The Design and Behavior of Bolted Beam-to-Column Frame Connections Under Cyclical Loading

by

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Thesis

Presented to the Faculty of the Graduate School
of The University of Texas at Austin
in Partial Fulfillment
of the Requirements
for the Degree of

Master of Science in Engineering

The University of Texas at Austin
May 1996

The Design and Behavior of Bolted Beam-to-Column

Frame Connections Under Cyclical Loading

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Acknowledgments

The author wishes to express gratitude to his graduate advisor, Dr. Joseph Yura, for his willingness to devote countless hours in teaching and sharing his experience in bolted steel connections and steel behavior. The ideas and experience that Dr. Yura brought to the design, testing, and analysis of the bolted connections created ideal learning conditions. The support and encouragement of Dr. Yura was greatly appreciated.

Thanks goes to Dr. Michael Englehardt for reviewing this thesis and for bringing greater insight on connection behavior for seismic application. Dr. Englehardt also provided great assistance in furnishing the test set-up, the beams, and the columns for the tests in this thesis. Thanks also goes to Trade Arbed Steel Corporation for providing the steel for the tees, Falcon Steel for fabricating the specimens, Nucor-Yamato Steel Corporation for providing the bolts, nuts, and washers and AISC for providing finical support.

This study was conducted at Fergusson Structural Engineering Laboratory at the University of Texas at Austin and was made possible by the support provided by the lab's staff.

Abstract

In the 1994 Northridge earthquake many steel buildings suffered damage to the full penetration groove weld of the moment resisting beam-to-column connections. In response to these failures new welding procedures, and reinforced connections have been designed, tested and implemented. The result has been an increase in the cost of the all welded connections. All bolted beam-to-column connections are not new. Historically the all bolted connection has been more expensive to build, however preliminary estimates show that with the increased cost of the welded connection, this may no longer be true. The greater expense of welded connections have prompted the investigation of all bolted connections.

The bolted connection consists of a tee stub with the flange bolted to the column flange and the stem bolted to the beam flange. The web of the beam is then connected by using a bolted shear tab. Tee stubs have been tested under monotonic loads and bolts have been studied for prying forces, but limited work has been performed in testing hangers under cyclical load and even less in testing full scale beam-to-column connections using the tee stub hanger under cyclical loads.

Tests were performed on five full scale bolted moment-resisting beam-to-column assemblies at the University of Texas at Austin. These connections were evaluated for there behavior, rotation capacities, moment capacities and modes of failure. Stiffness, ductility, loss of connection strength, and energy dissipating ability have been used in creating guidelines and recommendations for the design of bolted moment-resisting connections in seismic regions. A design example is given to give guidance in using the recommendations and to show some of the limitations of bolted connections.

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Chapter 1 Introduction

For many years welded flange and bolted web, or all welded, beam-to-column connections have been the choice of many designers in moment resisting steel frames. The reason has been the ease of design and the economics of construction. In areas of high seismic risk these connections have been used extensively because of the belief that their response was very ductile when loaded beyond the elastic limit. Research conducted by E. P. Popov and R. B. Pinkney, [19] E. P. Popov and R. M. Stephen, [18] and E. P. Popov and V. V. Bertero[17] at the University of California at Berkeley in the late sixties and early seventies generally supported the notion of ductile behavior of such connections. During the nineteen eighties and early nineties new test data became available that cast doubts on these ideas. [6,21] The tests showed that welded connections typically fractured through the full penetration groove welds joining the flange of the beam to the column. This connection failure is not only a brittle mode, but it occurs at unpredictably low levels of plastic beam rotation. However, since no significant damage of this type had been reported after major earthquakes, response to these findings was slow. The 1994 Northridge Earthquake showed that this phenomena was not unique to the laboratory. [4,5]

In response to these failures a test program was conducted at the University of Texas to determine the cause of the failures and to learn how to avoid future brittle failures in welded connections. [4] The connections that were found to perform well in this test series had a higher cost of construction than the traditional welded design. The higher cost has created interest in some older types of connections that have not been used primarily for economic reasons. Bolted connections of the type shown in Figures 1.1 and 1.2 could be used instead of

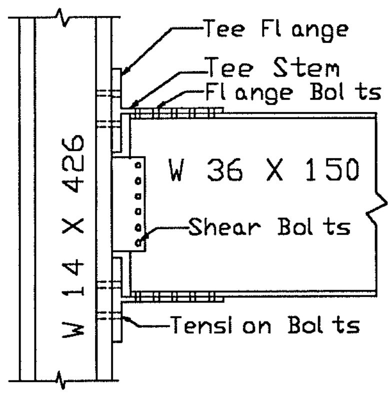

Figure 1.1 Example of Connection Used in Test Program

welded connections. This connection consists of a tee-stub with its flanges bolted to the flange of the column and the web or stem bolted to the flange of the beam. The web of the beam is connected by use of a shear tab that is bolted to the beam and welded to the column.

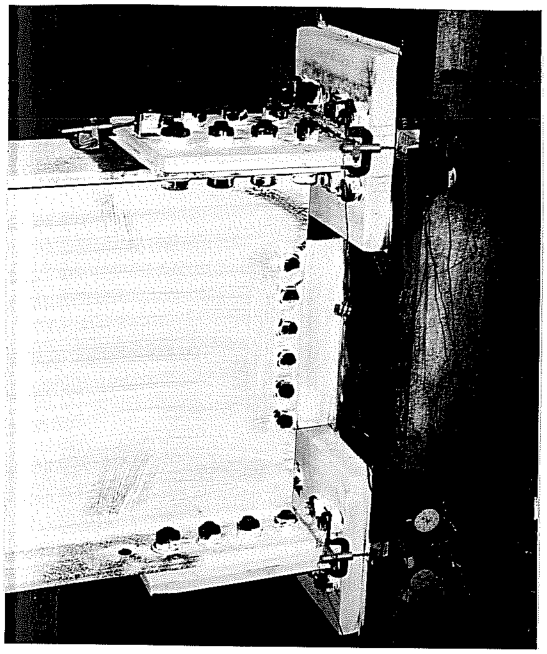

Figure 1.2 Connection Used in Test Program

A literature search showed that there is very limited research available on the design and behavior of the complete connection assembly under cyclical loads. A few papers deal with just the hanger (tee-stub) connection under monotonic loading, [3,13] and many papers consider the prying forces on the tension bolts. [3,7,13,15,20] Research has also been conducted on end plate [2,7,13,14,15,16] and top and bottom flange plate connections [2,16] under cyclical loads. End plate connection has a plate fully welded to the end of the beam as if the plate was a column flange. The welding is done in a shop and is supposedly of higher quality than field welds. The end plate of the beam is then bolted to the column flange in the field with tension bolts. The top and bottom flange plate connection has plates welded to the column flange with full penetration grove welds in the shop and then the plates are bolted to the beam flanges in the field.

Five full scale tests were conducted at the University of Texas to study the performance of bolted connections in a moment resisting frame. This thesis gives the results of the tests conducted on bolted connections. This thesis will focus on the experimental behavior of the different connection elements. The response of the beam flange bolts in shear including the effects of bearing deformation and friction resistance were studied. In addition, response of the tee stub flange in bending, the response of the tension bolts during cycling in tight and snug conditions and the effects of prying forces on the bolts and the connection were examined. Details of the test program and test results are presented in chapter three and four but first, the current design procedures for such a connection are reviewed in the next chapter. The test data is used to evaluate the current design procedures and suggestions are made on future design of these connections in chapter five. A summary is provided in chapter six.

Chapter 2 Review of Current Design Procedures

A moment frame in a seismic zone of high risk can be designed as either a Special Moment Frame (SMF) or as an Ordinary Moment Frame (OMF). [1,12] SMF's have greater detailing requirements, but they may be designed for lower seismic loads due to the increased available ductility. In defining SMF's AISC states in Section 7.1 of the Seismic Provisions Commentary, "Non-ductile behavior is inhibited so that nonlinear response to large earthquake motions can occur in components of the frame having capability of ductile behavior." [1] This statement indicates that the joint region needs to be ductile. AISC then states, "Flexural hinges can form in the beams and column and shear yielding can occur in the area of the panel zone," and Section 7.2 of the AISC Seismic Provisions Commentary states, "The special limitations provided for these joints (SMF's) are intended to assure that inelastic hinging... will not take place in the joinery." [1] The AISC Seismic Provisions and the Uniform Building Code (UBC) give two classes of connections, those with welded flanges and alternate connections. [1,12] Guidance given to alternate connections is that, by calculation, they must be designed for 125 percent of the capacity of the connecting elements and remain elastic. [1,12] These provisions exclude any idea that the joint may be ductile enough to withstand seismic loading while allowing the connected elements to remain elastic. The idea that the connecting elements must remain elastic is as sound as the ability of the engineer to reliably predict the strength and ductility of a connection. In the case where it can be shown that the joinery is both strong and ductile enough, the economy of limiting damage in the beams and columns may be a more sensible solution. Even though the code excludes the use of most bolted connections as SMF's, they still may be designed as alternate connections forcing the inelastic action into the beam or column. Otherwise they must be designed as OMF's.

Section 7.2.d of the AISC Seismic Provisions and Section 2212.5.1.3 of the UBC state that alternate connections, bolted connections, designed by calculation must be designed for 125 percent of the design strengths of the connected elements. [1,12] Typically weak beam systems are designed, which means the design is for 125 percent of the plastic moment (M_p) of the beam. AISC does not prohibit the use of the common assumption that all of the beam moment is carried by the beam flanges. This means for the tee connection the pulling force (T) it is required to carry is $1.25M_p$ divided by the depth of the beam (d). This increased pulling force should be used checking the various limit states of the connection elements such as:

1. Shear capacity of the bolts connecting the flange of the beam to the stem of the tee.
2. Yielding of the gross section of the tee stem.
3. Fracture of the net section of the tee stem.
4. Buckling of the tee stem.
5. Block shear of the tee stem and the flange of the beam.
6. Bolt tear out of the shear bolts through the tee stem and flange of the beam. This is controlled by proper bolt spacing and end distance in the AISC code. [1]
7. Bolt bearing stresses in the tee stem and beam flange which must be limited by 2.4 times the ultimate tensile stress (F_u) of the material.

The shear connection between the tee stem and the beam flange is not specifically required to be a slip-critical connection. Section 5.a of the AISC Specification for Structural Joints states , slip-critical joints should be used where "joints (are) subjected to significant load reversal,... (and) joints in which, in the judgment of the engineer, any slip would be critical to the performance of the joint or the structure and so designated on the contract plans and specifications." [1] Astaneh suggests that slip should not occur prior to 1.25 times the unfactored service load but, less than 80% of the plastic moment of the beam. [2] This is to allow the structure to remain stiff under service and wind loads while dissipating energy by slipping during earthquakes.

The tee flange and the tension bolts may be sized by procedures in the AISC manual or by an alternate procedure given by Astaneh. [3] The AISC LRFD manual gives a six step procedure for calculating the required flange thickness of the tee stub [1]:

1. Determine the size and number of bolts required such that B (design tensile strength of the bolts) $\geq T = \frac{1.25Mp}{d}$
2. Estimate required flange thickness and choose a trial section and design quantities, such as hole locations.
3. Calculate β (a design coefficient used to calculate flange thickness, calculated from pulling force, bolt capacity, section properties of tee flange, and bolt location.)
4. Calculate $t_{req'd}$, (required thickness of the flange of the tee, calculated from, pulling force, material properties, section properties, and bolt location); if sufficient continue otherwise try a new section.
5. Calculate the Prying force (Q), (calculated from section properties, bolt capacity, and bolt location). $B \geq T + Q$ If the bolt capacity is not sufficient either the bolt strength or the flange thickness must be increased.
6. If prying forces are to be avoided, another equation for $t_{req'd}$ is given.

Astaneh gives a simplified approach which calculates a T_0, for a given tee stub configuration and bolt capacity. [3] Astaneh gives three conditions that must be checked to find which controls the design. The first condition is the tee flange failure mode, characterized by a plastic moment forming through the gross section at the web and a plastic moment forming through the net section at the tension bolt line. The second condition is tension bolt failure with prying forces, where the bolts are at their ultimate condition and a plastic moment forms through the gross section through at the web. The third condition is bolt failure, where no prying forces are acting at the time of bolt failure and no hinges have formed in the tee flange [3]. The equations may be derived from statics by drawing the free body diagram and moment diagram, as

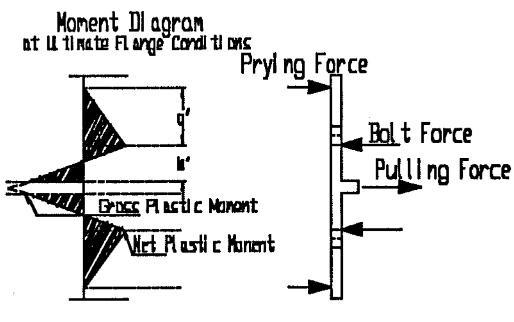

Figure 2. 1 Moment Diagram of Tee

shown in Figure 2.1, and checking the three conditions. The equations may be found in Reference 3. Astaneh's equations are written for allowable stress design (ASD) and assumes a factor of safety of 2. The LRFD manual approach includes a reduction factor, ϕ, in the constant of the design formula for $t_{req'd}$. When the factor of safety and reduction factor are removed from the above equations they give answers which are comparable.

The design procedure for the tee stub flange and tension bolts is theoretically a limit state analysis. By using the plastic moment and using the ultimate tensile capacity of the bolts the simple plastic theory it is implied that the maximum load capacity is calculated. The theory assumes that bending theory is applicable at failure of the hanger and that the bolts are capable of achieving and sustaining their ultimate load without changing the geometry of the connection. In reality the bolts yield and the prying force is relieved. The prying force's point of application may move further from the bolt line increasing distance a′ in Figure 2.1 and sustaining a constant moment at the bolt line in the tee flange. Another possibility is the moment at the bolt line may be reduced by redistributing the moment in the tee flange from the bolt line to the tee stem. Both scenarios allow the prying force to be relieved. If the bolt force remains constant and the prying force is relieved the pulling force may increase. The amount of each that happens is dependent upon the original geometry of the connection.

Fracture on the net section of the beam is handled in the UBC in section 2212.5.1.4 by the use of a simple ratio of the net and gross areas of the beam flanges,[12] $\frac{A_e}{A_g} \geq \frac{1.25F_y}{F_u}$. If this equation is satisfied the net beam section has been checked for fracture. The design of the web connection is not clearly defined in the code. The web connection must be able to carry the shear from the load combination 1.2D + (0.5L or 0.2S) plus the shear resulting from the ultimate moment of the connection. [1] The moment capacity of the web is not addressed in the code for alternate connections, except that the overall connection must be 125 percent stronger than the connected elements. The only guidance is for connections with welded flanges. If the flanges carry less than 70 percent of the beams moment, the web connection must carry at least 20 percent of the moment of the beam web.[1,12] Which is approximately 6 percent of the total moment depending on the beam section.

Chapter 3 Test Program

The objective of this test program was to predict the failure of several bolted connections, find if under cyclical loads these failure modes governed, find the level of ultimate plastic rotations bolted connections could achieve and to compare the plastic rotation to the 0.03 radians now being called for by the profession. It was a desire to ultimately find a connection that was feasible and able to achieve 0.03 radians of rotation, without brittle failure.

The connection assembly utilized a W36 X 150 beam, and a W14 X 426 column for all of the tests. The column and beam lengths and the entire test set-up were identical to the all welded connections also being tested at the University of Texas [5]. This was done for the following reasons. First, extra beam and column sections that were not used for the all welded connection tests where already available and could be used for the bolted connection test. Furthermore, it allows direct correlation between the bolted and the all welded connections

3.1 Test Setup

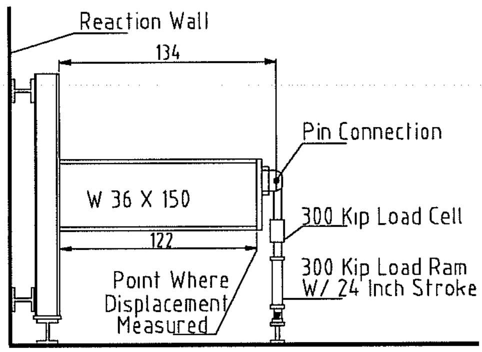

Figure 3.1 Test Setup

The test setup is shown in Figures 3.1, 3.2 and 3.3. The column was 136 inches from center to center of the end supports and the ends were allowed to rotate as if they were points of inflection. A cantilever beam was centered between the column supports and the hydraulic ram was attached to the end of the beam. The beam end load was applied at 134 inches from the column face and also allowed to rotate. The beam was white washed to show the yield patterns of the beam as it was loaded. The deflection was measured at 122 inches from the column face by a linear pot attached to each side of the beam flange. By

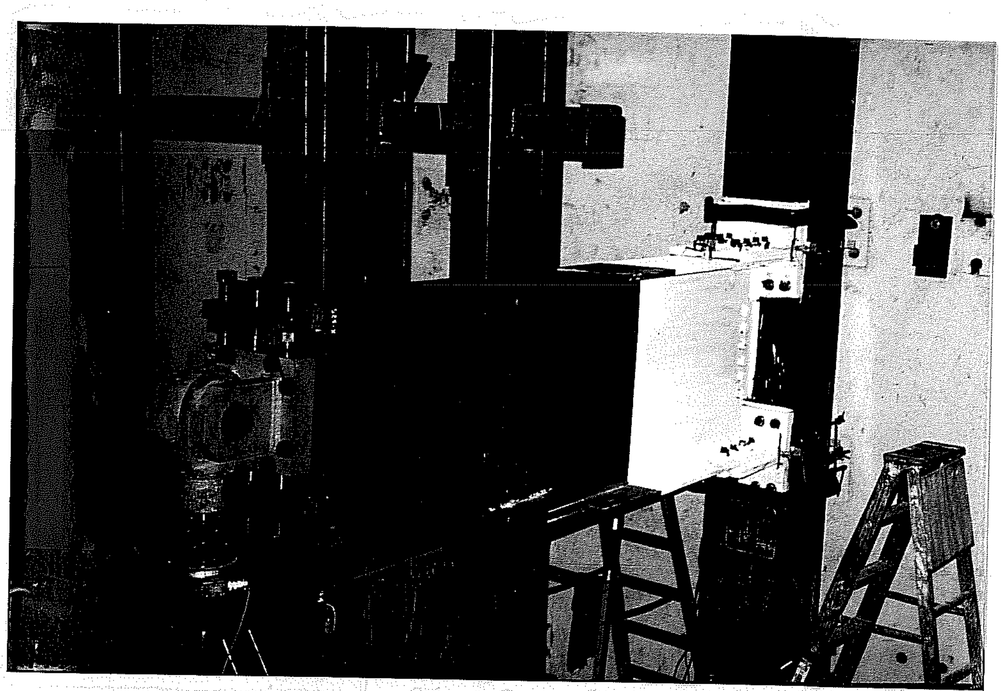

Figure 3.2 Test Setup

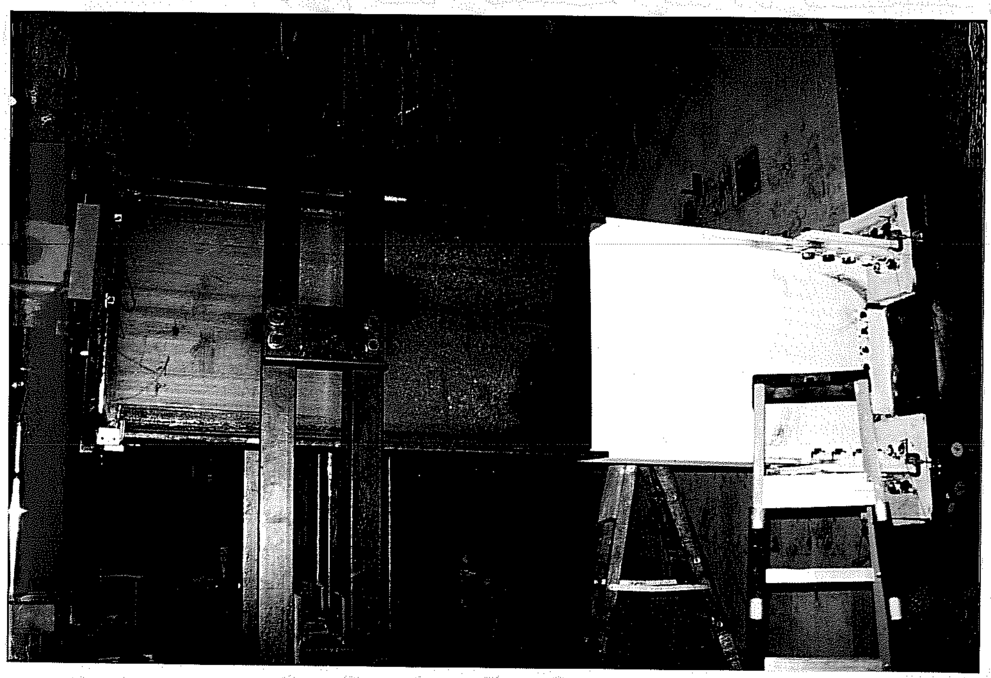

Figure 3.3 Test Setup

having two pots it was possible to check for beam torsion and erroneous readings. The hydraulic ram was a double acting 300 kip ram with a 24 inch stroke. A 300 kip load cell was attached for load readings, but these readings were also checked with intermittent pressure gage readings. All linear pots, strain gages and the load cell were connected to a single data acquisition system.

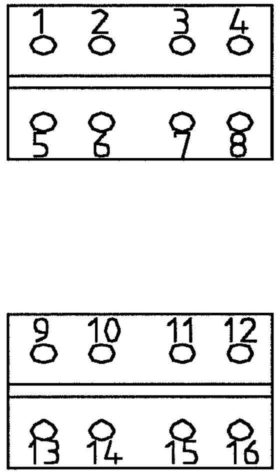

Figure 3.4 Bolt Numbering

The tension bolts were instrumented with strain gages that were placed in a 2 millimeter hole that was drilled through the center of the bolt head. The gages were in the grip of the bolt. Bolt numbering is given in Figure 3.4. A linear pot was placed on both edges of each tee stem to measure the center displacement of the tee flange from the column flange, as shown in Figure 3.5. A feeler gage was used on the first test to check the measurements of the pots at the bottom tee at the extreme conditions of each cycle. The displacement at the bolt line was also checked in the same manner.

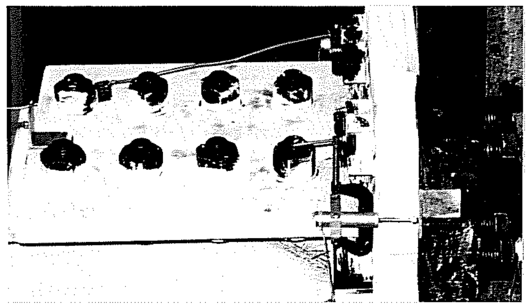

Figure 3.5 Linear Pot Measuring Tee Displacement

A linear pot was placed in the center of the top and bottom beam flanges to measure the relative displacement of the beam flange and the tee stem. This measurement includes three contributions of deformation: bolt slip, bolt bearing and beam and web elongation in the bolt zone. The numbering of these pots is given in Figure 3.6. A pot is shown in Figure 3.7. Bolt slip was also measured manually by scratching a line in the white wash at a bolt line. As the beam flange and the tee stem slipped past each

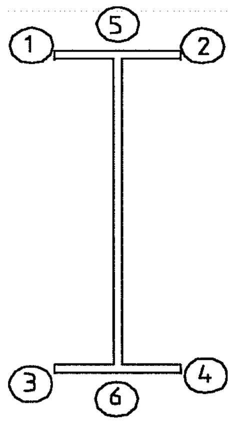

Figure 3.6 Linear Pot Numbering

other the line became offset and it was measured with a millimeter ruler at the most extreme conditions. Since no column stiffeners were used at the moment connections the change in distance between the column flanges was also measured with a millimeter ruler for specimen 2 and then with dial gages on subsequent specimens at both bolt lines for the bottom beam flange. The dial gages measured to the thousandth of an inch and were attached to the unloaded column flange.

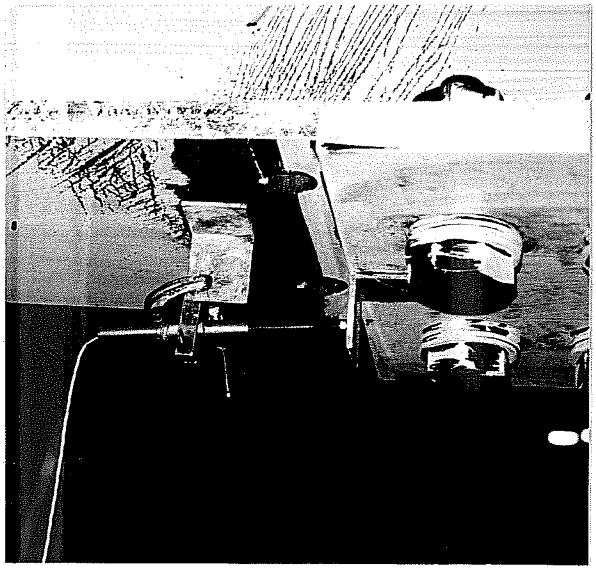

Figure 3.7 Linear Pot Measuring Bearing and Slip

3.2 Test Specimens

The test specimens were designed so that the various modes of failure could be established and that in most instances, the connection would fail and not the beam. This was accomplished by targeting the design strength of the connection at approximately 30 percent and 75 percent of the moment capacity of the beam. Five different connection configurations were tested and are described as follows.

Specimen 1-

In design it is common to assume that the moment in a connection is taken by a force couple where only the flanges of the beam participate. Specimen 1 was a simple connection with only the web bolted to the shear tab. This test was designed to determine the moment-rotation characteristics of the web alone so that the tee stubs forces in the subsequent tests could be established and compared with the common design assumptions.

Specimen 2-

The second specimen utilized tee stubs cut from the beam material. The nominal tee dimensions are given in Figure 3.8. This tee was chosen for two reasons. First beam and tee material properties would be the same and there would not be large discrepancies in the ratio of the strength of the tee to the beam. Secondly by utilizing the beam, tees could easily be fabricated. The holes in the tee flange were drilled 1/8 inch oversized for 7/8 inch A325 tension bolts with strain gages. These bolts were only snug tightened to determine if there were any adverse effects from not pretensioning them. The tee stub was 16 inches wide, as shown in Figure 3.8 approximately the same width as the column flange and approximately 4 inches greater than the beam flange width. In this tee, the controlling limit states were in the stem of the tee. Four 1 1/8 A490 flange bolts were used in 1 5/16 inch holes (1/8 inch oversized holes). The beam flange holes were punched and the tee stem holes were drilled. These bolts were then tightened 1/3 of a turn past snug condition as specified in the AISC manual for the turn of the nut method of tightening.

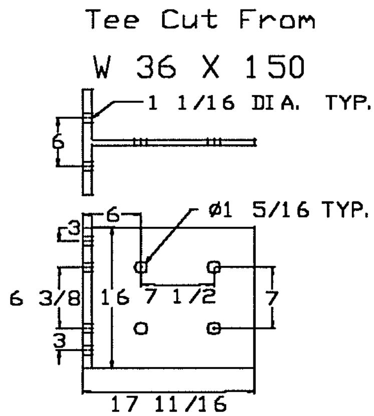

Figure 3.8 Specimen 2 Tee Detail

Specimen 3-

Specimen 3 again utilized the tee stub from the beam material. The difference was in the length of the web stem and in the size of the bolts. The detail may be seen in Figure 3.9. The stem was made longer so there would be room for ten bolts. The width of the tee remained at 16 inches. The flange of the tee utilized 1 inch A490 bolts in standard holes. The top tee used new bolts and did not have strain gages in

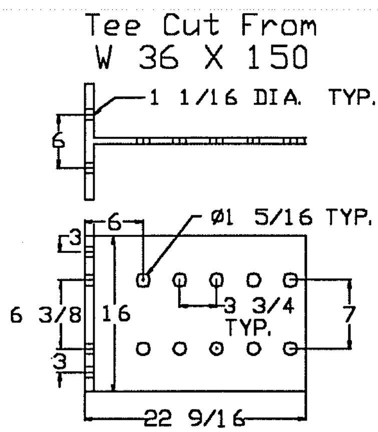

Figure 3.9 Specimen 3 Tee Detail

them. The bottom tee reused bolts with strain gages from the testing of specimen 4. These bolts had under gone some inelastic action which caused some difficulty in properly pretensioning them. The tee stem was drilled with ten 1 5/16 inch holes and the corresponding holes in the beam flange were punched for the 1 1/4 inch A490 bolts. Both surfaces were sand blasted for increased slip resistance. An attempt was made to tighten the bolts 1/3 of a turn past snug condition as specified in the AISC manual for the turn of the nut method of tightening. Difficulties in tightening will be included in the discussion of the test results. In this specimen the failure limit state was expected to be associated with the flange of the tee.

Specimen 4-

Specimen 4 was a WT 20 X 186 and can be seen in Figure 3.10. The 1 3/16 inch holes in the flange were punched by the fabricator and 1 inch A490 bolts were used (1/8 inch oversized). The bolts were strain gaged so the load could be determined throughout the test. The tee stub was 16 inches wide, as shown in Figure 3.10 approximately the same width as the column flange and approximately 4 inches greater than the beam flange width. The tee stem and the flange of the beam were punched by the fabricator with eight 1 5/16 inch holes for the 1 1/4 inch A490 bolts. Both surfaces were sand blasted for increased slip resistance. An attempt was made to pretension all of the bolts. With the 12 inch gage of the tension bolts, the failure limit state of the tee flange, bending was expected to control the performance of this specimen.

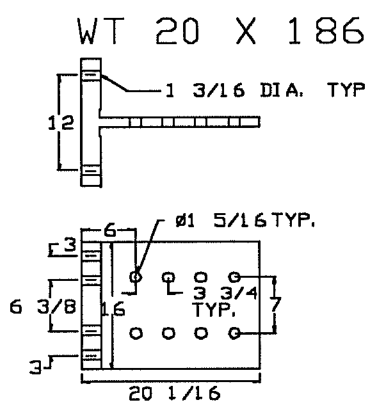

Figure 3.10 Specimen 4 Tee Detail

Specimen 5-

The fifth specimen used the WT 20 X 186 again but with closer tension bolt spacing as seen in Figure 3.11. The holes in the flange were originally punched for 1 1/8 inch bolts, but were modified by drilling to accept 1 1/4 inch A490 bolts. The bolts had strain gages in them so the load could be determined. The tee width is 16 inches as in specimen 4. Eight 1 1/4 inch A490 bolts were placed in the tee stem in standard size holes as in specimen 4. Both surfaces were sand blasted for increased slip resistance. An attempt was made to pretension all of the bolts. No failure of the connection was expected in this test; the limit state of the beam yielding (plastic hinge) was controlling.

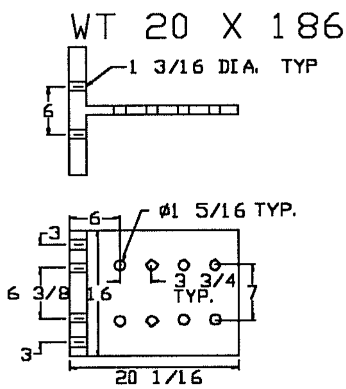

Figure 3.11 Specimen 5 Tee Detail

3.3 Bolt Behavior

Several randomly selected bolts with strain gages were tested of each bolt size. This was done to determine a calibration between bolt load and strain and to find the bolt behavior under a tension load. Load, strain, and for some, over-all axial elongation, were measured.

Three of the A490 bolts were tested, all three of them to failure. Figure 3.12 shows the typical load strain curve for the 1 inch A490 bolts used in test specimens three and four. The average ultimate bolt load was 96 kips and the accumulated permanent strain was 210 micro-inch / inch. Figure 3.13 shows the load displacement curve for the same bolt. Axial elongation was not originally measured, but it was found that at a point in the loading both load and strain would decrease as the overall length increased. The reason for this is that the strain gage was located in the body of the bolt and the inelastic action took place in the root of the threaded portion of

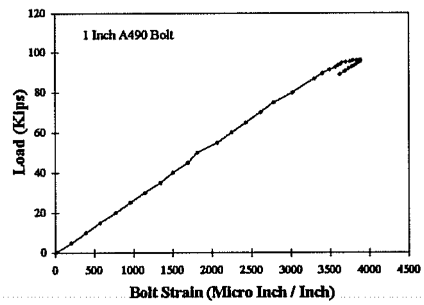

Figure 3.12 Strain of a 1 Inch A490 Bolts

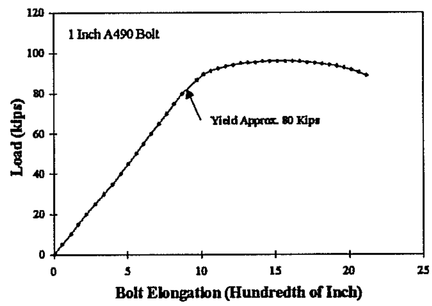

Figure 3.13 Elongation of a 1 Inch A490 Bolt

the bolt. As straining increases in the root and the load begins to decrease with the necking down of the section, the body of the bolt responds in an elastic manner and strain decreases. This action was very pronounced in the A490 bolts where the yield to ultimate tensile strength ratio is high. For the A490 bolts the body of the bolt is just reaching yield as the root is reaching ultimate. The consequence is that the body essentially remains elastic. For specimens utilizing A490 tension bolts, this helped in analyzing the data for the connection. As the connection is cycled and permanent strain is accumulated in the bolts, the bolt force will usually be able to be calculated from the theory of elasticity. The yield point is approximately 80 kips and the total deformation is 0.212 inches. The nut on the bolt was positioned furthest from the head while still being completely threaded, so that the maximum number of threads could participate in the inelastic action.

Five 7/8 inch A325 bolts were tested and, two of them were tested to failure. Figure 3.14 shows a typical load strain curve for a 7/8 inch A325 bolt used in the first connection test. The bolts were loaded and unloaded several times to find the bolt behavior after it had become inelastic. A noticeable amount of inelastic strain can be seen in this curve. Large amount of inelastic action caused the gage wire to be severed in the test fixture and the final unloading and decrease in strain before fracture is not shown. Of the two bolts tested to failure one failed by fracture of the net section and the other by striping of the threads. The average ultimate bolt load was 67 kips and the accumulated permanent strain was 622 micro-inch / inch. The load displacement curve for the same bolt is shown in Figure 3.15. This curve shows when the yielding of the root

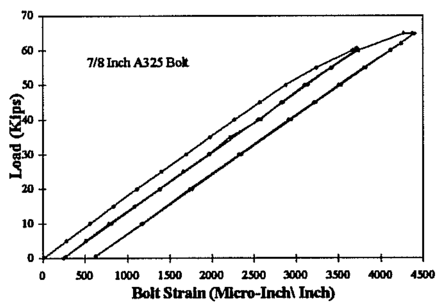

Figure 3.14 Strain of a 7/8 Inch A325 Bolt

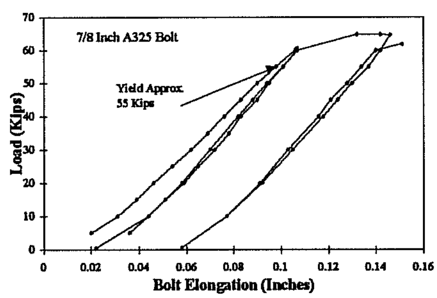

Figure 3.15 Elongation of a 7/8 Inch A325 Bolt

section occurs. The yield point is approximately 55 kips and the total deformation was not recorded. Mill certificates for these bolt were not available.

The 1 1/4 inch A490 tension bolts were never tested to failure. The mill certificates gave an average ultimate strength of 159.7 kips by the testing of five bolts. The shear flange bolts and the shear tab bolts were not intended to be broken and no problems from them arose during testing, so no material properties were found in addition to the mill certificate properties. The average values from the mill certificates for all of the bolts used in specimens one and three through five, are in Table 3.1.

Table 3.1 Mill Certificate Bolt Properties

Bolt Size / Bolt Use	Core Hardness (RC)	Tensile Strength (Kips)
1" - 6 1/2" A490 / Tension	33.3	93.6
1 1/4" - 8" A490 / Tension	30.4	159.7
1 1/4" - 4 1/2" A490 / Shear	34.2	To short to test
1" - 2 3/4" A325 / Shear	28.8	To short to test

3.4 Material and Section Properties

Actual beam, tee and hole dimensions were measured with a micrometer and section properties calculated. The plastic moment was based on measured coupon stresses and the different stresses for the web and the flange were accounted for. These values are reported in Tables 3.2, 3.3 and 3.4.

Table 3.2 Beam Dimensions

	Average Flange Thickness (in.)	Average Flange Width (in.)	Average Flange Hole Dia. (in.)	Average Web Hole Dia. (in.)	Calculated Plastic Moment, M_p (kip-in.)	Calculated Moment of Inertia, I_x (in.^{4})
Beam One	0.905 (0.873-0.925)	12.083 (12.063-12.125)	1.307 (1.303-1.310)	1.040 (1.018-1.057)	23189	9143
Beam Two	0.911 (0.885-0.928)	12.042 (12.063-12.125)	1.308 (1.305-1.310)	1.075 (1.052-1.090)	23215	9156

* values in parentheses are ranges from which the average was calculated

Table 3.3 Tee Stub Dimensions

	Average Top Flange Thickness (in.)	Average Bottom Flange Thickness (in.)	Average Web Thickness (in.)
Specimen Two	0.910 (0.895-0.925)	0.913 (0.885-0.946)	0.642 (0.633-0.655)
Specimen Three	0.914 (0.883-0.945)	0.912 (0.898-0.927)	0.641 (0.632-0.650)
Specimen Four	2.038 (2.011-2.056)	2.038 (2.013-2.061)	1.164 (1.155-1.182)
Specimen Five	2.054 (2.036-2.073)	2.058 (2.045-2.069)	1.161 (1.152-1.172)

* values in parentheses are ranges from which the average was calculated

Table 3.4 Initial Hole Diameter of Tee Stubs

	Average Flange Hole Diameter (in.)	Average Web Hole Diameter (in.)
Specimen Two	1.063 (1.062-1.064)	1.325 (1.320-1.334)
Specimen Three	1.060 (1.059-1.061)	1.327 (1.320-1.333)
Specimen Four	1.150 (1.131-1.164)	1.315 (1.310-1.319)
Specimen Five	1.305 (1.302-1.312)	1.315 (1.311-1.318)

* values in parentheses are ranges from which the average was calculated

Two tensile coupons were tested of each material type, location and orientation of those listed in Table 3.6. Material coupons were standard .505 inch diameter round coupons with 2 inch gage length and 1 1/2 inch wide flat plate coupons with an 8 inch gage length. For the WT 20 X 186 tees, both the flange and the web coupons were cut perpendicular to the rolling direction; these coupons were all .505 inch rounds. The purpose of this is that the stresses in the tees would be in this direction. Since the W 36 X 150 were used for tees and the beam, coupons were taken in the rolling direction for the web and the flanges and also perpendicular to the rolling direction for the flanges. Both plate and round coupons were taken to compare the results. Material properties are shown in Tables 3.5 and 3.6. The coupons were loaded at a moderate strain rate but were held at zero strain rate for 5 minutes at three points in the yield plateau and once at the ultimate peak to obtain static values. Note that for the W 36 X 150 section, the static yield value was 37.8 ksi compared to the mill certificate yield of 51 ksi.

Table 3.5 Mill Certificate Steel Properties

Specimen / Material Type	Mill Yield Point Ksi	Mill Ultimate Tensile Ksi	Mill Percent Elongation
W 36 X 150 / ASTM A36	51.00	63.00	29 %
W 40 X 372 / ASTM A572 Gr. 50	59.60	80.19	24.5 %

Table 3.6 Coupon Tests Steel Properties

Specimen / Material Type	Location/ Orientation To Rolling Direction	Coupon Type	Static Yield Point Ksi	Static Ultimate Tensile Ksi	Percent Elongation	Strain at onset of Strain Hardening
W 36 X 150 / ASTM A36	Flange / Parallel	Plate	37.65	57.21	31.3 %	0.0208
W 36 X 150 / ASTM A36	Flange / Parallel	505 Round	37.99	59.55	41.2 %	0.0209
W 36 X 150 / ASTM A36	Web / Parallel	Plate	44.58	59.53	27.4 %	0.0236
W 36 X 150 / ASTM A36	Flange / Perpendicular	505 Round	37.84	60.25	33.5 %	0.0180
W 40 X 372 / ASTM A572 Gr. 50	Flange / Perpendicular	505 Round	49.17	70.71	28.2 %	0.0178
W 40 X 372 / ASTM A572 Gr. 50	Web / Perpendicular	505 Round	50.86	73.10	30.0 %	0.0148

*Values are averages of two coupons

3.5 Test Procedure

The testing consisted of a slowly applied load that was cycled up and down. Since specimen 1 was only checking for the level of load which the web carried it was only cycled once up and then down. For the moment connections each loading stage was cycled three times before the next stage was applied. The exception to this is specimen 5 where once initiation of beam fracture was observed the specimen was cycled five times at each load stage to ascertain if the fracture would progress. Specimen 2 carried a lower initial slip load then the remainder of the specimens and had oversized holes to causing it to slip further. For this reason the first stage was loaded until the load displacement plot showed that slip of the bolts had started to occur and the next loading stage was loaded until the specimen showed an increase in stiffness, due to the bolts going into bearing. The third stage began displacement controlled loading were 1 1/2 inches of end displacement was achieved. The stages thereafter were increased by increments of a half of an inch until three inches of total displacement was reached where the displacement was incremented by one inch. Specimens three thru five load controlled the first loading stage and were loaded to 75 kips or until slip was initiated. Since the highest slip load achieved was 80 kips the second load stage was displacement controlled to 1 inch of end displacement. The subsequent loading stages then proceeded at .5 inch increments until the specimen lost load carrying capacity.

Chapter 4 Test Results

All data will be plotted against end load for comparison reasons, and individual calculations will be made to convert to other types of loading, i.e. bolt load. To give a measure of magnitude, Table 4.1 gives reference loads, calculated using measured section and material properties. Test criteria of plastic rotations of 0.03 radians were imposed, while trying to achieve energy dissipation, different ultimate load levels and failure modes. Each component of the connection having an effect on the load-to-end displacement history will be discussed individually. Hypotheses will be made to explain the behavior.

Table 4.1 Reference Loads

	Specimen 2	Specimen 3	Specimen 4	Specimen 5
End Load Corresponding to Plastic Moment at Column Face (Kips)	173	173	173	173
End Load Corresponding to Plastic Moment at the Bolt Line Closest to Load Ram (Kips)	192	204	198	198

For an overview, the load-to-end displacement histories for each specimen are shown in Figures 4.1, 4.2, 4.3, 4.4, and 4.5. The load envelopes for each specimen are given in Figures 4.6, 4.7, 4.8 and 4.9, where important events of each test are shown, and a comparison against the stiffness of an experimental welded connection is shown. Specimen 2 reached a maximum end load of 110 kips and average displacement of 4.40 inches. The test was stopped because the column with its shear tab was to be used by specimens three and five and there was concern that the shear tab would be damaged due to the large end displacements that had been caused by the excessive bearing deformation. Specimen 3 reached a maximum end

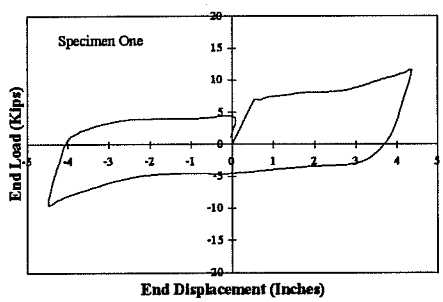

Figure 4.1 End Displacement Specimen 1

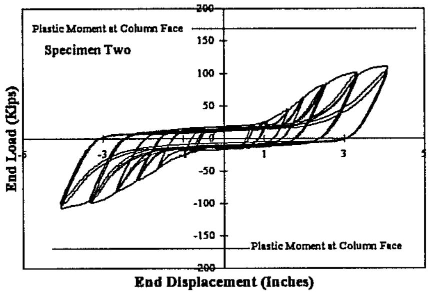

Figure 4.2 End Displacement of Specimen 2

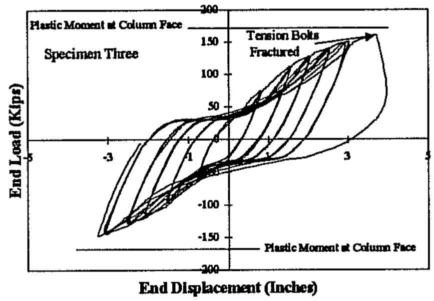

Figure 4.3 End Displacements of Specimen 3

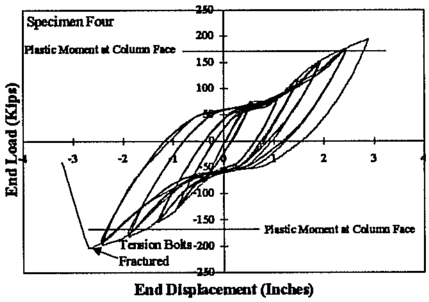

Figure 4.4 End Displacement of Specimen 4

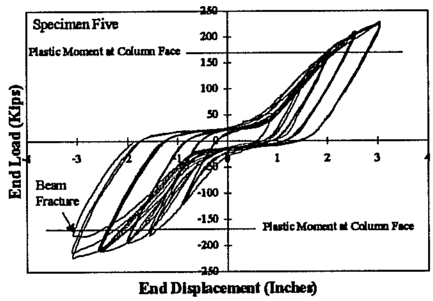

Figure 4.5 End Displacement of Specimen 5

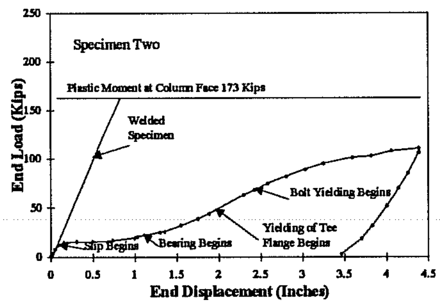

Figure 4.6 Load Envelope of Specimen 2

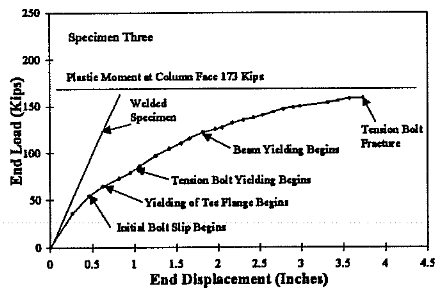

Figure 4.7 Load Envelope of Specimen 3

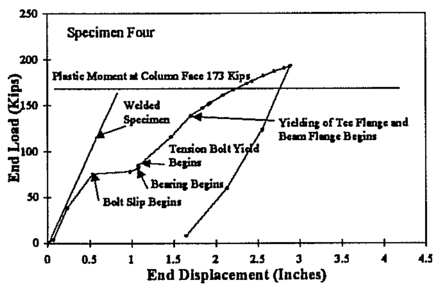

Figure 4.8 Load Envelope of Specimen 4

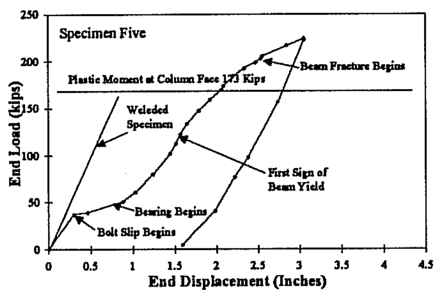

Figure 4.9 Load Envelope of Specimen 5

load of 159 kips and average end displacement of 3.73 inches. The test was stopped when the row of tension bolts below the bottom beam flange fractured. Specimen 4 reached a maximum end load of 201 kips and average end displacement of 2.9 inches. The test was stopped when the row of tension bolts above the top beam flange fractured. Specimen 5 reached a maximum end load of 221 kips and average end displacement of 3.0 inches. The test was stopped when a fractured progressed through the top beam flange and began to unload the connection.

4.1 Shear Connection

Specimen 1 was tested for determining the moment capacity of the web shear connection. The load displacement curve for the beam is shown in Figure 4.1. The maximum end load was 11 kips, 10% of the load for specimen 2, the weakest moment connection and 5% of the strongest specimen, specimen 5. The connection was rigid until the connection slipped where large displacements where achieved with little increase in load. What this means is that for calculations ignoring the web contribution are less accurate at low levels of load.

4.2 Beam Behavior

In moment connections, especially bolted moment connections, the compression flange is stiffer than the tension flange at the column face. The unproportionate stiffnesses causes the neutral axis to shift toward the compression flange of the beam and cause the compression flange to experience higher forces. Away from the connection region the neutral axis moves back toward the beam center line. By force equilibrium, if no axial force is in the member, the tension force must equal the compression force. For equilibrium to be satisfied the web must have a greater tension force than compression force and the difference in the flange forces can only be as great as the force the web can realize. By lowering the neutral axis the web will take a greater share of the moment than shown in specimen 1, helping to relieve stress in the tension flange for a given moment. However, the most efficient location for the neutral axis in a symmetric beam is the beam center line. At the bolt line closest to the ram, the moment is 11 % greater than the moment associated with the fully plastic gross section, assuming the web is fully participating and using the appropriate measured yield stresses for the flange and the web.

The beam steel was A36 and the connection was designed with the nominal AISC manual properties.[1] The UBC requires that the ratio of the net area (A_e) to the gross area (A_g) of the beam flanges be greater than 1.25 times the ratio of the yield stress to the ultimate tensile stress[12], $\frac{A_e}{A_g} \geq \frac{1.25F_y}{F_u}$. The beam tested met the design requirements of the UBC, where $\frac{A_e}{A_g} = 0.784 \geq \frac{1.25F_y}{F_u} = 0.776$ (F_y = 36 ksi, F_u = 58 ksi). The measured properties reported in chapter three are very close to manual design properties, making $\frac{1.25F_y}{F_u} = 0.819$ (F_y = 38 ksi, F_u = 58 ksi), which means the test beam violated the UBC recommendation by 5 percent. If the mill certificate material properties were used, $\frac{1.25F_y}{F_u} = 0.99$ which means that virtually no holes may be put in the beam.

Yielding in the beam flanges was observed adjacent to the bolts at an end load of 40 kips for specimen 2. Specimens 3, 4 and 5 had signs of yielding around 120 kips. All of the specimens showed signs of yielding prior to the gross section yield moment at the flange bolt line closest to the ram. The bolts going into bearing could be causing bearing stresses to cause local yielding. If the entire projected bolt area is used in calculating the bearing stress, specimens two, three, four and five, would have bearing stresses of 27 ksi, 29 ksi, 33 ksi, and 39 ksi respectfully. The phenomena of the bolt rotating causing higher bearing stresses on the edge of the hole, as discussed in section 4.3, could cause earlier bearing yield than calculated. Even though specimen 2 had the lowest calculated stresses, bolt rotation would have been the most prevalent with the over-sized holes. For specimens three four and five a more likely reason for early yield would have been the reduced section of the beam due to the bolt holes. The reduced net section gives a lower yield moment causing first yield at the bolt line closest to the load to be 141 kips for specimen 3 and 136 kips for specimens 4 and 5, close to the 120 kips where yield was first seen. Yielding may have occurred prior to that observed at 120 kips, because it is only the inside flange face that could be observed at the bolt line and 141 kips and 136 kips are calculated for first yield at the extreme fiber. The theoretical values assumes the entire section is acting to resist the moment. The web has been shown to carry little moment, specimen 1, causing the flanges to carry a greater portion of the moment near the column face. At the bolt line the web will carry less moment than the theoretical value but more of the moment than calculated at the column face. By carrying less than the theoretical value, premature yielding may have occurred in the flanges. The stresses near the bolt are not evenly distributed as assumed in the theory, this may also cause the discrepancy in the values.

Early yielding on the net section is acceptable if the ultimate strength is high enough to allow the gross section to yield and produce the required ductile beam rotations. Accounting for the flanges and web having different yield stresses the end load required to produce a full plastic section at the bolt line closest to the ram, 116.75 inches from the point of action of the load, is 196 kips for specimens four and five. Specimen 4 reached 201 kips before the tension bolts fractured and specimen 5 reached 221 kips before the section fractured. No fracture could be seen in the bolt holes until the tees had been removed and the beam was inspected. At this time some necking in the beam flange could be detected as well as the fracture as shown in Figure 4.10. With loads as high as 221 kips specimen 5 had no visual signs of beam hinging or local buckling of the beam flanges. When the specimen reached a load of approximately 200 kips some necking and small fractures could start to be seen. At this time each load stage was cycled five times instead of three to see if the fracture would progress. At a

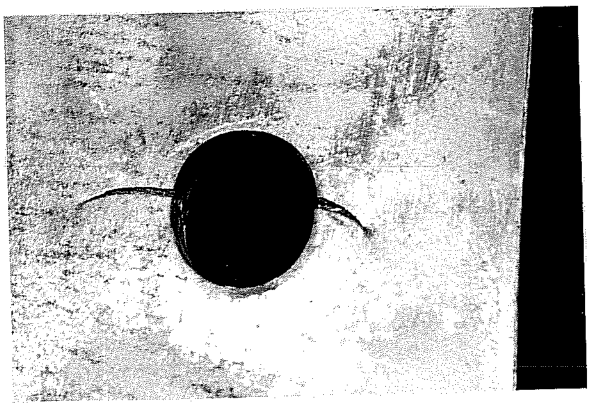

Figure 4.10 Beam Fracture of Specimen 4

displacement of 3 inches and 221 kips the fracture became severe and at each load cycle the end load was reduced. Figures 4.11 and 4.12 shows the extent of the fracture and Figure 4.13 shows the fracture in the adjacent hole.

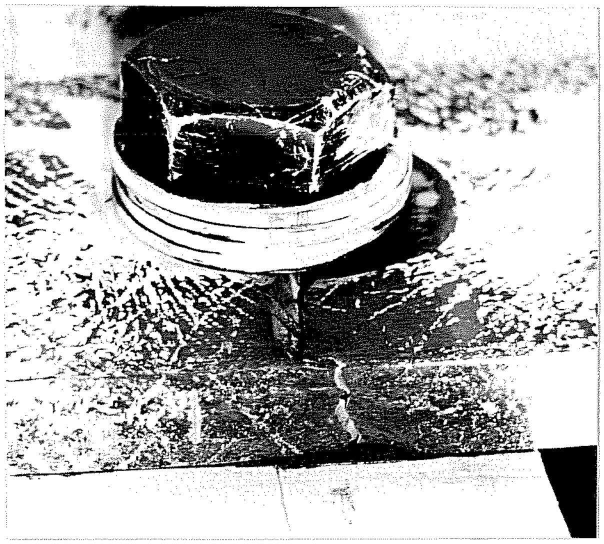

Figure 4.11 Beam Fracture of Specimen 5

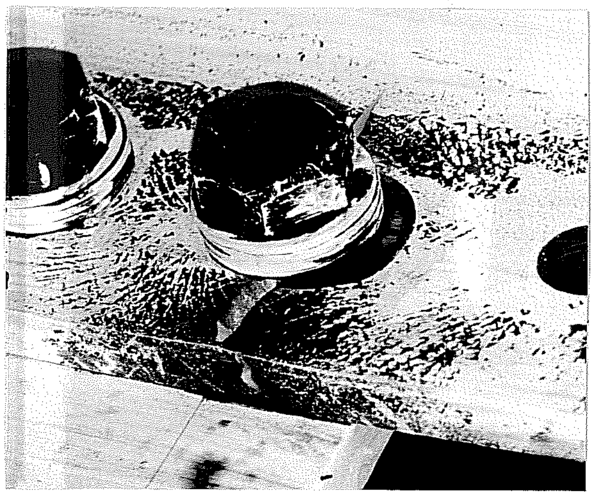

Figure 4.12 Beam Fracture of Specimen 5

A plastic hinge did not form when the plastic moment was exceeded because the beam was reinforced in the connection region. The tee stem was substantially stiffer than the beam flanges. The moment at the edge of the tee was still 11 percent greater than the plastic moment. If the beam is thought to be fully confined in the connection region then it is reasonable to assume the plastic moment will occur at $d_{BEAM}/2$ from the end of the tee, 98.75 inches from the point of loading. At this distance the moment at the maximum end load was only 21,824 inch-kips 94 % of the plastic moment of 23,215 inch-kips.

The material in the cross-section including the bolt holes did show good ductility and force redistribution. Yield lines in the web extended 8 inches from the tension flange showing that the web was caring a substantial amount of the moment. Even though the tension region showed good ductility the bolt holes did not elongate very much and fracture always started at the bolt hole. As noted earlier fracture started at approximately 200 kips of end load. As seen in Figure 4.14 yield lines are not seen directly next to the beam bolt holes. This is attributed to the beam bolt holes being punched, rather than drilled, straining the material adjacent to the holes. The yielding from the punching uses up some of the material ductility causing premature fracture at the holes. Gaylord, Gaylord, and Stallmeyer [9] recommend a reduction factor of 0.85 for punched holes rather than the 1/16 inch used by AISC [1] for tension members. A reduction of 0.85 would mean a reduction of approximately 11/16 inch per hole for beam tested.

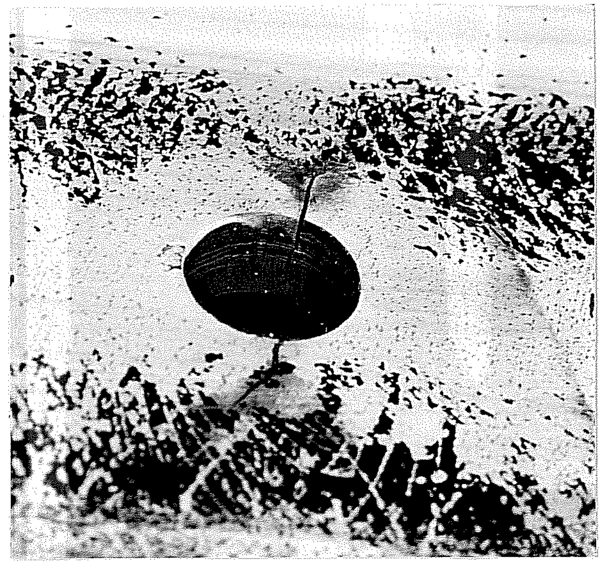

Figure 4.13 Beam Fracture in the Non-Bolted Hole

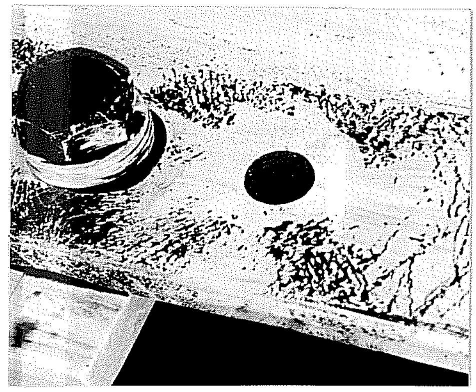

Figure 4.14 Yielding Near a Punched Hole

In calculating a failure moment for the net section of the beam several problems arise. First there is the problem of possible shear lag; essentially only the beam flanges are connected in such a way to take the tension forces from bending. The shear lag reduction factor in the AISC specification was developed for members in pure tension, where an element in the flange at a given stress is equally important as an element in the web of the same stress. [1,9] In a beam the distance from the neutral axis is important. An alternate method of calculating the importance of the web needs to be used to find the load where the beam will fracture. It can be assumed from the force level in the beam, that the neutral axis is at the mid-depth of the beam, but this is not known. The AISC code [1] requires an additional 1/16 of an inch be used in calculating the net section, where Gaylord, Gaylord, and Stallmeyer [9] suggest a reduction factor of 0.85 for the use of punched holes. Observations from the test would indicate that the real behavior was between the AISC 1/16 inch and the reduction of the 0.85 factor of 11/16 inch as stated earlier. In the region of the flange near the web the steel is confined and is restricted from necking down increasing the fracture strength of the steel and raising the moment where fracture occurs.

Several methods of calculating the fracture moment were investigated including the use of the shear lag reduction, the punched hole reduction, the AISC 1/16 inch reduction, and the use of an effective depth of web participation. All methods including the use of the shear lag reduction factor were substantially different than the measured failure moment at the fracture location. The method which calculated the moment the closest utilized the strain compatibility at the flange web intersection and the observation that yielding was observed 8 inches into the web. It is realized that stress strain relationships and depth of yielding in the web are factors not known in design, so a simplified method was also found that gave very close results to the more accurate method. The simplified method utilized some canceling errors in the calculation. The web is considered to be in a complete state of yield canceling the error of not considering the added moment from the increased stress from the part of the web in strain hardening and the fillets. By using the smaller hole increase of 1/16 of an inch the increased strength from the confined steel from the flange to web region of the beam is canceled.

4.3 Tee Stub to Beam Shear Connection.

Slip between the beam flange and the tee stem can produce large end displacement with small amounts of slip movement. In example, a 1/16 inch slip displacement corresponds to 0.42 inches of displacement at 122 inches from the column face. A connection should be designed so that under service and wind loads slip does not occur, providing a stiff structure. For seismic loads slip provides a controlled method of energy dissipation.

Specimen 2, with only four beam flange bolts, was designed for a low slip load, and used oversized holes. The connection slip surfaces were mill scale providing a calculated slip end load of 26 kips, based on a slip coefficient of 0.33 and the minimum specified bolt tension of 80 kips. Slip occurred at approximately 15 kips, 11 kips less than expected, possibly due to the cutting oils not being removed prior to assembly reducing the slip coefficient. Once the connection slipped the oversized holes would allow a theoretical slip of 3/16 inch and produce an average end displacement of 1.27 inches in each direction before the bolts begin bearing. Figure 4.15 shows the measured relative average movement of the beam and the top tee. The total slip prior to first bearing was about one half of the theoretical slip because the holes were not perfectly aligned.

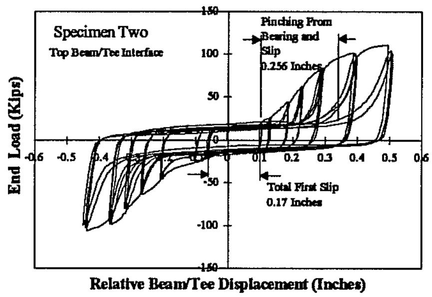

Figure 4.15 Bearing and Slip Displacement of Specimen 2

Once slip had occurred the flange bolts moved into bearing and the connection regained stiffness allowing the end load to increase. Bearing deformation permanently deforms a hole and consequently as the second cycle is loaded the connection must slip further before bearing and increasing stiffness. The connection still reaches the same load and displacement, however, as Figure 4.15 shows, the loops become pinched and there is a loss of energy dissipating ability. The small number of bolts and the oversized holes dramatized this effect for specimen 2. The linear pots mounted on the beam (pot numbers five and six) measured an average pinching of the slip-to-end load curve of 0.256 inches, as illustrated on Figure 4.15. The pinching varied depending on which flange was measured and if the movement of the beam was downward or upward. The measurements varied from 0.201 inches to 0.296 inches. Bearing deformation, as measured with calipers after the completion of the test is seen in Table 4.2. The beam / tee-side refers to the side of the tee stem and beam flange in contact, while the outside refers to the side not in contact with the other element. The code with its limit of $2.4F_u$ on the bearing stress implicitly gives a limit on bearing deformation of 25% of the original hole diameter. [1,8] The bearing deformation of the specimen 2 tees and beam fall within this limit, however most of the end beam deflection was due to the oversized holes and the elongation of the holes due to bearing. The other connection elements and the beam bending contributed very little to the final beam displacement. The bearing stress on the beam holes were 123 ksi and the holes in tee stem were 87 ksi, which corresponds to 2.04 and 1.50 times the coupon ultimate stress respectively. Bearing stresses were calculated by using the ultimate end load less the slip load and by using the full projected bolt area, assuming constant stress in the bolt hole.

Table 4.2 Bearing Deformation of Specimen Two

	Top Tee (in.)	% Change	Bottom Tee (in.)	% Change	Top Beam Flange (in.)	% Change
Original Size	1.325	NA	1.325	NA	1.307	NA
Beam / Tee-side	1.577 (1.552-1.608)	19 %	1.577 (1.567-1.586)	19 %	1.456 (1.433-1.483)	11 %
Outside	1.520 (1.498-1.540)	15 %	1.570 (1.532-1.610)	18 %	1.338 (1.311-1.390)	2 %

* values in parentheses are ranges from which the average was calculated

A general trend for the bearing deformation was for the sliding surface side of the hole to be greater than the outside surface. Figure 4.16, 4.17, and 4.18 explains why the side of the holes on the sliding surface have a larger hole deformation. Figure 4.16 shows how the bolt rotates in the hole and bears on the edge of the hole which has a very small bearing area. Figure 4.17 shows a possible stress distribution in on of the slipping plates. By force equilibrium the summation of the forces in the hole must equal the pulling force on the plate. This corresponds to the larger bearing deformation seen on the tee to beam

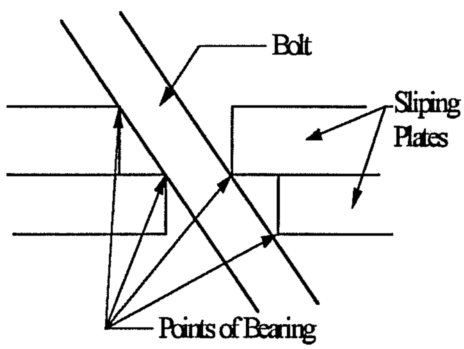

Figure 4.16 Rotation of a Bolt

interface than on the outside of the plates as shown Figure 4.18 and in Tables 4.2, 4.3, 4.4, and 4.5.

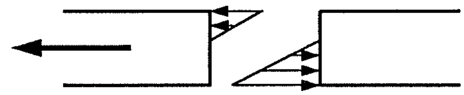

Figure 4.17 Equilibrium of Bearing Forces

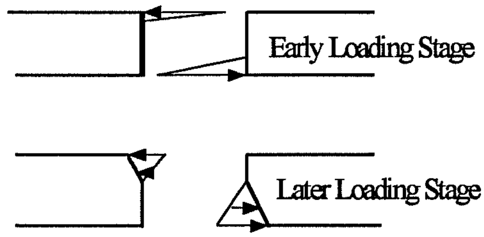

Figure 4.18 Change in Bearing Stress and Deformation

Specimens three through five all experienced unexpectedly low levels of load when slip occurred. After specimen 4 was tested the clamping force was checked in a Skidmore Willhelm bolt calibrator. A maximum of 85 kips clamping was found for the 1 1/4 inch A490 bolts as delivered with a more realistic load of 80 kips, as determined by visual observation, for the bolts installed in specimen 4. The clamping force was limited by the torque capacity of the impact wrench. This is 22 kips below the minimum specified of 102 kips. To remedy this the threads and faying surface of the nut were waxed. This allowed the minimum clamping force to be reached. Approximately a week elapsed between these tests and the time specimens 3 and 5 were tested. In specimen 3 additional washers were needed because the bolts were too long. It was probable that not enough washers were used and the nuts were turned into the thread run out area of the bolt. However, when specimen 5 also showed low levels of slip the bolts were re-tested and this time with waxed nuts only 56 kips could be reached. All conditions appeared to be the same. The same impact wrench, lab air outlet, and hoses were used and the compressor was at full pressure. The hoses were checked for leaks and two small ones were found and fixed, but no change in the maximum tension was found. The wrench did seem to be acting differently and may have been the problem.

Specimen 3 was expected to slip at approximately 140 kips, based on a slip coefficient of 0.5 and the minimum specified clamping force of 102 kips, instead it slipped at 45 kips. The sand blasting helped the first cycle of slip but the slip load was reduced by 15 kips a 33% reduction in the second cycle as seen in Figure 4.19. The test verified the expectation of the loss of slip resistance due to the abrasion of the slip surface. As the load was cycled the slip load did fluctuate and it was hypothesized that particles from the abrasion of the slip surfaces and the bearing deformation increased the slip coefficient for some cycles. The bearing stress for specimen 3 was reduced by 60% in the tee and 51% in the beam over specimen 2, by using ten bolts instead of four bolts and changing to 1 1/4 inch bolts from the 1 1/8 inch bolts. Table 4.3 shows the level of bearing deformation in Specimen Three, an average reduction of 84%. The smaller amount of pinching can be seen in Figure 4.19, a reduction of relative displacement from 0.256 inches at 100 kips for specimen 2 to 0.0526 inches at 140 kips for specimen 3, an 80% reduction. The bearing stress on the beam holes were 42 ksi and 60 ksi in the holes of the tee stem, which corresponds to 0.73 and 1.00 times the coupon ultimate stress, respectively. Even though there was a 60 % and 51 % reduction in bearing stress from specimen 2 to specimen 3, there was a 79 % and 93 % reduction in bearing deformation of the tee and the beam respectively. This difference is attributed to the non-linear behavior of bearing deformation.

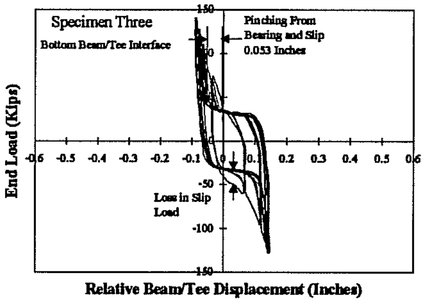

Figure 4.19 Bolt Slip and Bearing of Specimen 3

Table 4.3 Bearing Deformation of Specimen Three

	Top Tee (in.)	% Change	Bottom Tee (in.)	% Change	Top Beam Flange (in.)	% Change
Original Size	1.327	NA	1.327	NA	1.308	NA
Beam / Tee-side	1.379 (1.345-1.431)	3.9 %	1.378 (1.346-1.417)	3.8 %	1.319 (1.315-1.322)	0.8 %
Outside	1.364 (1.328-1.410)	2.8 %	1.365 (1.333-1.395)	2.9 %	1.309 (1.308-1.310)	0.1 %

Specimen 4 was expected to slip at 110 kips but, instead initially slipped at 75 kips a high for all the tests. Specimen 4 also experienced a drop of 20 kips in the slip load for the cycle following the initial slip, a 27% reduction, as shown in Figure 4.20. Figure

4.21 shows the overall behavior of the beam slip and bearing for specimen 4. Pinching for this test was very minimal at an end load of 200 kips the average pinch was 0.0215 inches, which corresponds to approximately an eighth of an inch of end displacement. Table 4.4 shows the permanent bearing deformation after the test. The averages for top and bottom tee were close so only the top tee is reported for specimen 4 and five. The bearing stress on the beam flange holes was an average 66 ksi and 51 ksi for the tee stem, approximately 1.14 times and 0.70 times the coupon ultimate stress respectively.

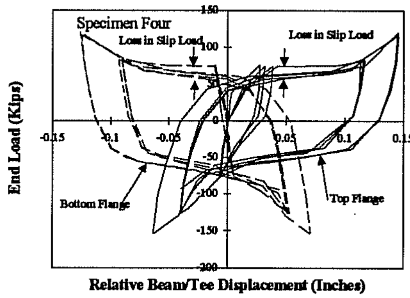

Figure 4.20 Loss of Slip Load of Specimen 4

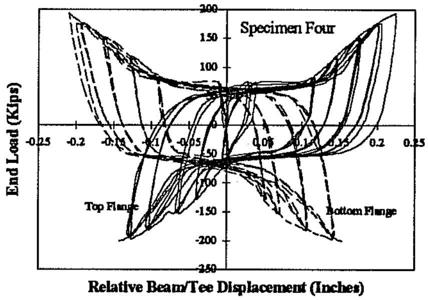

Figure 4.21 Bearing and Slip Deformation of Specimen 4

Table 4.4 Bearing Deformation of Specimen Four

	Top Tee (in.)	% Change	Top Beam Flange (in.)	% Change
Original Size	1.315	NA	1.308	NA
Beam / Tee-side	1.342 (1.331-1.353)	2.1 %	1.355 (1.351-1.359)	3.6 %
Outside	1.319 (1.317-1.324)	0.3 %	1.328 (1.325-1.330)	1.5 %

* values in parentheses are ranges from which the average was calculated

The initial slip of specimen 5 was 35 kips, 75 kips below the expected 110 kips. By the second cycle the slip load was 25 kips, a 29% reduction. For the three tests with the sand blasted slip surfaces an average reduction of the slip load of 30% occurred for the cycle following the initial slip. If the coefficient of friction is assumed to be 0.5 for the blasted surface, then the coefficient for the cycle after slip for the four tests is approximately that of a mill scale surface. Both linear pots did not function properly and no slip data is available for the test of specimen 5. No comparison of pinching can be made directly. Specimen 5 experienced greater bearing deformation than specimen 4, as shown in Table 4.5, however specimen 5 did experience greater bearing stresses, 80 ksi on the beam flange holes and 63 ksi on the tee stub web holes, approximately 1.38 times and 0.86 times the coupon ultimate stress respectively.

Table 4.5 Bearing Deformation of Specimen Five

	Top Tee (in.)	% Change	Top Beam Flange (in.)	% Change
Original Size	1.315	NA	1.308	NA
Beam / Tee-side	1.391 (1.354-1.415)	5.8 %	1.445 (1.435-1.449)	10.5 %
Outside	1.315 (1.311-1.322)	0.0 %	1.371 (1.360-1.381)	4.8 %

4.4 Tee Stub Flange and Tension Bolt Behavior

The tension bolt behavior and the tee stub flange behavior is closely linked to each other, complex, and difficult to separate. An overview of the bolt and tee flange behavior of this test program will be given in this thesis, however a more in depth investigation is given in Reference 23 (Tension Bolt Behavior in Moment Connections for Seismic Applications). Several tee stub sizes, bolt sizes and configurations were tested giving a variety of different load displacement responses. Some of the key characteristics of each specimen need to be discussed before comparisons to the predicted values can be made.

The hysteresis loops for the center displacement of specimen 2 is shown in Figure 4.22 and 4.23. As the tee of specimen 2 was cycled into the inelastic range three distinct slopes form after the initial displacement. The first slope is distinctive of specimen 2 and not seen in the other specimens. Slope two and three can be seen

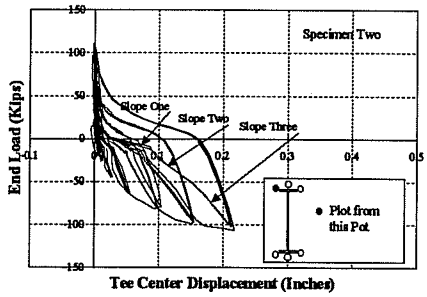

Figure 4.22 Tee Flange Displacement of Specimen 2

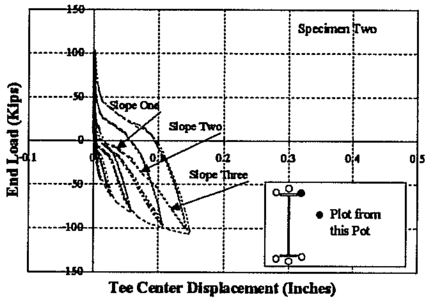

Figure 4.23 Tee Flange Displacement of Specimen 2

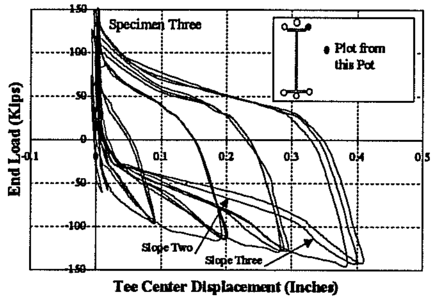

Figure 4.24 Tee Flange Displacement of Specimen 3

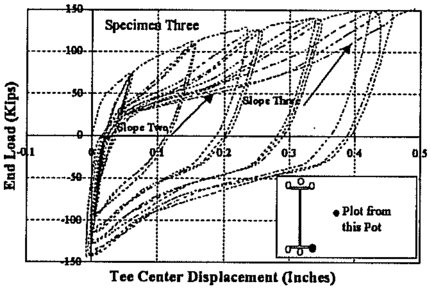

Figure 4.25 Tee Displacement of Specimen 3

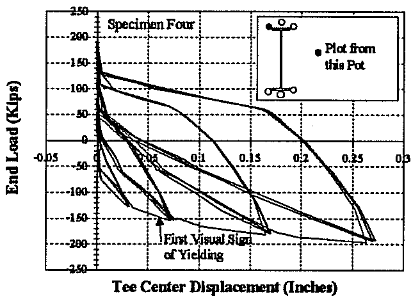

Figure 4.26 Tee Flange Displacement of Specimen 4

in specimen 3, Figure 4.24 and 4.25, while specimen 4 only shows slope two, Figure 4.26 and 4.27. The tension bolts of specimen 2 were snug tight and did not have the required minimum tension, unlike the other specimens. Slope one is very flat with relatively large displacements occurring with little increase in load. Loss of pretension and the bolts becoming loose contribute to causing the flat behavior of slope one. Figure 4.28 compares a few loading cycles to approximately the same displacement. Figure 4.28 shows a close comparison between specimens 2 and 3's initial load envelope, where subsequent cycles show the loss of bolt tension and the flatter slope one in specimen 2.

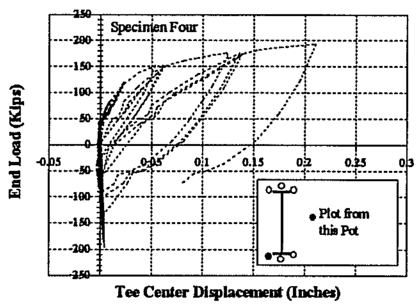

Figure 4.27 Tee Flange Displacement of Specimen 4

Slope three exhibited in specimens 2 and 3 is characterized by an increase in stiffness which occurs at approximately the same load, for a given specimen. However, at each loading stage the displacement at which the increase in stiffness occurs is greater. This slope is explained by observing enlarged holes on the column side of the tee after the tee had been removed. A significant amount of bearing deformation occurred in these holes as can be seen in Table 4.6. The deformation only took place on the column side of the tee and no deformation could be measured on the outside face. The hole elongation was always away from the tee stem, which mean the bearing did not occur due to beam shear which would have caused deformation on both sides of the hole. The increase in stiffness and bearing deformation is attributed to the tee stub going into a membrane action and putting the tee into tension instead of pure bending. Calculations show that at measured displacement stiffness due to pure membrane action would be substantial.

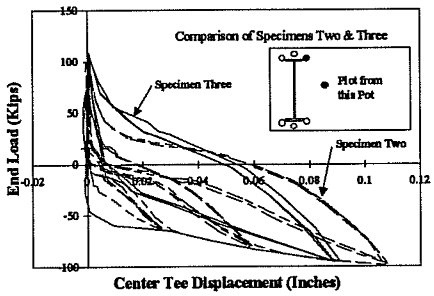

Figure 4.28 Comparison of Specimen 2 and 3 Tee Displacement

Table 4.6 Bearing of Tension Bolts

	Orientation to Beam	Initial Average Hole Diameter (in.)	Final Hole Diameter on Column Side (in.)	Change in Hole Diameter (in.)
Specimen Two	Outside Row	1.063	1.079 (1.073-1.086)	0.016
Specimen Two	Inside Row	1.060	1.083 (1.079-1.086)	0.023
Specimen Three	Outside Row	1.060	1.178 (1.173-1.188)	0.118
Specimen Three	Inside Row	1.060	1.146 (1.138-1.155)	0.086

* values in parentheses are ranges from which the average was calculated

For the tee to have bearing deformation the tension in the tee flange causes the tension bolts to carrying membrane shear in addition to the tension forces. Using a linear relationship between bearing stress and hole elongation up to a bearing stress of $2.4F_u$ and a hole elongation of 1/4 of the bolt diameter and assuming a triangular stress distribution across the thickness of the flange, the membrane shear force would be 34 kips per bolt for the outside bolt row of specimen 3. This is just an example of the possible magnitude of the membrane shear force applied to these bolts. With out knowing the stress distribution in the bolt hole, over the loading history, an accurate force can not be calculated.

Slope two of specimens 2,3 and 4 is characterized by being less stiff than the initial slope and it is always the first slope to appear as the tee is cycled into the inelastic range. This stiffness is the result of two different things. When the bolts are pretensioned they do not elongate until the applied load is in excess of the initial load. As the bolts become inelastic they lose the initial pre-load and the start to respond sooner in the loading history making the connection less stiff.

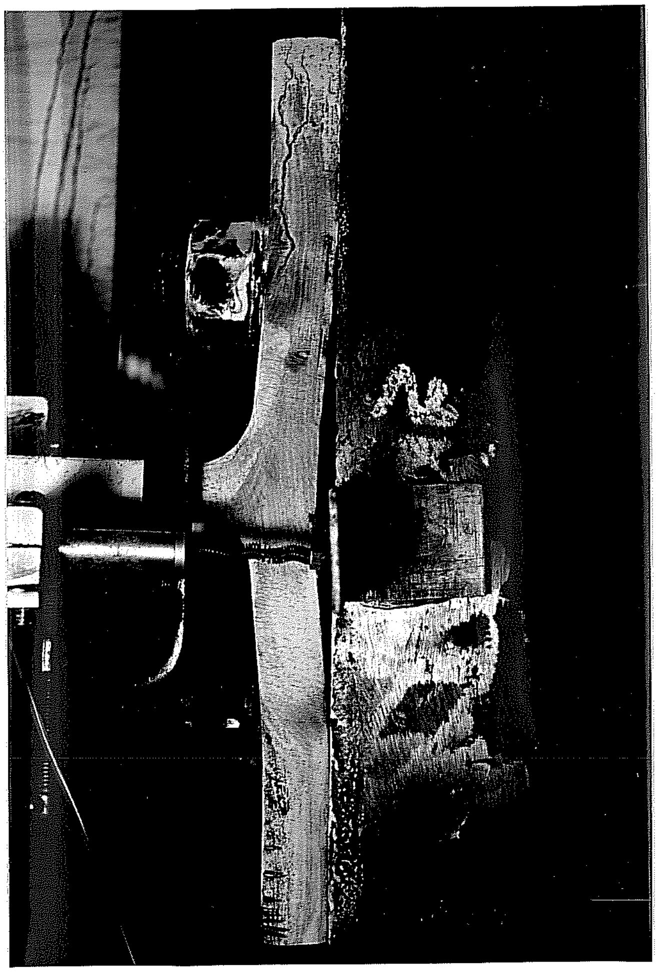

Figure 4.29 Tee Flange Bending Stage 1

The second is that as the tee starts to become inelastic the tee has permanent bending deformation that is not reversed with reversed loading. Figures 4.29, 4.30, 4.31 and 4.32 shows the cycle of the bending of the tees as the load is cycled. When the tees are straight, the tee between the bolts can be thought of as a fix-fix beam. The prying forces allow a negative moment at the bolt lines. As the tee is loaded out from the column, hinges form at the bolt line and the tee stem, Figure 4.29 and 4.30. When the load is reversed no prying force is available so, the tee flange acts as a simply supported beam in the reverse direction with the supports at the bolt lines, Figure 4.31. Hinges then forms at the stem and the tee is bent straight between the bolt lines, but no with reverse prying force, no reverse hinges form at the bolt lines, and the tee rocks around the bolts leaving the flange bent away from the column, Figure 4.32. When the load is reversed again, no prying force can be applied to the tee flanges bent away from the column until the flange tip of the tee rock back the other direction and make contact with the column. The connection now acts as if it is simply supported. A simply supported beam is less stiff than a fixed beam, changing the slope of the curve, from the initial slope to slope two.

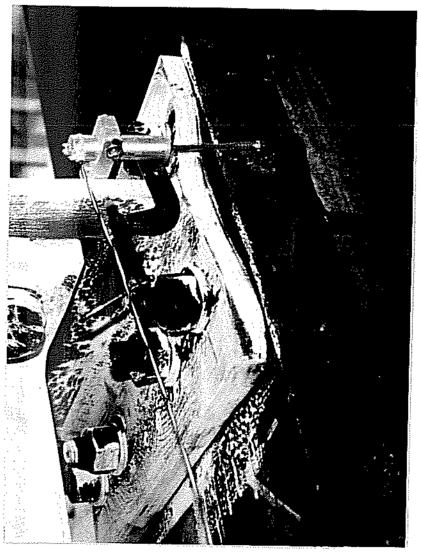

Figure 4.30 Tee Flange Bending Stage 1

When the bolts are placed close to the web the flange may act more like a deep beam, or a shear beam. For specimen 2 and three with relatively thin flanges the span to depth ratio was 4.7. if the fillets are accounted for the ratio becomes 2.9. The tee flange being a rectangular section is not limited in bending ability by phenomena like local buckling and if the bend is sufficiently wide, fracture is not a problem in limiting the flanges deformation capacity. The tee flange is capable of deforming into a geometry which carrying the applied forces differently. This is seen in the membrane action of specimens two and three.

Table 4.7 gives the calculated ultimate design loads, based on the Astaneh method[3] given in chapter two for each specimen along with the load at failure, the type of failure that was predicted to control capacity is also shown. Failure here is defined as the point where the tee fails to carry equal or greater load at increasing levels of displacement.

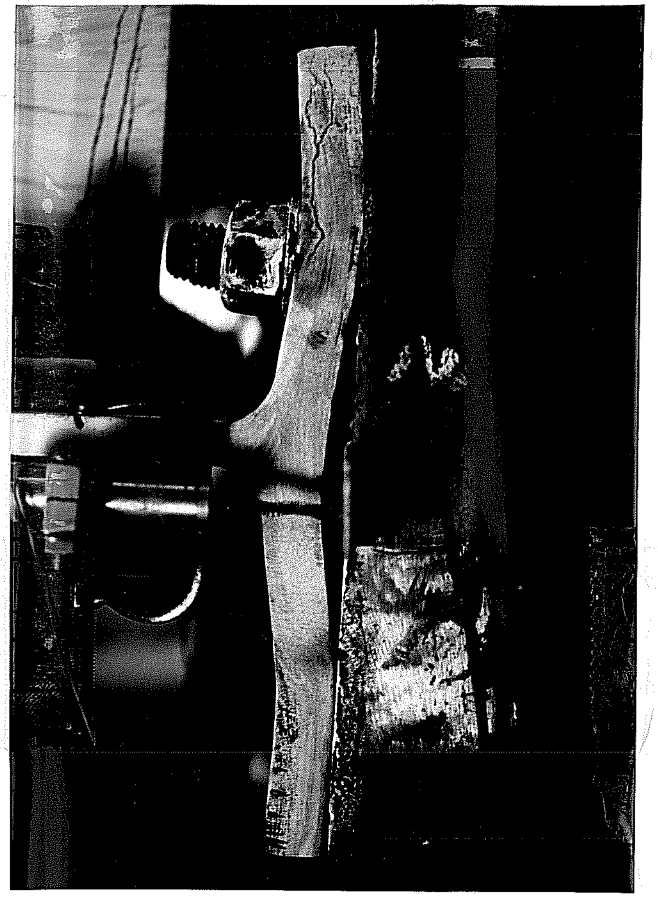

Figure 4.31 Tee Flange Bending Stage 2

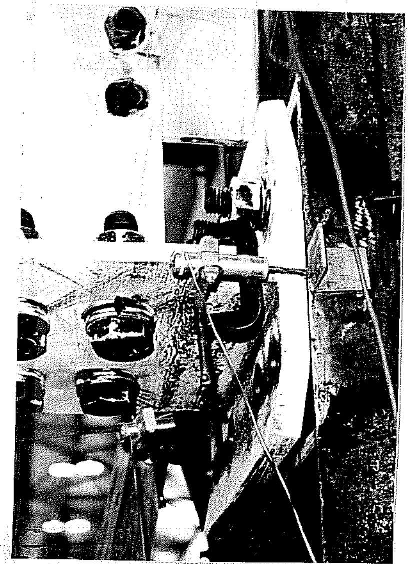

Figure 4.32 Tee Flange Bending Stage 3

All design loads were calculated using actual tee measurements and coupon stresses. Table 4.7 shows that even the closest of design values are different by approximately 40 percent. The predicted failure mode was not necessarily the actual failure mode.

Table 4.7 Tee Flange Failure Loads

	Tee Design Load	Predicted Failure Type	Corresponding End Load	End Load at Tee Failure
Specimen Two	194 kips	Flange Failure	52 kips	NA
Specimen Three	194 kips	Flange Failure	52 kips	159 kips
Specimen Four	469 kips	Bolt Failure with Prying Forces	126 kips	201 kips
Specimen Five	1168 kips	Bolt Failure with Prying Forces	314 kips	NA

From observing the initial slopes of Figures 4.22, 4.23, 4.24, and 4.25, yielding of the tee flange starts to occur at an end load of approximately 50 kips. From a review of the experimental bolt tension data, which is discussed in detail in Reference 23 (Tension Bolt Behavior in Moment Connections for Seismic Applications), yielding of the bolts started to occur at approximately 65 kips of end load for specimen 2 and 85 kips of end load for specimen 3. Specimen 2 had a maximum displacement of 0.218 inches and an average of 0.183 inches but, never received a load which caused failure of the tee. Displacement at the first sign of yield was an average 0.03 inches. Specimen 3 had a maximum displacement at failure of 0.58 inches and a corresponding end displacement, due to the tee deformation only, of 1.95 inches. Displacement at the first sign of yield was 0.02 inches.

Specimen 4 had the bolts reach yield before the tee yielded. The bolts yielded at and end load of 75 kips and are seen in Figure 4.26 and 4.27 starting to become less stiff at this load. yielding was first observed in the top tee on the first cycle to -151 kips of end load. By looking at Figure 4.26 it appears that yielding did start in this cycle but probably closer to 130 kips of end load. At yield the tee displacement at the center of the tee was approximately 0.03 inches. The ultimate displacement was 0.336 inches for the top tee and 0.210 inches for the bottom tee. Figure 4.33 directly compares the slopes of and loading envelope of the top and bottom tee. With end displacement of the beam controlling the loading,

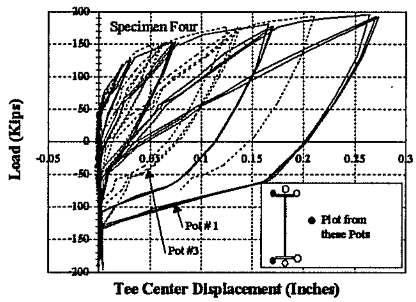

Figure 4.33 Comparison of Top and Bottom Tee Flange Displacement

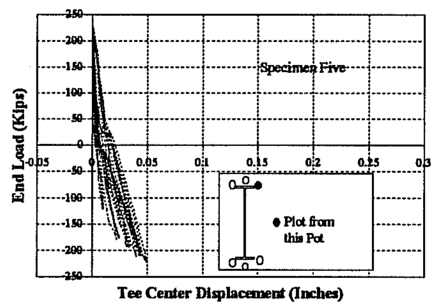

Figure 4.34 Tee Flange Displacement of Specimen 5

maximum end load for each cycle was not the same. However, the loading envelopes and the loading and unloading slopes are similar. It should be noted that the calculated ultimate end load of specimen 4 with no prying forces was 205 kips.

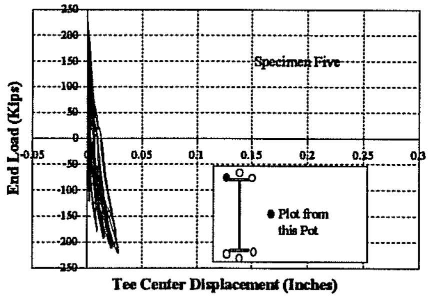

Figure 4.35 Tee Flange Displacement of Specimen 5

Specimen 5 did not yield the bolts or the tee. Figures 4.34 and 4.35 shows the displacement of the bottom tee. Some displacement can be seen but most of the movement is attributed to pot slip and errors in readings. The tee of specimen 5 forced the failure into the beam.

4.5 Column Flange Behavior

Measurements for column flanges bending were made only at the extreme displacement positions for each set of cycles. No measurements where taken for specimen 1 because the flanges where not connected to the beam. Specimen 2 had the measurements made with a ruler and were too small to read on a millimeter scale. The dial gages for specimens three through five showed minimal movement of the column flanges. Specimen 3 had movements of approximately 0.02 inches at an end load of 150 kips, specimen 4 had movements of 0.015 inches at an end load of 150 kips, and specimen 5 had movements of 0.03 inches at end loads of 220 kips. These kind of movements may be critical to a welded connection, however they are negligible to the bolted connection. Column flange bending is limited by AISC, but large bending displacements due to thin column flanges may cause additional prying forces in the bolts.

Chapter 5 Design Implications

Governing codes have few guidelines and recommendations in the design of bolted connections in seismic zones. This test program was a preliminary program to investigate the feasibility of steel bolted moment-resisting frame connections in seismic zones, and to identify excellent and poor behavior of the connecting elements. Some practical recommendations to designers considering bolted moment connection designs are given and several areas of concern are identified.

Fracture is an undesirable failure mode and needs to be avoided. This test program had fracture of the beam in the high strength specimens prior to achieving 0.03 radians of rotation and the classical plastic hinge never formed. For a system where the flexural hinge is to occur in the beam it is recommended that the moment diagram be drawn through the point where the moment at the depth of the beam divided by two ($d_{BEAM}/2$) from the end of the connecting element, the tee stem in this case, be equal to the plastic moment of the beam (M_p). From this diagram the moment at the critical section may be determined and this moment is the required flexural strength (M_u) of the element being designed, as shown in Figure 5.1. There is a concern of some designers that connecting elements may have different strengths than the beam and the beam itself may have a larger yield to ultimate ratio than the reduction factor.

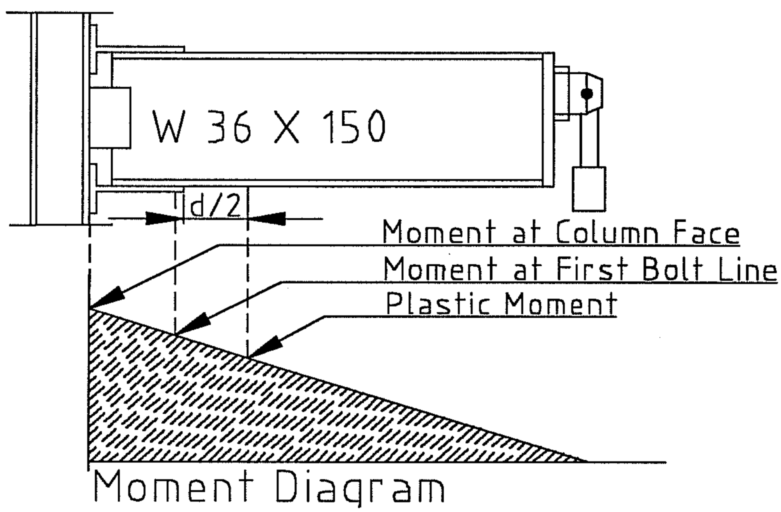

Figure 5.1 Assumed Moment Diagram of Beam

Beam fracture of the net section should be checked in a manor that does not assume the point of fracture occurs at the point that flexural hinging occurs, as does the current codes. It is proposed that a fracture moment is calculated at the critical net section, the bolt line closest to the inflection point, and is checked against the moment as previously determined. A fracture moment of the net section may be calculated by using the mid-depth of the beam as the neutral axis and by calculating the moment capacity as if the section is a hybrid section. The net effective area of the tension flange is used with a stress equal to the ultimate tensile strength of the material. The web is treated as if the entire web is at a yield condition. The fracture moment capacity is then equal to the fracture moment capacity of the flange (M_{Ff}) plus the yield moment capacity of the web (M_{Fw}). This is done by first calculating the net effective area of the flange. For connections where the bolts are not staggered:

$$A_{ef} = \left[B_f - N \times (d' + 0.0625)\right] \times t_f \qquad (5\text{-}1)$$

where:

N = Number of bolts in a line

t_f = Thickness of beam flange

B_f = Width of beam flange

d' = Nominal diameter of bolt hole

The fracture moment capacity of the flange is then:

$$M_{Ff} = A_{ef} \times F_{uf} \times (d_{BEAM} - t_f) \qquad (5\text{-}2)$$

where:

F_{uf} = Ultimate tensile strength of the flange

d_{BEAM} = Overall depth of the beam

The yield moment capacity of the web is then:

$$M_{Fw} = \frac{(d_{BEAM} - 2 \times t_f)^2 \times t_w \times F_{yw}}{4} \qquad (5\text{-}3)$$

where:

t_w = Thickness of the web

F_{yw} = Yield point of the web

The fracture moment capacity of the net section is then:

$$M_F = M_{Ff} + M_{Fw} \tag{5-4}$$

and

$$\phi \times M_F \geq M_u \tag{5-5}$$

where:

$\phi = 0.9$ for bending

It should be noted that the critical section for the beam usually is the bolt line furthest from the column. This is because the moment in the beam is reduced at each bolt line.

The author calculated the fracture moment of several sections varying the beam depth and the ratio of the plastic modulus of the flange to that of the web (Z_f/Z_w). Standard holes were assumed for 1 inch diameter bolts; the net flange width was taken as the gross flange width minus $2 \times (1 + 1/16 + 1/16)$ using the standard AISC procedure[1] for two holes. The fracture moment was compared to the moment taken at the bolt line with the plastic moment $d_{BEAM}/2$ away. The inflection point was held constant for all trials at 130 inches from the bolt line. By comparing different sections the feasibility of calculating a fracture moment could be studied. When a fracture reduction factor of 0.75 for the flange and a bending reduction factor of 0.9 for the web was used no sections were found to meet the required strength for A36 steel with a ratio of $F_u/F_y = 1.61$. The sections varied between depths of 36 inches and 18 inches and Z_f/Z_w ratios of 1.90 to 5.70. If the reduction factor was relaxed to 0.9 for both the web and the flange, then some sections would be satisfactory. If an over strength factor, or grade 50 steel is used then even these sections will no longer meet the strength requirements.

The study showed that as the plastic modulus ratio increased the sections could more easily meet the requirements. A more important factor was how the flange material was distributed. Flanges of the same area, but that were thinner and wider had higher fracture moments, given the same lever arm. The wider flange had a smaller percentage of material removed with the same size holes. An option not studied was the use of bolts smaller than 1 inch in diameter. For the larger sections this option is not very practical. For a W 36 X 150, using the same inflection point as in the study, 18, 1 inch A490 bolts would be required. For sections with less moment capacity smaller bolts are an option.

In the design of frames, the problem of fracture becomes a more serious problem using deep beams. In illustration, if one inch square holes are cut into a beam flange and all of the deformation is assumed to occur in the hole region, the maximum beam rotation is a function of the ultimate strain of the steel and the beam depth, $\Theta = \dfrac{2 \times \varepsilon_u \times d_{hole}}{d_{BEAM}}$. If the ultimate strain is assumed constant at 0.25 inch / inch the ultimate beam rotation for a 36 inch beam is 0.014 radians, where as for an 18 inch deep beam the ultimate beam rotation is 0.028 radians, close to the desired 0.03 radians. The fracture moment formulations presented herein have met force equilibrium but strain compatibility is not considered. What this means is that shallow beams may be able to provide the required rotations of 0.03 radians without forming the gross section plastic hinge required by the force calculations. Allowing a beam section not meeting force requirements to alternately meet deformation requirements may be an option. It is difficult for deep beams to meet either the strain or the force requirements making it necessary to use a greater number of shallow beams in a frame to meet stiffness requirements.

An option to avoid beam fracture would be to use fillet welds to connect the beam flange to the tee stem, eliminating the net section beam fracture problem. The connection would still be designed the same way with the hinge forming $d_{BEAM}/2$ from the end of the tee stem. Fillet welding the tee stem would eliminate bolt slip and stiffen the connection.

In checking fracture of the tee, the moment at the bolt line closest to the column is determined as shown above, and then the tension force pulling on the tee is found by dividing by the depth of the beam. If the section of the tee stem changes between bolt lines then the force is reduced by an amount equal to the shear capacity of the previous bolt lines. Once the required ultimate strength of the web section is determined the net section strength is checked in accordance with AISC design procedures. [1] Gross section yielding is checked in the same way, but the moment at the column face should be used.

In designing in accordance with current codes [1,12] that do not allow any inelastic action to occur in the joinery the current design procedures for the flange of the tee and the tension bolts are not completely correct, but give a good estimate of where inelastic action will begin to occur. In forcing the beam to hinge the required design strength should be determined at the column face as stated above. The design examples given in the AISC manual [1] and by Astaneh [3] use the factored bolt capacity in determining the thickness of the tee flange. This is overly conservative. By reducing the bolt capacity the required flange thickness increases, however a reduction factor of 0.9 for bending is already present in the calculation. The bolt reduction should only be used in determining bolt size. In determining the flange thickness the true bolt capacity should be used, placing only the bending reduction into the calculation. The tension bolts need to be pretensioned. If any inelastic action does occur, loss of pretension will occur. If the pretension is low, snug tight bolts for instance, the bolts will become loose and pinching of the hysteresis loops may occur.

Premature slip of the flange bolts had detrimental effects on the test specimens. Specimen 4 which had the highest slip load also had the lowest level of pinching in the hysteresis loops due to bearing deformation. It is also known that premature slip can cause excessive undesirable drift under service and wind loads. Close attention needs to be paid to the inspection of the tightening of the flange bolts. It is recommended that the slip load be above the required design strength for the connection under service and wind loads, calculated using factored loads and mean unreduced slip loads. The maximum level of the slip load should be below the load where any inelastic action occurs in the connection. How far below the maximum load is debatable. Tests showed that sand blasting increased the slip load for the first cycle of slip, but then was ineffective in increasing the slip resistance of subsequent cycles. It is recommended that sand blasting in slip critical connections to increase slip resistance not be used in seismic zones.

The friction associated with the slip of the connection has been demonstrated as a good method of dissipating energy. [10,11] The constant non-destructive loops due to friction of these tests demonstrate this dissipating ability. Bolted Slotted Hole Energy Dissipaters of references 10 and 11 are good examples of the phenomena of bolt slip. When friction and bolt slip are the sole method of energy dissipation and are properly design they hold much potential. However, if bolt slip is not the sole method of energy dissipation slotted and oversized holes should not be used, unless specific testing on the subject can show no detrimental effects. The use of oversized holes in the testing of specimen 2 showed that the increased slip distance allowed excessive beam rotations with no increase in load.

The flange bolts and bolt bearing should be designed for the force calculated by dividing the moment at the column face by the beam depth. Bearing gives good energy dissipation in the first cycle of loading, but subsequent cycles do not have this same ability due to the permanent elongation of the hole. As limited by AISC, bearing stresses may not exceed 2.4 times the ultimate tensile stress of the connected material.[1] The excessive level of pinching in the hysteresis loops of specimen 2 while still being below this level suggests that in seismic applications this level of stress may need to be lowered. If the code value of 0.25 inches for 1 inch bolts was used at $2.4F_u$ and assuming that both connecting plates where at the critical value, for a 36 inch deep beam this would correspond to a beam rotation of 0.014 radians in each direction. If the beam depth was reduced to 20 inches the beam rotation would increase to 0.025 radians in each direction. At $1.38F_u$, a 10.5 percent increase in the original hole diameter and a final hole diameter 15.6 percent of the bolt diameter for specimen 5 was experienced, and at $1.5F_u$, an 11 percent increase in the original hole diameter of specimen 2 was experienced. A 10 percent bearing deformation of a 1 5/16 inch hole corresponds to a pinch in the end load to end displacement curve of approximately 0.9 inches in each direction or a beam rotation of 0.007 radians in each direction. To help control this excessive pinching it is recommended that bearing stress be kept below $1.4F_u$ and more preferably $1.0\ F_u$.

The bolt pretension clearly helped the response of the connection and the level of bearing deformation, it is also recommended that the bearing force acting on each hole be reduced by the corresponding mean unreduced friction force applied by the clamping of that bolt. In example, a one inch thick A36 steel plate with one inch bolts with an applied load of 120 kips would have an unreduced bearing stress of 120 ksi or $2F_u$. If the bolts are A490's the minimum clamping force is 64 kips giving a mean slip force of 21 kips for a mill scale surface. The reduced bearing stress is then 99 ksi or $1.65F_u$. The provision of $1.0F_u$ calculated using a reduced bearing stress should never violate the provision of $2.4F_u$ using an unreduced bearing stress. To ensure the $2.4F_u$ bearing provision is never violated the bolt diameter can be limited by 6.3 times the thickness of the connected elements, $d \leq 6.3t$. If the bolt diameter is 1 inch then the minimum plate thickness would be 0.16 inches.

The philosophy of making the beam the sole dissipater of energy and sole contributor to inelastic rotation in a connection is a product of all welded connections, where the weld is inherently brittle. A connection that can demonstrate good ductile load displacement response outside the column should be acceptable even if that response is not completely from the beam. As seen in these tests ductile response can be achieved through the inelastic action of the tees. It is of concern that enough stiffness can be given to the connection if all of the inelastic action is to occur in the tee stub. A connection with the response shared between tee action, bolt slip, and beam hinging may be desirable. By reducing the demands of beam rotation local buckling may be limited and loss in beam strength may be avoided.

These tests show the inadequacy of the current design procedures for predicting the strength of a tee stub in tension. More testing is needed to determine a reliable method of determining the load displacement response of a tee stub hanger. Ultimately bolt failure needs to be avoided. Information gathered from this test program would suggest several things. First that through membrane action of the tees, tee loads can reach loads capable of fracturing the tension bolts. It was shown that substantial membrane forces were present in the bolt failure of specimen 3. Secondly the failure mode, bolt failure with prying forces at ultimate loads becomes bolt failure without prying forces (specimen 4). The maximum tee displacements stated for specimens three and four are at bolt failure and would be unconservative to use as maximum design displacements. The hysteresis loops for the tee displacement showed some pinching due to the bending of the tees. This may be avoided by providing a second row of bolts that would provide a reverse prying force and widen the loops for the tee displacement. These bolts would also provide strength backup for the first bolt row. It should be kept in mind that these bolts do not contribute to the bolt resisting strength until the bolts of the first row become inelastic. A problem with providing a second row of bolts is that it moves the first row of bolts closer to the web and reduces the level of deformation that may be provided by the tee flange before entering into membrane action. Since bolt failure must be avoided the tension bolts should be designed above the maximum probable moment the beam can achieve.

A design example using the recommendations of this chapter is given in the Appendix.

Chapter 6 Summary

The test program tried to predict the failure of several bolted connections, find if under cyclical loads these failure modes governed, find the level of ultimate plastic rotations bolted connections could achieve and to compare the plastic rotation to the 0.03 radians now being called for by the profession. It was a desire to ultimately find a connection that was feasible and able to achieve 0.03 radians of rotation, without brittle failure. Five full scale tests were performed on bolted moment-resisting beam-to-column assemblies at the University of Texas at Austin. These connections were evaluated for there behavior, rotation capacities, moment capacities and modes of failure. Stiffness, ductility, loss of connection strength, and energy dissipating ability have been used in creating guidelines and recommendations for the design of bolted moment-resisting connections in seismic regions.

Specimen 2 was the only specimen to reach 0.03 radians of non-elastic rotation, but only reached 65 percent of the nominal plastic moment. Specimens 3, 4 and 5 did not reach the required rotations, due to brittle failure. Beam fracture was the most severe failure mode and caused the most concern. A method of calculating the fracture moment and to predict the beam fracture was found, but beam sections able to avoid beam fracture are limited. By limiting the beam sections able to preclude fracture, the versatility of bolted connections are limited. Possible ways to avoid the limitations would be to field weld the tee stem to the beam flange, or to increase the number of frames with special moment connections in the structure and better distribute the seismic loads. The use of punched holes may have caused earlier fracture of the beam flange than if drilled holes were used.

With adequate research beam damage may be limited by allowing the tees to participate in the inelastic rotations of the connection. Current design procedures are very conservative and do not predict the strength of the tee flange well. The tee flange is very ductile but the tension bolts must be designed to preclude fracture. All bolts need to be properly tensioned to minimum specified levels for the connection to have good energy dissipating ability.

Slip of the beam flange under high clamping force can dissipate large amounts of energy and limit damage to the connection. Bolt bearing deformation causes pinching of the hysteresis loops and loss of energy dissipating ability. By limiting the bearing stresses to $1.0F_u$ pinching of the hysteresis loops is controlled.

Bolted moment-resisting beam to column connections under this test program have failed to produce the large rotations now being called for in the design profession, without producing brittle failures. Beam fracture and the lack of understanding of the load displacement histories of tee stub hanger connections limited the success of the connection behavior. A bolted connection providing a safe ductile response is possible with adequate research. Some of the topics that need further investigation are as follows:

1. A method to calculate a useable tee strength and to predict the load displacement history. Guidelines need to be set to insure bolt failure will be avoided.
2. The method presented in this thesis for calculating the moment at which fracture occurs in a beam needs to be checked for reliability and correctness.
3. Does the point where a plastic hinge forms really occur at $d_{BEAM}/2$ from the end of the tee.
4. What effect does different tee stem thickness have on the point where the plastic hinge forms.
5. What effect does the shear tab have on the fracture moment strength.
6. What effect does punched holes versus drilled holes have on the fracture moment strength.
7. How much of an effect does over-sized holes have on the bearing deformation of an eccentric shear connection.

Appendix

Given: A W 24 X 142 spans a 30 foot column face to column face clear span. Columns are W 14 X 426's. Service loads consist of a 4.5 kip/foot dead load (including the beam) and a 3 kip/foot live load. The design of the connection is required.

Solution:

Beam Properties:

$F_y = 36$ ksi

$F_u = 58$ ksi

$Z_x = 418$ in^3

$M_p = 15{,}048$ kip-in

$d_{BEAM} = 24.74$ in

$B_f = 12.9$ in

$t_f = 1.09$ in

$t_w = 0.65$ in

First design the tee stem to beam flange shear connection. Assume 18 1 inch A490 bolts with two bolts per row in the beam flange to tee stem connection. Use 3d (d = Bolt diameter) center to center bolt spacing and 1.5d end distance. Assume first bolt line 6 inches from the column face. Connection length is then:

$$6'' + 3'' \times 8 + 3'' \times 1.5 = 31.5''$$

The plastic hinge is then assumed to form away from the column face a distance:

$$\frac{24.74''}{2} + 31.5'' = 43.87''$$

If the inflection point is assumed to be at center span then the moment at the column face is:

$$M_{u\text{ Column Face}} = 15{,}048 \text{ kip-in} \times \frac{180''}{180''-43.87''} = 19{,}897 \text{ kip-in}$$

The pulling force on the tee is then:

$$T = \frac{19{,}897\text{kip}-\text{in}}{24.74''} = 804 \text{ kips}$$

Determine the number of shear bolts, where a 1 in A490 bolt with the threads excluded from the shear plane, has a capacity of 45.9 kips (AISC LRFD manual Table: 1-D, Shear):

$$\text{Bolts Required} = \frac{804\text{kips}}{45.9\text{kips}/\text{bolt}} = 17.5 \text{ bolts}$$

Assumption of 18 bolts OK

Check for slip above the service load. First determine service moment:

$$\omega_{service} = 1.6 \text{ x } 3 \text{ kips/ft} + 1.2 \text{ x } 4.5 \text{ kips/ft} = 10.2 \text{ kips/ft}$$

$$M_{service} = \frac{10.2\text{kips}/\text{ft} \times 30'^2}{12} \times 12\text{in}/\text{ft} = 9{,}180 \text{ kip-in}$$

Determine the slip moment using a slip coefficient of 0.33 and the minimum specified bolt tension of 64 kips (AISC LRFD manual Table: J3.1, Minimum Bolt Tension):

$$M_{slip} = 18 \text{ bolts} \times 0.33 \times 64 \text{ kips} \times 24.74'' = 9{,}405 \text{ kip-in}$$

$$M_{slip} > M_{service} \quad \text{OK}$$

Check bearing of beam flange:

$$\text{Reduced Bearing Load} = \frac{804\text{kips}}{18\text{bolts}} - 0.33 \times 64 \text{ kips} = 23.55 \text{ kips}$$

$$\text{Reduced Bearing Stress} = \frac{23.5\text{kips}}{1'' \times 1.09''} = 21.60 \text{ ksi} < 1.0F_u = 58 \text{ ksi} \quad \text{OK}$$

Check Beam Fracture. First calculate the required moment at the bolt line closest to the inflection point:

$$M_{u\,\text{Bolt Line}} = 15{,}048 \text{ kip-in} \times \frac{(180''-30'')}{(180''-43.87'')} = 16{,}581 \text{ kip-in}$$

Compare with the fracture moment of the beam. Calculate the effective flange area:

$$A_{ef} = [12.9'' - 2 \times (1'' + 0.0625'' + 0.0625'')] \times 1.09'' = 11.609 \text{ in}^2$$

Calculate the fracture moment capacity of the flange:

$$M_{Ff} = 11.609 \text{ in}^2 \times 58 \text{ ksi} \times (24.74'' - 1.09'') = 15{,}923 \text{ kip-in}$$

Calculate the yield moment capacity of the web:

$$M_{fw} = [24.74'' - 2 \times 1.09'']^2 \times 0.65'' \times 36 \text{ ksi} \times 0.25 = 2{,}977 \text{ kip-in}$$

$$\phi M_F = 0.9 \times (15{,}923 \text{ kip-in} + 2{,}977 \text{ kip-in}) = 17{,}010 \text{ kip-in}$$

$$\phi M_F = 17{,}010 \text{ kip-in} > 16{,}581 \text{ kip-in} = M_{u\,\text{Bolt Line}} \quad \text{OK}$$

Design the tee stub. Use 8, 1 1/4 inch A490 tension bolts, with a design strength of 103.5 kips (AISC LRFD manual Table: 1-A, Bolts and Rivets, Tension on Gross Area):

$$B = 8 \times 103.5 \text{ kips} = 828 \text{ kips} > 804 \text{ kips OK}$$

Calculate true bolt capacity:

$$B = 8 \times 112.5 \text{ ksi} \times 1.227 \text{ in}^2 = 1104 \text{ kips}$$

Find the minimum stem thickness by checking gross yielding and net fracture. Assume tee is 16 inches wide across column the flange ($B_{f\,\text{Column}}$ = 16.695 in.). Assume A572 grade 50 steel. Check gross yielding:

$$t_{w\,\text{Req'd}} = \frac{804\text{kips}}{16'' \times 50\text{ksi} \times 0.9} = 1.12''$$

The moment at the bolt line closest to the column may be reduced for checking net section Fracture:

$$M_{u\,\text{Bolt Line}} = 15{,}048 \text{ kip-in} \times \frac{(180''-6'')}{(180''-43.87'')} = 19{,}234 \text{ kip-in}$$

$$T_{\text{bolt Line}} = \frac{19{,}234\text{kip} - \text{in}}{24.74''} = 777\text{kips}$$

$$t_{w\ \text{Req'd}} = \frac{777\text{kips}}{[16''-2.25''] \times 65\text{ksi} \times 0.75} = 1.16''$$

A tee with a 31.5 inch long stem and a thickness of 1.16 inches is required. Try a tee cut from an A36, W 40 X 372.

Tee Properties:

$F_y = 50$ ksi

$F_u = 65$ ksi

$t_f = 2.05$ in

$t_w = 1.16$ in

$k = 3.25$ in

$k_1 = 1.75$ in

$B_f = 16.06$ in

Check assumption of 6 inch from column face to first bolt line, where $H_2 = 2$ inches and $C_1 = 1.69$ inches (AISC LRFD manual Table: Threaded Fasteners, Assembling Clearances):

$$2.05'' + 2.0'' + 1.69'' = 5.74'' < 6.0'' \quad \text{OK}$$

Determine minimum bolt space from tee stem, where $C_3 = 1.25$ (AISC LRFD manual Table: Threaded Fasteners, Assembling Clearances):

$$\text{center-to-center of bolts} = (1.75'' + 1.25'') \times 2 = 6.00''$$

Determine design distances:

$$b = \frac{6''-1.16''}{2} = 2.42''$$

$$a = 1.25 \text{ x } 2.42'' = 3.025''$$

$$b' = 2.42'' - \frac{1.25''}{2} = 1.795''$$

$$a' = 3.025'' + \frac{1.25''}{2} = 3.65''$$

Determine the required thickness of the tee flange (using equations from Reference 3).

$$\delta = 1 - \frac{1.3125''}{4''} = 0.672$$

$$T_o = \frac{1104\text{kips}}{1 + \frac{0.672 \times 1.795''}{1.672 \times 3.65''}} = 922 \text{ kips}$$

922 kips < 804 kips (plate failure governs)

$$t_f = \left[\frac{4 \times \frac{804\text{kips}}{8} \times 1.795''}{0.9 \times 4'' \times 50\text{ksi} \times 1.672} \right]^{\frac{1}{2}} = 1.55'' < 2.05'' \text{ OK}$$

Design the shear tab and check the column according to AISC [1] and UBC [12] provisions.

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Vita

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During the summers 1989 through 1991 he was employed by LTM Inc. General Contractors, testing construction materials. During the summers 1992 through 1994 he was employed by LTM Inc. General Contractors, preparing construction estimates and surveying. He received the degree of Bachelor of Science from Oregon State University in March, 1994. In August, 1994, he entered The Graduate School at The University of Texas at Austin.

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