ABSTRACT

ROYSTER, PRESTON BLAZE. Localized Low Cycle Fatigue Failure of Welded Steel Moment Connections. (Under the Direction of Dr. Tasnim Hassan and Dr. Emmett Sumner.)

Welded steel moment frames were once thought to be one of the most ductile systems for resisting seismic loading. Welded steel moment connections (WSMCs) consisting of complete joint penetration welds between the beam flanges and the column flange along with a bolted shear tab connecting the beam web to the column flange gave the frame its resistance to lateral loads. However, the unexpected brittle failures initiated by cracks near or at the welds between the bottom beam flange and the column flange during the Northridge (1994) and Kobe (1995) Earthquakes challenged this belief. Over the last decade, researchers have improved the global performance of the connection by improving the weld access hole and weld details. However, cracking near the weld is still observed in the experiments of these “improved” connections. Through all of these efforts to comprehend the behavior of the welded steel moment connection, the local behavior of the joint leading to the global failures that have been observed is not fully understood.

The primary goal of this research is to observe and understand the local behavior of the welded steel moment connection through analysis and a set of material and structural experiments. Small coupons of beam flanges, developed according to the ASTM specifications, heat treated to different temperatures were tested using monotonic, displacement-controlled, and force-controlled loading to obtain the properties of the steel at different distances from the weld (within the heat affected zone). Additional experiments were conducted on base and weld metal coupons. Large scale structural experiments were conducted on the bottom beam flange to column flange connection, which consisted of a short section of the bottom beam flange welded to the column flange using a complete joint penetration weld. These simpler structural specimens
were subjected to uniaxial load reversals along the direction of the beam. This configuration facilitated the observation of the local failure mechanisms in a large scale global specimen.

Finite element simulations of the full-scale welded steel moment connections and the simplified bottom beam flange welded joint were conducted using the ANSYS software package with its multilinear material model. These pretest analyses were utilized to develop the loading protocol for the experimental program.

Through experimentation of the heat treated coupons this research observed changes in material behavior associated with exposure temperature from welding. Such heterogeneous changes in material behavior in the heat affected zone may influence the formation of cracks near the weld. Brittle failures of the bottom beam flange tee joints were observed under constant amplitude displacement controlled load reversals. Recorded data from the strain gages located near the complete joint penetration welds demonstrated the presence of ratcheting (defined by accumulation of strains with cycles). This strain ratcheting response may also influence the formation of cracks near the weld and thus may also be responsible for the brittle failure of WSMCs.

In conclusion, this research investigates the local failure of the welded steel moment connection in order to understand the global brittle failure mechanisms. The change in material properties and the presence of ratcheting at the WSMCs influences its local crack initiated brittle failures. However, further research is needed to more completely understand the local behavior of the welded steel moment connections.
Localized Low Cycle Fatigue Failure of Welded Steel Moment Connections

by

Preston Blaze Royster

A thesis submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the Degree of Master of Science Civil Engineering Raleigh, NC November 5, 2007

APPROVED BY:

________________________
Dr. Tasnim Hassan (Chair)

________________________
Dr. Emmett Sumner (Co-Chair)

________________________
Dr. James Nau
DEDICATION

to The Roysters....
BIOGRAPHY

Preston Blaze Royster was born on April 13th, 1981 in Oxford, NC to his parents, Richard Royster and Deborah Royster. He grew up in Oxford, NC and attended elementary and middle schools in the Granville County school system. Preston went to J.F. Webb High School, where he graduated with the class of 1999.

Upon completion of high school, Preston attended North Carolina State University from 1999 – 2004. During his time at NCSU, he co-oped with Granite Construction Company and worked on the construction of I-85 through Durham. He graduated in 2004 with a degree in Civil Engineering. Preston graduated with a perfect 4.0 and was the valedictorian of the undergraduate class.

Prior to returning to school to pursue his Master’s Degree, Preston went to work full time with Granite Construction Company as a foreman. He worked with Granite for 7 months before choosing to return to school to pursue his Master’s Degree, which he will complete in December of 2007.
ACKNOWLEDGEMENTS

I would like to thank the members of my committee, Dr. Tasnim Hassan, Dr. Emmett Sumner, and Dr. James Nau, for their support and guidance through this project. I would also like to thank the Civil Engineering Department at NC State University for funding. In addition, SteelFab deserves acknowledgement for donating the steel specimens, Buckner Steel Erection for welding services, and the National Science Foundation for funding this research project. This would not have been possible without the assistance of Jerry Atkinson in day to day laboratory activities and Bill Dunleavy for computer and electronics assistance.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIST OF TABLES</td>
<td>viii</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>ix</td>
</tr>
<tr>
<td>1 INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 BACKGROUND</td>
<td>1</td>
</tr>
<tr>
<td>1.2 LESSONS LEARNED FROM PAST EARTHQUAKES</td>
<td>3</td>
</tr>
<tr>
<td>1.3 DESIGN MODIFICATIONS FOR IMPROVED DUCTILITY</td>
<td>7</td>
</tr>
<tr>
<td>1.4 LOW CYCLE FATIGUE</td>
<td>12</td>
</tr>
<tr>
<td>1.5 MOTIVATION</td>
<td>13</td>
</tr>
<tr>
<td>1.6 TEST SPECIMENS AND EXPERIMENTS</td>
<td>20</td>
</tr>
<tr>
<td>2 ANALYTICAL INVESTIGATION</td>
<td>22</td>
</tr>
<tr>
<td>2.1 FULL-SCALE WELDED STEEL MOMENT CONNECTION MODEL</td>
<td>22</td>
</tr>
<tr>
<td>2.1.1 MODELING</td>
<td>22</td>
</tr>
<tr>
<td>2.1.2 RESULTS</td>
<td>36</td>
</tr>
<tr>
<td>2.2 BOTTOM FLANGE TEE JOINT SPECIMENS</td>
<td>40</td>
</tr>
<tr>
<td>2.2.1 MODELING</td>
<td>42</td>
</tr>
<tr>
<td>2.2.2 RESULTS</td>
<td>44</td>
</tr>
<tr>
<td>2.3 CONCLUSIONS</td>
<td>50</td>
</tr>
<tr>
<td>3 EXPERIMENTAL INVESTIGATION</td>
<td>51</td>
</tr>
<tr>
<td>3.1 INTRODUCTION</td>
<td>51</td>
</tr>
<tr>
<td>3.2 FULL-SCALE EXTERIOR WELDED STEEL MOMENT CONNECTIONS</td>
<td>52</td>
</tr>
<tr>
<td>3.2.1 BASIC SETUP</td>
<td>53</td>
</tr>
</tbody>
</table>
5.3 CONCLUSIONS........................................................................................................ 133
REFERENCES .............................................................................................................. 136
APPENDIX.................................................................................................................. 140
LIST OF TABLES

Table 1.1 Summary of Castiglioni [2005] Test Results ...........................................17
LIST OF FIGURES

Fig. 1.1 Beam-Column Connection Fractures during 1994 Northridge Earthquake [FEMA 350] .................................................................3

Fig. 1.2 Plastic Hinge Development in Steel Beams .................................4

Fig. 1.3 Common Zone of Fracture Initiation in Beam-Column Connections ....5

Fig. 1.4 Sketch of a Typical Welded Steel Moment Connection .................7

Fig. 1.5 Sketch of Pre-Northridge Connection .....................................9

Fig. 1.6 Sketch of Post-Northridge Connection ....................................9

Fig. 1.7 Radius Cut Reduced Beam Section (RBS) Moment Connection .......12

Fig. 1.8 Finite Element Results by Castiglioni [2005] ............................17

Fig. 1.9 Welded Steel Moment Connection .......................................19

Fig. 1.10 Section A-A and Welding Sequence 1 .................................19

Fig. 1.11 Section A-A and Welding Sequence 2 .................................20

Fig. 2.1 Idealized Welded Steel Moment Connection Test Setup ...............23

Fig. 2.2 Generic Stress-Strain Curve for A992 Steel (Ricles, et. al [2000]) ....25

Fig. 2.3 Generic Stress-Strain Curve for E70 Weld Metal (Ricles, et. al [2000]) 26

Fig. 2.4 Mesh 1: Coarse Finite Element Mesh ..................................27

Fig. 2.5 Mesh 4: Fine Finite Element Mesh .....................................28

Fig. 2.6 Close-up View of Mesh 4 at the Beam to Column Connection ........29

Fig. 2.7 von Mises Stress in the Top Beam Flange in the Base Metal (25 kips Load) ........31

Fig. 2.8 Axial Strain in the Top Beam Flange in the Base Metal (25 kips Load) ........32

Fig. 2.9 von Mises Stress in the Top Beam Flange in the Base Metal (35 kips Load) ....33

Fig. 2.10 Axial Strain in the Top Beam Flange in the Base Metal (35 kips Load) ........34

Fig. 2.11 SAC Loading Protocol with Increasing Amplitudes ......................35
Fig. 2.12 Multilinear Material Model for the Monotonic Uniaxial Stress-Strain Curve
..................................................................................................................36

Fig. 2.13 Axial Strain Distribution Across the Width of the Flange .................37

Fig. 2.14 von Mises Stress (ksi) Distribution Near the Beam-Column Connection
(Max. Stress = 71.345 ksi) ..................................................................................38

Fig. 2.15 von Mises Stress (ksi) Concentrations Near the Bottom Beam Flange Edges
and the Weld Access Hole ..............................................................................39

Fig. 2.16 von Mises Stress (ksi) Concentration in the Center of the Bottom Beam Flange
..........................................................................................................................40

Fig. 2.17 Sketch of the Bottom Beam Flange Tee Joint Specimen and the Loading Direction
.........................................................................................................................41

Fig. 2.18 Finite Element Mesh for the Bottom Beam Flange Tee Joint Specimen ....44

Fig. 2.19 Full-Scale Welded Steel Moment Connection and Bottom Beam Flange Tee Joint Axial Strain Simulation Comparison ........................................45

Fig. 2.20 von Mises Stress (ksi) Contour Plot for the Bottom Beam Flange Tee Joint
(Max. Stress = 74.07 ksi) ..................................................................................47

Fig. 2.21 von Mises Stress (ksi) Concentrations in the Bottom Beam Flange Edges
..........................................................................................................................48

Fig. 2.22 von Mises Stress (ksi) Concentrations in the Center of the Bottom Flange
..........................................................................................................................49

Fig. 3.1 Interior and Exterior Connections in Moment Frames .........................52

Fig. 3.2 Sketch of the Full-Scale Exterior Welded Steel Moment Connection to be Tested .................................................................54

Fig. 3.3 Full-Scale Exterior Welded Steel Moment Connection Specimen and Test Setup .......................................................................................55

Fig. 3.4 SAC Loading Protocol ...........................................................................57

Fig. 3.5 Top and Bottom Flange Welding Passes .............................................58

Fig. 3.6 Origin of the Bottom Flange Tee Joint Specimens .............................59
Fig. 4.4 Stress Amplitudes for the Displacement-Controlled Loading .....................91
Fig. 4.5 Mean Strain Values for the Force-Controlled Coupons ...............................93
Fig. 4.6 Monotonic Stress-Strain Response of the Extensometer for Specimen C-M-W .................................................................94
Fig. 4.7 Monotonic Stress-Strain Response of the Strain Gages for Specimen C-M-W .................................................................95
Fig. 4.8 Side View of the Welded Coupon’s Welded Connection ..............................96
Fig. 4.9 Strain Means and Amplitudes (from Average Strain Gage Values) for C-W-FC .................................................................98
Fig. 4.10 Stress-Strain (Extensometer Strain) for C-W-FC ....................................99
Fig. 4.11 Stress-Strain (Strain Gage) Response for C-W-DC .................................100
Fig. 4.12 Displacement History for T-MA-B-4A/B .............................................104
Fig. 4.13 Load Versus Displacement Relationship for T-MA-B-4A/B ..................105
Fig. 4.14 Local Buckling of the Flange in Specimen T-MA-B-4A/B .....................106
Fig. 4.15 Fractured Beam in Specimen T-MA-B-4A/B ......................................106
Fig. 4.16 Displacement History of Specimen T-CA-B-1A/B ...............................108
Fig. 4.17 Cracks Near the Complete Joint Penetration Weld in the Beam Flange in Specimen T-CA-B-1A/B .........................................................109
Fig. 4.18 Close Up View of the Crack at the Edge of the Beam Flange ..............109
Fig. 4.19 Load Versus Displacement Relationship for T-CA-B-1A/B ..................111
Fig. 4.20 Fractured Beam in Specimen T-CA-B-1A/B ......................................111
Fig. 4.21 Load Versus Displacement Relationship for Phase 3 for Specimen T-CA-B-1A/B .................................................................112
Fig. 4.22 Displacement History of Specimen T-CA-B-2A/B ...............................114
Fig. 4.23 Load Versus Displacement Relationship for Specimen T-CA-B-2A/B .......115
Fig. 4.24 Load Verses Displacement Relationship for Phase 5 for Specimen T-CA-B-2A/B .................................................................116
Fig. 4.25 Failure of Specimen T-CA-B-2A/B .........................................................116
Fig. 4.26 Displacement History for T-MA-B-3A/B ..................................................118
Fig. 4.27 Specimen T-MA-B-3A/B Flange Buckling .............................................119
Fig. 4.28 Load Verses Displacement Relationship for T-MA-B-3A/B .....................119
Fig. 4.29 Strain Means and Amplitudes for T-CA-B-1A SG-R1-O-4 .....................123
Fig. 4.30 Strain Means and Amplitudes for T-CA-B-1B SG-R1-O-4 ......................123
Fig. 4.31 Strain Means and Amplitudes for T-CA-B-1B SG-R1-I-1 .......................124
Fig. 4.32 Strain Means and Amplitudes for T-CA-B-2A SG-R1-I-5 .......................125
Fig. 4.33 Strain Means and Amplitudes for T-CA-B-2A SG-R1-I-7 .......................125
Fig. 4.34 Strain Means and Amplitudes for T-MA-B-3A SG-R1-O-4 ....................127
Fig. 4.35 Strain Means and Amplitudes for T-MA-B-3A SG-R1-O-1 ....................127
Fig. A.1 Displacement History for T-CA-B-1A/B ................................................142
Fig. A.2 Load-Displacement Response for T-CA-B-1A/B .................................143
Fig. A.3 Phase 1 Load-Displacement Response for T-CA-B-1A/B .......................143
Fig. A.4 Phase 2 Load-Displacement Response for T-CA-B-1A/B .......................144
Fig. A.5 Phase 3 Load-Displacement Response for T-CA-B-1A/B .......................144
Fig. A.6 Phase 4 Load-Displacement Response for T-CA-B-1A/B .......................145
Fig. A.7 Phase 5 Load-Displacement Response for T-CA-B-1A/B .......................145
Fig. A.8 Phase 6 Load-Displacement Response for T-CA-B-1A/B .......................146
Fig. A.9 Phase 7 Load-Displacement Response for T-CA-B-1A/B .......................146
Fig. A.10 Phase 8 Load-Displacement Response for T-CA-B-1A/B ......................147
Fig. A.11 Displacement Amplitudes and Means for T-CA-B-1A/B ......................147
Fig. A.12 Load Amplitudes and Means for T-CA-B-1A/B .................................. 148
Fig. A.13 Strain Amplitudes and Means for T-CA-B-1A SG-R1-O-7 ...................... 148
Fig. A.14 Strain Amplitudes and Means for T-CA-B-1A SG-R3-O-4 .................... 149
Fig. A.15 Strain Amplitudes and Means for T-CA-B-1A SG-R1-I-3 .................... 149
Fig. A.16 Strain Amplitudes and Means for T-CA-B-1A SG-R1-O-4 .................... 150
Fig. A.17 Strain Amplitudes and Means for T-CA-B-1A SG-R2-O-4 .................... 150
Fig. A.18 Strain Amplitudes and Means for T-CA-B-1A SG-R1-O-1 .................... 151
Fig. A.19 Strain Amplitudes and Means for T-CA-B-1B SG-R1-O-6 .................... 151
Fig. A.20 Strain Amplitudes and Means for T-CA-B-1B SG-R1-O-7 .................... 152
Fig. A.21 Strain Amplitudes and Means for T-CA-B-1B SG-R1-O-4 .................... 152
Fig. A.22 Strain Amplitudes and Means for T-CA-B-1B SG-R2-O-4 .................... 153
Fig. A.23 Strain Amplitudes and Means for T-CA-B-1B SG-R1-I-1 .................... 153
Fig. A.24 Strain Amplitudes and Means for T-CA-B-1B SG-R3-O-4 .................... 154
Fig. A.25 Strain Amplitudes and Means for T-CA-B-1B SG-R1-I-5 .................... 154
Fig. A.26 Average Strain Amplitudes and Means for T-CA-B-1A SG-R1-O-1 and
SG-R1-I-1 ........................................................................................................ 155
Fig. A.27 Displacement History for T-CA-B-2A/B ............................................. 157
Fig. A.28 Load-Displacement Response for T-CA-B-2A/B ................................. 158
Fig. A.29 Phase 1 Load-Displacement Response for T-CA-B-2A/B ..................... 158
Fig. A.30 Phase 2 Load-Displacement Response for T-CA-B-2A/B ..................... 159
Fig. A.31 Phase 3 Load-Displacement Response for T-CA-B-2A/B ..................... 159
Fig. A.32 Phase 4 Load-Displacement Response for T-CA-B-2A/B ..................... 160
Fig. A.33 Phase 5 Load-Displacement Response for T-CA-B-2A/B ..................... 160
Fig. A.34 Phase 6 Load-Displacement Response for T-CA-B-2A/B ......................161
Fig. A.35 Phase 7 Load-Displacement Response for T-CA-B-2A/B ......................161
Fig. A.36 Phase 8 Load-Displacement Response for T-CA-B-2A/B ......................162
Fig. A.37 Phase 9 Load-Displacement Response for T-CA-B-2A/B ......................162
Fig. A.38 Phase 10 Load-Displacement Response for T-CA-B-2A/B ....................163
Fig. A.39 Phase 11 Load-Displacement Response for T-CA-B-2A/B ....................163
Fig. A.40 Phase 12 Load-Displacement Response for T-CA-B-2A/B ....................164
Fig. A.41 Phase 13 Load-Displacement Response for T-CA-B-2A/B ....................164
Fig. A.42 Phase 14 Load-Displacement Response for T-CA-B-2A/B ....................165
Fig. A.43 Displacement Amplitudes and Means for T-CA-B-2A/B .....................165
Fig. A.44 Load Amplitudes and Means for T-CA-B-2A/B ..............................166
Fig. A.45 Strain Amplitudes and Means for T-CA-B-2B SG-R3-O-4 ....................166
Fig. A.46 Strain Amplitudes and Means for T-CA-B-2B SG-R1-I-1 .....................167
Fig. A.47 Strain Amplitudes and Means for T-CA-B-2B SG-R1-I-7 .....................167
Fig. A.48 Strain Amplitudes and Means for T-CA-B-2B SG-R1-I-5 .....................168
Fig. A.49 Strain Amplitudes and Means for T-CA-B-2B SG-R1-O-7 ....................168
Fig. A.50 Strain Amplitudes and Means for T-CA-B-2A SG-R1-I-5 .....................169
Fig. A.51 Strain Amplitudes and Means for T-CA-B-2A SG-R1-I-7 .....................169
Fig. A.52 Strain Amplitudes and Means for T-CA-B-2B SG-R1-O-2 ....................170
Fig. A.53 Strain Amplitudes and Means for T-CA-B-2B SG-R1-O-4 ....................170
Fig. A.54 Strain Amplitudes and Means for T-CA-B-2B SG-R1-O-1 ....................171
Fig. A.55 Strain Amplitudes and Means for T-CA-B-2B SG-R2-O-4 ....................171
Fig. A.56 Strain Amplitudes and Means for T-CA-B-2A SG-R3-O-4 ....................172
Fig. A.57 Strain Amplitudes and Means for T-CA-B-2A SG-R1-O-7 ..................172
Fig. A.58 Strain Amplitudes and Means for T-CA-B-2A SG-R1-O-4 ..................173
Fig. A.59 Strain Amplitudes and Means for T-CA-B-2A SG-R1-O-1 ..................173
Fig. A.60 Strain Amplitudes and Means for T-CA-B-2A SG-R1-O-2 ..................174
Fig. A.61 Strain Amplitudes and Means for T-CA-B-2A SG-R1-I-1 ..................174
Fig. A.62 Strain Amplitudes and Means for T-CA-B-2A SG-R2-O-4 ..................175
Fig. A.63 Displacement History for T-MA-B-3A/B ..................................177
Fig. A.64 Load-Displacement Response for T-MA-B-3A/B ..........................178
Fig. A.65 Phase 1 Load-Displacement Response for T-MA-B-3A/B .................178
Fig. A.66 Phase 2 Load-Displacement Response for T-MA-B-3A/B .................179
Fig. A.67 Phase 3 Load-Displacement Response for T-MA-B-3A/B .................179
Fig. A.68 Phase 4 Load-Displacement Response for T-MA-B-3A/B .................180
Fig. A.69 Phase 5 Load-Displacement Response for T-MA-B-3A/B .................180
Fig. A.70 Phase 6 Load-Displacement Response for T-MA-B-3A/B .................181
Fig. A.71 Phase 7 Load-Displacement Response for T-MA-B-3A/B .................181
Fig. A.72 Phase 8 Load-Displacement Response for T-MA-B-3A/B .................182
Fig. A.73 Phase 9 Load-Displacement Response for T-MA-B-3A/B .................182
Fig. A.74 Displacement Amplitudes and Means for T-MA-B-3A/B ..................183
Fig. A.75 Load Amplitudes and Means for T-MA-B-3A/B ..........................183
Fig. A.76 Strain Amplitudes and Means for T-MA-B-3A SG-R3-O-4 .................184
Fig. A.77 Strain Amplitudes and Means for T-MA-B-3B SG-R3-O-4 .................184
Fig. A.78 Strain Amplitudes and Means for T-MA-B-3B SG-R2-O-4 .................185
Fig. A.79 Strain Amplitudes andMeans for T-MA-B-3B SG-R1-I-1 .................185
<table>
<thead>
<tr>
<th>Figure Number</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>A.80</td>
<td>Strain Amplitudes and Means for T-MA-B-3A SG-R1-O-4</td>
<td>186</td>
</tr>
<tr>
<td>A.81</td>
<td>Strain Amplitudes and Means for T-MA-B-3B SG-R1-I-7</td>
<td>186</td>
</tr>
<tr>
<td>A.82</td>
<td>Strain Amplitudes and Means for T-MA-B-3B SG-R1-O-7</td>
<td>187</td>
</tr>
<tr>
<td>A.83</td>
<td>Strain Amplitudes and Means for T-MA-B-3B SG-R1-I-5</td>
<td>187</td>
</tr>
<tr>
<td>A.84</td>
<td>Strain Amplitudes and Means for T-MA-B-3A SG-R1-O-6</td>
<td>188</td>
</tr>
<tr>
<td>A.85</td>
<td>Strain Amplitudes and Means for T-MA-B-3B SG-R1-O-6</td>
<td>188</td>
</tr>
<tr>
<td>A.86</td>
<td>Strain Amplitudes and Means for T-MA-B-3A SG-R1-O-4</td>
<td>189</td>
</tr>
<tr>
<td>A.87</td>
<td>Strain Amplitudes and Means for T-MA-B-3A SG-R1-I-5</td>
<td>189</td>
</tr>
<tr>
<td>A.88</td>
<td>Strain Amplitudes and Means for T-MA-B-3A SG-R2-O-4</td>
<td>190</td>
</tr>
<tr>
<td>A.89</td>
<td>Strain Amplitudes and Means for T-MA-B-3A SG-R1-O-7</td>
<td>190</td>
</tr>
<tr>
<td>A.90</td>
<td>Strain Amplitudes and Means for T-MA-B-3A SG-R1-I-7</td>
<td>191</td>
</tr>
<tr>
<td>A.91</td>
<td>Strain Amplitudes and Means for T-MA-B-3A SG-R1-O-1</td>
<td>191</td>
</tr>
<tr>
<td>A.92</td>
<td>Strain Amplitudes and Means for T-MA-B-3A SG-R1-I-1</td>
<td>192</td>
</tr>
<tr>
<td>A.93</td>
<td>Displacement History for T-MA-B-4A/B</td>
<td>194</td>
</tr>
<tr>
<td>A.94</td>
<td>Load-Displacement Response for T-MA-B-4A/B</td>
<td>195</td>
</tr>
<tr>
<td>A.95</td>
<td>Phase 1 Load-Displacement Response for T-MA-B-4A/B</td>
<td>195</td>
</tr>
<tr>
<td>A.96</td>
<td>Phase 2 Load-Displacement Response for T-MA-B-4A/B</td>
<td>196</td>
</tr>
<tr>
<td>A.97</td>
<td>Phase 3 Load-Displacement Response for T-MA-B-4A/B</td>
<td>196</td>
</tr>
<tr>
<td>A.98</td>
<td>Phase 4 Load-Displacement Response for T-MA-B-4A/B</td>
<td>197</td>
</tr>
<tr>
<td>A.99</td>
<td>Phase 5 Load-Displacement Response for T-MA-B-4A/B</td>
<td>197</td>
</tr>
<tr>
<td>A.100</td>
<td>Phase 6 Load-Displacement Response for T-MA-B-4A/B</td>
<td>198</td>
</tr>
<tr>
<td>A.101</td>
<td>Phase 7 Load-Displacement Response for T-MA-B-4A/B</td>
<td>198</td>
</tr>
<tr>
<td>A.102</td>
<td>Phase 8 Load-Displacement Response for T-MA-B-4A/B</td>
<td>199</td>
</tr>
</tbody>
</table>
Fig. A.103 Phase 9 Load-Displacement Response for T-MA-B-4A/B .................199
Fig. A.104 Phase 10 Load-Displacement Response for T-MA-B-4A/B ..............200
Fig. A.105 Phase 11 Load-Displacement Response for T-MA-B-4A/B ..........200
Fig. A.106 Phase 12 Load-Displacement Response for T-MA-B-4A/B .........201
Fig. A.107 Phase 13 Load-Displacement Response for T-MA-B-4A/B ..........201
Fig. A.108 Phase 14 Load-Displacement Response for T-MA-B-4A/B ........202
Fig. A.109 Phase 15 Load-Displacement Response for T-MA-B-4A/B ........202
Fig. A.110 Displacement Amplitudes and Means for T-MA-B-4A/B ...........203
Fig. A.111 Load Amplitudes and Means for T-MA-B-4A/B .......................203
1.1 Background

Major seismic events have devastated the world since the beginning of time. Since 1900 there have been many major Earthquakes in the US alone: San Francisco in 1906, Long Beach in 1929, Imperial Valley in 1940 and 1979, San Fernando in 1971, Loma Prieta in 1989, and Northridge in 1994. Earthquakes are among the most costly and deadly natural disasters.

In the 1960s structural engineers regarded the welded steel moment frame as the most ductile system in the building code [FEMA 350]. Steel structures are considered ductile if they can sustain large inelastic deformations without significant degradation in strength and development of instability. Many engineers believed that if welded steel moment frames were exposed to seismic loading, the damage would be limited to ductile yielding of members or connections. For this reason, many large commercial, industrial, and institutional buildings were constructed using welded steel moment frames in the western United States [FEMA 350].

The 1988 Uniform Building Code (UBC) adopted a prescriptive moment connection design for seismic applications based on west coast building practices at the time. The qualification for this connection came from research performed by Popov and Stephen [FEMA 350]. This connection was subsequently adopted by the 1992 AISC Seismic Provisions and other building codes. The prequalified connection was supposed to result in ductile, plastic behavior due to the formation of a plastic hinge in the beam at
the face of the column or in the column panel zone or a combination of the two. The Northridge Earthquake in 1994 challenged this connection and demonstrated its inability to meet the seismic demands. It was observed that when the hinge developed in the panel zone, the resulting column deformation resulted in large secondary stresses on the beam flange to column flange joint which led to brittle failure of the flange connection [FEMA 350]. When the plastic hinge formed in the beam at the face of the column, large inelastic strains occurred in and adjacent to the weld metal and the surrounding heat affected zone, and led to brittle failure [FEMA 350].

On January 17, 1994 at approximately 4:30 a.m., the Northridge Earthquake struck southern California. Its epicenter was located west-northwest of Los Angeles and south-southwest of Northridge. The magnitude 6.7 quake shook the ground for only 15 seconds. Left behind were 9,000+ injured people and 51 dead along with a $20-40 billion loss of property [“Northridge Earthquake” 2005]. Seven thousand buildings were severely damaged and deemed unsafe to occupy, and 22,000 buildings were moderately damaged and authorized for “limited use.” Amazingly, no steel structures, with the exception of the scoreboard assembly at Anaheim Stadium, failed. However, welded steel moment frames (WSMF) suffered huge, unexpected cracks at the welded steel moment connections (WSMC). The damaged buildings ranged from one to 26 stories tall and as old as 30 years up to buildings under construction. These brittle failures were observed in the weld between the bottom beam flange and the column flange as shown in Fig. 1.1(a). In some cases, the cracks continued through the column flange right above the weld as shown in Fig. 1.1(b). Amazingly, the same types of brittle connection failures were observed in WSMF following the Kobe Earthquake exactly one year after
the Northridge Earthquake. Many observations on failure mechanisms were learned, and many were addressed through research over the last decade as presented herein.

![Image of beam-column connection fractures]

**Fig. 1.1 – Beam-column connection fractures during 1994 Northridge earthquake [FEMA 350]**

### 1.2 Lessons Learned from Past Earthquakes

Welded steel moment frames get their ductility from inelastic behavior of the beam-to-column connections. The inelastic response comes from plastic hinging in the beam (Fig. 1.2), plastic hinging in the column (less desirable), plastic shear deformation in the column panel zone, or a combination of the three. Contrary to popular belief, the brittle fractures initiated in the connections at low levels of plastic demand, and in some cases, the structure remained in the elastic range of motion during the Northridge and Kobe Earthquakes [FEMA 350]. Under constant, small amplitude tests where the beam doesn’t develop its full plastic moment capacity, brittle fractures were observed near the weld metal and heat affected zone [Castiglioni 2005]. Brittle fractures were also observed in connections with strong panel zones prior to the development of the plastic...
moment in the beam [Han, et al. 2006]. It was thought that the pre-Northridge WSMF connections would be capable of sustaining large plastic rotations without significant strength degradation. The fractures observed typically initiated at the complete joint penetration (CJP) weld between the bottom beam flange and the column flange (Fig. 1.3). The cracks propagated along different paths depending on individual joint configuration [FEMA 350]. According to FEMA and the SAC Joint Venture, the typical WSMF beam-to-column connection used prior to the Northridge Earthquake had a number of features that rendered it susceptible to brittle failure.

![Diagram of plastic hinge development in steel beams](image)

**Fig. 1.2 – Plastic hinge development in steel beams**
Early investigations of the Northridge Earthquake connection failures identified a number of fractures that appeared to be due to a lack of through thickness strength of the column flange material [FEMA 350]. Through thickness strength is critical in large column sections with thick flanges (greater than 1"). After rolling, the thickness of the flange affects the rate of cooling and the strength as the center cools much slower than the outside faces. Immediately following the earthquake it was suggested to limit the stress demand on the through thickness direction in the column flange to 40 ksi, thus ensuring that through-thickness yielding did not occur. After more extensive investigation, it was identified that most fractures initiated in defects in the complete joint penetration weld root instead of a result of low through thickness strength in the flange. Base metal toughness is important in the prevention of brittle fracture in the base metal in the highly stressed areas of the connection. The k-area in wide flanged rolled sections, the intersection of the flange and the web, has low toughness and may therefore be prone to cracking as a result of welding operations. Thus, it was recommended that fabricators
exercise extreme care when welding in, near, or to the k-area. Preheat temperatures, pre-welding cleaning, and welding passes should be executed with precision in order to ensure that the weld is high quality. It has also been suggested that weld backing bars and runoff tabs should be removed following the completion of the complete joint penetration welds, and a reinforcing fillet weld should be placed on the bottom of the beam flange in place of the backing. [FEMA 350]

The lack of ductility displayed in the pre-Northridge WSMC prompted research by the Federal Emergency Management Agency (FEMA) as well as the SAC Joint Venture, a partnership between the Structural Engineers Association of California (SEAOc), the Applied Technology Council (ATC), and the California Universities for Research in Earthquake Engineering (CUREe). The SAC Joint Venture focused research efforts on many different areas of the welded moment connection. Weld metal matching, base metal and weld metal toughness, backing bar effects, fillet weld reinforcement, residual stress effects, continuity plates, column flange thickness, panel zone deformations, reduced beam section details, weld access hole details, and column flange through-thickness fractures have all been investigated. Solutions to overcome the lack of ductility displayed by the connection are discussed in the following.

1.3 Design Modifications for Improved Ductility

To discuss the different failures observed during the Northridge earthquake and the SAC research, different regions of a typical welded steel moment connection are defined in Fig. 1.4. Region 1 is the top and bottom flange complete joint penetration groove welds and weld access holes. Region 2 is the shear tab that is welded to the
column flange and bolted to the beam web. Region 3 is the continuity plates that are parallel to the beam flanges and welded to the column flanges and the column web. Region 3 is also the doubler plate which lies in the plane of the column web in the column panel zone, the region between the continuity plates. It is welded in place with fillet welds on the top and bottom and complete joint penetration welds on the sides.

Fig. 1.4 – Sketch of a typical welded steel moment connection

Many of the cracks in pre- and post-Northridge connections can be traced back to stress concentrations in the beam-to-column joint [Barsom, et al. 1999]. Stress concentrations in a welded steel moment connection may be caused by the geometry of the joint or by imperfections or crack-like discontinuities in the weld metal, the base metal, or the heat affected zone.
First, the backing bar in the pre-Northridge connection creates a severe stress concentration because the unwelded portion between the backing bar and the column flange acts as a pre-existing crack (region 1 in Fig. 1.5). This pre-existing crack promotes earlier crack initiation and propagation [Nakashima, et al. 1998]. In the post-Northridge connection, the backing bar has been removed and a reinforcing fillet weld has been placed, as shown in Fig. 1.6, to eliminate these cracks. This reduces the stress concentration in region 1 in Fig. 1.5 [Barsom, et al. 1999]. This post-Northridge modification has also been shown to increase the rotational capacity of the connection [Barsom, et al. 1999]. Finite element analysis of welded joints performed by Chi, et al. (1997) also has shown that the backing bar does create some notch effect. However, it is far more significant when the backing is left in place since it obscures the effective detection of significant flaws that may be present in the weld root. These flaws represent a far more severe notch effect than the backing bar itself. Thus, it is recommended that the backing bar be removed from the beam bottom flange weld joints to allow identification and correction of weld root flaws [FEMA 350].
Regions 2 and 3 in Figs. 1.5 and 1.6 are areas where weld imperfections and weld geometry can cause stress concentrations that may initiate and propagate fatigue cracks. The majority of fatigue cracks in welded members initiate at a weld toe or weld termination (weld start and stop) where stress concentrations are high and weld discontinuities may be present [Barsom, et al. 1999]. Regions 4 and 5 in Figs. 1.5 and 1.6 are fatigue crack initiation sites corresponding to the weld access hole. Region 4 is the intersection of the beam web and flange at the weld access hole, while region 5 refers to the roughness of the flame cut weld access hole surface. The severity of the beam-to-web intersection increases as the angle, $\Theta$, increases and as the radius, $r$, decreases. A rougher flame cut surface can also lead to higher stress concentrations. Tests conducted by Nakashima, et al. [1998] showed that cracks initiated and propagated from the toe of
the conventional weld access hole. The design configuration and geometry of welded beam-to-column joints also produces high triaxial stresses at the midlength of the complete joint penetration weld. The welding procedure results in a lack of fusion along the weld root, and large defects at the midlength of the weld between the beam bottom flange and the column flange, where the stresses are the most severe. [Barsom, et al. 1999]

In order to address the stress concentrations induced by weld access hole geometry, Ricles, et al. (2000) conducted finite element analysis on nine different configurations. The variables investigated included the length of the hole, the length of the flat portion of the hole, slope of the access hole, and the radius of the access hole. From their studies the geometry of the weld access hole was optimized to reduce stress concentrations. Subsequently, they tested several full scale welded steel moment connections to failure using the modified weld access hole geometry. No failures occurred at the modified weld access hole [Ricles, et al. 2000].

Welded steel moment connection fractures from the Northridge earthquake were also caused by low toughness of the weld filler metal used in making the flange welds [Ricles, et al. 2000]. This problem has been solved by overmatching the weld metal to the base metal, or making the weld metal stronger than the base metal. Overmatching prevents weld root flaws at the weld-to-column interface, however it does not offer any benefit at the weld-to-beam flange interface where yielding concentrates in the heat affected zone [Dong, et al. 1999]. The heat affected zone is the portion of base metal that is subjected to high temperatures during welding, high enough to produce microstructural changes without melting. The properties of the heat affected zone depend on the base
metal, the number of weld passes, and the welding conditions employed, thus the mechanical properties of the heat affected zone can vary considerably from point to point. The influence of the base metal material property changes induced by welding has not drawn much attention yet.

One proposed method to mitigate fatigue cracks in welded steel moment connections is the radius cut reduced beam section (RBS) moment connection [FEMA 350]. Fig. 1.7 depicts a typical RBS moment connection. The goal of the RBS is to force the plastic hinge to form in the reduced beam section away from the column face, thus reducing high stress and strain demands on the critical beam flange welds while still dissipating energy [Deierlein, et al. 1999]. Test specimens incorporating the RBS have exhibited good overall performance with no fractures occurring at story drift angles less than 0.04 radians. The primary benefit of RBS cuts is to significantly reduce inelastic strain demand at critical locations near the beam flange welds [Engelhardt, et al. 2000]. However, some of the European tests have demonstrated problems with the RBS beam-column connection. Failures have been observed in the welds due to high stress concentrations, thus RBS is not considered as an acceptable solution by many [Mazzolani 2000]. The RBS moment connection also results in lower capacity of the beam at the reduced beam section. This could lead to larger members, thus higher material costs. In addition, because the flanges are cut to form the RBS, the beam is not as stable laterally as the uncut beam would be. As a result, special attention must be paid to lateral bracing of the flanges at the reduced beam section. While the RBS connection has shown promise as a means to eliminate the fatigue cracks at the complete joint penetration welds, this approach to a solution only treats the symptom of the problem. It eliminates
the groove welds as the weakest link in the connection and creates a failure in the RBS. It does not attempt to understand or alleviate the fatigue crack problem in welded steel moment connections.

Fig. 1.7 – Radius cut reduced beam section (RBS) moment connection

In order to assure plastic hinging, and ductile behavior, a strong column-weak beam design should be employed [FEMA 350]. The prequalified connection used prior to Northridge was presumed to result in the formation of a plastic hinge in the beam at the face of the column or in the column panel zone or as a combination of the two [FEMA 350]. Welded moment connections should be configured to either force the inelastic action, plastic hinging, away from the face of the column, where performance is less dependent on materials and workmanship, or employ optimum welded joint design and quality assurance measures. It is also important that weld metals have tensile and
yield strengths that are close to those of the beam flanges being welded [FEMA 350]. The majority of successful tests on welded moment connections have been conducted using either 58 or 70 ksi weld metal, compared to 50 ksi yield strength for typical structural steel [FEMA 350].

1.4 Low Cycle Fatigue

The discussion above demonstrated that the ten years of research following the Northridge Earthquake has resulted in modified designs of welded steel moment frames for improving its ductility compared to pre-Northridge welded steel moment frame. However, little work has been done to understand the local failure mechanisms of the welded steel moment connections. Testing of post-Northridge welded steel moment connections have shown that many failures are initiated from localized, low cycle fatigue cracks near the welds [Barsom, et al. 1999]. Research by the European community has also demonstrated localized, low cycle fatigue fracture mechanisms and concluded that an “exhaustive understanding of the low-cycle fatigue strength of steel members and joints” is required [Mazzolani, 2000]. The primary objective of this research is to investigate the influence of local fatigue mechanisms on inducing brittle failure of welded steel moment connections.

1.5 Motivation

The current trend of earthquake engineering is gradually moving towards “Performance-Based” design. The emphasis of this concept is to design a structure that will exhibit specific performance under a prescribed level of seismic intensity
Seismology Committee, 1999]. It is essential that the cause and mechanism of failure of welded steel moment frames be understood to allow it to effectively and reliably resist seismic demands. In order to fully understand the failure mechanism, localized crack initiation should be studied along with global failure mechanisms. Extensive efforts have been made to quantify structural parameters such as drift, strength, and stability in FEMA 350. There are Steel Construction Manuals that focus on the global performance of the structure [LRFD Manual of Steel Construction]. However, missing from all of these guidelines and design methods is a basic correlation between localized member behavior and global structural performance. The local condition at one location will not dictate the performance of a structure; whereas the local condition of many structural members does dictate the performance of the entire structure [Rodgers, et al. 2006]. While many design procedures can predict the performance of a global structural parameter under prescribed load conditions, those same procedures do not attempt to correlate these quantities with local structural performance. This is not a shortcoming of the current guidelines and design procedures; however it is a limitation on our knowledge of local failure conditions. Post-Northridge researchers have mainly addressed the global level mitigation of the failure risk; however the fundamental reasons for the brittle failures at the local level were not uncovered.

Part of the motivation for this research was demonstrated in previous research performed at NC State University that explored the localized failures of welded piping joints under low cycle fatigue loading [Lu, 2003]. The prescribed loading in the test was displacement controlled and symmetrical, thus the strain responses measured at the weld toe of the butt weld joint were expected to be symmetrical. However, the recorded axial
strain gradually increased with each cycle. This phenomenon is known as ratcheting or cyclic creep. Using our current knowledge base, we can not explain the strain ratcheting observed under a displacement-controlled loading. Ratcheting is known to occur in metallic materials as they undergo inelastic cyclic loading, along the directions of force-controlled loading. Hence, it is anticipated that the occurrence of ratcheting at the welded joint under displacement-controlled loading could be influenced by residual stresses near the welded joints.

In conjunction with the ratcheting phenomenon discovered in the welded piping joints, welding procedure played a key role in the fatigue life of the joint [Lu, 2003]. Two different welding procedures were investigated by Lu [2003] for piping joints: quarter circle welding and full circle welding. For the full circle welding procedure, the joint was welded in one continuous pass around the outside of the pipe. For the quarter circle welding, the pipe was welded in four passes that spanned only one quarter of the pipe’s circumference in each step in a symmetrical manner. The joints were then tested to failure. Each welded joint failed where the stress concentration was the highest, at the top or bottom toe of the weld. The full circle welded piping joint was subjected to 300 cycles before failure, whereas the quarter circle welded joint was subjected to 560 cycles. This same study [Lu, 2003] also demonstrated the effect of welding on increasing ratcheting strain rate which was attributed to the presence of residual stress in the piping joint. Different amounts of residual stress were also shown to be related to the weld sequence. Therefore, the welding procedure can have a significant effect on the residual stress and thus the fatigue life of a joint.
Castiglioni (2005) observed the influence of loading history on the seismic performance of welded steel moment connections. He conducted several full scale tests on welded steel moment connections with varying constant amplitude loadings. He observed through analysis that below a certain amplitude, the majority of the strain accumulates in the weld resulting in a brittle failure. This observation corresponds to his experimental results. Table 1 shows that the specimens C3-30, C3-50A, and C3-50B subjected to small amplitudes failed in a brittle manner. Loads larger than that amplitude, caused the beam to buckle, thus strain accumulated in the buckled region in the following cycles and a ductile failure occurred in specimens C3-100, C3-125A, and C3-125B (Table 1). The intermediate amplitude test, C3-75, failed under a combination of these two mechanisms (Table 1). Fig. 1.8 shows the strain response from finite element analysis of the welded joint connection. Each displays the ratcheting phenomena, the accumulation of strain with cycles under displacement-controlled loading. Fig. 1.8(a) represents the strains when the joint is subjected to small amplitude cycles. Strain accumulates in the weld rather than in the beam. Fig. 1.8(b) depicts strains from a large amplitude cycle. Once the plastic hinge forms after the first cycle, the strain in the weld is relieved, while strain accumulates in the plastic hinge in the beam [Castiglioni, 2005].
Because of the occurrence of fatigue crack-initiated failure in many post-Northridge experiments, the weld interface conditions or residual stress at the weld are anticipated to be leading contributors to the fatigue initiated failures. The NC State University study on welded piping joints demonstrates the significance of welding procedure and sequence, and thus the residual stress on fatigue life. The presence of high residual stress in WSMC has been demonstrated by Dong et al. [1999] and the failure cracks are shown to initiate in the weld toe region [Suita et al., 1998, Barsom et al., 1999,
Ricles et al., 2000]. If the influence of residual stress on fatigue damage accumulation in WSMC is revealed, fatigue life can be enhanced through optimizing the weld sequence. The SAC Joint Venture did not investigate any aspect of the welding procedure (number and size of weld beads and their sequence and the weld speed) and its influence on strain accumulation near the weld toe and base metals due to high temperature cycles during welding. Thus, a study is needed to capture the influence of welding procedure on failures of welded steel moment connections.

Fig. 1.9 displays a typical welded steel moment connection. Figs. 1.10 and 1.11 show a cut out (section A-A in Fig. 1.9) of the bottom beam flange. Fig. 1.10 illustrates a weld sequence option for applying the complete joint penetration groove weld to the bottom beam flange. When using this sequence, the welder makes all the passes required to fill the groove from one side of the beam web (1 and 2). Each pass is stepped back slightly to prevent all of the start-stop points from falling on top of each other. Once one side of the weld is complete, the welder finishes the weld by filling the groove on the opposite side of the beam web (3 and 4). Another weld sequence option is shown in Fig. 1.11. In this sequence, the welder places the first bead from one side of the beam web (1), and then completes the bead by moving to the other side of the beam and finishing the pass (2). The next bead is then started from the same side that the last bead was finished from and pulled from the opposite side of the web to the outer edge of the flange (3). This bead is then completed by switching sides of the beam and completing the pass (4). These two weld sequence options may cause different residual stress distributions in the bottom beam flange and the column flange. One of these weld sequence options can adversely affect the fatigue life of the connection. Due to similar reasons, weld bead size
can also cause differences in fatigue life of the welded steel moment connection. None of these factors are taken into consideration in the design of the connection, yet they can greatly influence the service life of the joint.

Fig. 1.9 – Welded steel moment connection

Fig. 1.10 – Section A-A and welding sequence 1
The objective of this research is to investigate the local low cycle fatigue failure mechanisms observed in post-Northridge welded steel moment connections. Steel is a ductile material; however local ductility is limited due to local instabilities. Ductility is also adversely affected under cyclic loading by cracking of the base material or near welds as a consequence of low cycle fatigue. Brittle fracture of steel members and connections occur under restrained conditions, which are present at the complete joint penetration weld at the beam flange-column flange interface [Mazzolani 2000]. In order to investigate the local welding effects on low cycle fatigue, tests will be performed on numerous plate specimens as well as simplified beam-column tee-joint specimens.

1.6 Test Specimens and Experiments

The small plate specimens (see Chapter 2 for a sketch) will investigate the heterogeneous material properties of the base metal near welds. Tensile tests to failure
will be performed on base metal specimens as well as specimens that are conditioned by exposure to various high temperatures to determine variation of the material properties of the heat affected zone. Cyclic tests will be performed to find the cyclic material properties of the unconditioned and temperature conditioned specimens. Welded plate specimens will be cyclically loaded to determine the properties of the base and weld metals.

In addition to the plate specimens, tests will be conducted on tee-joints of the bottom beam flange and column (see Chapter 2 for a sketch). These tests will consist of a bottom beam flange WT-section attached to short columns on each end by a bottom beam flange complete joint penetration groove weld. This entire specimen will be subjected to tension and compression loading along the axis of the WT beam in order to simulate the load reversal experienced by the beam flange in the welded steel moment connection. Pretest finite element analysis will be performed on a full scale beam-column connection (see Chapter 2 for a sketch) using material properties of base and weld metals found in the literature. The results from this analysis will yield displacements and strains that correspond to varying levels of rotation experienced by the beam-column connection. These displacements and strains will determine the displacement cycles to be prescribed to the small tee-joint specimens, so the results of the test can represent the local failure mechanism of the full scale welded steel moment connection. The results obtained from the small tee test, will be compared to the strains acquired from the finite element analysis to see if there is good correlation.
CHAPTER 2
ANALYTICAL INVESTIGATION

2.1 Full-scale Welded Steel Moment Connection Model

The finite element study used in this research project involved modeling the full scale post-Northridge welded steel moment connection and bottom beam flange beam-column tee-joints using members of the same size and geometry as those used in the experiments. In this thesis, only the bottom beam flange tee joint test will be performed. However, modeling the full-scale welded steel moment connection is necessary to ensure that the behavior of the bottom beam flange tee-joint simulates the behavior of the full-scale connection. Thus, the full-scale test was modeled first. The general purpose nonlinear finite element analysis program ANSYS 9.0 was used to create the 3-D simulation models of the two test setups.

2.1.1 Modeling

Fig. 2.1 shows the idealized centerline model of the full-scale welded steel moment connection to be tested. The beam in the test setup is a 126 in. W18x55, and the column is a 132 in. W14x74. The column has a length of 108 in. between the supports on the strong wall. The beam span from the centerline of the column to the point of load application is 114 in. In the finite element model, the length of the column is 108 in. to represent the span between the supports on the strong wall, and thick plates are included at each end to eliminate stress concentrations caused by boundary conditions placed on the column nodes. The beam includes a web stiffener in the test setup as well as the model. This eliminates any problems with web buckling and crippling due to loads applied during testing, and it eliminates stress concentrations at the point of load and displacement application. By eliminating the stress concentrations at the supports and
loads, the areas of interest, in and near the beam flange to column flange full penetration groove welds, can be studied more closely. All of the geometry of the test setups are included in the finite element models. The weld access hole detail complies with current AISC Seismic Design Specifications and FEMA 350. The holes in the continuity plates are also included in the model.

Fig. 2.1 – Idealized welded steel moment connection test setup

One discrepancy between the actual full-scale welded steel moment connection model and test setup is the attachment of the beam web to the column flange. In reality, the beam flange is bolted with high strength bolts to a shear tab that is connected to the column flange using fillet welds. In the model, we assume that the beam web is attached to the column flange by welding, neglecting the bolts, the shear tab, and the fillet welds.
Previous finite element analysis and experiments have shown that the beam web attachment details are not as significant as the flange attachment details to the performance of the joint [Ricles 2000]. For this research, the area of interest is the vicinity near the beam flange welds.

The purpose of this modeling is experimental guidance, so shell elements provide a quick and efficient way to simulate the experimental test setup. Because shell elements are used, it is not possible to model the geometry of the full penetration groove weld exactly. For these simulation models, the weld is modeled as a 5/8 in. strip (the average width of the weld) at the face of the columns. This strip has the same mesh density as the surrounding areas, however the material property is that of 70 ksi weld metal as opposed to A992 steel which has a yield stress of 55 ksi.

The 3-D simulation model of the full-scale welded steel moment connection and the bottom beam flange beam-column joint are both meshed using eight-noded shell elements (element SHELL93 in the ANSYS element library). These elements have six degrees of freedom at each node (translations and rotations about the nodal x, y, and z directions). The deformation shapes are quadratic, and it has plasticity, stress stiffening, large deflection, and large strain capabilities.

Generic material properties for A992 steel and E70 weld metal used were obtained from Ricles, et.al [2000]. The A992 stress-strain curve is shown in Fig. 2.2, and the E70 weld metal stress-strain curve is shown in Fig. 2.3. The A992 steel has a modulus of elasticity of 29000 ksi. It yields at 55 ksi, and has an ultimate strength of 70 ksi. The yield plateau begins at a yield strain of 0.001897 and ends at a strain 11 times higher than the yield strain, 0.02086. The ultimate strain is 120 times the yield strain,
The E70 weld metal also has a modulus of elasticity of 29000 ksi. It reaches a stress of 70 ksi and a strain of 0.002414 at yielding. The ultimate stress reaches 90 ksi at a strain of 0.1. In order to more accurately simulate the actual test setups, the generic material properties for the A992 steel will be replaced in the finite element model by actual material properties from uniaxial tests performed on coupons of the base metal used in the welded steel moment connection.

![Stress-Strain Curve for A992 Steel](image)

**Fig. 2.2 – Generic stress-strain curve for A992 steel (Ricles, et. al [2000])**
To test convergence of the 3-D full-scale welded steel moment connection model and achieve a balance between accuracy and computational time, four different meshes were tested. The coarsest mesh (Mesh 1), shown in Fig. 2.4, consists of 2” square shell elements. In this full-scale connection model, there are approximately 3500 elements and 10900 nodes. The mesh was then refined near the beam to column connection because this is the area of interest and it is where the majority of the inelastic action should occur. After one mesh refinement, the model consists of 1” shell elements near the beam-column joint and 2” shell elements everywhere else. This mesh (Mesh 2) contains approximately 8100 elements and 24600 nodes. The next refinement contains ½” shell elements near the beam-column joint, 2” shell elements at the ends of the beam and
column, and 1” shell elements in between. This mesh (Mesh 3) contains approximately 14800 elements and approximately 45000 nodes. The finest mesh, and the last mesh refinement, contains ¼” shell elements near the beam-column interface, 1” shell elements at the beam and column ends, and ½” shell elements in between. This mesh (Mesh 4) contained approximately 53300 elements and 161000 nodes. Fig. 2.5 displays the finest mesh considered. Fig. 2.6 is a closeup view of mesh 4 near the beam to column connection. After conducting the convergence study on the full-scale welded steel moment connection model, the chosen mesh size will be used on the bottom beam flange beam-column joint model to allow comparisons to be made between stress and strain distributions across the width of the beam flange.

Fig. 2.4 – Mesh 1: Coarse finite element mesh
Fig. 2.5 – Mesh 4: Fine finite element mesh
To simulate the supports imposed on the experimental full-scale welded steel moment connection, boundary conditions were placed on the finite element model. The nodes at the bottom of the column and on the opposite face from the beam are pinned: they are not allowed to translate in the x (parallel to the beam), y (parallel to the column), or z (perpendicular to the beam and column) direction and they are not allowed to rotate about the x or y axes. In order to restrain the nodes at the top of the column on the opposite side from the beam, a roller support is used: the nodes are not allowed to translate in the x or z direction or rotate about the x or y axes.

Fig. 2.6 – Close-up view of mesh 4 at the beam to column connection
To test convergence of the different mesh sizes, the full-scale welded steel moment connection model was run with two different applied loads. Although the experiments and the simulation model will be ran using displacement controlled loading, convergence can be evaluated when a load is applied. Two different loads were placed at the beam end to determine the effect of yielding on convergence. One load (25 kips) causes minimal yielding in the base metal near the weld, while the other load (35 kips) causes substantial yielding in the base metal near the weld. To evaluate convergence, beam tip displacements and the stresses and strains in the base metal in the beam flange at the weld and at the intersection of the beam flange and web at the weld access hole were analyzed.

By analyzing the displacements at the end of the beam, the convergence of the global behavior of the full-scale welded steel moment connection model can be determined. The global behavior of the models converged quickly. With the 25 kip load applied on the beam of the coarse mesh, the maximum displacement is -0.718204045". The displacement after one mesh refinement was -0.718693349"; after two refinements, the displacement went to -0.719018371"; and with the finest mesh, the displacement is -0.719354545". The maximum difference between these values is less than 0.05%, thus the global performance of all the meshes is adequate.

To determine local convergence, the stresses and strains across the beam flange were analyzed. Fig. 2.7 shows the von Mises stress distribution in the base metal across the width of the top beam flange (for this model, the top and bottom flanges gave similar results) at the weld interface when a 25 kip load is applied on the beam. Mesh 1 is the coarsest mesh, mesh 4 is the finest mesh, and mesh 2 and 3 are in between. This loading
causes minimal yielding in the beam, so the model is mostly elastic. The stress in the bottom flange is a mirror image of the top stress, so it is not shown. The stress converges between the third and fourth mesh refinements. Fig. 2.8 shows the x-direction strain (strain in the direction of the W18x55 beam) distribution in the base metal across the width of the top beam flange at the weld interface when a 25 kip load is applied. Once again, the strains converge between meshes 3 and 4.

![Graph showing von Mises stress in the top beam flange in the base metal (25 kip load)](image)

**Fig. 2.7 – von Mises stress in the top beam flange in the base metal (25 kip load)**
In addition to the 25 kip load which caused minimal yielding in the base metal, models were ran using a 35 kip load on the beam that caused significant yielding in the beam flange. Figure 2.9 shows the von Mises stress distribution in the base metal across the width of the top flange of the beam at the weld interface (at the same nodes used for the model with a 25 kip load applied) when a 35 kip load is applied. The stress converges between meshes 3 and 4 once again. The bottom flange is a mirror image of the top flange, so it is not shown. Figure 2.10 shows the x-direction strain in the base metal across the width of the top beam flange at the weld interface when a 35 kip load is applied. The strains do not converge as well as the stresses due to the small slope in the
strain hardening region of the stress-strain curve of A992 steel because a small change in stress yields a large change in strain.

Fig. 2.9 – von Mises stress in the top beam flange in the base metal (35 kip load)
Fig. 2.10 – Axial strains in the top flange in the base metal (35 kip load)

After reviewing the results of the convergence study, Mesh 4 (the finest mesh) was chosen to simulate the full-scale welded steel moment connection and the bottom beam flange tee joint specimen. The beam tip displacements were less than 0.05% different from mesh to mesh, therefore all of the meshes considered gave satisfactory results for global performance. The stress values seemed to converge between Mesh 3 and Mesh 4. The largest percent difference between any of the values for stress and strain was less than 8%. Thus, all of the meshes gave relatively good results for the local behavior. Computational time was not a factor in this choice.

Upon choosing a mesh, the full-scale welded steel moment connection model was run using the SAC loading protocol (Fig. 2.11) and a multilinear fit of the base metal
stress-strain curve (Fig. 2.12). The material model used for these more detailed analyses was a multilinear model with kinematic hardening. The modulus of elasticity remained the same at 29000 ksi, the yield stress is 53 ksi, and the strain hardening region follows the monotonic test results obtained from the base metal test. This analysis does not contain any of the data from the temperature conditioned coupons that give material properties in the heat affected zone. It also does not contain any data on residual stresses due to the welding. To examine the behavior of the full-scale specimen, displacements were applied at the beam end that corresponded to the different joint rotations prescribed by the loading protocol. This analysis was done in part for experimental guidance when testing the bottom flange tee joint specimens as well as for observations of the connection behavior when the full-scale tests are performed at a later date.

**SAC Loading Protocol**

![SAC Loading Protocol](image)

**Fig. 2.11 – SAC Loading Protocol with increasing amplitudes**
2.1.2 Results

Fig. 2.13 shows the distribution of axial strains across the width of the flange for the first 5 phases of the SAC loading protocol. These strains are in the x-direction (along the length of the beam) and located on the outside of the beam flange in the base metal approximately 1.3125 in. from the face of the column. They represent the tensile strains along the top and bottom beam flanges. The compressive strains are very close to these values, so they aren’t shown. The first two sets of cycles in the SAC loading protocol result in no inelastic action according to the model. In the third step, there is a small amount of yielding in the center of the flange, and in the fourth step, the strains increase and the region of inelastic action expands from the center out towards the beam flange.
tips. The fifth step results in the yielding of the entire flange according to the finite element model. Step 6 and beyond gave strains that are higher than the range of the strain gages, so these results are not needed as the focus of the research is the local behavior.

![Strain Distribution](Image)

**Fig. 2.13 – Axial strain distribution across the width of the flange**

The contour plot shown in Fig. 2.14 displays the concentration of von Mises stress near the beam to column joint. The plot also verifies the validity of the connection design because the highest stress levels occur in the beam flanges, thus our test specimens should exhibit the desired behavior with most of the inelastic action developed near the complete joint penetration groove welds at the top and bottom beam flanges. As desired, the model predicts failure near the beam flange to column flange connection. The full-scale moment connections show specific areas of stress concentrations near the
weld access hole and beam flange as shown in Fig. 2.15 and Fig. 2.16 respectively. The center of the beam flange and the flange tips near the weld are of particular interest as well as in the weld access hole itself.

Fig. 2.14 – Von Mises stress (ksi) distribution near the beam-column connection

(Max. stress = 71.345 ksi)
Fig. 2.15 – Von Mises stress (ksi) concentrations near the bottom beam flange edges and the weld access hole
The primary focus of this research is to understand the local response of the bottom flange tee joint specimens. This specimen was developed to study the local behavior of the complete joint penetration groove weld while using smaller specimens than the full-scale welded steel moment connections. The bottom flange tee joint test specimen includes two bottom beam flange to column flange welded joints as shown on Fig. 2.17. In the full-scale connection, the joint sees large moments at the face of the column. These moments can be broken down into a tensile and compressive force in the
top and bottom beam flanges. Thus, the bottom flange tee joint will be subjected to axial tension and compression to simulate the moments in the beam. The specimen also provides two sets of data to be collected from the bottom beam flange joint as opposed to only one set of data from the standard full-scale welded moment connection. It was necessary to model the full-scale welded steel moment connection in order to verify the connection design, gain an understanding its behavior, and verify that its behavior is similar to that of the bottom beam flange tee joint specimen. Ultimately, the results from the bottom flange tee joint specimens will be compared to the results given by the full-scale connection tests.

Fig. 2.17 – Sketch of the bottom beam flange tee joint specimen and the loading direction
2.2.1 Modeling

A model of the bottom flange tee joint was created using the same material properties, elements, and mesh that was used in the full-scale welded steel moment connection model. Fig. 2.17 shows the sketch of the bottom flange tee joint specimen and loading direction. The W14x74 columns are 48 in. long each and the modified WT-9x27.5 section is 36 in. long. The beam flange is located 22.375” from the bottoms of the column. There are 5/8” continuity plates inside the column in line with the beam flange just like in the full-scale connection. These plates stiffen the column flange and prevent the column web from buckling under high compressive loads. There are also ½” plates welded to the column web. These plates act similarly to the doubler plate in the full-scale connection’s panel zone. It also helps prevent the column web from crushing. The modified WT-9x27.5 section has a weld access hole in the web, and it is connected to the column flange with a complete joint penetration weld between the beam flange and the column flange and fillet welds along the beam web and column flange. The beam flange is welded at the same position using the same procedure used on the bottom beam flange of the full-scale connection. In the model, the complete joint penetration weld is modeled using a 5/8” wide strip of shell elements that have the weld metal material properties. These connections are modeled the same way as they are in the full-scale welded steel moment connection model.

Fig. 2.18 shows the bottom flange tee joint connection model. It consists of ¼” shell elements in the vicinity of the bottom beam flange complete joint penetration weld and ½” shell elements further away from the joint. The size of the specimen did not allow larger elements to be used further from the connections. This model contains
36910 shell elements and 111681 nodes. The boundary conditions on this model under tensile displacement consisted of fixing all of the nodes on the inside face of the column from each end to 14” from the end against translating in the x-direction (in the direction of the WT9x27.5 beam). This dimension is consistent with the size of the plates used to lock the specimen to the strong floor and the support beam (see Chapter 3 for test setup details). The matching nodes on the inside face of the other column were given the desired displacement in the x-direction in order to match the strains with those observed in the bottom flange of the full-scale connection at each SAC step. When the specimen undergoes a compressive force, all of the nodes on the outside face of one column are restrained from translating in the x-direction. The desired displacement is imposed on all of the nodes on the outside face of the opposite column in the x-direction. This is consistent with the boundary conditions seen by the specimen during the test as it bears on the spreader beam and the strong floor.
2.2.2 Results

Because an eventual goal of this research is to compare the results obtained in the bottom beam flange tee joint specimens to the results of the full-scale connection, a correlation between the beam tip displacements (governed by joint rotations from the SAC Protocol) and the displacements applied to the bottom beam flange tee joint specimen must be determined. We decided to use the strain intensities and distributions to provide correlation between the two independent models. Initially, the bottom beam flange tee joint model was run using incremental displacements to get a feel for the stress

Fig. 2.18 – Finite element mesh for the bottom beam flange tee joint specimen
and strain intensities and distributions. Once a correlation between displacement and strain was reached, the model was run with displacements that gave similar strain distributions across the beam flange width to those seen in the finite element simulation model of the full-scale connection. Fig. 2.19 shows the correlation between the axial strain distributions for the full-scale connection model at a rotation of 0.02 radians and the bottom flange tee joint model at a displacement of 0.05 inches in the beam. The axial strain values shown are the axial strains on the outside of the beam flange in the direction of the beam. The shape of the distribution is similar meaning that the bottom flange tee joint specimen under axial tension and compression can reasonably replicate the behavior of the beam in the full-scale welded steel moment connection.

Fig. 2.19 Full-scale welded steel moment connection and bottom beam flange tee-joint axial strain simulation comparison
Fig. 2.20 displays the stress contour plot for the bottom flange tee joint specimen. Once again, higher stresses are concentrated near the beam to column connection, thus verifying the design of our test specimen. The simulation predicts failure in the region of interest, near the complete joint penetration groove weld. The stress contours in the region of interest are similar for the full-scale connection simulation and the bottom flange tee joint simulation. When comparing the bottom flange tee joint simulation (Fig. 2.20) to the full-scale welded steel moment connection (Fig. 2.14), the areas of interest are the connections of the beam flanges to the columns. The highest stresses and the most intense stress concentrations are located in this region. This is also the area where the failures have been observed in previous research. The similarities between the two simulations provide support for the validity of the bottom flange tee joint specimen as an alternative to the full-scale welded steel moment connection.
Fig. 2.20 – Von Mises stress (ksi) contour plot for the bottom beam flange tee joint

(Max. stress = 74.07 ksi)
Fig. 2.21 – Von Mises stress (ksi) concentrations in the bottom beam flange edges
By comparing the stress contour plots in Fig. 2.15 and Fig. 2.21, it is evident that the two model simulations yield similar stress distributions on the inside face of the beam near the column face. Similarly, Fig. 2.16 and Fig. 2.22 display the similarity of von Mises stress contours on the outside face of the beam flanges from the two model simulations. These plots show similarities in the stress distribution in the area of interest, the region near the complete joint penetration groove weld and the weld access hole. Thus, the bottom beam flange tee joint specimen should reasonably simulate the behavior of the full-scale welded steel moment connection.
2.3 Conclusions

The full-scale welded steel moment connection model verifies our specimen design and provides some insight into the behavior of the joint. The bottom flange tee joint model also verifies the design of our simplified specimen and provides insight into it behavior. By comparing the two models, it can be concluded that the bottom flange tee joint specimen reasonably simulates the behavior of the full-scale welded steel moment connection. The strain distributions across the width of the flange in the area of interest are similar, the stress contours are similar near the complete joint penetration groove weld, and the areas of stress concentration are similar in both models. According to the results of the bottom beam flange tee joint model, this test will reasonably simulate the behavior of the full-scale welded steel moment connection while giving two sets of data about the local behavior in a smaller, cheaper specimen.
CHAPTER 3
EXPERIMENTAL INVESTIGATION

3.1 Introduction

Many modern steel buildings, from low to high rise structures, rely on steel moment frames to resist lateral loads from wind and earthquakes. Wind loads are applied externally and can be easily visualized, while earthquake loads come from inertial forces as the ground and the building foundation undergo lateral and vertical oscillation accelerations. Under small lateral loads, frames are designed to remain elastic, however during severe earthquakes, elastic design does not yield good performance. During severe events, the frame must yield and react in a ductile manner in order to dissipate energy. To withstand seismic loading safely, special detailing is required in moment connections of the moment frame. In order for the frame to reach design ductility and strength levels, the connections must remain intact so plastic hinges can form in the beam and absorb the energy generated during an earthquake. During the Northridge Earthquake, steel moment connections fractured in a brittle manner prior to plastic hinge formation, resulting in insufficient performance of the lateral load resisting system.

Following the Northridge Earthquake, numerous investigations have yielded various solutions to the brittle connection fractures. These solutions improved the performance of the steel moment frame, however steel moment connections still experience low cycle fatigue fracture in the region near the complete joint penetration weld between the bottom beam flange and column flange. Thus, the local failure mechanism of the joint is not fully understood. In order to gain understanding of this failure mechanism, the experimental program included three types of specimens: (1) full-scale exterior welded
steel moment connections, (2) bottom beam flange tee joints, and (3) tests on plates of base and weld metals.

3.2 Full-Scale Exterior Welded Steel Moment Connections

In order to understand the correlation between global and local failures of welded steel moment connections, cyclic experiments on full-scale exterior connections should be performed. These specimens are the basis for all of the other experiments in this project even though they will be performed in later research. Moment frames contain both interior and exterior connections as shown in Fig. 3.1. Exterior connections consist of a beam connected (welded and bolted) to a column, whereas interior connections consist of two beams connected to a column, one on each side (Fig. 3.1).

![Fig. 3.1 – Interior and exterior connections in moment frames](Fig_3.1.png)
3.2.1 Basic Setup

The full-scale exterior connection (see Figs. 3.2 and 3.3) consist of a W18x55 A992 beam and a W14x74 A992 column designed and detailed according to AISC Seismic Design Provisions [LRFD Manual of Steel Construction], FEMA 350 [FEMA 350 2000], and AWS D1.1. The web of the beam is connected to the column flange by a shear tab that is connected to the beam web with high strength bolts and welded to the column flange. The beam flanges are welded to the column flange using a complete joint penetration groove weld according to the post-Northridge design recommendations [FEMA 350]. The connection also contains continuity plates in the column that coincide with the top and bottom beam flanges and a doubler plate in the column panel zone between the continuity plates. All of the connection’s field bolting and welding will be performed in the vertical position, as the joint is oriented in the moment frame. Fig. 3.2 depicts the connection details.
Fig. 3.2 – Sketch of the full-scale exterior welded steel moment connection to be tested

The full-scale specimen will be positioned vertically during testing, as it is oriented in an actual moment frame, and securely fastened at the top and bottom of the column to a strong wall to prevent lateral movement. Rollers will be placed between the column and the supports to allow rotation at the column supports, such that those points in the column represent inflection points. A concrete or steel block will be placed between the specimen and the strong floor to provide vertical support. Rollers will also be placed between the column and the block to allow horizontal translation of the column end as the column rotates. Load will be applied near the beam end by a 220 kip actuator. Fig. 3.3 shows the full-scale exterior connection test setup. The W18x55 beam is 125.75
in. long, and the load is applied 114 in. from the column centerline. The W14x74 column is 132 in. long, and the wall supports are spaced at 108 in.

In order to gain an understanding of the effect of the weld procedure on connection performance, welding will be closely monitored. Weld sequence, start and stop locations, bead size and length, weld speed, weld direction, temperature cycles, voltage, and current will be recorded. An eventual outcome of these tests and future finite element analysis will be an optimized weld procedure that increases fatigue resistance of the welded steel moment connection. The full-scale connection tests will provide correlation between local parameters and global performance. They will also provide validation of the bottom beam flange tee joint tests.

Fig. 3.3 – Full-scale exterior welded steel moment connection specimen and test set-up
3.2.2 Loading Protocol Options

There are different types of loading for the full scale welded steel moment connections. One type of loading can be that developed by the SAC Joint Venture (see Fig. 3.4). It is a displacement-controlled cyclic loading path based on joint rotation with increasing displacement amplitudes. This loading was used during the SAC Joint Venture tests, and it is used to validate moment connection seismic performance. The other type of loading can be constant amplitude cycles. Castiglioni [2005] demonstrated the effects of constant amplitude loading cycles on connection performance. He showed that small displacement, constant amplitude cycles cause brittle connection failure in the region near the weld, while large displacement, constant amplitude cycles cause ductile failure through local buckling. The loading to be prescribed will be determined by future finite element simulation of the connection or results from the bottom beam flange tee joint tests and simulations.
3.3 Bottom Beam Flange Tee Joint Specimens

The main objective of this investigation is to study and understand the local failure mechanisms in the welded steel moment connection through testing bottom flange tee joint specimens. The bottom flange joint is studied instead of the top flange for two reasons: (1) the bottom beam flange to column flange joint is more susceptible to fatigue crack initiation than the top flange [Ricles et. al, 2000] and (2) the welding sequence is influenced by the presence of the beam web and the weld access hole (see Fig. 3.5). The web of the beam hinders the completion of a single weld pass across the width of the beam flange. Therefore, the weld is not as uniform as the top flange weld. The start and
stop points in the bottom flange weld may cause discontinuities or higher residual stresses in the joint that lead to low cycle fatigue failure.

In order to test the bottom beam flange by itself, a simplified test specimen needed to be fabricated. When a moment connection is subjected to lateral loading such as seismic events, the joint sees a reversing bending moment that can be broken into a tensile and compressive force in the top and bottom beam flanges (see the first two schematics in Fig. 3.6). If the connection is further simplified by cutting the beam in half, we can represent the bottom beam flange of the welded steel moment connection as a uniaxial test specimen (the last two schematics in Fig. 3.6). By welding a section of the column to each end of a short section of the beam, the bottom beam flange tee joint test specimen includes two bottom beam flange to column flange welded joints. This allows two sets of data to be collected from a single bottom beam flange joint test.
3.3.1 Specimen Fabrication and Test Setup

The bottom beam flange tee joint specimen shown in Fig. 3.7 consists of a 36” modified WT9x27.5 A992 beam and two 48” W14x74 A992 columns. The modified WT9x27.5 is cut from a W18x55 with a web height of 7.5” from the outside of the flange. It is connected to the W14x74 columns by complete joint penetration groove welds at the flanges and fillet welds along the web. The weld access hole is identical to that on the full-scale welded moment connection. The welding procedures used to place the full penetration groove weld will also be identical to those used in the full scale test specimens, and they will also be in accordance with AWS D1.1 [2004]. The W14x74 columns contain 5/8” continuity plates, which are also present in the full scale specimens,
and 1/2” doubler plates which represent the doubler plate in the panel zone of the full scale test specimen and prevent web yielding.

---

**Fig. 3.7 – Bottom beam flange tee joint specimen**
Fig. 3.8 shows a view of the weld access hole, which has the modified geometry, per the SAC research, in the web of the bottom beam flange tee joint specimen. The weld access hole is identical to those cut into the webs of wide flange beams used in welded steel moment connections. The figure also shows the backing bar which is tack welded to the column flange. Fig. 3.9 shows the same connection after the complete joint penetration weld has been placed. As shown in Fig. 3.5, the weld sequence in the bottom flange was interrupted by the beam web. Each successive bead was started on the opposite side of the web from which the welder was positioned and pulled to the outside edge of the flange (see Fig. 3.11). This method fills the groove evenly across the width of the flange one bead at a time. Fig. 3.10 shows the completed bottom beam flange complete joint penetration weld. Note in this figure that the backing bar has been
removed from the joint by cutting it away with a torch, and a reinforcing fillet weld has been applied in its place. This reinforcing fillet weld is another post-Northridge modification that improves the fatigue life of the connection by reducing stress concentrations in the beam flange to column flange joint.

Fig. 3.9 – Complete joint penetration groove weld placed to join the bottom beam flange to the column flange
The welding procedure was closely monitored, as in the full-scale specimen, and all details were be recorded. The purpose of this close observation is to detect any subtle differences welding procedure has on the fatigue life of the connection. As seen in previous work on piping at NCSU [Lu 2003], welding procedure has an impact on the fatigue life of the joint. For this project, all of the bottom beam flange tee joint specimens were fabricated using the same welding procedure (see Fig. 3.11). After completing the root pass across the width of the beam flange, the welder would complete each successive bead before proceeding to the next one. In Fig. 3.11, the weld passes are numbered in the order of completion. An alternate welding procedure which will be investigated in a later project is shown in Fig. 3.12. When using this procedure, the welder fills the groove on one side of the web after completing the root pass. Then, he
completes the weld by switching to the other side of the web and finishing each pass. Since, all of the bottom flange tee joint specimens in this project have the same welding procedure, the only variables in the weld are the bead size and order in which they were placed. The beam web was welded to the column using a fillet weld. In the welded steel moment connection, the web is bolted to a shear tab that is welded to the column with a fillet weld. Instead of attaching a short shear tab to the column in the bottom flange tee joint specimen, it was decided to simply weld the beam web to the column. The focus of this test is to observe the behavior of the beam flange to column flange connection, so the connection of the web was not as critical.
In order to monitor the behavior of the bottom beam flange tee joint specimen, a combination of instruments were used. The global behavior of the specimen will be monitored by load cells, short longfellows, and LVDT’s. The load cells integrated into the two 440 kip actuators were utilized to measure force applied on the specimen. Two 2 in. short longfellows were mounted between the column flanges to measure the axial displacements in the beam. Short longfellows were used to provide high resolution in the displacement data because very small displacements were anticipated according to the finite element analysis. Fig. 3.13 shows the mounting locations of the instruments. One of the short longfellows was mounted approximately 1 in. from the beam flange at its
centerline, while the other one was mounted at the edge of the beam web along the
column centerline. The short longfellow mounted next to the flange was used in data
analysis because the behavior near the region of the beam flange to column flange
complete joint penetration weld is the focus of the research. The short longfellow
mounted next to the beam web was used to ensure equal displacements were applied to
both sides of the beam. Unequal displacements between the two short longfellows would
indicate bending in the specimen, whereas purely axial forces are desired to simulate the
compressive and tensile forces present in the beam flanges in the full-scale connection
when it is subjected to bending moments induced by lateral loads. The LVDT’s in the
actuators were used to control the test because the loading is displacement controlled,
however there was a large difference in the applied displacements and those seen by the
specimen due to elastic motion of the test setup components. The two additional LVDT’s
were used to measure lateral deflections of the beam as it is loaded. They were mounted
at the midpoint of the beam, perpendicular to each other, to ensure there are no large
deflections that may indicate a global buckling failure mechanism (only one is shown in
Fig. 3.13). In order to understand the local failure mechanisms near the complete joint
penetration groove weld, the specimen must fail locally or accumulate local damage in
the region near the weld.
In addition to load and displacement measurements, nine strain gages were used to monitor each beam flange to column flange connection (18 total strain gages per specimen). The gages were 1/16 in. uniaxial gages oriented parallel to the beam and in
the direction of loading. Fig. 3.14 shows the gage locations. The first row, closest to the weld, is located 1 5/16 in. from the column flange and approximately 3/8 in. from the weld. This is our main area of interest as we are primarily focused on the local failure mechanism near the weld. Four gages are mounted on the outside of the beam, and three gages are mounted inside the beam (web side) in the first row. One gage is in the second row, 2 13/16 in. from the column flange and 1 ½ in. from the first row, along the beam’s centerline. There is also one gage in the third row. It is 4 5/16 in. from the column flange and 1 ½ in. from the second row, and it is mounted on the beam’s centerline as well. The purpose of the second and third row of strain gages is to monitor the strain distribution along the beam length. Fig. 3.14 is shown looking down on the bottom beam flange tee joint specimen from the web side of the beam. In the results section, the designation for the strain gage location is SG-R1-O-2, for example. “SG” stands for strain gage. “R1” indicates the gage is in row 1 (similarly, “R2” for row 2 and “R3” for row 3). “O” indicates the gage is located on the outside (opposite of the web) of the beam (“I” means the gage is on the inside, or web side of the beam); and “2” indicates the gage location across the width of the flange.
3.3.2 Loading Protocol

The bottom beam flange tee joint specimens will be loaded using an increasing amplitude loading protocol, similar to the SAC Protocol, and constant displacement amplitude cyclic loading. When testing the full scale welded steel moment connection, the applied loading causes a bending moment and shear force at the connection. The bending moment in the full scale test can be broken into a tensile and compressive force in the top and bottom beam flange. For this reason, the bottom beam flange tee joint specimen will be tested in axial tension and compression to simulate the bending seen in the beam to column joint in the full-scale moment connection. Fig. 3.15 shows a sketch of the test setup. In the test setup, the specimen will be oriented vertically as shown in Fig. 3.16. It will be anchored to the strong floor by a combination of four high strength
post-tensioning bars and two built-up beams made from two channel sections and two plates. At the other end of the specimen, two 440 kip actuators apply load which is combined by using a strong spreader beam made of two large channels and two thick plates. The specimen is connected to the spreader beam by four more high strength post-tensioning bars and four pairs of channels. The loading induced by the actuators will be applied in such a way to minimize bending in the modified WT9x27.5 flange, thus ensuring maximum tensile and compressive forces will be present at the welds. Fig. 3.17 shows the actual test setup with a specimen in place.

![Fig. 3.15 – Bottom beam flange tee joint test schematic](image-url)
Fig. 3.16 – Bottom beam flange tee joint test setup
3.3.3 Basic Results

The global response of the bottom beam flange tee joint specimen is best assessed by looking at the load-displacement relationship. Fig. 3.18 shows an example of the load-displacement data from one of the tested specimens. The load is found by summing the loads from each actuator, and the displacement is recorded by the short longfellow mounted next to the flange of the modified WT9x27.5 beam. These global plots are also used to display information collected from full-scale welded steel moment connection tests from other research. In these tests, the load is applied at the beam tip, and the displacements are measured at the same point.
Load-displacement plots can indicate several aspects of the connection’s behavior. The size and area covered by the hysteresis loops indicates the level of ductility displayed by the system. Connections that yield large hysteresis loops generally dissipate large amounts of energy and behave in a ductile manner with a progressive failure mechanism, such as buckling. On the other hand, systems that yield small, narrow loops generally dissipate little energy, lack ductility, and fail in a more sudden, brittle manner. Load-displacement plots also show damage accumulation. Changes in the slope of the linear portion of the loop indicate a change in stiffness (a flatter slope equals less stiffness and a steeper slope equals higher stiffness). Loss of stiffness can be attributed to the accumulation or propagation of cracks or other damage in the system.
The strain gages yield similar curves to the load-displacement hysteresis loops when the strain is plotted verses load. The load-strain hysteresis loops can display the same connection behavior traits (ductility, energy dissipation, and failure mode) as the load-displacement hysteresis loops on a more local level. In addition to the common traits, the load-strain graph can indicate the presence of strain accumulation or ratcheting in the connection by the movement of the loops towards higher or lower strains while the specimen is subjected to constant displacement amplitude cycles. In order to determine the ratcheting behavior of the connection, the load-strain response can be further analyzed by determining the mean and amplitude strain value for each hysteresis loop. The mean strain is the average of the positive (tensile in most cases) and negative (primarily compressive) strain in one particular loop, and the strain amplitude is half the difference between the positive and negative peak strains. By plotting the mean strain verses the cycle number (Fig. 3.19), trends can be noted that indicate the presence of ratcheting in the connection, whereas the strain amplitudes indicates a cyclic hardening or softening response of the material. Upward or downward trends in the mean strain generally indicate ratcheting is occurring in the material being tested. Note in the third load step (cycles 14-41) in Fig. 3.19 that the mean strain is indicating a ratcheting response. By analyzing the global behavior (load-displacement plots) and the local behavior (mean strain and strain amplitude trends) of the connection correlations can be made between the local response and the global failure mode of the connection.
3.4 Tests on Smooth Plates of Base and Weld Metals

In order to determine the uniaxial material properties of the base metal, the heat affected zone, and the weld metal, a series of monotonic and cyclic tests will be performed on base metal, heat treated, and welded coupons. The coupons are 2.5” wide at the ends and 2” wide in the 5” gage length (see Fig. 3.20), and they are cut from the edges of the flange of a W18x55 rolled section which is 0.63” thick.
There are three distinct regions near the top and bottom beam flange complete joint penetration groove welds: unaffected base metal, the heat affected zone, and the weld itself. The heat affected zone is the region that experiences the extreme temperature changes associated with the welding procedure. In this region, the microstructure of the base metal is changed. There are five different regions inside the heat affected zone [Johnson, et. al, 2000]. The coarse grained heat affected zone is closest to the weld, and it experiences temperatures in excess of 1100°C. Next is the fine grained heat affected zone which reaches temperatures between 875°C and 1100°C. Next is the intercritical heat affected zone which is heated from 700°C to 875°C. The subcritical heat affected zone follows the intercritical heat affected zone, and it reaches temperatures between 575°C and 700°C. Finally, there is the base metal which never reaches 575°C. In order
to determine the material properties of the metal in this region, we will conduct uniaxial, monotonic and cyclic, tests on coupons that are conditioned to different temperature levels. This will aid in further understanding of the joint behavior.

![Fig. 3.21 – Welded coupon bevel dimensions](image)

In order to obtain the material properties of the weld metal, uniaxial tests were conducted on welded coupon specimens. These were fabricated by welding two pieces of the W18x55 beam flange together. The end of the flange to be welded was fabricated as shown in Fig. 3.21. One of the flange pieces was cut flat while the other piece had a double bevel machined on the end. This configuration is different than the geometry of the complete joint penetration weld between the beam flange and column flange in the welded steel moment connection. The flange was machined in this way to allow weld metal to be placed on each side of the specimen, thus minimizing the amount of bending in the coupon that is inherent with welding. The coupons were then cut and machined from the welded beam flange to the same dimensions as the other coupon specimens. Strain gages were placed in the center of the weld metal on each side of the coupon.
Averaging the strains from each side eliminates the effect of the initial bending of the specimen caused by welding. An extensometer will also be placed across the weld. Because the gage length of the extensometer is larger than the weld, the strains it records combine the weld behavior and a small portion of the heat affected zone. These results may prove beneficial in understanding the local behavior of the interface between the weld and the base metal in the connection.

3.4.1 Test Setup

A 200 kip hydraulic uniaxial material testing MTS machine was used to conduct all of the coupon tests. Fig. 3.22 shows the MTS machine with a specimen gripped and ready to be tested. The MTS machine is equipped with hydraulic wedge grips that are used for both monotonic and cyclic tensile and compressive loads. Once the specimen is mounted in the fixtures of the machine, an extensometer (also shown in Fig. 3.22) is placed at approximately the midheight of the specimen to measure axial elongation or shortening over the gage length (20 mm). These displacement values are then converted to strain values. The MTS machine has a built-in load cell that was used to measure the force applied on the specimen. These values are then converted to uniaxial stress by dividing by the cross-section area of the coupon.
In addition to the extensometer, some specimens were instrumented with strain gages. The gages are uniaxial with a gage length of 1/16”. When the gages were used, they were mounted at the midheight of the coupon in the center on both sides. By mounting gages on both sides of the coupon, the strains of the two gages can be averaged to report an average strain at the center of the cross-section and eliminate any bending present in the specimen. The strain gages were primarily used to validate the initial data given by the extensometer, and they were not used extensively because of the small strain.
range and strain shift errors when the strain amplitude exceeds 0.4%. In most of the coupon tests, the strain exceeded the range of the gage and the adhesive, and the gage failed or gave incorrect results. However, strain gages were used on all of the welded coupons at the center of the weld. The gage length of the extensometer is larger than the width of the weld, so gages are needed to record the weld metal response. Once again, gages were placed on both sides of the coupon to eliminate the bending strains in the specimen.

3.4.2 Loading Protocol

The cut out location of the coupons corresponds to the tensile coupon cut out location outlined in ASTM A370. The coupons will be labeled according to the type of test being performed as well as the temperature conditioning to which they were exposed. For example, specimen C-DC-500 indicates the type of specimen (C means coupon), DC indicates it was subjected to displacement controlled loading (similarly, M = monotonic loading and FC = force controlled loading), and the temperature it was exposed to was 500°C (similarly, Base = unconditioned, 625 = 625°C, 800 = 800°C, and 1050 = 1050°C). The specimens used to obtain the monotonic material properties were pulled at a rate of 0.1 in/min through yielding until initial strain hardening occurred, then they were pulled at 0.625 in/min until failure. The MTS machine used for this research was not equipped with strain-controlled loading capability, so displacement-controlled loading was used instead of the strain-controlled loading for determining the cyclic material responses.

To obtain the cyclic material properties, the specimens are tested in displacement control loading with limits of approximately +/- 1000 με up to +/- 10000 με. Each
specimen is gripped such that the grips are approximately even with the transition from the 2 ½” width to the 2” width in the specimen. Next, the distance between the grips is measured and displacement values are calculated corresponding to strain values of 0.1%, 0.2%, 0.3%, 0.5%, 0.7%, 0.9%, 1.0%, 1.1%, and 1.2% using this distance as the gage length. These displacements were then used as the amplitude for the displacement controlled cycles. For each specimen, one cycle was performed at each of the first two amplitudes. Then, two cycles were performed for each of the next two amplitudes. Finally, three cycles were performed at each of the remaining two amplitudes.

Force controlled tests were also performed on the coupon specimens. The purpose of these tests is to understand the ratcheting behavior in the base, the heat affected zone, and the weld metals. To begin with, the specimens were loaded until they reached the strain hardening region of the stress-strain curve in displacement controlled mode at a monotonic rate of 0.1 in./min. Once the specimen began hardening, it was unloaded to zero load. The mean force for each specimen was then calculated as 15% of its yield force. The amplitude was calculated as 90% of the observed load at yielding for each specimen. Each coupon was then cycled for 30 cycles in force controlled mode with the calculated mean and amplitude.

3.4.3 Basic Results

Each type of smooth plate test gives different information about the three regions present in the welded steel moment connection: unaffected base metal, the heat affected zone, and the weld metal. The monotonic tests give typical material properties such as the yield stress and strain, the ultimate stress, the ultimate strain, and an indication of
ductility. Fig. 3.23 shows the results of a typical monotonic test. The yield stress occurs at the stress-strain curve plateau where the strains increase while the stress remains relatively constant. The ultimate stress is the highest stress the specimen reaches before the stress begins to fall and considerable necking of the specimen occurs. The ultimate strain is the strain at rupture of the specimen, and it is not shown in the figure due to removing the extensometer prior to failure to avoid damage. The ductility of the specimen is proportional to the area under the stress-strain curve (the more area under the curve, the higher the ductility). Comparing these parameters for all of the specimens will aid in understanding the connection behavior and failure.

![Monotonic stress-strain response](image)

**Fig. 3.23 – Monotonic stress-strain response**

The displacement-controlled loading on the coupons yielded hysteresis loops when stress is plotted verses strain (see Fig. 3.24). The change in stress amplitude of the
loops, under constant displacement amplitudes, indicates cyclic hardening or softening of the material. If the strain amplitude increases, the material is cyclic hardening, and if they decrease, the material is cyclic softening. The strain amplitude of each hysteresis loop from each cycle for each specimen can also be compared to show trends in the material behavior.

![Figure 3.24](image-url)  
**Fig. 3.24 – Displacement-controlled stress-strain response**

The force-controlled tests were conducted to observe the ratcheting behavior for the base, the weld, and the heat affected zone metals. As the specimen undergoes force-controlled cycles with positive mean force, the hysteresis loops move to the right (the mean strain increases after each cycle). Fig. 3.25 shows such a response. The rate at which the loops move can determine the ratcheting behavior of the material. Once again,
the peaks from each loop in each cycle will be used to determine the mean and amplitude strains. When these values are plotted verses cycle number, the trends in both the mean and the amplitude can be used to determine the ratcheting response for the base, the weld, and the heat affected zone metals.

![Fig. 3.25 – Force-controlled stress-strain response](image-url)
CHAPTER 4
DISCUSSION OF RESULTS

4.1 Coupon Test Results

In order to obtain the material properties of the structural steel and the weld metal used in this study, multiple small coupon tests were performed. The monotonic material properties were obtained by loading the coupons uniaxially to failure, while the cyclic material properties were obtained using two different loading types: displacement-controlled and force-controlled. The displacement-controlled tests demonstrate strain hardening and/or softening of the material, and the force-controlled tests will be used to determine the ratcheting material properties in a later study.

In order to understand the local behavior near the complete joint penetration weld between the bottom beam flange and column flange in the welded steel moment connection, coupons from the unconditioned base metal will be tested as well as heat treated specimens. When welding the beam flanges to the column, high temperatures are generated in the base metal. These high temperature cycles cause a heat affected zone to form near the weld. There are five different regions in the heat affected zone which are differentiated by the microstructure changes that occur in the base metal as a result of welding [Johnson, et. al, 2000]. For this reason, we will test coupons conditioned to 500°C, 625 °C, 800 °C, and 1050 °C in addition to the unconditioned base metal. The temperatures chosen correspond to different regions of the heat affected zone. The heat treated coupons were heated in a large oven to the desired temperature and allowed to air cool to room temperature. Due to the temperature changes and the associated changes in microstructure, the monotonic and cyclic material properties change and affect the behavior of the welded steel moment connection.
In addition to the unconditioned and heat treated specimens, welded coupons were also tested. These specimens were fabricated by welding two pieces of the beam flange together. The coupons were then cut and machined from the edges of the welded flange. The welded coupons will also aid in determining the effect of the weld on the behavior of the welded steel moment connection.

4.1.1 Monotonic Coupon Test Results

Five coupons (one unconditioned specimen and one each conditioned to 500°C, 625 °C, 800 °C, and 1050 °C) were tested monotonically to determine the effect of different levels of temperature conditioning on their uniaxial material properties. The unconditioned specimen (C-M-Base) yielded between 52 and 53 ksi and displayed an ultimate tensile stress of approximately 67 ksi (see Fig. 4.1) which is typical for A992 grade 50 steel. The specimen also displayed a large amount of ductility as it failed at approximately 320000 µε. The ultimate strain is not shown in the figure because the extensometer was removed from the specimen to avoid damage when the specimen fails. The 500°C specimen (C-M-500) displayed similar behavior to specimen C-M-Base. The yield stress and the ultimate stress were approximately the same and the ductility was comparable for the two specimens. This supports the observations made by Johnson, et. al [SAC/BD-00/13, 2000] that the material does not change until the temperature reaches 575°C, the lower limit of the intercritical portion of the heat affected zone.

With each successive temperature, the yield stress and the ultimate stress dropped for each heat treated specimen (see Fig. 4.1). The yield stress and the ultimate stress for the 625°C specimen (C-M-625) was less than that of the C-M-500 at approximately 51
ksi and 66 ksi, respectively. The yield stress and ultimate stress for the 800ºC specimen (C-M-800) was less than that of C-M-625 at approximately 49 ksi and 65 ksi, respectively. Finally, the yield stress and the ultimate stress for the 1050ºC specimen (C-M-1050) is lower than that of C-M-800 at 39 ksi and 60 ksi, respectively. The difference between specimens C-M-800 and C-M-1050 is the largest of all the monotonic coupon specimens. Ductility remained roughly the same for all of the specimens, however. As stated, the extensometer was removed from all of the specimens prior to failure to prevent damage to the instrument. Based on these results, there is a significant loss in strength in the heat affected zone near the complete joint penetration weld. These local changes in material properties may affect the low-cycle fatigue performance of the connection near the weld.

![Stress-Strain Graph](image)

**Fig. 4.1 – Monotonic stress-strain behavior for the coupon specimens**
4.1.2 Displacement-Controlled Coupon Test Results

In order to determine the cyclic material properties of the steel near the weld, displacement-controlled cyclic coupon tests were conducted. Displacement-controlled constant amplitude cycles show upward or downward trends in the load and stress values when exposed to constant displacements. These trends can indicate strain hardening or softening of the material which can be important in understanding the local connection behavior during seismic or cyclic loading. The displacement-controlled tests can also display differences in behavior for the different regions near the complete joint penetration weld.

Once again, the unconditioned base metal specimen was tested first to provide comparisons with the heat treated specimens. Fig. 4.2 shows the hysteresis loops for the unconditioned specimen (C-DC-Base). The initial displacements applied to the specimen does not cause the specimen to exceed the yield stress of the material, therefore the response is linear elastic. Once the specimen yields, the behavior changes drastically. Unlike in the monotonic case where there is a pronounced yield plateau, there is no yield plateau once the specimen is exposed to significant load reversal. Another trait that was observed for all exposure temperatures was the stability in the displacement-controlled response. The material’s hysteresis loops stabilize and follow the same path as those before it after only a couple of cycles. This behavior indicates a material that is predictable when used in applications exposed to cyclic loading.

Fig. 4.3 shows the hysteresis loops for C-DC-1050. One significant difference in behavior between the heated specimen (C-DC-1050) and the unconditioned specimen (C-
DC-Base) is the lower peak stress of the heated coupon. This means that when the specimens are exposed to higher temperatures, they lose some of their strength and become softer. This softening could possibly lead to failures in the neighborhood next to the weld in the welded steel moment connection.

Fig. 4.2 – Unconditioned coupon response to displacement-controlled loading
Fig. 4.3 - 1050°C conditioned coupon response to displacement-controlled loading

Fig. 4.4 is a summary of the stress amplitudes for each cycle for each specimen. Only the amplitude for the first cycle at each displacement amplitude is shown. During the first four displacement amplitudes, all of the specimens displayed similar behavior as most of the behavior was linear elastic. During the next three displacement amplitudes, material behavior varied greatly as all of the specimens experienced large amounts of yielding. During the last two displacement amplitudes, the behavior stabilized quickly for all of the specimens. During the last amplitude, C-DC-500 experienced a large amount of buckling, so its results are not shown. However, it’s behavior up to this point was very similar to specimen C-DC-Base. As the temperature increased, stress amplitudes decreased for the displacement-controlled coupons.
4.1.3 Force-Controlled Coupon Test Results

In order to obtain the ratcheting parameters of the material near the weld, force-controlled tests were conducted. By limiting and controlling the load values, the strain response of the specimen can be isolated. For these tests, one coupon from the virgin steel will be tested along with three heat treated specimens. The conditioned coupons were exposed to temperatures of 625°C, 800°C, and 1050°C. Based on the results of the monotonic and displacement-controlled coupon tests, it was concluded that a 500°C specimen was not needed because the behavior exhibited in this temperature range is very similar to that of the unconditioned coupon specimens.
The initial portion of the force-controlled tests consisted of monotonic loading into the strain hardening region of the stress-strain curve. These initial results supported the findings in the monotonic tests: yield stress decreases as the exposure temperature increases. Following this initial phase of the test, the coupons were unloaded to a mean stress which was calculated based on the yield stress for each specimen. Once the mean stress was reached, the coupons were loaded in compression to the negative load peak (the mean stress minus the stress amplitude), and then the specimens were loaded back to the mean stress. Following these steps, the coupons were cycled in force-controlled loading with stress means and amplitudes based on the yield stress.

Each specimen exhibited positive ratcheting (strain accumulation in the tensile direction) during the force-controlled loading as expected. However, the rate of ratcheting, the amount of increase in the mean stress from one cycle to the next, for each specimen was different (see Fig. 4.5). The mean stress for specimen C-FC-625 increased at a faster rate than the mean stress for specimen C-FC-Base. The mean stresses for specimens C-FC-800 and C-FC-1050 both increased at a much slower rate than specimen C-FC-Base. Finally, specimen C-FC-1050 had the slowest rate of increase of all of the force-controlled coupons. Thus, the exposure temperature plays a role in the fatigue behavior and the rate of ratcheting in structural steel. As the exposure temperature increases, the yield stress decreases and the rate of ratcheting decreases.
4.1.4 Welded Coupon Results

In order to determine the material properties of the weld metal, welded coupon specimens were fabricated and tested. These specimens were made to resemble the geometry of the complete joint penetration groove weld between the beam flange and column flange. These coupons were cut from the same base metal as all of the other coupons and the beams used in the bottom flange tee joint specimens and the full-scale welded steel moment connection specimens.

The monotonic results for the welded coupons yielded unexpected results. Two methods were used to measure the strain in the weld: strain gages mounted at the center of the weld and an extensometer that extended across the weld. Two gages, one on each
side of the specimen, were averaged to eliminate the effects of bending in the specimens. During the tests, the weld metal yielded at approximately 55 ksi instead of the expected 70 ksi. Fig. 4.6 shows the stress-strain response of specimen C-M-W with the extensometer and strain gage results, and Fig. 4.7 shows the stress-strain response for the strain gages. This lower than expected yield stress may be acceptable in the case where the extensometer is used because it extends beyond the weld metal. The length covered by the extensometer spans the weld metal, the heat affected zone, and the base metal, which all have different material properties as shown by our previous experiments. However, the same results were not expected when looking at the small gage length of the strain gages which were mounted at the center of the weld.

![Monotonic stress-strain response of the extensometer and strain gage for specimen C-M-W](image)

**Fig. 4.6 – Monotonic stress-strain response of the extensometer and strain gage for specimen C-M-W**

94
This lower than expected value of the yield stress for the weld metal may be attributed to the geometry of the welded joint at the location of the strain gages (see Fig. 4.8). The cross-section of the coupon at the location of the strain gages consists primarily of weld metal, however there is a small portion of the base metal in the cross-section. This portion of the base metal will yield at a low stress because it was exposed to the highest temperatures during fabrication of the coupon. The yield stress for specimen C-M-1050 was much lower than that of the unconditioned base metal (C-M-Base). The cross-sectional area of the weld metal is smaller than the dimensions of the coupon, which was used to calculate the stresses, at the location of the strain gages. Thus, the
yield stress of the weld metal would appear lower than it actually is. For the same reasons, the displacement-controlled specimens and the force-controlled specimens will appear to yield at lower than expected stresses. The extensometer results may be useful, however, in explaining the behavior of the welded region (weld metal, heat affected zone, and base metal) as a whole.

When the monotonic specimen ruptured, it did not fail like the base metal and conditioned coupons had. The base metal coupons experienced significant necking at the middle of the coupon prior to failure. The welded coupon experienced necking in the regions above and below the weld (midway between the weld and the transition from wider ends to the gage width). We expected the specimen to crack adjacent to the weld, however it failed away from the weld in the base metal. This also leads us to believe that the weld metal is stronger than the monotonic results show.

Fig. 4.8 – Side view of the welded coupon’s welded connection
Fig. 4.9 shows the strain means and amplitudes for the gages on specimen C-W-FC. The welded coupon was exposed to the same loading routine as the base metal and heat treated coupons with stress means and amplitudes based on the yield stress. Fig. 4.10 shows the strain response of the extensometer which displays strains in the same range as those observed in the other force-controlled coupon tests. However, the weld metal experienced much smaller local strains when compared to the extensometer. The mean strain for the gages ranged from a little more than 2000 µε to a little more than 2500 µε, while the extensometer read mean strains from 8000 µε to 16000 µε. This could be caused by the difference in stiffness of the weld metal compared to the base metal and the heat affected zone. The stiffer weld metal experienced less strain than the softer material surrounding it.
Fig. 4.9 – Strain means and amplitudes (from average strain gage values) for C-W-FC
The displacement-controlled welded coupons gave more unexpected results. Once again, the strain gages and the extensometer gave significantly different results. Fig. 4.11 shows the strain response of the strain gages mounted in the center of the weld metal. The weld metal displayed similar stability to the base metal specimens, however the loops moved slightly towards the tensile strain direction with each cycle. This is a common characteristic of ratcheting. As the displacement amplitude increased for C-W-DC, a small crack began to form at the weld metal-base metal interface in the side of the coupon. This crack appeared to grow with each successive cycle. This behavior is consistent with failure mechanisms seen following the Northridge Earthquake and the SAC research. This crack could have formed because of the occurrence of ratcheting due to the cyclic loading that was not present in the monotonic specimen.

\[
\begin{align*}
\sigma_{xm} &= 46.0 \text{ ksi} \\
\sigma_{xa} &= 7.6 \text{ ksi}
\end{align*}
\]
4.2 Bottom Beam Flange Tee Joint Specimens

The bottom flange tee joint testing program consisted of four specimens which yielded eight sets of data on the local behavior near the complete joint penetration groove weld. The specimen name, T-CA-B-1A/B for example, denotes several variables that were tested. The first letter, T, refers to the type of test, which is the bottom beam flange tee joint test in this case. The second designation refers to the type of loading applied to the specimen: CA stands for constant displacement amplitude and MA stands for multiple displacement amplitude loading. The next designation stands for the welding sequence, A or B. “A” refers to the welding sequence in which the welder fills the groove on one side of the web with weld material before switching to the other side of the web to complete the weld. “B” refers to the welding sequence in which the welder alternates
sides of the web after placing each weld bead. This sequence fills the groove evenly from the top to the bottom as opposed to filling one side of the groove at a time (see Chapter 3 for details about welding sequence). All of the specimens in this program were fabricated using welding sequence “B” to isolate the effect of the loading protocol on the connection behavior. Specimens fabricated using sequence “A” will be tested later to determine the effect of the welding sequence on joint performance. The final designation in the specimen name is the specimen number (1-4) and the end of the specimen being examined (A or B) since each specimen has two different sets of strain data, one from each end.

After completing the test setup, the first bottom flange tee joint test, specimen T-MA-B-4A/B, was secured to the floor and spreader beam (see Chapter 3 for test setup details). After the test was started, several problems arose. First, applying the desired displacements accurately was very difficult. The loading was controlled by imputing displacements into the actuator controls. Due to all of the elastic movements of components in the test setup, the displacements seen by the specimen are much different than the displacements travelled by the actuators. The stiffness of the setup in compression was approximately twice the stiffness of the setup in tension. Because controlling displacements was difficult, it was nearly impossible to match the desired displacements and strain values from the finite element analysis that corresponded to the SAC loading protocol. Thus, it was decided to increase the displacements in a manner similar to the SAC loading protocol, but not to worry about matching the displacements exactly. We also decided to run more cycles at each displacement step to give a greater opportunity to identify the presence of ratcheting.
In this first test, we also discovered that the bottom flange tee joint fails due to local flange buckling prior to ratcheting or cracking near the complete joint penetration weld when the specimen is exposed to an increasing loading protocol. While local flange buckling is a local failure mechanism, it is a ductile failure mechanism. The intent of this research is to investigate the brittle failure of the connection seen during the Northridge Earthquake and the SAC research tests, so we decided to try constant displacement amplitude loading in the next two specimens. It was also decided to perform the tests at two different displacement amplitudes to see the effect of amplitude on the local behavior.

In addition to the lack of control experienced during the first test, the majority of the data from the test was lost due to a malfunction in the data acquisition system. Thus, we decided to test the final specimen using an increasing displacement amplitude loading similar to the SAC protocol. This would provide a complete set of data for the increasing amplitude protocol for comparison with the constant displacement amplitude tests.

4.2.1 Global Behavior of the Bottom Beam Flange Tee Joint Specimens

Previous experiments conducted on full-scale welded steel moment connections revealed two distinct modes of failure: ductile and brittle. Ductile failures typically involve buckling of the flanges, large displacements, and large amounts of energy dissipation. In general, brittle failures are initiated by the propagation of cracks that form near the complete joint penetration groove weld between the bottom beam flange and column flange. Little energy is dissipated prior to failure as the connection is exposed to
a small number of plastic cycles. These two failure modes were also observed in the bottom beam flange tee joint tests conducted in this research program.

The first test, T-MA-B-4A/B, was loaded using a multiple, increasing displacement amplitude loading protocol similar to the SAC protocol. As stated earlier, obtaining accurate displacements in the specimen was difficult due to the complexity of the setup, so the displacements and strains seen in the specimen do not correspond to those in the SAC loading protocol. We also ran more cycles in each loading phase than in the SAC protocol.

Fig. 4.12 shows the displacement amplitudes seen by T-MA-B-4A/B in the first test. This increasing displacement amplitude loading resembles the SAC loading protocol. Due to a data acquisition system malfunction on the second day of testing, the majority of the data in the middle of the test was lost. The first day of testing involved small, elastic cycles to get a feel for the control of the setup and to make sure the setup worked. During the second day, the specimen was cycled from small cycles in the elastic range to large cycles well into the plastic region of the response. On the third day, we cycled the specimen from small cycles until failure to get an idea about the global failure of the specimen, and to attempt to regain some of the lost data from the previous day.
Fig. 4.12 – Displacement history for T-MA-B-4A/B

This specimen failed in a ductile manner due to local flange buckling of the bottom beam flange. A crack initiated in the edge of the flange in the buckled region, and it propagated across the width if the flange until the specimen ruptured. This ductile behavior can be seen in Fig. 4.13. The large displacement values indicate large amounts of energy dissipation, a characteristic of ductile failure mechanisms. The drop in loading values is indicative of a loss in section area due to propagation of the crack in the flange. Prior to failure, the web yielded first during phase 3 of the loading (see Fig. 4.12). Initial yielding first occurred in the flange during phase 4. With each subsequent cycle and phase, yielding in both the flange and web continued. During the compression stage of phase 8, the flange began to buckle locally near the complete joint penetration groove.
weld (see Fig. 4.14). With each successive increase in displacement, the lateral
displacement of the flange continued to grow in the buckled region. Finally, the crack
formed in the buckled portion of the flange away from the weld. The crack propagated
until the specimen failed in a predictable manner (see Fig. 4.15).

Fig. 4.13 – Load verses displacement relationship for T-MA-B-4A/B
Fig. 4.14 – Local buckling of the flange in specimen T-MA-B-4A/B

Fig. 4.15 – Fractured beam in specimen T-MA-B-4A/B
In the second test, specimen T-CA-B-1A/B, was exposed to a constant
displacement amplitude loading protocol. In the first experiment, we observed that
increasing from small displacements to large displacements caused the flange to buckle
locally leading to a ductile failure. Castiglioni [2005] observed ductile failure
mechanisms at high displacement constant amplitude tests and brittle failure mechanisms,
cracks near the weld and small amounts of energy dissipation, at smaller displacement
constant amplitude tests. So, we decided to employ a small, constant displacement
amplitude loading protocol during the second experiment.

Fig. 4.16 shows the displacement history of specimen T-CA-B-1A/B. Phases 1
and 2 were used to make sure the specimen was mounted in the test setup properly and to
gain a feel for the control of the loading. The displacement amplitude was increased in
small increments until a desirable displacement amplitude was reached and to avoid
overloading the specimen. Because the control of the specimen was not exact, we chose
a range of displacement amplitudes we felt would give the desired response. During
phase 2, slight yielding occurred in the flange and web of the bottom beam flange tee
joint specimen. Phase 3 was the constant amplitude portion of the experiment with
amplitudes of +/- 0.1 in. The purpose of this portion of the loading is to determine
whether or not ratcheting is present and to see if any cracking occurs in the specimen.
During this phase, the web buckled, while the flange remained relatively straight. Cracks
were observed near the complete joint penetration groove weld along the base metal-weld
metal interface. The cracks were visible at the edges of the flanges and the outside of the
beam flange (see Fig. 4.17 and Fig. 4.18).
Fig. 4.16 – Displacement history of specimen T-CA-B-1A/B
Fig. 4.17 – Cracks near the complete joint penetration weld in the beam flange in specimen T-CA-B-1A/B

Fig. 4.18 – Close up view of the crack at the edge of the beam flange
During phase 4, the web continued to buckle and eventually ruptured, while the flange remained relatively straight with only slight local lateral movements. Once the web ruptured, we decided to begin increasing the displacement amplitude in the tensile direction while remaining fairly constant in the compressive direction. The purpose of this loading was to propagate the cracks that formed during phase 3 and cause brittle failure without buckling the flange and causing a ductile response. In Fig. 4.19, the ruptured web is evident with the drop in the loading value. In phases 5, 6, 7, and 8, we increased the tensile displacement amplitude until the specimen failed. Specimen T-CA-B-1A/B failed suddenly and without warning with a slight increase in the tensile displacement amplitude following phase 8. The cracks that formed near the weld propagated suddenly across the width of the flange causing a brittle fracture to occur. Fig. 4.20 shows the failed specimen. Another indication of a brittle failure can be seen in Fig. 4.19. The hysteresis loops continue to have positive slopes at the tensile load peaks. These upward slopes mean that the specimen that the specimen still has strength carrying capabilities beyond the applied load. Flat, or downward, slopes indicate a loss of strength and can predict an eminent failure, a characteristic of ductile behavior. Notice the slope in Fig. 4.13 in the compressive peaks. These predict the inevitable failure of the specimen, whereas T-CA-B-1A/B (Fig. 4.19) shows no indication of failure until it occurs.
Fig. 4.19 – Load verses displacement relationship for T-CA-B-1A/B

Fig. 4.20 – Fractured beam in specimen T-CA-B-1A/B
Specimen T-CA-B-1A/B exhibited some ratcheting or strain softening during phase 3 of the loading, the constant amplitude portion of the test. Fig. 4.21 shows the load-displacement response of the specimen during the third phase. The specimen was exposed to constant amplitude displacement loading of approximately +/-0.1 in. During this portion of the test, the loading values decreased with each successive cycle. This behavior could have some affect on the low cycle fatigue behavior of the connection.

Fig. 4.21 – Load verses displacement relationship for phase 3
for specimen T-CA-B-1A/B
In the third bottom flange tee joint test, specimen T-CA-B-2A/B was loaded with another constant displacement amplitude loading protocol. The amplitude of the loading for this experiment was smaller than the previous constant displacement amplitude test (T-CA-B-1A/B) at +/- 0.08 in. compared to +/- 0.1 in. The purpose of this amplitude was to determine the effect of the smaller amplitude on the performance of the connection and to see if the smaller amplitude causes ratcheting. Fig. 4.22 shows the displacement history of test 3. Phases 1-4 of the loading ensure the specimen is secure in the test frame and determine the level of control of the specimen. Phase 5 is the constant amplitude portion of the test. Once phase 5 was completed, phases 6-8 involve one big cycle followed by 6-7 constant amplitude cycles from +/- 0.08 in. The amplitude of the big cycle at the start of phases 6-8 was increased in each phase. Phases 9-14 were alternating cycles of high amplitude cyclic tensile displacements (phases 9, 11, and 13) and constant displacement amplitude cycles (phases 10, 12, and 14). The purpose of phases 6-14 were to cause damage (cracks or fractures) during the initial high amplitude cycle (first cycle of phases 6-8) or the high tensile displacement cycles (phases 9, 11, and 13) and observe the damage propagation in the following constant displacement amplitude cycles (constant displacement portion of phases 6-8 and phases 10, 12, and 14). Finally, the specimen was pulled monotonically to failure. Fig. 4.23 shows the load-displacement response of the test.
As with the two previous tests, the web yielded first, followed by slight yielding of the flange during phase 2. The flange and web yielded more during each successive phase. During phase 5 (the constant displacement amplitude portion of the test), the specimen did not display the same behavior as the previous specimen had demonstrated. The hysteresis loops did not move, and the specimen did not appear to harden or soften as the cycles continued (see Fig. 4.24). Thus, the slight change in displacement amplitude from 0.1 inches to 0.08 inches affected the behavior of the specimen. In the latter phases, the flange began to buckle, so the compressive displacements were limited in an attempt to induce a brittle failure as with T-CA-B-1A/B. During phase 7, cracks formed in the edges of the flange near the weld metal-base metal interface. These cracks are shown in
Cracks also formed in the web in the weld access hole. During phase 8, cracks formed in the center of the flange on the outside of the beam at the weld metal-base metal interface. The same cracking pattern observed in specimen T-CA-B-1A/B occurred in specimen T-CA-B-2A/B. The cracks did not propagate as expected during phases 9, 11, and 13, however. It appears as if the reversal of loading aids in crack propagation. Finally, the specimen was pulled to failure monotonically. The flange ruptured next to the complete joint penetration groove weld as seen in Fig. 4.25.

Fig. 4.25 – Load verses displacement relationship

For specimen T-CA-B-2A/B
Fig. 4.24 – Load verses displacement relationship for phase 5 for specimen T-CA-B-2A/B

Fig. 4.25 – Failure of specimen T-CA-B-2A/B

Cracks at the weld metal-base metal interface
The fourth bottom flange tee joint specimen, T-MA-B-3A/B, was loaded using a combination of an increasing displacement amplitude loading protocol and a constant displacement amplitude loading protocol (see Fig. 4.26). The first 5 phases of the test involved loading the specimen with an increasing displacement amplitude to provide some more information on the behavior of the specimen when loaded in this manner in addition to the results given by specimen T-MA-B-4A/B. It also provides more correlation between our results and the results of past full-scale welded steel moment connection tests loaded using the SAC loading protocol. Phases 6-9 resemble the latter loading of specimen T-CA-B-2A/B with one large cycle followed by six smaller constant displacement amplitude cycles. The first cycle of each phase increased in amplitude with each phase. Once again, the theory was to create some damage during the initial large cycle and propagate that damage during the subsequent constant displacement amplitude portion of the loading phase.
As in the other tests, the web yielded first followed by slight yielding of the flange in phases 3 and 4. During phase 5, the web began to buckle. During phase 6, the flange began buckling (see Fig. 4.27). In the first cycle of phase 8, the flange buckled more severely. No cracking was observed near the weld in any of the phases of loading. Because the flange continued to buckle, the specimen was exhibiting ductile behavior and the test was stopped. Fig. 4.28 shows the load-displacement relationship for T-MA-B-3A/B.
Fig. 4.27 – Specimen T-MA-B-3A/B flange buckling

Fig. 4.28 – Load verses displacement relationship for T-MA-B-3A/B
4.2.2 Local Behavior of the Bottom Beam Flange Tee Joint Specimens

Strain is defined as the geometrical deformation caused by the action of stress on a physical body. It is calculated by examining the change between two body states: the initial state and the final state. The mathematical definition is the change in length (final length minus initial length) divided by the initial length. Thus, theoretically, the strain is only dependent on the displacement. However, in piping tests conducted at NCSU [Lu, 2003], the measured strain near the welded joints accumulated over time during constant displacement amplitude tests. This phenomenon of strain accumulation is known as ratcheting.

The primary focus of this research is the local behavior of the bottom beam flange to column flange connection. One objective, in particular, was to determine whether or not ratcheting is present in the connection and its effect on local behavior. In order to monitor this behavior, nine strain gages were placed along the width of the flange at each complete joint penetration groove weld in the bottom flange tee joint specimens.

The strains recorded by these gages yield hysteresis loops when strain is plotted verses load similar to the load-displacement hysteresis loops used to evaluate global behavior of the specimen. By imputing these load-strain relationships into a “peak-picking” computer program, we were able to determine the maximum and minimum strains for each cycle. These values are then used to calculate the mean strain and the strain amplitude for each cycle (Eqn. 1 and Eqn. 2). By plotting the mean strain and strain amplitude verses cycle, we can determine whether or not any local phenomenon, such as ratcheting, may have been present in the connection.
\[
\varepsilon_{\text{mean}} = \frac{\varepsilon_1 + \varepsilon_2}{2} \quad \text{(Eqn. 1)}
\]

\[
\varepsilon_{\text{amplitude}} = \frac{|\varepsilon_1 - \varepsilon_2|}{2} \quad \text{(Eqn. 2)}
\]

Positive ratcheting is characterized by increasing mean strain. In a uniaxial loading situation, the hysteresis loops gradually move towards the tensile strain direction, and the specimen gradually gets longer. Negative ratcheting is characterized by decreasing mean strain on the other hand. The hysteresis loops gradually move towards the compressive strain direction, and the specimen gets shorter.

Near the complete joint penetration weld in the beam-column joint, multiaxial stresses are present instead of uniaxial stresses. This is due to interactions between residual stress and stress due to applied loading that combine to create complex stresses in different directions that are not well understood. Thus, ratcheting is difficult to quantify in the welded steel moment connection.

Ratcheting can also be described as damage accumulation. This damage accumulation can lead to material degradation which could induce cyclic softening. Positive ratcheting (tensile direction ratcheting) may initiate fatigue cracking, whereas negative ratcheting, compressive direction ratcheting, may initiate buckling. Thus, it is important to understand ratcheting to determine its effect on the performance of the welded steel moment connection.

During the first bottom beam flange tee joint test (specimen T-MA-B-4A/B), we experienced a malfunction in the data acquisition system that caused most of the strain data to be lost. Thus, this test was primarily used to evaluate global behavior and for experimental guidance for the subsequent tests.
Despite the malfunction in the first test, we observed ratcheting in all three subsequent bottom flange tee joint tests. This behavior was most pronounced in the constant displacement amplitude experiments, however it was also seen in the multiple, or increasing, displacement amplitude tests. The second (T-CA-B-1A/B) and third (T-CA-B-2A/B) tests were both constant amplitude tests, and ratcheting was observed in each.

During the second experiment (T-CA-B-1A/B), ratcheting was observed during the constant displacement portion of the loading from cycle 14 to cycle 41 in phase 3 in Fig. 4.16. The displacement amplitude during this sequence was approximately +/- 0.1 inches. Location played a role in the presence of ratcheting during this test. The center strain gage on the outside of the flange (see Fig. 4.28 for T-CA-B-1A SG-R1-O-4 and Fig 4.29 for T-CA-B-1B SG-R1-O-4) and the strain gages at the edges of the flange nearest to the weld displayed the most pronounced ratcheting behavior (see Fig. 4.30 for T-CA-B-1B SG-R1-I-1). The gages located further from the weld did not display the ratcheting behavior seen closer to the weld. This could be due to the residual stresses present near the weld.
Fig. 4.29 – Strain means and amplitudes for T-CA-B-1A SG-R1-O-4

Fig. 4.30 – Strain means and amplitudes for T-CA-B-1B SG-R1-O-4
The third test (T-CA-B-2A/B) also displayed ratcheting during the constant
displacement portion of the loading from cycle 27 to cycle 50 in phase 5 in Fig. 4.22.
The loading amplitude during this segment of the experiment was approximately +/- 0.08
inches. Ratcheting was present in the same locations as in the prior test. However, in
this test, it was more pronounced at the edges of the flange near the weld rather than at
the center of the flange near the weld. Once again, there was no ratcheting away from the
weld, which can be attributed to the lack of residual stress away from the complete joint
penetration weld. The ratcheting is shown in Fig. 4.31 and Fig. 4.32 by the increase in
strain amplitude while the mean stress remains relatively constant between cycles 27 and
50. The ratcheting in this test was not as pronounced as in the prior experiment because
of the smaller displacement amplitudes applied to the specimen.
Fig. 4.32 – Strain means and amplitudes for T-CA-B-2A SG-R1-I-5

Fig. 4.33 – Strain means and amplitudes for T-CA-B-2A SG-R1-I-7
During the final experiment (T-MA-B-3A/B), a multiple, or increasing, displacement amplitude was employed to provide some comparison to full-scale welded steel moment connection tests loaded with the SAC loading protocol. The SAC loading protocol calls for six cycles during the first three amplitudes, four cycles during the next constant amplitude, and two cycles for each additional amplitude. We chose to run each displacement amplitude for 6-8 cycles to determine if ratcheting was present or not.

During phases five and six of the loading (see Fig. 4.26), we observed ratcheting in the same locations as in the two previous tests: the center and edges of the flange near the weld. Fig. 4.33 shows the strain means and amplitudes for gage T-MA-B-3A SG-R1-O-4, and Fig. 4.34 show the strain means and amplitudes for T-MA-B-3A/B SG-R1-O-1. Fig. 4.33 shows ratcheting in phases 5-9 (cycle 29 to cycle 71), while Fig. 4.34 shows ratcheting in phases 5 and 6 (cycle 29 to cycle 51). The amplitudes during these phases was between +/- 0.08 and 0.1 inches, the same as the amplitudes used in the constant amplitude portion of the previous tests that demonstrated ratcheting while exposed to constant displacement amplitude loading.

All of the results for all of the bottom flange tee joint specimens are presented in the Appendix.
Fig. 4.34 – Strain means and amplitudes for T-MA-B-3A SG-R1-O-4

Fig. 4.35 - Strain means and amplitudes for T-MA-B-3A SG-R1-O-1
5.1 Observations

The goal of this research was to gain an understanding of the local behavior of the welded steel moment connection. In order to accomplish this goal, numerous experiments and analyses were performed. Specimens ranging from small tensile coupons to large scale bottom flange tee joint assemblies were subjected to varying loading schemes to better understand the local effect of the complete joint penetration weld between the beam flange and the column flange on the global behavior of the connection. Some of the coupons were exposed to high temperatures to gain insight into differences in material properties between the base metal and the heat affected zone adjacent to the weld. Some of the coupons were welded to better understand the behavior of the weld metal as well as the behavior of the steel adjacent to the weld in a simple specimen. The bottom flange tee joints were fabricated with the intent of replicating the behavior of the full-scale welded steel moment connection by using a smaller specimen subjected to loading that is less complex than that seen in a typical moment connection experiment. They also provided two sets of local data for the critical bottom beam flange to column flange complete joint penetration groove weld that showed problems during the Northridge Earthquake and subsequent research as opposed to one set of data from typical full-scale moment connection tests.

The coupon tests offered some insights into the behavior of the welded steel moment connection. The monotonically loaded base metal and heat treated coupons showed trends in the yield stress, ultimate stress, and ductility of the material as exposure temperature increases. As expected, the yield stress decreases as exposure temperature
increases. There was a significant drop in yield stress from the unconditioned specimen (C-M-Base) and the coupon exposed to 1050ºC (C-M-1050). It dropped from approximately 53 ksi to 39 ksi. The ultimate stress also decreased from approximately 67 ksi to 60 ksi. There was little change in the ductility of the specimens as a result of the heat treatment.

The displacement-controlled coupons displayed similar results as the exposure temperature increased. The yield stress and the ultimate stress for these specimens decreased as the exposure temperature increased in most cases. The exception to the rule was C-DC-625. It displayed lower strain amplitudes than the other specimens at all displacement amplitudes once yielding occurred. Another behavioral characteristic displayed by the displacement-controlled coupons was stability in behavior when exposed to cyclic loading. The material exhibited stable hysteresis loops at all displacement levels and showed no signs of strain hardening or softening.

The force-controlled coupons were used to observe and determine the ratcheting, or strain accumulation, parameters for the base material with different levels of temperature exposure. They also reinforced the results from the monotonic tests that yield stress decreases as temperature increases. A general observation from these tests was that the rate at which the specimen ratchets, or accumulates strain, decreased as the exposure temperature increased. The exact ratcheting parameters will be determined in later research.

Of all the coupon tests, the welded coupons yielded perhaps the most surprising and interesting results. First, the monotonic specimen did not yield at the anticipated yield stress, and the specimen failed in an unexpected location. Instead of necking near
the weld in the center of the coupon or fracturing in the base metal next to the weld, the
coupon necked on both sides of the weld (midway between the weld and the grips) and
ruptured in the base metal away from the weld. This leads me to believe that the weld
metal was indeed stronger than the base metal, and the startling data was a result of
instrumentation problems instead of material defects. The results may however be useful
in understanding the region adjacent to the weld. The displacement-controlled welded
coupon exhibited the same stability in the weld metal and the area surrounding the weld
as the base metal coupons had. One interesting observation was the weld metal’s
tendency for the mean stress to increase towards the tensile direction with each increase
in displacement amplitude, whereas the base metal and the heat treated coupons’ mean
stress remained close to zero. Finally, the force-controlled welded coupon displayed
ratcheting behavior just as the base metal coupons had shown. However, the weld
metal’s ratcheting rate (determined by the readings from the strain gages mounted on the
weld) was much slower than the ratcheting rate of the weld metal, heat affected zone, and
base metal region (determined by the extensometer that extended beyond the weld metal).
This behavior may be why the connections have been failing in the region adjacent to the
complete joint penetration weld.

During the bottom flange tee joint tests, two distinct failure modes were observed:
ductile failure and brittle failure. The increasing displacement amplitude tests (specimens
T-MA-B-4A/B and T-MA-B-1A/B) failed in a ductile manner characterized by local
buckling of the flange. Once the local buckling became severe, cracks formed in the
flange and propagated predictably with each cycle until the flange ruptured. The constant
displacement amplitude tests (specimens T-CA-B-1A/B and T-CA-B-2A/B) failed in a
brittle manner. These failures were characterized by the formation of small cracks along the weld metal-base metal interface. The cracks in specimen T-CA-B-1A/B propagated slowly until the specimen ruptures suddenly and without warning with a slight increase in the loading amplitude. Cracks formed in the same areas in specimen T-CA-B-2A/B. However, this specimen did not fail in the same manner as specimen T-CA-B-1A/B because we decided to pull it to failure monotonically prior to a brittle fracture occurring. Castiglioni [2003] observed similar results in his research. He witnessed ductile failures during constant displacement amplitude loading when the amplitudes were large and brittle failures when the amplitudes were small. In our tests, the multiple amplitude loading reached displacements that were large enough to cause ductile failures before brittle failures could occur. However, in our constant displacement amplitude tests, the small amplitudes were applied for enough cycles for local damage to initiate and propagate, thus a brittle failure occurred.

In addition to the global failure mechanisms observed in the bottom flange tee joint tests, local behavior that may contribute to the failure of the connection was observed. Ratcheting, a strain accumulation phenomenon was observed in three of the test specimens. It was more pronounced and easier to observe when constant displacement amplitudes were applied to the connections, however it was observed during multiple or increasing amplitude tests as well. Ratcheting has been shown to decrease fatigue life of welded piping joints [Lu, 2003], thus it may have the same detrimental effect in welded steel moment connections.
5.2 Recommendations

While all of the results of this research have indicated that welding and the associated high temperatures affect the performance of the welded steel moment connection, there is not enough evidence to draw any concrete conclusions that warrant any changes to the design or fabrication of the joint. There are just too many variables that affect the performance of the connection that could not be examined by this research. However, I would recommend testing of bottom flange tee joint specimens that are fabricated with a different welding procedure, namely the other procedure mentioned in chapters 1 and 3. These tests would need to be performed at the same displacement amplitude cycles to isolate the welding procedure as the primary variable being examined.

It was shown in this research that the loading protocol affects the behavior and failure mechanism of the bottom beam flange tee joint specimens. Thus, multiple or increasing displacement amplitude cycle loading protocols are not as beneficiary in determining the low cycle fatigue behavior of the connection.

Another shortcoming of this testing program was the inaccurate measurement of the strength of the weld metal. In order to determine the true material properties of the weld metal, pure specimens made of just the weld metal need to be fabricated and tested. The complex geometry of the welded specimens do not give true material properties of just the weld metal. Our results are a combination of the behavior of the base metal, the heat affected zone, and the weld metal. These results may be important in understanding the behavior in the region near the complete joint penetration weld in the welded steel
moment connection. However, the material properties of the weld metal itself would be beneficial.

In conclusion, ductile failures and brittle failures of the bottom beam flange to column flange joint were observed during this testing program. In addition to witnessing the different global failure mechanisms, various local parameters that affect the behavior of the connection were recorded. High temperatures introduced by welding the beam flange to the column flange change the local material characteristics of the base metal. These high temperature cycles also cause large internal and residual stresses in the connection that affect its fatigue life. Finally, ratcheting was observed in the connection. This behavioral trait is not well understood and should be examined more closely to determine its influence on the low cycle fatigue life of the welded steel moment connection.

5.3 Conclusions

This research makes an attempt to observe multiple variables and their effect on the behavior of the bottom beam flange to column flange connection. The first variable that could play a role in the fatigue life of the joint is the welding procedure. However, the welding sequence used on all of the bottom beam flange tee joint specimens was the same in this testing program. This is one variable that needs to be further investigated in future work by testing specimens fabricated using different sequences.

Another variable in these experiments was the weld bead size, which was different for every specimen. Specimen T-MA-B-1A/B was fabricated first and had a larger number of small weld beads when compared to specimen T-MA-B-4A/B which had a smaller
number of larger weld beads. Specimens T-CA-B-2A/B and T-CA-B-3A/B were in the middle when it came to weld bead size. In all of the experiments, weld bead size played an insignificant role in the behavior and fatigue life of the specimen.

The one variable that made a significant difference in the behavior of the bottom flange tee joint specimens was the loading protocol. The specimens loaded using the increasing displacement amplitude loading (similar to the SAC loading protocol) all failed in a ductile manner beginning with buckling of the beam flange. The specimens loaded using the constant displacement amplitude loading exhibiting cracking near the complete joint penetration weld between the beam flange and column flange and failed in a brittle manner. This is similar to the observations made by Castiglioni [2005]. Despite the differences in global behavior as a result of the loading protocol, ratcheting was observed in all of the bottom beam flange tee joint specimens. The observed ratcheting was most prominent in the center of the flange and at the edges of the flange in the row of gages closest to the weld. None of the observed ratcheting and strain accumulation were present in the finite element model results. This is a shortcoming of the analysis techniques used.

The coupon tests also yielded valuable information related to the behavior of the bottom beam flange to column flange connection. From the monotonic tests, we see that the yield stress decreases as exposure temperature increases. This demonstrates the changes in material behavior in the heat affected zone adjacent to the weld. The displacement-controlled and force-controlled coupons also demonstrated differences in the material behavior as temperature increased. The displacement-controlled specimens
reached lower levels of stress as the temperature increased at the same displacements. As the temperature increased, the rate of ratcheting slowed for the force-controlled coupons.

Our initial attempt at modeling the full-scale welded steel moment connection and using the results as experimental guidance did not work well. This was primarily due to the lack of control of the test setup.
References


Johnson, Matt Q., Bill Mohr, and John Barsom. “Evaluation of Mechanical Properties in


This Appendix contains the complete results from all the bottom beam flange tee joint specimens. Because the results for each specimen were similar, they were not all presented in the text. A loading history, load-displacement plots, and strain mean and amplitude plots are given for each specimen.

T-CA-B-1A/B .................................................................................................................141
T-CA-B-2A/B .................................................................................................................156
T-CA-B-3A/B .................................................................................................................176
T-CA-B-4A/B .................................................................................................................193
Specimen T-CA-B-1A/B was the second of four bottom beam flange tee joint specimens that were tested for this research. Phases 1 and 2 consisted of elastic cycles to ensure that the test setup and the instrumentation was functioning properly. The third phase was the constant amplitude portion of the loading. The loading amplitude during this phase was +/- 0.1 inches. Loading phases 4, 5, 6, 7, and 8 consisted of increasing amplitudes in the tensile direction. The purpose of this loading was to prevent the flange from buckling as the web had already ruptured and force the failure to occur near the weld. During the third loading phase, ratcheting was observed at the locations of the gages closest to the weld.

During phase 1 and 2, the specimen displayed mostly elastic behavior except for some yielding (observed by flaking of the white wash in areas of inelastic behavior) occurred in the web. During phase 3, the flange began to yield. With each successive increase in loading amplitude, the flange yielded more until failure. Cracks were first observed near the weld at the edge of the beam flange during phase 5. More cracks initiated during phases 6 and 7 at the edges of the flange and at the center of the beam flange. Finally, the specimen failed in an unpredictable, brittle failure.

Following is the displacement history, load-displacement graphs, and strain mean and amplitude plots.
Fig. A.1 – Displacement history for T-CA-B-1A/B
Fig. A.2 – Load-displacement response for T-CA-B-1A/B

Fig. A.3 – Phase 1 load-displacement response for T-CA-B-1A/B
Fig. A.4 – Phase 2 load-displacement response for T-CA-B-1A/B

Fig. A.5 – Phase 3 load-displacement response for T-CA-B-1A/B
Fig. A.6 – Phase 4 load-displacement response for T-CA-B-1A/B

Fig. A.7 – Phase 5 load-displacement response for T-CA-B-1A/B
Fig. A.8 – Phase 6 load-displacement response for T-CA-B-1A/B

Fig. A.9 – Phase 7 load-displacement response for T-CA-B-1A/B
Fig. A.10 – Phase 8 load-displacement response for T-CA-B-1A/B

Fig. A.11 – Displacement amplitudes and means for T-CA-B-1A/B
Fig. A.12 – Load amplitudes and means for T-CA-B-1A/B

Fig. A.13 – Strain amplitudes and means for T-CA-B-1A SG-R1-O-7
Fig. A.14 – Strain amplitudes and means for T-CA-B-1A SG-R3-O-4

Fig. A.15 – Strain amplitudes and means for T-CA-B-1A SG-R1-I-3
Fig. A.16 – Strain amplitudes and means for T-CA-B-1A SG-R1-O-4

Fig. A.17 – Strain amplitudes and means for T-CA-B-1A SG-R2-O-4
Fig. A.18 – Strain amplitudes and means for T-CA-B-1A SG-R1-O-1

Fig. A.19 – Strain amplitudes and means for T-CA-B-1B SG-R1-O-6
Fig. A.20 – Strain amplitudes and means for T-CA-B-1B SG-R1-O-7

Fig. A.21 – Strain amplitudes and means for T-CA-B-1B SG-R1-O-4
Fig. A.22 – Strain amplitudes and means for T-CA-B-1B SG-R2-O-4

Fig. A.23 – Strain amplitudes and means for T-CA-B-1B SG-R1-I-1
Fig. A.24 – Strain amplitudes and means for T-CA-B-1B SG-R3-O-4

Fig. A.25 – Strain amplitudes and means for T-CA-B-1B SG-R1-I-5
Fig. A.26 – Average strain amplitudes and means for T-CA-B-1A SG-R1-O-1 and SG-R1-I-1
Specimen T-CA-B-2A/B was the third of the four bottom beam flange tee joint specimens that were tested for this research. Loading phases 1 and 2 displayed mostly elastic behavior. During phases 3 and 4, the web and the flange began yielding slightly. Phase 5 was the constant amplitude portion of the loading. The displacement amplitude was +/- 0.08 inches, and ratcheting was observed in the strain gage locations closest to the weld. Yielding in the flange and the web continued during this loading phase. Phases 6, 7, and 8 all began with a large displacement amplitude cycle followed by constant amplitude cycles of 0.08 inches. The initial large displacement cycle during these phases increased with each subsequent phase. The reason for this loading was to cause some damage with the large cycle and propagate the damage with the constant amplitude cycles.

Phases 9-14 alternated between one phase of high cycles in the tensile direction followed by a phase of tensile and compressive cycles. This loading prevented the specimen from buckling and failing in a brittle manner. It also attempted to cause some damage with the large tensile cycles and propagate the damage during the constant displacement amplitude cycles. Finally, the specimen was pulled to failure monotonically.

Cracks were first observed at the edges of the flange next to the weld during phase 7. More cracks were observed during phase 8 at both edges of the flange and at the center of the flange next to the weld. The monotonic loading was an attempt to force the cracks to propagate and cause the specimen to fail, however the base metal ruptured in the weld access hole away from the weld. Following is the displacement history, load-displacement graphs, and strain mean and amplitude plots.
T-CA-B-2A/B Displacement History

Fig. A.27 – Displacement History for T-CA-B-2A/B
Fig. A.28 – Load-displacement response for T-CA-B-2A/B

Fig. A.29 – Phase 1 load-displacement response for T-CA-B-2A/B
Fig. A.30 – Phase 2 load-displacement response for T-CA-B-2A/B

Fig. A.31 – Phase 3 load-displacement response for T-CA-B-2A/B
Fig. A.32 – Phase 4 load-displacement response for T-CA-B-2A/B

Fig. A.33 – Phase 5 load-displacement response for T-CA-B-2A/B
Fig. A.34 – Phase 6 load-displacement response for T-CA-B-2A/B

Fig. A.35 – Phase 7 load-displacement response for T-CA-B-2A/B
Fig. A.36 – Phase 8 load-displacement response for T-CA-B-2A/B

Fig. A.37 – Phase 9 load-displacement response for T-CA-B-2A/B
Fig. A.38 – Phase 10 load-displacement response for T-CA-B-2A/B

Fig. A.39 – Phase 11 load-displacement response for T-CA-B-2A/B
Fig. A.40 – Phase 12 load-displacement response for T-CA-B-2A/B

Fig. A.41 – Phase 13 load-displacement response for T-CA-B-2A/B
Fig. A.42 – Phase 14 load-displacement response for T-CA-B-2A/B

Fig. A.43 – Displacement amplitudes and means for T-CA-B-2A/B
Fig. A.44 – Load amplitudes and means for T-CA-B-2A/B

Fig. A.45 – Strain amplitudes and means for T-CA-B-2B SG-R3-O-4
Fig. A.46 – Strain amplitudes and means for T-CA-B-2B SG-R1-I-1

Fig. A.47 – Strain amplitudes and means for T-CA-B-2B SG-R1-I-7
Fig. A.48 – Strain amplitudes and means for T-CA-B-2B SG-R1-I-5

Fig. A.49 – Strain amplitudes and means for T-CA-B-2B SG-R1-O-7
Fig. A.50 – Strain amplitudes and means for T-CA-B-2A SG-R1-I-5

Fig. A.51 – Strain amplitudes and means for T-CA-B-2A SG-R1-I-7
Fig. A.52 – Strain amplitudes and means for T-CA-B-2B SG-R1-O-2

Fig. A.53 – Strain amplitudes and means for T-CA-B-2B SG-R1-O-4
Fig. A.54 – Strain amplitudes and means for T-CA-B-2B SG-R1-O-1

Fig. A.55 – Strain amplitudes and means for T-CA-B-2B SG-R2-O-4
Fig. A.56 – Strain amplitudes and means for T-CA-B-2A SG-R3-O-4

Fig. A.57 – Strain amplitudes and means for T-CA-B-2A SG-R1-O-7
Fig. A.58 – Strain amplitudes and means for T-CA-B-2A SG-R1-O-4

Fig. A.59 – Strain amplitudes and means for T-CA-B-2A SG-R1-O-1
Fig. A.60 – Strain amplitudes and means for T-CA-B-2A SG-R1-O-2

Fig. A.61 – Strain amplitudes and means for T-CA-B-2A SG-R1-I-1
Fig. A.62 – Strain amplitudes and means for T-CA-B-2A SG-R2-O-4
Specimen T-MA-B-3A/B was the final bottom beam flange tee joint specimen tested for this research. The first five phases of this loading was an attempt to replicate a SAC-like loading protocol with increasing displacement amplitudes in each phase. There were two primary differences between this loading and the SAC loading protocol: number of cycles and displacement amplitudes. A higher number of cycles was used in our protocol in an attempt to see if ratcheting was present in the joint, and the amplitudes are different because of the lack of control over the specimen.

Phases 6-9 consisted of on large initial cycle followed by a series of smaller constant displacement amplitude cycles. The reason for this loading was to cause some damage in the first cycle and attempt to propagate the damage during the constant displacement amplitude cycles.

Yielding was observed first in the web during the second phase of loading. The flange began to yield during phase 4. No cracks near the weld were observed at any point during the loading of specimen T-MA-B-3A/B. During phase 6 the flange began to buckle. This buckle increased with each successive loading phase. Finally, the buckle became severe enough that the test was stopped following the ninth loading phase. The behavior was similar to that demonstrated by T-MA-B-4A/B which failed in a brittle manner.

Following is the displacement history, load-displacement graphs, and strain mean and amplitude plots.
Fig. A.63 – Displacement history for T-MA-B-3A/B
Fig. A.64 – Load-displacement response for T-MA-B-3A/B

Fig. A.65 – Phase 1 load-displacement response for T-MA-B-3A/B
Fig. A.66 – Phase 2 load-displacement response for T-MA-B-3A/B

Fig. A.67 – Phase 3 load-displacement response for T-MA-B-3A/B
Fig. A.68 – Phase 4 load-displacement response for T-MA-B-3A/B

Fig. A.69 – Phase 5 load-displacement response for T-MA-B-3A/B
Fig. A.70 – Phase 6 load-displacement response for T-MA-B-3A/B

Fig. A.71 – Phase 7 load-displacement response for T-MA-B-3A/B
Fig. A.72 – Phase 8 load-displacement response for T-MA-B-3A/B

Fig. A.73 – Phase 9 load-displacement response for T-MA-B-3A/B
Fig. A.74 – Displacement amplitudes and means for T-MA-B-3A/B

Fig. A.75 – Load amplitudes and means for T-MA-B-3A/B
Fig. A.76 – Strain amplitudes and means for T-MA-B-3A SG-R3-O-4

Fig. A.77 – Strain amplitudes and means for T-MA-B-3B SG-R3-O-4
Fig. A.78 – Strain amplitudes and means for T-MA-B-3B SG-R2-O-4

Fig. A.79 – Strain amplitudes and means for T-MA-B-3B SG-R1-I-1
Fig. A.80 – Strain amplitudes and means for T-MA-B-3A SG-R1-O-4

Fig. A.81 – Strain amplitudes and means for T-MA-B-3B SG-R1-I-7
Fig. A.82 – Strain amplitudes and means for T-MA-B-3B SG-R1-O-7

Fig. A.83 – Strain amplitudes and means for T-MA-B-3B SG-R1-I-5
Fig. A.84 – Strain amplitudes and means for T-MA-B-3A SG-R1-O-6

Fig. A.85 – Strain amplitudes and means for T-MA-B-3B SG-R1-O-6
Fig. A.86 – Strain amplitudes and means for T-MA-B-3A SG-R1-O-4

Fig. A.87 – Strain amplitudes and means for T-MA-B-3A SG-R1-I-5
Fig. A.88 – Strain amplitudes and means for T-MA-B-3A SG-R2-O-4

Fig. A.89 – Strain amplitudes and means for T-MA-B-3A SG-R1-O-7
Fig. A.90 – Strain amplitudes and means for T-MA-B-3A SG-R1-I-7

Fig. A.91 – Strain amplitudes and means for T-MA-B-3A SG-R1-O-1
Fig. A.92 – Strain amplitudes and means for T-MA-B-3A SG-R1-I-1
T-MA-B-4A/B

Specimen T-MA-B-4A/B was the first bottom beam flange tee joint specimen tested for this research. It was loaded using an increasing displacement amplitude loading similar to the SAC loading protocol. The differences between this loading sequence the SAC protocol is the number of cycles at each displacement amplitude and the amplitudes. The displacements are different because of the difficulty of controlling the test.

This specimen failed in a ductile manner due to local buckling of the flange. With each increase in displacement, the displacement in the buckled region increased. Eventually a crack developed at the edge of the flange in the buckled region. With each successive cycle, the crack propagated until the specimen ruptured.

Following is the displacement history and load-displacement graphs.
Fig. A.93 – Displacement history for T-MA-B-4A/B
Fig. A.94 – Load-displacement response for T-MA-B-4A/B

Fig. A.95 – Phase 1 load-displacement response for T-MA-B-4A/B
Fig. A.96 – Phase 2 load-displacement response for T-MA-B-4A/B

Fig. A.97 – Phase 3 load-displacement response for T-MA-B-4A/B
Fig. A.98 – Phase 4 load-displacement response for T-MA-B-4A/B

Fig. A.99 – Phase 5 load-displacement response for T-MA-B-4A/B
Fig. A.100 – Phase 6 load-displacement response for T-MA-B-4A/B

Fig. A.101 – Phase 7 load-displacement response for T-MA-B-4A/B
Fig. A.102 – Phase 8 load-displacement response for T-MA-B-4A/B

Fig. A.103 – Phase 9 load-displacement response for T-MA-B-4A/B
Fig. A.104 – Phase 10 load-displacement response for T-MA-B-4A/B

Fig. A.105 – Phase 11 load-displacement response for T-MA-B-4A/B
Fig. A.106 – Phase 12 load-displacement response for T-MA-B-4A/B

Fig. A.107 – Phase 13 load-displacement response for T-MA-B-4A/B
Fig. A.108 – Phase 14 load-displacement response for T-MA-B-4A/B

Fig. A.109 – Phase 15 load-displacement response for T-MA-B-4A/B
Fig. A.110 – Displacement amplitudes and means for T-MA-B-4A/B

Fig. A.111 – Load amplitudes and means for T-MA-B-4A/B