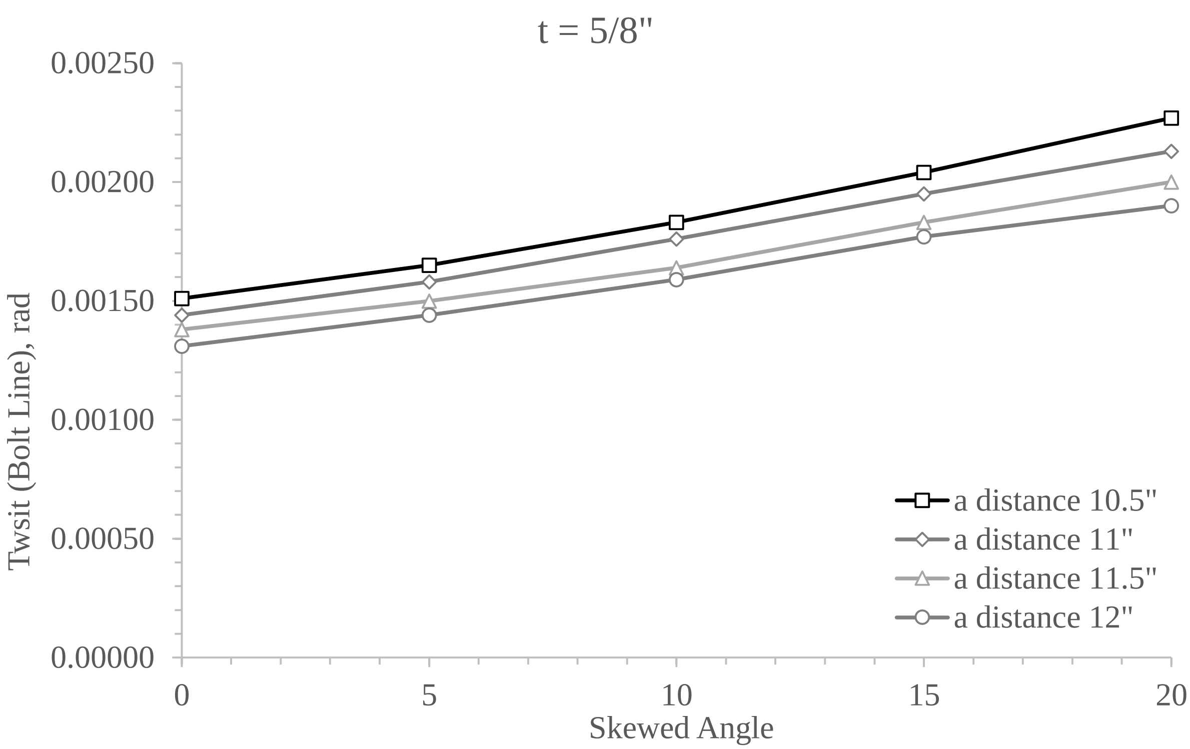
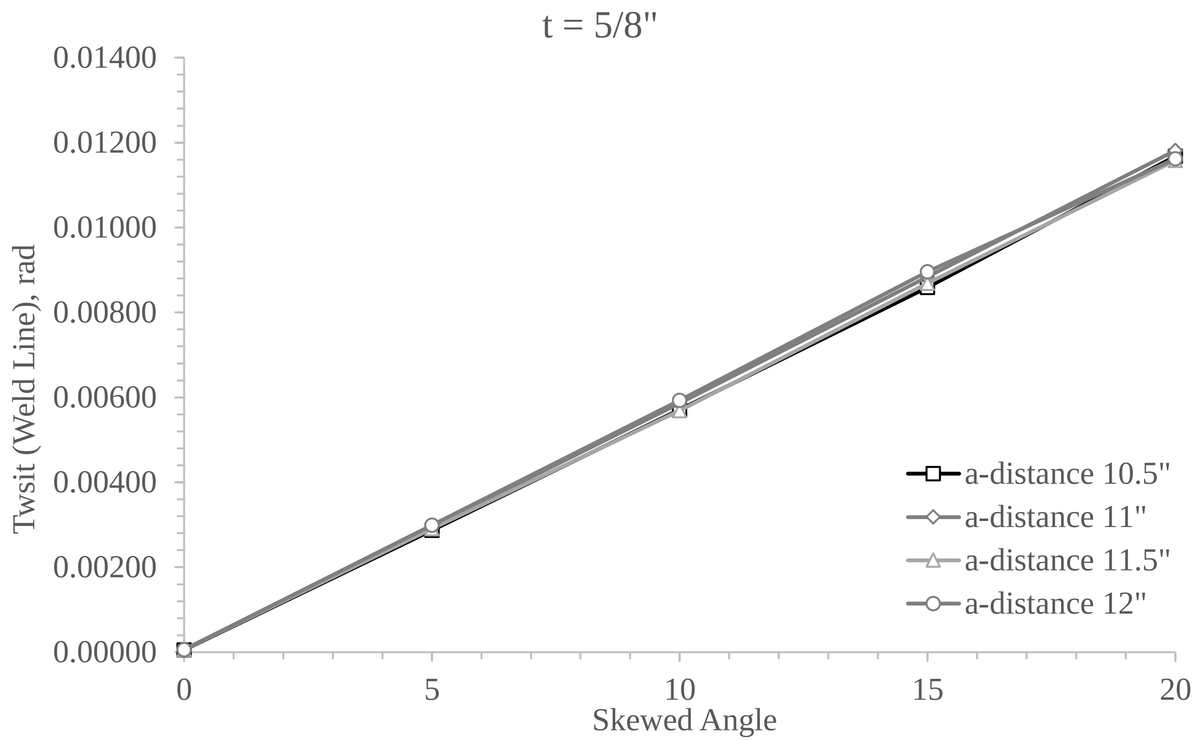


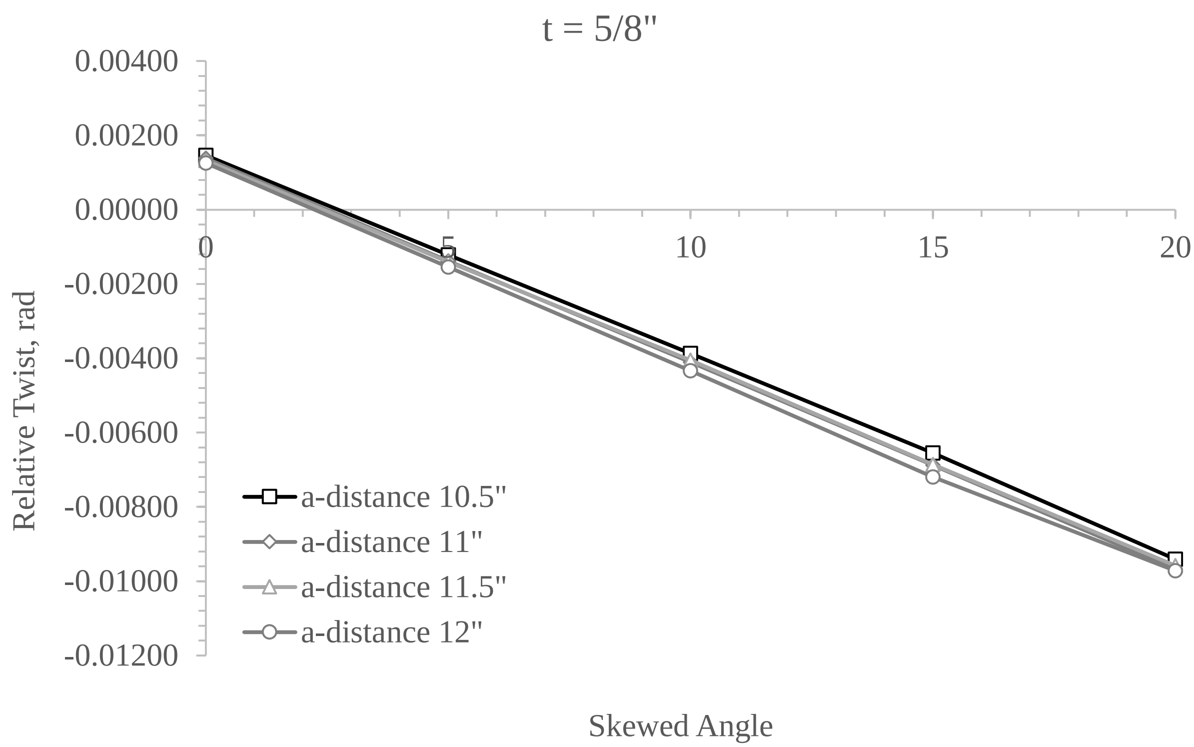
Plate Thickness = 1/2"
x 10^5
1.95
1.9
1.85
1.8
1.75
1.7
Connection Shear, lbs
20
15
10
5
0
Skewed Angle, degrees
12
11.5
11
10.5
a-distance, in
x 10^5
1.9
1.88
1.86
1.84
1.82
1.8
1.78
1.76
1.74

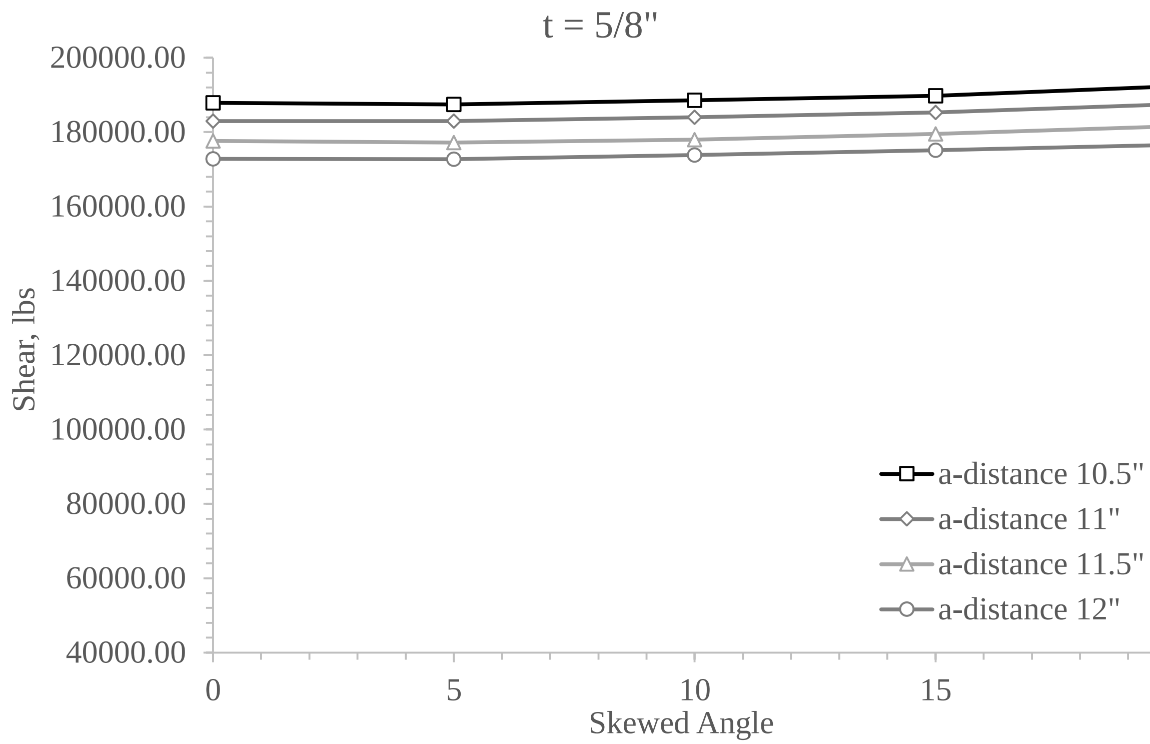
Eight Bolted Connections (t = 5/8”) Curves






t = 5/8"
0.01400
0.01200
0.01000
0.00800
0.00600
0.00400
0.00200
0.00000
Twsit (Weld Line), rad
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


t = 5/8"
0.00400
0.00200
0.00000
-0.00200
-0.00400
-0.00600
-0.00800
-0.01000
-0.01200
Relative Twist, rad
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"



t = 5/8"
200000.00
180000.00
160000.00
140000.00
120000.00
100000.00
80000.00
60000.00
40000.00
Shear, lbs
0
5
10
15
20
Skewed Angle
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"

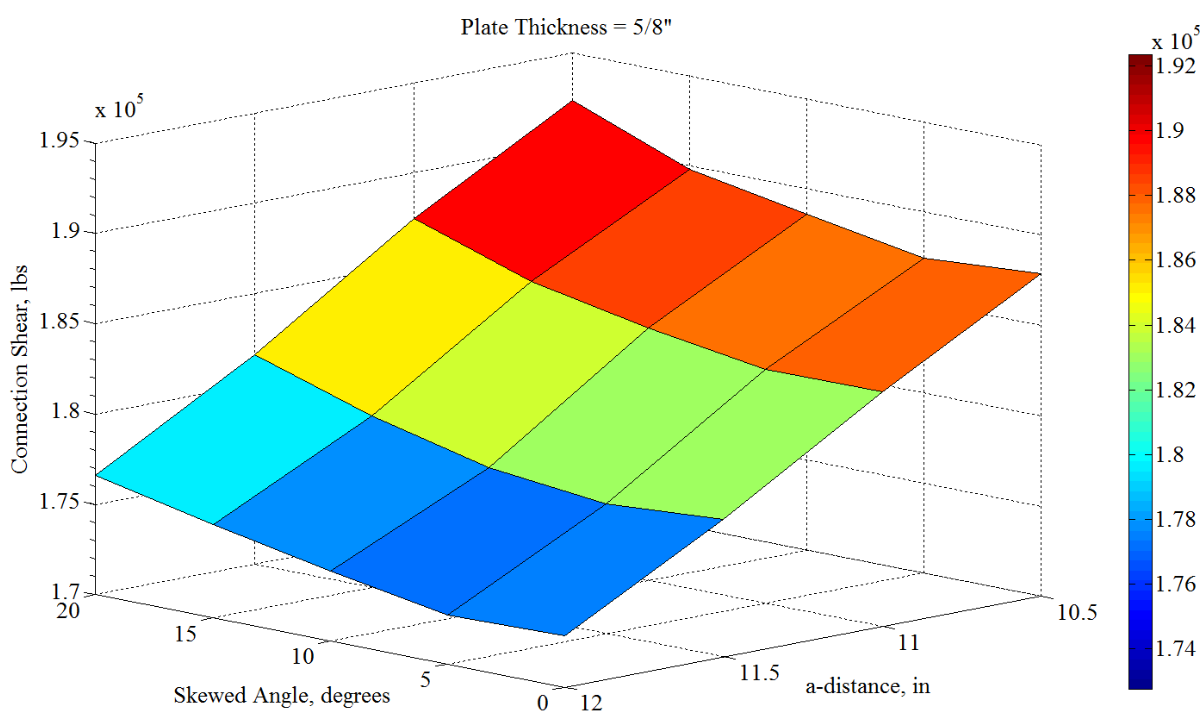
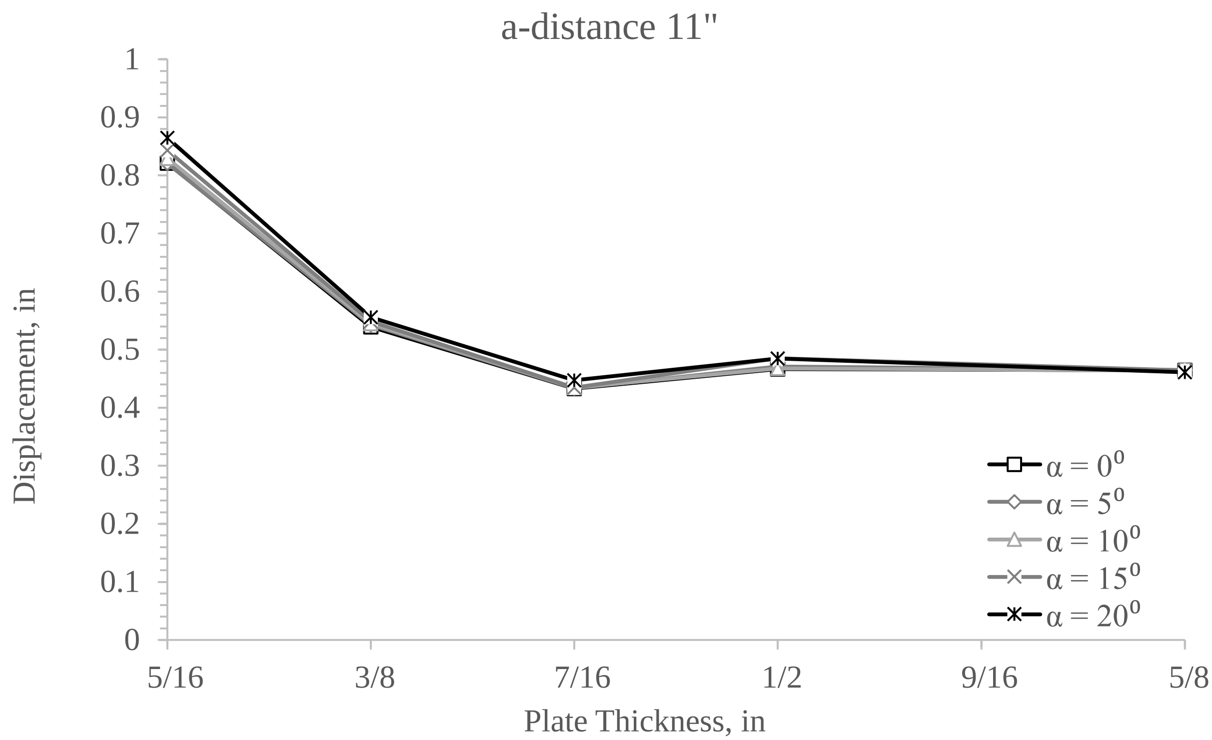
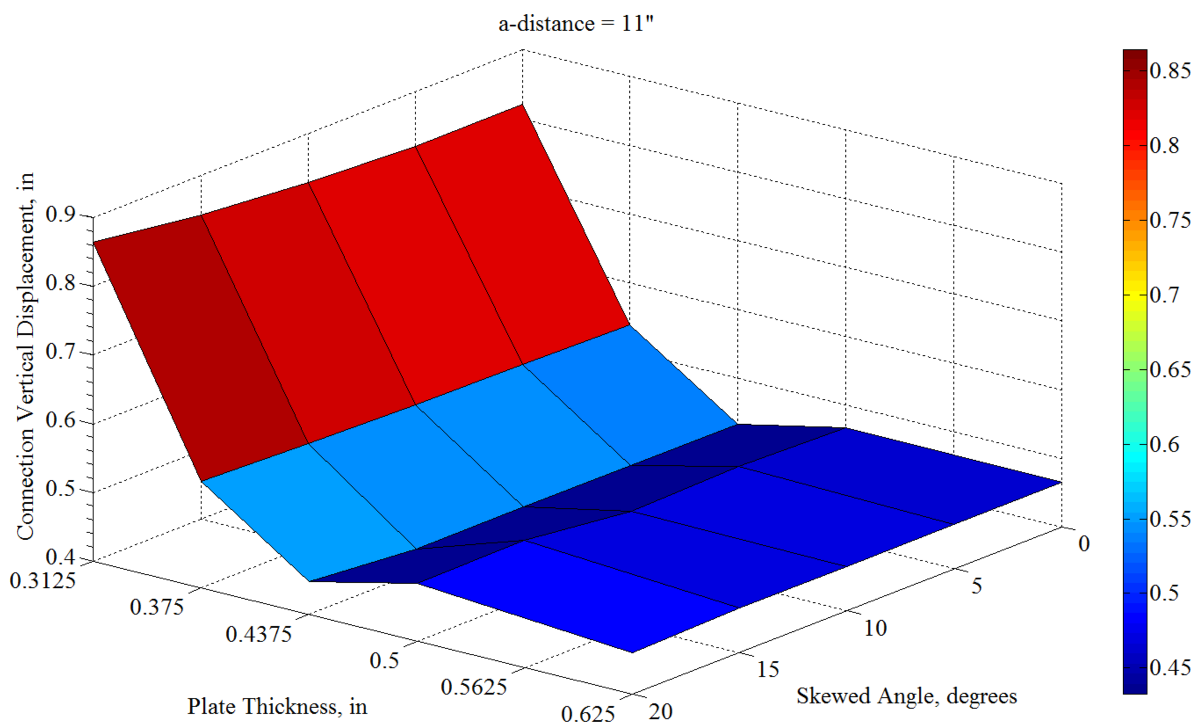
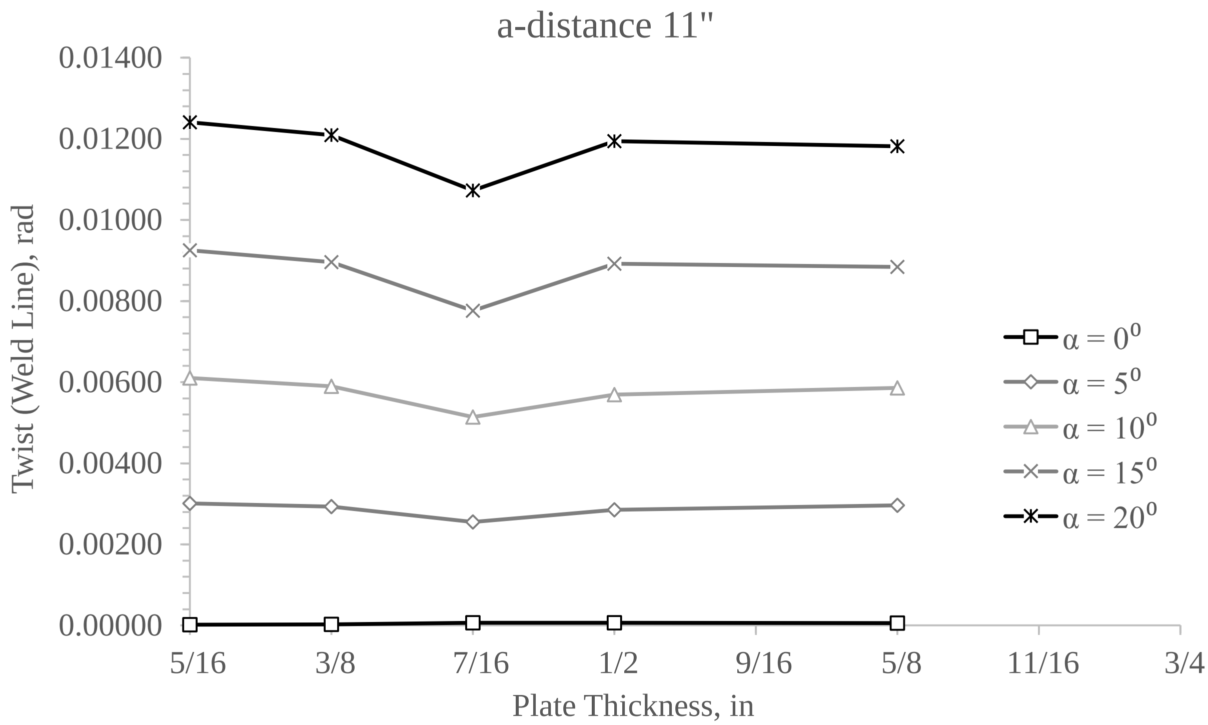


Plate Thickness = 5/8"
x 10^5
1.95
1.9
1.85
1.8
1.75
1.7
Connection Shear, lbs
20
15
10
5
0
Skewed Angle, degrees
12
11.5
11
10.5
a-distance, in
x 10^5
1.92
1.9
1.88
1.86
1.84
1.82
1.8
1.78
1.76
1.74

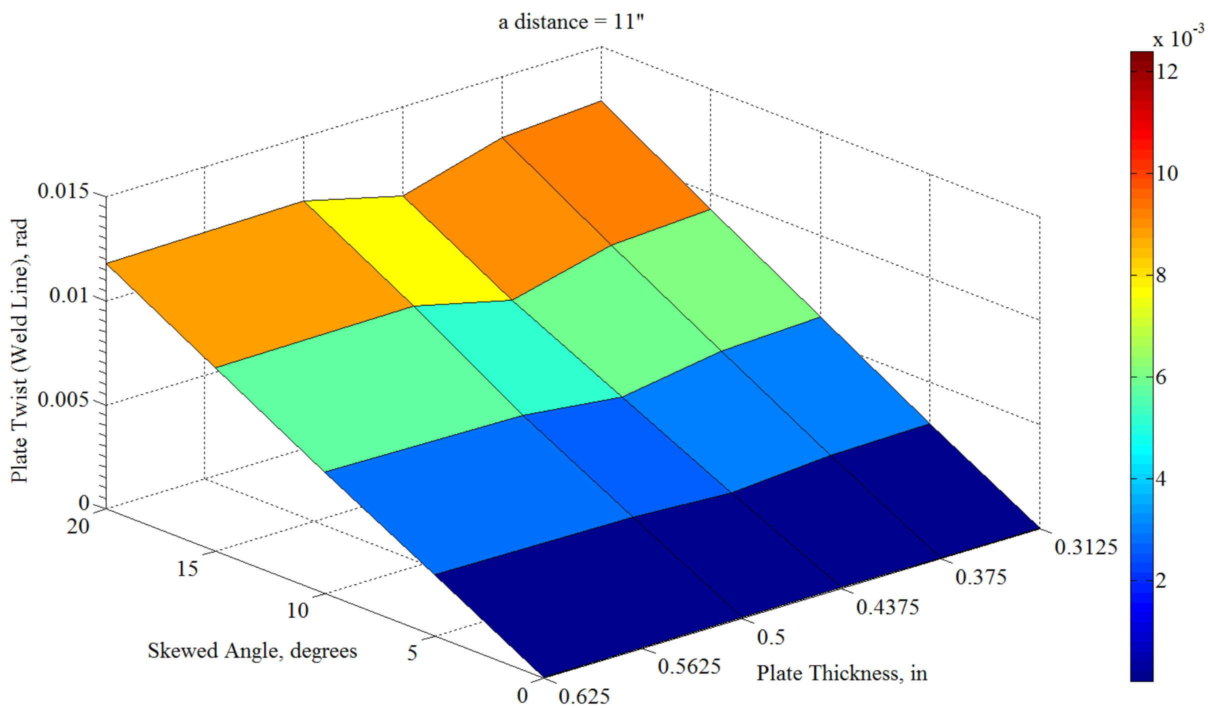
Eight Bolted Connections (a-distance = 11”) Curves



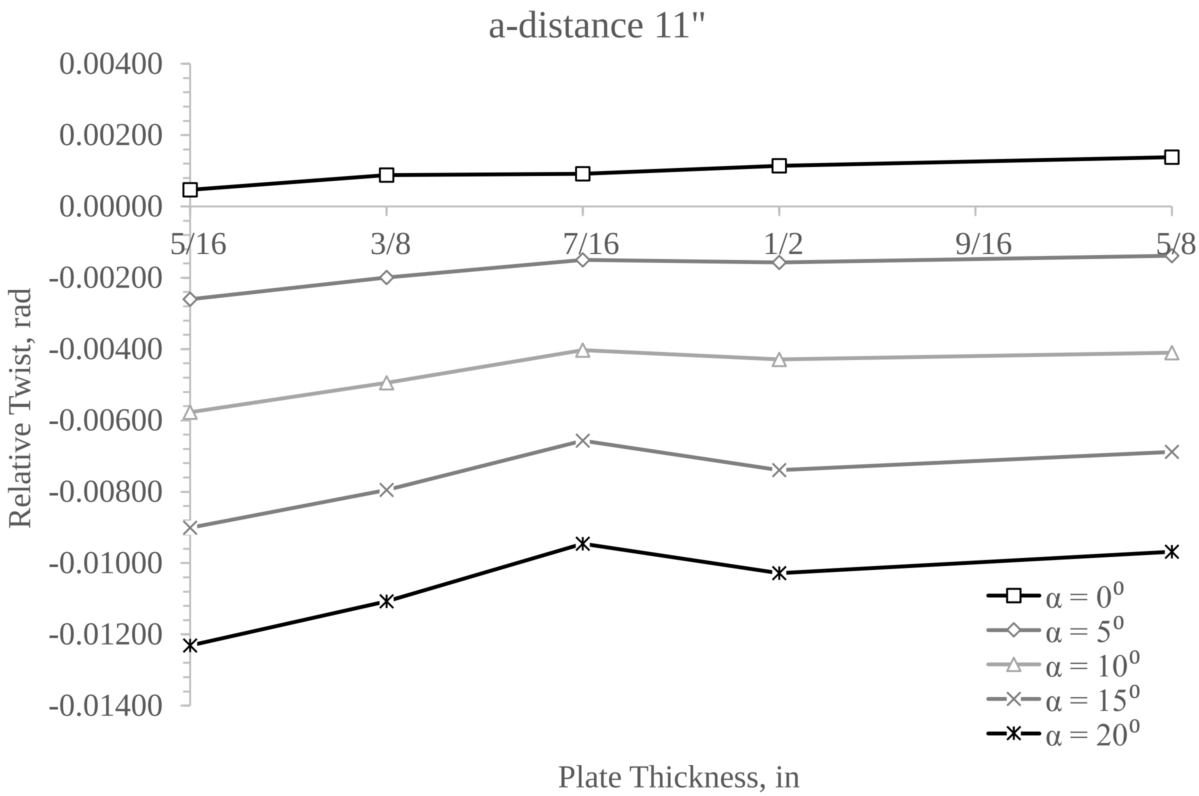


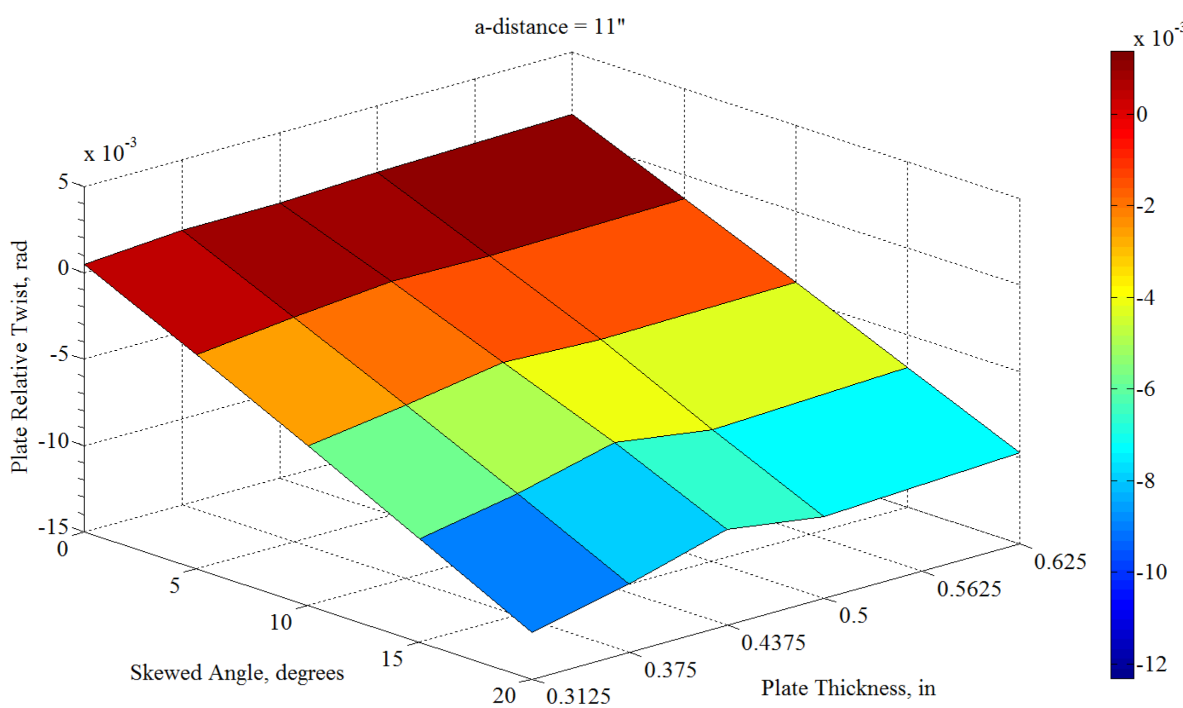


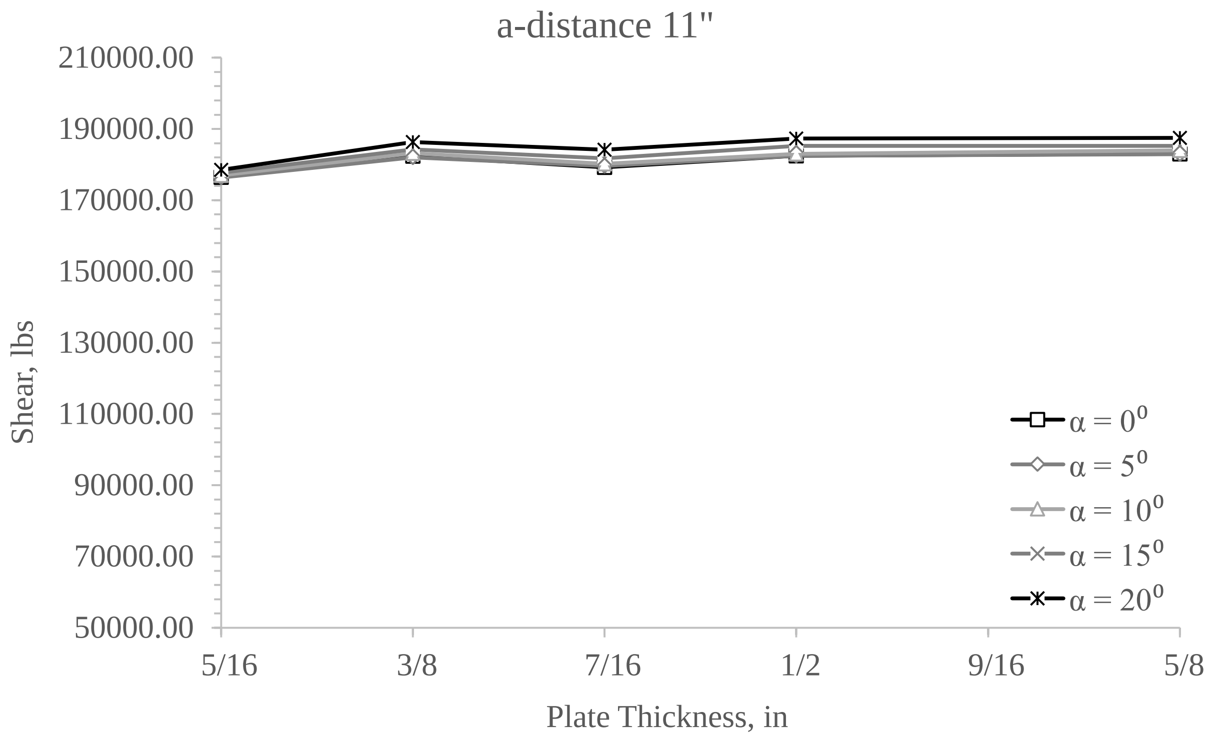
a-distance 11"
0.01400
0.01200
0.01000
0.00800
0.00600
0.00400
0.00200
0.00000
Twist (Weld Line), rad
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness, in
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰



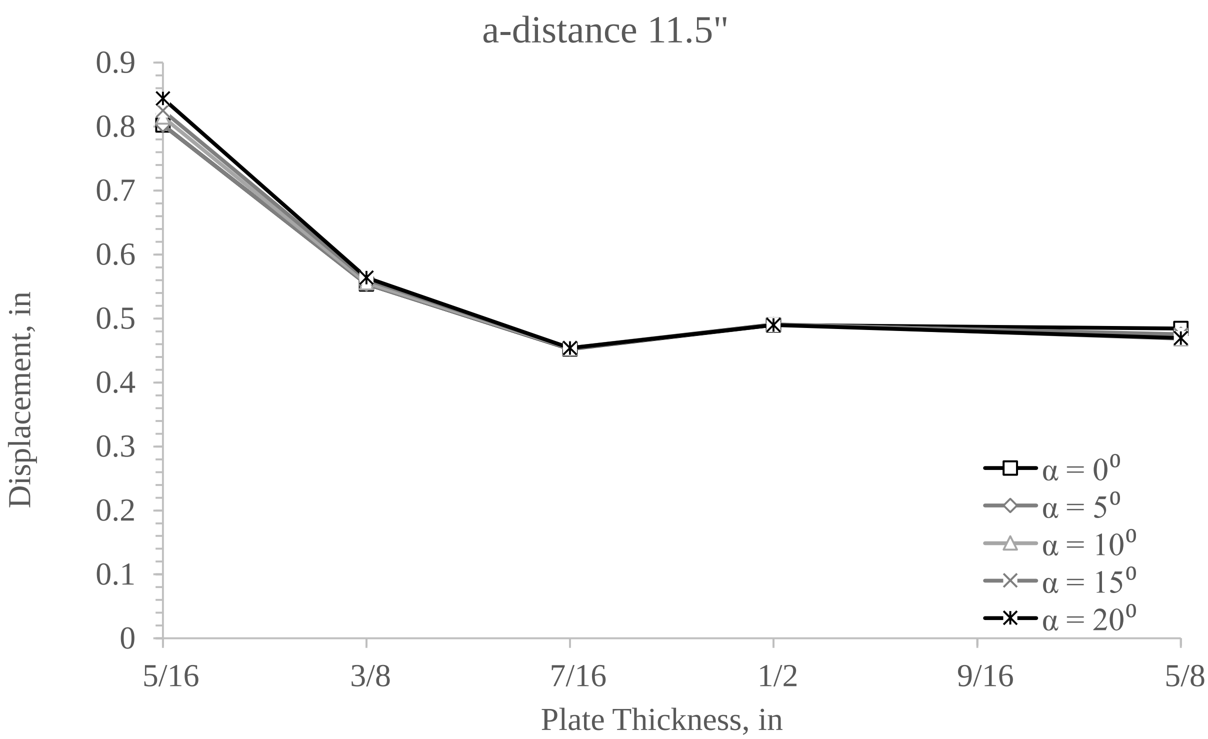
a distance = 11"
x 10-3
12
10
8
6
4
2
0.015
0.01
0.005
0
Plate Twist (Weld Line), rad
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in

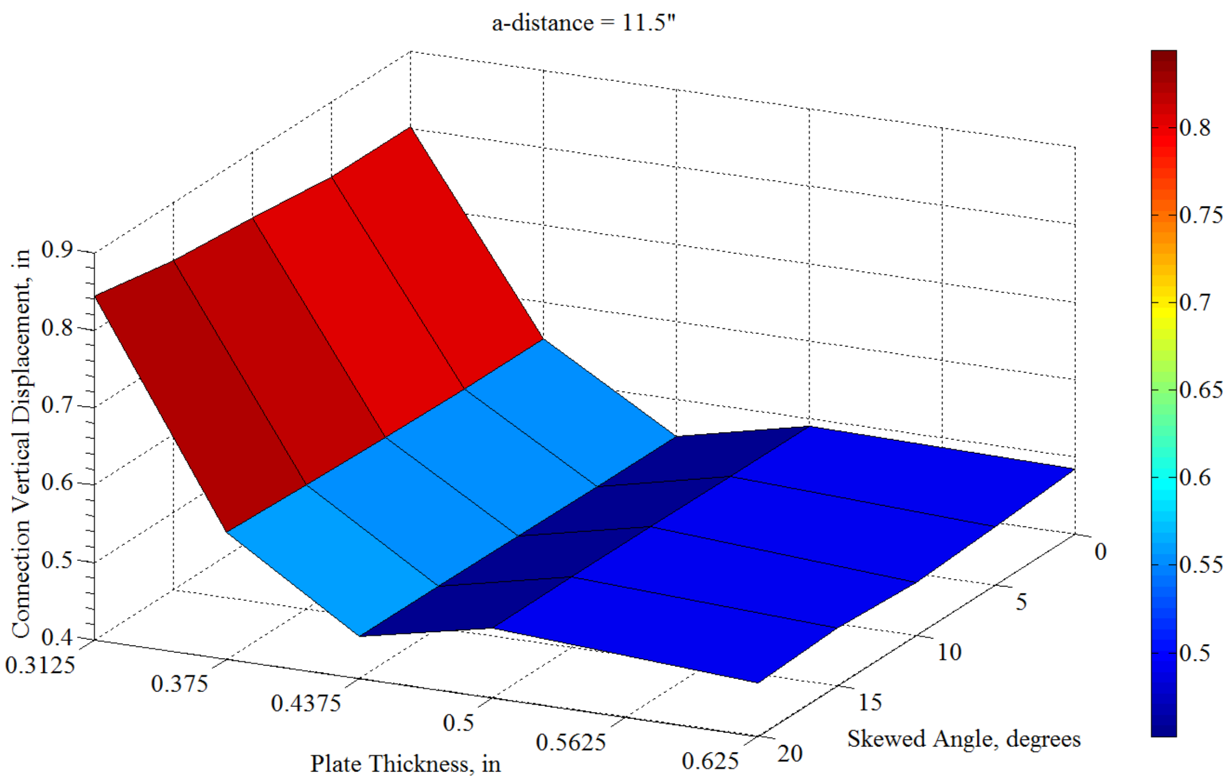

a-distance 11"
0.00400
0.00200
0.00000
-0.00200
-0.00400
-0.00600
-0.00800
-0.01000
-0.01200
-0.01400
5/16
3/8
7/16
1/2
9/16
5/8
Relative Twist, rad
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰
Plate Thickness, in

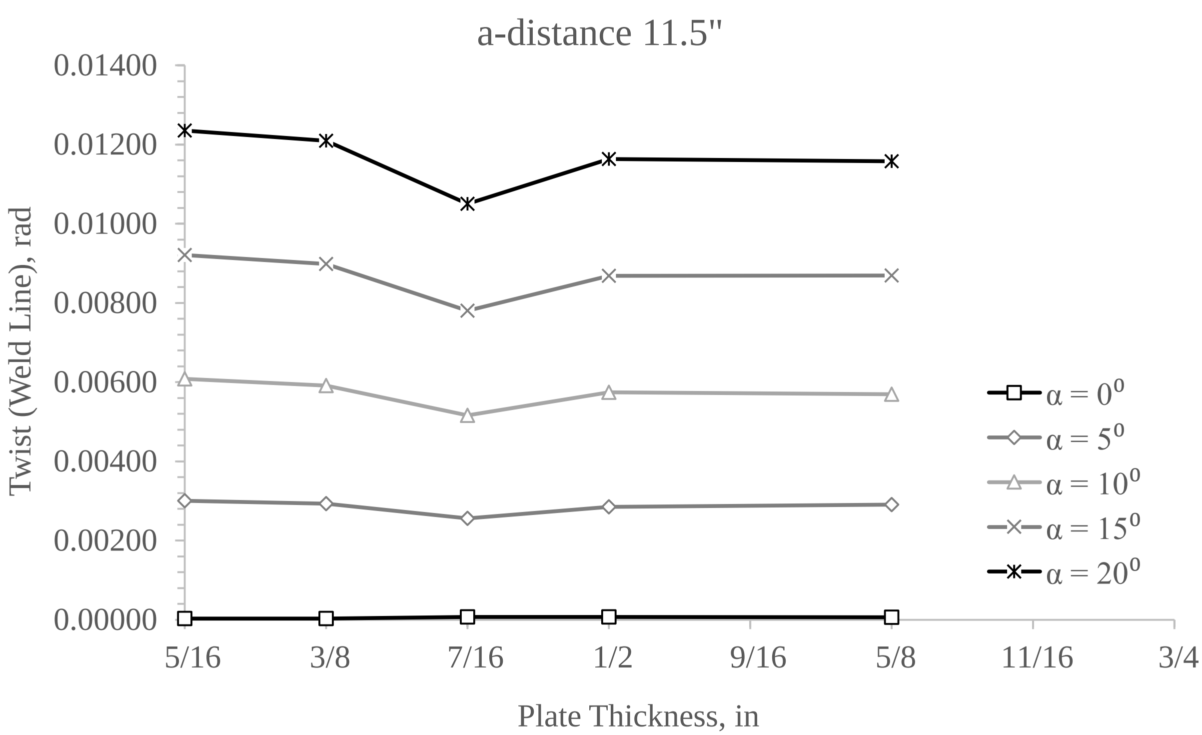

a-distance = 11"
x 10-3
5
0
-5
-10
-15
Plate Relative Twist, rad
0
5
10
15
20
Skewed Angle, degrees
0.3125
0.375
0.4375
0.5
0.5625
0.625
Plate Thickness, in
x 10-3
0
-2
-4
-6
-8
-10
-12

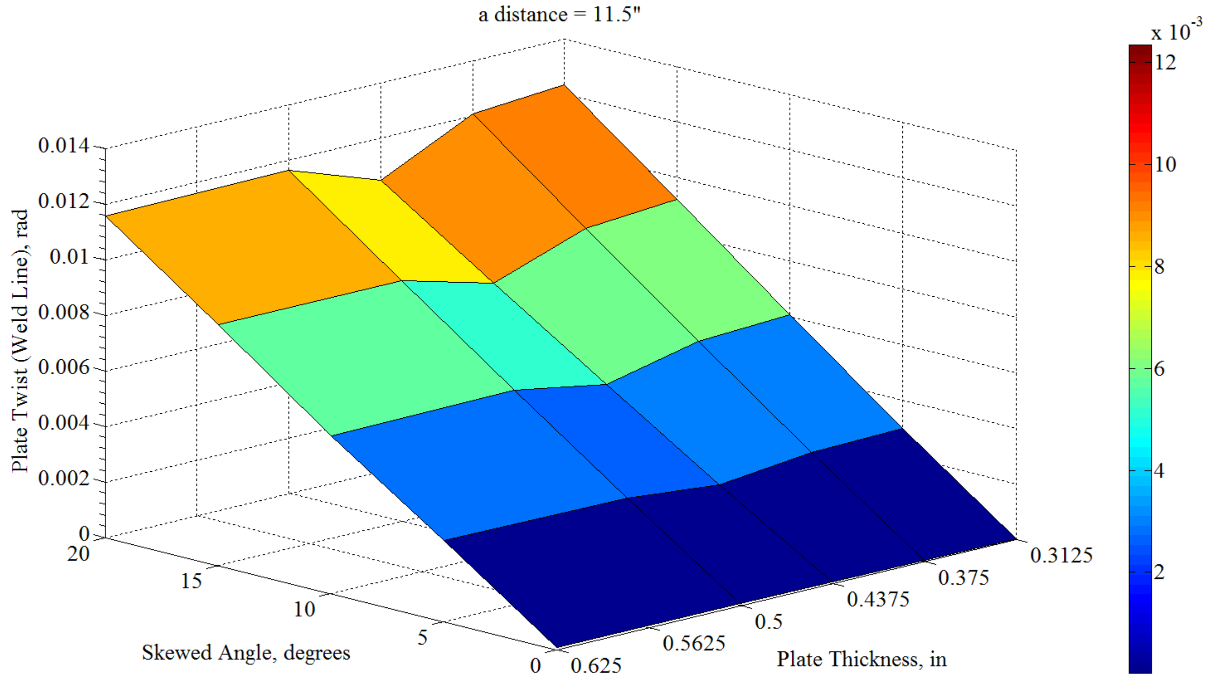


Eight Bolted Connections (a-distance = 11.5”) Curves

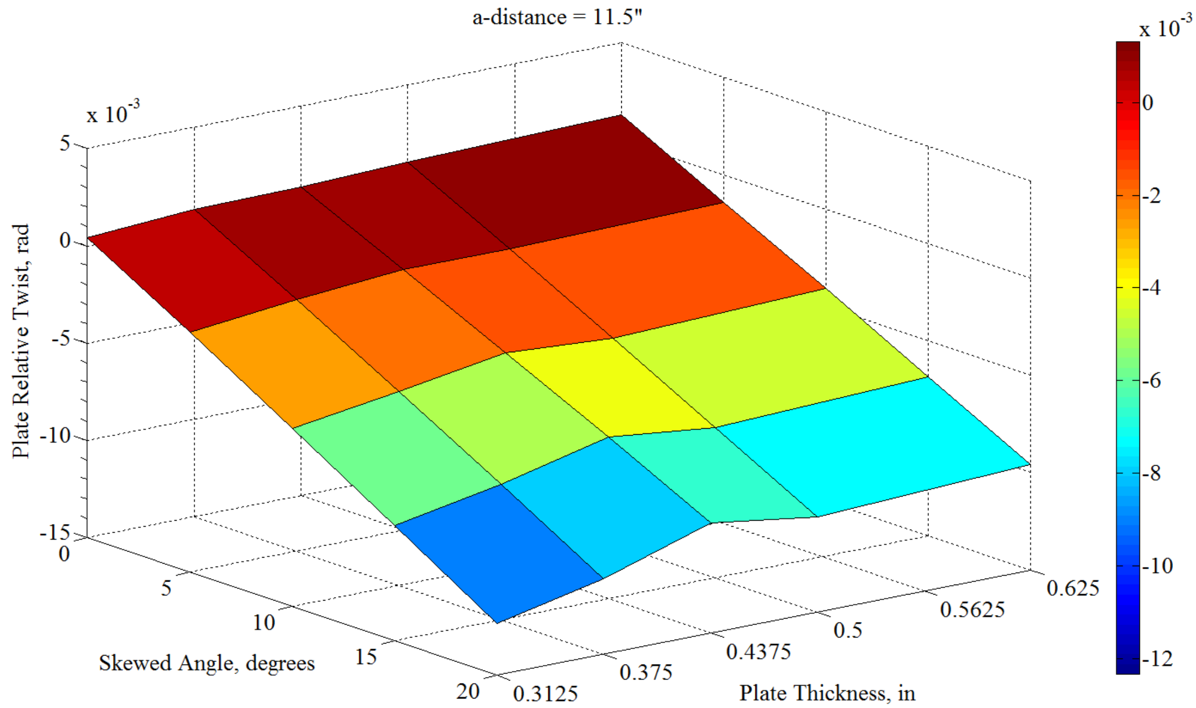


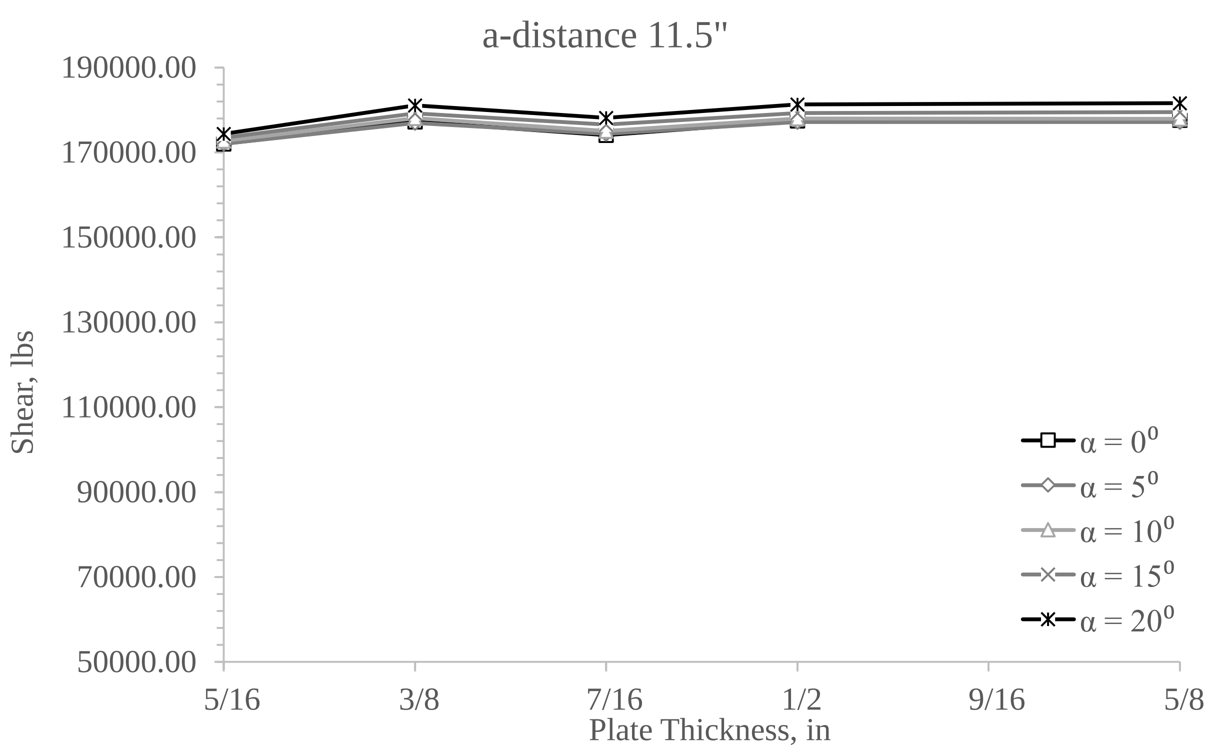

a-distance = 11.5"
Connection Vertical Displacement, in
0.9
0.8
0.7
0.6
0.5
0.4
0.3125
0.375
0.4375
0.5
0.5625
0.625
Plate Thickness, in
0
5
10
15
20
Skewed Angle, degrees
0.8
0.75
0.7
0.65
0.6
0.55
0.5


a-distance 11.5"
Twist (Weld Line), rad
0.01400
0.01200
0.01000
0.00800
0.00600
0.00400
0.00200
0.00000
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness, in
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

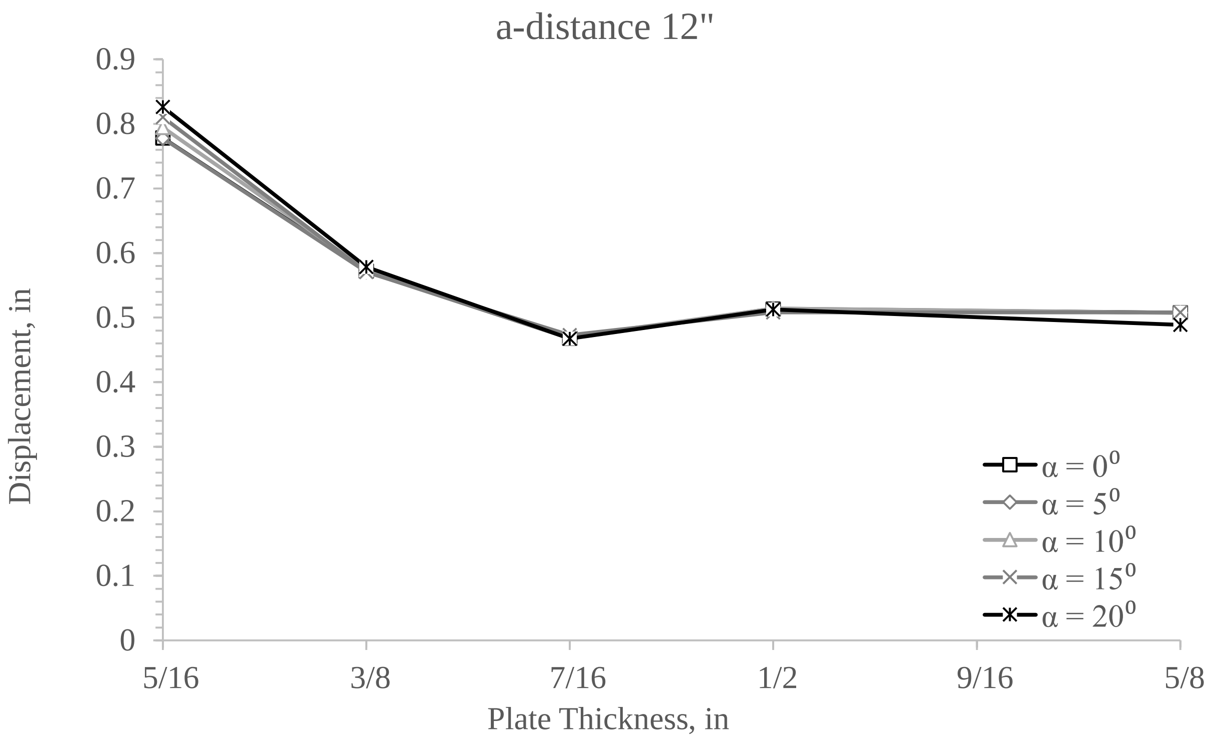

a distance = 11.5"
x 10^-3
12
10
8
6
4
2
0.014
0.012
0.01
0.008
0.006
0.004
0.002
0
Plate Twist (Weld Line), rad
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in

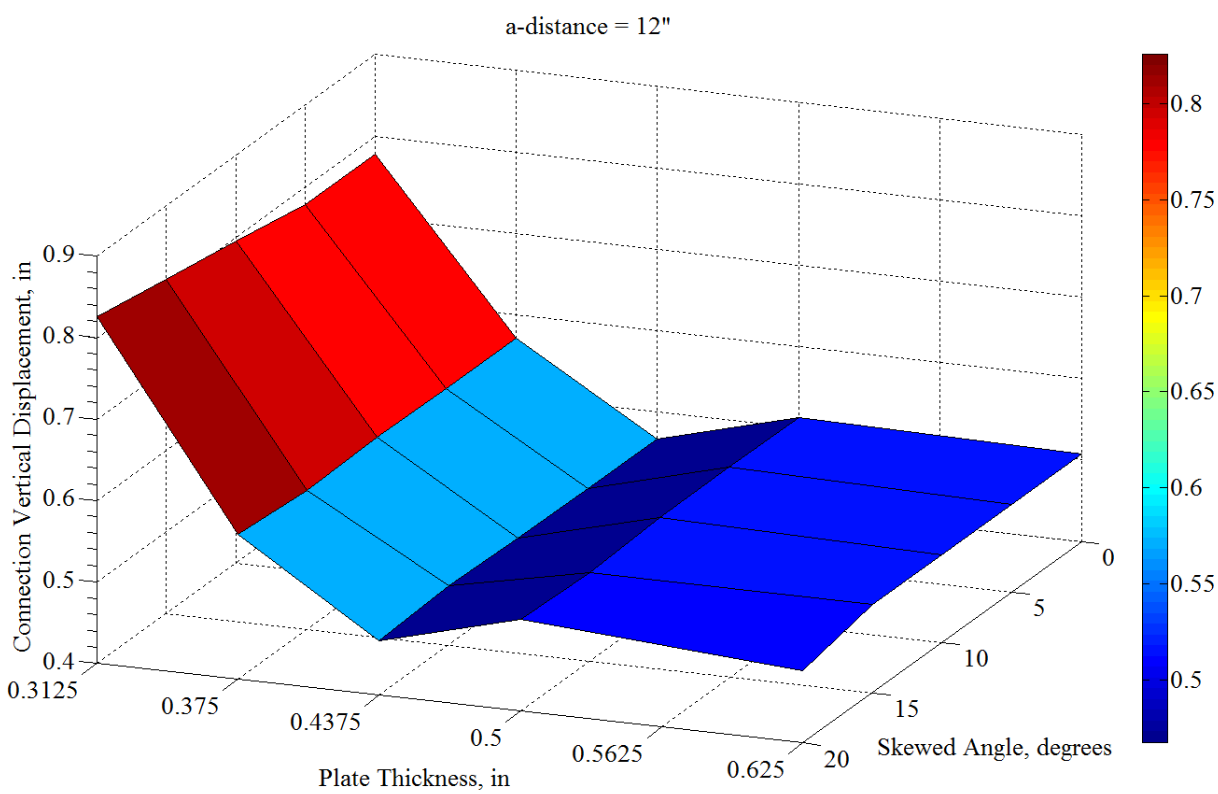

a-distance 11.5"
0.00400
0.00200
0.00000
-0.00200
-0.00400
-0.00600
-0.00800
-0.01000
-0.01200
-0.01400
Relative Twist, rad
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

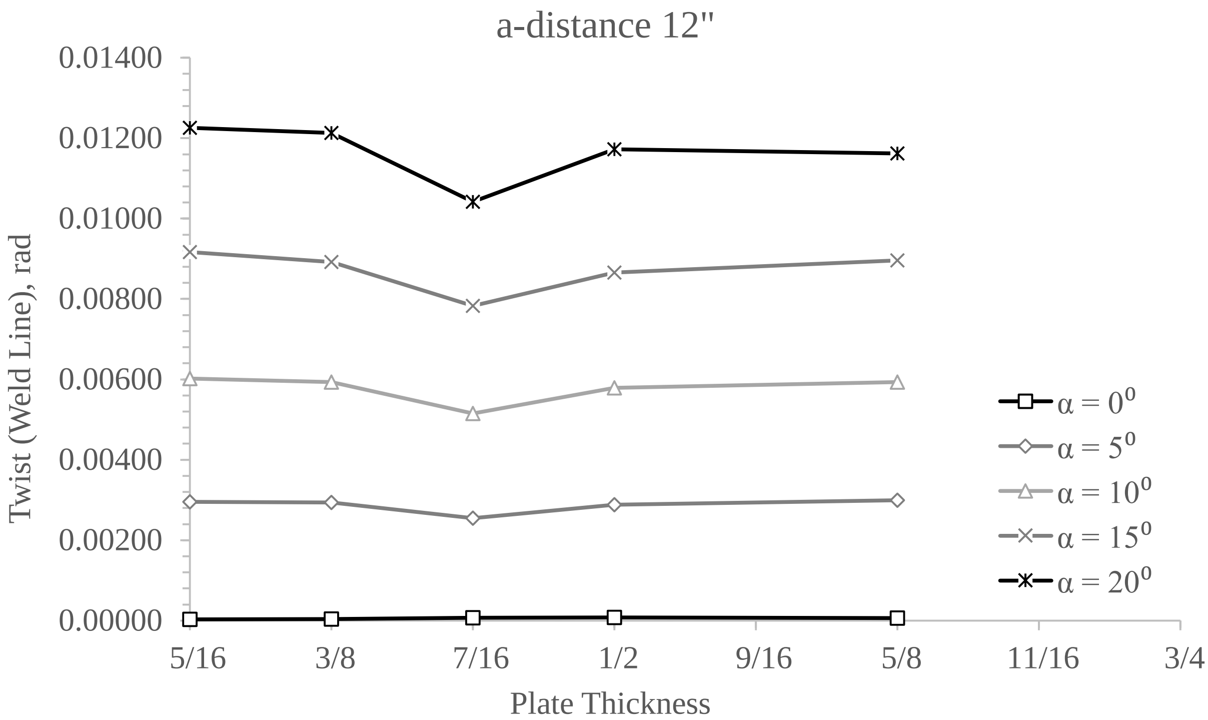

a-distance = 11.5"
Plate Relative Twist, rad
Skewed Angle, degrees
Plate Thickness, in
x 10^-3

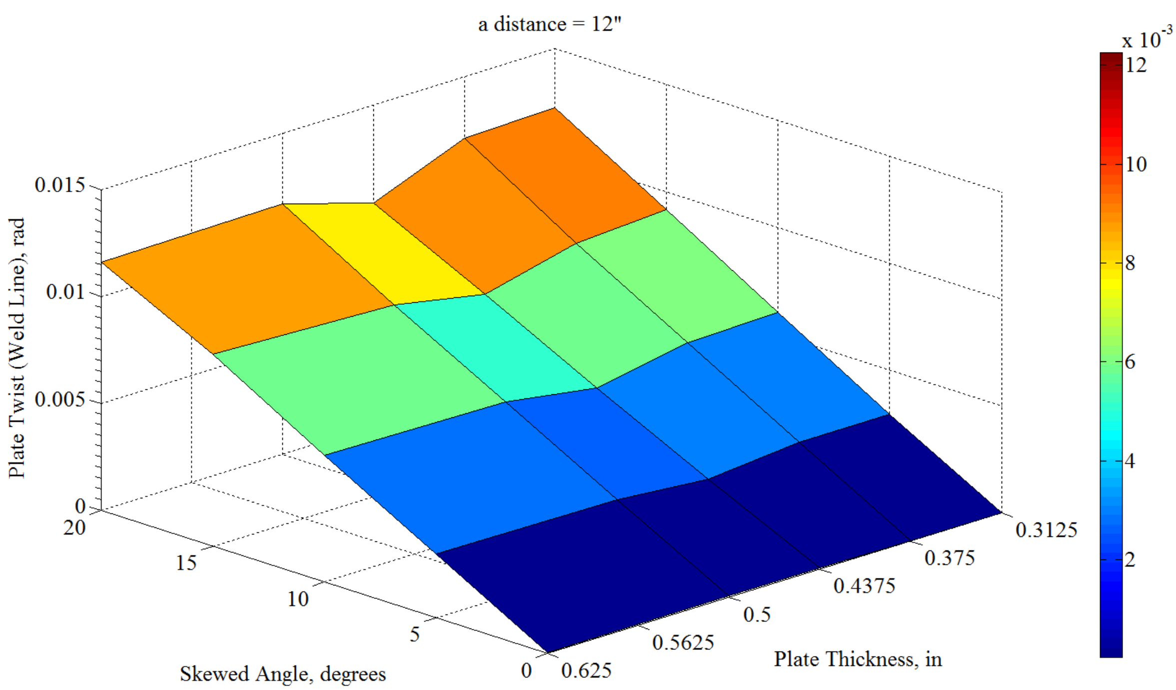

a-distance 11.5"
Shear, lbs
Plate Thickness, in
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

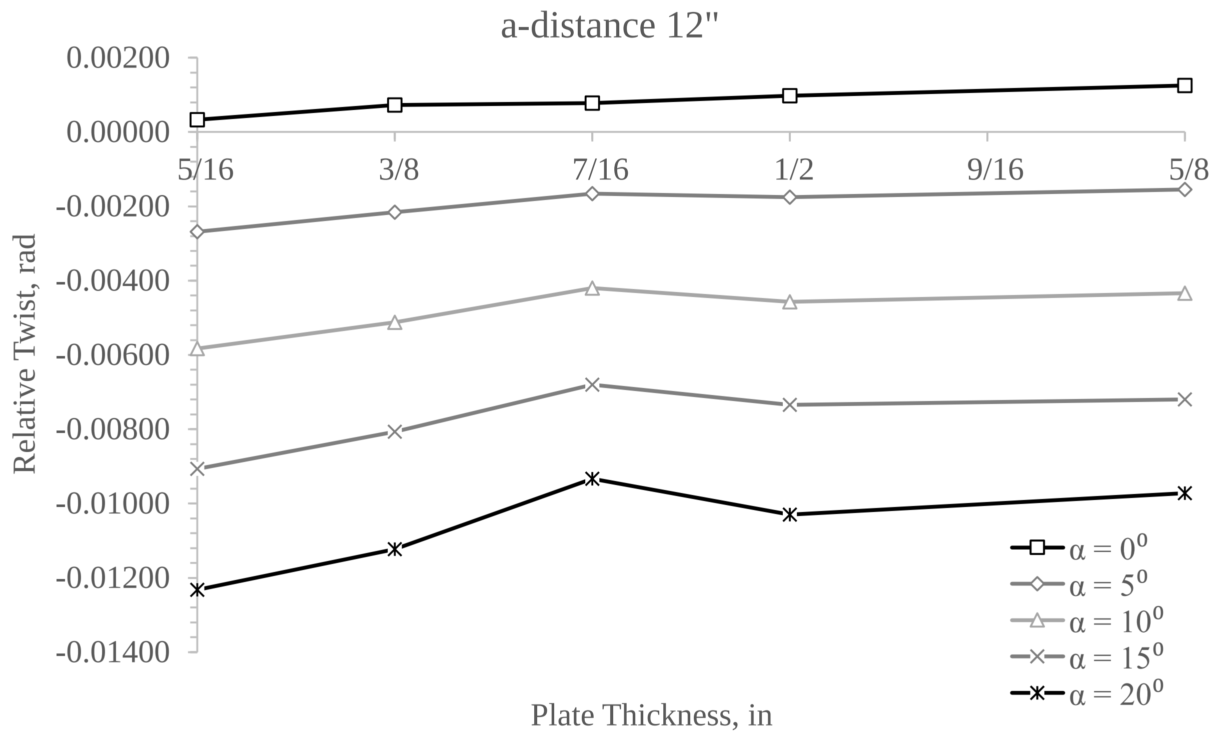
Eight Bolted Connections (a-distance = 12”) Curves



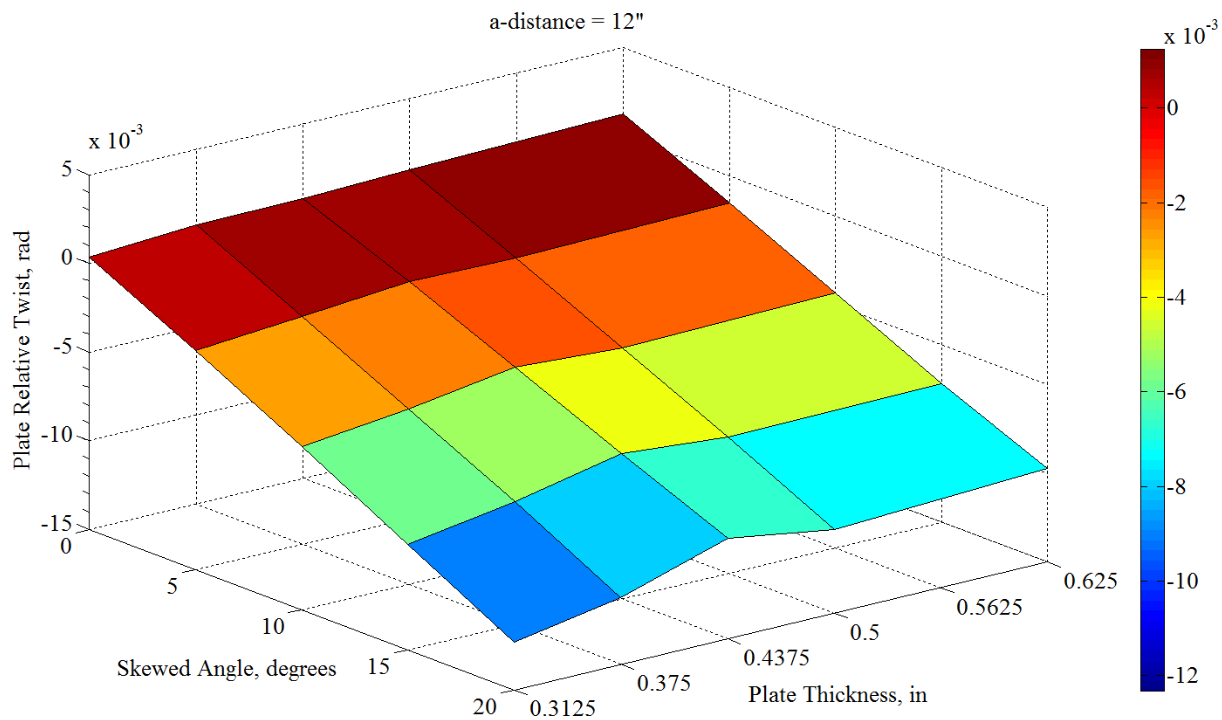



a-distance 12"
0.01400
0.01200
0.01000
0.00800
0.00600
0.00400
0.00200
0.00000
Twist (Weld Line), rad
5/16
3/8
7/16
1/2
9/16
5/8
11/16
3/4
Plate Thickness
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

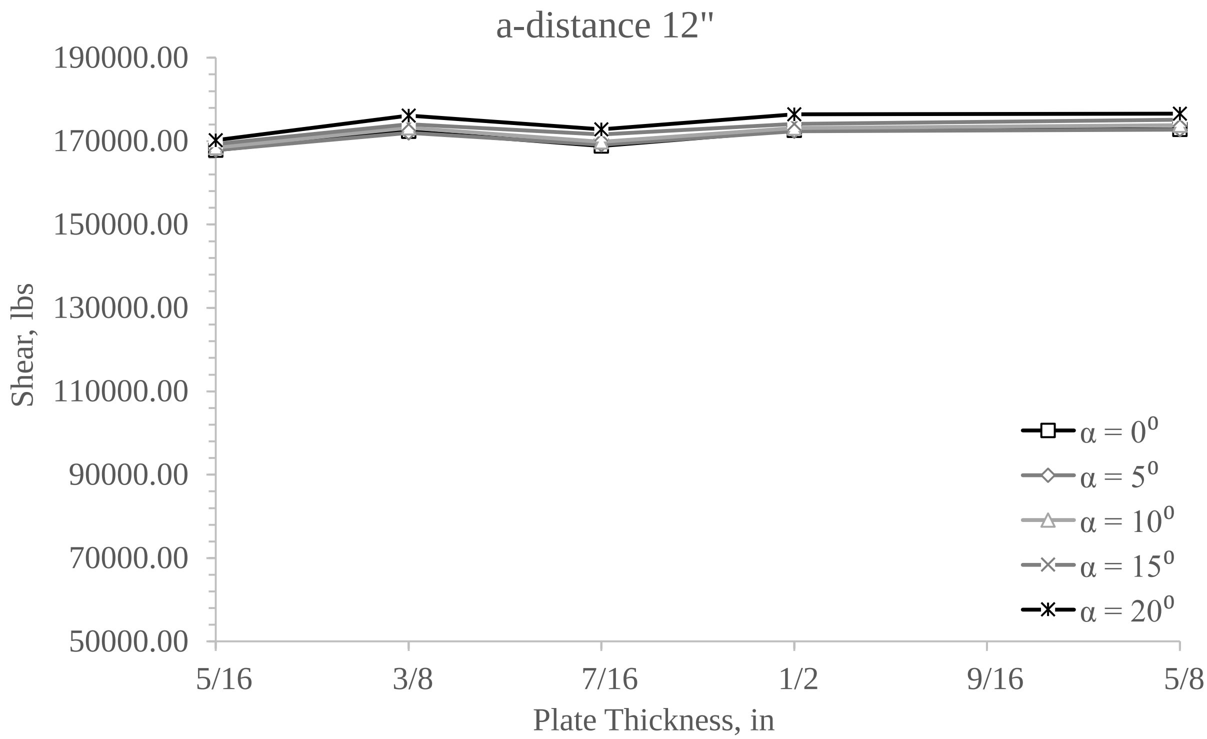

a distance = 12"
Plate Twist (Weld Line), rad
0.015
0.01
0.005
0
20
15
10
5
0
Skewed Angle, degrees
0.625
0.5625
0.5
0.4375
0.375
0.3125
Plate Thickness, in
x 10-3
12
10
8
6
4
2



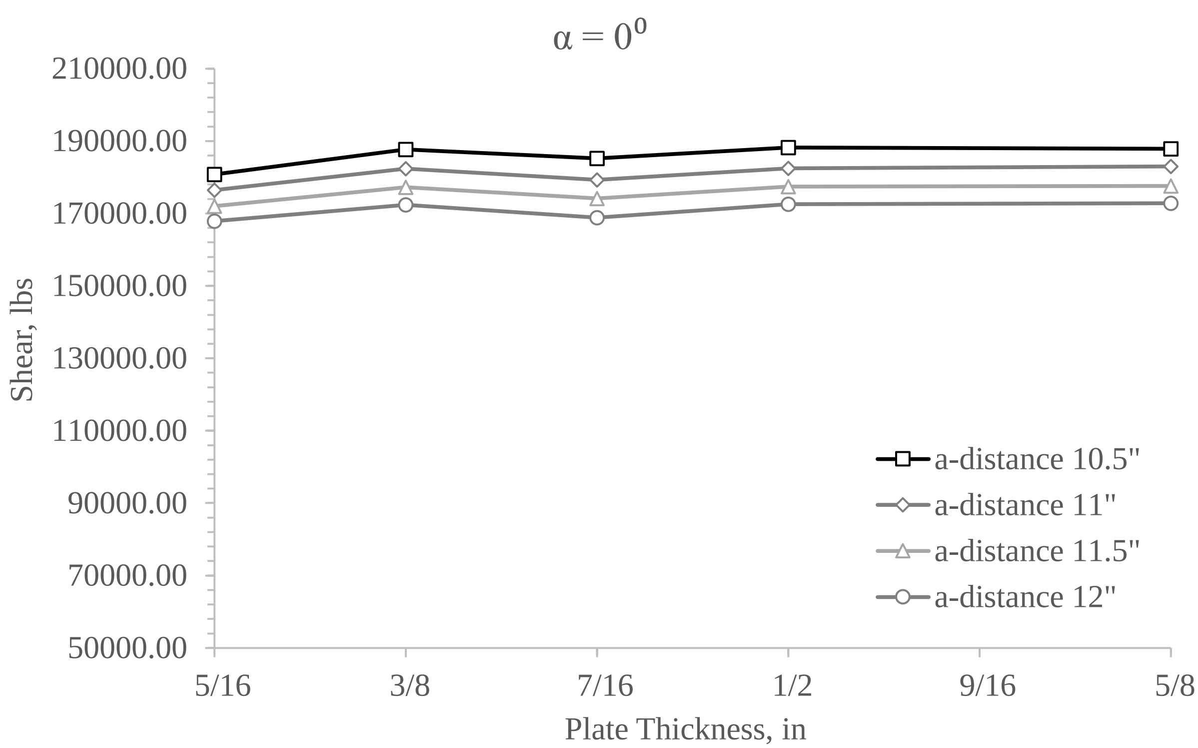
a-distance 12"
0.00200
0.00000
-0.00200
-0.00400
-0.00600
-0.00800
-0.01000
-0.01200
-0.01400
5/16
3/8
7/16
1/2
9/16
5/8
Relative Twist, rad
Plate Thickness, in
α = 0⁰
α = 5⁰
α = 10⁰
α = 15⁰
α = 20⁰

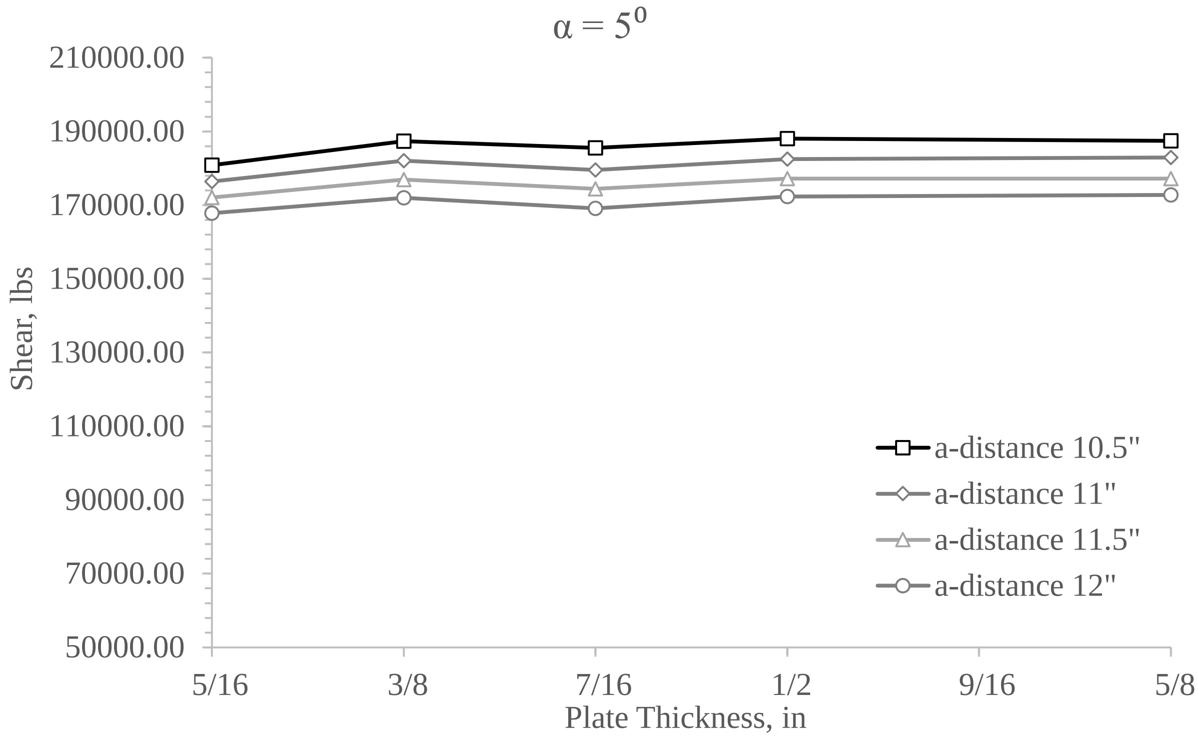


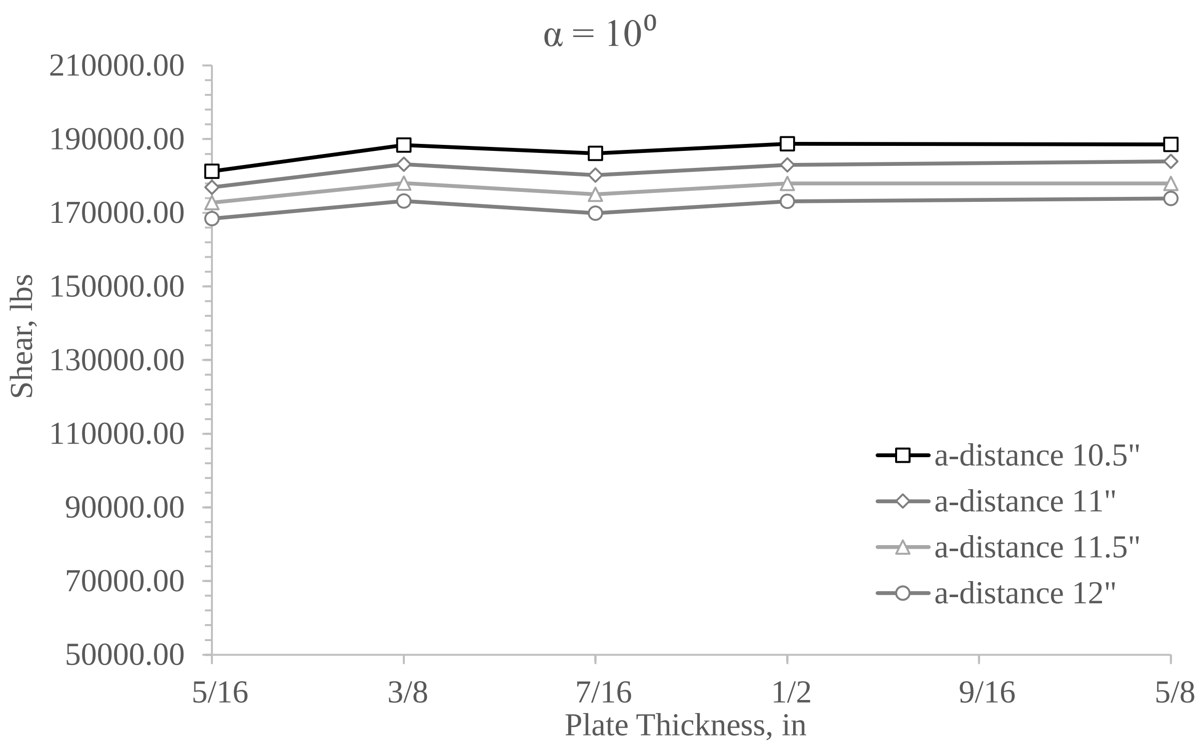
a-distance = 12"
x 10^-3
5
0
-5
-10
-15
Plate Relative Twist, rad
0
5
10
15
20
Skewed Angle, degrees
0.3125
0.375
0.4375
0.5
0.5625
0.625
Plate Thickness, in
x 10^-3
0
-2
-4
-6
-8
-10
-12

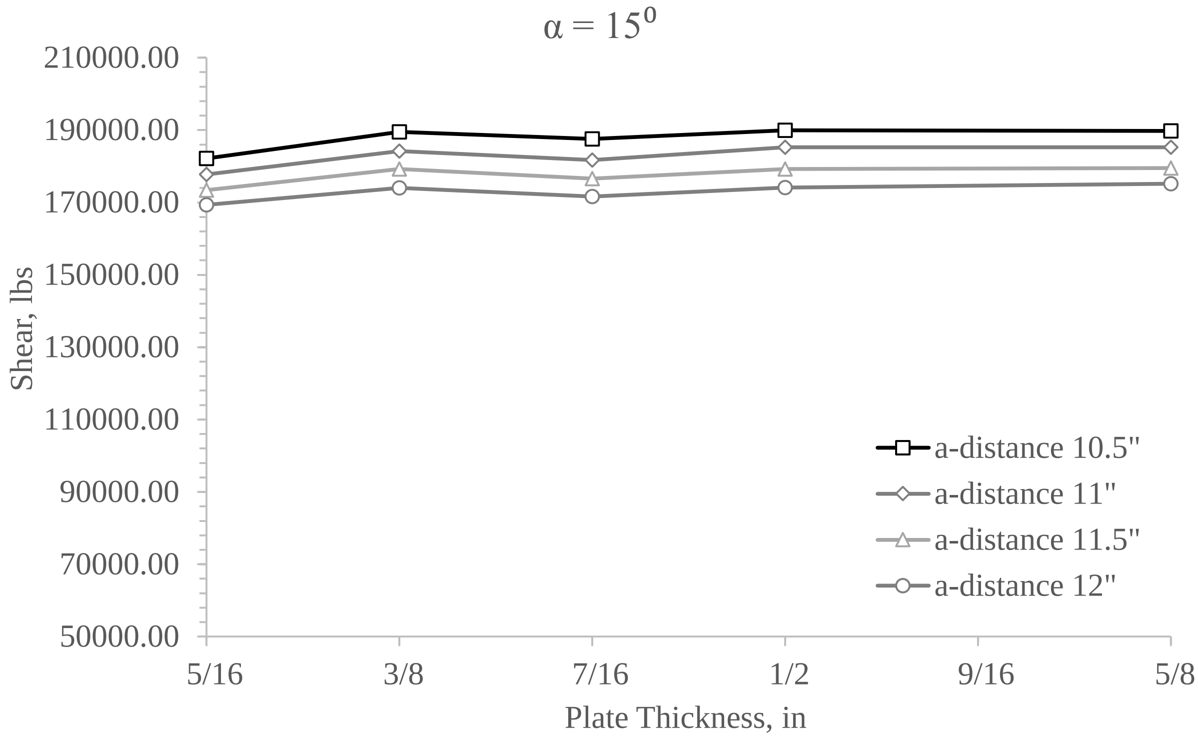


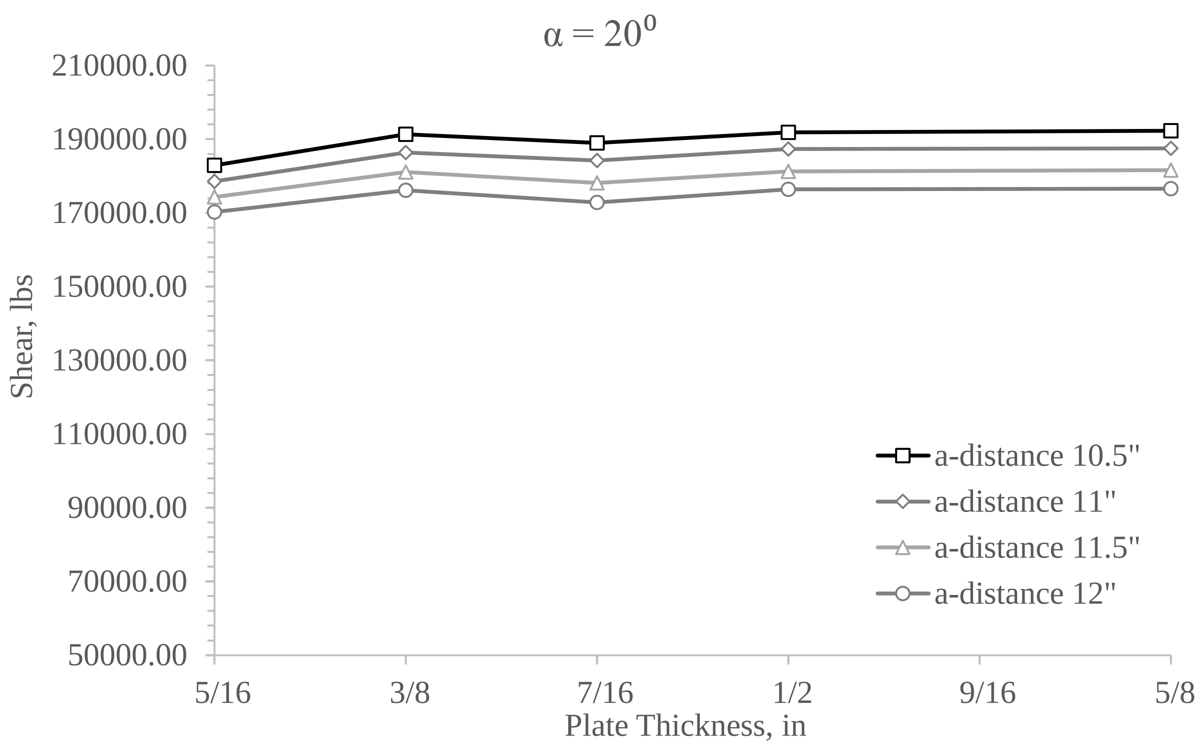
Eight Bolted Connections (0^0, 5^0, 10^0, 15^0, and 20^0) Curves




α = 5⁰
210000.00
190000.00
170000.00
150000.00
130000.00
110000.00
90000.00
70000.00
50000.00
Shear, lbs
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


α = 10⁰
210000.00
190000.00
170000.00
150000.00
130000.00
110000.00
90000.00
70000.00
50000.00
Shear, lbs
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


α = 15⁰
210000.00
190000.00
170000.00
150000.00
130000.00
110000.00
90000.00
70000.00
50000.00
Shear, lbs
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"


α = 20⁰
210000.00
190000.00
170000.00
150000.00
130000.00
110000.00
90000.00
70000.00
50000.00
Shear, lbs
5/16
3/8
7/16
1/2
9/16
5/8
Plate Thickness, in
a-distance 10.5"
a-distance 11"
a-distance 11.5"
a-distance 12"

Appendix C (Errors of Using the Proposed Equations – Chapter 7)

C1. Three Bolted Connections

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
1/4	7	0	1.4987	1.5190	-1.35	---	---	---
		5	1.5110	1.5155	-0.29	0.00657	0.0069	-5.02
		10	1.5662	1.5119	3.47	0.01416	0.0138	2.54
		15	1.4947	1.5084	-0.91	0.02163	0.0207	4.30
		20	1.4892	1.5048	-1.05	0.02750	0.0276	-0.36
	7.5	0	1.7379	1.7410	-0.18	---	---	---
		5	1.6246	1.6645	-2.46	0.00551	0.0073	-31.58
		10	1.6318	1.5880	2.68	0.01513	0.0145	4.16
		15	1.5566	1.5115	2.90	0.02292	0.0218	5.10
		20	1.3895	1.4350	-3.27	0.02828	0.0290	-2.55
	8	0	1.7322	1.7310	0.07	---	---	---
		5	1.6368	1.6472	-0.63	0.00743	0.0082	-10.36
		10	1.5724	1.5633	0.58	0.01549	0.0154	0.58
		15	1.4845	1.4795	0.34	0.02333	0.0226	3.13
		20	1.3891	1.3956	-0.47	0.02902	0.0298	-2.69
	8.5	0	1.6575	1.6790	-1.30	---	---	---
		5	1.6261	1.6209	0.32	0.00753	0.0084	-10.89
		10	1.5932	1.5628	1.91	0.01563	0.0157	-0.45
		15	1.5169	1.5047	0.80	0.02362	0.0231	2.41
		20	1.4215	1.4466	-1.77	0.02955	0.0304	-2.88

t (in)	a-distance (in)	α (degrees)	Relative Twist, rad			Shear, lbs		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
1/4	7	0	0.05082	0.0500	1.61	55052	55063	-0.02
		5	0.05022	0.0509	-1.35	55134	55166	-0.06
		10	0.05104	0.0518	-1.49	55345	55269	0.14
		15	0.05061	0.0527	-4.13	55362	55372	-0.02
		20	0.05522	0.0536	2.93	55452	55474	-0.04
	7.5	0	0.04627	0.0460	0.58	53946	53990	-0.08
		5	0.04590	0.0466	-1.42	53792	53791	0.00
		10	0.04585	0.0471	-2.73	53631	53592	0.07
		15	0.04783	0.0477	0.38	53493	53393	0.19
		20	0.04798	0.0482	-0.46	53099	53193	-0.18
	8	0	0.04011	0.0390	2.77	51680	51589	0.17
		5	0.03907	0.0390	0.31	51311	51381	-0.14
		10	0.03764	0.0389	-3.35	51064	51172	-0.21
		15	0.03772	0.0389	-3.00	51027	50963	0.12
		20	0.04045	0.0388	4.08	50778	50755	0.05
	8.5	0	0.03566	0.0350	1.85	48905	48851	0.11
		5	0.03490	0.0350	-0.14	48774	48761	0.03
		10	0.03378	0.0349	-3.32	48532	48672	-0.29
		15	0.03428	0.0349	-1.66	48605	48582	0.05
		20	0.03564	0.0348	2.36	48543	48493	0.10

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
5/16	7	0	1.0453	1.0520	-0.64	---	---	---
		5	1.0493	1.0495	-0.02	0.00817	0.0081	1.47
		10	1.0550	1.0470	0.76	0.01647	0.0161	2.25
		15	1.0552	1.0445	1.01	0.02412	0.0242	-0.12
		20	1.0298	1.0420	-1.18	0.03235	0.0322	0.46
	7.5	0	1.1319	1.1340	-0.19	---	---	---
		5	1.1378	1.1358	0.18	0.00825	0.0083	-0.61
		10	1.1374	1.1376	-0.02	0.01665	0.0166	0.30
		15	1.1385	1.1394	-0.08	0.02441	0.0249	-2.01
		20	1.1406	1.1412	-0.05	0.03335	0.0332	0.45
	8	0	1.2053	1.2050	0.02	---	---	---
		5	1.2099	1.2062	0.31	0.00822	0.0083	-0.97
		10	1.2018	1.2074	-0.47	0.01658	0.0166	-0.12
		15	1.2039	1.2086	-0.39	0.02432	0.0249	-2.38
		20	1.2142	1.2098	0.36	0.03340	0.0332	0.60
	8.5	0	1.2775	1.2780	-0.04	---	---	---
		5	1.2772	1.2780	-0.06	0.00819	0.0083	-1.34
		10	1.2794	1.2780	0.11	0.01666	0.0166	0.36
		15	1.2826	1.2780	0.36	0.02450	0.0249	-1.63
		20	1.2748	1.2780	-0.25	0.03334	0.0332	0.42

t (in)	a-distance (in)	α (degrees)	Relative Twist, rad			Shear, lbs		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
5/16	7	0	0.04053	0.0400	1.31	56160	55814	0.62
		5	0.03475	0.0360	-3.45	55792	56155	-0.65
		10	0.03037	0.0319	-5.04	56255	56497	-0.43
		15	0.02756	0.0279	-1.05	57026	56838	0.33
		20	0.02381	0.0238	0.04	57249	57179	0.12
	7.5	0	0.03797	0.0370	2.55	53372	53007	0.68
		5	0.03217	0.0329	-2.11	52998	53370	-0.70
		10	0.02770	0.0287	-3.61	53490	53732	-0.45
		15	0.02473	0.0246	0.73	54237	54094	0.26
		20	0.02088	0.0204	2.30	54564	54457	0.20
	8	0	0.03549	0.0350	1.38	50524	50124	0.79
		5	0.02974	0.0309	-3.73	50135	50528	-0.78
		10	0.02531	0.0267	-5.49	50653	50932	-0.55
		15	0.02227	0.0226	-1.26	51477	51336	0.27
		20	0.01837	0.0184	-0.16	51873	51740	0.26
	8.5	0	0.03361	0.0330	1.81	48055	47665	0.81
		5	0.02789	0.0288	-3.08	47651	48040	-0.82
		10	0.02345	0.0245	-4.48	48155	48415	-0.54
		15	0.02026	0.0203	0.05	48915	48790	0.26
		20	0.01629	0.0160	1.78	49298	49165	0.27

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
3/8	7	0	0.9585	0.9940	-3.70	---	---	---
		5	0.9966	0.9677	2.91	0.00849	0.0086	-0.71
		10	0.9746	0.9413	3.41	0.01680	0.0161	4.17
		15	0.9037	0.9150	-1.25	0.02357	0.0237	-0.34
		20	0.8734	0.8886	-1.74	0.03074	0.0312	-1.50
	7.5	0	1.1094	1.1120	-0.24	---	---	---
		5	1.1093	1.1105	-0.11	0.00865	0.0087	0.00
		10	1.1136	1.1090	0.41	0.01763	0.0173	1.87
		15	1.1089	1.1075	0.13	0.02604	0.0260	0.35
		20	1.1020	1.1060	-0.36	0.03484	0.0346	0.69
	8	0	1.1866	1.1890	-0.20	---	---	---
		5	1.1920	1.1901	0.16	0.00865	0.0088	-1.16
		10	1.1934	1.1912	0.19	0.01768	0.0175	1.02
		15	1.1900	1.1923	-0.20	0.02617	0.0263	-0.31
		20	1.1932	1.1934	-0.02	0.03526	0.0350	0.74
	8.5	0	1.2390	1.2490	-0.81	---	---	---
		5	1.2587	1.2513	0.59	0.00861	0.0088	-1.63
		10	1.2581	1.2536	0.36	0.01766	0.0175	0.91
		15	1.2619	1.2559	0.47	0.02625	0.0263	0.00
		20	1.2489	1.2582	-0.74	0.03507	0.0350	0.20

t (in)	a-distance (in)	α (degrees)	Relative Twist, rad			Shear, lbs		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
3/8	7	0	0.03709	0.0370	0.24	57232	57237	-0.01
		5	0.03258	0.0332	-1.75	57241	57235	0.01
		10	0.02790	0.0293	-5.02	57236	57234	0.00
		15	0.02466	0.0255	-3.20	57230	57232	0.00
		20	0.02170	0.0216	0.46	57229	57230	0.00
	7.5	0	0.03650	0.0360	1.37	54950	54547	0.73
		5	0.03100	0.0320	-3.23	54634	54909	-0.50
		10	0.02646	0.0280	-5.82	54818	55270	-0.82
		15	0.02362	0.0240	-1.61	55750	55632	0.21
		20	0.02017	0.0200	0.84	56200	55994	0.37
	8	0	0.03452	0.0340	1.51	52200	51807	0.75
		5	0.02918	0.0300	-2.64	51927	52183	-0.49
		10	0.02465	0.0259	-5.07	52103	52560	-0.88
		15	0.02169	0.0219	-0.74	53050	52937	0.21
		20	0.01806	0.0178	1.44	53523	53314	0.39
	8.5	0	0.03280	0.0320	2.44	49513	49163	0.71
		5	0.02776	0.0280	-0.68	49331	49537	-0.42
		10	0.02324	0.0239	-2.84	49448	49910	-0.94
		15	0.02007	0.0199	1.10	50427	50284	0.28
		20	0.01630	0.0158	3.07	50833	50657	0.35

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
1/2	7	0	0.9376	0.9410	-0.36	---	---	---
		5	0.9428	0.9416	0.13	0.00950	0.0098	-3.16
		10	0.9442	0.9422	0.22	0.01825	0.0186	-1.92
		15	0.9438	0.9428	0.10	0.02714	0.0274	-0.96
		20	0.9401	0.9434	-0.35	0.03586	0.0362	-0.95
	7.5	0	0.9730	0.9710	0.20	---	---	---
		5	0.9734	0.9738	-0.05	0.00924	0.0097	-4.98
		10	0.9758	0.9766	-0.08	0.01782	0.0184	-3.25
		15	0.9770	0.9794	-0.24	0.02659	0.0271	-1.92
		20	0.9851	0.9822	0.29	0.03551	0.0358	-0.82
	8	0	1.0474	1.0530	-0.53	---	---	---
		5	1.0501	1.0534	-0.31	0.00927	0.0098	-5.18
		10	1.0686	1.0537	1.39	0.01818	0.0185	-1.76
		15	1.0520	1.0541	-0.19	0.02682	0.0273	-1.60
		20	1.0482	1.0544	-0.59	0.03550	0.0360	-1.41
	8.5	0	1.0964	1.1150	-1.70	---	---	---
		5	1.1515	1.1176	2.94	0.00946	0.0089	6.45
		10	1.1203	1.1202	0.00	0.01805	0.0177	1.94
		15	1.0975	1.1228	-2.31	0.02656	0.0266	0.04
		20	1.1364	1.1254	0.97	0.03625	0.0354	2.34

t (in)	a-distance (in)	α (degrees)	Relative Twist, rad			Shear, lbs		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
1/2	7	0	0.02925	0.0290	0.85	58572	58257	0.54
		5	0.02525	0.0263	-3.96	58402	58595	-0.33
		10	0.02258	0.0235	-4.07	58730	58933	-0.35
		15	0.01976	0.0208	-5.01	58993	59271	-0.47
		20	0.01833	0.0180	1.80	59968	59610	0.60
	7.5	0	0.02779	0.0270	2.84	55080	54698	0.69
		5	0.02403	0.0243	-1.12	54883	55091	-0.38
		10	0.02145	0.0216	-0.70	55208	55484	-0.50
		15	0.01860	0.0189	-1.61	55530	55877	-0.63
		20	0.01708	0.0162	5.15	56722	56271	0.80
	8	0	0.02715	0.0270	0.55	52381	52103	0.53
		5	0.02344	0.0242	-3.24	52219	52462	-0.47
		10	0.02080	0.0214	-2.88	52778	52821	-0.08
		15	0.01768	0.0186	-5.20	52883	53179	-0.56
		20	0.01597	0.0158	1.06	53843	53538	0.57
	8.5	0	0.02635	0.0260	1.33	49677	49569	0.22
		5	0.02305	0.0231	0.00	50197	49969	0.45
		10	0.01989	0.0201	-1.06	50109	50369	-0.52
		15	0.01663	0.0172	-3.13	50169	50768	-1.20
		20	0.01481	0.0142	4.12	51691	51168	1.01

C2. Five Bolted Connections

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
5/16	10.5	0	0.3975	0.3980	-0.13	---	---	---
		5	0.3935	0.3930	0.13	0.00193	0.0018	6.74
		10	0.3847	0.3880	-0.86	0.00375	0.0036	4.00
		15	0.3810	0.3830	-0.52	0.00552	0.0054	2.17
		20	0.3726	0.3780	-1.45	0.00715	0.0072	-0.70
	11	0	0.4019	0.4060	-1.02	---	---	---
		5	0.4012	0.3960	1.30	0.00196	0.0018	8.16
		10	0.3915	0.3860	1.40	0.00381	0.0036	5.51
		15	0.3807	0.3760	1.23	0.00554	0.0054	2.53
		20	0.3690	0.3660	0.81	0.00714	0.0072	-0.84
	11.5	0	0.4222	0.4280	-1.37	---	---	---
		5	0.4244	0.4180	1.51	0.00204	0.0019	9.31
		10	0.4126	0.4080	1.11	0.00397	0.0037	6.80
		15	0.4004	0.3980	0.60	0.00576	0.0056	3.65
		20	0.3881	0.3880	0.03	0.00744	0.0074	0.54
	12	0	0.4467	0.4520	-1.19	---	---	---
		5	0.4439	0.4370	1.55	0.00210	0.0019	9.52
		10	0.4248	0.4220	0.66	0.00403	0.0038	5.71
		15	0.4130	0.4070	1.45	0.00585	0.0057	2.56
		20	0.3939	0.3920	0.48	0.00746	0.0076	-1.88

t (in)	a-distance (in)	α (degrees)	Relative Twist, rad			Shear, lbs		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
5/16	10.5	0	0.07187	0.0720	-0.18	73714	73562	0.21
5/16	10.5	5	0.07342	0.0737	-0.38	73350	73431	-0.11
5/16	10.5	10	0.07475	0.0754	-0.87	73130	73300	-0.23
5/16	10.5	15	0.07676	0.0771	-0.44	73146	73168	-0.03
5/16	10.5	20	0.07873	0.0788	-0.09	73160	73037	0.17
5/16	11	0	0.06826	0.0680	0.38	71589	71525	0.09
5/16	11	5	0.06970	0.0696	0.14	71368	71353	0.02
5/16	11	10	0.07127	0.0712	0.10	71044	71180	-0.19
5/16	11	15	0.07280	0.0728	0.00	70972	71008	-0.05
5/16	11	20	0.07473	0.0744	0.44	70926	70836	0.13
5/16	11.5	0	0.06431	0.0640	0.48	69915	69899	0.02
5/16	11.5	5	0.06581	0.0657	0.17	69797	69739	0.08
5/16	11.5	10	0.06723	0.0674	-0.25	69494	69579	-0.12
5/16	11.5	15	0.06899	0.0691	-0.16	69354	69420	-0.09
5/16	11.5	20	0.07113	0.0708	0.46	69338	69260	0.11
5/16	12	0	0.06062	0.0600	1.02	68325	68294	0.04
5/16	12	5	0.06210	0.0618	0.48	68013	67996	0.02
5/16	12	10	0.06324	0.0636	-0.57	67607	67698	-0.13
5/16	12	15	0.06555	0.0654	0.23	67408	67400	0.01
5/16	12	20	0.06778	0.0672	0.86	67137	67102	0.05

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
3/8	10.5	0	0.8241	0.8190	0.62	---	---	---
		5	0.8225	0.8240	-0.18	0.00460	0.0046	1.09
		10	0.8237	0.8290	-0.64	0.00915	0.0091	0.55
		15	0.8301	0.8340	-0.47	0.01376	0.0137	0.80
		20	0.8448	0.8390	0.69	0.01839	0.0182	1.03
	11	0	0.8414	0.8430	-0.19	---	---	---
		5	0.8454	0.8480	-0.31	0.00461	0.0046	1.30
		10	0.8502	0.8530	-0.33	0.00922	0.0091	1.30
		15	0.8492	0.8580	-1.04	0.01379	0.0137	1.02
		20	0.8521	0.8630	-1.28	0.01830	0.0182	0.55
	11.5	0	0.8613	0.8650	-0.43	---	---	---
		5	0.8728	0.8700	0.32	0.00460	0.0046	0.00
		10	0.8774	0.8750	0.27	0.00921	0.0092	0.11
		15	0.8798	0.8800	-0.02	0.01386	0.0138	0.43
		20	0.8811	0.8850	-0.44	0.01842	0.0184	0.11
	12	0	0.9037	0.9020	0.19	---	---	---
		5	0.9035	0.9020	0.17	0.00459	0.0046	-0.22
		10	0.9034	0.9020	0.15	0.00917	0.0092	-0.33
		15	0.9110	0.9020	0.99	0.01388	0.0138	0.58
		20	0.9104	0.9020	0.92	0.01846	0.0184	0.33

t (in)	a-distance (in)	α (degrees)	Relative Twist, rad			Shear, lbs		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
3/8	10.5	0	0.05666	0.0550	2.93	89167	88725	0.50
		5	0.05500	0.0552	-0.36	89106	89322	-0.24
		10	0.05445	0.0554	-1.74	89499	89918	-0.47
		15	0.05512	0.0556	-0.87	90238	90515	-0.31
		20	0.05753	0.0558	3.01	91584	91112	0.52
	11	0	0.05345	0.0520	2.71	85943	85498	0.52
		5	0.05179	0.0519	-0.12	85862	86089	-0.26
		10	0.05114	0.0517	-1.10	86259	86679	-0.49
		15	0.05148	0.0516	-0.14	87005	87270	-0.30
		20	0.05297	0.0514	2.96	88324	87861	0.53
	11.5	0	0.05054	0.0500	1.07	82876	82442	0.52
		5	0.04899	0.0498	-1.65	82861	83049	-0.23
		10	0.04822	0.0496	-2.86	83214	83656	-0.53
		15	0.04848	0.0494	-1.90	83973	84263	-0.35
		20	0.04971	0.0492	1.03	85355	84870	0.57
	12	0	0.04825	0.0470	2.59	80085	79642	0.55
		5	0.04658	0.0467	-0.26	79999	80215	-0.27
		10	0.04572	0.0464	-1.49	80342	80789	-0.56
		15	0.04590	0.0461	-0.44	81132	81362	-0.28
		20	0.04701	0.0458	2.57	82385	81935	0.55

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
7/16	10.5	0	0.721922934	0.7130	1.24	---	---	---
		5	0.716273897	0.7180	-0.24	0.00463	0.0046	1.73
		10	0.714860845	0.7230	-1.14	0.00893	0.0091	-1.90
		15	0.718714879	0.7280	-1.29	0.01339	0.0137	-1.94
		20	0.747075914	0.7330	1.88	0.01857	0.0182	1.99
	11	0	0.693227801	0.6830	1.48	---	---	---
		5	0.695720726	0.7030	-1.05	0.00435	0.0046	-4.60
		10	0.733683716	0.7230	1.46	0.00880	0.0091	-3.41
		15	0.70496175	0.7430	-5.40	0.01268	0.0137	-7.65
		20	0.789095768	0.7630	3.31	0.01880	0.0182	3.19
	11.5	0	0.720009704	0.7240	-0.55	---	---	---
		5	0.723353626	0.7240	-0.09	0.00433	0.0042	4.16
		10	0.742348621	0.7240	2.47	0.00858	0.0083	3.26
		15	0.726803655	0.7240	0.39	0.01259	0.0125	1.11
		20	0.727954675	0.7240	0.54	0.01692	0.0166	1.89
	12	0	0.749336635	0.7490	0.04	---	---	---
		5	0.749872559	0.7540	-0.55	0.00430	0.0042	2.33
		10	0.754159553	0.7590	-0.64	0.00840	0.0084	0.00
		15	0.784973586	0.7640	2.67	0.01303	0.0126	3.30
		20	0.757427608	0.7690	-1.53	0.01694	0.0168	0.83

t (in)	a-distance (in)	α (degrees)	Relative Twist, rad			Shear, lbs		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
7/16	10.5	0	0.04737	0.0460	2.89	89202	88582	0.69
		5	0.04505	0.0453	-0.55	88791	89156	-0.41
		10	0.04399	0.0446	-1.39	89150	89730	-0.65
		15	0.04386	0.0439	-0.09	90086	90304	-0.24
		20	0.04444	0.0432	2.79	91424	90878	0.60
	11	0	0.04447	0.0440	1.06	84950	84331	0.73
		5	0.04250	0.0435	-2.35	84692	85131	-0.52
		10	0.04216	0.0430	-1.99	85694	85930	-0.28
		15	0.04135	0.0425	-2.78	86046	86730	-0.79
		20	0.04262	0.0420	1.45	88271	87529	0.84
	11.5	0	0.04289	0.0420	2.08	81873	81492	0.46
		5	0.04099	0.0413	-0.63	81645	81985	-0.42
		10	0.04027	0.0405	-0.57	82296	82478	-0.22
		15	0.03980	0.0398	0.13	82838	82971	-0.16
		20	0.03983	0.0390	2.08	83741	83464	0.33
	12	0	0.04157	0.0410	1.37	79011	78553	0.58
		5	0.03963	0.0403	-1.69	78731	79098	-0.47
		10	0.03874	0.0396	-2.22	79152	79643	-0.62
		15	0.03879	0.0389	-0.28	80438	80188	0.31
		20	0.03840	0.0382	0.52	80883	80733	0.18

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
1/2	10.5	0	0.70798461	0.7280	-2.83	---	---	---
		5	0.74267152	0.7280	1.98	0.00507	0.0046	9.27
		10	0.74400951	0.7280	2.15	0.00979	0.0092	6.03
		15	0.71330144	0.7280	-2.06	0.01411	0.0138	2.20
		20	0.71495844	0.7280	-1.82	0.01881	0.0184	2.18
	11	0	0.76794245	0.7770	-1.18	---	---	---
		5	0.76954137	0.7670	0.33	0.00501	0.0046	9.18
		10	0.77527235	0.7570	2.36	0.00976	0.0091	6.76
		15	0.73178229	0.7470	-2.08	0.01389	0.0137	1.73
		20	0.73083529	0.7370	-0.84	0.01847	0.0182	1.46
	11.5	0	0.78722633	0.7890	-0.23	---	---	---
		5	0.79179625	0.7890	0.35	0.00494	0.0048	3.85
		10	0.79398023	0.7890	0.63	0.00961	0.0095	1.14
		15	0.79562117	0.7890	0.83	0.01443	0.0143	1.25
		20	0.79653818	0.7890	0.95	0.01924	0.0190	1.25
	12	0	0.82194824	0.8330	-1.34	---	---	---
		5	0.85911816	0.8380	2.46	0.00509	0.0049	3.73
		10	0.83789714	0.8430	-0.61	0.00971	0.0098	-0.93
		15	0.83790309	0.8480	-1.21	0.01458	0.0147	-0.82
		20	0.85860309	0.8530	0.65	0.01986	0.0196	1.31

t (in)	a-distance (in)	α (degrees)	Relative Twist, rad			Shear, lbs		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
1/2	10.5	0	0.04128	0.0410	0.68	90111	89988	0.14
		5	0.03980	0.0400	-0.38	90441	90421	0.02
		10	0.03863	0.0389	-0.70	90771	90854	-0.09
		15	0.03736	0.0379	-1.31	90899	91288	-0.43
		20	0.03722	0.0368	1.13	92049	91721	0.36
	11	0	0.04063	0.0400	1.55	87231	86955	0.32
		5	0.03847	0.0388	-0.86	87057	87242	-0.21
		10	0.03738	0.0376	-0.59	87459	87528	-0.08
		15	0.03604	0.0364	-1.00	87407	87815	-0.47
		20	0.03583	0.0352	1.76	88488	88101	0.44
	11.5	0	0.03928	0.0390	0.71	83885	83506	0.45
		5	0.03731	0.0380	-1.85	83803	84009	-0.25
		10	0.03619	0.0370	-2.24	84176	84512	-0.40
		15	0.03540	0.0360	-1.69	84796	85015	-0.26
		20	0.03512	0.0350	0.34	85904	85518	0.45
	12	0	0.03828	0.0380	0.73	81028	80731	0.37
		5	0.03668	0.0370	-0.87	81243	81273	-0.04
		10	0.03535	0.0360	-1.84	81435	81816	-0.47
		15	0.03455	0.0350	-1.30	82022	82358	-0.41
		20	0.03440	0.0340	1.16	83350	82900	0.54

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
5/8	10.5	0	0.6995	0.6990	0.07	---	---	---
		5	0.7003	0.7140	-1.96	0.00518	0.0051	1.54
		10	0.7435	0.7290	1.95	0.01047	0.0102	2.58
		15	0.7329	0.7440	-1.51	0.01537	0.0153	0.46
		20	0.7487	0.7590	-1.38	0.02090	0.0204	2.39
	11	0	0.7285	0.7280	0.07	---	---	---
		5	0.7293	0.7330	-0.51	0.00513	0.0049	4.48
		10	0.7314	0.7380	-0.90	0.00990	0.0098	1.01
		15	0.7642	0.7430	2.77	0.01535	0.0147	4.23
		20	0.7340	0.7480	-1.91	0.01975	0.0196	0.76
	11.5	0	0.7535	0.7590	-0.73	---	---	---
		5	0.7546	0.7640	-1.25	0.00508	0.0049	3.54
		10	0.7907	0.7690	2.74	0.01023	0.0098	4.20
		15	0.7935	0.7740	2.46	0.01529	0.0147	3.86
		20	0.7659	0.7790	-1.71	0.01977	0.0196	0.86
	12	0	0.7845	0.7980	-1.72	---	---	---
		5	0.8209	0.7980	2.79	0.00526	0.0048	8.75
		10	0.7870	0.7980	-1.40	0.00980	0.0096	2.04
		15	0.7885	0.7980	-1.20	0.01465	0.0144	1.71
		20	0.7910	0.7980	-0.88	0.01963	0.0192	2.19

t (in)	a-distance (in)	α (degrees)	Relative Twist, rad			Shear, lbs		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
5/8	10.5	0	0.03089	0.0300	2.88	90993	90534	0.50
		5	0.02916	0.0292	-0.14	90804	91209	-0.45
		10	0.02873	0.0284	1.15	91656	91884	-0.25
		15	0.02784	0.0276	0.86	92398	92559	-0.17
		20	0.02758	0.0268	2.83	93572	93234	0.36
	11	0	0.03035	0.0300	1.15	87610	87193	0.48
		5	0.02871	0.0292	-1.71	87437	87763	-0.37
		10	0.02795	0.0284	-1.61	87877	88333	-0.52
		15	0.02750	0.0276	-0.36	89120	88903	0.24
		20	0.02691	0.0268	0.41	89619	89473	0.16
	11.5	0	0.02986	0.0290	2.88	84332	83985	0.41
		5	0.02831	0.0283	0.21	84184	84599	-0.49
		10	0.02787	0.0275	1.33	85105	85213	-0.13
		15	0.02714	0.0268	1.44	85897	85827	0.08
		20	0.02658	0.0260	2.18	86547	86442	0.12
	12	0	0.02949	0.0290	1.66	81399	81179	0.27
		5	0.02832	0.0282	0.42	81732	81660	0.09
		10	0.02728	0.0274	-0.44	81690	82141	-0.55
		15	0.02659	0.0266	-0.04	82430	82622	-0.23
		20	0.02629	0.0258	1.86	83454	83103	0.42

C3. Eight Bolted Connections with Lateral Bracing

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
5/16	10.5	0	0.8393	0.8340	0.63	---	---	---
		5	0.8430	0.8440	-0.12	0.00304	0.0031	-1.97
		10	0.8403	0.8540	-1.63	0.00609	0.0062	-1.81
		15	0.8678	0.8640	0.44	0.00933	0.0093	0.32
		20	0.8731	0.8740	-0.10	0.01234	0.0124	-0.49
	11	0	0.8214	0.8140	0.90	---	---	---
		5	0.8207	0.8240	-0.40	0.00301	0.0031	-2.99
		10	0.8296	0.8340	-0.53	0.00610	0.0062	-1.64
		15	0.8429	0.8440	-0.13	0.00925	0.0093	-0.54
		20	0.8646	0.8540	1.23	0.01241	0.0124	0.08
	11.5	0	0.8025	0.7970	0.69	---	---	---
		5	0.8025	0.8070	-0.56	0.00300	0.0031	-3.33
		10	0.8149	0.8170	-0.26	0.00608	0.0062	-1.97
		15	0.8250	0.8270	-0.24	0.00921	0.0093	-0.98
		20	0.8442	0.8370	0.85	0.01235	0.0124	-0.40
	12	0	0.7784	0.7720	0.82	---	---	---
		5	0.7778	0.7870	-1.18	0.00295	0.0031	-3.39
		10	0.7947	0.8020	-0.92	0.00602	0.0061	-1.33
		15	0.8102	0.8170	-0.84	0.00917	0.0092	0.22
		20	0.8261	0.8320	-0.71	0.01226	0.0122	0.49

t (in)	a-distance (in)	α (degrees)	Shear, lbs		
			FEA	Proposed Equation	Error (%)
5/16	10.5	0	180755	180445	0.17
		5	180811	181017	-0.11
		10	181267	181588	-0.18
		15	182179	182160	0.01
		20	182928	182731	0.11
	11	0	176437	176095	0.19
		5	176383	176656	-0.15
		10	176956	177217	-0.15
		15	177751	177778	-0.01
		20	178557	178338	0.12
	11.5	0	172047	171724	0.19
		5	172050	172324	-0.16
		10	172733	172924	-0.11
		15	173440	173524	-0.05
		20	174352	174124	0.13
	12	0	167849	167490	0.21
		5	167840	168113	-0.16
		10	168450	168736	-0.17
		15	169316	169359	-0.03
		20	170226	169982	0.14

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
3/8	10.5	0	0.5335	0.5310	0.47	---	---	---
		5	0.5328	0.5360	-0.60	0.00292	0.0030	-2.74
		10	0.5376	0.5410	-0.63	0.00591	0.0060	-1.52
		15	0.5430	0.5460	-0.55	0.00898	0.0090	-0.22
		20	0.5490	0.5510	-0.36	0.01210	0.0120	0.83
	11	0	0.5390	0.5380	0.19	---	---	---
		5	0.5425	0.5430	-0.09	0.00293	0.0030	-2.39
		10	0.5447	0.5480	-0.61	0.00590	0.0060	-1.69
		15	0.5487	0.5530	-0.78	0.00896	0.0090	-0.45
		20	0.5556	0.5580	-0.43	0.01209	0.0120	0.74
	11.5	0	0.5537	0.5520	0.31	---	---	---
		5	0.5542	0.5570	-0.51	0.00293	0.0030	-2.39
		10	0.5559	0.5620	-1.10	0.00591	0.0060	-1.52
		15	0.5608	0.5670	-1.11	0.00898	0.0090	-0.22
		20	0.5638	0.5720	-1.45	0.01209	0.0120	0.74
	12	0	0.5718	0.5700	0.31	---	---	---
		5	0.5710	0.5700	0.18	0.00294	0.0030	-2.04
		10	0.5738	0.5700	0.66	0.00593	0.0060	-1.18
		15	0.5703	0.5700	0.05	0.00892	0.0090	-0.90
		20	0.5786	0.5700	1.49	0.01213	0.0120	1.07

t (in)	a-distance (in)	α (degrees)	Shear, lbs		
			FEA	Proposed Equation	Error (%)
3/8	10.5	0	187716	186995	0.38
		5	187376	187931	-0.30
		10	188409	188867	-0.24
		15	189499	189803	-0.16
		20	191335	190739	0.31
	11	0	182370	181610	0.42
		5	182050	182623	-0.31
		10	183189	183636	-0.24
		15	184218	184649	-0.23
		20	186352	185662	0.37
	11.5	0	177277	176544	0.41
		5	176947	177536	-0.33
		10	178092	178529	-0.24
		15	179232	179521	-0.16
		20	181096	180513	0.32
	12	0	172362	171633	0.42
		5	172006	172591	-0.34
		10	173192	173549	-0.21
		15	174065	174507	-0.25
		20	176122	175465	0.37

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
7/16	10.5	0	0.422802762	0.4240	-0.28	---	---	---
		5	0.423329707	0.4240	-0.16	0.00259	0.0026	-0.39
		10	0.424162676	0.4240	0.04	0.00523	0.0052	0.57
		15	0.422322883	0.4240	-0.40	0.00785	0.0078	0.64
		20	0.42135507	0.4240	-0.63	0.01053	0.0104	1.23
	11	0	0.432803322	0.4310	0.42	---	---	---
		5	0.433563314	0.4360	-0.56	0.00255	0.0027	-3.92
		10	0.434467302	0.4410	-1.50	0.00514	0.0053	-3.11
		15	0.434453517	0.4460	-2.66	0.00776	0.0080	-2.45
		20	0.447280687	0.4510	-0.83	0.01073	0.0106	1.21
	11.5	0	0.452234835	0.4520	0.05	---	---	---
		5	0.452909667	0.4520	0.20	0.00256	0.0026	-1.56
		10	0.4534585	0.4520	0.32	0.00516	0.0052	-0.78
		15	0.453886534	0.4520	0.42	0.00780	0.0078	0.00
		20	0.454041552	0.4520	0.45	0.01050	0.0104	0.95
	12	0	0.467696368	0.4680	-0.06	---	---	---
		5	0.468055354	0.4680	0.01	0.00255	0.0026	-1.96
		10	0.469847365	0.4680	0.39	0.00515	0.0052	-0.97
		15	0.472938568	0.4680	1.04	0.00783	0.0078	0.38
		20	0.467582735	0.4680	-0.09	0.01042	0.0104	0.19

t (in)	a-distance (in)	α (degrees)	Shear, lbs		
			FEA	Proposed Equation	Error (%)
7/16	10.5	0	185231	184767	0.25
		5	185515	185731	-0.12
		10	186142	186696	-0.30
		15	187565	187660	-0.05
		20	189027	188624	0.21
	11	0	179269	178590	0.38
		5	179566	179795	-0.13
		10	180210	180999	-0.44
		15	181746	182204	-0.25
		20	184202	183408	0.43
	11.5	0	174106	173601	0.29
		5	174409	174631	-0.13
		10	175028	175660	-0.36
		15	176601	176690	-0.05
		20	178158	177720	0.25
	12	0	168876	168401	0.28
		5	169164	169431	-0.16
		10	169855	170461	-0.36
		15	171601	171491	0.06
		20	172807	172521	0.17

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
1/2	10.5	0	0.453855428	0.4550	-0.25	---	---	---
		5	0.455402346	0.4550	0.09	0.00287	0.0029	-1.05
		10	0.454904266	0.4550	-0.02	0.00576	0.0058	-0.69
		15	0.453349355	0.4550	-0.36	0.00868	0.0087	-0.23
		20	0.45059139	0.4550	-0.98	0.01158	0.0116	-0.17
	11	0	0.466417846	0.4650	0.30	---	---	---
		5	0.471087778	0.4700	0.23	0.00285	0.0030	-5.26
		10	0.467495617	0.4750	-1.61	0.00569	0.0060	-5.45
		15	0.485432715	0.4800	1.12	0.00892	0.0090	-0.90
		20	0.484442758	0.4850	-0.12	0.01194	0.0120	-0.50
	11.5	0	0.489805539	0.4900	-0.04	---	---	---
		5	0.490289521	0.4900	0.06	0.00285	0.0029	-1.75
		10	0.490752432	0.4900	0.15	0.00574	0.0058	-1.05
		15	0.490703526	0.4900	0.14	0.00868	0.0087	-0.23
		20	0.489729594	0.4900	-0.06	0.01163	0.0116	0.26
	12	0	0.513072457	0.5140	-0.18	---	---	---
		5	0.514072417	0.5140	0.01	0.00288	0.0029	-0.69
		10	0.514234371	0.5140	0.05	0.00579	0.0058	-0.17
		15	0.507950467	0.5140	-1.19	0.00866	0.0087	-0.46
		20	0.512715501	0.5140	-0.25	0.01172	0.0116	1.02

t (in)	a-distance (in)	α (degrees)	Shear, lbs		
			FEA	Proposed Equation	Error (%)
1/2	10.5	0	188207	187512	0.37
		5	188022	188432	-0.22
		10	188734	189353	-0.33
		15	189966	190273	-0.16
		20	191837	191194	0.34
	11	0	182470	181600	0.48
		5	182467	182860	-0.22
		10	183015	184119	-0.60
		15	185292	185379	-0.05
		20	187356	186639	0.38
	11.5	0	177446	176678	0.43
		5	177179	177657	-0.27
		10	177983	178636	-0.37
		15	179285	179615	-0.18
		20	181287	180593	0.38
	12	0	172569	171823	0.43
		5	172352	172771	-0.24
		10	173141	173719	-0.33
		15	174098	174668	-0.33
		20	176437	175616	0.47

t (in)	a-distance (in)	α (degrees)	Displacement, in			Twist $_{weld}$, rad		
			FEA	Proposed Equation	Error (%)	FEA	Proposed Equation	Error (%)
5/8	10.5	0	0.4389	0.4350	0.89	---	---	---
		5	0.4322	0.4350	-0.65	0.00287	0.0029	-1.05
		10	0.4333	0.4350	-0.39	0.00571	0.0058	-1.58
		15	0.4333	0.4350	-0.39	0.00859	0.0087	-1.28
		20	0.4375	0.4350	0.57	0.01168	0.0116	0.68
	11	0	0.4645	0.4650	-0.11	---	---	---
		5	0.4651	0.4650	0.02	0.00296	0.0030	0.34
		10	0.4640	0.4650	-0.22	0.00586	0.0059	-0.68
		15	0.4645	0.4650	-0.11	0.00884	0.0089	-0.11
		20	0.4612	0.4650	-0.82	0.01182	0.0118	0.17
	11.5	0	0.4845	0.4810	0.72	---	---	---
		5	0.4752	0.4760	-0.17	0.00291	0.0029	0.34
		10	0.4682	0.4710	-0.60	0.00569	0.0058	-1.93
		15	0.4746	0.4660	1.81	0.00869	0.0087	-0.12
		20	0.4695	0.4610	1.81	0.01158	0.0116	-0.17
	12	0	0.5077	0.5110	-0.65	---	---	---
		5	0.5074	0.5060	0.28	0.00299	0.0029	3.01
		10	0.5071	0.5010	1.20	0.00593	0.0058	2.19
		15	0.5083	0.4960	2.42	0.00896	0.0087	2.90
		20	0.4888	0.4910	-0.45	0.01162	0.0116	0.17

t (in)	a-distance (in)	α (degrees)	Shear, lbs		
			FEA	Proposed Equation	Error (%)
5/8	10.5	0	187861	186953	0.48
		5	187470	188082	-0.33
		10	188584	189212	-0.33
		15	189805	190341	-0.28
		20	192340	191470	0.45
	11	0	183008	182282	0.40
		5	182953	183414	-0.25
		10	183983	184545	-0.31
		15	185281	185677	-0.21
		20	187502	186809	0.37
	11.5	0	177630	176718	0.51
		5	177193	177745	-0.31
		10	177935	178773	-0.47
		15	179483	179800	-0.18
		20	181622	180828	0.44
	12	0	172817	172232	0.34
		5	172729	173229	-0.29
		10	173846	174227	-0.22
		15	175147	175224	-0.04
		20	176595	176221	0.21

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1. Mustafa Mahamid, **Mutaz Al Hijaj**, Behavior of Stiffened Skewed Extended Shear Tab Connections, Geotechnical and Structural Engineering Congress Conference, Phoenix, Arizona, United States of America, February 2016.
2. **Mutaz Al Hijaj**, Mustafa Mahamid, Behavior of Skewed Extended Shear Tab Connections, 2015 Structures Congress Conference, Portland, Oregon, United States of America, April 2015.

Regards,
Mutaz Al Hijaj

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