TENSION CAPACITY OF BOLTED SINGLE ANGLE SHEAR CONNECTION

Final Report

Prepared by

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Abstract

The International Building Code (IBC) and the Building Code of the City of New York are incorporating requirements for proven axial capacity of shear connections with the intent of increasing the structural integrity of buildings. The language of these code changes indicate that the tension capacity should be determined under axial loading alone, not shear and axial loading combined. The New York building code recommends that the axial capacity be equal or greater than the shear capacity of the connection, but not less than 10 kips.

There have been many previously reported studies on the capacity of connections when shear and axial tension act simultaneously, but research on the capacity of shear connections under axial tensile load alone is very limited. A study in the United Kingdom investigated the tension capacity of all bolted double angle connections, but the report indicated that the results would be expected to be very different for an asymmetric connection, such as a single angle.

This research is aimed at experimental determination of the axial capacity of asymmetric single angle connections and proposing simple analytical models suitable for design use. Experiments test six all-bolted single angle connections under axial tension. Under increasing tension, asymmetric connections unfold such that the line of action of the force no longer remains normal to the connection. In a typical floor system this lateral translation would be prevented by the composite action of the slab or decking supported by the members. This effect on the tension capacity of single angle connections is a subject of these tests, which were designed to examine the difference between the laterally restrained and unrestrained cases.

The experiment results agree with the new language in the codes. The observed tension capacity of a standard all-bolted single angle connection with a single row of four bolts is significantly (~35%) higher than its shear capacity. On average, the tension capacity of the connection is found to be nearly 90 kips. The results show that the tension capacity of the single angle connection improves very slightly (~6%) when a lateral restraint is present. Two primary failure modes are identified: 1) shear fracture of the angle and 2) tensile failure of the bolts. For each failure mode, a model to evaluate the

tension capacity is proposed. The predicted capacity using these simple models are slightly conservative (~10%) with respect to the observed strength.

Acknowledgments

The authors would like to acknowledge the extensive contribution of Mark Wagner, supervisor at the Material Testing Laboratory of the George Washington University, toward planning and conducting the tests throughout this project. Thanks are also expressed to Cives Steel Company who donated the material required for the testing and Sean Boynton at Cives who helped with the material fabrication and delivery. This research has been sponsored by American Institute of Steel Construction (AISC). Thanks are expressed to Tom Schlafly and Kurt Gustafson for their insights to the problem that helped in initial design of the test-setup.

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This report has been prepared for The American Institute of Steel Construction (AISC) to present the results of tests performed at The George Washington University. The Department of Civil and Environmental Engineering was contracted by AISC to develop and execute a testing procedure to determine the axial capacity of an all bolted single angle shear connection.

Background

The National Institute of Standards and Technology (NIST) conducted a three year investigation in response to the collapse of the World Trade Center (WTC) Towers in New York City to study the factors contributing to and probable causes of post impact collapse of the WTC. NIST made a number of recommendations in its Final Report including a call to increase the structural integrity of buildings [NIST, 2008]. These recommendations were developed into proposed code changes by The National Council of Structural Engineers Associations (NCSEA). The International Building Code (IBC) and the Building Code of the City of New York are incorporating requirements for proven axial capacity of shear connections in an effort to increase resistance against unanticipated loads.

The language of these code changes indicate that the tension capacity should be determined under axial loading alone, not shear and axial loading combined. There have been many previously reported studies on the capacity of connections when shear and axial tension act simultaneously, but research on the capacity of shear connections under axial tensile load alone is very limited. The New York building code recommends that the axial capacity be equal or greater than the shear capacity of the connection, but not less than 10 kips. A study in the United Kingdom investigated the tension capacity of all bolted double angle connections, but the report indicated that the results would be expected to be very different for an asymmetric connection such as a single angle.

The all bolted single angle shear connection (Figure 1) is one of the most economical floor beam to girder connections currently used in the United States. It is the preferred shear connection for fabricators with operations favoring shop bolting over shop welding. Since there is currently very limited information concerning the tensile performance of single angle connections, the capacity for the resistance of unanticipated loads needs to be determined by testing.

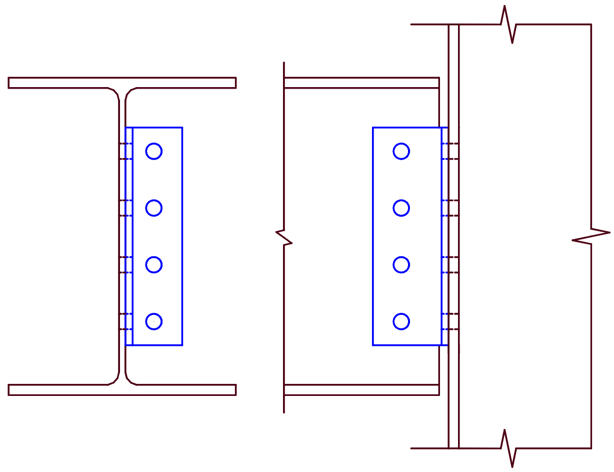


Figure 1: All-Bolted Single Angle Connection

Because a single angle shear connection in tension is an eccentric connection, the angle will begin to unfold as the tension becomes increasingly large. This manifests as a lateral translation of the floor beam so that it is no longer normal to the girder. In a typical floor system the beam and girder will act compositely with the slab or decking supported by the members, therefore resisting this lateral translation through the connection. Currently it is unknown how much effect this will have on the tension capacity of single angle shear connections. Thus, the difference between the laterally restrained case and laterally unrestrained case is considered in the experimental design.

Objectives

This project was designed to be the initial investigation of what may become a larger study concerning the axial tension capacity of selected common types of shear connections currently used in the United States. The objectives of this project are:

- To determine experimentally the axial capacity of typical all-bolted single angle connections in absence of shear.
- To conduct at least four tests assessing the tension capacity of two different cases of an all bolted single angle shear connection – at least two restrained against lateral movement and two unrestrained.

- To determine whether there are significant differences in performance between the restrained/unrestrained cases of an all bolted single angle shear connection.
- To identify the failure modes of an all-bolted single angle shear connection in axial tension.
- To propose simple analytical models to predict tension capacity of the connection.
- To gain information which can be used to develop additional tests to determine the capacity of different types of shear connections commonly used in the United States.

Connection Design

The connection type and overall dimensions were selected by AISC to best represent current United States practice. The connection specified is an L4x3x3/8 A36 angle with four rows of bolts. Each leg of the angle has one vertical row of bolts with four bolts in each vertical row, as illustrated in Figure 2. Bolts specified are ¾ in. A325 hex head snug tightened. The test is intended to model a standard simple beam to girder connection with the 4 in. leg connected to the beam web and the 3 in. leg connected to the girder web.

The remaining details of the connection are designed to meet the criteria for bolted connections in the AISC Manual. Standard holes with a nominal diameter of 13/16 in. are used. The bolt spacing is set at the preferred minimum of three times the nominal bolt diameter for a spacing of 3.0 in. Bolts are placed at the minimum edge distance from sheared edges, 1¼ in. All bolts on each side of the angle are oriented with the heads in the same direction and placed so the threaded end of the beam side of the connection faces the bolt heads on the girder side of the connection. Dimensions can be seen in Figure 2.

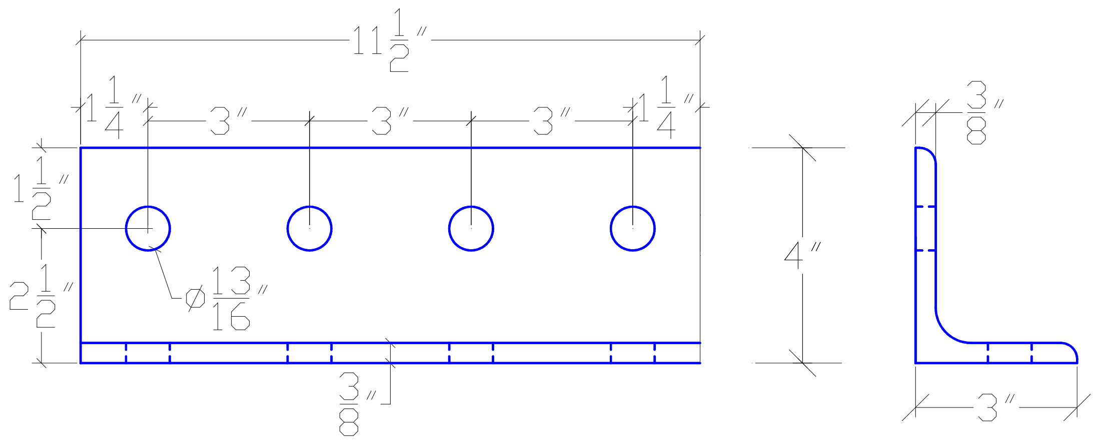


Figure 2: Single Angle Connection Details

Test Setup and Experiment Design

Based on previous research conducted on double angle connections, it was determined that an inverted tee arrangement would be the most convenient way to apply a tensile load to the connection [Jarret, 1990]. Such an arrangement could be conveniently setup on a Tinius Olsen (TO) machine (Figure 3).

Figure 3: Tinius Olsen Tension-Compression Machine

The initial setup designed in collaboration with AISC consisted of an HSS section to model the beam web and a W-section to model the web of the girder. This initial test setup failed to produce the desired failure mode. The criteria of selection of the HSS section, the details of the test setup, and the results of trial test run using this setup is described in Appendix A.

Based on the results of an initial test setup (Appendix A), the test setup was redesigned. The alternate setup to test the single-angle connection for the restrained case is shown in Figure 4. The setup uses a 10 in. long and 5/8 in. thick gusset plate (models web of supported beam) attached to the top beam with a bolted double angle connection and attached to the bottom beam with the single-angle connection. The plate thickness matches the thickness of the web of the HSS section (Appendix A). For the restrained case, a 1.75 x 12 x 6 in. block of steel is bolted at the bottom beam, close to the plate, to prevent the out-of-plane movement. Teflon plates are used to ensure low friction sliding. Since the length of the plate is significantly less than the length of the HSS used in the initial setup (10 in. compared to 24 in.), the out-of-plane displacements would be expected to be much lower. The alternate setup to test the single-angle connection for the unrestrained case is the same as that shown in Figure 4 with the block of steel on the bottom beam removed.

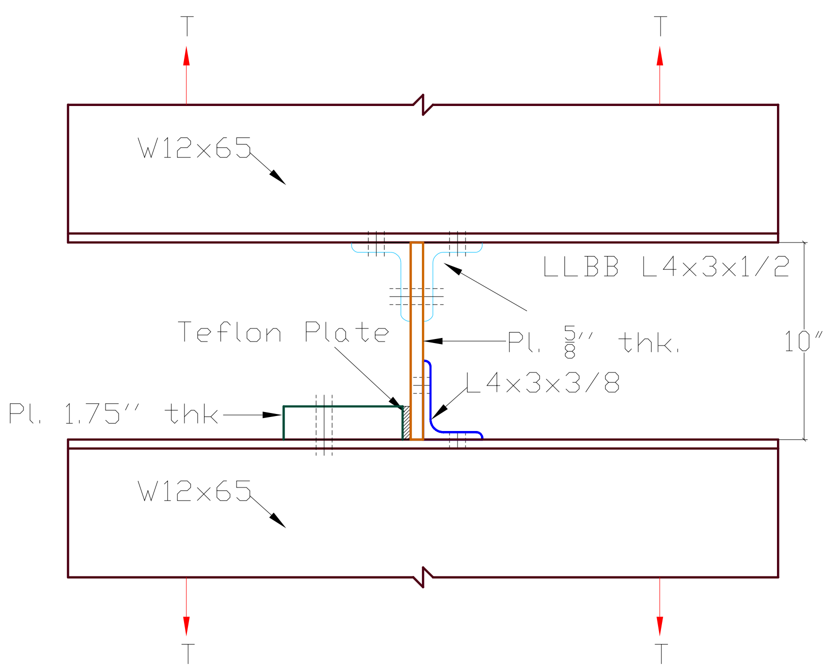


Figure 4: Test Setup-B with a Gusset Plate, Restrained Case

Figure 5 shows the test piece mounted on the TO machine for the unrestrained and the restrained case. The 1.75 in. thick steel block for the restrained case is held in place by using four C-clamps (Figure 5(b)). The trial run using these setups produced the desired failure mechanism, with the single angle as the weakest link.

(a) Unrestrained Case (UNR-3) (b) Restrained Case (RES-3)

Figure 5: Test Piece Mounted on the TO Machine

Test Results and Discussion

Table 1 summarizes the test results of the six specimens. The test specimens are labeled to distinguish the unrestrained (UNR) and the restrained (RES) case. Three specimens for each case were tested.

Two specimens (UNR-1 and RES-1) failed by complete fracture (Mode A) of the 4 in. angle leg attached to the gusset plate, as illustrated in Figure 6. The RES-2 specimen showed only a partial fracture (Mode B) of the 4 in. angle leg. In these three specimens, bearing failure at bolt hole edges of both the 3 in. and 4 in. legs, and yielding of bolts connecting the angle to the gusset plate can also be observed. In the specimen RES-1, bearing failure along the bolt hole edges of the gusset plate was observed. These failure states of the angle and gusset plate are illustrated in Figure 7 and Figure 8.

Table 1: Test Results of Single-Angle Shear Connections in Tension

Specimen No.	Failure Mode	Ultimate Load (Kips)	Disp. Vs. Load Data	Strain Gage Data
UNR-1	A	85.5	No	No
UNR-2	C	73.1	Yes	No
UNR-3	C	108.0	Yes	Yes
RES-1	A	93.2	Yes	No
RES-2	B	93.8	Yes	No
RES-3	C	95.3	Yes	Yes

Notes:- UNR – Unrestrained, RES – Restrained.

Failure Modes:- A – Full fracture of the 4 in. leg attached to the gusset.
B – Partial fracture of the 4 in. leg attached to the gusset.
C – Bolt failure.

The other three specimens (UNR-2, UNR-3, and RES-3) failed by tension failure of the bolts (Mode C) attaching the 3 in. leg to the top flange of the beam. This failure mode is shown in Figure 9. The double angle connection, attaching the gusset plate to the top beam, did not show signs of distress in any of the test runs.

The average tensile load capacity of the single angle connection in the unrestrained case was about 89 kips. The observed average load capacity of the restrained case was about 94 kips. As illustrated in Figure 6, both the unrestrained and restrained cases show similar lateral displacement. This out-of-plane displacement is significantly less than that observed in Setup-A using the HSS section (Appendix A). Figure 10 shows some of the factors driving the lateral movement: bowing-up of the beam flange, bending of the 3 in. leg, and flexibility of the 4 in. angle leg that appears to be rotating about the heel of the angle shape. Using the current setup, even with a firmer lateral support than the current steel block with C-clamps, the effects of these factors cannot be reduced. This may be one of the reasons for the observed similar distortion behavior and connection strength for the two cases.

For specimens UNR-2, UNR-3, RES-1, RES-2, and RES-3 displacement vs. load data was recorded. For the unrestrained cases the data is presented in Figure 11. As noted in the figure, the load at which the first bolt failed is taken as the capacity for UNR-2.

Beyond this point, the multiple bolt failures induce noise in the recorded data. The UNR-3 specimen also failed by bolt failure, but this did not affect the recorded data.

As shown in Figure 12, the load-displacement behavior for the restrained case is similar to that of the unrestrained case. Due to incorrect machine setting during the test of the RES-1 specimen, there was considerable noise in the recorded data near the beginning of the test, as seen in the figure. However, this does not affect the ultimate capacity of the connection. The noise induced due to multiple bolt failures in case of RES-3 specimen can be observed in Figure 12.

In order to gain more insight into the relative behavior of the unrestrained and restrained cases, two strain gages were mounted on the gusset plate. This data could also be used to calibrate future analytical models of the test setup. These gage locations are indicated in Figure 13. One strain gage was located on the front (Gage-1) above the single angle and the other one at the back (Gage-2) of the plate. The gages were installed on specimens UNR-2, UNR-3, and RES-3. Due to error in data acquisition settings, the data from the gages mounted on the specimen UNR-2 could not be recorded. The data from the other two specimens is summarized in Table 2. The strains noted in the table are the strains recorded at the failure point.

Table 2: Measured Strain Data

Specimen No.	Strain Gage Data	Gage – 1 (Peak Strain) X10^{-6}	Gage – 2 (Peak Strain) X10^{-6}
UNR-2	Yes	-damaged-	-damaged-
UNR-3	Yes	2145	-999
RES-3	Yes	1683	-729

Note:- These strains were recorded at the failure point for the specimens.

For the unrestrained and restrained cases, the strain vs. displacement data is presented in Figure 14 and Figure 15. The load vs. displacement curve for the two cases is also shown in the figures. As noted previously, the two cases have similar behavior. This assertion is corroborated by the measured strain data. The relative difference between the two gages for the two cases case is insignificant.

Simple Model to Calculate Tension Capacity

The test results indicate that the single angle all bolted shear connection fails mainly in two ways: 1) shear fracture of the angle and 2) tension failure of bolts. Separate models predicting the tension capacity can be developed to account for these failure mechanisms. The models presented in this report are based upon appropriate modifications to those presented in Publication P212 [SCI and BCSA, 2002] for double-angle all bolted connections. The models are calibrated to the experimental observations for the single-angle connections.

When a single-angle connection is subjected to high tension, it opens up and the typical deformed shape is shown in Figure 16. The displacement Δ_1 is observed to be in the range of 1 to 1.5 in. at failure. Since the connection is asymmetric, the angle leg connected to the vertical plate shows an out-of-plane movement. The horizontal displacement Δ_2 was not recorded in the tests. This out-of-plane displacement is observed even for the cases where a horizontal restraint is provided. There are four critical regions in the angle which are subjected to high plastic strains under the action of combined tension, shear, and moment. These are located at the bolt centerlines and near the rib of the angle. These regions are marked as A, B, B', and C in the figure.

Based upon these large displacement assumptions a stick model of the angle can be built as shown in Figure 17. Typically, at failure plastic hinges are formed at or near the critical sections and the two angle legs undergo a double curvature bending. The inflection points are located at distances l_1 and l_2 from the bolt centerlines as shown in the figure. The P212 publication [SCI and BCSA, 2002] uses similar stick models to develop rigorous and simple models to determine the tension capacity of a double-angle connection.

Angle Shear Fracture

The rigorous method (Appendix B P212) to account for the limit state of shear fracture is based upon combined action of tension, shear, and moment with the failure criteria based upon the Von-Mises failure theory. The solution of the associated interaction equation to derive the tension capacity is solved numerically and the solution is dependent upon the observed displacements. Since the displacement data is not recorded during the experiments, a complete solution of using the rigorous method is not feasible.

A simple model can be adopted to predict the shear fracture capacity. This model is based upon the shear capacity of the net effective area at the critical section. The tension capacity (T) based upon shear fracture is:

$$T = 0.6\ F_y\ L_e\ t \qquad \text{(Eq. 1)}$$

where,

F_y – yield capacity

L_e – net effective connection length (Figure 18)

$L_e = 2e + (n-1)p - nD_h$

e – end distance

p – pitch

D_h – hole diameter

n – number of bolts

t – thickness of the angle

Figure 18illustrates the evaluation of the net effective length of the connection to evaluate the shear fracture strength. For the test connection, e = 1.25 in., p = 3 in., D_h = 13/16 in., and n = 4. Thus, L_e = 8.25 in. The angle thickness is 3/8 in. The nominal yield strength is 36-ksi. However, it is reasonable to assume that the actual yield strength of the angle could be anywhere in between 36-ksi to 50-ksi, so for further evaluation an average yield strength of 43-ksi is assumed. Therefore, using Eq. 1 the tension capacity (T) of a single-angle connection based upon the shear fracture limit state equals 80 kips.

Three specimens (UNR-1, RES-1, RES-2) failed through this mechanism. The average strength observed equals 91 kips (Table 1). Thus the simple analytical model (Eq. 1) provides a conservative estimate of the tension capacity. The predicted capacity is about 12% lower than the observed strength. An appropriate strength reduction factor (Φ) could be applied to Eq. 1 for design purposes.

Bolt Tensile Failure

Failure was also observed by tension failure of the bolts along the critical section ‘C’ (Figure 17). Due to prying action on these bolts the effective tensile capacity of the bolts is reduced. The critical angle leg is assumed to undergo double curvature bending with an inflection point located at a distance of l_2 from section ‘C’. Figure 19 shows the stick model with the critical forces associated with this prying failure mechanism. The prying force is assumed to act a distance of 2*t* from the bolt centerline, where *t* is the angle thickness. The farther away the point of inflection is from C, the greater will be the prying action. A prying ratio bolt force (B) over shear force (V) can be defined to characterize the effect of prying action on the bolt tensile strength. This ratio can be evaluated using moment equilibrium about section ‘E’ for a free-body D-C-E (Figure 19). The section ‘D’ is through the point of inflection and the moment at this section is zero. The prying ratio B/V is thus given by

$$B/V = (2t+l_2)/2t \qquad \text{(Eq. 2)}$$

The upper bound for the distance l_2 is distance between the bolt centerline at C and the plastic hinge at B′. It is reasonable to assume that the plastic hinge at sections near the heal of the angle is formed at a distance of [(t + r)+ t] from the heel, where r is the angle’s rolled radius. The distance (t+r) is approximately noted in the AISC manual for all standard angles as k and x_p. For the test angle 4x3x3/8 the distance (t+r) equals 0.311 in. Thus the plastic hinge is located at a distance of approximately 0.686 in. from the heel. The gage for a 3 in. leg is 1.75, thus the distance of the section B′ from the bolt centerline at C is equal to approximately1.064 in. Thus, a worst case upper bound of distance l_2 gives a prying ratio for the test angle of 2.42. The worst tensile strength reduction factor would therefore be 0.41 (=1/2.42). In the publication P212 Appendix D [SCI and BCSA, 2002], the point of inflection is assumed to be located at approximately half the distance between the bolt centerline and the plastic hinge near the heel. Using this assumption, the prying ratio for the test angle is equal to 1.71. This prying ratio as applied by the publication P212 [SCI and BCSA, 2002] to the double-angle connection is 2.13.

Assuming the worst case, the tension capacity (T) based upon the bolt tensile failure is given by

$$T = 0.4\, n\, F_t\, A_b \qquad \text{(Eq.3)}$$

where,

F_t – bolt tension capacity

A_b – Bolt tension area

n – number of bolts

For ¾ in. A325 bolts, F_t is 90 ksi and A_b is 0.442 in.^{2}. Therefore, assuming the worst case location of the inflection point, the tension capacity of the single angle connection is equal to about 64 kips. Assuming that the inflection point is located at half the distance between the bolt centerline and plastic hinge location, the reduction factor is about 0.6, which provides the tension capacity of about 96 kips.

Three specimens (UNR-2, UNR-3, RES-3) failed by bolt tensile failure. The average observed capacity equals 92 kips. The lower bound of the predicted tension capacity (64 kips) is about 30% lower than the observed capacity. This was judged to be an overly conservative approach to estimating the lower bound tension capacity.

Using a more realistic assumption for the location of the inflection point as midway between the bolt centerline and the plastic hinge near the heel, the predicted capacity closely matches the observed capacity, and is thus not conservative. This assumption for the location of the inflection point is the same as assumed in P212.

In order to obtain a somewhat conservative estimate, a distance ratio of 0.7 was chosen. This is more conservative than the midway assumption but is not overly conservative as with the worst case. Using this ratio, for the test angle the prying ratio is about 2 and the reduction factor is 0.5. Thus Eq. 3 can be revised to account for a more reasonable location of the inflection point. The tension capacity of the single-angle connection assuming tensile bolt failure is given by

$$T = 0.5\, n\, F_t\, A_b \qquad \text{(Eq. 4)}$$

Using Eq. 4 the predicted tension capacity is equal to about 80 kips. This predicted capacity is about 13% lower than the observed capacity. An appropriate strength reduction factor (Φ) could be applied to Eq. 4 for design purposes.

Comparison of Shear and Tensile Capacity

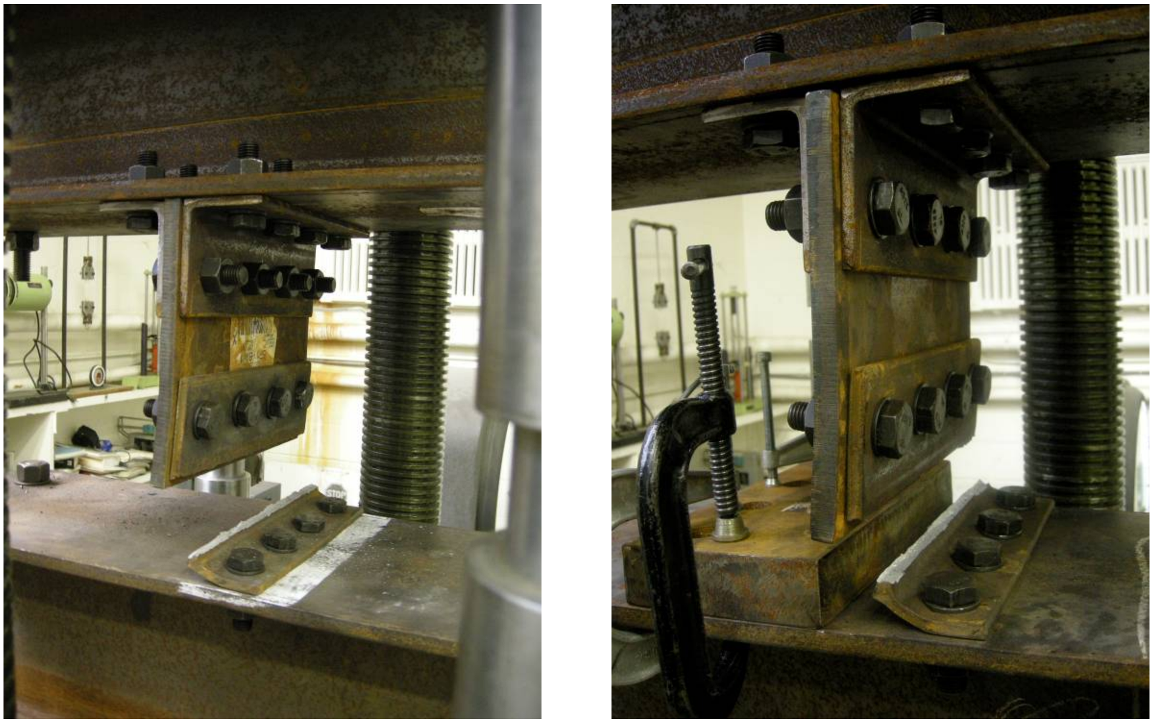
The shear capacity of an all bolted single-angle connection can be obtained from Table 10-10 of the AISC manual. For the test angle, using Case-I and four bolt rows, the capacity coefficient 'C' is 3.07. For ¾ in. A325 bolts, the unfactored shear strength per bolt (r_n) is 21.2 kip/bolt. This provides a shear capacity of the connection as 65 kips.

The average observed tension capacity is about 92 kips. From the shear fracture model (Eq. 1) the predicted tension capacity is 91 kips. Using the bolt tensile failure model (Eq. 5) the predicted capacity is 80 kips. These are significantly higher than the shear capacity of the test connection. On average, the tensile capacity is about 35% higher than the shear capacity.

Conclusions

The experiments demonstrate that the axial capacity of the test connection is significantly (~35%) higher than its shear capacity, which meets the requirement of the new language in the New York building code. On average, the tension capacity of the connection is found to be nearly 90 kips. The results show that the tension capacity of the single angle connection improves only slightly (~6%) when a lateral restraint is present.

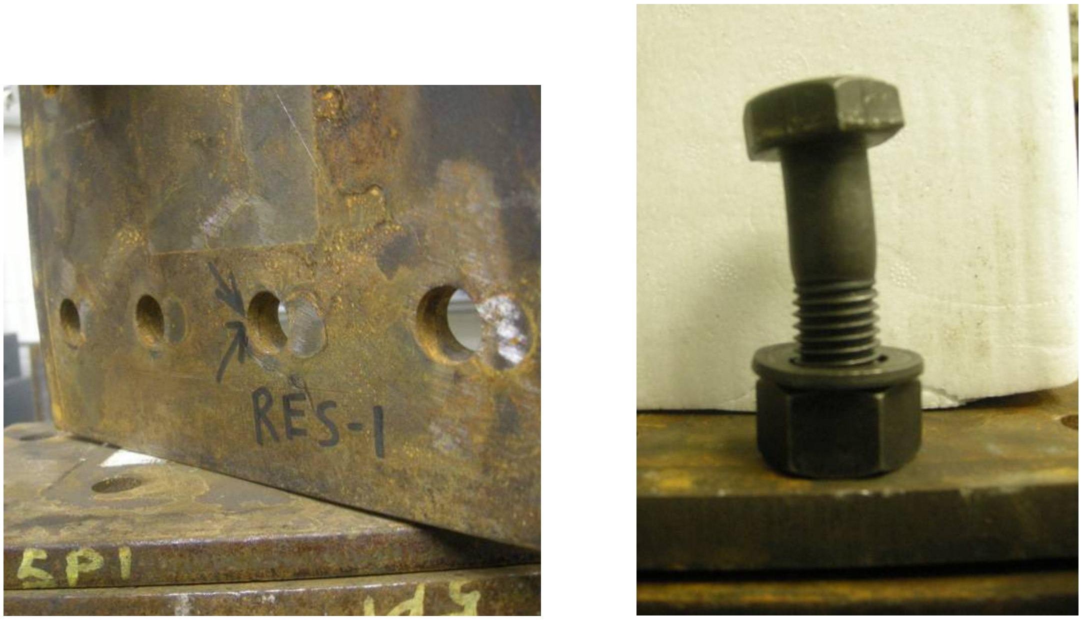
Two primary failure modes are identified: 1) shear fracture of the angle and 2) tensile failure of bolts. For each failure mode, a simple model to evaluate tension capacity is proposed. For shear fracture, the tension capacity governed by the combined action of shear, tension, and moment is given by $0.6\ F_y\ L_e\ t$. For the bolt tension failure, the tension capacity based upon prying action is given by $0.5\ n\ F_t\ A_b$. The predicted tension capacity for both the limit states are 12-13% lower than the observed capacity.



(a) Unrestrained Case, UNR-1 (b) Restrained Case, RES-1

Figure 6: Fracture Failure of Single Angle Connection

Figure 7: Typical Failure States of the Single Angle



(a) Bearing Failure in Some Gusset Plates (b) Yielding of Bolts

Figure 8: Typical Failure States of the Gusset Plate and Bolt

(a) Unrestrained Case, UNR-3 (b) Restrained Case, RES-3

Figure 9: Bolt Failure in the Single Angle Connection

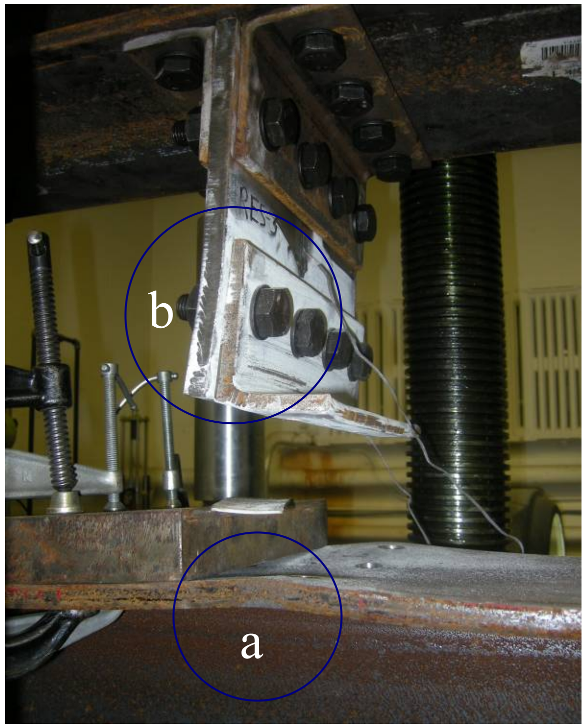


Figure 10: Some Causes of Out-of-Plane Movement of the Gusset Plate: (a) Bowing-up of Flange (b) Flexibility of the 4 in. leg

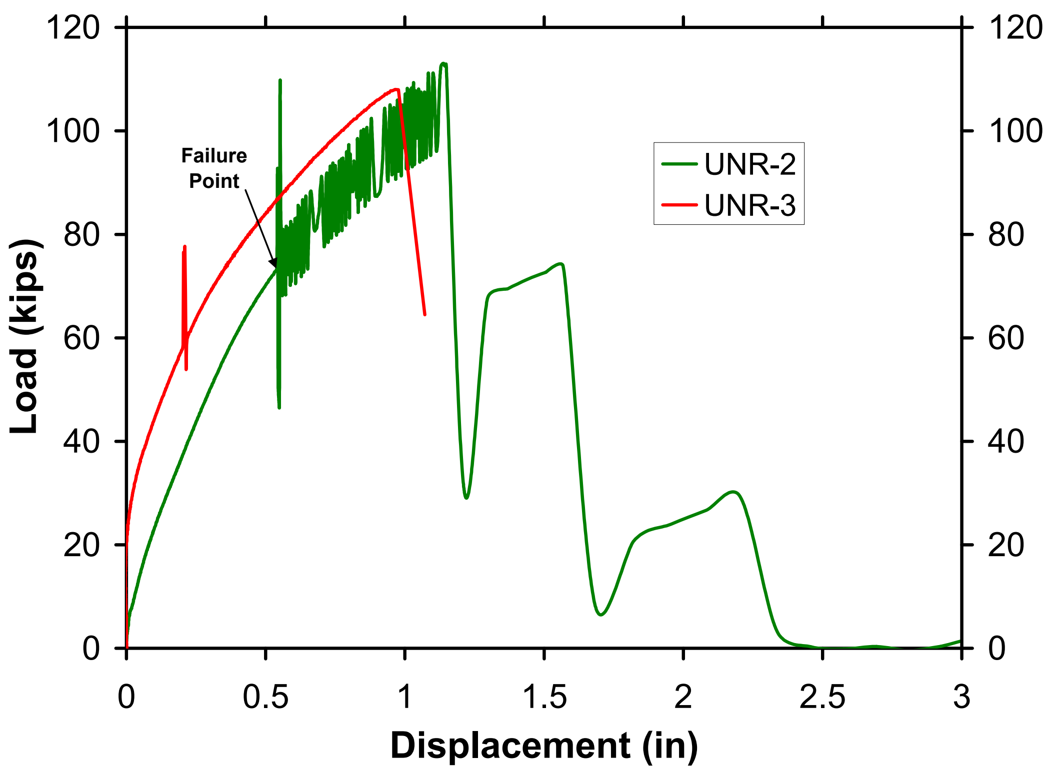


Figure 11: Load Vs. Displacement Curves for Unrestrained Case

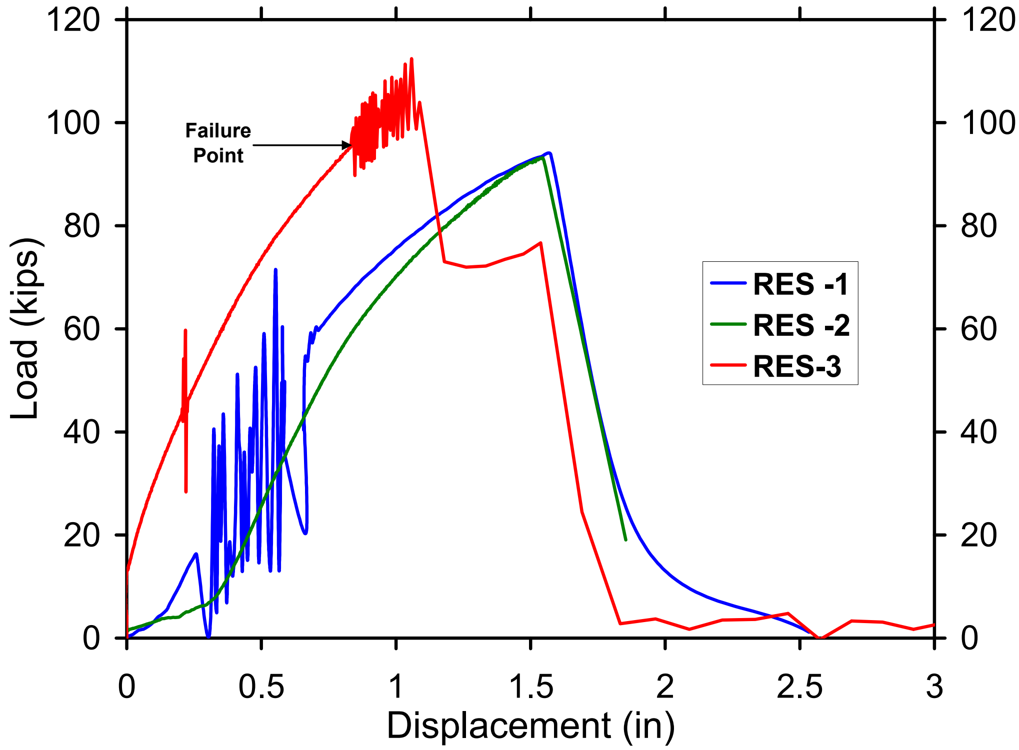


Figure 12: Load Vs. Displacement Curves for Restrained Case

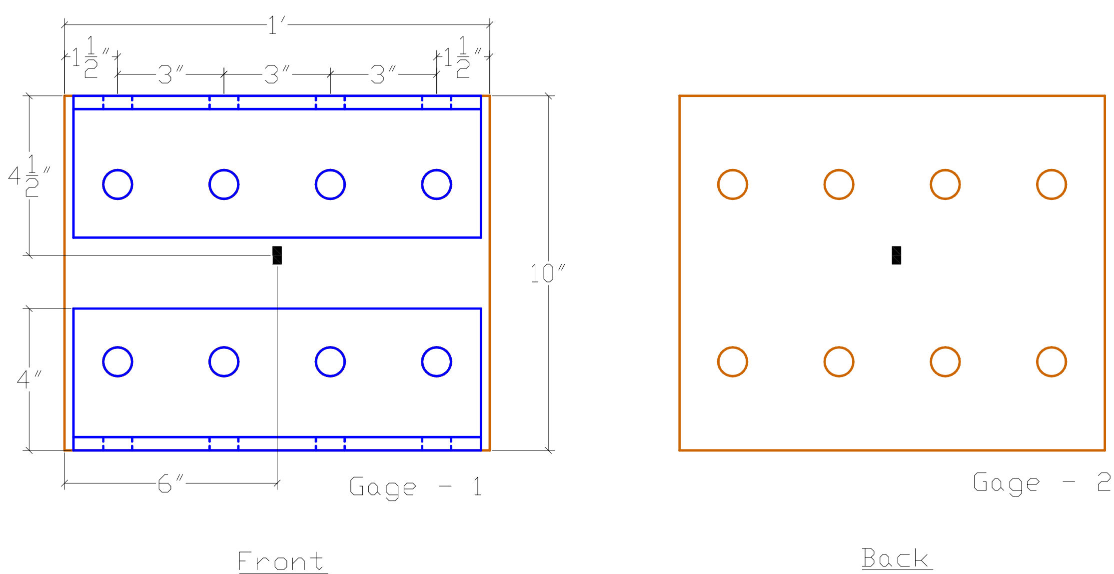


Figure 13: Strain Gage Locations

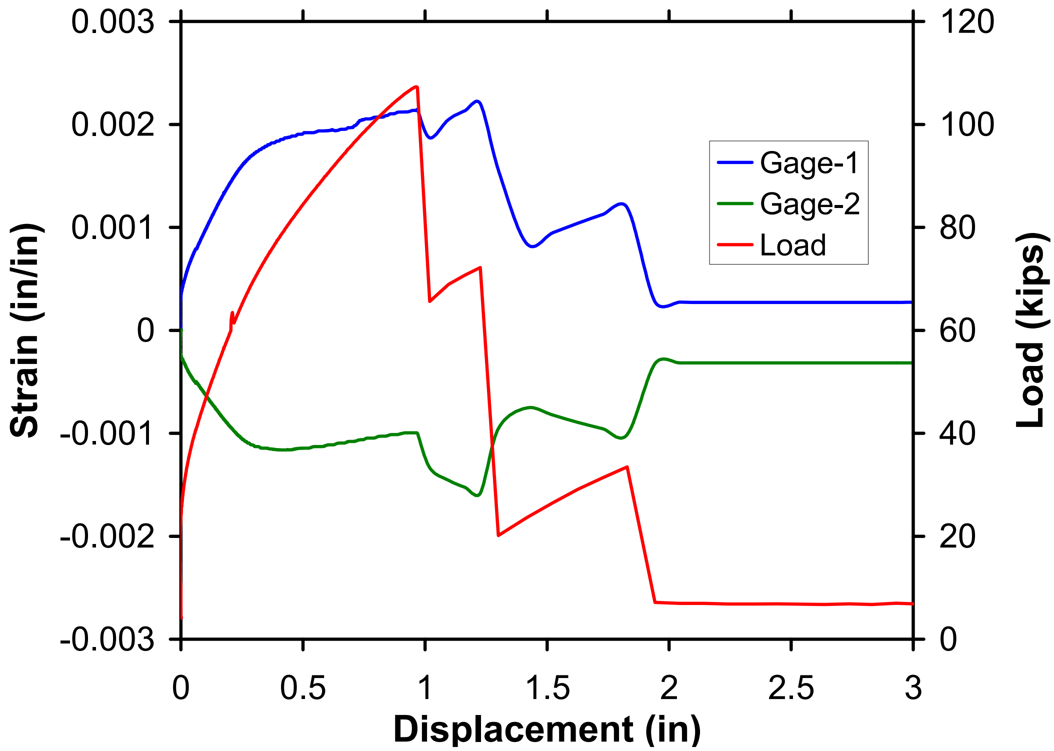


Figure 14: Measured Strain for the Unrestrained Case (UNR-3)

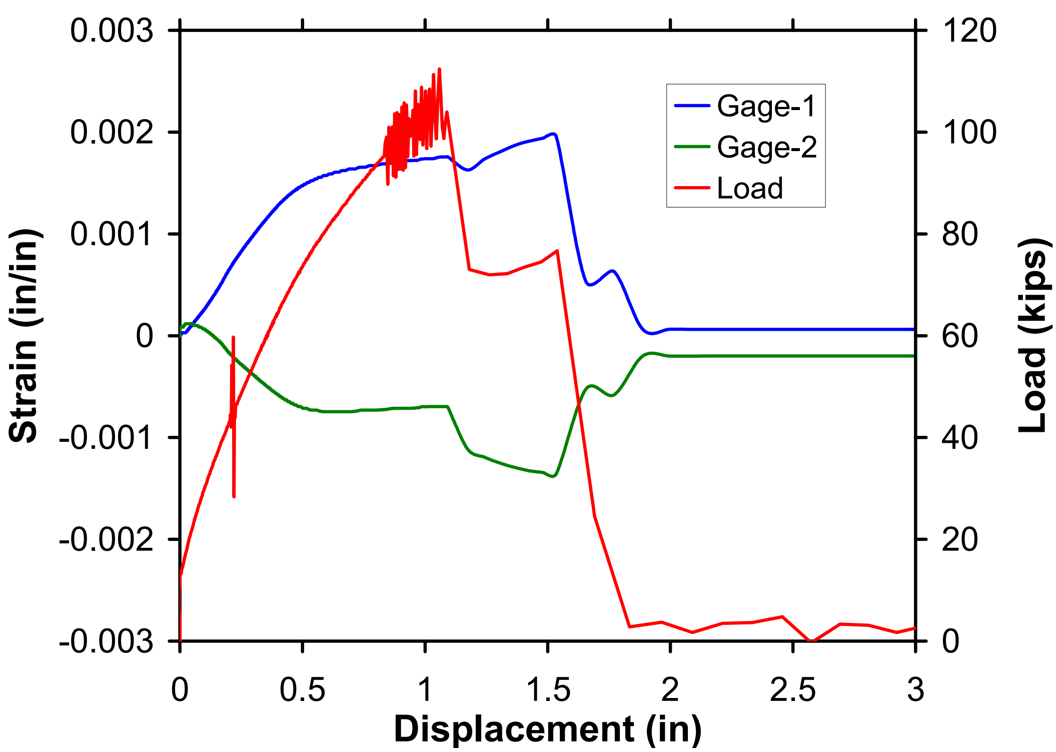


Figure 15: Measured Strain for the Restrained Case (RES-3)

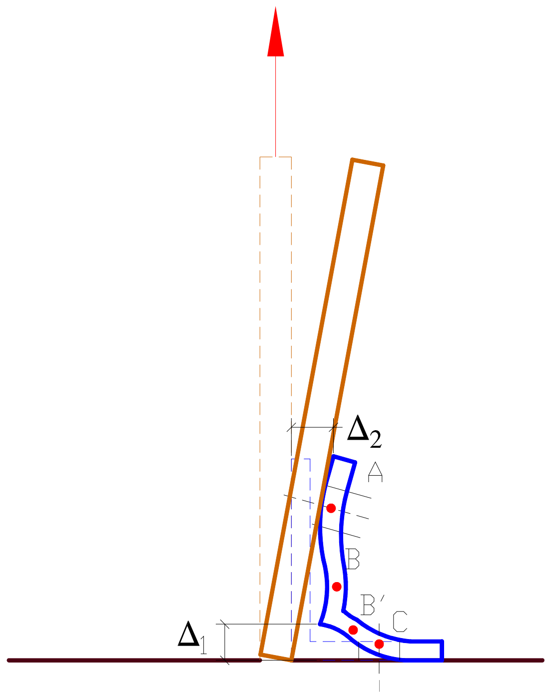


Figure 16: Deformed Shape of a Single Angle Connection Subjected to High Tension

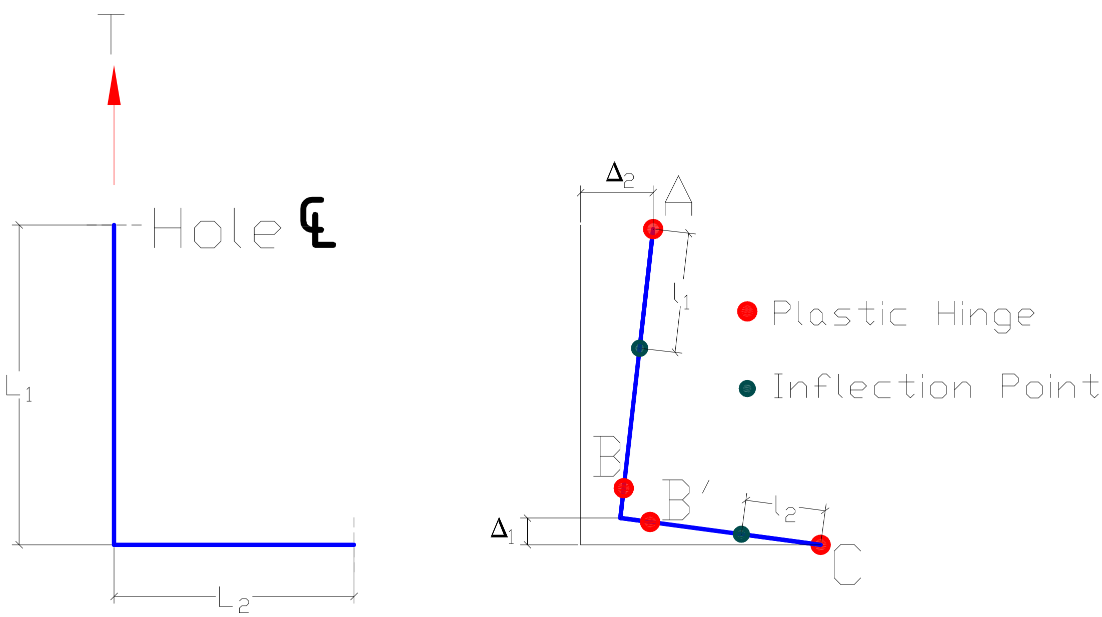


Figure 17: Stick Model for Single Angle Connection Under Tension

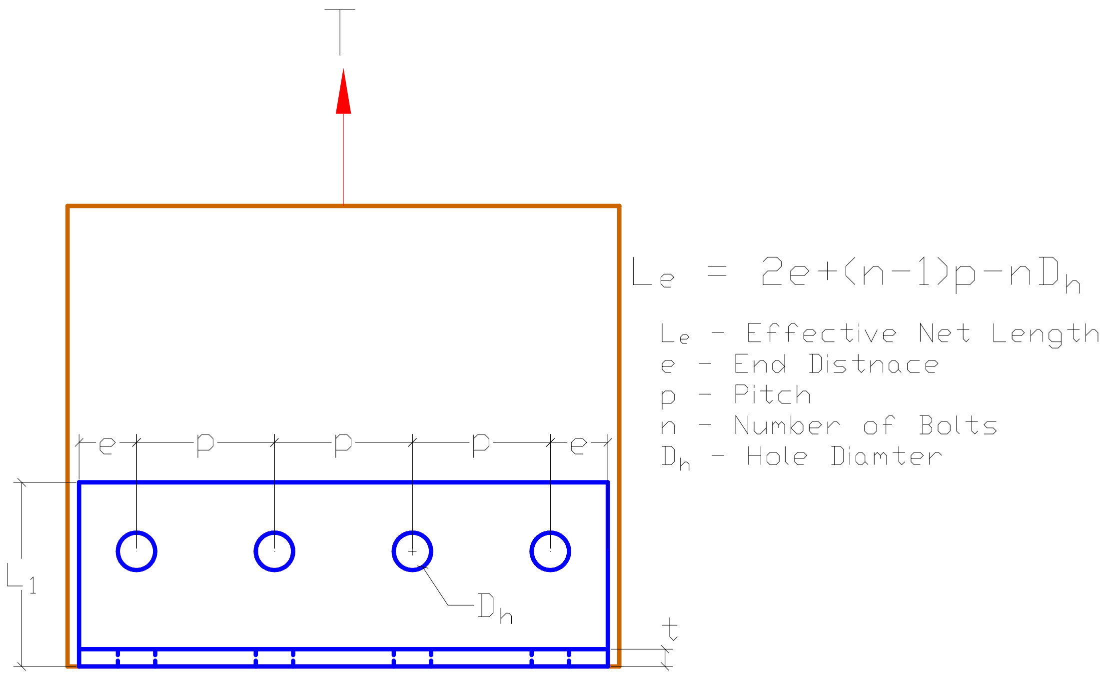


Figure 18: Effective Net Connection Length to Evaluate Shear Fracture Strength

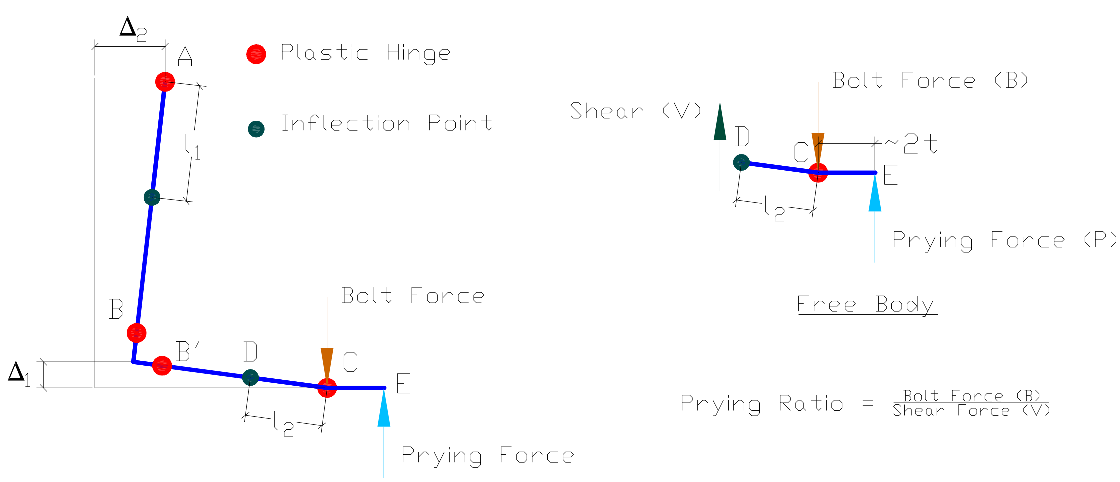


Figure 19: Approximate Evaluation of Prying Ratio

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Appendix A

The initial test setup designed in collaboration with AISC consisted of an HSS section to model the beam web and a W-section to model the girder web. The HSS section was selected because the side of the HSS section across from the single angle can be used to apply a lateral restrained to prevent any out-of-plane movement. This arrangement can be seen in Figure 20.

The selection of the HSS section was determined by a number of factors. The outer and inner dimensions of the HSS section were required to be great enough to allow for all four of the bolts to be connected with enough edge distance, as well as enough room to be able to conveniently connect the members. The thickness of the HSS section was chosen by determining what thickness would provide sufficient yield strength so that the HSS section would remain elastic up until the failure of the angle. The ultimate strength of the angle was found to be as high as 180 kips (the variability is because the Manual lists a range of F_u for A36 steel). Based on these criteria, an HSS14x14x5/8 was chosen as the minimum square HSS section that would meet the strength and geometric requirements.

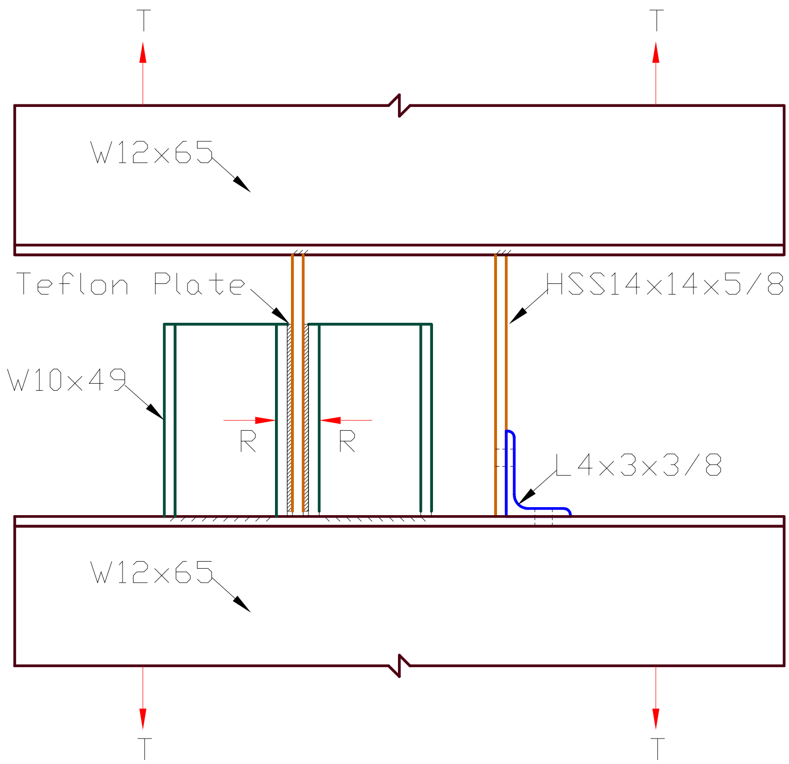

Figure 20: Test Setup-A with an HSS section, Restrained Case

The selection of the W shape was governed by criteria similar to those of the HSS section. The width of the flange had to be sufficient for all of the bolts to be connected while meeting the edge distance requirements. As with the HSS section, a minimum flange width of 10 in. was chosen. The second criterion was that the section should be stiff enough so as not to excessively deflect, as well as capable of resisting the bending moment created by the angle pulling at the center of the beam. Since the cross head of the TO machine is 2 ft. wide, it was decided that the beam would be approximately 3 ft. long with a 6 in. overhang on each edge. For a tension force of approximately 200 kip, the maximum moment in the beam would be 150 kip. All W sections with a flange of at least 10 in. satisfied the moment criteria. A W12x65 section met these requirements and was selected for the test.

In the trial run using this setup, the two transverse welds attaching the HSS to the top beam failed when the load was increased to about 20 kips. The connection of the HSS to the top beam was then reinforced using two angles welded to the HSS and the beam flange. The test was then resumed. As the load was increased to beyond 60 kips, both the angles used to reinforce the top connection started to yield (Figure 21(a)). One tore apart at a load of approximately 70 kips, as shown in Figure 21(b). At this stage there was no sign of distress in the single angle connection.

(a) Load ~ 60 kips (b) Load ~ 70 kips

Figure 21: Failure Modes for Test Setup-A, Unrestrained Case

These results clearly show the HSS-to-beam weld as the weakest link, even after reinforcement. Additionally, the test showed that the out-of-plane displacement is significantly more than expected and the proposed test-setup for the restrained case is unlikely to be adequate.