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I, John M. Schroeder, hereby submit this original work as part of the requirements for the degree of Master of Science in Civil Engineering.

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Moment-Rotation Curves for Shear Tab Connections Using Finite Element Modeling and Experimental Data

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Moment-Rotation Curves for Shear Tab Connections
Using Finite Element Modeling and Experimental Data

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School of Advanced Structures
College of Engineering and Applied Science
University of Cincinnati

By

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Abstract
This thesis presents the results of research into using finite element modeling software to create moment-rotation curves for "simple" gravity connections. In current design practice, the rotational stiffness of gravity connections is ignored to ease calculations and due to limited availability of data on the response of gravity connections to rotational demands. Research is presented that shows when the rotational stiffness of these connections is added to the lateral load resisting system, the building response to seismic effects is improved, with smaller associated story drifts and decreases to the moments experienced in the moment frames and in the gravity beams.

In an effort to increase the amount of data on the rotational stiffness of gravity connections available to researchers, this thesis presents the design of two theoretical shear tab connections. The finite element modeling software ABAQUS is used to create moment-rotation curves for these theoretical connections. To demonstrate finite element modeling's ability to accurately predict the response of the theoretical connections, the results of several experiments performed on shear tab connections are reproduced in ABAQUS.

The finite element modeling of the experimental shear tab connections shows that this type of modeling is able to reproduce connection specific events such as bolt slippage, beam-column bearing, and shear tab failure. Additional research is presented indicating finite element modeling's ability to predict net section fracture and block shear rupture of connection components.

Lastly, steps taken to create the finite element models of the theoretical shear tab connections are presented along with the final moment-rotation curves for the connections.
Acknowledgments

I would like to thank all of those who made this thesis a reality. Foremost I would like to thank my advisor and committee chair, Dr. Gian Rassati. Dr. Rassati always made himself available for consultation, guidance, and conversation. Without his instruction and leadership this project would have been orders of magnitude more difficult. I’d also like to thank Dr. James Swanson whose input and observations helped address many issues.

To those at Simpson, Gumpertz, and Heger who taught me the importance of doing more than the minimum amount of work and investigating all possible outcomes.

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# Table of Contents

List of Figures ...................................................................................................... vii  
List of Tables ........................................................................................................ ix  
Notation and Abbreviations ................................................................................. xi  

Chapter 1:  Introduction  
1.1 Idealized Connections ............................................................................................... 1  
1.2 Objective....................................................................................................................... 4  

Chapter 2:  Literature Review  
2.1 Jones, Kirby, and Nethercot (1983) .............................................................................. 5  
2.2 Larson (1996) ............................................................................................................... 7  
2.3 SSRC (1998) ................................................................................................................. 9  
2.4 Liu and Astaneh-Asl (2000) .......................................................................................... 9  
2.5 Astaneh-Asl, Liu, and McMullin (2002) ....................................................................... 12  
2.6 Crocker and Chambers (2004) .................................................................................... 12  
2.7 Barber (2011) .............................................................................................................. 14  
2.8 Ruffley (2011) ............................................................................................................. 16  

Chapter 3:  Methodology  
3.1 Methodology............................................................................................................... 19  

Chapter 4:  Finite Element Modeling: Experimental Connections  
4.1 Creating the Model Parts ............................................................................................ 20  
4.1.1 Dimensions.............................................................................................................. 20  
4.1.2 Partitioning and Meshing.......................................................................................... 23  
4.1.2.1 Columns.............................................................................................................. 24  
4.1.2.2 Beams ............................................................................................................... 25  
4.1.2.3 Shear Tabs ....................................................................................................... 26  
4.1.2.4 Bolts ............................................................................................................... 26
Chapter 5: Finite Element Modeling: Theoretical Connections

5.1 Connection Loading ................................................................. 48
5.2 Connection Design ................................................................. 49
  5.2.1 Beam Loading and Design .................................................. 50
  5.2.2 Column Loading and Design .............................................. 51
  5.2.3 Shear Tab and Bolt Loading and Design ................................ 51
5.3 Creating the Model Parts .......................................................... 53
  5.3.1 Dimensions ................................................................. 53
  5.3.2 Partitioning and Meshing ................................................... 54
    5.3.2.1 Columns ............................................................. 55
    5.3.2.2 Beams ............................................................... 55
    5.3.2.3 Shear Tabs .......................................................... 56
    5.3.2.4 Bolts ................................................................. 56
  5.3.3 Material Properties ........................................................... 56
5.4 Assembling the Connection ....................................................... 57
5.5 Bolt Pretensioning and Connection Loading ................................. 60
5.6 Results .................................................................................. 61
  5.6.1 Theoretical Connection 1 .................................................. 61
  5.6.2 Theoretical Connection 2 .................................................. 63
  5.6.3 Theoretical Model Summary and Findings .............................. 66
Chapter 6: Conclusions

6.1 Conclusions ........................................................................................................................ 67

6.2 Suggestions for Future Research ........................................................................................ 69

References .................................................................................................................................. 71

Appendix A .................................................................................................................................. 73

Appendix B .................................................................................................................................. 79

Appendix C .................................................................................................................................. 84

Appendix D .................................................................................................................................. 104
List of Figures

Figure 1-1: Generic Moment Rotation Curves................................................................. 2
Figure 1-2: AISC Moment-Rotation Curve................................................................. 3

Figure 2-1: Larson Test Set-up Elevation ................................................................. 8
Figure 2-2: Larson Specimen 1 ............................................................................. 8
Figure 2-3: Force-Displacement Curve for Larson Specimen 1 .............................. 8
Figure 2-4: Liu and Astaneh-Asl's Test Set-up Isometric ........................................... 10
Figure 2-5: Liu and Astaneh-Asl's Test Set-up Elevation ......................................... 10
Figure 2-6: Simplified Moment-Rotation Curve ....................................................... 13
Figure 2-7: ETABS' Hinge Parameters .................................................................. 14
Figure 2-8: Sensitivity Analysis .............................................................................. 17

Figure 4-1: FEM Bolt Cross-Section ....................................................................... 22
Figure 4-2: Column Partitions ............................................................................... 24
Figure 4-3: Column Mesh ...................................................................................... 24
Figure 4-4: Beam Partitions ................................................................................... 25
Figure 4-5: Beam Mesh ......................................................................................... 25
Figure 4-6: Shear Tab Partitions ............................................................................ 26
Figure 4-7: Shear Tab Mesh .................................................................................. 26
Figure 4-8: Bolt Partitions ....................................................................................... 27
Figure 4-9: Bolt Mesh ............................................................................................. 27
Figure 4-10: Larson Specimen 1 Model ................................................................. 30
Figure 4-11: Liu and Astaneh-Asl Specimen 1A Model ........................................... 30
Figure 4-12: Liu and Astaneh-Asl Specimen 2A Model ........................................... 30
Figure 4-13: Crocker and Chambers Specimen I Model .............................................................. 30
Figure 4-14: Crocker and Chambers Specimen II Model .......................................................... 30
Figure 4-15: Crocker and Chambers Specimen III Model ........................................................ 30
Figure 4-16: Larson Specimen 1 Model Results ................................................................. 35
Figure 4-17: Larson Specimen 1 Initial Stiffness ................................................................. 35
Figure 4-18: FEM Specimen 1 Stress Contours ................................................................. 35
Figure 4-19: Liu and Astaneh-Asl Specimen 1A Model Results .............................................. 36
Figure 4-20: Liu and Astaneh-Asl Specimen 1A Initial Stiffness ............................................ 36
Figure 4-21: FEM Specimen 1A Stress Contours .............................................................. 37
Figure 4-22: Liu and Astaneh-Asl Specimen 2A Model Results .............................................. 38
Figure 4-23: Liu and Astaneh-Asl Specimen 2A Close Up .................................................. 38
Figure 4-24: Liu and Astaneh-Asl Specimen 2A Initial Stiffness ............................................ 39
Figure 4-25: FEM Specimen 2A Stress Contours ............................................................... 40
Figure 4-26: Crocker and Chambers Specimen I Model Results .......................................... 41
Figure 4-27: Crocker and Chambers Specimen I Initial Stiffness .......................................... 41
Figure 4-28: FEM Specimen I Stress Contours ................................................................. 42
Figure 4-29: Crocker and Chambers Specimen II Model Results .......................................... 43
Figure 4-30: Crocker and Chambers Specimen II Initial Stiffness ........................................ 43
Figure 4-31: FEM Specimen II Stress Contours ................................................................. 44
Figure 4-32: Crocker and Chambers Specimen III Model Results ......................................... 45
Figure 4-33: Crocker and Chambers Specimen III Initial Stiffness ........................................ 45
Figure 4-34: FEM Specimen III Stress Contours ............................................................... 46
Figure 4-35: FEM Specimen III Bolt Stress Contours .......................................................... 46
Figure 5-1: Theoretical Connection 1 Model ................................................................. 58
Figure 5-2: Theoretical Connection 2 Model ................................................................. 58
Figure 5-3: Theoretical Connection 1 Model Results ...................................................... 62
Figure 5-4: Theoretical Connection 1 Close Up ............................................................. 62
Figure 5-5: Theoretical Connection 1 Initial Stiffness .................................................... 62
Figure 5-6: Theoretical Connection 1 Stress Contours .................................................. 63
Figure 5-7: Theoretical Connection 1 Bolt Stress Contours ......................................... 63
Figure 5-8: Theoretical Connection 2 Model Results .................................................... 64
Figure 5-9: Theoretical Connection 2 Close Up ............................................................. 64
Figure 5-10: Theoretical Connection 2 Initial Stiffness ............................................... 64
Figure 5-11: Theoretical Connection 2 Stress Contours .............................................. 65
Figure 5-12: Theoretical Connection 2 Bolt Stress Contours ...................................... 65
# List of Tables

Table 2-1: Gravity Connection Experiment Summary ................................................................. 6
Table 2-2: Larson Test Specimens ............................................................................................... 7
Table 2-3: Liu and Astaneh-Asl Test Specimens ........................................................................ 11
Table 2-4: Crocker and Chambers Test Specimens .................................................................... 13

Table 4-1: Experimental Models' Component Size Summary ......................................................... 21
Table 4-2: Experimental Models' Component Dimensions .......................................................... 21
Table 4-3: Experimental Models' Bolt Dimensions ..................................................................... 23
Table 4-4: Experimental Models' Material Yield Strength .......................................................... 28
Table 4-5: Specimens 1, 1A, and 2A Model Boundary Conditions ............................................. 31
Table 4-6: Specimens I, II, and III Model Boundary Conditions ................................................. 31
Table 4-7: Experimental Models' Bolt Pretension Temperatures ................................................. 33
Table 4-8: Experimental Models' Beam Tip Displacement .......................................................... 33

Table 5-1: Theoretical Connections' Live Loadings ..................................................................... 49
Table 5-2: Theoretical Connections' Dead Loading ................................................................. 49
Table 5-3: Theoretical Connections' Beam Loading ................................................................. 50
Table 5-4: Theoretical Connections' Beam Capacities .............................................................. 50
Table 5-5: Theoretical Connections' Column Loading and Capacity ........................................... 51
Table 5-6: Theoretical Connections' Loading and Capacities ...................................................... 52
Table 5-7: Theoretical Models' Component Size Summary ......................................................... 53
Table 5-8: Theoretical Models' Component Dimensions .......................................................... 53
Table 5-9: Theoretical Models' Bolt Dimensions ..................................................................... 54
Table 5-10: Theoretical Models' Material Yield Strength .......................................................... 57
Table 5-11: Theoretical Models' Boundary Conditions ................................................................. 59
Table 5-12: Theoretical Models' Bolt Pretension Temperatures .................................................. 60
Table 5-13: Theoretical Models' Beam Tip Displacement .......................................................... 60
Notation and Abbreviations

A:   Cross sectional area of the beam
bf:  Width of flange
CoF: Coefficient of friction
d:   Depth of the beam section
DoF: Degree of freedom
FEM: Finite element modeling
h:   Height of the beam web
I_x: Moment of inertia about the x-axis
L:   Length of the beam
M_p: Plastic moment of the beam
PR:  Partially restrained
r_x: Radius of gyration about x-axis
R_y: Ratio of expected yield stress to specified minimum yield stress
S_x: Elastic section modulus about the x-axis
t_f: Thickness of flange
t_w: Thickness of web
Z_x: Plastic section modulus about the x-axis
Chapter 1: Introduction

1.1 Idealized Connections

When designing beams in current design practice, assumptions are often made about the types of connections that will be used in the building when it is constructed. These connections are assumed to be either fully restrained, also known as fixed, where there is no relative rotation between the connected elements, or pinned, also known as simple, where there is unrestricted rotation between the connected elements. For the majority of gravity beams in buildings the assumption is that their connections will be simple, while beams in the moment frame of the building’s lateral load resisting system are assumed to be fixed.

However these assumptions are based on extremely idealized conditions that do not occur in reality. For example, one of the most commonly used gravity connections in steel structures is the shear tab. In this connection, a steel plate is shop welded to the column face, then, during construction, the beam is bolted to the plate. In theory, the plate deforms and allows beam rotation with no moment transferred between the connected members. In reality a complex loading occurs through the bolts and the plate is forced to bend to accommodate the rotation of the beam, with some amount of moment transfer.

These idealizations and simplifications for the connections have consequences on the internal load experienced not only by the connected elements but also for other members in the building. Stiff connections, such as fixed connections, generally cause larger moments in the beam near its connections, while connections with little or no stiffness, such as pinned connections, create larger moments along the length of the beam. Therefore, the connection type or stiffness
determines the amount of moment transferred among members as well as affects the maximum moment experienced by the beams.

More importantly, when all connections in gravity beams are assumed to be pinned, any contributions of the connections to the lateral load resisting system are ignored. Therefore it would be beneficial to account for the rotational stiffness of these gravity connections. However this requires that the assumptions of pinned or fixed connections be replaced with the assumption of partially restrained (PR) connections. Figure 1-1 displays several generic moment rotation curves and shows the difference between fixed, PR, and pinned connections. In this figure the vertical axis is the moment transferred between the connected members while the horizontal axis is the relative rotation between the elements. Line 1, the horizontal axis, represents a pinned connection with no moment transfer as the members rotate freely between themselves. Line 2, the vertical axis, shows a fixed connection where no relative rotation occurs with unlimited moment transfer. Lastly, lines 3 through 5 show PR connections of varying stiffness. The shape of each line represents how that PR connection responds to different rotational demands.

As discussed earlier, the idealized conditions of fixed and pinned connections do not occur in reality and therefore all connections are actually partially restrained. However to simplify
design, AISC breaks moment-rotation curves into three areas observed in Figure 1-2. Connections above a stiffness of 20 EI/L are assumed to be fixed, where I and L refer to the connecting beam. For connections below 2 EI/L "it is acceptable to consider the connection to be simple (in other words, rotates without developing moment)" (2005a).

So if we begin to look at gravity connections, specifically shear tab connections, as partially restrained instead of perfectly pinned, we can begin to add their rotational stiffness to that of the lateral load resisting system. Additionally, if we are to account for the rotational stiffness of shear tab connections in the design of buildings, a way to predict the moment-rotation behavior of various connections must be developed. To establish equations to predict this moment-rotation behavior, experimental data must be analyzed. Unfortunately, few experiments have been performed on shear tab connections and as a result not much data is available on connections with varying member sizes or configurations.
1.2 Objective

Therefore, the goal of this thesis is to create moment-rotation curves that predict the response of theoretical shear tab connections. Specifically, this thesis uses a finite element analysis program to model shear tab connections with practical beam and columns sizes so that moment-rotation curves can be created for the connections. To ensure that the predictions of the finite element models are accurate, the results of several experimental tests performed by other researchers will be reproduced by the program.

This thesis will show that finite element modeling can reproduce the response of full-scale tests on shear tab connections and therefore it can accurately predict the response of theoretical shear tab connections. This will allow future researchers the opportunity to test theories about shear tab connections before moving into expensive and time-consuming experimental tests. Additionally, practicing engineers will be able to use the partially-restrained assumption for shear tab connections to add the rotational stiffness of the gravity connections to the lateral load resisting system, making the lateral load resisting system lighter and reducing the overall cost of the structure.
Chapter 2: Literature Review

The following sections will summarize the various technical papers reviewed in the creation of this thesis. An attempt will be made to convey the important findings of each paper without going into exhaustive detail about the paper's specifics. The review is ordered chronologically.

2.1 Jones, Kirby, and Nethercot (1983)

In their paper, Jones, Kirby, and Nethercot reviewed the benefits that come from incorporating PR connections into the design of structures. They discussed how including PR connections in design will reduce beam moments and provide for slimmer and less costly designs. The slimmer designs of these beams could result in savings of up to 20% over conventional designs (Jones, Kirby, and Nethercot, 1983).

They also presented a summary of several methods for integrating PR connections into the design of structures. These methods involved the use of already established procedures for analysis of indeterminate frames and modifying them to allow for the inclusion of PR connections. The three analysis procedures discussed in the paper are the slope-deflection method, moment-distribution method, and matrix stiffness method. In all three methods the general analysis procedure remains the same. However in the slope-deflection method and moment-distribution method several factors are altered to reflect the PR connections. At the same time, in the matrix stiffness method the initial member stiffness matrices are amended through the use of a correction matrix to incorporate the PR connections. All three procedures use a common factor known as the semi-rigid connection factor $Z$ in their adjustments. This results in the same limitation being present in all three procedures. The semi-rigid connection factor assumes a linear relationship between moment and rotation, even though "experiments had
clearly shown that moment-rotation curves were non-linear over the whole range for most types of connections" (Jones, Kirby, and Nethercot, 1983).

Jones, Kirby, and Nethercot also summarized the research that has occurred on simple and PR connections up until that point in time. The majority of research at that time went into web angle, end plate, welded top plate and seat, and T-stub connections. Table 2-1 is a portion of the summary provided by Jones, Kirby, and Nethercot, specifically listing the experiments that related to gravity connections.

### Table 2-1: Gravity Connection Experiment Summary
(Jones, Kirby, and Nethercot, 1983)

<table>
<thead>
<tr>
<th>Type of Connection</th>
<th>Experiment Date</th>
<th>Country of Origin</th>
<th>No. of Tests</th>
<th>Type of Fastener</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Web Cleats</td>
<td>1942</td>
<td>U.S.A.</td>
<td>4</td>
<td>Welded</td>
</tr>
<tr>
<td></td>
<td>1967</td>
<td>Canada</td>
<td>4</td>
<td>Bolted</td>
</tr>
<tr>
<td></td>
<td>1968</td>
<td>Canada</td>
<td>8</td>
<td>Bolted</td>
</tr>
<tr>
<td></td>
<td>1977</td>
<td>Canada</td>
<td>43</td>
<td>Bolted</td>
</tr>
<tr>
<td>Double Web Cleats</td>
<td>1934</td>
<td>UK</td>
<td>3</td>
<td>Rivets</td>
</tr>
<tr>
<td></td>
<td>1936</td>
<td>U.S.A.</td>
<td>7</td>
<td>Rivets</td>
</tr>
<tr>
<td></td>
<td>1959</td>
<td>U.S.A.</td>
<td>4</td>
<td>Rivets/Bolts</td>
</tr>
<tr>
<td></td>
<td>1959</td>
<td>Canada</td>
<td>4</td>
<td>Bolts</td>
</tr>
<tr>
<td></td>
<td>1961</td>
<td>Canada</td>
<td>4</td>
<td>Bolts</td>
</tr>
<tr>
<td></td>
<td>1961</td>
<td>Canada</td>
<td>6</td>
<td>Bolts</td>
</tr>
<tr>
<td></td>
<td>1969</td>
<td>U.S.A.</td>
<td>9</td>
<td>Rivets/Bolts</td>
</tr>
<tr>
<td></td>
<td>1977</td>
<td>U.S.A.</td>
<td>3</td>
<td>Bolts</td>
</tr>
<tr>
<td>Header Plate</td>
<td>1967</td>
<td>Canada</td>
<td>16</td>
<td>Bolts</td>
</tr>
</tbody>
</table>

Lastly, they present different approaches for modeling PR connections. The different procedures discussed in the paper included linear methods as well as polynomial curve and B-spline curve fitting methods. The linear methods involved decreasing the beam stiffness near the connection or using rotational springs to represent the connection. The polynomial curve-fitting method required fitting non-dimensional polynomial series to standardized moment-rotation curves.
Further work in this method has produced moment-rotation functions for web angle, header plate, end plate, T-stub, and top and seat angle connections. This procedure also uses a size effect factor, C, to account for various dimensions in the geometry of the connection. The B-spline curve fitting method “requires the division of the range of connection rotations into a finite number of smaller ranges, then within each of these ranges a cubic function is fitted in turn, but first and second derivative continuity is maintained between ranges” (Jones, Kirby, and Nethercot, 1983). Of the three procedures, the B-spline curve fitting method is considered the most accurate, producing generally smooth lines with a numerical description of the moment rotation curve.

2.2 Larson (1996)

The focus of Larson’s thesis was the experimental testing of several top and bottom flange T-stub connections. The objective of Larson’s research included obtaining data on the performance of T-stub connections under cyclic loading. After the Northridge earthquake of the early 1990s, many fully welded connections experienced brittle failure of the welds, which required a need for further understanding of how non-welded connections perform under such loadings.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Connection Type</th>
<th>Column Size</th>
<th>Beam Size</th>
<th>T-Stub Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Simple</td>
<td>W14x426</td>
<td>W36x150</td>
<td>NA</td>
</tr>
<tr>
<td>2</td>
<td>T-Stub</td>
<td>W14x426</td>
<td>W36x150</td>
<td>Cut from W36x150</td>
</tr>
<tr>
<td>3</td>
<td>T-Stub</td>
<td>W14x426</td>
<td>W36x150</td>
<td>Cut from W36x150</td>
</tr>
<tr>
<td>4</td>
<td>T-Stub</td>
<td>W14x426</td>
<td>W36x150</td>
<td>WT20x186</td>
</tr>
<tr>
<td>5</td>
<td>T-Stub</td>
<td>W14x426</td>
<td>W36x150</td>
<td>WT20x186</td>
</tr>
</tbody>
</table>

Larson tested a total of five connections. Of the five connections tested, four were T-stub connections. He also tested one shear tab connection to “determine the moment-rotation
characteristics of the web alone” (Larson, 1996). A list of all of Larson’s test specimens can be seen in Table 2-2. The shear tab connected the flange of the column to the web of the beam by eight 1 inch diameter A325 bolts, see Figure 2-2. In his testing Larson used W36x150 beams and W14x426 columns for all connections. Unlike the T-stub specimens, the shear tab connection was only loaded for one cycle and was not tested to failure. A portion of the resulting force-displacement curve of Specimen 1 was reproduced from Larson’s paper and can be seen in Figure 2-3.

Figure 2-1: Larson Test Set-up Elevation
(Larson, 1996)

Figure 2-2: Larson Specimen 1
(Larson, 1996)

Figure 2-3: Force-Displacement Curve for Larson Specimen 1
(Larson, 1996)
2.3 SSRC (1998)
This technical memorandum summarizes the different meanings of the term yield stress and highlights the meaning of the term static yield stress. ASCE E6 defines yield strength as "the stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain" (SSRC, 1998). The common understanding of yield strength is what the paper clarifies as the dynamic yield stress. However the dynamic yield strength is affected by a number of factors that are specific to the test specimen and machine, the largest factor being the strain rate of the machine during testing. Larger strain rates lead to higher values of dynamic yield stress.

Static yield stress is defined as the "average stress during actual yielding in the plastic range at zero strain rate" (SSRC, 1998). The paper then summarizes the testing method which involves loading a test sample to yielding, then placing the testing machine's strain rate at zero. After five minutes, a recording of the load on the specimen is taken. This process is repeated three times. Testing in this manor means that the static yield stress is independent of the strain rate, the size of the specimen and the testing machine (SSRC, 1998). A relationship between the dynamic and static yield stresses is then presented for some grades of steel. The relationship is expressed through the dynamic yield stress ratio which is a ratio of dynamic to static yield stresses. For A36, A441, and A514 grades of steel, the average dynamic yield stress ratios are 1.126, 1.070, and 1.040 respectively.

2.4 Liu and Astaneh-Asl (2000)
Liu and Astaneh-Asl’s research attempted to gain an understanding of the lateral resistance of simple connections after events like the Northridge earthquake suggested that these connections added to the lateral stability of buildings even though they were not designed for these loadings.
They performed eight full-scale tests on shear tab connections. The connections were configured with two beams connected to one column, see Figure 2-4 and Figure 2-5.

A W14x90 column made of A572 Gr. 50 steel was used in all tests. The majority of specimens included a concrete floor slab supported by the beams in the connection. This slab was cast on top of a 20 gauge metal deck with 3 in. ribs and with welded wire fabric as the primary reinforcing. Liu and Astaneh-Asl constructed two specimens, 1A and 2A, without a floor slab as a control, see Table 2-3. All beams were also made of A572 Gr. 50 steel while the plates and angles were made of A36 steel.

The researchers applied the lateral load via a drift angle measured as the lateral displacement at the top of the column divided by the distance between the top and bottom of the supporting column. To simulate the gravity load seen in the theoretical building Liu and Astaneh-Asl added actuators, located 5 ft. 6 in. away from the centerline of the column, to the system to provide additional vertical loading on the connections. The experiments used a combined dead and live load total of 100 psf for the gravity loading (Liu and Astaneh-Asl, 2000). The resulting moments
in the connections from the applied gravity loads were 20-25% of the maximum moment the connections experienced during testing (Liu and Astaneh-Asl, 2000).

Table 2-3: Liu and Astaneh-Asl Test Specimens (Liu and Astaneh-Asl, 2000)

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>Beam</th>
<th>Shear tab dimensions (mm)</th>
<th>Bolts on web</th>
<th>Seat Angle (mm)</th>
<th>Floor Slab</th>
<th>Slab reinforcement</th>
<th>Concrete in column web</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>W18x35</td>
<td>305x292x9.5</td>
<td>4-22 mm</td>
<td>None</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A325</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2A</td>
<td>W24x55</td>
<td>457x114x9.5</td>
<td>6-22 mm</td>
<td>None</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A325</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3A</td>
<td>W18x35</td>
<td>305x292x9.5</td>
<td>4-22 mm</td>
<td>None</td>
<td>Yes</td>
<td>Wire mesh + nominal reinforcement</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A325</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4A</td>
<td>W18x35</td>
<td>305x292x9.5</td>
<td>4-22 mm</td>
<td>None</td>
<td>Yes</td>
<td>D16 (No. 5) reinforcing bar</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A325</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5A</td>
<td>W18x35</td>
<td>None</td>
<td>None</td>
<td>Stiffened</td>
<td>Yes</td>
<td>Wire mesh + nominal reinforcement</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6A</td>
<td>W24x55</td>
<td>457x114x9.5</td>
<td>4-22 mm</td>
<td>None</td>
<td>Yes</td>
<td>Wire mesh + nominal reinforcement</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A325</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7A</td>
<td>W24x55</td>
<td>457x114x9.5</td>
<td>4-22 mm</td>
<td>None</td>
<td>Yes</td>
<td>Wire mesh + nominal reinforcement</td>
<td>None</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A325</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8A</td>
<td>W24x55</td>
<td>457x114x9.5</td>
<td>4-22 mm</td>
<td>203x102x19</td>
<td>Yes</td>
<td>Wire mesh + nominal reinforcement</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A325</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Testing of the Specimens 1A and 2A showed that the connections experienced 15 to 20% of the beams moment capacity before failure (Liu and Astaneh-Asl, 2000). Failure of both specimens was similar with shear tab yielding, beam and column flange contact, and fracture of the shear tab. The addition of the floor slab almost doubled the maximum moment capacities of the simple connections while not lowering the rotational capacities of the connections. This was due to loss of composite action with the concrete above rotations of 0.04 radians. The connections failed at approximately 30% and 60% of the W18x35 and W24x55 beam capacities respectively.
(Liu and Astaneh-Asl, 2000). After loss of composite action the connections performed very similarly to the simple connections without the floor slabs present.

2.5 Astaneh-Asl, Liu, and McMullin (2002)

This paper is a summary of 2 sets of research performed at the University of California, Berkley. The first set of research concerned the behavior of shear tabs under gravity loadings. These tests attempted to identify the relationship between shear, bending moment, and rotation of the connection as well as defining limit states and the effects of geometry and material properties. The researchers performed tests on 15 different connections, with varying number of bolts, bolt types, hole types, plate dimensions, steel strength, weld size, and supporting element. While testing, they found most connections performed similarly, with more flexibility at higher rotations and more rigid behavior at lower rotations. The testing showed that the connections are not perfect pins and do develop moments that lower the load carrying capacity of the connection. While developing a design philosophy for shear connections, they intended for shear failure of the connection to occur at the same time as the beam plastic failure and that ductile failure modes (plate yielding and bolt bearing) would occur before brittle failure modes (weld, plate, or bolt fracture). The second set of research contained a summary of Liu and Astaneh-Asl experiments in the 2000 paper.

2.6 Crocker and Chambers (2004)

In their paper, Crocker and Chambers summarized the results of full-scale shear tab connection experiments they performed in the early 2000s. They tested three shear tab connections cyclically up to rotations of 0.06 radians, see Table 2-4. Test Specimens I, II, and III had plate depths of 9, 12, and 18 inches, with 3, 4, and 6 bolts respectively (Crocker and Chambers, 2004). Their test set-up involved a column with fixed end supports connected through the shear tab to a
beam supported by an actuator at the far end. During the testing, Specimen III's top and bottom most bolts experienced shear failure at a beam rotation of 0.04 radians followed by the next top bolt experiencing shear failure at a rotation of 0.05 radians (Crocker and Chambers, 2004). In Specimens I and II, the researchers observed no shear failure in any of the bolts.

Table 2-4: Crocker and Chambers Test Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Column</th>
<th>Beam</th>
<th>Shear Tab</th>
<th>Bolt dia. (in)</th>
<th>Bolt Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>W1x4398</td>
<td>W18x55</td>
<td>9x4 1/4x3/8</td>
<td>(3) 3/4</td>
<td>A325</td>
</tr>
<tr>
<td>II</td>
<td>W18x55</td>
<td></td>
<td>12x4 1/4x3/8</td>
<td>(4) 3/4</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>W24x84</td>
<td></td>
<td>18x4 1/4x3/8</td>
<td>(6) 3/4</td>
<td></td>
</tr>
</tbody>
</table>

Analysis of the moment rotation curves created from the experiments led Crocker and Chambers to the conclusion that the number of bolts in the connection determines the connections stiffness (2004). From examining Specimen III they observed that after the first two bolts failed, Specimen III had the same stiffness as Specimen II, see Figure 2-6. In addition, when the third bolt failed, the stiffness of Specimen III became the same as Specimen I.

![Figure 2-6: Simplified Moment-Rotation Curve](Image)

From the testing, Crocker and Chambers found that the performance for shear tab connections "is dependent upon the magnitude of the rotation demand, connection size, and connection
detailing" (2004). Crocker and Chambers also created several equations for predicting bolt deformations given a specific design rotation of the connection.

2.7 Barber (2011)

This paper discusses analysis performed on three different experimental connections in an attempt to quantify the lateral stiffness contribution of shear tab connections. Barber used the structural analysis program ETABS to create computer models of the shear tab connections tested in Liu and Astaneh-Asl (Specimens 1A and 2A) and Larson (Specimen 1). She then took default ETABS definitions for moment-rotation curves of connections, based on FEMA hinge properties, and altered the default values so that the resulting moment-rotation curves matched the data seen in the original experiments. Figure 2-7 illustrates the five points through which the ETABS hinge parameters relate moment and rotation of a connection.

A=Origin
B=Yield point
C=Ultimate pushover capacity
D=Residual pushover strength
E=Total failure

Figure 2-7: ETABS' Hinge Parameters
(Barber, 2011)

From these altered hinge parameters, Barber created equations to calculate moment and hinge parameters for other beam and column connections, the equations can be seen below. She specifically created these equations to only use properties of the beam used in the connection.
With the moment-rotation parameters of the connection defined, the connection could now be classified as a PR connection rather than the original title of pinned connection.

<table>
<thead>
<tr>
<th>Point</th>
<th>Moment Parameter</th>
<th>Rotation Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$0$</td>
<td>$0$</td>
</tr>
<tr>
<td>B</td>
<td>$\frac{M_p L^2}{\left(\frac{h}{t_w}\right)^2} \leq 0.06 M_p$</td>
<td>$0$</td>
</tr>
<tr>
<td>C</td>
<td>$\frac{M_p A^2 r_x t_w}{S_x I_x} \left(\frac{2 L t_w}{b_f}\right)^2 \leq 0.17 M_p$</td>
<td>$t_w \frac{t_f}{t_r} \left(\frac{d}{2}\right)^d$</td>
</tr>
<tr>
<td>D</td>
<td>$\frac{M_p Z_x d b_f}{2 t_f d \left(\frac{t_r + t_f}{t_w}\right)} \leq 0.14 M_p$</td>
<td>$t_w \left(\frac{d}{2}\right) \left(\frac{r_x}{Z_x}\right)^{0.25} \left(\frac{r_x t_w b_f}{2 S_x h t_f}\right)^2$</td>
</tr>
<tr>
<td>E</td>
<td>$\frac{M_p d}{2 d \left(\frac{S_x}{I_x}\right)\left(\frac{t_r + t_f}{t_w}\right)} \leq 0.26 M_p$</td>
<td>$b_f^2 r_x t_w^2 \left(\frac{S_x}{I_x}\right)\left(\frac{d}{r_x}\right) \leq 1.75 \text{Rotation}_{point \ C}$</td>
</tr>
</tbody>
</table>

Barber then analyzed three theoretical buildings to view the effect of including the rotational stiffness of the PR shear tab connections. The three buildings she examined varied in the number of stories, from three to eight, but the buildings mostly used moment frame lateral load resisting systems. The ratio of non-rigid to rigid connections in the buildings also varied between one and four. Barber analyzed several models of each building in ETABS. As a control, for each building Barber ran one model with gravity and lateral loads while using traditional pinned gravity connections. On subsequent models, Barber applied lateral loads with the gravity connections using the new PR connection parameters and performed displacement and force based pushover analyses. From these models, Barber concluded that by accounting for the rotational stiffness of the gravity connections, the buildings, as a whole, could experience a 24% minimum increase to the base shear and a 10% minimum increase to base shear (2011).
Alternately, accounting for the rotational stiffness of the gravity connections decreased the average story drift by a minimum of 22% (2011). Additionally, the maximum moment experienced by the gravity beam and the beams of the lateral load resisting decreased by 6% and 22% respectively (Barber, 2011).

2.8 Ruffley (2011)

This paper presents the results of work done to recreate experimental tests performed on top and seat angle connections in a finite element modeling program. Ruffley used the finite element modeling program ABAQUS to recreate experimental top-and-seat angle connections and several component tests. The top-and-seat angle connection tests included two tests performed by Schrauben (1999). The component tests included two T-stub tests performed by Swanson and a bolt shear test by Moore (1999 and 2007).

Before starting on creation of the connection and component finite element models, Ruffley performed a sensitivity analysis to determine the type and size of the finite elements required in his modeling. He compared 8-node and 20-node elements with both fine and course meshes on a cantilevered beam. Eight-node elements are generally not desirable for bending due to "a phenomena called shear locking which causes unrealistic increases in stiffness" (Ruffley, 2011). However the 8-node elements utilized in the finite element analysis program use reduced integration that eases the effect of the shear locking phenomenon. Figure 2-8 compares the end load vs. beam tip displacement results of those four sensitivity analysis models. From these results Ruffley determined "a coarse meshed, 8-node-element schemed beam behaves identically to a fine meshed, 20-node-element schemed beam" (2011). Therefore for his connection and component models, Ruffley used 20-node elements in areas expected to experience large
inelastic deformations and 8-node elements in all other areas including the bolts of the connections.

In the analysis of the component tests, the finite element analysis program correctly reproduced the results of experiments that produced net section fracture in a T-stub as well as block shear failure in the second T-stub. The finite element models did not use fracture mechanics, so confirmation of these limit states came from analysis of the stress contours of the components being tested and engineering judgment.

Analysis of the first top-and-seat angle connection showed that the modeling could correctly reproduce several of the observations made during the experimental testing. During testing, the original researchers observed top angle prying, beam web crippling, and top tension bolt failure. The finite element model confirmed all of these observations. Additionally, the beam end load

Figure 2-8: Sensitivity Analysis  
(Ruffley, 2011)
required in the model to reach the experiment's beam tip displacement was within 1.7% of the value seen in the experiment (Ruffley, 2011).

Many of the same observations made by the researchers for the first top-and-seat angle experiment also occurred during the second top-and-seat angle connection test. In addition to the original observations, the researcher's comments included notable yielding of the k-line on the shear leg of the top angle and eventual plastic hinge development in the shear leg of the top angle during the second top-and-seat angle connection test. Similar to the finite element model of the first connection, the model for the second top-and-seat angle connection test reproduced all of these observations.
Chapter 3: Methodology

3.1 Methodology

To complete the objectives of this thesis several finite element models were created and compared with the results of actual experiments from other researchers. Once the models showed that they could accurately predict the results observed in the experimentation, additional shear tab connections were designed to use practical beam, column, and shear tab sizes. These theoretical connections were also modeled using finite element software. From the results of the finite element modeling of the designed connections, moment-rotation curves for the theoretical connections have been created.

The experimental connections modeled were the shear tab connection tested by Larson, Specimen 1, the two bare steel connections tested by Liu and Astaneh-Asl, Specimens 1A and 2A, and the three connections tested by Crocker and Chambers, Specimens I, II, and III. The computer program used for the finite element modeling was ABAQUS version 6.10-2 released on August 12, 2010. This thesis used ABAQUS due to the wide variety of options available, versatility of the program to model specific conditions, and the promising results observed by Ruffley. The loadings and load combinations for the theoretical connections followed ASCE 7-10 (2010). Additionally the connections were designed according to AISC's Steel Construction Manual and Specifications for Structural Steel Buildings to ensure practical design and sizes of members (2005b and a).
Chapter 4: Finite Element Modeling: Experimental Connections

Before the analysis of the finite element models can begin of the finite element models, several steps must be taken to ensure that the analysis runs properly. Additionally, the conditions of the original experiments must be reproduced as closely as possible. These steps are taken to ensure that the models provide accurate results. The finite element modeling program, ABAQUS, requires that all of the pieces of the connection model such as bolts, beams, and columns be created separately and then assembled together. Once the pieces are created and then assembled, constraints and boundary conditions must be assigned to tell ABAQUS how the parts will interact with each other and which degrees of freedom (DoF) are available for each piece. Lastly displacements or loads can be applied to the assembled connection and the analysis can be performed. The following sections will describe each of these steps.

4.1 Creating the Model Parts

4.1.1 Dimensions

The first step in reproducing the connections in ABAQUS is to recreate the components used in the experiment. All of the researchers reported the sizes of the columns and beams used in their connections. Consulting AISC, the dimensions of the various sizes can be obtained (2005b). Table 4-1 summarizes the component sizes used in each of the experimental connections. Table 4-2 lists the various dimensions used in the modeling of each specimen.

In their papers most of the researchers reported the dimensions of the shear tabs used in the connections. However in his description of the shear tab connection, Larson did not mention the size of the shear tab used. To gain sufficient information on the dimensions of Larson's shear tab connection, research was performed on Larson's advisor's previous work. Larson's advisor had
worked on welded column connections using the same column and beam sizes as those used in Larson's experiments (Englehardt, 1994). Therefore it was hypothesized that the shear tabs used in those experiments were the same size as the shear tabs used in Larson's experiments.

Table 4-1: Experimental Models' Component Size Summary

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Column</th>
<th>Beam</th>
<th>Shear Tab</th>
<th>Bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W14x426</td>
<td>W36x150</td>
<td>25&quot;x5&quot;x5/8&quot;</td>
<td>(8) 1&quot; Dia.</td>
</tr>
<tr>
<td>1A</td>
<td>W14x90</td>
<td>W18x35</td>
<td>12&quot;x11 1/2&quot;x3/8&quot;</td>
<td>(4) 7/8&quot; Dia.</td>
</tr>
<tr>
<td>2A</td>
<td>W24x55</td>
<td>18&quot;x4 1/5&quot;x3/8&quot;</td>
<td>(6) 7/8&quot; Dia.</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>W14x398</td>
<td>W18x55</td>
<td>9&quot;x4 1/4&quot;x3/8&quot;</td>
<td>(4) 3/4&quot; Dia.</td>
</tr>
<tr>
<td>II</td>
<td>W14x398</td>
<td>12&quot;x4 1/4&quot;x3/8&quot;</td>
<td>(6) 3/4&quot; Dia.</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>W24x84</td>
<td>18&quot;x4 1/4&quot;x3/8&quot;</td>
<td>(6) 3/4&quot; Dia.</td>
<td></td>
</tr>
</tbody>
</table>

Table 4-2: Experimental Models' Component Dimensions

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Column Height (in.)</th>
<th>Column Depth (in.)</th>
<th>Column Web Thick. (in.)</th>
<th>Column Flange Thick. (in.)</th>
<th>Column Length (in.)</th>
<th>Column Depth (in.)</th>
<th>Column Web Thick. (in.)</th>
<th>Column Flange Thick. (in.)</th>
<th>Beam Length (in.)</th>
<th>Beam Depth (in.)</th>
<th>Beam Width (in.)</th>
<th>Beam Thick. (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>136</td>
<td>18.7</td>
<td>1.88</td>
<td>3.04</td>
<td>133.00</td>
<td>35.9</td>
<td>0.625</td>
<td>0.940</td>
<td>25</td>
<td>5.00</td>
<td>0.625</td>
<td></td>
</tr>
<tr>
<td>1A</td>
<td>120</td>
<td>14.0</td>
<td>0.44</td>
<td>0.71</td>
<td>142.28</td>
<td>17.7</td>
<td>0.300</td>
<td>0.425</td>
<td>12</td>
<td>11.50</td>
<td>0.375</td>
<td></td>
</tr>
<tr>
<td>2A</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>142.50</td>
<td>23.6</td>
<td>0.395</td>
<td>0.505</td>
<td>18</td>
<td>4.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>194</td>
<td>18.3</td>
<td>1.77</td>
<td>2.85</td>
<td>156.25</td>
<td>18.1</td>
<td>0.390</td>
<td>0.630</td>
<td>9</td>
<td></td>
<td>12</td>
<td>0.375</td>
</tr>
<tr>
<td>II</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24.1</td>
<td>0.470</td>
<td>0.770</td>
<td>18</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Additionally, in Liu and Astaneh-Asl's Specimen 1A the beam connects to the column's weak axis. To stiffen the column, Liu and Astaneh-Asl placed stiffener plates above and below the shear tab. They called out the stiffener plates as being 0.5 inches in thickness. However, they were not specific with the other dimensions of the plates. Therefore, an assumption that the plates would fill the entire space between the flanges and the web was made. Initial runs of the model revealed that the plates were too large and therefore a width of 6.5 in. was preferable.
Results from these initial runs also suggested that the shear tab was welded to the stiffener plates although it was not mentioned by Liu and Astaneh-Asl.

The creation of the bolts used in the testing presented an additional challenge due to the threads present in the bolt shank. The strength of structural bolts is dependent on a 75% reduction of the bolt material yield stress (AISC, 2005a). This 75% reduction "accounts for the approximate ratio of the effective area of the threaded portion of the bolt to the area of the shank of the bolt for common sizes" (AISC, 2005a). AISC uses equation (4-1) to find the effective area of the threaded portion of bolts based on the bolt diameter, d, and the threads per inch, n, of the bolt (AISC, 2005b). By combining the value of $A_{net}$ with equation (4-2) the threaded portion of the shank can be approximated as a hollow tube. See Figure 4-1 for a cross-section of the bolt, washer, and nut assembly that illustrates the void created in the shank to account for the missing area due to the threads.

\[ A_{net} = 0.7854 \times \left( d_b - \frac{0.9743}{n} \right)^2 \]  

\[ A_{Hollow\ Tube} = \frac{\pi}{4} (d_{outer}^2 - d_{inner}^2) \]

Figure 4-1: FEM Bolt Cross-Section

Table 4-3 summarizes the dimensions of the bolts used in the finite element modeling. Thread height and diameter, under the shank heading in Table 4-3, refer to the height and diameter of the void created to account for the difference in area between the threaded and non-threaded portions of the shank.
### Table 4-3: Experimental Models' Bolt Dimensions

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Size/Dia. (in.)</th>
<th>Height (in.)</th>
<th>Width (in.)</th>
<th>Grip (in.)</th>
<th>Thread Height (in.)</th>
<th>Thread Dia. (in.)</th>
<th>Thick. (in.)</th>
<th>Height (in.)</th>
<th>Width (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1/2</td>
<td>39/64</td>
<td>1 5/8</td>
<td>1.250</td>
<td>0.3435</td>
<td>0.4778</td>
<td>0.1565</td>
<td>63/64</td>
<td>1 5/8</td>
</tr>
<tr>
<td>1A</td>
<td>7/8</td>
<td>35/64</td>
<td>1 7/16</td>
<td>0.675</td>
<td>0.2185</td>
<td>0.4212</td>
<td>0.1565</td>
<td>55/64</td>
<td>1 7/16</td>
</tr>
<tr>
<td>2A</td>
<td>7/8</td>
<td>35/64</td>
<td>1 7/16</td>
<td>0.770</td>
<td>0.2185</td>
<td>0.4212</td>
<td>0.1565</td>
<td>55/64</td>
<td>1 7/16</td>
</tr>
<tr>
<td>II</td>
<td>3/4</td>
<td>15/32</td>
<td>1 1/4</td>
<td>0.765</td>
<td>0.2255</td>
<td>0.3704</td>
<td>0.1495</td>
<td>47/64</td>
<td>1 1/4</td>
</tr>
<tr>
<td>III</td>
<td>3/4</td>
<td>15/32</td>
<td>1 1/4</td>
<td>0.845</td>
<td>0.2255</td>
<td>0.3704</td>
<td>0.1495</td>
<td>47/64</td>
<td>1 1/4</td>
</tr>
</tbody>
</table>

#### 4.1.2 Partitioning and Meshing

The next step in the creation of the connection components is the partitioning and meshing of each piece. Meshing is the process by which the finite element modeling program breaks the parts of the model down into finite elements. The computer is able to take each connection piece, analyze its shape, and based on the user's criteria place the finite element mesh on the part. For all of the experimental models, the finite element program used C3D8R elements on all of the model parts. C3D8R elements are three-dimensional 8-node linear blocks with reduced integration and hourglass control.

However some shapes are too complex for the program to mesh without assistance. Therefore partitioning is used to segment the part into easier to mesh geometrical shapes such as rectangles. The segmentation is mainly used to aid the meshing process, during analysis the part is still considered to be whole. In addition to aiding the meshing process, partitioning can be used to separate areas that will have different sized finite elements. Generally, smaller finite elements are more accurate and better able to capture the true stresses and strains that would occur in the actual component in the experiment. The same meshing and partitioning was performed on each part for each specimen's model to provide consistency.
4.1.2.1 Columns

For each model, the column flanges are separated from the webs using partitioning to allow the program to mesh the part properly. In addition, the column is partitioned twice along its length. This is done to allow for finer meshing around the connection. The partitions along the length occur at one beam depth above and below the centerline of the connection. For Specimens 1, 1A, and 2A the researchers placed the connection centerline at the mid-height of the column. However in Crocker and Chambers' Specimens I, II, and III the connection centerline occurs 11 inches above the column mid-height.

The meshing of the column consisted of two different sizes. The model used an approximate mesh size of 1.5 inches in the middle portion of each column while using a mesh size of 3 inches on the ends of the columns. However, a 1.5 or 3 inch mesh spacing would result in only one finite element thick flanges. Since one finite element could not capture the true deflected shape or stresses of the flanges, the models utilized a minimum of two finite elements per flange.
thickness. Figure 4-2 and Figure 4-3 illustrate both the partitioning and finite element mesh present in the models.

4.1.2.2 Beams

Much of the partitioning of the beams is similar to the columns. In addition to the partitioning of the beam flanges from the web, the beams are segmented along their lengths, with the cuts occurring at one and two beam depths from the connection. This was done to decrease the mesh size closer to the connection. The areas around the bolt holes in the beam's web required additional partitioning so that the meshing could be performed properly. To meet the meshing requirements, the model broke the bolt holes into quarters with the partitions covering the same area as the shear tab that would be connected to the beam, see Figure 4-4.

The meshing of the beam consisted of four different sizes. The areas around the bolt holes used an approximate mesh size of 0.25 inches, while the portion of the beam closest to the connection consisted of a 1.5 inch mesh. The middle portion of the beam adopted a mesh size of 3 inches. The remainder of the beam's length employed a 6 inch mesh size. However, a 3 or 6 inch mesh spacing would result in only one finite element thick flanges. Since one finite element could not capture the true deflected shape or stresses of the flanges, the model required a minimum of two
finite elements per flange thickness. Figure 4-5 illustrates the finite element mesh present in the models.

### 4.1.2.3 Shear Tabs

The shear tabs required partitioning around the bolt holes to allow for proper meshing. Similar to the beam web bolt holes, the model partitioned each hole into quarters. The shear tabs' required only one mesh size, 0.25 inches, due to their small size and proximity to the connection. Figure 4-6 and Figure 4-7 illustrate both the partitioning and finite element mesh present in the models.

![Figure 4-6: Shear Tab Partitions](image1)

![Figure 4-7: Shear Tab Mesh](image2)

### 4.1.2.4 Bolts

The bolts employed in each model required multiple partitions along their lengths due to the diameter changes and the void created in their shanks. The first and second segmentations separated the head and the nut/washer assembly from the shank of the bolt. The third partition occurred along the shank length, at the transition from the hollow tube, representing the threads, to the solid portion of the shank. The bolts required only one mesh size, 0.25 inches, due to their
small size and proximity to the connection. Figure 4-8 and Figure 4-9 illustrate both the partitioning and finite element mesh present in the models.

![Figure 4-8: Bolt Partitions](image1)

![Figure 4-9: Bolt Mesh](image2)

### 4.1.3 Material Properties

For each of the models, the material properties came from the information provided in the researcher's papers in so far as possible. Liu and Astaneh-Asl provided yield stress information from coupon tests performed on the steel used in their connections. Therefore the models for their connections use this data in their material properties. Larson provided static yield stress data for the beam and column material used in his connection but not for the plate used in the shear tab. He did list the steel grades used in each connection's components. Crocker and Chambers provided the steel grades used in the bolts and shear tabs of their connections. However, they did not mention the results of any coupon tests for this material and did not state the steel grades used in the column and beam portions of the connections. Therefore the models assumed these components were made of A992 Grade 50 steel.

Combining the provided steel grade designations with AISC's ratio of expected yield stress to specified minimum yield stress, $R_y$, allowed for an expected yield stress to be calculated for each
grade and used in the models (2005c). Each of the finite element models used an elastic-
perfectly plastic material model.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Column Steel</th>
<th>Yield Str. (ksi)</th>
<th>Beam Steel</th>
<th>Yield Str. (ksi)</th>
<th>Shear Tab Steel</th>
<th>Yield Str. (ksi)</th>
<th>Bolts Steel</th>
<th>Yield Str. (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A572 Gr. 50</td>
<td>54.42</td>
<td>A36</td>
<td>42.77</td>
<td>A36</td>
<td>46.80**</td>
<td>A325</td>
<td>120**</td>
</tr>
<tr>
<td>1A</td>
<td>A572 Gr. 50</td>
<td>52.07</td>
<td>A572 Gr. 50</td>
<td>55.11</td>
<td>A36</td>
<td>45.98</td>
<td>A325</td>
<td></td>
</tr>
<tr>
<td>2A</td>
<td>A992 Gr. 50*</td>
<td>55.00</td>
<td>A992 Gr. 50</td>
<td>55.00</td>
<td>A36</td>
<td>46.80**</td>
<td>A325</td>
<td></td>
</tr>
<tr>
<td>I</td>
<td>A992 Gr. 50*</td>
<td>55.00</td>
<td>A992 Gr. 50</td>
<td>55.00</td>
<td>A36</td>
<td>46.80**</td>
<td>A325</td>
<td></td>
</tr>
</tbody>
</table>

*Assumed steel grade and yield strength value used in model
**Assumed yield strength value used in model

For all of the experimental tests, the researchers used A325 bolts. Since there is no AISC approved value of $R_y$ for bolts, the models use the minimum specified yield stress for A325 steel. All of the models also used the standard values for other steel properties such as Young's modulus and Poisson's ratio: 29000 ksi and 0.3 respectively. Additionally, the bolt material included one more material property. The bolt material in each model included steel's coefficient of thermal expansion with a value of 6.5x10^{-6} /°F. The reason for including this material property will be explained in later sections.

4.2 Assembling the Connection

After the models contained sufficient information for each part, assembly of the connection could begin. Once the connection's geometry is reproduced, the finite element modeling program requires definition of several quantities and parameters to understand how the various parts interact with each other. Figure 4-10 through Figure 4-15 show the complete and assembled specimen connections in the finite element modeling program.
The first parameter that requires definition is each model's contact surfaces. The finite element program needs every pair of surfaces that could come into contact to be defined. During the analysis, if two surfaces come into contact and they are not defined as a contact pair, the surfaces will move through each other. Examples of contact pairs that required definition include the shear tab and beam web interaction and each bolt's shank with the inside of that bolt's hole.

Incorporated with naming the contact pairs is the definition of the interaction properties. Specifically the interaction properties define normal and tangential behavior for the contact surfaces. Each model used a penalty friction formulation in the interaction property's tangential behavior with a friction coefficient defined for each model. Based on the research of Ruffley, the models used an initial value of 0.2 for the coefficient of friction (2011). For normal behavior, every model used the "Hard" contact and default settings for the interaction property's pressure-overclosure and constraint enforcement method, respectively, while allowing separation after contact.

Next, the models required the definition of constraints and boundary conditions. These define how the model is supported, what degrees of freedom (DoF) those supports allow, and how they are applied to the model. All of the models contain three reference points, with one at each end of the column and one at the beam end away from the connection. All three reference points are located along each members' centerline. The boundary conditions for the model are imposed on these reference points.
Figure 4-10: Larson Specimen 1 Model

Figure 4-11: Liu and Astaneh-Asl Specimen 1A Model

Figure 4-12: Liu and Astaneh-Asl Specimen 2A Model

Figure 4-13: Crocker and Chambers Specimen I Model

Figure 4-14: Crocker and Chambers Specimen II Model

Figure 4-15: Crocker and Chambers Specimen III Model
Table 4-5 summarizes which DoF the boundary conditions restricted for each reference point in the finite element models of Specimens 1, 1A, and 2A. These specimens shared the same boundary conditions because in all three experiments, the column and beam ends used pinned supports. These pin supports allowed unrestrained rotation in-plane and restricted torsional rotation. Table 4-6 summarizes which DoF the boundary conditions restricted for each reference point in the finite element models of Specimens I, II, and III. In their tests, Crocker and Chambers used fixed connections at the top and bottom of the columns. Therefore none of the DoF at the top and bottom of the column are unrestricted.

Table 4-5: Specimens 1, 1A, and 2A Model Boundary Conditions

<table>
<thead>
<tr>
<th>Reference Point</th>
<th>Restrained DoF</th>
<th>Unrestrained DoF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam End</td>
<td>U1, U2, U3, UR1</td>
<td>UR2, UR3</td>
</tr>
<tr>
<td>Column Top</td>
<td>U1, U2, U3, UR2</td>
<td>UR1, UR3</td>
</tr>
<tr>
<td>Column Bottom</td>
<td>U1, U2, U3</td>
<td>UR1, UR2, UR3</td>
</tr>
</tbody>
</table>

Table 4-6: Specimens I, II, and III Model Boundary Conditions

<table>
<thead>
<tr>
<th>Reference Point</th>
<th>Restrained DoF</th>
<th>Unrestrained DoF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam End</td>
<td>U1, U2, U3, UR1</td>
<td>UR2, UR3</td>
</tr>
<tr>
<td>Column Top</td>
<td>U1, U2, U3, UR1, UR2, UR3</td>
<td>-</td>
</tr>
<tr>
<td>Column Bottom</td>
<td>U1, U2, U3, UR1, UR2, UR3</td>
<td>-</td>
</tr>
</tbody>
</table>

During initial steps in the analysis, the models imposed additional boundary conditions on the bolts in the connection. These additional boundary conditions turned the washer/nut assembly portion of the bolts into a fixed connection. The finite element modeling program required these boundary conditions to allow for proper pretensioning of the bolts. During later analysis steps, the program removed the boundary conditions on the bolts, eliminating any effect they would have on the final results of the model.
Lastly, the model requires several constraints that define how the boundary conditions affect the model. Since the reference points are not a physical piece of any of the connection's parts, the constraints tell the finite element program how the reference points and model parts interact. Specifically, a coupling constraint is used to attach each reference point to the corresponding cross-section where the reference point is located. A coupling constraint takes the translations and rotations experienced by the reference point and extrapolates them to each of the finite element nodes on the associated surface. After these three coupling constraints, a fourth constraint occurs between the column surface and the edge of the shear tab in each model. The fourth constraint is a tie constraint and forces the finite element nodes along the shear tab's edge to have the same translations and rotations as the nodes on the column's surface. This fourth restraint reproduces the weld that connects the column and shear tab together.

### 4.3 Bolt Pretension and Connection Loading

To bring the various model parts into solid contact, the bolts in the model must be pretensioned like they would be in a real connection. Ruffley reproduced bolt pretension forces in his models by applying a temperature differential to the shanks of the simplified model bolts (2011). Using equations (4-3) and (4-4), equation (4-5) was derived to give a temperature differential that could be applied to the bolts to reproduce the proper bolt pretension. However this equation can only produce approximate temperature differentials because it does not take into account the deflections in the shear tab and beam web as the bolt shank length decreases. The required pretension force in the bolts came from AISC (2005a). Table 4-7 summarizes the temperature differentials applied to the bolts in each model to achieve the proper pretension force.
The final step in creating the finite element models is applying the deflections necessary to reproduce the rotations experienced by the experimental connections. Section 4.2 mentioned U2 was a restricted DoF for the end of the beam away from the connection. Specifically, DoF U2 is the vertical DoF used to apply the necessary displacements from the experiments. In all of the experimental tests, the largest rotations the connections experienced occurred with the end of the beam lifted upwards. Therefore, the finite element program initially lifted each beam tip upwards by an amount, listed in Table 4-8, based on small displacement theory and the maximum rotations listed in the original papers and data. Some of the tip displacements required adjustment to bring the rotation of the connection models into agreement with the experimental data.

Table 4-8: Experimental Models' Beam Tip Displacement

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Rotation (rad.)</th>
<th>Tip Disp. (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-</td>
<td>4.300</td>
</tr>
<tr>
<td>1A</td>
<td>0.1497</td>
<td>21.300</td>
</tr>
<tr>
<td>2A</td>
<td>0.0906</td>
<td>10.65</td>
</tr>
<tr>
<td>I</td>
<td>0.0600</td>
<td>9.375</td>
</tr>
<tr>
<td>II</td>
<td>0.0600</td>
<td>9.375</td>
</tr>
<tr>
<td>III</td>
<td>0.0600</td>
<td>7.120</td>
</tr>
</tbody>
</table>
4.4 Results

Each model generally required multiple cycles of analysis to properly match the results from the experimental tests. The changes in the models between cycles often consisted of fine tuning the bolt pretension temperature differential, the friction coefficient, and the various boundary conditions. The following sections summarize some of the large changes and present the results of the final models.

4.4.1 Larson Specimen 1

The model corresponding to Larson's Specimen 1 required many versions and changes in an attempt to properly match the experimental data. The model's initial changes centered on adjusting the coefficient of friction so that the slip planes matched between the model results and experimental data as well as attempting to explain why the model results showed the beam bearing on the column at higher rotations. Since it was not possible to know the exact pretension force in the experiment's connection bolts, the finite element model's coefficient of friction can be altered to bring the slip planes of the model and experiment into agreement. The final coefficient of friction used in the model of Larson's Specimen 1 was 0.225. The problem of bearing between the beam and column resolved when the model used the shear tab dimensions seen in Englehardt's experiments. The results of the final model are shown in Figure 4-16.

Additional revisions to the model attempted to more closely match the initial stiffness of the experiment and model. These changes included adjusting reference point locations, changing finite element types, and altering boundary conditions. In the end, none of these changes were able to bring the initial stiffness of the model and experiment into closer agreement. A plot of the initial portion of both the finite element model and experiment data, shown in Figure 4-17,
indicates that the results of the final model had an initial stiffness 43.6% greater than that seen in the experiment.

Figure 4-16: Larson Specimen 1 Model Results

Figure 4-17: Larson Specimen 1 Initial Stiffness

Figure 4-18: FEM Specimen 1 Stress Contours

Figure 4-18 shows the stress contours in the model overlaid on top of the connection's deflected shape at the end of the analysis. In the figure, since the bolt stresses are much larger than the rest of the model, the bolts and their stresses have been omitted. The stresses shown in Figure 4-18, are von Mises' stresses with the contours set so that the any stresses above the yield stress of the shear tab, 46 ksi, appear grey. In his description of the test, Larson did not mentioned any shear
tab yielding or bolt failures during the testing. These contours match that description with only small portions of yielding in the shear tab around the bolt holes.

4.4.2 Liu and Astaneh-Asl Specimen 1A

Initial analytical runs of the finite element model for Liu and Astaneh-Asl's Specimen 1A attempted to have the slip plane of the model match the experimental data. Similar to the Larson model, this involved changes to the friction coefficient. The final friction coefficient used in the model of Liu and Astaneh-Asl's Specimen 1A was 0.3.

![Figure 4-19: Liu and Astaneh-Asl Specimen 1A Model Results](image)

![Figure 4-20: Liu and Astaneh-Asl Specimen 1A Initial Stiffness](image)

Initial runs also ignored the stiffener plates in the column. The results from these runs showed that the model had an initial stiffness that was approximately half that of the experiment. When the model incorporated the column stiffeners, the difference between the initial stiffness values became much lower. The initial column stiffener plates used in the models were made to be the same width as the column's flanges, but this led to the beam end bearing on the end of the plate at higher rotations. Since the data for the experiment showed no sign of bearing between the beam and column, the stiffener plate's width was reduced by a half inch, resulting in the 6.5 in. width mentioned in Section 4.1.1. This reduced width eliminated the bearing between the plate.
and beam. The results of the final model are shown in Figure 4-19. A plot of the initial portion of both the finite element model and experiment data, shown in Figure 4-20, indicates that the results of the final model had an initial stiffness within 2.7% of that seen in the experiment. Figure 4-20 also contains a plot of the stiffness AISC considers to be the upper limit for pinned connections, $2EI/L$, where $I$ and $L$ refer to the connecting beam.

![Figure 4-21: FEM Specimen 1A Stress Contours](image)

Figure 4-21 shows the stress contours in the model overlaid on top of the connection's deflected shape at the end of the analysis. In the figure, since the bolt stresses are much larger than the rest of the model, the bolts and their stresses have been omitted. The stresses shown in Figure 4-21, are von Mises' stresses with the contours set so that any stresses above the yield stress of the shear tab, 46 ksi, appear grey. These finite element models are not set up with fracture mechanics and therefore cannot predict when or if a given component of the connection would fracture in a real connection. Therefore, the program lets a piece yield indefinitely until the analysis is completed. At that point engineering judgment must be used to determine if the yielded piece would have fractured in a real connection. Liu and Astaneh-Asl describe the failure mode of Specimen 1A as "fracture of tab after much slip and yielding" (2000). Based on
engineering judgment and the stress contours in Figure 4-21, the finite element model matches this description.

4.4.3 Liu and Astaneh-Asl Specimen 2A

Similar to the Larson model and Liu/Astaneh-Asl's Specimen 1A model, the initial runs of Liu and Astaneh-Asl's Specimen 2A model involved changes to the friction coefficient in an effort to have the slip planes between the experimental data and finite element model match. This required a final friction coefficient of 0.3, the same value used in the finite element models of Liu and Astaneh-Asl's Specimen 1A.

After the initial runs to confirm the proper friction coefficient value, the Specimen 2A model required no other analysis runs. Figure 4-22 and Figure 4-23 present the results of the final model analysis. The model predicts the beam end coming into bearing with the columns flange at a rotation of approximately 0.06 radians. While not mentioned by Liu and Astaneh-Asl in their paper, the experiment data shows bearing between the beam and column, indicated by the large increase in stiffness, at rotations between 0.06-0.07 radians. A plot of the initial portion of
both the finite element model and experiment data, shown in Figure 4-24, shows that the results of the final model had an initial stiffness within 11.9% of that seen in the experiment. Figure 4-24 also contains a plot of the stiffness AISC considers to be the upper limit for pinned connections, $2EI/L$, where I and L refer to the connecting beam.

![Figure 4-24: Liu and Astaneh-Asl Specimen 2A Initial Stiffness](image)

Figure 4-24 shows the stress contours in the model overlaid on top of the connection's deflected shape at the end of the analysis. In the figure, since the bolt stresses are much larger than the rest of the model, the bolts and their stresses have been omitted. The stresses shown in Figure 4-25, are von Mises' stresses with the contours set so that any stresses above the yield stress of the shear tab, 46 ksi, appear grey. These finite element models are not set up with fracture mechanics and therefore cannot predict when or if a given component of the connection would fracture in a real connection. Therefore, the program lets a piece yield indefinitely until the analysis is completed. At that point engineering judgment must be used to determine if the yielded piece would have fractured in a real connection. Liu and Astaneh-Asl describe the failure mode of Specimen 2A as "fracture of tab after much slip and yielding" (2000). Based on
engineering judgment and the stress contours in Figure 4-25, the finite element model matches this description.

Figure 4-25: FEM Specimen 2A Stress Contours

The finite element model's inability to predict component failure is also the reason for the high end moment the model experiences at the end of the analysis, see Figure 4-22. Since the model cannot predict and reproduce the fracture of the shear tab, it cannot redistribute the stresses in the finite elements around the bolt holes that should have failed. These high stresses located so far from the connection's neutral axis cause the high end moments observed in Figure 4-22.

4.4.4 Crocker and Chambers Specimen I

Initial analytical runs of the finite element model for Crocker and Chambers' Specimen I attempted to have the slip plane of the model match the experimental data. Similar to the previous models, this involved changes to the friction coefficient. The final friction coefficient used in the models of Crocker and Chambers' Specimen I was 0.37. Other than adjusting the friction coefficient, the Crocker Specimen I model required very little work to obtain results that closely matched the experimental data, see Figure 4-26. A plot of the initial portion of both the
finite element model and experiment data, shown in Figure 4-27, indicates that the results of the final model had an initial stiffness within 8.0% of that seen in the experiment. Figure 4-27 also contains a plot of the stiffness AISC considers to be the upper limit for pinned connections, $2EI/L$, where $I$ and $L$ refer to the connecting beam.

The large difference between the size of the slip planes between the experimental and finite element model data is believed to be due to two factors. The first is that all of the models positioned the bolts in the center of the bolt holes. This is an idealized condition, rarely reproduced in the field. The second factor is that it was not possible to obtain the original cyclic test data. Instead, Crocker and Chambers provided simplified moment-rotation data that only has peak values from the various cycles.

Figure 4-26: Crocker and Chambers Specimen I Model Results

Figure 4-27: Crocker and Chambers Specimen I Initial Stiffness

Figure 4-28 shows the stress contours in the model overlaid on top of the connection's deflected shape at the end of the analysis. In the figure, since the bolt stresses are much larger than the rest of the model, the bolts and their stresses have been omitted. The stresses shown are von Mises' stresses with the contours set so that any stresses above the yield stress of the shear tab, 46 ksi, appear grey. These finite element models are not set up with fracture mechanics and therefore
cannot predict when or if a given component of the connection would fracture in a real connection. Crocker and Chambers stated that Specimen I "did not reach any limit states during testing" (2004). Figure 4-28 appears to confirm the result that they observed. While there is some yielding of the shear tab around the top and bottom bolt holes, it is not enough to indicate fracture of the plate.

![Figure 4-28: FEM Specimen I Stress Contours](image)

### 4.4.5 Crocker and Chambers Specimen II

Initial analytical runs of the finite element model for Crocker and Chambers' Specimen II attempted to have the slip plane of the model match the experimental data. Similar to the previous models, this involved changes to the friction coefficient. The final friction coefficient used in Crocker and Chambers' Specimen II was 0.37. Other than adjusting the friction coefficient, the Crocker Specimen II model required very little work to obtain results that closely matched the experimental data, see Figure 4-29. A plot of the initial portion of both the finite element model and experiment data, shown in Figure 4-30, indicates that the results of the final model had an initial stiffness within 4.7% of that seen in the experiment. Figure 4-30 also
contains a plot of the stiffness AISC considers to be the upper limit for pinned connections, 2EI/L, where I and L refer to the connecting beam.

![Figure 4-29: Crocker and Chambers Specimen II Model Results](image)

![Figure 4-30: Crocker and Chambers Specimen II Initial Stiffness](image)

The large difference between the size of the slip planes between the experimental and finite element model data is believed to be due to two factors. The first is that all of the models positioned the bolts in the center of the bolt holes. This is an idealized condition, rarely reproduced in the field. The second factor is that it was not possible to obtain the original cyclic test data. Instead, Crocker and Chambers provided simplified moment-rotation data that only has peak values from the various cycles.

Figure 4-31 shows the stress contours in the model overlaid on top of the connection's deflected shape at the end of the analysis. In the figure, since the bolt stresses are much larger than the rest of the model, the bolts and their stresses have been omitted. The stresses shown are von Mises' stresses with the contours set so that any stresses above the yield stress of the shear tab, 46 ksi, appear grey. These finite element models are not set up with fracture mechanics and therefore cannot predict when or if a given component of the connection would fracture in a real
connection. Crocker and Chambers stated that Specimen II "did not reach any limit states during testing" (2004). Figure 4-31 appears to confirm the result that they observed. While there is some yielding of the shear tab around the top and bottom bolt holes, it is not enough to indicate fracture of the plate.

![Figure 4-31: FEM Specimen II Stress Contours](image)

**4.4.6 Crocker and Chambers Specimen III**

Initial analytical runs of the finite element model for Crocker and Chambers’ Specimen III attempted to have the slip plane of the model match the experimental data. Similar to the previous models, this involved changes to the friction coefficient. The final friction coefficient used in Crocker and Chambers’ Specimen III was 0.45. Figure 4-32 shows the results of the Crocker and Chambers’ Specimen III finite element model. A plot of the initial portion of both the finite element model and experiment data, shown in Figure 4-33, indicates that the results of the final model had an initial stiffness within 2.6% of that seen in the experiment. Figure 4-33 also contains a plot of the stiffness AISC considers to be the upper limit for pinned connections, $2EI/L$, where $I$ and $L$ refer to the connecting beam.
In the experimental data, the large drops in stiffness that occur at rotations of 0.04 and 0.05 radians is the result of bolts failing in the connections during the experiments. Additional models attempted to reproduce the effect of the bolt's sudden removal in the finite element program. However after several attempts the models could not converge on a solutions for these more advanced variations. Due to time constraints, the model variations that attempted to reproduce the fractured bolts were aborted.

The large difference between the size of the slip planes between the experimental and finite element model data is believed to be due to two factors. The first is that all of the models positioned the bolts in the center of the bolt holes. This is an idealized condition, rarely reproduced in the field. The second factor is that it was not possible to obtain the original cyclic test data. Instead, Crocker and Chambers provided simplified moment-rotation data that only has peak values from the various cycles.

Figure 4-34 shows the stress contours in the model overlaid on top of the connection's deflected shape at the end of the analysis. In the figure, since the bolt stresses are much larger than the rest of the model, the bolts and their stresses have been omitted. The stresses shown are von Mises'
stresses with the contours set so that any stresses above the yield stress of the shear tab, 46 ksi, appear grey. These finite element models are not set up with fracture mechanics and therefore cannot predict when or if a given component of the connection would fracture in a real connection. In the experiment “Specimen III did reach bolt shear limit states when the three bolts were sheared prior to reaching 0.06 radians rotation” (Crocker and Chambers, 2004). Figure 4-35 shows the stress contours present in the bolt at the top of the connection at the end of the analysis. This is one of the bolts that fractured at a rotation of 0.04 radians in the experiment. Similar to Figure 4-34, the stress contours in Figure 4-35 are set so that stresses above the yield stress of the bolt, 120 ksi, appear grey. The large amount of grey areas along the length of the bolt shank and across its diameter in Figure 4-35 strongly support the observations made by Crocker and Chambers.

4.4.7 Experimental Model Summary and Findings
The final results of Liu and Astaneh-Asl's Specimens 1A and 2A as well as Crocker and Chambers’ Specimens I, II, and III show finite element modeling’s strong ability to replicate the findings of an full-scale experimental test. All five of these finite element models correctly
matched the slip planes seen in the experiments. The models also recreated the initial stiffness within 12% of the values seen in the experiments.

More importantly, the models predicted specific events in all models. The Specimen 1A finite element model correctly showed the effect of the column stiffener plate on the connection's initial stiffness. Specimen 2A's model matched the bearing that occurred in the connections experimental testing. Both models also predicted the shear tab failure experienced by both connections during their testing. Additionally, Specimen III’s model very strongly supported the bolt failure observations made by the researchers.

The finite element model of Larson's Specimen 1 was able to correctly match the slip planes seen in Larson's testing. The model also matched the general shape of the experiment's results. However, the model could not predict either the initial stiffness or moment values experienced after the connection slipped during the testing. This is likely due to a lack of information regarding the testing and access to the original test data.
Chapter 5: Finite Element Modeling: Theoretical Connections

This portion of the thesis modeling focuses on the creation of moment-rotation curves for theoretical shear tab connections. The design and sizes of the theoretical connections are based on current design procedures. Based on the results of Chapter 4, finite element modeling can predict the response of these theoretical connections and produce the moment-rotation curves that describe their response to rotational demands. The following sections will discuss the design, construction, and finite element model results of two theoretical connections.

5.1 Connection Loading

The theoretical connections designed for this thesis were meant to be based on current design procedures. The sizes of the connections were also to be practical. Therefore, a theoretical building was created to contain the theoretical connections. The building's intended size and purpose was a five story office building with storage located on the second floor. The first floor is 15 feet tall with remaining stories being 12 feet tall. The spacing between columns is 30 feet center-to-center in both the X and Y directions. Filler beams connect to the girders at 7.5 feet on center.

The theoretical connections occur where the girders connect to the columns. Connection 1 occurs at the fourth floor (ceiling of the third floor) and supports office space. Connection 2 occurs at the second floor (ceiling of the first floor) and supports storage space. The difference in floors and design function was an effort to obtain connections with some differences in member sizes as well as number of required bolts.

Each of the connections support live loads from their intended purposes as well as live loads from finishes and equipment along with dead loads from the floor system. The weights of the
equipment and finishes come from ASCE's recommendations (2010). Additional loadings for finishes not listed in ASCE are based on engineering judgment and marked in Table 5-1 and Table 5-2. Information on the metal decking used in the floor system, the required concrete floor thickness, and weight of the decking comes from Corrugated Metal (2012).

<table>
<thead>
<tr>
<th>Table 5-1: Theoretical Connections' Live Loadings</th>
<th>Table 5-2: Theoretical Connections' Dead Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Live Loadings (psf)</strong></td>
<td><strong>Floor Dead Loadings (psf)</strong></td>
</tr>
<tr>
<td><strong>Conn. 1</strong></td>
<td><strong>Conn. 1</strong></td>
</tr>
<tr>
<td><strong>Conn. 2</strong></td>
<td><strong>Conn. 2</strong></td>
</tr>
<tr>
<td>Office Space</td>
<td>50</td>
</tr>
<tr>
<td>Storage Space</td>
<td>-</td>
</tr>
<tr>
<td>Carpet*</td>
<td>1</td>
</tr>
<tr>
<td>Flooring</td>
<td>-</td>
</tr>
<tr>
<td>Ceiling</td>
<td>3</td>
</tr>
<tr>
<td>Partitions (moveable)</td>
<td>4</td>
</tr>
<tr>
<td>Partitions*</td>
<td>-</td>
</tr>
<tr>
<td>Lighting*</td>
<td>2</td>
</tr>
<tr>
<td>Ventilation</td>
<td>4</td>
</tr>
<tr>
<td>Total</td>
<td>64</td>
</tr>
<tr>
<td></td>
<td>110</td>
</tr>
<tr>
<td></td>
<td><strong>Decking (2in. 19 ga.)</strong>*</td>
</tr>
<tr>
<td></td>
<td>2.55</td>
</tr>
<tr>
<td></td>
<td><strong>Concrete (4 in. thick)</strong></td>
</tr>
<tr>
<td></td>
<td>49</td>
</tr>
<tr>
<td></td>
<td><strong>Total</strong></td>
</tr>
<tr>
<td></td>
<td>51.55</td>
</tr>
<tr>
<td></td>
<td><strong>Manufacturer listed value (Corrugated Metal, 2012)</strong></td>
</tr>
</tbody>
</table>

* Amount not listed in ASCE 7

5.2 Connection Design

The design of the members used in the connections follows AISC's design procedures for shear tab connections (2005b). AISC specifies two kinds of shear tab connection configurations: conventional and extended. Both theoretical connections follow the guidelines for conventional configuration shear tab connections. The guidelines for conventional configuration connections includes a limit of one bolt line, maximum distance between bolt line and weld, and restrictions on horizontal/vertical edge distances and shear tab plate thickness (AISC, 2005b).
5.2.1 Beam Loading and Design

The first step in designing the connections involved the design of the beam and columns required to support the loadings. Both members' designs were based on LRFD and the applicable limit states listed in AISC. All the loadings for the members are increased according to LRFD load combination 2 which requires a 20% increase to dead loads and a 60% increase to live loads.

| Table 5-3: Theoretical Connections' Beam Loading | Table 5-4: Theoretical Connections' Beam Capacities |
|---|---|---|---|
| | Conn. 1 | Conn. 2 | Limit State | Conn. 1 (W24x76) | Conn. 2 (W27x94) |
| P (k) | 39 | 56 | Yielding | 9,000 | 12,510 |
| M_U (k-in) | 7,144 | 10,232 | LTB | 8,806 | 12,477 |
| Z_MIN (in^4) | 156 | 224 |
| Δ_MAX (in) | 1 |
| I_MIN (in^4) | 1,831 | 3,185 |
| Beam | W24x76 | W27x94 |

The beam loading in the connection is determined by the filler beams collecting the dead and live loads and transferring the load to the connection beam. The connection beams then transmit the load to the columns for transfer to the foundation. Trial sizes for the beams were chosen based on the moments caused by the point loads from the filler beams and the allowable deflection listed in ASCE 7. Table 5-3 summarizes the loadings on the beam in the connection, the ultimate moment the beam is required to resist, and the trial beam size chosen for each connection. Both trial beam sizes were selected to meet the stricter requirement of deflection. Table 5-4 summaries the capacity of the trial beam sizes based on the applicable limit states. Both trial beams exceed the ultimate load required and will be used in the theoretical connection. Appendix A provides more information on the computation of the applied loads and Appendix B
provides more information on the computation of these capacities. Both theoretical connections used A992 Grade 50 for the beam material.

5.2.2 Column Loading and Design

The columns in the connections gather the load from the gravity beams and transfer it to the foundation. Table 5-5 summarizes the axial load on each connection's column, the column size used in the connection, and the capacity of that column's size. The capacity of the column is based on the length of the column and the effective length factor $K$. A beam with simple connections at each end is given a $K$ value of 1 in current design practice (AISC, 2005b).

<table>
<thead>
<tr>
<th>Conn.</th>
<th>L (ft)</th>
<th>$P_u$ (k)</th>
<th>Column</th>
<th>$\phi P_N$ (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conn. 1</td>
<td>12</td>
<td>398</td>
<td>W10x45</td>
<td>410</td>
</tr>
<tr>
<td>Conn. 2</td>
<td>15</td>
<td>784</td>
<td>W12x79</td>
<td>809</td>
</tr>
</tbody>
</table>

Table 5-5: Theoretical Connections' Column Loading and Capacity

Appendix A provides more information on the computation of the applied loads and Appendix B provides more information on the computation of these capacities. Both theoretical connections used A992 Grade 50 for the column material.

5.2.3 Shear Tab and Bolt Loading and Design

Design soon began on the shear tab and bolts that would be used to connect the beam and columns in the theoretical connection. AISC has specific requirements on the dimensions of the shear tab used in a conventional configuration (2005b). A plate with dimensions of 12"x4"x1/2" meets all of AISC's shear tab dimension requirements for both connections. Therefore both connections used a 12"x4"x1/2" shear tab with a single set of bolt holes and a horizontal edge
distance of 1.75 inches. The 1.75 inch horizontal edge distance was applied to the beam webs as well, resulting in a 0.5 inch gap between the beam end and the column face. AISC requires that the design check the following limit states for the shear tab: yielding and rupture (2005b). In addition, a check was performed on the assumed 5/8 inch weld that connects the shear tab to the column. Table 5-6 summarizes the limit state capacities of both shear tabs for the connections.

Table 5-6: Theoretical Connections' Loading and Capacities

<table>
<thead>
<tr>
<th></th>
<th>Conn. 1</th>
<th>Conn. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Yielding</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RU (k)</td>
<td>59</td>
<td>84</td>
</tr>
<tr>
<td>( \Phi R_N (k) )</td>
<td>115</td>
<td>115</td>
</tr>
<tr>
<td><strong>Rupture</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RU (k)</td>
<td>59</td>
<td>84</td>
</tr>
<tr>
<td>( \Phi R_N (k) )</td>
<td>120</td>
<td>108</td>
</tr>
<tr>
<td><strong>Weld</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RU (k)</td>
<td>59</td>
<td>84</td>
</tr>
<tr>
<td>( \Phi R_N (k) )</td>
<td>167</td>
<td>167</td>
</tr>
<tr>
<td><strong>Shear</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RU (k/bolt)</td>
<td>20</td>
<td>21</td>
</tr>
<tr>
<td>( \Phi R_N (k/bolt) )</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td><strong>Bearing</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RU (k)</td>
<td>59</td>
<td>84</td>
</tr>
<tr>
<td>( \Phi R_N (k) )</td>
<td>127</td>
<td>147</td>
</tr>
<tr>
<td><strong>Block shear</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RU (k)</td>
<td>59</td>
<td>84</td>
</tr>
<tr>
<td>( \Phi R_N (k) )</td>
<td>100</td>
<td>103</td>
</tr>
</tbody>
</table>

The theoretical connections also used 7/8 inch diameter bolts. Theoretical connection 1 used 3 bolts in a single line resulting in a 4 inch spacing between the bolts. Theoretical connection 2 used 4 bolts in a single line resulting in a 3 inch spacing between the bolts. The design of the bolts checked the following limit states: shear strength, block shear, and bearing (AISC, 2005b). Appendix B contains more information on the minimum dimensions required of the shear tab as well as the calculations of the limit state checks for the shear tab and bolts. Appendix A provides more information on the computation of the applied loads.
5.3 Creating the Model Parts

5.3.1 Dimensions

The first step in reproducing the connections in ABAQUS is to recreate the components from the design. Consulting AISC's Steel Construction Manual the dimensions of the various sizes can be obtained (2005b). Table 5-7 summarizes the component sizes used in each of the theoretical shear tab connections. Table 5-8 lists the dimensions used in the modeling of each connection.

<table>
<thead>
<tr>
<th>Table 5-7: Theoretical Models' Component Size Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connection</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 5-8: Theoretical Models' Component Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
</tbody>
</table>

The creation of the bolts used in the theoretical connections followed a similar procedure as those created for the experimental models due to the threads present in the bolt shank. To account for AISC's 75% reduction of the bolt material strength, the threaded portion of the bolt shank was approximated as a hollow tube. Equation (4-1) for the effective area of the threaded portion of bolts and equation (4-2) for the area of a hollow tube were combined to find the dimensions of the void required in the shank to reproduce the proper cross-sectional area. See Figure 4-1 for a cross-section of the bolt, washer, and nut assembly that illustrates the void created in the shank to account for the missing area due to the threads. Table 5-9 also

53
summarizes the dimensions of the bolts used in the finite element modeling. Thread height and diameter, under the shank heading in Table 5-9, refer to the height and diameter of the void created to account for the difference in area between the threaded and non-threaded portions of the shank.

<table>
<thead>
<tr>
<th>Conn.</th>
<th>Size/Dia. (in.)</th>
<th>Head Height (in.)</th>
<th>Head Width (in.)</th>
<th>Grip (in.)</th>
<th>Thread Height (in.)</th>
<th>Thread Dia. (in.)</th>
<th>Thick. (in.)</th>
<th>Height (in.)</th>
<th>Width (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7/8</td>
<td>35/64</td>
<td>1 7/16</td>
<td>0.94</td>
<td>0.2185</td>
<td>0.4212</td>
<td>0.1565</td>
<td>55/64</td>
<td>1 7/16</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td>0.99</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

5.3.2 Partitioning and Meshing

The next step in the creation of the connection components is the partitioning and meshing of each piece. Meshing is the process by which the finite element modeling program breaks the parts of the model down into finite elements. The computer is able to take each connection piece, analyze its shape, and based on the user's criteria place the finite element mesh on the part. For all of the theoretical connection models, the finite element program used C3D8R elements on all of the model parts. C3D8R elements are three-dimensional 8-node linear blocks with reduced integration and hourglass control.

However some shapes are too complex for the program to mesh without assistance. Therefore partitioning is used to segment the part into easier to mesh geometrical shapes such as rectangles. The segmentation is mainly used to aid meshing, during analysis the part is still considered to be whole. In addition to aiding the meshing process, partitioning can be used to separate areas that will have different sized finite elements. Generally, smaller finite elements are more accurate
and able to capture the true stresses and strains that would occur in the real part. The same meshing and partitioning was performed for each specimen's model to provide consistency.

5.3.2.1 Columns

For each model, the column flanges are separated from the webs using partitioning to allow the program to mesh the part. In addition the column is partitioned twice along its length. This is done to allow for finer meshing around the connection. The partitions along the length occur at one beam depth above and below the centerline of the connection. For both theoretical connection models the connections centerline occurs at mid-height of the column.

The meshing of the column consisted of two different sizes. The model used an approximate mesh size of 1.5 inches in the middle portion of each column while using a mesh size of 3 inches on the ends of the columns. However, a 1.5 or 3 inch mesh spacing would result in only one finite element thick flanges. Since one finite element could not capture the true deflected shape or stresses of the flanges, a minimum of two finite elements were required per flange thickness.

5.3.2.2 Beams

Much of the partitioning of the beams is similar to the columns. In addition to the partitioning of the beam flanges from the web, the beams are segmented along their lengths, with the cuts occurring at one and two beam depths from the connection. This was done to decrease the mesh size closer to the connection. The areas around the bolt holes in the beam's web required additional partitioning so that meshing could be performed properly. To meet the meshing requirements, the model broke the bolt holes into quarters with the partitions covering the same area as the shear tab that would be connected to the beam.
The meshing of the beam consisted of four different sizes. The areas around the bolt holes used an approximate mesh size of 0.25 inches, while the portion of the beam closest to the connection consisted of a 1.5 inch mesh. The middle portion of the beam adopted a mesh size of 3 inches. The remainder of the beam's length employed a 6 inch mesh size. However, a 3 or 6 inch mesh spacing would result in only one finite element thick flanges. Since one finite element could not capture the true deflected shape or stresses of the flanges, a minimum of two finite elements were required per flange thickness.

5.3.2.3 Shear Tabs

The shear tabs required partitioning around the bolt holes to allow for proper meshing. Similar to the beam web bolt holes, the model partitioned each hole into quarters. The shear tabs' required only one mesh size, 0.25 inches, due to their small size and proximity to the connection.

5.3.2.4 Bolts

The bolts employed in each model required multiple partitions along their lengths due to the diameter changes and the void created in their shanks. The first and second segmentations separated the head and the nut/washer assembly from the shank of the bolt. The third partition occurred along the shank length, at the transition from the hollow tube, representing the threads, to the solid portion of the shank. The bolts required only one mesh size, 0.25 inches, due to their small size and proximity to the connection.

5.3.3 Material Properties

For each of the models, the material properties used came from the initial design of the connections. Combining these steel grade designations with AISC's ratio of expected yield stress to specified minimum yield stress, $R_{y}$, allowed for an expected yield stress to be calculated for
each grade and used in the models (2005c). Each of the finite element models used a material model of elastic-perfectly plastic.

All of the theoretical connections used A325 bolts. Since there is no AISC approved value of $R_y$ for bolts, the models use the minimum specified yield stress for A325 steel. The models also used the standard values for other steel properties such as Young's modulus and Poisson's ratio: 29000 ksi and 0.3 respectively. Additionally, the bolt material properties included steel's coefficient of thermal expansion with a value of $6.5 \times 10^{-6} /{^\circ\text{F}}$.

<table>
<thead>
<tr>
<th>Table 5-10: Theoretical Models' Material Yield Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Conn.</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
</tbody>
</table>

5.4 Assembling the Connection

Once the models contained sufficient information for each part, assembly of the connection could begin. Once the connection's geometry is reproduced, the finite element modeling program requires definition of several quantities and parameters to understand how the various parts interact with each other. Figure 5-1 and Figure 5-2 show the complete and assembled theoretical connections in the finite element modeling program.

The first parameter that requires definition is the models contact surfaces. The finite element program needs every pair of surfaces that could come into contact to be defined. During the analysis, if two surfaces come into contact and they are not defined as a contact pair, the surfaces will move through each other. Examples of contact pairs that required definition include the shear tab and beam web interaction and each bolt’s shank with the inside of that bolt's hole.
Incorporated with naming the contact pairs is the definition of the interaction properties. Specifically the interaction properties define normal and tangential behavior for the contact surfaces. Each model used a penalty friction formulation in the interaction property's tangential behavior with a friction coefficient defined for each model. For normal behavior, every model used the "Hard" contact and default settings for the interaction property's pressure-overclosure and constraint enforcement method, respectively, while allowing separation after contact.

Next, the models required the definition of constraints and boundary conditions. These define how the model is supported, what degrees of freedom (DoF) those supports allow, and how they are applied to the model. All of the models contain three reference points, with one at each end of the column and one at the beam end away from the connection. All three reference points are located along each members' centerline. The boundary conditions for the model are imposed on these reference points. Table 5-11 summarizes which DoF the boundary conditions restricted for...
each reference point. Both theoretical connections used pin connections at the ends of the column and beam members, similar to the models for Specimens 1, 1A, and 2A.

Table 5-11: Theoretical Models' Boundary Conditions

<table>
<thead>
<tr>
<th>Reference Point</th>
<th>Restrained DoF</th>
<th>Unrestrained DoF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam End</td>
<td>U1, U2, U3, UR1</td>
<td>UR2, UR3</td>
</tr>
<tr>
<td>Column Top</td>
<td>U1, U2, U3, UR2</td>
<td>UR1, UR3</td>
</tr>
<tr>
<td>Column Bottom</td>
<td>U1, U2, U3</td>
<td>UR1, UR2, UR3</td>
</tr>
</tbody>
</table>

During initial steps in the analysis, the model imposed additional boundary conditions on the bolts in the connection. These additional boundary conditions turned the washer/nut assembly portion of the bolts into a fixed connection. The finite element modeling program required these boundary conditions to allow for proper pretensioning of the connections' bolts. During later analysis steps, the program removed the boundary conditions on the bolts, eliminating any effect they would have on the final results of the model.

Lastly, the model requires several constraints that define how the boundary conditions affect the model. Specifically, a coupling constraint is used to attach each reference point to the corresponding cross-section where the reference point is located. A coupling constraint takes the translations and rotations experienced by the reference point and extrapolates them to each of the finite element nodes on the associated surface. After these three coupling constraints, a fourth constraint occurs between the column surface and the edge of the shear tab in each model. The fourth constraint is a tie constraint and forces the finite element nodes along the shear tab's edge to have the same translations and rotations as the nodes on the column's surface. This fourth restraint reproduces the weld that connects the column and shear tab together.
5.5 Bolt Pretensioning and Connection Loading

To bring the various model parts into solid contact, the bolts in the model must be pretensioned like they would be in real connection. Similar to the experimental connections, the theoretical connection finite element models use a temperature differential to pretension the bolts. Equation (4-5) gives an approximate temperature differential to achieve the required pretension force. The value is approximate because equation (4-5) does not take into account the deflections in the shear tab and beam web as the bolt shank length decreases. The required pretension force in the bolts came from AISC (2005a). Table 5-12 summarizes the temperature differentials applied to the bolts in the theoretical connection models.

Table 5-12: Theoretical Models' Bolt Pretension Temperatures

<table>
<thead>
<tr>
<th>Connection</th>
<th>Temp. Diff. (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-520</td>
</tr>
<tr>
<td>2</td>
<td>-500</td>
</tr>
</tbody>
</table>

Table 5-13: Theoretical Models' Beam Tip Displacement

<table>
<thead>
<tr>
<th>Connection</th>
<th>Tip Disp. (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>10.81</td>
</tr>
</tbody>
</table>

The final step in creating the finite element models is applying deflections. Section 5.4 mentioned U2 was a restricted DoF for the end of the beam away from the connection. DoF U2 is the vertical DoF used to apply the necessary displacements. Since the experiments administered their deflections by applying deflections to the beam end, the theoretical connection models do the same. Table 5-13 summarizes the tip displacements imposed on the theoretical connection models. The desired final rotation for both of the theoretical connections was 0.1 radians.
5.6 Results

The finite element models for the theoretical models used information taken from the experimental connection models. The models used variations of this information to create moment-rotation response envelopes for both connections. The following sections summarize the results of the finite element models for the theoretical connections.

5.6.1 Theoretical Connection 1

It is not practical to measure the exact pretension present in every bolt installed in a building. Therefore theoretical connection 1’s finite element model examined a range of values for the model’s coefficient of friction (CoF). As mentioned earlier, changing the CoF is a simple way of observing the effect that higher and lower bolt pretension forces have on the connection's deflection response.

Therefore the biggest influence that the experimental connection models had on the theoretical connections was determining the range of values for the CoF used in the theoretical connection models. The theoretical connection models took the average CoF used in the experimental connection finite element models of Liu and Astaneh-Asl’s Specimens 1A and 2A as well as Crocker and Chambers’ Specimens I, II, and III. Therefore the model started with a CoF value of 0.36 for theoretical connection 1. Additional models with CoF values 20% higher and lower were also analyzed to create the moment-rotation curve envelope seen in Figure 5-3 and Figure 5-4. The CoF from the finite element model of Larson’s Specimen 1 was not included because the results of that connection could not be properly reproduced.

Figure 5-5 displays initial stiffness for all three models of theoretical connection 1. In addition, Figure 5-5 displays a trendline for the model with the average CoF and a plot of the stiffness
AISC considers to be the upper limit for pinned connections, 2EI/L, where I and L refer to the connecting beam.

Figure 5-3: Theoretical Connection 1 Model Results

Figure 5-4: Theoretical Connection 1 Close Up

Figure 5-5: Theoretical Connection 1 Initial Stiffness

All three variations of theoretical connection 1's models predict the beam end coming into bearing with the columns flange at rotations of approximately 0.056 radians. This bearing between the beam and column moves the neutral axis of the connection towards the bearing surface (Crocker and Chambers, 2004). As the neutral axis moves further away from the lower bolts in the connection, those bolts experience larger force transfer through the bolts and the bolt holes. Examining the stress contours of the shear tab, Figure 5-6, a large amount of the shear
tab is yielding around the lowest bolt hole. By comparison, the stress contours for the bolt, Figure 5-7, shows that yielding has begun in the shank but has not proceeded through the entire diameter. This suggests that the likely failure mode this connection would experience in a full-scale test would be failure of the shear tab around the lowest bolt hole.

So the finite element model's inability to predict component failure is the reason for the high end moment the model experiences at the end of the analysis, see Figure 5-3. Since the model cannot predict and reproduce the fracture of the shear tab, it cannot redistribute the stresses in the finite elements around the bolt holes that should have failed. These high stresses located so far from the connection's neutral axis cause the high end moments observed.

5.6.2 Theoretical Connection 2

Similar to theoretical connection 1, theoretical connection 2’s finite element model examined a range of values for the model’s CoF. The theoretical connection 2 model took the average CoF used in the experimental connection finite element models of Specimens 1A, 2A, I, II, and III. Therefore, the model started with a CoF value of 0.36 for theoretical connection 2. Additional models with CoF values 20% higher and lower were also analyzed to create the moment-rotation
curve envelope seen in Figure 5-8 and Figure 5-9. The CoF from the finite element model of Larson’s Specimen 1 was not included because the results of that connection could not be properly reproduced.

Figure 5-8: Theoretical Connection 2 Model Results

Figure 5-9: Theoretical Connection 2 Close Up

Figure 5-10 displays initial stiffness for all three models of theoretical connection 2. In addition, Figure 5-10 displays a trendline for the model with the average CoF and a plot of the stiffness AISC considers to be the upper limit for pinned connections, 2EI/L, where I and L refer to the connecting beam.

$$y = 47264x + 8.7546$$

$$R^2 = 0.9779$$

Figure 5-10: Theoretical Connection 2 Initial Stiffness
All three variations of theoretical connection 2's models predict the beam end coming into bearing with the columns flange at rotations of approximately 0.0575 radians. This bearing between the beam and column moves the neutral axis of the connection towards the bearing surface (Crocker and Chambers, 2004). As the neutral axis moves further away from the lower bolts in the connection, those bolts experience larger force transfer through the bolts and the bolt holes. Examining the stress contours of the shear tab, Figure 5-6, a large amount of the shear tab is yielding around the lowest bolt hole. By comparison, the stress contours for the lowest bolt, Figure 5-7, shows that yielding has begun in the shank but has not proceeded through the entire diameter. This suggests that the likely failure mode this connection would experience in a full-scale test would be failure of the shear tab around the lowest bolt hole.

So the finite element model's inability to predict component failure is the reason for the high end moment the model experiences at the end of the analysis, see Figure 5-8. Since the model cannot predict and reproduce the fracture of the shear tab, it cannot redistribute the stresses in the finite
elements around the bolt holes that should have failed. These high stresses located so far from the connection's neutral axis cause the high end moments observed.

5.6.3 Theoretical Model Summary and Findings

The final results of the theoretical connection models show finite element modeling's strong ability to predict the response of a connection. The experimental connection models have already shown that finite element modeling can very accurately reproduce experimental connection response. Therefore, the results of the theoretical connection models are very likely the responses these connections would exhibit during full-scale testing.

Both of the theoretical connection models predicted several events for each connection. Events predicted by the models included initial stiffness, bolt slippage, beam-column bearing, and potential failure states of the connection.

Both of the models predict bearing of the beam on the column at approximately 0.056 radians of rotation. As mentioned earlier, the bearing experienced by the connections is one of the factors contributing to their believed failure mode. It's important to note that the minimum expected rotation of special moment frames in building design is 0.04 radians (AISC, 2005c). Therefore both of these connections would be able to contribute to the lateral load resisting system before bearing occurred and potential excessive yielding and failure of the shear tab.
Chapter 6: Conclusions

6.1 Conclusions

The creation of ABAQUS finite element models representing the experimental shear tab connections of Larson, Liu, Astaneh-Asl, Crocker, and Chambers has shown that finite element modeling is able to accurately reproduce the results of experimental testing. All models predicted the onset of bolt slippage in addition to specific events that occurred in each connection. Specimen 1A's model correctly showed the effect of column stiffener plates on the connection's initial stiffness. Other events include bearing of the beam on the column in Specimen 2A, as well as the lack of shear tab yielding or fracture in Larson's specimen 1, and bolt failure in Crocker and Chambers’ Specimen III.

The finite element models were also able to very closely match the initial stiffness of both Liu and Astaneh-Asl's connections as well as Crocker and Chambers’ connections. All of the models predicted initial stiffness values within 2-12% of the values observed by the researchers during testing. This indicates that the finite element models have a high degree of both precision and accuracy when analyzing the initial stiffness of the connections.

Modeling of these connections was generally straightforward with few major sources of uncertainty. In the experimental connections, the largest factor affecting the response of the connection was the coefficient of friction and by association the pretension force in the bolts. Most of the other factors that greatly affect the response of a connection, such as material properties, member sizes, and connection geometry, can be easily measured or recorded.
However individual bolt pretension and specific friction coefficient values are generally unreported.

The accuracy with which finite element modeling was able to reproduce the experimental connection data, predict connection specific events, and calculate initial stiffness, shows that it can be a valid technique for finding the response of other theoretical connections. Connection characteristics such as bolt slippage, initial stiffness, bearing between connected elements, and possible failure modes for the connection can all be predicted by finite element modeling.

This allows for the possibility for future researchers to explore their theories before pursuing expensive and time-intensive full scale experimentation. Finite element modeling can be used to analyze connections to study their expected failure modes and deflection response.

Recent research by others on shear tab connections, has shown that the lateral stiffness of these connections can greatly add to the lateral stiffness of the resisting system. The addition of these PR shear tab connections to the building analysis can reduce inter-story drift by almost 22% when compared to the results of an analysis with assumptions of perfectly pinned shear tab connections, while reducing the moment experienced by the beams in the lateral resisting system. In the future, when more "simple" gravity connections have been analyzed, engineers can begin to account for the rotational stiffness of these connections in the design of lateral load resisting systems, instead of neglecting the connection's strength as in current design practice. Resulting in light and cheaper designs overall.
6.2 Suggestions for Future Research

While this thesis attempted to address every issue that arose, additional research is needed to improve the conclusions drawn. The following list indicates some of the potential research opportunities for future students looking to further this study.

- Contact P.C. Larson to obtain more detailed information on the materials, dimensions, and geometry used in his Specimen 1. With this new information attempt to more accurately reproduce the results of Larson's experiment using finite element modeling.
- Use additional ABAQUS features such as Restart and Model Change to more closely reproduce the experimental results of Crocker and Chambers' Specimen III. Restart and Model Change present ways to model the effect of the disappearance of the bolt from the connection.
- Use new research in ABAQUS damage mechanics to reproduce the Crocker and Chambers' Specimen 3 results and properly predict and recreate the bolt failure in the connection.
- Continued production of moment-rotation curves for theoretical gravity connections. To allow future designers the confidence to use gravity connections as PR connections, more data is needed on the response of gravity connections to rotational demands. In addition to shear tab connections, other types of simple connections can be investigated, including single and double web angle connections and weak axis shear tab connections.
- A sensitivity analysis on the effect that material properties, connection configuration, and other factors have on the response of shear tab or other gravity connections.
References


American Society of Civil Engineers (ASCE) (2010). ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers


Larson, P.C. (1996) “The design and behavior of bolted beam-to-column frame connections under cyclical loading,” thesis, presented to the University of Texas at Austin, TX, in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.


Ruffley, D.J. (2011) "A finite-element approach for modeling top-and-seat angle components and moment connections,"


Appendix A

Connection Loading
The following section explains how the loadings used in the design of the two theoretical connections were calculated. Table 5-1 and Table 5-2 summarize the loadings taken from ASCE 7-10 explained in more detail in section 5.1. The total live and dead loads are then increased according to LRFD's procedure and ASCE 7-10's load combinations (2010).

Table 5-1: Theoretical Connections' Live Loadings

<table>
<thead>
<tr>
<th>Connection 1</th>
<th>Conn. 1</th>
<th>Conn. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Office Space</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>Storage Space</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Carpet*</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>Flooring</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td>Ceiling</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>Partitions (moveable)</td>
<td>4</td>
<td>-</td>
</tr>
<tr>
<td>Partitions*</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>Lighting*</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Ventilation</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Total</td>
<td>64</td>
<td>110</td>
</tr>
</tbody>
</table>

* Amount not listed in ASCE 7

Table 5-2: Theoretical Connections' Dead Loading

<table>
<thead>
<tr>
<th>Connection 1</th>
<th>Conn. 1</th>
<th>Conn. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Decking (2 in. 19 ga.)*</td>
<td>2.55</td>
<td>2.55</td>
</tr>
<tr>
<td>Concrete (4 in. thick)</td>
<td>49</td>
<td>49</td>
</tr>
<tr>
<td>Total</td>
<td>51.55</td>
<td>51.55</td>
</tr>
</tbody>
</table>

*Manufacturer listed value (Corrugated Metal, 2012)

These factored loadings are converted into distributed loads applied along the filler beams, based on the filler beam spacing. The filler beams are spaced at 7.5 feet and are 30 feet long. Case 1 from Table 3-23 in AISC is used to find the maximum moment experienced by the filler beams (2005b).
Equation F2-1 from AISC is used to determine the minimum plastic section moduli for the filler beams (2005a). Additionally, deflection equations from case 1 of Table 3-23 in AISC can be used to give minimum moment of inertias for the filler beams (2005b). The allowable deflection, \( \Delta \), is from ASCE 7-10's equation of \( \Delta_{\text{ALLOW}} = L/360 \) which is equal to 1 inch for both connections (2010). From these minimum values, trial sizes for the filler beams are then selected. The filler beams are then connected to the gravity beam of each connection, turning the distributed load and self-weight of the trial beam sizes into three point loads along the length of the gravity beam.

<table>
<thead>
<tr>
<th>Connection 1</th>
<th>Connection 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w_1 = 164.26 \text{ psf} \cdot 7.5' = 1,231.95 \text{ plf} )</td>
<td>( w_2 = 237.86 \text{ psf} \cdot 7.5' = 1,783.95 \text{ plf} )</td>
</tr>
<tr>
<td>( w_1 = 1.232 \text{ klf} )</td>
<td>( w_2 = 1.784 \text{ klf} )</td>
</tr>
</tbody>
</table>

\[
M_u = M_{\max} = \frac{w \cdot L^2}{8}
\]

\[
M_{\max, \text{filler},1} = \frac{1.232 \text{ klf} \cdot 30^2}{8} = 139 \text{ k} - \text{ft}
\]

\[
M_{\max, \text{filler},1} = 1663 \text{ k} - \text{in}
\]

\[
M_{\max, \text{filler},2} = \frac{1.784 \text{ klf} \cdot 30^2}{8} = 201 \text{ k} - \text{ft}
\]

\[
M_{\max, \text{filler},2} = 2408 \text{ k} - \text{in}
\]

<table>
<thead>
<tr>
<th>Connection 1</th>
<th>Connection 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_u \leq 0.9 \cdot M_n = 0.9 \cdot M_p = 0.9 \cdot F_y \cdot Z_x )</td>
<td>( Z_{\min} = \frac{M_u}{0.9 \cdot F_y} )</td>
</tr>
<tr>
<td>( Z_{\min, \text{filler},1} = \frac{1663 \text{ k} - \text{in}}{0.9 \cdot 50 \text{ ksi}} = 36.96 \text{ in}^3 )</td>
<td>( Z_{\min, \text{filler},2} = \frac{2408 \text{ k} - \text{in}}{0.9 \cdot 50 \text{ ksi}} = 53.51 \text{ in}^3 )</td>
</tr>
</tbody>
</table>

\[
\Delta = \frac{5 \cdot w \cdot L^4}{384 \cdot E \cdot I}
\]

\[
I_{\min, \text{filler},2} = 774 \text{ in}^4
\]

\[
I_{\min, \text{filler},2} = 1122 \text{ in}^4
\]
The maximum moments occurring in the gravity beams were calculated using the moment equations in cases 7 and 9 from Table 3-23 in AISC (2005b). Combining these moment values with the yielding limit state equation from AISC, minimum plastic section moduli were calculated (2005a).

<table>
<thead>
<tr>
<th>Connection 1</th>
<th>Connection 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Filler Beam: W21x44</td>
<td>Filler Beam: W21x55</td>
</tr>
<tr>
<td>$P_1 = (1.232 \text{ klf } + 0.044 \text{ klf}) \cdot 30' = 38.28 \text{ k}$</td>
<td>$P_2 = (1.784 \text{ klf } + 0.055 \text{ klf}) \cdot 30' = 55.17 \text{ k}$</td>
</tr>
<tr>
<td>$P_1 \approx 39 \text{ k}$</td>
<td>$P_2 \approx 56 \text{ k}$</td>
</tr>
</tbody>
</table>

After obtaining the minimum plastic section moduli, the deflection equations from AISC's Table 3-23 were used to find the minimum moment of inertia the beams in the theoretical connection would require. In these equations, \( P_{LL} \) stands for the point load from the factored live load only. The allowable deflection, \( \Delta \), is from ASCE 7-10's equation of \( \Delta_{\text{ALLOW}} = \frac{L}{360} \) which is equal to 1 inch for both connections (2010).
These minimum plastic section moduli and moments of inertia led to trial beam sizes for the gravity beams in the shear tab connections. Additional calculations were performed to adjust the initial loadings, listed above, to reflect the new beam sizes. This involved including the trial beam's dead weight to the moments created by the filler beam's point loads. Connection 1 used W 21x44 filler beams while connection 2 used W21x55 filler beams. Then the connection beams self-weight was added to the moments calculated above.

<table>
<thead>
<tr>
<th>Connection 1</th>
<th>Connection 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity Beam: W24x76</td>
<td>Gravity Beam: W27x94</td>
</tr>
<tr>
<td>$M_{u,1} = \left( \frac{39 \cdot 30'}{2} + \frac{0.076 \cdot 1.2 \cdot 30^2}{8} \right) \cdot 12$</td>
<td>$M_{u,2} = \left( \frac{56 \cdot 30'}{2} + \frac{0.094 \cdot 1.2 \cdot 30^2}{8} \right) \cdot 12$</td>
</tr>
<tr>
<td>$M_{u,1} = 7,144 \text{ k-in}$</td>
<td>$M_{u,2} = 10,233 \text{ k-in}$</td>
</tr>
</tbody>
</table>

The loadings applied to the columns in the connections came from estimates of each floor's weight. The load on the columns is then a summation of the weight of the floors above. Since
no dead and lives load estimates were created for the roof, the calculations assume the roof weighs half the weight of the office floor. No trial sizes were selected for the columns.

<table>
<thead>
<tr>
<th>Office Floor:</th>
<th>Roof:</th>
<th>Storage:</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_0 = 0.076 klf \cdot \frac{30\gamma}{2} \cdot 1.2 \cdot 2 + 39 k \cdot 4 \approx 159 k$</td>
<td>$P_R = \frac{1}{2} \cdot 159 k \approx 80 k$</td>
<td>$P_S = 0.094 klf \cdot \frac{30\gamma}{2} \cdot 1.2 \cdot 2 + 56 k \cdot 4 \approx 227 k$</td>
</tr>
</tbody>
</table>

$P_{u,1} = 80 + 159 \cdot 2 = 398 k$

$P_{u,2} = 80 + 159 \cdot 3 + 227 = 784 k$

The shear tabs in the theoretical connections are required to resist the shear transferred from the gravity beam to the column. The value of the shear at each connection is the result of the point loads applied to the gravity beam and the self-weight of the beam. The bolts in the connection must also collectively resist the transferred shear, so each bolt is assumed to resist an equal amount of the shear. The design assumed Connection 1 would require three bolts while Connection 2 would require four bolts.

<table>
<thead>
<tr>
<th>Connection 1</th>
<th>Connection 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_{U,1} = 37 k \cdot 3 \cdot \frac{1}{2} + 0.076 klf \cdot 1.2 \cdot 30' \cdot \frac{1}{2}$</td>
<td>$V_{U,2} = 54 k \cdot 3 \cdot \frac{1}{2} + 0.094 klf \cdot 1.2 \cdot 30' \cdot \frac{1}{2}$</td>
</tr>
<tr>
<td>$V_U = 57 k$</td>
<td>$V_U = 83 k$</td>
</tr>
<tr>
<td>$R_{U,1} = \frac{57 k}{3 \text{ bolts}} = 19 k/\text{bolt}$</td>
<td>$R_{U,2} = \frac{83 k}{4 \text{ bolts}} = 21 k/\text{bolt}$</td>
</tr>
</tbody>
</table>
Appendix B
Connection Limit State Checks
The following section explains how the design checks undertaken to ensure the theoretical connections met all current limit state checks. The design calculations utilize some of the design aids AISC makes available in its design manual (2005b).

First the trial beams sizes are checked. AISC states that "for guidance in determining the appropriate sections of this chapter to apply, Table User Note F1.1 may be used" (2005a). Table User Note F1.1 bases the appropriate sections on the slenderness classification of the beam's web and flanges. From AISC Table B4.1, the webs and flanges of both connection's beams fall into the compact slenderness classification (2005a). Therefore, only the limit states of yielding and lateral-torsional bucking (LTB) need to be considered in the beam design. The yielding limit state check follows equation F2-1 provided by AISC, while the LTB limit state makes use of the design guide Table 3-2 in AISC (2005a and b).

<table>
<thead>
<tr>
<th></th>
<th>Connection 1</th>
<th>Connection 2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Yielding</strong></td>
<td>( \Phi M_n = \Phi \cdot F_y \cdot Z_x )</td>
<td></td>
</tr>
<tr>
<td>( \Phi M_{n,1} )</td>
<td>0.9 \cdot 50 \text{ ksi} \cdot 200 \text{ in}^3</td>
<td>0.9 \cdot 50 \text{ ksi} \cdot 278 \text{ in}^3</td>
</tr>
<tr>
<td>( \Phi M_{n,1} )</td>
<td>9,000 \text{ k} - \text{ in}</td>
<td>12,510 \text{ k} - \text{ in}</td>
</tr>
<tr>
<td><strong>LTB</strong></td>
<td>( \Phi M_n = C_B \cdot (\Phi M_p - BF \cdot (L_B - L_P)) )</td>
<td></td>
</tr>
<tr>
<td>( \Phi M_{n,1} )</td>
<td>1 \cdot \left( 750 \text{ k} - \text{ ft} - 22.5 \text{k} \cdot \left( 7.5' - 6.78' \right) \right)</td>
<td>1 \cdot \left( 1,040 \text{ k} - \text{ ft} - 28.8 \text{k} \cdot \left( 7.5' - 7.49' \right) \right)</td>
</tr>
<tr>
<td>( \Phi M_{n,1} )</td>
<td>734 \text{ k} - \text{ ft} = 8,806 \text{ k} - \text{ in}</td>
<td>1,040 \text{ k} - \text{ ft} = 12,477 \text{ k} - \text{ in}</td>
</tr>
</tbody>
</table>

The design of the columns in the connections used the design guide Table 4-1 available in AISC (2005b). To use this guide, the design assumed the columns were braced at each floor with an appropriate effective length factor of 1.
The bolts used in the theoretical connections are made of A325 grade steel and the threads of the bolt are assumed to be in the shear plane of the connection. AISC provides design aid Table 7-1 to calculate the capacity of various bolt sizes (2005b). For the theoretical connections, 7/8 inch diameter bolts are used. Table 7-1 indicates that the capacity of 7/8 inch diameter, A325 bolts in single shear is 21.6 kips with the thread included in the shear plane. Therefore Connection 1 requires three 7/8 inch diameter bolts while Connection 2 requires at least four 7/8 inch diameter bolts.

Part 10 of AISC's construction manual deals with shear tab connections and lists the required limit states that the shear tab and bolts must meet (2005b). AISC places multiple restrictions on the dimensions of the connection. Several of the restrictions important to the size of the connection are (AISC, 2005b):

- The distance from the bolt line to the weld line must be equal to or less than 3 1/2 in.
- The horizontal edge distance must be equal to or greater than 2\(d_b\) for both the plate and the beam web
- The vertical edge distance must satisfy AISC Specification Table J3.4 requirements
- Either the plate or the beam web must satisfy \(t \leq d_b/2 + 1/16\) in.
- The minimum plate length be one-half the T-dimension of the beam to be supported

<table>
<thead>
<tr>
<th>Connection 1</th>
<th>Connection 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>(K \cdot L_1 = 1 \cdot 12' = 12')</td>
<td>(K \cdot L_2 = 1 \cdot 15' = 15')</td>
</tr>
<tr>
<td>(\Phi P_{n,1} = 410 \text{ k})</td>
<td>(\Phi P_{n,2} = 809 \text{ k})</td>
</tr>
</tbody>
</table>
Following these restrictions, the trial plate size is 12" x 4" x 1/2" for both connections. Therefore Connection 1 has a 4 inch spacing between bolts with a 2 inch vertical edge distance while Connection 2 has a 3 inch bolt spacing with a 1.5 inch vertical edge distance. AISC also requires that the limit states of bolt shear, block shear rupture, bolt bearing, shear tab yielding, and shear tab rupture be met (AISC, 2005b). Except for bolt shear which was addressed earlier, all the limit states were checked using equations in Chapter J of AISC's specification (2005a). From these equations the design determined that block shear in the beam web is not a concern of the design. Also bearing and tearout of the bolts in the beam web were not concerns. Results of the other limit states can be seen below.
<table>
<thead>
<tr>
<th>Bolt Bearing</th>
<th>Connection 1</th>
<th>Connection 2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bolt</strong></td>
<td>$\Phi R_n = \Phi \cdot 1.2 \cdot L_c \cdot t \cdot F_u \leq \Phi \cdot 2.4 \cdot d \cdot t \cdot F_u$</td>
<td>$L_c = 3 \text{ in.} - \frac{7}{8} \text{ in.} = 2.125 \text{ in.}$</td>
</tr>
<tr>
<td>$L_c = 4 \text{ in.} - \frac{7}{8} \text{ in.} = 3.125 \text{ in.}$</td>
<td>$L_c \geq 2d \therefore \Phi R_{n,1} = \Phi \cdot 2.4 \cdot d \cdot t \cdot F_u$</td>
<td>$L_c \geq 2d \therefore \Phi R_{n,1} = \Phi \cdot 2.4 \cdot d \cdot t \cdot F_u$</td>
</tr>
<tr>
<td>$\Phi R_{n,1} = 0.75 \cdot 2.4 \cdot \frac{7}{8} \text{ in.} \cdot 0.44 \text{ in.} \cdot 58 \text{ ksi}$</td>
<td>$\Phi R_{n,2} = 45.68 \text{ k/center bolt}$</td>
<td>$\Phi R_{n,2} = 45.68 \text{ k/center bolt}$</td>
</tr>
<tr>
<td>$\Phi R_{n,1} = 45.68 \cdot (1 + 40.72 \cdot 2)$</td>
<td>$\Phi R_{n,1} \approx 127 k$</td>
<td>$\Phi R_{n,1} \approx 147 k$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Block Shear</th>
<th>$R_n = 0.6 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.6 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_{gv} = 0.5 \text{ in.} \cdot 10 \text{ in.} = 5 \text{ in}^2$</td>
<td>$A_{gv} = 0.5 \text{ in.} \cdot 10.5 \text{ in.} = 5.25 \text{ in}^2$</td>
</tr>
<tr>
<td>$A_{nv} = 0.5 \text{ in.} \left(10 \text{ in.} - 2.5 \left(\frac{7}{8} \text{ in.} + \frac{1}{16} \right)\right)$</td>
<td>$A_{nv} = 0.5 \text{ in.} \left(10.5 \text{ in.} - 3.5 \left(\frac{7}{8} \text{ in.} + \frac{1}{16} \right)\right)$</td>
</tr>
<tr>
<td>$= 3.83 \text{ in}^2$</td>
<td>$= 3.609 \text{ in}^2$</td>
</tr>
<tr>
<td>$A_{nt} = 0.5 \text{ in.} \left(\frac{3}{4} \text{ in.} - \frac{1}{2} \left(\frac{7}{8} \text{ in.} + \frac{1}{16} \right)\right)$</td>
<td>$A_{nt} = 0.5 \text{ in.} \left(\frac{3}{4} \text{ in.} - \frac{1}{2} \left(\frac{7}{8} \text{ in.} + \frac{1}{16} \right)\right)$</td>
</tr>
<tr>
<td>$= 0.64 \text{ in}^2$</td>
<td>$= 0.64 \text{ in}^2$</td>
</tr>
<tr>
<td>$\Phi R_{n,1} = 0.75 \cdot 133.12 k \approx 100 k$</td>
<td>$\Phi R_{n,2} = 0.75 \cdot 137.92 k \approx 103 k$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Yielding</th>
<th>$\Phi R_n = \Phi \cdot 0.6 \cdot F_y \cdot A_g$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Phi R_{n,1} \approx 130 k$</td>
<td>$\Phi R_{n,2} \approx 130 k$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rupture</th>
<th>$\Phi R_n = \Phi \cdot 0.6 \cdot F_u \cdot A_{nv}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Phi R_{n,1} \approx 120 k$</td>
<td>$\Phi R_{n,2} \approx 108 k$</td>
</tr>
</tbody>
</table>
Appendix C

Black and White Figures
Figure 4-16: Larson Specimen 1 Model Results
Figure 4-17: Larson Specimen 1 Initial Stiffness

\[ y = 14.92x \]

\[ y = 21.432x + 0.75 \]

\[ R^2 = 0.9561 \]
Figure 4-19: Liu and Astaneh-Asl Specimen 1A Model Results
Figure 4-20: Liu and Astaneh-Asl Specimen 1A Initial Stiffness
Figure 4-22: Liu and Astaneh-Asl Specimen 2A Model Results
Figure 4-23: Liu and Astaneh-Asl Specimen 2A Close Up
Figure 4-24: Liu and Astaneh-Asl Specimen 2A Initial Stiffness
Figure 4-26: Crocker and Chambers Specimen I Model Results
Figure 4-27: Crocker and Chambers Specimen I Initial Stiffness

\[ y = 27193x \]

\[ R^2 = 0.952 \]

\[ y = 29263x + 7.281 \]
Figure 4-29: Crocker and Chambers Specimen II Model Results
Figure 4-30: Crocker and Chambers Specimen II Initial Stiffness

The graph illustrates the relationship between moment and rotation for Specimen II, FEM Conn. II, and AISC Pinned Stiffness. The equations and their respective $R^2$ values are:

- **Specimen II**: $y = 51907x$
- **FEM Conn. II**: $y = 54369x + 18.088$, $R^2 = 0.9342$
- **AISC Pinned Stiffness**: $y = 51907x$
Figure 4-32: Crocker and Chambers Specimen III Model Results
Figure 4-33: Crocker and Chambers Specimen III Initial Stiffness

\[ y = 142313x \]

\[ R^2 = 0.9472 \]

\[ y = 138550x + 37.308 \]
Figure 5-3: Theoretical Connection 1 Model Results
Figure 5-4: Theoretical Connection 1 Close Up
Figure 5-5: Theoretical Connection 1 Initial Stiffness

- $y = 36052x + 13.314$
- $R^2 = 0.9538$
Figure 5-8: Theoretical Connection 2 Model Results
Figure 5-9: Theoretical Connection 2 Close Up
Figure 5-10: Theoretical Connection 2 Initial Stiffness

\[ y = 47264x + 8.7546 \]
\[ R^2 = 0.9779 \]
Appendix D

Color Figures
Figure 4-16: Larson Specimen 1 Model Results
Figure 4-17: Larson Specimen 1 Initial Stiffness

\[ y = 14.92x \]

\[ y = 21.432x + 0.75 \]

\[ R^2 = 0.9561 \]
Figure 4-19: Liu and Astaneh-Asl Specimen 1A Model Results
Figure 4-20: Liu and Astaneh-Asl Specimen 1A Initial Stiffness
Figure 4-22: Liu and Astaneh-Asl Specimen 2A Model Results
Figure 4-23: Liu and Astaneh-Asl Specimen 2A Close Up
Figure 4-24: Liu and Astaneh-Asl Specimen 2A Initial Stiffness
Figure 4-26: Crocker and Chambers Specimen I Model Results
Figure 4-27: Crocker and Chambers Specimen I Initial Stiffness

\[ y = 27193x \]

\[ R^2 = 0.952 \]

\[ y = 29263x + 7.281 \]
Figure 4-29: Crocker and Chambers Specimen II Model Results
Figure 4-30: Crocker and Chambers Specimen II Initial Stiffness

Equation for Specimen II:

\[ y = 51907x \]

Equation for FEM Conn. II:

\[ y = 54369x + 18.088 \]

\[ R^2 = 0.9342 \]
Figure 4-32: Crocker and Chambers Specimen III Model Results
Figure 4-33: Crocker and Chambers Specimen III Initial Stiffness

\[ y = 138550x + 37.308 \]
\[ R^2 = 0.9472 \]

\[ y = 142313x \]
Figure 5-3: Theoretical Connection 1 Model Results
Figure 5-4: Theoretical Connection 1 Close Up
Figure 5-5: Theoretical Connection 1 Initial Stiffness

$y = 36052x + 13.314$

$R^2 = 0.9538$
Figure 5-8: Theoretical Connection 2 Model Results
Figure 5-9: Theoretical Connection 2 Close Up
Figure 5-10: Theoretical Connection 2 Initial Stiffness

Theoretical Connection 2 Initial Stiffness

\[ y = 47264x + 8.7546 \]

\[ R^2 = 0.9779 \]