I, Cory W Larkin, hereby submit this original work as part of the requirements for the degree of Master of Science in Civil Engineering.

It is entitled:
A Modified Design Procedure for the Fused Steel Coupling Beam System

Student's name: Cory W Larkin

This work and its defense approved by:

Committee chair: Gian Rassati, Ph.D.
Committee member: Thomas Baseheart, Ph.D.
Committee member: James Swanson, Ph.D.
A Modified Design Procedure for the Fused Steel Coupling Beam System

Master’s Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science in the Department of Civil Engineering

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

By: Cory W. Larkin
Bachelor of Science in Civil Engineering – University of Cincinnati, 2012

Committee Chair: Dr. Gian A. Rassati
Committee Members: Dr. T. Michael Baseheart
Dr. James A. Swanson

Date: 8/20/2013
Abstract

In 2005 and 2012, Dr. Patrick J. Fortney and Steven J. Mitchell worked on the development of the fused coupling beam (FCB) concept and application. Mitchell enhanced Dr. Fortney’s design and obtained more desirable results.

A FCB is a coupling beam that would be found in a coupled core wall system. Unlike a typical coupling beam, the FCB has three sections: two embedded steel beams and one smaller “link” beam that is found at mid-span. This middle link is designed to take all of the damage in the system during a seismic event and then simply be replaced afterwards. Mitchell’s FCB had excellent results; however, yielding occurred in the embedded beams under a 2% rotation of the system. This was an undesirable behavior for the FCB. The scope of work that is presented uses finite element modeling to simulate Mitchell’s beam; based on that model a modified design procedure was created in order to prevent yielding in the embedded beams prior to that of the fuse section. The modified design procedure was then tested using finite element modeling with the same parameters that were found in the validation study of Mitchell’s FCB.

Finite element modeling was performed using Abaqus as the analysis tool. The first models were created per the specifications laid out in Mitchell’s thesis and all later models were created based off the updated design and the parameters from Mitchell’s model (stiffness, material properties, etc.). Results show that the modified design procedure has shifted the first yield of the FCB from the embedded beam into the web of the fuse. This behavior is the desired result for the FCB which enables the link to be replaced if yielding should occur.
Acknowledgments

I would like to thank my advisor, Dr. Gian A. Rassati for his continual support throughout the development of my research. The topic came directly from him and his ideas have been the highlight of my graduate studies. Dr. Rassati was always willing to lend a helping hand during all phases of research and was extremely knowledgeable on the topic. Acting alongside Dr. Rassati was Dr. James A. Swanson. Through our weekly meetings his advice was most welcome and used to better understand certain aspects of my work. His suggestions were often used to better understand certain aspects of finite element modeling and other engineering software.

I would like to thank Dr. T. Michael Baseheart for his willingness to sit on my thesis committee. His comments on my Thesis Proposal were most welcome and helpful. I would also like to thank Steven J. Mitchell for all of his advice and resources. Through him I was able to obtain a better understanding of coupling beams and their applications. He was always willing to meet with me to either explain certain aspects of the FCB or to give me valuable data that was absolutely essential to the completion of my research.

I would like to thank my family for their continued support during my studies. They have worked very hard to help me get to where I am today. Whether it was financial support or emotional support, they provided it all. I am certain without their help I would not be where I am today.
# Table of Contents

List of Tables ................................................................................................................................ vii

List of Figures ................................................................................................................................ viii

List of Symbols ................................................................................................................................ xii

Chapter 1-Introduction to Coupled Core Wall Systems and Coupling Beams............................... 1

Chapter 2-Previous Research and Literature Review ..................................................................... 4

2.1 Overview of Dr. Fortney’s FCB....................................................................................... 4

2.2 Overview of Mitchell’s FCB............................................................................................ 7

2.3 Scope of Current Research ............................................................................................... 9

Chapter 3-Finite Element Modeling ............................................................................................. 10

3.1 Abaqus Part Assembly ................................................................................................... 11

3.2 Abaqus Part Meshing ..................................................................................................... 12

3.3 Abaqus Interactions........................................................................................................ 14

Chapter 4- Modeling of Mitchell’s FCB....................................................................................... 16

4.1 Pre-Tensioning ............................................................................................................... 16

4.2 Stiffness Testing............................................................................................................ 18

4.3 Material Properties ........................................................................................................ 20

4.4 Cyclic Modeling............................................................................................................ 22

4.4.1 Non-Linear Stiffness Modeling .............................................................................. 24

4.5 Results from Mitchell’s FCB Abaqus Modeling............................................................ 25
Chapter 5- Updated Design Methodology for Fuse Coupling Beams ................................................... 30

5.1 Overview of the FCB Design Methodology ............................................................................. 30

5.2 Updated Design Procedure for the FCB .................................................................................. 31

5.2.1 Overall Design of the FCB ................................................................................................. 31

5.2.2 Design of Fuse Link .......................................................................................................... 32

5.2.3 Design of Embedded Beam ............................................................................................... 35

5.2.4 Connection Design .......................................................................................................... 37

Chapter 6- Results for the Updated FCB ....................................................................................... 38

6.1 Scenario One Results ............................................................................................................. 39

6.1.1 Von Mises Criterion .......................................................................................................... 40

6.1.2 Shear Stress ...................................................................................................................... 42

6.1.3 Tensile Stress ................................................................................................................... 44

6.1.4 Plastic Equivalent Strain ................................................................................................. 45

6.1.5 Ultimate Capacities ......................................................................................................... 46

6.1.6 Stress Comparisons ......................................................................................................... 48

6.2 Scenario Two Results ............................................................................................................ 49

6.2.1 Von Mises ......................................................................................................................... 49

6.2.2 Shear Stress ..................................................................................................................... 51

6.2.3 Tensile Stress .................................................................................................................. 53

6.2.4 Plastic Equivalent Strain ................................................................................................. 54
6.2.5 Stress Comparisons........................................................................................................... 55

Chapter 7- Summary and Conclusions ......................................................................................... 56

7.1 Suggestions for Future Research........................................................................................... 57

References.................................................................................................................................... 59

Appendix A – Bolt Pre-Tensioning ............................................................................................ 61

A.1 Modeling and Results............................................................................................................ 61

Appendix B – Stiffness Trials...................................................................................................... 66

B.1 Modeling and Results............................................................................................................ 66

Appendix C – Material Properties ............................................................................................... 73

C.1 Modeling and Results ............................................................................................................ 73

Appendix D – Example Calculation ............................................................................................ 79

D.1 Loading and Material Properties........................................................................................ 79

D.2 Fuse Link Design ................................................................................................................ 80

D.3 Embedded Beam Design ................................................................................................... 84

D.4 Final Design ....................................................................................................................... 89

D.5 Connection Design ............................................................................................................. 90
List of Tables

Table 3.1: Part assembly list and dimensions ................................................................. 11
Table 3.2: Part mesh attributes....................................................................................... 13
Table 4.1: Bolt Pre-tension iteration trials............................................................... 17
Table 6.1: Comparison of fuse link and embedded beams rotation at yield.......... 48
Table 6.2: Comparison of fuse link and embedded beams rotation at yield.......... 55
Table A.1: Flange bolt data for the final iteration in bolt pre-tensioning ................ 63
Table A.2: End plate bolt data for the final iteration in bolt pre-tensioning ............ 64
Table B.1: Data for the non-linear stiffness of the FCB .......................................... 68
Table C.1: Abaqus input data for 1/4” coupon ....................................................... 74
Table C.2: Abaqus input data for 3/8” coupon ....................................................... 75
Table C.3: Abaqus input data for 1/2” coupon ....................................................... 76
Table C.4: Abaqus input data for 5/8” coupon ....................................................... 77
Table D.1: Beam Final Dimensions ......................................................................... 89
List of Figures

Figure 1.1: Reactions for a typical CCW system (Fortney 2005) ................................................................. 1

Figure 1.2: Plan of CCW system (Left) and Elevation A-A (Right) ............................................................ 2

Figure 2.1: Steel coupling beam assembly (Fortney 2005) ......................................................................... 5

Figure 2.2: Fuse coupling beam assembly (Fortney 2005) ....................................................................... 6

Figure 2.3: Fuse coupling beam assembly at an 11% rotation (Fortney 2005) ............................................ 7

Figure 2.4: Fuse coupling beam end-plate bending at 14% rotation (Left) and fuse coupling beam set up (Right) .............................................................................................................................................. 8

Figure 3.1: Final assembly of the FCB in Abaqus ......................................................................................... 12

Figure 3.2: Embedded beam partitioned (Left) and meshed (Right). Swept meshed regions are shown around the bolt holes while structured meshed regions encompass the rest of the part... 14

Figure 4.1: Fully pre-tensioned model ........................................................................................................ 18

Figure 4.2: Trial runs in Abaqus, stiffness testing ....................................................................................... 19

Figure 4.3: Interpolated stiffness curves superimposed onto 2% rotation data ............................................ 20

Figure 4.4: Abaqus results at 14% rotation with true-stress/strain defined .................................................. 21

Figure 4.5: Preliminary cyclic results ........................................................................................................ 22

Figure 4.6: Non-linear stiffness results ....................................................................................................... 23

Figure 4.7: Non-linear spring usage in Abaqus ......................................................................................... 24

Figure 4.8: Representation of non-linear stiffness of spring ....................................................................... 25

Figure 4.9: Shear stress distribution, S23 .................................................................................................. 26
Figure 4.10: Tensile stress distribution, S33 ................................................................. 26

Figure 4.11: Von Mises stress distribution in the web of the FCB ................................ 27

Figure 4.12: Von Mises stress distribution in the flanges of the FCB .......................... 28

Figure 4.13: Distribution of plastic strain, PEEQ .......................................................... 29

Figure 5.1: Elevation view of a typical FCB ............................................................... 31

Figure 5.2: Fuse link dimensions ............................................................................... 32

Figure 5.3: Elevation view of FCB end plates (Left) and top view of flange splice plates (Right) .......................................................................................................................... 37

Figure 6.1: Deformed shape of scenario one ............................................................... 38

Figure 6.2: Deformed shape of scenario two .............................................................. 39

Figure 6.3: Von Mises state of stress in the embedded beam flanges ................................ 40

Figure 6.4: Von Mises state of stress in the fuse link web ........................................ 41

Figure 6.5: Von Mises state of stress in the embedded beam web .............................. 42

Figure 6.6: Shear stress, S23, of the fuse link ............................................................. 43

Figure 6.7: Shear stress, S23, of the embedded beam .................................................. 44

Figure 6.8: Tensile stress, S33, of the embedded beam .............................................. 45

Figure 6.9: Plastic equivalent strain at a 10.7% chord rotation .................................. 46

Figure 6.9: Force-Displacement of updated fuse and Mitchell’s fuse .......................... 47

Figure 6.10: Force-Displacement of updated fuse vs. 2nd updated fuse ..................... 48

Figure 6.11: Von Mises state of stress in the fuse link web ......................................... 50
Figure 6.12: Von Mises state of stress in the embedded beam web................................. 50

Figure 6.13: Von Mises state of stress in the embedded beam flanges ............................ 51

Figure 6.14: Shear stress, S23, of the fuse link.............................................................. 52

Figure 6.15: Shear stress, S23, of the embedded beam ................................................ 52

Figure 6.16: Tensile stress, S33, of the embedded beam................................................ 53

Figure 6.17: Plastic equivalent strain at a 5% end rotation ........................................ 54

Figure A.1: Stress distribution in bolt shaft................................................................. 62

Figure A.2: Pre-tension force versus Abaqus steps.................................................... 65

Figure B.1: Non-Linear Stiffness.................................................................................. 67

Figure B.2: Non-Linear Stiffness.................................................................................. 72

Figure C.1: Original 1/4” coupon data versus true stress/strain values...................... 74

Figure C.2: Original 3/8” coupon data versus true stress/strain values...................... 75

Figure C.3: Original 1/2” coupon data versus true stress/strain values...................... 76

Figure C.4: Original 5/8” coupon data versus true stress/strain values...................... 77

Figure D.1: Moment and shear transfer between fuse link (Right) and embedded beam (Left). 89

Figure D.2: Fuse (Left) and embedded (Right) beam notations.................................. 89

Figure D.3: Bolt spacing and edge distances............................................................... 94

Figure D.4: Block shear case one .............................................................................. 99

Figure D.5: Block shear case two .............................................................................. 100

Figure D.6: Block shear case three .......................................................................... 101
List of Symbols

$A_{gv}$ = Gross Area Subject to Shear, in$^2$

$A_{lw}$ = Link Web Area (Excluding Flanges), in$^2$

$A_n$ = Net Area of Connecting Part, in$^2$

$A_{nt}$ = Area in Tension of Connecting Part, in$^2$

$A_{nv}$ = Area in Shear of Connecting Part, in$^2$

$b_{ff}$ = Fuse Flange Width, in

$b_{fe}$ = Embedded Beam Flange Width, in

$b_{sp}$ = Width of Stiffener Plate, in

$C_a$ = Ratio of Required Strength to Available Strength

$d$ = Depth of Member or Bolt Diameter, in

$e$ = Length of Fuse Link, in

$F_y$ = Yield Stress, ksi

$F_u$ = Ultimate Stress, ksi

$E$ = the modulus of elasticity, ksi

$h_{we}$ = Embedded Beam Web Height, in

$h_{wf}$ = Fuse Web Height, in

$I_{xx}$ = Moment of Inertia, in$^4$

$I_{xx,net}$ = Net Moment of Inertia, in$^4$

$K$ = Stiffness, K-in/rad
$l_c =$ Clear Distance, in

$L_{emb} =$ Length of Embedded Beam, in

$M =$ Moment at Wall/Beam Interface, K-in

$M_{nf} =$ Nominal Flexural Strength, K-in

$M_{nf} =$ Nominal Flexural Strength of Link, K-in

$M_{pf} =$ Plastic Flexural Strength of Link, K-in

$\phi M_{pf} =$ Design Plastic Flexural Strength of FCB Link, k-in

$M_{fr} =$ Moment Transferred from Fuse Link to Embedded Beam, K-in

$M_u =$ Required Flexural Strength, K-in

$M_{wall} =$ Moment of FCB at the Wall Face, K-in

$M_{y,net} =$ Moment at Yielding of the Net Section, K-in

$P_{M-Conn} =$ Force at Moment Connection, K

$P_{Flange} =$ Force on Flange of Embedded Beams, K

$P_{Splice} =$ Force at Flange Splice Plates, K

$P_u =$ Required Axial Strength, K

$P_y =$ Axial Yield Strength, K

$\phi r_n =$ Design Capacity of a Single Bolt, K

$\phi R_n =$ Design Capacity of a Connection, K

$R_y =$ Ratio of the Expected Yield Stress to the Specified Minimum

$S23 =$ Shear Stress, ksi
\[ S_{33} = \text{Tensile/Compressive Stress, ksi} \]

\[ S_e = \text{Section Modulus, in}^3 \]

\[ S_{\text{net}} = \text{Net Section Modulus, in}^3 \]

\[ t = \text{Thickness of Plate, in} \]

\[ t_{fe} = \text{Embedded Beam Flange Thickness, in} \]

\[ t_{ff} = \text{Fuse Flange Thickness, in} \]

\[ t_{sp} = \text{Thickness of Stiffener Plate, in} \]

\[ t_{we} = \text{Embedded Beam Web Thickness, in} \]

\[ t_{wf} = \text{Fuse Web Thickness, in} \]

\[ \Delta T = \text{Change in Temperature, Magnitude} \]

\[ V_{ne} = \text{Expected Shear Strength of Embedded Beam, K} \]

\[ \phi V_{ne} = \text{Design Shear Strength of Embedded Beam, K} \]

\[ V_{nf} = \text{Expected Shear Strength of FCB Link, K} \]

\[ \phi V_{nf} = \text{Design Shear Strength of FCB Link, K} \]

\[ V_p = \text{Nominal Shear Strength of Link, K} \]

\[ \phi V_p = \text{Design Shear Strength of Link, K} \]

\[ V_u = \text{Shear Demand on Beam, K} \]

\[ V_{u,\text{emb}} = \text{Required Shear Strength of the Embedded Beam} \]

\[ V_{u,\text{emb-}\Omega} = \text{Required Shear Strength of the Embedded Beam with Overstrength Demands} \]

\[ \Omega V_{\text{beam}} = \text{Required Overstrength Shear Strength of FCB Link, K} \]
\[ Z = \text{Plastic Section Modulus, in}^3 \]

\[ \alpha = \text{Coefficient of Thermal Expansion, in}/(\text{in}\cdot\text{degree}) \]

\[ \varepsilon_{\text{nom}} = \text{Nominal Strain, in/in} \]

\[ \varepsilon_{\text{pl}} = \text{True Strain, in/in} \]

\[ \lambda_{\text{hd}} = \text{Web Ductility Limit} \]

\[ \lambda_{\text{md}} = \text{Flange Ductility Limit} \]

\[ \sigma = \text{Pre-Tension Stress, ksi} \]

\[ \sigma_{\text{true}} = \text{True Stress in Steel, ksi} \]

\[ \sigma_{\text{nom}} = \text{Nominal Stress in Steel, ksi} \]

\[ \Omega = \text{Over Strength Factor} \]

\[ \theta = \text{End Rotation of FCB, Radians} \]

\[ \tau = \text{Shear Stress, ksi} \]
**Chapter 1-Introduction to Coupled Core Wall Systems and Coupling Beams**

Coupled core wall (CCW) systems have been proven to be very efficient in resisting seismic loads. A CCW system is typically found in the center of mid-to high-rise structures. They are primarily lateral force resisting systems but also function as a gravity system.

In a CCW system, if the openings are placed in a regular pattern and the coupling beams are designed properly, during a seismic event, where a typical wall system would resist overturning moment in the form of flexure in the wall piers and the input energy is dissipated through plastic deformation of the wall piers, a CCW system will resist that same moment in the form of compressive and tensile axial forces creating a couple that resists a large part of the overturning moment (SEI/ASCE 2007).

![Diagram of a typical CCW system](image)

**Figure 1.1:** Reactions for a typical CCW system (Fortney 2005)
Figure 1.2 shows the typical layout of a CCW system.

![Plan of CCW system (Left) and Elevation A-A (Right) (Fortney 2005)](image_url)

**Figure 1.2:** Plan of CCW system (Left) and Elevation A-A (Right) (Fortney 2005)

When considering the effect the coupling beams have on wall piers, the axial forces created during a seismic or wind event are the sum of the shear forces in the beams on the floors above the one under consideration. When coupling beams are designed properly over the height of a building, they will develop a plastic hinge, in shear or flexure, while also undergoing end rotations. During a seismic event, coupling beams must yield prior to the wall piers and demonstrate a stable hysteretic response in order for the CCW system to operate properly (Fortney 2005).

Steel coupling beams are a type of coupling beam used in a CCW system. They can be designed to show energy absorbing characteristics similar to those of an eccentrically braced frame. In a CCW system, the beams are embedded into the concrete pier walls with enough embedment and anchorage to transfer the required forces in a composite system. Design of a shear-critical steel coupling beam should ensure that the web yields in shear while the flanges remain elastic.
In this investigation, the fused steel coupling beam will be considered as the element of energy dissipation in CCW systems. The fused coupling beam (FCB) is similar to the steel coupling beam, except it has a removable link at the mid-span of the beam. The intent is for shear yielding to take place in this removable link while the remainder of the beam is unaffected. Once the link has yielded, it can then be replaced by a new one, saving the structure from being damaged indefinitely.

Two versions of the FCB have been previously investigated (Fortney 2005; Mitchell 2012). The goal of this research is to continue the advancement of the FCB system by using finite element software and the results of previous research in order to lay out a modified design procedure. This investigation will take into account different loading situations such as rotating wall piers. The procedure will be validated by using models of previous research results and their design procedures. A modified design procedure for the FCB system has been created and is detailed in the remainder of this investigation.
Chapter 2-Previous Research and Literature Review

Steel Coupling Beams have been researched and proven to be a viable option as a link beam in a CCW system. Previous studies at the University of Cincinnati have been performed by Dr. Patrick J. Fortney and Steven J. Mitchell.

2.1 Overview of Dr. Fortney’s FCB

In 2005, Dr. Fortney designed a compilation of experimental, parametric, and analytical studies which were carried out in order to further understand the behavior and constructability of CCW systems. The research focused on the design approach of CCW systems which Dr. Fortney describes as “minimizing steel congestion while maintaining satisfactory structural behavior”. In his thesis he talks about several configurations of link beams such as diagonally reinforced concrete coupling beams, un-encased steel beams, un-encased steel beams with a steel fuse link at mid span, and a composite steel and concrete beam utilizing a vertical web shear plate with headed shear studs. The beam that is the most relevant in the context of this research is the steel beam with the fuse link at mid span which will be referred to as the Fuse Coupling Beam (FCB) (Fortney 2005).

Dr. Fortney’s design of the FCB closely follows the design of the steel coupling beam. The design also closely follows the design of links in an eccentrically-braced frame. When designed properly, the steel links show excellent energy-absorbing capabilities where they are the primary mode of energy dissipation. When the steel link experiences an event beyond a design event, the flange remains elastic while the web yields in shear. Stiffeners are used to delay web buckling. Inside the wall, a fixed point is found one third of the embedment length from the face of the wall. Therefore a smaller stiffness is calculated. Vertical bars, or auxiliary transfer
bars, were added to reduce the loss of stiffness which also allows for a more symmetric
distribution of strength during reverse loadings. At the wall/beam interface, web stiffener plates
were added to delay the spalling of concrete (Fortney 2005).

![Steel coupling beam assembly](image)

**Figure 2.1:** Steel coupling beam assembly (Fortney 2005)

Experimental testing of the steel coupling beam took the beam through 11% rotation with
no damage. The web was first to yield and the shear capacity was very close to the measured
value at first yield which was 565kN. The strength of the beam increased with loading beyond
first yield and reached a maximum shear of 934kN at approximately 3% rotation. No major
damage to the wall piers was evident. Hairline cracks were seen at 1% rotation and beyond 3%
rotation cracks were 3mm wide (Fortney 2005).

The design of the FCB followed the design of the steel coupling beam very closely. The
FCB consists of three steel built-up I-shapes with the outer two pieces being connected to a wall
pier in a similar fashion as the steel coupling beam shown above. While the third piece, or the
“link” beam, connects the outer beams at mid-span. Its intent is to reduce post-seismic repair and
damage. Ensuring that the damage occurs in the fuse is done by proportioning the beam to remain elastic past a loading where the web of the fuse has yielded. The following figure displays the FCB set up (Fortney 2005).

![Fuse coupling beam assembly](image)

**Figure 2.2: Fuse coupling beam assembly (Fortney 2005)**

Dr. Fortney designed two specimens having the same depth and length but different web thicknesses. FCB-1 and FCB-2 were designed with an arbitrary percentage of shear capacity relative to the main section. One was designed with 70% of the main sections’ shear capacity and the other with only 50%. This resulted in an 8.4mm thick web in FCB-1 and a 6mm thick web in FCB-2. The fuse was also designed to have slip-critical connections at the web and flange splice locations, which needed to transfer the shear and moment along the coupling beam. Concerning the main sections outside the fuse, they were designed per the steel coupling beam design procedure (Fortney 2005).
The FCB at 50% shear capacity (FCB-50) was tested first and once it reached yield, with the main section still in the elastic range, it was replaced with the FCB at 70% shear capacity (FCB-70). FCB-50 made it to 2% chord rotation and reached approximately two times its calculated shear capacity, while the main section reached 96% of its capacity. The first yield in the web of the fuse occurred at approximately 329kN and at a displacement of 4.1mm. Once FCB-70 was in place, testing took place up to 11% rotation at which point the beam was considered to have failed, showing considerable damage at the fillet welds. First yield occurred at approximately 485kN and at a displacement of 7.9mm. Dr. Fortney states that the beam behaved as intended up to 4% rotation (Fortney 2005).

![Fuse coupling beam assembly at an 11% rotation (Fortney 2005)](image)

**Figure 2.3:** Fuse coupling beam assembly at an 11% rotation (Fortney 2005)

### 2.2 Overview of Mitchell’s FCB

In 2012 Mitchell submitted his thesis on “The Development of a Steel Fuse Coupling Beam for Hybrid Coupled Wall Systems”. He took Dr. Fortney’s FCB and improved on the design procedure with the goal of protecting the welds up to large strain demands. His beam,
when tested, was able to reach a 14% rotation without serious damage to the system. However, what was observed did not match with the desired performance of the FCB. The FCB experienced yielding in the main sections prior to 2% rotation which is not the intended behavior of the system. The intent is in fact for all of the yielding to occur in the fuse section up to large strain demands. (Mitchell 2012).

Mitchell’s FCB has a noticeably different connection layout with endplates transferring shear in a different fashion and flange splice plates still carrying the moment. His FCB is the primary focus of this research (Mitchell 2012).

**Figure 2.4:** Fuse coupling beam end-plate bending at 14% rotation (Left) and fuse coupling beam set up (Right) (Mitchell 2012)
2.3 Scope of Current Research

During Mitchell’s experiment, yielding of the embedded beams was measured prior to 2% rotation (Mitchell 2012). The objective of this research is to produce a modified design procedure for the FCB system. The objective will be reached by modeling Mitchell’s FCB in a finite element program. Once that model has been validated against existing experimental data, observations will be made in order to modify the design procedure. Finite element modeling will again be used to model the modified FCB and confirm that the objective has been reached by looking at different loading scenarios such as rotation of wall piers or a simple vertical displacement.
Chapter 3-Finite Element Modeling

For this research project the finite element software Abaqus was used. Abaqus has three main products used in analysis: Abaqus Standard, Abaqus Explicit, and Abaqus CFD. This study will focus mainly on the use of Abaqus Standard with the occasional use of Abaqus Explicit.

Abaqus Standard can solve linear and nonlinear problems involving the static, dynamic, thermal, electrical, and electromagnetic response of components. It solves problems implicitly at each solution increment where Abaqus Explicit solves a problem in “real time” in pre-determined user increments. The graphical interface in which models can be created and will be used is Abaqus CAE (Complete Abaqus Environment). It allows the user to quickly create the desired geometry, assign material properties, define contact interactions, and build models in either Standard or Explicit (Dassault Systemes 2011a).

In the context of this research, Abaqus was used to model the FCB. First, the FCB tested by Mitchell was modeled. The Abaqus model used the exact geometry from his experimental testing data spanning between the wall piers. For the portion of the beam that is inside the wall piers rotational springs were used for the end restraints in order to model the stiffness at the wall/beam interface. A series of Abaqus tests were performed to determine the stiffness of the FCB by comparing force versus displacement values.

The FCB model has six stiffener plates, four flange plates, eight end plates, two embedded sections, and one fuse section. The model contains eighty four bolts, each having a nut and washer (Mitchell 2012). Every bolt in the model was pre-tensioned to the minimum levels specified in AISC 360-10.
Once the model was created, pre-tensioned, and the proper stiffness had been achieved, a
displacement controlled load was then applied on one end of the beam and compared against
Mitchell’s results. Once the re-creation of Mitchell’s FCB was achieved, a modified design
procedure was created. Using the results found in Abaqus for Mitchell’s FCB, tests on the new
design can be carried out by using Abaqus as a simulation tool.

3.1 Abaqus Part Assembly

When starting an Abaqus model each individual part must be drawn in 2-D, extruded into
3-D, and then cut and shaped into the final geometry desired. The following table lists all of the
parts and their required dimensions in order to make the FCB by Mitchell:

<table>
<thead>
<tr>
<th>Part</th>
<th>Quantity</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embedded Beam</td>
<td>2</td>
<td>Flange- 9” x 5/8” x 9-3/8”  Web- 9-3/4” x 1/2” x 9-3/8”</td>
</tr>
<tr>
<td>Fuse Beam</td>
<td>1</td>
<td>Flange- 9” x 5/8” x 1’-6”   Web- 9-3/4” x 3/8” x 1’-6”</td>
</tr>
<tr>
<td>Flange Splice Plates</td>
<td>4</td>
<td>1’- 1/2” x 9” x 5/8”</td>
</tr>
<tr>
<td>A490 Bolts</td>
<td>84</td>
<td>End Plate Bolts – 1.802”, Flange Plate Bolts- 2.427”, 3/4”Ø</td>
</tr>
<tr>
<td>Nuts</td>
<td>84</td>
<td>Per AISC Table 7-14</td>
</tr>
<tr>
<td>Washers</td>
<td>84</td>
<td>Per AISC Table 7-14</td>
</tr>
</tbody>
</table>

Each part was made per the shop drawing for the experimentally tested FCB with the
exception of the bolts, nuts, and washers (Mitchell 2012). Those were dimensioned per AISC
360-10, 2010. Once, all of the parts were created in the parts module they could then be
individually imported into the assembly module in Abaqus. After being rearranged and
manipulated, the FCB was assembled (Dassault Systemes 2011a).
Figure 3.1: Final assembly of the FCB in Abaqus

3.2 Abaqus Part Meshing

Meshing the FCB required the use of structured and swept meshing techniques. Structured mesh is a technique that meshes a part by using predetermined mesh topologies where a swept mesh is used to mesh complex shapes and involves two phases (Dassault Systemes 2011a):

- The mesh generator creates a mesh on one side of the part, called the source side.
- It then copies the nodes of that mesh, one layer at a time (one element size), until it reaches the target side.
The benefits of using a swept mesh are that it can be used on any type of edge, be it straight or curved (Dassault Systemes 2011a). For the FCB, when using the swept meshing technique, it was optimal to use the “medial axis algorithm” option. This algorithm first breaks apart the region being meshed into a group of simpler regions. It then uses structured meshing techniques to fill the simple region. This technique can only be used for quadrilateral and hexahedral meshing. In the case of the FCB this technique created more well shaped elements than alternative meshing techniques (Dassault Systemes 2011a).

Seed sizes for each part were selected on a part by part basis in order to form the most well-shaped elements possible. A seed is a marker that can be placed along the boundary of a region that allows the user to specify the target mesh density for that part (Dassault Systemes 2011a). The following table illustrates how each part was meshed and seeded:

**Table 3.2: Part mesh attributes**

<table>
<thead>
<tr>
<th>Part</th>
<th>Mesh Technique</th>
<th>Seed Size</th>
<th>Number of Elements</th>
<th>Type of Element</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embedded Beam</td>
<td>Structured/Swept</td>
<td>0.20</td>
<td>28,187</td>
<td>C3D8R</td>
</tr>
<tr>
<td>Fuse Beam</td>
<td>Structured/Swept</td>
<td>0.20</td>
<td>42,226</td>
<td>C3D8R</td>
</tr>
<tr>
<td>Flange Splice Plates</td>
<td>Structured/Swept</td>
<td>0.25</td>
<td>5,724</td>
<td>C3D8R</td>
</tr>
<tr>
<td>A490 Bolts-Flange</td>
<td>Swept</td>
<td>0.10</td>
<td>2327</td>
<td>C3D8R</td>
</tr>
<tr>
<td>A490 Bolts-End Plate</td>
<td>Swept</td>
<td>0.10</td>
<td>2132</td>
<td>C3D8R</td>
</tr>
<tr>
<td>A563 Nuts</td>
<td>Swept</td>
<td>0.11</td>
<td>497</td>
<td>C3D8R</td>
</tr>
<tr>
<td>A436 Washers</td>
<td>Swept</td>
<td>0.15</td>
<td>48</td>
<td>C3D8R</td>
</tr>
</tbody>
</table>
Partitions enable the use of both swept and structured mesh on some parts in the FCB. A partition simply divides two or more areas of a region into user defined shapes. Partitions become part of the assembly once created and can be treated as a separate entity.

![Embedded beam partitioned and meshed](image)

**Figure 3.2:** Embedded beam partitioned (Left) and meshed (Right).

Swept meshed regions are shown around the bolt holes while structured meshed regions encompass the rest of the part.

### 3.3 Abaqus Interactions

Before loads can be applied to the model, certain interactions and constraints need to be defined. Between each surface the coefficient of friction has been initially defined as 0.30 with “normal” mechanical properties. This interaction property is then applied to 430 surface-to-surface interactions where a surface to surface interaction is defined as a region that will come into contact with another region during loading. Once they make contact they will maintain all interaction and material properties given to each individual region (Dassault Systemes 2011b).

Along with surface to surface interactions, the model has 168 tie constraints. In the FCB model a tie constraint is defined where two regions are already in contact and movement is
restricted completely (Dassault Systemes 2011b). All of the tie constraints in the model are as follows:

- Nut tied to A490 Flange Bolt
- Nut tied to A490 End Plate Bolt
- Washer tied to Nut

Since the FCB was now assembled and all of the basic properties were set, the validation process could take place.
Chapter 4- Modeling of Mitchell’s FCB

In order to re-create Mitchell’s results, his experimental force-displacement data was compared against the analytical data from Abaqus. In his experiment he had strain gauges attached to various parts of the beam. Usable force-displacement data was then measured directly (Mitchell 2012). The analytical data is obtained directly from Abaqus and plotted into Excel.

For an appropriate reproduction of experimental results, Abaqus needed to properly model standard bolt pre-tensioning, rotational stiffness at the beam/wall interface, inelastic material properties and physical interactions. The end goal is to have a model that can adequately reproduce the results of the FCB by Mitchell. This model will then be used as a design basis to further develop the design procedure for the FCB.

4.1 Pre-Tensioning

Per AISC 360-10, the minimum pre-tension for a 3/4” A490 bolt is 35 kips (ANSI/AISC 360-10 2010). This value will be the target value for all bolts in the FCB assembly. The method that was used to pretension the bolts is based on the following equation:

\[ \sigma = E\alpha\Delta T \]  

Equation 4-1

To calculate an initial guess, an over-estimated value for temperature was used such that it resulted in a 100 ksi stress, yielding a 44 kip pretension value. It was necessary to be conservative on the initial guess since Abaqus takes into account the stiffness of the plates when pre-tensioning a bolt. This means that, the thicker the plate, the stiffer it is, requiring a smaller temperature applied to it.
Using $\alpha$, the coefficient of thermal expansion, as an assumed value of 0.00001 (in/in*degree) and $E$, the modulus of elasticity, as 29000 ksi, iterations began until Abaqus obtained a 35 kip pre-tension in every bolt, with a tolerance of 0.5 kips.

**Table 4.1**: Bolt Pre-tension iteration trials

<table>
<thead>
<tr>
<th>Iteration</th>
<th>Flange Bolt Temperature (Degrees)</th>
<th>End Plate Bolt Temperature (Degrees)</th>
<th>Flange Bolt Pre-tension Achieved (Kips)</th>
<th>End Plate Bolt Pre-tension Achieved (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-346.00</td>
<td>-346.00</td>
<td>30.87</td>
<td>28.34</td>
</tr>
<tr>
<td>2</td>
<td>-380.00</td>
<td>-400.0</td>
<td>33.22</td>
<td>29.47</td>
</tr>
<tr>
<td>3</td>
<td>-405.8</td>
<td>-662.8</td>
<td>35.29</td>
<td>47.12</td>
</tr>
<tr>
<td>4</td>
<td>-405.8</td>
<td>-482.4</td>
<td>35.29</td>
<td>37.45</td>
</tr>
<tr>
<td>5</td>
<td>-405.8</td>
<td>-457.1</td>
<td>35.29</td>
<td>35.93</td>
</tr>
<tr>
<td>6</td>
<td>-405.8</td>
<td>-448.9</td>
<td>35.29</td>
<td>35.37</td>
</tr>
</tbody>
</table>

In order to extract the pre-tension data from Abaqus, all of the elements in one layer were selected at the center of the stressed bolt and the axial stress data, S22 or S33, was plotted depending on the orientation in the global coordinate system. The desired results are to not only have a 35 kip minimum pre-tension in every bolt, but also to have a uniform stress distribution in the bolts themselves.
4.2 Stiffness Testing

To account for the experimental FCB’s rotation at the wall/beam interface, rotational springs were introduced in the analytical model. The springs released the end rotations from the idealized “fixed-fixed” scenario to a slightly less rigid connection. In order to approximate an appropriate stiffness value for the springs, a few assumptions were made.

\[ K = \frac{M}{\theta} \]  

Equation 4-2
Where the moment, \( M \), is the moment for the fixed-fixed beam from the Abaqus analysis and the rotation, \( \theta \), is an assumed value in radians. Assuming that end-rotations are less than one degree; we can solve for a few reasonable values for stiffness and analyze the results.

![Force-\( \Delta \) with Rotational Springs](image)

**Figure 4.2:** Trial runs in Abaqus, stiffness testing

As can be seen in the figure above, with a decrease in stiffness, the initial slope of the line is smaller and the total force felt at maximum displacement is less as well. Once the initial trial runs were finished, they could then be superimposed onto Mitchell’s force-displacement data. The goal is to model the initial stiffness as closely as possible accounting for the uncertainty in the experimental boundary conditions. This was done by interpolating between the slopes of the different trial runs and the desired slope.
From the results, it can be observed that a stiffness of 570000 (K-in/rad) can be used for further testing. This value was later altered slightly for other simulations due to cyclic testing results, but was considered satisfactory.

4.3 Material Properties

After bolt pre-tensioning was completed and a reasonable stiffness value was obtained, simulations were run up to a 14% beam rotation in order to model the maximum rotation achieved during experimental testing. It became evident that the only way to obtain a better representation of the experimental data was to modify the material properties. During experimental testing of the FCB, coupon tests were performed on different thickness coupons.
Abaqus requires the user to input the true-stress and true-strain when defining plastic properties, where strain is zero at the yield stress (Dassault Systemes 2011b). The plots for 14% rotation using the original stress-strain data and the true stress-strain values are plotted in Figure 4.4.

\[
\sigma_{true} = \sigma_{nom}(1 + \varepsilon_{nom}) \quad \text{Equation 4-3}
\]

\[
\varepsilon_{ln}^{pl} = \ln(1 + \varepsilon_{nom}) - \frac{\sigma_{true}}{E} \quad \text{Equation 4-4}
\]

Since the experimental coupon data had hundreds of points, the data was thinned out while still capturing the plastic properties of the material. This was done for every thickness of material in the model. To explore the influence of the friction coefficient on the strength at 14% rotation, its value was changed to 0.4 and the results were analyzed, confirming that 0.3 was an appropriate value for it.

**Figure 4.4:** Abaqus results at 14% rotation with true-stress/strain defined
4.4 Cyclic Modeling

Once the model adequately represented Mitchell’s FCB, cyclic modeling was then carried out. Initially, isotropic hardening was being used for modeling. This approach was not desirable and was switched over to kinematic hardening. It was then determined that the previous stiffness value was too high and the new value that better represents the data is 330000 (K-in/rad). Preliminary results showed that while the stiffness was modeled appropriately for the initial loading, all other cycles required a non-linear stiffness, corresponding to a progressive loss of stiffness of the test setup. A material overstrength value of 10% was added to the stress-strain data, giving a more desirable curve as shown in the figure seen below.

![Figure 4.5: Preliminary cyclic results](image-url)
Several stiffness values were evaluated during the non-linear stiffness modeling but it was found that a value of 75000 (K-in/rad) was the most appropriate.

**Figure 4.6:** Non-linear stiffness results

As can be seen in Figure 4.6, as reversal loadings are applied there is a definite change in stiffness. It is postulated that the post-tensioning provided to prevent any rotation of the wall piers was not 100% effective during the experiment, causing different stiffnesses at different stages of loading and for different loading directions. This accidental rotation was accounted for during the evolution of the updated design procedure.
4.4.1 Non-Linear Stiffness Modeling

In order to model non-linear stiffness in Abaqus, a series of values had to be assigned to the appropriate spring. A minimum of three points are required. The first and last value corresponds to the spring stiffness at a specified moment and rotation. The value of the float point will determine where stiffness will change in the model. Figure 4.7 represents the proper usage of a non-linear spring with one float point in an input file:

*Spring, nonlinear, elset="Non-Linear Spring 1-spring"
4
-75000., -1.
0., 0.
328676., 1.
*Element, type=Spring1, elset="Non-Linear Spring 1-spring"
2, 1

Figure 4.7: Non-linear spring usage in Abaqus

The first line specifies the name of the spring. On the second line the 4 represents the degree of freedom the spring is acting on. The following three lines represent the moments and rotations desired on either side of the float point to define the slope of the element’s moment-rotation behavior (Dassault Systemes 2011a; Dassault Systemes 2011b).
Figure 4.8: Representation of non-linear stiffness of spring

4.5 Results from Mitchell’s FCB Abaqus Modeling

Based on strain gage measurements, it was estimated that the flanges of the embedded portion of the FCB that Mitchell tested yielded before the fuse link. Ideally, the link should yield in shear prior to any yielding in the embedded beams. Once the FCB was modeled adequately in Abaqus it was then compared to multiple states of stress in order to determine how best to solve the issue at hand. Those stress states were tensile stress (S33), shear stress (S23) and Von Mises Stress Criterion. Plastic equivalent strain (PEEQ) was also observed for post-yield behavior.
Figure 4.9: Shear stress distribution, S23

Figure 4.10: Tensile stress distribution, S33
Figure 4.9 and Figure 4.10 together demonstrate that the fuse is not yielding properly. Both the flange of the embedded beam and the web of the fuse yield at approximately 2.8% chord rotation. It can also be noted that the stress concentrations shown for S33 are focused around the back row of bolts in the flange. From this observation the modified design procedure needs to account for the net area at the bolt holes. Along with S33 and S23, the Von Mises yield criterion is a useful measure of the combined results of all the stress components (S11, S22…S23 etc.) plotted on the FCB. Figure 4.11 is set to show failure at the true yield stress as determined from Equation 4-3.

Figure 4.11: Von Mises stress distribution in the web of the FCB
Figure 4.12: Von Mises stress distribution in the flanges of the FCB

The Von Mises criterion shows that the combined stress components cause the web of the link and the embedded beam to yield at the same time. The chord rotation associated with the failure is approximately 1.5%. Also, as previously stated from the S33 and S23 comparison, the flanges show yielding at the bolt hole locations.

Not only are the results of first yield important but so are the results after yielding has occurred. The equivalent plastic strain (PEEQ) can show the strain distribution after yielding has occurred. At first yield, stress is equal to the defined yield stress and plastic strain is equal to zero. As the beam experiences higher states of yielding, PEEQ captures the strain values at the yield locations.
Figure 4.13: Distribution of equivalent plastic strain, PEEQ

The strain distribution shown in Figure 4.13 is shown at a 2.8% chord rotation. This distribution corresponds to the intended behavior, concentrated in the link and then spread out towards the embedded beams.
Chapter 5- The Modified Design Methodology for Fuse Coupling Beams

There is no standardized way to develop a FCB. The Seismic Provisions, known as AISC 341-10, contain equations to design a link in an eccentrically braced frame (EBF). As discussed earlier, the FCB design procedure will closely follow that of an EBF. Also available is the research that Mitchell has completed. The methodology for the updated design procedure will be a combination of the Seismic Provisions and Mitchell’s research. A full example will be included in Appendix D.

5.1 Overview of the FCB Design Methodology

The primary goal of the FCB is to have the fuse link fail in shear prior to the beams embedded in the wall. A traditional coupling beam is meant to develop a plastic hinge at some distance away from the wall/beam interface. In this application, the fuse link will act as the plastic hinge. The fuse link will be developed as Mitchell developed it, which is as part of a steel coupling beam. The fuse link will be designed closely to Mitchell’s procedure, and the embedded portions will have to provide at least the same capacity as the fuse link. This means the embedded beams are proportioned based on the fuse link details. All bolted connections will need to transfer the bending moment by means of flange splice plates and the shear force by means of end plates. By keeping the embedded beams below yielding during a seismic event, the fuse can then be replaced, thus saving the structure and creating a positive economical consequence (Mitchell 2012).
5.2 Updated Design Procedure for the FCB

This section contains the design procedure for the FCB. It is an enhancement to the current design procedure developed by Mitchell. The design will focus solely on the portion of the beam that is between the pier walls. For the design of the hybrid connection of the embedded portion of the beam, the procedure developed in AISC 341-10 for steel coupling beams and Mitchell’s Thesis should be followed, both referenced in this report.

5.2.1 Overall Design of the FCB

The procedure for the design of the FCB will closely follow the procedure that Mitchell developed with a few improvements. FCB components will be designed based on the sizing of the fuse link. This is considered an “inside-out” design approach. In order to start sizing a coupling beam, an analysis similar to that of Mitchell’s is required in order to find the forces needed for design. Figure 5.1 shows the general layout of the typical FCB (Mitchell 2012).

Figure 5.1: Elevation view of a typical FCB between wall faces
5.2.2 Design of Fuse Link

Before sizing the fuse link, shear and moment demands must be determined based on Mitchell’s analysis. Along with shear and moment demands, axial forces in the beams must be determined for subsequent checks. Once the loads are determined, the beam can then be designed for shear strength. Sizing of the beam has to also take into consideration the width of the beam that is to be embedded into the pier walls and available floor to floor heights. The dimensions that are required for the design are as follows:

![Fuse link dimensions](image)

**Figure 5.2:** Fuse link dimensions
Before solving for shear strength an axial force check must be performed.

If \( \frac{P_u}{P_y} \leq 0.15 \) then \( V_p = 0.6F_yA_{tw} \) \( \text{Equation 5-1} \)

or

If \( \frac{P_u}{P_y} > 0.15 \) then \( V_p = 0.6F_yA_{tw} \sqrt{1 - \left(\frac{P_u}{P_y}\right)^2} \) \( \text{Equation 5-2} \)

Where \( P_u \) is the axial force felt in the FCB and \( P_y \) is the cross sectional area of the fuse link times \( F_y \). Once \( V_p \) is found it can be said that (ANSI/AISC 341-10 2010):

\[ \phi V_p = \phi V_{nf} \] \( \text{Equation 5-3} \)

Where \( V_{nf} \) is the nominal shear strength of the fuse and \( \phi V_{nf} \) is the design shear strength of the fuse. For this limit state \( \phi = 0.9 \) (ANSI/AISC 341-10 2010).

Now that the shear demand has been found, the fuse link flexural strength can be found. Although the fuse is not expected to fail in flexure it is a safe check to make. The check is simply

\[ \phi M_{nf} = \phi M_{pf} = \phi F_yZ \] \( \text{Equation 5-4} \)

In this case \( \phi \) also equals 0.9, \( F_y \) is the yield stress and \( Z \) is the plastic section modulus. Once you have \( M_{pf} \) one must then back check to see if the correct \( V_p \) was used during the shear strength design. If \( 2M_{pf}/e < V_p \) then the design is still okay, otherwise \( 2M_{pf}/e \) needs to be substituted back in the shear strength design as the new \( V_p \) value (ANSI/AISC 341-10 2010).

Now that a proper \( V_p \) value is obtained, a check to see if the fuse link is shear critical is required. Just as the links in an EBF, the fuse links primary mode of energy dissipation is through shear yielding. To check if the link is shear critical use the following equation:
\[ e \leq 1.6 \frac{M_{pf}}{V_p} \quad \text{Equation 5-5} \]

Where \( e \) is the length of the fuse link (ANSI/AISC 341-10 2010).

In order for local buckling to be precluded, the fuse link needs to be considered a highly ductile member. For the web of the fuse link a highly ductile member is required since the beam is shear critical. The flange ductility can be checked as moderately ductile since it is unlikely to yield. In the case of a highly ductile member, the use of \( C_a \) is only considered when axial loads are transferred to the system. For web ductility (ANSI/AISC 341-10 2010):

\[ 2.45 \sqrt{\frac{E}{F_y}} (1 - .93C_a) > \frac{h_{wf}}{t_{wf}} \quad \text{Equation 5-6} \]

For flange ductility:

\[ .38 \sqrt{\frac{E}{F_y}} > \frac{b_{ff}}{2t_{ff}} \quad \text{Equation 5-7} \]

Stiffener plates are required based on AISC 341-10. In the case of the FCB, a link rotation of 0.08 radians can be assumed which leads to stiffener spacing of (ANSI/AISC 341-10 2010):

\[ \text{Spacing} = 30t_w - \frac{d}{5} \quad \text{Equation 5-8} \]

and a width of

\[(b_f - 2t_{wf}) \text{ not less than the larger of } 0.75t_{wf} \text{ or } 3/8 \text{in} \quad \text{Equation 5-9} \]
5.2.3 Design of Embedded Beam

The design of the embedded beam will be a two-step process. First, design the fuse normally and then calculate the shear demand of the embedded beam using an overstrength factor, $\Omega_{FCB}$, which is suggested to be taken as 1.20 from Abaqus results. Once the shear design of the embedded beam is completed, the moment demand should be evaluated using the shear demand without the overstrength factor. Once the second evaluation is completed, be sure to double check whether the design of the embedded beam caused the depth of the member to change. If it did, adjust the web height of the fuse beam accordingly and make sure the design of the fuse beam is still adequate.

The shear demand on the embedded beam is taken directly from the shear capacity of the fuse link. Due to expected material strength factors, the shear that the embedded beams have to resist is:

$$ V_{u,emb-\Omega} = \Omega_{FCB} 1.1 R_y V_{nf} $$  \hspace{1cm} \textbf{Equation 5-10} \hspace{1cm}$$

Where $R_y$ is the expected material strength found in AISC 341-10, the 1.1 accounts for material strain hardening, $\Omega_{FCB}$ is the overstrength factor of 1.2 and $V_{nf}$ is the shear capacity from the fuse link design. This shear demand is then applied to the embedded beam (ANSI/AISC 341-10 2010).

The flexural demand on the beam is taken as the moment transferred from the fuse link plus the inherent moment from the shear force applied at the end of the beam. This shear force does not have the overstrength factor applied to it. For the moment transferred, it is the shear demand multiplied by the moment arm of the link as shown in the following equation (Mitchell 2012):
\[ M_{tr} = 1.1R_yV_{nf} \cdot \left( \frac{e}{2} \right) \]  \hspace{1cm} \text{Equation 5-11}

In order to find the maximum moment in the embedded beam, the moment at the wall face must be found. That moment is shown in the following equation (Mitchell 2012):

\[ M_{wall} = M_{tr} + 1.1R_yV_{nf} \cdot L_{emb} = M_u \]  \hspace{1cm} \text{Equation 5-12}

From earlier observations in this report it was found that the stress concentration was greatest around the bolt holes for the embedded beam. For this reason, rather than using the section modulus for the gross area, the section modulus for the net area will be used. This area will be taken at the location of the row of bolts closest to the wall. Therefore the governing equation is given as:

\[ M_{y,net} = S_{net}F_y \]  \hspace{1cm} \text{Equation 5-13}

Where \( S_{net} \) is the net section modulus and \( F_y \) is the yield stress. The beam is considered adequate in flexure if the moment at the wall face is lower than the yield moment.

Stiffener plates should follow the same procedure as the fuse link. See Equation 5-8 and Equation 5-9 for stiffener design. Having designed the fuse link and the embedded beams, the dimensions of the beam have been confirmed.
5.2.4 Connection Design

Connections in the FCB will involve flange splice plates for moment transfer and end plates for shear transfer. The bolts will be considered slip-critical in order to provide a predictable behavior during cyclic loading. All end plates and stiffeners will be welded to the FCB following the procedure laid out by AISC 360-10. All other connections will be bolted, following all applicable limit state checks in AISC 360-10.

Figure 5.3: Elevation view of FCB end plates (Left) and top view of flange splice plates (Right)

These connections are designed to be replaceable, excluding the end plates on the embedded beams. A full design example with commentary is included in Appendix D.
Chapter 6- Results for the Updated FCB

Once the FCB was designed following the modified design procedure, it was then modeled using the same input load history from Mitchell’s FCB in Abaqus. This includes bolt pretension values, modified stiffness values through the use of rotational springs, material strength data and meshing techniques. There are two main scenarios that will be considered for the modeling of the updated FCB:

1. Mitchell’s FCB test setup with an applied vertical displacement to represent chord rotation. The Abaqus model applies a displacement on one end of the beam in order to recreate the behavior in the lab.

![Figure 6.1: Deformed shape of scenario one](image)

As Figure 6.1 suggests, the loading shown will put the beam into double curvature. This shape is exactly the same as the initial modeling of Mitchell’s FCB.
2. A more realistic scenario for a FCB would be to apply rotations directly at the beam/wall interface only. The rotation was arbitrarily chosen as 5% and was applied to both ends of the model. No vertical displacements were added to this scenario.

Figure 6.2: Deformed shape of scenario two

The deformed shape of scenario two puts the beam into double curvature. It is not typical for walls to displace as much vertically as they do horizontally during a seismic event. This model of the FCB treats the beam as if the walls were being pushed over rather than being pulled up and down.

6.1 Scenario One Results

Similarly to the FCB modeling of Mitchell’s beam in Abaqus, scenario one provides the same loading and parameters. The desired results for scenario one would achieve yielding in the fuse link beam prior to the yielding of the embedded beams. Limit states that will be investigated are the following:

- Von Mises Yield Criterion
- Shear stress, S23, of the fuse link and embedded beam webs
- Tensile stress, S33, of the embedded beams
- Equivalent plastic strain, PEEQ

In order to plot the results for the aforementioned limit states the stress/strain contours were assigned limits based on the thickness of the portion of the beam under consideration.

### 6.1.1 Von Mises Criterion

Von Mises describes the state of all stress components in a particular element. The following plots show the exact instance the Von Mises stresses are considered to have reached yielding in a certain portion of the beam. Looking at the embedded beams only, the following figure shows the state of stress in the flanges only.

**Figure 6.3:** Von Mises state of stress in the embedded beam flanges
Yielding is occurring at the bolt holes and at the part of the flange on the wall side of the bolts. This state is observed to occur at a chord rotation of approximately 4.45%. In order for this beam to have performed adequately, the fuse web must already have yielded. The following plot demonstrates the state of stress in the fuse link’s web only:

![Figure 6.4: Von Mises state of stress in the fuse link web](image)

Figure 6.4 demonstrates that yielding in the fuse link’s web has occurred prior to the yielding of the embedded beam’s web. The fuse link has yielded at a chord rotation of approximately 1.4%. Results show that the fuse link has yielded prior to the embedded beam. Next, the web yielding of the fuse link and the embedded beam need to be compared to see if flange yielding or web yielding is the prominent failure mode.
Figure 6.5: Von Mises state of stress in the embedded beam web

The above stress contours show that the webs of the embedded beams have yielded at an approximate chord rotation of 2.6%. This rotation is lower than the rotation required to yield the beam flanges and therefore controls the design.

Results from these Von Mises plots show that the fuse is indeed shear critical. It also demonstrates the desired failure mechanisms for the updated FCB have been achieved. In order to validate these results, other stress states must be considered as well.

6.1.2 Shear Stress

Since the FCB has been designed to be shear critical, it must be confirmed that the fuse link is failing in shear prior to that of the embedded beam. As opposed to Von Mises where all states of stress are considered simultaneously, shear stress or S23 will be considered the primary
failure mode of the FCB webs. The following shear stresses are plotted based on the thickness of the part under consideration. The point at which the FCB has yielding is determined based on a relationship that the Tresca Yield Criterion specifies.

\[ \tau = \frac{1}{\sqrt{3}} F_y \]  

Equation 6-1

Figure 6.6: Shear stress, S23, of the fuse link

The fuse link has yielded at approximately 2.6% chord rotation. Shear stress in the embedded beams is plotted in Figure 6.7.
Looking at only the regions in the embedded beams, it can be determined that the embedded beam has yielded at approximately 6.0% chord rotation. That means that after first yield in the fuse link occurred, the FCB was able to be rotated another 3.4% which is desirable.

6.1.3 Tensile Stress

The tensile stress, $S_{33}$, needs to be considered as a failure mode in the embedded beam flanges due to bending. Since the fuse link is shear critical, it is assumed that it will fail in shear before flexure, therefore $S_{33}$ can be ignored in the fuse link.
From observation, the highest stress concentrations are surrounding the last row of bolt holes on the assembly. The approximate chord rotation at yield stress is 3.9%. This means that flexural stress $S_{33}$ is a more prominent failure mode in the embedded beams than shear stress $S_{23}$.

### 6.1.4 Equivalent Plastic Strain

Equivalent plastic strain, $\text{PEEQ}$, shows the behavior of a material once it has yielded. In the case of the FCB, the desired result is for the fuse to continue yielding up to ultimate stress while the embedded beam is delayed in yielding.
Figure 6.9: Equivalent plastic strain at a 10.7% chord rotation

Figure 6.9 demonstrates the desired action of the FCB after yielding has occurred. The fuse link has gone much further towards its ultimate stress/strain than the embedded beam.

6.1.5 Ultimate Capacities

Prior to the analytical analysis of the FCB, Mitchell was able to plot the fuse capacities up to a 14% chord rotation. Using the same stiffness and material properties as Mitchell’s experiment, a capacity for the updated FCB can be compared to the experimental values. The experiment shows a resultant force at the wall/beam interface of 218 Kips at the maximum rotation. The Abaqus analysis of the updated FCB at its maximum rotation yields a 250 Kip reaction force.
Figure 6.9: Force-Displacement of updated fuse and Mitchell’s fuse

The force displacement curve shows that a higher ultimate strength can be achieved along with having the same stiffness as the original experiment.

A second FCB was created using the updated design procedure. The beam was designed for story shears and moments lower than that of the original FCB. This second beam was assigned the same stiffness and material properties as the aforementioned beam. As expected, this second fuse yielded a lower ultimate force at the beam/wall interface at a 14% rotation than the previous updated FCB.
Figure 6.10: Force-Displacement of updated fuse vs. 2nd updated fuse

6.1.6 Stress Comparisons

Yielding of the embedded beams compared to the yielding of the fuse link shows how well the FCB performs. Figure 6.11 shows the comparisons between the chord rotations associated with failure.

Table 6.1: Comparison of fuse link and embedded beams rotation at yield

<table>
<thead>
<tr>
<th>Yield Criterion</th>
<th>Fuse Link</th>
<th>Embedded Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Von Mises</td>
<td>1.4%</td>
<td>2.6%</td>
</tr>
<tr>
<td>S23</td>
<td>2.6%</td>
<td>6.0%</td>
</tr>
<tr>
<td>S33</td>
<td>N/A</td>
<td>3.6%</td>
</tr>
</tbody>
</table>
From Figure 6.11 it can be seen that in each instance the fuse link yields prior to the embedded beam. Stress S33 is not applicable for the fuse link since the fuse link is shear critical and the only mode for S33 to yield before S23 is through flexure. Since the moment is highest in the embedded beams, it can be concluded that S33 can be ignored in the fuse link.

6.2 Scenario Two Results

Scenario two demonstrates a more realistic behavior of the FCB. When a CCW system is laterally loaded it is in essence pushed over. This results in a rotational demand on the FCBs in the system rather than a vertical displacement. During loading the beam’s deflected shape is shown to be in double curvature. The same limit states that were investigated in scenario one will still be investigated in scenario two.

6.2.1 Von Mises

Since the FCB from scenario two has the same dimensions as scenario one, a contour plot of the Von Mises state of stress is comparable. In order to properly plot the contours, the stress distribution is shown for individual parts of the FCB. The following plot shows the onset of yielding in the fuse link. Its yield stress in Abaqus was assigned per the material thickness. This data comes from coupon test results from Mitchell’s experiment.
Figure 6.11: Von Mises state of stress in the fuse link web

The fuse is shown to have yielded at a 0.5% rotation. In order to determine if the fuse is performing as it should, the yielding in the embedded beams was investigated.

Figure 6.12: Von Mises state of stress in the embedded beam web
The rotation at yield for the embedded beam was determined to be 1.5% which is the desired result because the embedded beam has yielded after the fuse beam. Also, the rotation at yield for the flanges of the embedded beam must be taken into consideration.

Figure 6.13: Von Mises state of stress in the embedded beam flanges

As it turns out, the flange yields at a rotation of 3% and therefore is not the controlling mode of failure for the Von Mises stress criterion.

6.2.2 Shear Stress

Possibly the most critical state of stress to be considered is the shear stresses in the embedded beams and the fuse link. In order for the FCB to be considered shear critical in the link, the link must fail prior to the embedded beams.
Figure 6.14: Shear stress, S23, of the fuse link

From the above figure, shear yielding is taking place at the center of the link beam. The rotation at yield for the link beam is 1.4%.

Figure 6.15: Shear stress, S23, of the embedded beam
The FCB performs extremely well in shear, showing a 4.4% rotation at yield. That means that the FCB can rotate another 3% before the embedded beams experience any yielding in shear.

6.2.3 Tensile Stress

To determine whether or not shear or flexure controls the design of the embedded beams, S33 must be compared to S23. This is due to the fact that S33 is proportional to flexural strength in the FCB. As the flanges of the embedded beams bend, they go into tension or compression, where S33 can be used to evaluate the rotation at yield.

Figure 6.16: Tensile stress, S33, of the embedded beam

Note that the highest concentrations of stress are shown at the bolt holes. This coincides with the considerations taken in the updated design procedure. It is shown that the flanges yield
at a rotation of 3.3% and therefore are the controlling value when compared to the web of the embedded beam.

6.2.4 Equivalent Plastic Strain

PEEQ was investigated for scenarios one and two to ensure that the beam behaved properly past yield.

![Figure 6.17: Plastic equivalent strain at a 5\% end rotation](image)

Just as scenario one, scenario two shows the desired behavior of the FCB even at a very high amount of rotation. PEEQ shows that the link beam is shear critical.
6.2.5 Stress Comparisons

Yielding of the embedded beams compared to the yielding of the fuse link shows how well the FCB performs. Figure 6.11 shows the comparisons between the chord rotations associated with failure.

Table 6.2: Comparison of fuse link and embedded beams rotation at yield

<table>
<thead>
<tr>
<th>Yield Criterion</th>
<th>Fuse Link</th>
<th>Embedded Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Von Mises</td>
<td>0.5%</td>
<td>1.5%</td>
</tr>
<tr>
<td>S23</td>
<td>1.4%</td>
<td>4.4%</td>
</tr>
<tr>
<td>S33</td>
<td>N/A</td>
<td>3.3%</td>
</tr>
</tbody>
</table>

Again, the beam performs the way it should, albeit at smaller rotations than scenario one. These rotations are even more desirable when considering the expected rotational demands in a CCW system. The fuse link is still yielding prior to the embedded beam in all cases.
Chapter 7- Summary and Conclusions

The overall goal of this research was to develop a modified design procedure for the FCB to prevent premature yielding of the fuse. The procedure was developed also taking into account different loading situations such as rotating pier walls. This procedure was to be validated using Mitchell’s experimental results and his design procedure. A study of the FCB was carried out through analytical finite element modeling.

After the model of Mitchell’s FCB was completed, it was observed that yielding took place in the embedded beams at nearly the same time as it did in the fuse link, confirming the experimental observations. Not only did the embedded beams yield in flexure but they also yielded in shear at a rotation that was considered to be unacceptable. Since the flexural yielding was occurring at the bolt holes, a modification to the calculation of the moment of inertia of the beam was made to account for the net section rather than the gross section. This in turn caused the section modulus to decrease, forcing the design to include either a thicker flange or a wider flange. It is at the designer’s discretion as to how they increase flexural strength, however it is suggested that a thicker flange be used over a wider flange in order to not force the pier walls to become overly thick. Having taken care of the yielding in the flanges, the yielding in the web of the embedded beam needed to be considered. The new design procedure assigns an overstrength factor on the shear capacity of 1.20.

Abaqus models were created to model the newly designed beams. In the first model, the beam was assigned a vertical displacement at one end while keeping the other end fixed. A second model was run where the beam was assigned a rotation on one end of the beam while keeping the other end fixed. It was found that, in both cases, the FCB was performing as was
desired. Von Mises, shear stress and tensile/compressive stress were all compared in establishing adequacy. It was observed that the shear stress in the link beam was always higher than the shear stress in the embedded beams, meaning that the link beam always yielded in shear first, as expected. This type of yielding was the desired result in this investigation. Flexural yielding did occur in the embedded beams at large drifts and in both loading scenarios it caused yielding in the embedded beams prior to shear.

Being a primary mode of energy dissipation in a CCW system, the FCB is an economical investment worth making when designing for seismic loads. When a design earthquake should affect a building, the FCB will dissipate the energy at least as well as any other CCW system. However, the advantage comes when the FCB does yield and can then be replaced with no other damage to the system. Not only can it be replaced, saving money in repairs, but it will continue to gather strength even after the FCB yields everywhere. Through strain hardening, the FCB can withstand loads up to a minimum of a 14% chord rotation, thus being a safe alternative to traditional coupling beams.

7.1 Suggestions for Future Research

Future work that should accompany the updated design procedure would require physically testing the new FCB. It would yield better results if the beam were to be able to rotate at both ends rather than just displacing upwards. It was seen in this study that the beam under rotational loads was more critical than the beam with only an upward displacement. During experimentation, gages should be present at the center of the web on the fuse link, at various locations on the web of the embedded beam and between the bolt holes where stress concentrations appear to be highest.
Along with an experimental setup, an analytical model should be run in order to better define the story drift limits of design. The analytical model would include the updated fuse beams and their properties. If the experimental test should succeed and the analytical model succeeds, a design procedure that is code formatted should be written and submitted to AISC.

Furthermore, an investigation into recentering systems coupled with FCB systems should be studied. In a seismic event that causes the FCB’s to yield with no yielding of the embedded beams, a permanent story drift may result. In that case, the fuses may all be replaced but the deflections would remain. A possible system might incorporate post-tensioning to bring the structure back to its original position.
References


Appendices
Appendix A – Bolt Pre-Tensioning

During the original experiment, failure modes that concerned bolted connections were not observed to be an issue. All bolted connections remained intact. Due to this observation, the goal of the Abaqus model was not focused on failure of the bolts. The main focus for the bolts is for the model to be represented through adequate bolt pre-tensioning.

Currently AISC denotes that for a ¾ inch A490 bolt, the minimum pre-tension force is 35 kips. It was decided that the bolts would be modeled without threads for simplicity and since bolt shear was not an issue in the actual experiment. As an A490 bolt, the yield stress was taken as 150 ksi.

A.1 Modeling and Results

To apply a pre-tension strength to the bolts in the model a certain magnitude of temperature was applied to the shank of the bolts. At the same time, the bolt shank was tied to a nut which held that part of the bolt in place, thus applying a tension force to the part to the shank between the nut and the head of bolt. Figure A.1 shows the stress distribution in the shank of the bolt once it is pre-tensioned. It is observed that the highest stress concentrations are in the area of the bolt shank between the head of the bolt and the instance where the nut is tied to it. Some stresses occur in the area of the bolt that is tied to the nut but they are not as significant as the aforementioned.
Figure A.1: Stress distribution in bolt shaft

Since the pre-tensioning of the bolt is dependent on the length of the bolt (or thickness of connecting part), multiple iterations had to be completed in order to obtain the proper value of 35 kips. The final iteration of pre-tensioning required 32 steps in Abaqus standard. The final values for the Mitchell FCB model had a $\Delta T$ of 405.8 degrees for the flange plates and 448.9 degrees for the end plates.
**Table A.1**: Flange bolt data for the final iteration in bolt pre-tensioning

<table>
<thead>
<tr>
<th>Step</th>
<th>Flange Bolt Total Stress (Ksi)</th>
<th>Average Stress (Ksi)</th>
<th>Force in Bolt (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>1</td>
<td>0.060</td>
<td>0.001</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>0.111</td>
<td>0.002</td>
<td>0.001</td>
</tr>
<tr>
<td>3</td>
<td>0.194</td>
<td>0.003</td>
<td>0.001</td>
</tr>
<tr>
<td>4</td>
<td>0.319</td>
<td>0.005</td>
<td>0.002</td>
</tr>
<tr>
<td>5</td>
<td>0.507</td>
<td>0.007</td>
<td>0.003</td>
</tr>
<tr>
<td>6</td>
<td>0.788</td>
<td>0.012</td>
<td>0.005</td>
</tr>
<tr>
<td>7</td>
<td>1.210</td>
<td>0.018</td>
<td>0.008</td>
</tr>
<tr>
<td>8</td>
<td>1.842</td>
<td>0.027</td>
<td>0.012</td>
</tr>
<tr>
<td>9</td>
<td>2.791</td>
<td>0.041</td>
<td>0.018</td>
</tr>
<tr>
<td>10</td>
<td>4.215</td>
<td>0.062</td>
<td>0.027</td>
</tr>
<tr>
<td>11</td>
<td>6.350</td>
<td>0.093</td>
<td>0.041</td>
</tr>
<tr>
<td>12</td>
<td>9.553</td>
<td>0.140</td>
<td>0.062</td>
</tr>
<tr>
<td>13</td>
<td>14.36</td>
<td>0.211</td>
<td>0.093</td>
</tr>
<tr>
<td>14</td>
<td>21.56</td>
<td>0.317</td>
<td>0.140</td>
</tr>
<tr>
<td>15</td>
<td>32.37</td>
<td>0.476</td>
<td>0.210</td>
</tr>
<tr>
<td>16</td>
<td>48.59</td>
<td>0.715</td>
<td>0.316</td>
</tr>
<tr>
<td>17</td>
<td>72.92</td>
<td>1.072</td>
<td>0.474</td>
</tr>
<tr>
<td>18</td>
<td>109.4</td>
<td>1.609</td>
<td>0.711</td>
</tr>
<tr>
<td>19</td>
<td>164.1</td>
<td>2.414</td>
<td>1.066</td>
</tr>
<tr>
<td>20</td>
<td>246.3</td>
<td>3.622</td>
<td>1.600</td>
</tr>
<tr>
<td>21</td>
<td>369.5</td>
<td>5.433</td>
<td>2.400</td>
</tr>
<tr>
<td>22</td>
<td>554.3</td>
<td>8.152</td>
<td>3.601</td>
</tr>
<tr>
<td>23</td>
<td>831.7</td>
<td>12.23</td>
<td>5.403</td>
</tr>
<tr>
<td>24</td>
<td>1248</td>
<td>18.35</td>
<td>8.11</td>
</tr>
<tr>
<td>25</td>
<td>1804</td>
<td>26.53</td>
<td>11.72</td>
</tr>
<tr>
<td>26</td>
<td>2361</td>
<td>34.72</td>
<td>15.34</td>
</tr>
<tr>
<td>27</td>
<td>2918</td>
<td>42.91</td>
<td>18.96</td>
</tr>
<tr>
<td>28</td>
<td>3470</td>
<td>51.02</td>
<td>22.54</td>
</tr>
<tr>
<td>29</td>
<td>4006</td>
<td>58.92</td>
<td>26.03</td>
</tr>
<tr>
<td>30</td>
<td>4533</td>
<td>66.66</td>
<td>29.45</td>
</tr>
<tr>
<td>31</td>
<td>5051</td>
<td>74.28</td>
<td>32.82</td>
</tr>
<tr>
<td>32</td>
<td>5436</td>
<td>79.93</td>
<td>35.31</td>
</tr>
</tbody>
</table>
Table A.2: End plate bolt data for the final iteration in bolt pre-tensioning

<table>
<thead>
<tr>
<th>Step</th>
<th>End Bolt Total Stress (Ksi)</th>
<th>Average Stress (Ksi)</th>
<th>Force in Bolt (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>1</td>
<td>0.063</td>
<td>0.001</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>0.125</td>
<td>0.002</td>
<td>0.001</td>
</tr>
<tr>
<td>3</td>
<td>0.218</td>
<td>0.003</td>
<td>0.001</td>
</tr>
<tr>
<td>4</td>
<td>0.358</td>
<td>0.005</td>
<td>0.002</td>
</tr>
<tr>
<td>5</td>
<td>0.567</td>
<td>0.008</td>
<td>0.003</td>
</tr>
<tr>
<td>6</td>
<td>0.882</td>
<td>0.012</td>
<td>0.005</td>
</tr>
<tr>
<td>7</td>
<td>1.354</td>
<td>0.018</td>
<td>0.008</td>
</tr>
<tr>
<td>8</td>
<td>2.062</td>
<td>0.028</td>
<td>0.012</td>
</tr>
<tr>
<td>9</td>
<td>3.125</td>
<td>0.042</td>
<td>0.019</td>
</tr>
<tr>
<td>10</td>
<td>4.718</td>
<td>0.064</td>
<td>0.028</td>
</tr>
<tr>
<td>11</td>
<td>7.108</td>
<td>0.096</td>
<td>0.042</td>
</tr>
<tr>
<td>12</td>
<td>10.69</td>
<td>0.145</td>
<td>0.064</td>
</tr>
<tr>
<td>13</td>
<td>16.07</td>
<td>0.217</td>
<td>0.096</td>
</tr>
<tr>
<td>14</td>
<td>24.14</td>
<td>0.326</td>
<td>0.144</td>
</tr>
<tr>
<td>15</td>
<td>36.24</td>
<td>0.490</td>
<td>0.216</td>
</tr>
<tr>
<td>16</td>
<td>54.38</td>
<td>0.735</td>
<td>0.325</td>
</tr>
<tr>
<td>17</td>
<td>81.60</td>
<td>1.103</td>
<td>0.487</td>
</tr>
<tr>
<td>18</td>
<td>122.4</td>
<td>1.654</td>
<td>0.731</td>
</tr>
<tr>
<td>19</td>
<td>183.6</td>
<td>2.482</td>
<td>1.096</td>
</tr>
<tr>
<td>20</td>
<td>275.5</td>
<td>3.722</td>
<td>1.645</td>
</tr>
<tr>
<td>21</td>
<td>413.1</td>
<td>5.583</td>
<td>2.466</td>
</tr>
<tr>
<td>22</td>
<td>619.6</td>
<td>8.373</td>
<td>3.699</td>
</tr>
<tr>
<td>23</td>
<td>929.1</td>
<td>12.55</td>
<td>5.547</td>
</tr>
<tr>
<td>24</td>
<td>1393</td>
<td>18.82</td>
<td>8.32</td>
</tr>
<tr>
<td>25</td>
<td>2013</td>
<td>27.20</td>
<td>12.02</td>
</tr>
<tr>
<td>26</td>
<td>2632</td>
<td>35.57</td>
<td>15.72</td>
</tr>
<tr>
<td>27</td>
<td>3250</td>
<td>43.92</td>
<td>19.40</td>
</tr>
<tr>
<td>28</td>
<td>3847</td>
<td>51.99</td>
<td>22.97</td>
</tr>
<tr>
<td>29</td>
<td>4419</td>
<td>59.71</td>
<td>26.38</td>
</tr>
<tr>
<td>30</td>
<td>4979</td>
<td>67.28</td>
<td>29.72</td>
</tr>
<tr>
<td>31</td>
<td>5523</td>
<td>74.63</td>
<td>32.97</td>
</tr>
<tr>
<td>32</td>
<td>5924</td>
<td>80.06</td>
<td>35.37</td>
</tr>
</tbody>
</table>
In order to determine the tension in the bolts, a cross section of the bolt was taken and the tensile force in each element was measured. This total stress was tabulated in the second column of the above tables. That total stress was then divided by the number of elements and then divided again by the area of the bolt. That left the end result as an average tensile force of the cross section. The following Figure A.2 demonstrates how the bolt pre-tensioning was applied in the Abaqus model over the 32 steps.

![Bolt Pre-tensioning](image)

**Figure A.2:** Pre-tension force versus Abaqus steps
Appendix B – Stiffness Trials

Although the FCB was embedded into concrete pier walls, it cannot be assumed that it ever reached a perfectly “fixed-fixed” scenario. For this reason it was required to determine what stiffness the beam was displaying at the wall/beam interface.

Modeling began with releasing the degree of freedom responsible for rotation at the wall/beam interface. For this model it was degree of freedom four. Once it was released it was then replaced by a rotational spring. Initial guesses at stiffness were based on the analytical model’s end moments and an assumed rotation.

B.1 Modeling and Results

Stiffness modeling was first based on a rotation of 2%. In order to guess the stiffness accurately, the stiffness was overestimated and then underestimated. From that the stiffness could be interpolated based on the slope of the desired stiffness. It was found that for a 2% rotation, a rotational stiffness of 570,000 (K-in/rad) was adequate.

Once the 2% case was estimated accurately, the 14% rotation model was then tested. It turned out that the 2% case was an overestimate of the stiffness and the proper value is closer to 330,000 (K-in/rad). Now that the stiffness was determined, cyclic models were created and it was observed that during reverse loading the stiffness was non-linear. The non-linear portion of the stiffness was estimated to be 75,000 (K-in/rad), a significantly lower stiffness than the initial slope.


**Figure B.1**: Non-Linear Stiffness

In order to produce this curve the force and displacement values were found at the ends of the beams in Abaqus. The following data is the Abaqus output for the above curve.
Table B.1: Data for the non-linear stiffness of the FCB

<table>
<thead>
<tr>
<th>Step Time</th>
<th>Displacement (in)</th>
<th>Force (Kips)</th>
<th>Modified Force (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.000</td>
<td>0.000</td>
<td>0.002</td>
<td>-0.002</td>
</tr>
<tr>
<td>1.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>1.000</td>
<td>0.000</td>
<td>-0.003</td>
<td>0.003</td>
</tr>
<tr>
<td>1.000</td>
<td>0.000</td>
<td>-0.007</td>
<td>0.007</td>
</tr>
<tr>
<td>1.000</td>
<td>0.000</td>
<td>-0.014</td>
<td>0.014</td>
</tr>
<tr>
<td>1.000</td>
<td>0.000</td>
<td>-0.025</td>
<td>0.025</td>
</tr>
<tr>
<td>1.000</td>
<td>0.000</td>
<td>-0.040</td>
<td>0.040</td>
</tr>
<tr>
<td>1.000</td>
<td>0.000</td>
<td>-0.063</td>
<td>0.063</td>
</tr>
<tr>
<td>1.000</td>
<td>0.000</td>
<td>-0.098</td>
<td>0.098</td>
</tr>
<tr>
<td>1.001</td>
<td>0.001</td>
<td>-0.150</td>
<td>0.150</td>
</tr>
<tr>
<td>1.001</td>
<td>0.001</td>
<td>-0.229</td>
<td>0.229</td>
</tr>
<tr>
<td>1.001</td>
<td>0.001</td>
<td>-0.346</td>
<td>0.346</td>
</tr>
<tr>
<td>1.002</td>
<td>0.002</td>
<td>-0.522</td>
<td>0.522</td>
</tr>
<tr>
<td>1.003</td>
<td>0.003</td>
<td>-0.787</td>
<td>0.787</td>
</tr>
<tr>
<td>1.004</td>
<td>0.004</td>
<td>-1.183</td>
<td>1.183</td>
</tr>
<tr>
<td>1.006</td>
<td>0.006</td>
<td>-1.778</td>
<td>1.778</td>
</tr>
<tr>
<td>1.009</td>
<td>0.009</td>
<td>-2.671</td>
<td>2.671</td>
</tr>
<tr>
<td>1.013</td>
<td>0.013</td>
<td>-4.008</td>
<td>4.008</td>
</tr>
<tr>
<td>1.020</td>
<td>0.020</td>
<td>-6.014</td>
<td>6.014</td>
</tr>
<tr>
<td>1.030</td>
<td>0.030</td>
<td>-9.017</td>
<td>9.017</td>
</tr>
<tr>
<td>1.044</td>
<td>0.044</td>
<td>-13.513</td>
<td>13.513</td>
</tr>
<tr>
<td>1.067</td>
<td>0.066</td>
<td>-20.246</td>
<td>20.246</td>
</tr>
<tr>
<td>1.100</td>
<td>0.100</td>
<td>-30.342</td>
<td>30.342</td>
</tr>
<tr>
<td>1.150</td>
<td>0.150</td>
<td>-45.475</td>
<td>45.475</td>
</tr>
<tr>
<td>1.224</td>
<td>0.224</td>
<td>-68.043</td>
<td>68.043</td>
</tr>
<tr>
<td>1.249</td>
<td>0.249</td>
<td>-75.534</td>
<td>75.534</td>
</tr>
<tr>
<td>1.287</td>
<td>0.287</td>
<td>-86.730</td>
<td>86.730</td>
</tr>
<tr>
<td>1.343</td>
<td>0.343</td>
<td>-103.408</td>
<td>103.408</td>
</tr>
<tr>
<td>1.428</td>
<td>0.428</td>
<td>-128.103</td>
<td>128.103</td>
</tr>
<tr>
<td>1.528</td>
<td>0.528</td>
<td>-151.690</td>
<td>151.690</td>
</tr>
<tr>
<td>1.628</td>
<td>0.628</td>
<td>-158.903</td>
<td>158.903</td>
</tr>
<tr>
<td>1.728</td>
<td>0.728</td>
<td>-163.237</td>
<td>163.237</td>
</tr>
<tr>
<td>1.828</td>
<td>0.828</td>
<td>-166.739</td>
<td>166.739</td>
</tr>
<tr>
<td>1.928</td>
<td>0.928</td>
<td>-169.760</td>
<td>169.760</td>
</tr>
<tr>
<td>2.000</td>
<td>1.000</td>
<td>-171.756</td>
<td>171.756</td>
</tr>
<tr>
<td>2.000</td>
<td>1.000</td>
<td>-171.756</td>
<td>171.756</td>
</tr>
<tr>
<td>Step Time</td>
<td>Displacement (in)</td>
<td>Force (Kips)</td>
<td>Modified Force (Kips)</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------</td>
<td>--------------</td>
<td>-----------------------</td>
</tr>
<tr>
<td>2.000</td>
<td>1.000</td>
<td>-171.750</td>
<td>171.750</td>
</tr>
<tr>
<td>2.000</td>
<td>1.000</td>
<td>-171.744</td>
<td>171.744</td>
</tr>
<tr>
<td>2.000</td>
<td>1.000</td>
<td>-171.735</td>
<td>171.735</td>
</tr>
<tr>
<td>2.000</td>
<td>1.000</td>
<td>-171.721</td>
<td>171.721</td>
</tr>
<tr>
<td>2.000</td>
<td>1.000</td>
<td>-171.701</td>
<td>171.701</td>
</tr>
<tr>
<td>2.000</td>
<td>1.000</td>
<td>-171.670</td>
<td>171.670</td>
</tr>
<tr>
<td>2.000</td>
<td>1.000</td>
<td>-171.624</td>
<td>171.624</td>
</tr>
<tr>
<td>2.000</td>
<td>0.999</td>
<td>-171.554</td>
<td>171.554</td>
</tr>
<tr>
<td>2.001</td>
<td>0.999</td>
<td>-171.450</td>
<td>171.450</td>
</tr>
<tr>
<td>2.001</td>
<td>0.998</td>
<td>-171.295</td>
<td>171.295</td>
</tr>
<tr>
<td>2.001</td>
<td>0.998</td>
<td>-171.061</td>
<td>171.061</td>
</tr>
<tr>
<td>2.002</td>
<td>0.997</td>
<td>-170.710</td>
<td>170.710</td>
</tr>
<tr>
<td>2.003</td>
<td>0.995</td>
<td>-170.183</td>
<td>170.183</td>
</tr>
<tr>
<td>2.004</td>
<td>0.992</td>
<td>-169.393</td>
<td>169.393</td>
</tr>
<tr>
<td>2.006</td>
<td>0.988</td>
<td>-168.209</td>
<td>168.209</td>
</tr>
<tr>
<td>2.009</td>
<td>0.983</td>
<td>-166.431</td>
<td>166.431</td>
</tr>
<tr>
<td>2.013</td>
<td>0.974</td>
<td>-163.764</td>
<td>163.764</td>
</tr>
<tr>
<td>2.020</td>
<td>0.961</td>
<td>-159.762</td>
<td>159.762</td>
</tr>
<tr>
<td>2.022</td>
<td>0.956</td>
<td>-158.261</td>
<td>158.261</td>
</tr>
<tr>
<td>2.026</td>
<td>0.948</td>
<td>-156.009</td>
<td>156.009</td>
</tr>
<tr>
<td>2.031</td>
<td>0.937</td>
<td>-152.630</td>
<td>152.630</td>
</tr>
<tr>
<td>2.040</td>
<td>0.921</td>
<td>-147.560</td>
<td>147.560</td>
</tr>
<tr>
<td>2.052</td>
<td>0.896</td>
<td>-139.955</td>
<td>139.955</td>
</tr>
<tr>
<td>2.071</td>
<td>0.858</td>
<td>-128.547</td>
<td>128.547</td>
</tr>
<tr>
<td>2.099</td>
<td>0.802</td>
<td>-111.456</td>
<td>111.456</td>
</tr>
<tr>
<td>2.141</td>
<td>0.718</td>
<td>-85.811</td>
<td>85.811</td>
</tr>
<tr>
<td>2.204</td>
<td>0.592</td>
<td>-47.526</td>
<td>47.526</td>
</tr>
<tr>
<td>2.299</td>
<td>0.402</td>
<td>6.151</td>
<td>-6.151</td>
</tr>
<tr>
<td>2.324</td>
<td>0.352</td>
<td>15.917</td>
<td>-15.917</td>
</tr>
<tr>
<td>2.361</td>
<td>0.277</td>
<td>30.487</td>
<td>-30.487</td>
</tr>
<tr>
<td>2.418</td>
<td>0.165</td>
<td>52.211</td>
<td>-52.211</td>
</tr>
<tr>
<td>2.502</td>
<td>-0.004</td>
<td>83.799</td>
<td>-83.799</td>
</tr>
<tr>
<td>2.602</td>
<td>-0.204</td>
<td>116.968</td>
<td>-116.968</td>
</tr>
<tr>
<td>2.627</td>
<td>-0.254</td>
<td>122.582</td>
<td>-122.582</td>
</tr>
<tr>
<td>2.664</td>
<td>-0.329</td>
<td>129.400</td>
<td>-129.400</td>
</tr>
<tr>
<td>2.721</td>
<td>-0.441</td>
<td>137.577</td>
<td>-137.577</td>
</tr>
<tr>
<td>2.805</td>
<td>-0.610</td>
<td>146.592</td>
<td>-146.592</td>
</tr>
<tr>
<td>2.830</td>
<td>-0.660</td>
<td>148.747</td>
<td>-148.747</td>
</tr>
<tr>
<td>Step Time</td>
<td>Displacement (in)</td>
<td>Force (Kips)</td>
<td>Modified Force (Kips)</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------</td>
<td>-------------</td>
<td>----------------------</td>
</tr>
<tr>
<td>2.868</td>
<td>-0.735</td>
<td>151.637</td>
<td>-151.637</td>
</tr>
<tr>
<td>2.924</td>
<td>-0.848</td>
<td>155.351</td>
<td>-155.351</td>
</tr>
<tr>
<td>3.000</td>
<td>-1.000</td>
<td>159.695</td>
<td>-159.695</td>
</tr>
<tr>
<td>3.000</td>
<td>-1.000</td>
<td>159.695</td>
<td>-159.695</td>
</tr>
<tr>
<td>3.000</td>
<td>-1.000</td>
<td>159.695</td>
<td>-159.695</td>
</tr>
<tr>
<td>3.000</td>
<td>-1.000</td>
<td>159.695</td>
<td>-159.695</td>
</tr>
<tr>
<td>3.000</td>
<td>-1.000</td>
<td>159.695</td>
<td>-159.695</td>
</tr>
<tr>
<td>3.000</td>
<td>-0.999</td>
<td>159.523</td>
<td>-159.523</td>
</tr>
<tr>
<td>3.000</td>
<td>-0.999</td>
<td>159.428</td>
<td>-159.428</td>
</tr>
<tr>
<td>3.000</td>
<td>-0.999</td>
<td>159.285</td>
<td>-159.285</td>
</tr>
<tr>
<td>3.000</td>
<td>-0.998</td>
<td>159.071</td>
<td>-159.071</td>
</tr>
<tr>
<td>3.000</td>
<td>-0.997</td>
<td>158.750</td>
<td>-158.750</td>
</tr>
<tr>
<td>3.001</td>
<td>-0.995</td>
<td>158.268</td>
<td>-158.268</td>
</tr>
<tr>
<td>3.001</td>
<td>-0.993</td>
<td>157.545</td>
<td>-157.545</td>
</tr>
<tr>
<td>3.002</td>
<td>-0.990</td>
<td>156.461</td>
<td>-156.461</td>
</tr>
<tr>
<td>3.003</td>
<td>-0.984</td>
<td>154.835</td>
<td>-154.835</td>
</tr>
<tr>
<td>3.004</td>
<td>-0.977</td>
<td>152.397</td>
<td>-152.397</td>
</tr>
<tr>
<td>3.006</td>
<td>-0.965</td>
<td>148.741</td>
<td>-148.741</td>
</tr>
<tr>
<td>3.009</td>
<td>-0.947</td>
<td>143.258</td>
<td>-143.258</td>
</tr>
<tr>
<td>3.013</td>
<td>-0.921</td>
<td>135.034</td>
<td>-135.034</td>
</tr>
<tr>
<td>3.020</td>
<td>-0.881</td>
<td>122.699</td>
<td>-122.699</td>
</tr>
<tr>
<td>3.030</td>
<td>-0.821</td>
<td>104.205</td>
<td>-104.205</td>
</tr>
<tr>
<td>3.044</td>
<td>-0.732</td>
<td>76.518</td>
<td>-76.518</td>
</tr>
<tr>
<td>3.067</td>
<td>-0.598</td>
<td>35.390</td>
<td>-35.390</td>
</tr>
<tr>
<td>3.100</td>
<td>-0.397</td>
<td>-25.082</td>
<td>25.082</td>
</tr>
<tr>
<td>3.150</td>
<td>-0.095</td>
<td>-82.433</td>
<td>82.433</td>
</tr>
<tr>
<td>3.168</td>
<td>0.018</td>
<td>-103.032</td>
<td>103.032</td>
</tr>
<tr>
<td>3.196</td>
<td>0.187</td>
<td>-131.628</td>
<td>131.628</td>
</tr>
<tr>
<td>3.207</td>
<td>0.251</td>
<td>-139.993</td>
<td>139.993</td>
</tr>
<tr>
<td>3.223</td>
<td>0.346</td>
<td>-148.837</td>
<td>148.837</td>
</tr>
<tr>
<td>3.246</td>
<td>0.490</td>
<td>-157.598</td>
<td>157.598</td>
</tr>
<tr>
<td>3.282</td>
<td>0.704</td>
<td>-166.713</td>
<td>166.713</td>
</tr>
<tr>
<td>3.335</td>
<td>1.026</td>
<td>-175.308</td>
<td>175.308</td>
</tr>
<tr>
<td>3.355</td>
<td>1.147</td>
<td>-177.862</td>
<td>177.862</td>
</tr>
<tr>
<td>3.385</td>
<td>1.328</td>
<td>-181.317</td>
<td>181.317</td>
</tr>
<tr>
<td>3.430</td>
<td>1.600</td>
<td>-185.859</td>
<td>185.859</td>
</tr>
<tr>
<td>3.497</td>
<td>2.007</td>
<td>-191.648</td>
<td>191.648</td>
</tr>
<tr>
<td>Step Time</td>
<td>Displacement (in)</td>
<td>Force (Kips)</td>
<td>Modified Force (Kips)</td>
</tr>
<tr>
<td>-----------</td>
<td>-----------------</td>
<td>--------------</td>
<td>----------------------</td>
</tr>
<tr>
<td>3.597</td>
<td>2.612</td>
<td>-198.737</td>
<td>198.737</td>
</tr>
<tr>
<td>3.622</td>
<td>2.763</td>
<td>-200.311</td>
<td>200.311</td>
</tr>
<tr>
<td>3.660</td>
<td>2.990</td>
<td>-202.542</td>
<td>202.542</td>
</tr>
<tr>
<td>3.716</td>
<td>3.330</td>
<td>-205.630</td>
<td>205.630</td>
</tr>
<tr>
<td>3.737</td>
<td>3.458</td>
<td>-206.715</td>
<td>206.715</td>
</tr>
<tr>
<td>3.745</td>
<td>3.505</td>
<td>-207.116</td>
<td>207.116</td>
</tr>
<tr>
<td>3.757</td>
<td>3.577</td>
<td>-207.718</td>
<td>207.718</td>
</tr>
<tr>
<td>3.775</td>
<td>3.685</td>
<td>-208.609</td>
<td>208.609</td>
</tr>
<tr>
<td>3.802</td>
<td>3.846</td>
<td>-209.911</td>
<td>209.911</td>
</tr>
<tr>
<td>3.812</td>
<td>3.907</td>
<td>-210.386</td>
<td>210.386</td>
</tr>
<tr>
<td>3.827</td>
<td>3.997</td>
<td>-211.087</td>
<td>211.087</td>
</tr>
<tr>
<td>3.849</td>
<td>4.134</td>
<td>-212.104</td>
<td>212.104</td>
</tr>
<tr>
<td>3.883</td>
<td>4.338</td>
<td>-213.564</td>
<td>213.564</td>
</tr>
<tr>
<td>3.934</td>
<td>4.644</td>
<td>-215.603</td>
<td>215.603</td>
</tr>
<tr>
<td>3.950</td>
<td>4.745</td>
<td>-216.251</td>
<td>216.251</td>
</tr>
<tr>
<td>3.975</td>
<td>4.895</td>
<td>-217.198</td>
<td>217.198</td>
</tr>
<tr>
<td>3.981</td>
<td>4.933</td>
<td>-217.432</td>
<td>217.432</td>
</tr>
<tr>
<td>3.991</td>
<td>4.990</td>
<td>-217.801</td>
<td>217.801</td>
</tr>
<tr>
<td>4.000</td>
<td>5.046</td>
<td>-218.171</td>
<td>218.171</td>
</tr>
<tr>
<td>4.000</td>
<td>5.046</td>
<td>-218.171</td>
<td>218.171</td>
</tr>
</tbody>
</table>

By observation it was apparent that the data from Abaqus had the opposite sign when compared to the experimental data. The column labeled “Modified Force” is simply the force times negative one.

The modified stiffness, calculated using the modified force, was inputted into Abaqus through the command called “Non Linear Spring”. It requires a minimum of three points to describe the behavior of the spring being modified. Figure B.2 demonstrates how the command was implemented. The positive stiffness values correspond to a stiffness of 330,000 (K-in/rad) while the negative values correspond to 75,000 (K-in/rad). The point that the stiffness changes is defined as the float point and in this instance it was defined at a moment of 0 (K-in) and a rotation of 0 (rad) (Dassault Systemes 2011b).
Concurrent with the non-linear springs was the command called “Model Change”. This command allows the user to remove or add multiple different objects in Abaqus. In this case, it was used to remove and add different springs so a different spring stiffness could be used for each step of the model (Dassault Systemes 2011b).

**Figure B.2:** Non-Linear Stiffness
Appendix C – Material Properties

During the experimental stage for the FCB, steel coupons were ordered from the steel distributor. These coupons all had varying thicknesses in order to properly define the material properties. In every case, the yield stress was higher than the specified 50 ksi (Mitchell 2012).

In order to properly input the material data from the experiment into the Abaqus model, the experimental data had to be condensed. The strain gages recorded thousands of data points. This amount of data would cause a single Abaqus run to take an enormous amount of time to run. Due to this, for each thickness of coupon data, a select number of points were used. From these points the material data was then increased by 10% for both stress and strain and then converted to true stress and true plastic strain (see section 4.3).

C.1 Modeling and Results

For each coupon, data points were manually selected in order to mimic the stress-strain curves with all data points included. The number of points were selected arbitrarily for each coupon and visually inspected for accuracy. The following figures and graphs demonstrate the material data that was used for Abaqus modeling. Note that the true stress and strain values have been increased by 10% in the figures.
Table C.1: Abaqus input data for 1/4” coupon

<table>
<thead>
<tr>
<th>True Stress (ksi)</th>
<th>True Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>66.24</td>
<td>0.0000</td>
</tr>
<tr>
<td>66.35</td>
<td>0.0093</td>
</tr>
<tr>
<td>66.88</td>
<td>0.0145</td>
</tr>
<tr>
<td>68.24</td>
<td>0.0182</td>
</tr>
<tr>
<td>69.67</td>
<td>0.0216</td>
</tr>
<tr>
<td>71.00</td>
<td>0.0251</td>
</tr>
<tr>
<td>72.01</td>
<td>0.0285</td>
</tr>
<tr>
<td>73.30</td>
<td>0.0320</td>
</tr>
<tr>
<td>74.07</td>
<td>0.0354</td>
</tr>
<tr>
<td>77.29</td>
<td>0.0445</td>
</tr>
<tr>
<td>80.76</td>
<td>0.0611</td>
</tr>
<tr>
<td>83.65</td>
<td>0.0790</td>
</tr>
<tr>
<td>86.18</td>
<td>0.0975</td>
</tr>
<tr>
<td>88.06</td>
<td>0.1170</td>
</tr>
<tr>
<td>89.95</td>
<td>0.1335</td>
</tr>
</tbody>
</table>

Figure C.1: Original 1/4” coupon data versus true stress/strain values
Table C.2: Abaqus input data for 3/8” coupon

<table>
<thead>
<tr>
<th>True Stress (ksi)</th>
<th>True Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>62.78</td>
<td>0.0000</td>
</tr>
<tr>
<td>63.33</td>
<td>0.0079</td>
</tr>
<tr>
<td>64.06</td>
<td>0.0098</td>
</tr>
<tr>
<td>65.63</td>
<td>0.0135</td>
</tr>
<tr>
<td>66.97</td>
<td>0.0172</td>
</tr>
<tr>
<td>68.80</td>
<td>0.0226</td>
</tr>
<tr>
<td>69.94</td>
<td>0.0262</td>
</tr>
<tr>
<td>71.80</td>
<td>0.0333</td>
</tr>
<tr>
<td>72.24</td>
<td>0.0351</td>
</tr>
<tr>
<td>76.67</td>
<td>0.0515</td>
</tr>
<tr>
<td>79.56</td>
<td>0.0692</td>
</tr>
<tr>
<td>81.85</td>
<td>0.0874</td>
</tr>
<tr>
<td>83.87</td>
<td>0.1068</td>
</tr>
<tr>
<td>86.75</td>
<td>0.1385</td>
</tr>
</tbody>
</table>

Figure C.2: Original 3/8” coupon data versus true stress/strain values
Table C.3: Abaqus input data for 1/2” coupon

<table>
<thead>
<tr>
<th>True Stress (ksi)</th>
<th>True Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>62.83</td>
<td>0.0000</td>
</tr>
<tr>
<td>65.26</td>
<td>0.0051</td>
</tr>
<tr>
<td>67.94</td>
<td>0.0106</td>
</tr>
<tr>
<td>70.72</td>
<td>0.0164</td>
</tr>
<tr>
<td>73.10</td>
<td>0.0230</td>
</tr>
<tr>
<td>75.48</td>
<td>0.0307</td>
</tr>
<tr>
<td>77.51</td>
<td>0.0396</td>
</tr>
<tr>
<td>79.10</td>
<td>0.0493</td>
</tr>
<tr>
<td>80.87</td>
<td>0.0602</td>
</tr>
<tr>
<td>82.18</td>
<td>0.0717</td>
</tr>
<tr>
<td>83.64</td>
<td>0.0852</td>
</tr>
<tr>
<td>85.12</td>
<td>0.1002</td>
</tr>
</tbody>
</table>

Figure C.3: Original 1/2” coupon data versus true stress/strain values
### Table C.4: Abaqus input data for 5/8” coupon

<table>
<thead>
<tr>
<th>True Stress (ksi)</th>
<th>True Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>68.40</td>
<td>0.0000</td>
</tr>
<tr>
<td>72.21</td>
<td>0.0011</td>
</tr>
<tr>
<td>77.02</td>
<td>0.0081</td>
</tr>
<tr>
<td>80.69</td>
<td>0.0170</td>
</tr>
<tr>
<td>84.13</td>
<td>0.0276</td>
</tr>
<tr>
<td>87.69</td>
<td>0.0420</td>
</tr>
<tr>
<td>90.71</td>
<td>0.0602</td>
</tr>
<tr>
<td>92.28</td>
<td>0.0709</td>
</tr>
<tr>
<td>94.10</td>
<td>0.0845</td>
</tr>
</tbody>
</table>

**Figure C.4:** Original 5/8” coupon data versus true stress/strain values

---

77
In Abaqus it is possible to assign the above material properties to different parts of the FCB based on thickness. For example: the 1/4” coupon data was assigned to the fuse link end plates while the 1/2” data was assigned to the web. In the case where the flange of the embedded beam was 3/4” thick, the material data for the 5/8” coupon data was used.
Appendix D – Example Calculation

The following discussion and example will give a detailed explanation of how to design a FCB. It will contain the modified design procedure infused with Mitchell’s design parameters.

D.1 Loading and Material Properties

In Mitchell’s design he determined load demands on a 20-story structure in ETABS. The structure he modeled incorporated a full scale model of the FCB. In the experiment that he performed and even in the Abaqus models, a half scale model was used. The loads were taken from ETABS for story’s 5-8. The shear demand and moment demands are 392 kips and 593 k-ft, respectively. For more information on Mitchell’s ETABS model, refer to Mitchell’s Thesis.

Since the ETABS model was for the “real life” dimensions of the FCB and the experimental model was reduced to a half scale model of the FCB, the forces found in Mitchell’s analysis needed to be reduced accordingly. When scaling a model down, material strengths do not change. Knowing that force is a stress times area, therefore when cutting dimensions in half it means cutting areas by four. This implies that forces on the half scale model are a fourth of the full scale model. Since moment is force times distance, the moment on the half scale model is the force in the scaled model times the scaled distance which is one half by one fourth, totaling one eighth. The scaled forces are calculated in the following steps:

Shear Demand:

\[ V_u = \frac{V_{ETABS}}{4} = \frac{392 \text{kips}}{4} = 98.00 \text{ kips} \]
Moment Demand:

\[ M_u = \frac{M_{ETABS}}{8} = \frac{592^{k-ft}}{8} = 74.00 \text{ k-ft} \]

Material Properties:

Material properties are taken as standard properties and not the results from coupon testing. All steel is A572 Grade 50, therefore:

\[ F_y = 50 \text{ ksi}; \quad F_u = 65 \text{ ksi}; \quad E = 29,000 \text{ ksi} \]

D.2 Fuse Link Design

The model provided in Mitchell’s thesis gave recommended values for the widths and thicknesses of the web and flanges. However, these dimensions are found using an iterative process and new dimensions may be assigned later in this design. The following dimensions are the same dimensions that Mitchell used in his experiment.

Dimensions:

\[ t_{wf} = 0.375 \text{ in} \]
\[ h_{wf} = 9.75 \text{ in} \]
\[ b_{ff} = 9.00 \text{ in} \]
\[ t_{ff} = 0.625 \text{ in} \]
\[ e = 18 \text{ in} \]
Fuse Design Shear Strength:

If an axial load can be determined through a pushover analysis or a finite element analysis, it should then be considered for the design of the fuse link. In either case the provisions provided in AISC 341-10 should be considered. The following case is for no axial load.

\[ V_{nf} = 0.6F_yA_{lw} = 0.6 \times 50^{ksi} \times (9.75^{ln} \times 0.375^{ln}) = 109.7 \text{ kips} \]  
(AISC 341-10, F3-2)

\[ \phi V_{nf} = \phi 0.6F_yA_{lw} = 0.9 \times 109.7 \text{ kips} = 98.72 \text{ kips} > V_u = 98.00 \text{ kips} \]  
OKAY

Web Ductility:

Since there is no axial load the value for \( C_a \) from AISC can be taken as zero.

\[ \lambda_{hd} = 2.45 \sqrt{\frac{E}{F_y}} (1 - 0.93C_a) = 2.45 \sqrt{\frac{29000^{ksi}}{50^{ksi}} (1 - 0)} = 59.00 \]  
(AISC 341-10 Table D1.1)

\[ \frac{h_{wf}}{t_{wf}} = \frac{9.75^{ln}}{0.375^{ln}} = 26.00 < \lambda_{hd} = 59.00 \]  
OKAY

This means that the web of the section can be considered highly ductile.

Flange Ductility:

The flanges of the fuse only need to satisfy requirements for moderately ductile members.

\[ \lambda_{md} = 0.38 \sqrt{\frac{E}{F_y}} = 0.38 \sqrt{\frac{29000^{ksi}}{50^{ksi}}} = 9.152 \]  
(AISC 341-10 Table D1.1)
\[
\frac{b_{ff}}{2t_{ff}} = \frac{9.00^{in}}{(2 \times 0.625^{in})} = 7.20 < \lambda_{ma} = 9.152
\]

**Fuse Design Flexural Strength:**

\[
M_n = M_p = F_y Z
\]

\[
Z = \frac{A}{2} \times a
\]

\[
a = 2 \left( \frac{\left(\left(\frac{9.75^{in}}{2}\right) \times 0.375^{in}\right) \times 9.75^{in} / 4} {\left(\frac{9.75^{in}}{2}\right) \times 0.375^{in} + (9.00^{in} \times 0.625^{in})} \right) + \left(9.00^{in} \times 0.625^{in}\right)
\]

\[
a = 9.026^{in}
\]

\[
\frac{A}{2} = \left[\frac{0.375^{in} \times 9.75^{in} + (2 \times 9.00^{in} \times 0.625^{in})}{2}\right] = 7453^{in^2}
\]

\[
Z = 7453^{in^2} \times 9.026^{in} = 67.27^{in^3}
\]

\[
M_{nf} = M_{pf} = F_y Z = 50^{k_{sl}} \times 67.27^{in^3} = 3364 \text{kips in} = 280.3 \text{kips ft}
\]

\[
\phi M_{pf} = 0.9 \times 280.3^{kip-ft} = 252.3 \text{kips ft} > M_u = 74.00 \text{k ft}
\]

Before moving on in the calculations be sure to check and see if \(V_n\) for flexural yielding controls the design.

\[
V_{nm} = \frac{2M_{pf}}{e} = 2 \times \frac{3364^{kip-in}}{18^{in}} = 373.7 \text{kips} > V_{nf} = 109.7 \text{kips}
\]

Therefore \(V_{nf}\) Controls
Design Fuse to be Shear Critical:

\[ e \leq \frac{1.6M_{pf}}{V_{nf}} = 1.6 \times \frac{3364^{k-in}}{109.7\text{ kips}} = 49.06 \text{ in} > e = 18 \text{ in} \quad \text{OKAY} \]

Fuse Stiffeners:

Assuming a rotation of 0.08 radians is reached, the following AISC equation applies to stiffener spacing:

\[ \text{Stiffener Spacing} = 30t_{wf} - \frac{d}{5} = 30 \times 0.375^{in} - \frac{11^{in}}{5} = 9.05 \text{ in} \quad \text{(AISC 341-10, F3-3)} \]

Therefore, place one stiffener at the mid-span of the fuse link beam with a width and thickness of:

\[ b_{sp} = \frac{(b_{ff} - t_{wf})}{2} = \frac{(9^{in} - 0.375^{in})}{2} = 4.313 \text{ in} \]

\[ t_{sp} = \text{Greater of } (0.75 \times t_{wf})\text{ and } \frac{3}{8} \text{ in} = \text{Max}[(0.75 \times 0.375^{in}), \frac{3}{8} \text{ in}] = 0.375 \text{ in} \]
D.3  Embedded Beam Design

At this point the preliminary fuse link dimensions satisfy all requirements. Designing the embedded beam will be a two-step process:

1. Design the embedded beam for shear strength using the overstrength factor, $\Omega_{FCB}$, taken as 1.20.

2. After calculating the shear demand and strength, using a shear demand that does not employ the overstrength factor, design the beam for flexure. If the dimensions for the fuse change, the previous calculations in section D.2 need to be reconsidered.

**Dimensions:**

$t_{we} = 0.5$ in

$h_{we} = 9.75$ in

$b_{fe} = 9.00$ in

$t_{fe} = 0.625$ in

$L_{emb} = 9$ in

**Shear Demand on Embedded Beam:**

To assure shear yielding in the embedded beam occurs prior to shear yielding in the link beam, an overstrength factor, $\Omega_{FCB}$, is introduced. This factor is applied only to the shear demand on the embedded beam and will be taken as 1.20.

**Calculate Shear Demand:**

The overstrength shear demand on the embedded beam is:

$$V_{u,emb-\Omega} = 1.1R_y\Omega_{FCB}V_{nf} = 1.1 \times 1.1 \times 1.20 \times 109.7^{kips} = 159.3\ kips$$
Where $R_s$ is the ratio of the expected yield stress to the specified minimum yield stress of that material and 1.1 accounts for material strain hardening (AISC 341-10 Table A3.1).

**Calculate Shear Strength of Embedded beam:**

Based on experimental and analytical results it is known that the dimension for web thickness of 0.5 inches is too small, try a new web thickness of 0.625 inches.

$$V_{ne} = 0.6F_y h_{we} t_{we} = 0.6 \times 50^{ksi} \times 9.75^{in} \times 0.625^{in} = 182.8 \text{kips}$$

$$\phi V_{ne} = 0.9 \times 182.8^{kips} = 164.5 \text{kips} > V_{u,emb-\Omega} = 159.3 \text{kips} \quad \text{OKAY}$$

By changing $t_{we}$ from 0.5 inches to 0.625 inches, the shear strength is adequate.

**Moment Demand on Embedded Beams:**

The moment transferred from the fuse link to the embedded beam shown in the following figure.

**Figure D.1:** Moment and shear transfer between fuse link (Right) and embedded beam (Left)
The overstrength shear force is not required to solve for the moment demand on the embedded beam and is taken as:

\[ V_{u,emb} = 1.1R_yV_{nf} \]

\[ V_{u,emb} = 1.1 \times 1.1 \times 109.7 \text{kips} = 132.7 \text{kips} \]

\[ M_{tr} = V_{u,emb} \times \left( \frac{e}{2} \right) = 132.7 \text{kips} \times \left( \frac{18\text{in}}{2} \right) = 1194 \text{k-in} \]

\[ M_{wall} = M_{tr} + (V_{u,emb} \times L_{emb}) = 1194 \text{k-in} + (132.7 \text{kips} \times 9\text{in}) = 2389 \text{k-in} \]

\[ M_{wall} = 199.1 \text{k-ft} \]

**Flexural Strength:**

This step of the design procedure is the first instance where the procedure has been modified. Rather than using \( I_{xx} \) and \( S_e \) for the gross area at the wall face, \( I_{xx,net} \) and \( S_{e,net} \) are found at the net section which is the row of bolts closest to the wall face.

\[ I_{xx} = \left( \frac{(9\text{in} \times (11.00\text{in}))^3}{12} \right) - 2 \times \left( \frac{9\text{in}}{2} \right) - \left( \frac{0.625\text{in}}{2} \right) \times (9.75\text{in})^3 \]

\[ I_{xx} = 351.4 \text{in}^4 \]

\[ S_e = \frac{I_{xx}}{d/2} = \frac{351.4\text{in}^4}{11.00\text{in}/2} = 63.89 \text{in}^3 \]
\[ I_{xx,\text{net}} = I_{xx} - \left[ 8 \times \left( \frac{(0.8125 \text{ in} \times (0.625 \text{ in})^3)}{12} \right) + \left( 8 \times (0.8125 \text{ in} \times 0.625 \text{ in} \times (5.19 \text{ in})^2) \right) \right] \]

\[ I_{xx,\text{net}} = 351.4 \text{ in}^4 - 109.5 \text{ in}^4 = 241.9 \text{ in}^4 \]

\[ S_{e,\text{net}} = \frac{I_{xx,\text{net}}}{d/2} = \frac{241.9 \text{ in}^4}{11.00 \text{ in}/2} = 43.99 \text{ in}^3 \]

Notice the difference between the gross and net sections for the moment of inertia and the section modulus. Having decreased values will lower the flexural strength of the embedded beam thus forcing the use of a thicker flange. The original design called for a flange thickness of 0.625 inches but as shown in the next step it required a thickness of 0.75\text{in} to be adequate in flexure.

\[ M_{y,\text{net}} = S_{e,\text{net}} \times F_y = 43.99 \text{ in}^3 \times 50 \text{ ksi} = 2199 \text{ k } \text{ in} \]

\[ M_{y,\text{net}} = 183.3 \text{ k } \text{ ft} < M_{wall} = 199.1 \text{ k } \text{ ft} \quad \text{NOT OKAY} \]

Since the flexural check failed, try using a flange thickness of 0.75 inches. By using a flange thickness of 0.75 inches, the fuse web height needs to increase by 0.25 inches. Note that \( d \) is now 11.25 inches. The following values have been updated:

\[ V_{nf} = 112.5 \text{ kips} \]

\[ V_{u,emb} = 136.1 \text{ kips} \]

\[ M_{tr} = 1225 \text{ k } \text{ in} = 102.1 \text{ k } \text{ ft} \]

\[ M_{wall} = 2450 \text{ k } \text{ in} = 204.2 \text{ k } \text{ ft} \]
$I_{xx} = 421.0 \text{ in}^4$

$S_e = 74.84 \text{ in}^3$

$I_{xx,\text{net}} = 286.4 \text{ in}^4$

$S_{e,\text{net}} = 50.92 \text{ in}^3$

$M_{y,\text{net}} = S_{e,\text{net}} \times F_y = 50.92\text{in}^3 \times 50^{\text{ksi}} = 2546 \text{ k-in}$

$M_{y,\text{net}} = 212.2 \text{ k-ft} > M_{\text{wall}} = 204.2 \text{ k-ft}$

**OKAY**

**Embedded Beam Stiffeners:**

$Stiffener \text{ Spacing} = 30t_{we} - d/5 = 30 \times 0.625^{\text{in}} - 11.25^{\text{in}}/5 = 16.50 \text{ in}$

(AISC 341-10, F3-3)

In order to keep spalling of the concrete at the wall/beam interface to a minimum. A stiffener should be placed at that location.

$b_{sp} = (b_{fe} - t_{we})/2 = (9^{\text{in}} - 0.625^{\text{in}})/2 = 4.188 \text{ in}$

$t_{sp} = Greater \ of \ (0.75 \times t_{we}) and 3/8^{\text{in}} = \text{Max}[(0.75 * 0.625^{\text{in}}), 3/8^{\text{in}}] = 0.4688 \text{ in}$
D.4 Final Design

The following table and figure represent the final dimensions after design.

Final Beam Dimensions:

**Table D.1: Beam Final Dimensions**

<table>
<thead>
<tr>
<th>Embedded Beam</th>
<th>Dimension (in)</th>
<th>Link Beam</th>
<th>Dimension (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_{we}$</td>
<td>0.625</td>
<td>$t_{wf}$</td>
<td>0.375</td>
</tr>
<tr>
<td>$h_{we}$</td>
<td>9.75</td>
<td>$h_{wf}$</td>
<td>10.00</td>
</tr>
<tr>
<td>$b_{fe}$</td>
<td>9.00</td>
<td>$b_{ff}$</td>
<td>9.00</td>
</tr>
<tr>
<td>$t_{fc}$</td>
<td>0.75</td>
<td>$t_{ff}$</td>
<td>0.625</td>
</tr>
</tbody>
</table>

**Figure D.2:** Fuse (Left) and embedded (Right) beam notations
D.5 Connection Design

The FCB has two main connections through the use of flange splice plates and end plates. They help to transfer the shears and moments that the beam feels. When designing the connections it is important to keep symmetry in mind. The updated FCB investigated in this report will maintain the same geometry as Mitchell’s beam in order to maintain a certain level of similarity between the two procedures. The following checks will be made in order to verify plate thicknesses and number of bolts needed per connection:

- Connections will be slip critical
- Shear strength of bolts
- Shear rupture and yielding of end plates
- Bearing capacities and plowing of end plates
- Moment connection strength (flange plates)
- Determine thickness of flange plates
- Bearing and tear-out in flange splice plates
- Block shear failure in flange splice plates

Since the Abaqus model represented bolts without threads, the following calculations will be using 3/4in A490-X bolts. Note that all values being used to compute connection capacities come from the calculations for the beam with its final dimensions.

Slip Critical Connections (End Plates):

For the beam to be considered slip critical, the number of bolts selected for each end plate is dependent on the shear demand on the embedded beam $V_{u,emb}$, which is equal to 136.1 kips.
For:

\[ r_n = u \times D_u \times h_f \times T_b \times n_s \quad \text{(AISC 360-10, J3-8)} \]

\[ u = 0.5 \text{ (Class B Faying Surface)} \]

\[ D_u = 1.13 \text{ (Constant)} \]

\[ h_f = 1.00 \text{ (No Fillers)} \]

\[ T_b = 35 \text{ kips (3/4'' A490 Bolts)} \]

\[ n_s = 1.00 \text{ (Number of Slip Planes)} \]

\[ \phi = 1.00 \text{ (AISC 360-10, J3-4)} \]

\[ r_n = u \times D_u \times h_f \times T_b \times n_s = 0.5 \times 1.13 \times 1.00 \times 35^{kips} \times 1.00 = 19.78 \text{ kips} \]

\[ \phi r_n = 1.00 \times 19.78^{kips} = 19.78 \text{ kips} \]

\[ \text{Number of Bolts} = \frac{V_{u,emb}}{\phi r_n} = \frac{136.1^{kips}}{19.78^{k/bolt}} = 6.88 \text{ bolts, use 10} \]

Ten bolts are used in order to keep consistency with Mitchell’s experiment.

**Shear Strength of Bolts:**

The AISC Specification for the shear strength of bolted connections is dependent on connection length. The work by Raymond H.R. Tide titled “Bolt Shear Design Considerations”, states that if a connection is end loaded with a fastener pattern length greater than 38 inches, it is then applicable to reduce the values for \( F_{nv} \) found in AISC Table J3.2 by 83.3%. In this calculation it is not applicable to use such a reduction since the connection length is less than 38
inches and it is not an end-loaded condition. Note this is a change from Mitchell’s design since he used the 2005 Specifications in a different manner.

\[
\phi R_n = 0.75 \times \phi F_{nv} \times A_b \times \text{number of bolts} = 0.75 \times 84^{ksi} \times 0.4418^{in^2} \times 10^{bolts}
\]

\[
\phi R_n = 278.3 \text{ kips} > V_{u,emb} = 136.1 \text{ kips} \quad \text{(AISC 360-10, J3-1)} \quad \text{OKAY}
\]

**Bearing Capacity and Plowing in End Plates:**

In the end plates, the failure mode that the beam will be designed for is for plowing of the bolts through the fuse end plates prior to the embedded beam end plates. For the bearing capacity of the fuse end plate when deformation is not considered:

\[
\phi R_n = 0.75(3dtF_u n^{bolts}) = 0.75(3 \times 0.75^{in} \times 0.25^{in} \times 65^{ksi} \times 10^{bolts})
\]

\[
\phi R_n = 274.2 \text{ kips} < \text{Shear Capacity} = 278.3 \text{ kips} \quad \text{(AISC 360-10, J3-10b)} \quad \text{OKAY}
\]

For the bearing capacity of the embedded beam when deformation is considered:

\[
\phi R_n = 0.75(2.4dtF_u n^{bolts}) = 0.75(2.4 \times 0.75^{in} \times 0.375^{in} \times 65^{ksi} \times 10^{bolts})
\]

\[
\phi R_n = 329.1 \text{ kips} > \text{Shear Capacity} = 278.3 \text{ kips} \quad \text{(AISC 360-10, J3-10a)} \quad \text{OKAY}
\]

Since the bearing capacity is lower than the shear strength of the bolts in the fuse but not the embedded beams, plowing will occur in the fuse end plates prior to the embedded beams. Tear out is not a concern for the FCB. For tear-out to occur, the flange plate would have to fail first therefore meaning tear-out is not applicable.
Shear Rupture and Yield of End Plates:

For shear rupture in the fuse end plate, the flange thicknesses are not considered when calculating capacity.

\[ \phi R_n = 0.6 F_u A_{nv} \quad (\text{AISC 360-10, J4-2a}) \]

\[ A_{nv} = 2h_{wf} t = 2 \times 10.00^{\text{in}} \times 0.25^{\text{in}} = 5.00 \text{ in}^2 \]

\[ \phi R_n = 0.6 \times 65^{ksi} \times 5.00^{\text{in}^2} = 195.0 \text{ kips} > V_{u,emb} = 136.1 \text{ kips} \quad \text{OKAY} \]

Similarly to the previous calculation, contributions from the flanges will be ignored for shear yielding of the embedded beam.

\[ \phi R_n = 0.6 F_y A_g \quad (\text{AISC 360-10, J4-2b}) \]

\[ A_g = 2h_{we} t = 2 \times 9.75^{\text{in}} \times 0.375^{\text{in}} = 7.313 \text{ in}^2 \]

\[ \phi R_n = 0.6 \times 50^{ksi} \times 7.313^{\text{in}^2} = 219.4 \text{ kips} > V_{u,emb} = 136.1 \text{ kips} \quad \text{OKAY} \]

Moment Connections:

The force at the connection is as follows:

\[ P_{M-\text{conn}} = \frac{M_r}{d} = \frac{1225^{k-in}}{11.25^{\text{in}}} = 108.9 \text{ kips} \]

Since the connection will be designed to be slip critical, the following relationship applies:

\[ \text{number of bolts} = \frac{P_{M-\text{conn}}}{\phi R_n} = \frac{108.9^{\text{kips}}}{19.78^{k/bolt}} = 5.51 \text{ bolts, use 8} \]
Again, the same number of bolts Mitchell used will be used again in order to stay consistent while also maintaining symmetry.

**Determine the Thickness of Flange Plates:**

The following figure demonstrates the bolt spacing to be used throughout the rest of the design.

![Figure D.3: Bolt spacing and edge distances](image)

Taking the thickness of the flange splice plate as $t_{sp} = 0.625\text{in.}$, the force in the splice plate is solved similarly to $P_{M-Conn}$.

$$P_{splice} = \frac{M_{tr}}{(d + t_{sp})} = \frac{1225 \text{k-in}}{(11.25 \text{in} + 0.625 \text{in})} = 103.2 \text{kips}$$
Check gross-section yielding for flange plate:

$$\phi R_n = \phi F_y A_g = 0.9 \times 50^{k_{si}} \times (0.625^{in} \times 9^{in}) = 253.1 \text{ kips} > P_{splice} = 103.2 \text{ kips}$$

(AISC 360-10, J4-1) **OKAY**

Check net-section fracture for flange plate:

$$\phi R_n = \phi F_u A_n U$$

$$U = 1.00$$

$$A_n = t_{sp} \left( b_{sp} - \left( \frac{\text{number of bolts}}{2} \right) \times \left( \phi_{bolts} + \frac{1}{8^{in}} \right) \right)$$

$$A_n = 0.625^{in} \left( 9^{in} - \left( \frac{8^{bolts}}{2} \right) \times \left( 0.75^{in} + \frac{1}{8}^{2} \right) \right) = 3.438 \text{ in}^2$$

$$\phi P_n = 0.75 \times 65^{ksi} \times 3.438^{in^2} \times 1.00 = 167.6 \text{ kips} > P_{splice} = 103.2 \text{ kips}$$

(AISC 360-10, J4-1) **OKAY**

Check Bearing and Tear-out in Flange Splice Plates:

$$R_n = \text{Min. of } \begin{cases} 1.2 L_c t F_u \\ 2.4 d t F_u \end{cases}$$

(AISC 360-10, J3-10a)

**Exterior Bolts:**

$$R_n = 1.2 \times \left( 1.00^{in} - \left( \frac{0.75^{in} + \frac{1}{16}^{in}}{2} \right) \right) \times 0.625^{in} \times 65^{ksi} = 28.95 \text{ kips}$$

$$R_n = 2.4 \times 0.75^{in} \times 0.625^{in} \times 65^{ksi} = 73.13 \text{ kips}$$
$R_n = 28.95 \text{kips} \quad \text{Controls}$

Interior Bolts:

$$R_n = 1.2 \times \left( 2.25^{\text{in}} - \left( 0.75^{\text{in}} + \frac{1}{16}^{\text{in}} \right) \right) \times 0.625^{\text{in}} \times 65^{\text{ksi}} = 70.08 \text{kips}$$

$$R_n = 2.4 \times 0.75^{\text{in}} \times 0.625^{\text{in}} \times 65^{\text{ksi}} = 73.13 \text{kips}$$

$$R_n = 70.08 \text{kips} \quad \text{Controls}$$

Total Capacity:

$$\phi R_n = 0.75 \left( 4^{\text{bolts}} \times 28.95^{k/bolt} \right) + \left( 4^{\text{bolts}} \times 70.08^{k/bolt} \right)$$

$$\phi R_n = 297.1 \text{kips} > P_{\text{splice}} = 103.2 \text{kips} \quad \text{OKAY}$$

Bearing and Tear-out in Embedded Beam Flange:

$$P_{\text{flange.e}} = \frac{M_r}{(d - t_{fe})} = \frac{1225^{k-\text{in}}}{(11.25^{\text{in}} - 0.75^{\text{in}})} = 116.7 \text{kips}$$

Exterior Bolts:

$$R_n = 1.2 \times \left( 2.625^{\text{in}} - \left( 0.75^{\text{in}} + \frac{1}{16}^{\text{in}} \right) / 2 \right) \times 0.75^{\text{in}} \times 65^{\text{ksi}} = 129.8 \text{kips}$$

$$R_n = 2.4 \times 0.75^{\text{in}} \times 0.75^{\text{in}} \times 65^{\text{ksi}} = 87.75 \text{kips}$$

$$R_n = 87.75 \text{kips} \quad \text{Controls}$$
**Interior Bolts:**

\[ R_n = 1.2 \times \left(2.25^{\text{in}} - \left(0.75^{\text{in}} + \frac{1}{16^{\text{in}}} \right)\right) \times 0.75^{\text{in}} \times 65^{\text{ksi}} = 84.09 \text{ kips} \]

\[ R_n = 2.4 \times 0.75^{\text{in}} \times 0.75^{\text{in}} \times 65^{\text{ksi}} = 87.75 \text{ kips} \]

\[ R_n = 84.09 \text{ kips} \quad \text{Controls} \]

**Total Capacity:**

\[ \phi R_n = 0.75 \left(\left(4^{\text{bolts}} \times 87.75^{k/bolt}\right) + \left(4^{\text{bolts}} \times 84.09^{k/bolt}\right)\right) \]

\[ \phi R_n = 515.5 \text{ kips} > P_{\text{flange.e}} = 116.7 \text{ kips} \quad \text{OKAY} \]

**Bearing and Tear-out in Fuse Link Beam Flange:**

\[ R_n = \text{Min. of } \frac{1.5L_c R_t t F_u}{3.0d R_t t F_u} \quad (\text{AISC 360-10, J3-10a}) \]

\[ P_{\text{flange.f}} = \frac{M_t r}{d - t_{ff}} = \frac{1225^{k-in}}{(11.25^{in} - 0.625^{in})} = 115.3 \text{ kips} \]

AISC 341-10 Section A3.2 states that for an element that was computed using the expected capacity, the factor \( R_t \) may be used. Where \( R_t \) is the ratio of the expected tensile strength to the specified minimum tensile strength, \( F_u \), of that material (ANSI/AISC 341-10 2010).
Exterior Bolts:

\[ R_n = 1.5 \times \left( 2.750^{in} - \left( \frac{0.75^{in} + \frac{1}{16}^{in}}{2} \right) \right) \times 1.2 \times 0.625^{in} \times 65^{ksi} = 171.4 \text{ kips} \]

\[ R_n = 3 \times 0.75^{in} \times 1.2 \times 0.625^{in} \times 65^{ksi} = 109.7 \text{ kips} \]

\[ R_n = 109.7 \text{ kips} \quad \text{Controls} \]

Interior Bolts:

\[ R_n = 1.5 \times \left( 2.25^{in} - \left( \frac{0.75^{in} + \frac{1}{16}^{in}}{2} \right) \right) \times 1.2 \times 0.625^{in} \times 65^{ksi} = 105.1 \text{ kips} \]

\[ R_n = 3 \times 0.75^{in} \times 1.2 \times 0.625^{in} \times 65^{ksi} = 109.7 \text{ kips} \]

\[ R_n = 105.1 \text{ kips} \quad \text{Controls} \]

Total Capacity:

\[ \phi R_n = 0.75 \left( 4^{bolts} \times 109.7^{kip/bolt} \right) + \left( 4^{bolts} \times 105.1^{kip/bolt} \right) \]

\[ \phi R_n = 644.4 \text{ kips} > P_{flange,f} = 115.3 \text{ kips} \quad \text{OKAY} \]
Check Block Shear in Splice Plates:

Three scenarios of block shear were evaluated. They are demonstrated in the following calculations:

\[ \phi R_n = 0.75 \times \text{Min. of} \begin{cases} 0.6F_{uA_{nv}} + UF_{uA_{nt}} \\ 0.6F_{yA_{gv}} + UF_{uA_{nt}} \end{cases} \]  
(AISC 360-10, J4-3)

Case One:

![Figure D.4: Block shear case one](image)

\[ A_{gv} = 2 \times (1.00^{in} + 2.25^{in}) \times 0.625^{in} = 4.063 \text{ in}^2 \]

\[ A_{nv} = 4.063^{in^2} - 0.625^{in} \times \left( 3 \times (0.75^{in} + 1.25^{in}) \right) = 2.422 \text{ in}^2 \]

\[ A_{nt} = 0.625^{in} \times \left( (9^{in} - 1.00^{in} - 1.00^{in}) - (3 \times (0.75^{in} + 1.25^{in})) \right) = 2.734 \text{ in}^2 \]

\[ R_n = \left( (0.6 \times 65^{ksi} \times 2.422^{in^2}) + (1.00 \times 65^{ksi} \times 2.734^{in^2}) \right) = 272.2 \text{ kips} \quad \text{Controls} \]
\[
R_n = \left( (0.6 \times 50^{ksi} \times 4.063^{in^2}) + (1.00 \times 65^{ksi} \times 2.734^{in^2}) \right) = 299.6 \text{ kips}
\]

\[
\phi R_n = 0.75 \times 272.2^{kips} = 204.1 \text{ kips} > P_{splice} = 103.2 \text{ kips}
\]

**Case Two:**

![Block shear case two diagram](Image)

**Figure D.5:** Block shear case two

\[
A_{gv} = (1.00^{in} + 2.25^{in}) \times 0.625^{in} = 2.031 \text{ in}^2
\]

\[
A_{nv} = 2.031^{in^2} - 0.625^{in} \times \left( 1.5 \times (0.75^{in} + 1.25^{in}) \right) = 1.211 \text{ in}^2
\]

\[
A_{nt} = 0.625^{in} \times \left( (9^{in} - 1.00^{in}) - (3.5 \times (0.75^{in} + 1.25^{in})) \right) = 3.086 \text{ in}^2
\]

\[
R_n = \left( (0.6 \times 65^{ksi} \times 1.211^{in^2}) + (1.00 \times 65^{ksi} \times 3.086^{in^2}) \right) = 247.8 \text{ kips}
\]

Controls

\[
R_n = \left( (0.6 \times 50^{ksi} \times 2.031^{in^2}) + (1.00 \times 65^{ksi} \times 3.086^{in^2}) \right) = 261.5 \text{ kips}
\]

\[
\phi R_n = 0.75 \times 247.8^{kips} = 185.9 \text{ kips} > P_{splice} = 103.2 \text{ kips}
\]

OKAY
Case Three:

Figure D.6: Block shear case three

\[ A_{gv} = 2 \times (1.00^{in} + 2.25^{in}) \times 0.625^{in} = 4.063 \, \text{in}^2 \]

\[ A_{nv} = 4.063 \, \text{in}^2 - 0.625^{in} \times \left(3 \times (0.75^{in} + .125^{in})\right) = 2.422 \, \text{in}^2 \]

\[ A_{nt} = 0.625^{in} \times \left( (9^{in} - 3.00^{in}) - \left(3 \times (0.75^{in} + .125^{in})\right)\right) = 2.109 \, \text{in}^2 \]

\[ R_n = \left( (0.6 \times 65^{ksi} \times 2.422^{in^2}) + (1.00 \times 65^{ksi} \times 2.109^{in^2}) \right) = 231.6 \, \text{kips} \quad \text{Controls} \]

\[ R_n = \left( (0.6 \times 50^{ksi} \times 4.063^{in^2}) + (1.00 \times 65^{ksi} \times 2.109^{in^2}) \right) = 259.0 \, \text{kips} \]

\[ \phi R_n = 0.75 \times 231.6^{kips} = 173.7 \, \text{kips} > P_{splice} = 103.2 \, \text{kips} \quad \text{OKAY} \]
Welded Connections:

Welded connections were not designed in this investigation since Mitchell’s experiment proved that the weld design was adequate. For the design of welding connections, refer to the work by Mitchell or AISC 360-10, J2.