ABSTRACT

SLOAN, JOHN ELLIOT. The Seismic Behavior of Reinforced Concrete Members at Low Temperatures. (Under the direction of Dr. Mervyn Kowalsky and Dr. Tasnim Hassan)

While reinforced concrete structures depend on ductility for acceptable seismic performance, research on the material behavior of concrete and steel has indicated that the loss of ductility may occur under low temperatures. The current research program investigates the behavior of reinforced concrete column-type members under low temperatures (-20 degrees Celsius, -30 degrees Celsius, and -40 degrees Celsius, approximately) and compares the results to an identical specimen tested at ambient laboratory temperature (23 degrees Celsius). The columns are lightly reinforced, and were loaded in a reversed cyclic manner while inside of an environmental chamber. The results of the experimentation indicate moderate increases in column strength as the temperature decreases, as well as moderate decreases in ultimate displacement capacity as the temperature decreases. The hysteretic damping properties of the columns were not significantly affected by low temperatures, and the specimen tested at -40 degrees Celsius exhibited a shortening of the extent of plasticity.
THE SEISMIC BEHAVIOR OF REINFORCED CONCRETE MEMBERS AT LOW TEMPERATURES

by

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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, CONSTRUCTION, AND ENVIRONMENTAL ENGINEERING

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APPROVED BY:

_________________________ _________________________
Chair of Advisory Committee
For Papa

“I will arise and go to my father.”

Luke 15:18
NKJV

and Mom

“...knowing from whom you have learned them, and that from childhood you have known the Holy Scriptures, which are able to make you wise for salvation through faith which is in Jesus Christ.”

2 Timothy 3:14-15
NKJV
BIOGRAPHY

John Sloan was born in Richmond, Virginia, in 1980, the third of Fred and Grace Sloan’s four children. He grew up in Powhatan County and attended Powhatan High School, where he played on the baseball and basketball teams. Upon his graduation from PHS, he attended the Virginia Military Institute, majoring in civil engineering and playing on the baseball team. He graduated with the VMI class of 2002, and then attended the North Carolina State University in Raleigh to pursue graduate studies in structural engineering. During his graduate school studies in Raleigh he met his lovely wife Amy, and they were married on July 24, 2004. In August of 2004, he began working for URS Corporation, where he is currently working on the design of the Henley Street Bridge in Knoxville, Tennessee, under the supervision of NCSU alumnus Dr. Satrajit Das. God blessed John and Amy with their first baby, Joshua Hazen Sloan, on May 23, 2005.
ACKNOWLEDGEMENTS

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TABLE OF CONTENTS

LIST OF TABLES ........................................................................................................ VII
LIST OF FIGURES ..................................................................................................... VIII
1 INTRODUCTION ..................................................................................................... 1
  1.1 DUCTILITY ........................................................................................................... 1
  1.2 RESEARCH MOTIVATION ................................................................................... 2
  1.3 RESEARCH OBJECTIVES ................................................................................... 2
  1.4 RESEARCH OVERVIEW .................................................................................... 3
  1.5 RESEARCH SIGNIFICANCE ................................................................................ 3
  1.6 MATERIAL BEHAVIOR ...................................................................................... 6
    1.6.1 Concrete Strength ........................................................................................ 6
    1.6.2 Concrete Stress-Strain Behavior ................................................................. 9
    1.6.3 Steel Monotonic Stress-Strain Behavior ..................................................... 10
    1.6.4 Steel Cyclic Stress-Strain Behavior ............................................................ 10
    1.6.5 Impact of High Strain Rates on Steel and Concrete Behavior ...................... 11
    1.6.6 Impact of Low Temperatures on Concrete Behavior (Including Bond Strength) .................................................................................................................................. 12
    1.6.7 Impact of Low Temperatures on Steel Behavior .......................................... 13
  1.7 CONCLUSION .................................................................................................... 14
2 MATERIAL TESTING ............................................................................................ 16
  2.1 STEEL REINFORCEMENT TESTING ................................................................ 17
  2.2 CONCRETE CYLINDER TESTS ......................................................................... 24
  2.3 CONCLUSIONS .................................................................................................. 33
3 TEST SETUP, INSTRUMENTATION, AND TEST SPECIMEN DESIGN .......... 35
  3.1 ENVIRONMENTAL CHAMBER ......................................................................... 35
  3.2 ACTUATOR EXTENSION .................................................................................... 37
  3.3 THE FOOTING SUPPORT .................................................................................. 40
  3.4 INSTRUMENTATION .......................................................................................... 42
  3.5 TEST SPECIMEN DESIGN ................................................................................ 45
  3.6 COLUMN DESIGN ............................................................................................. 45
  3.7 FOOTING DESIGN .............................................................................................. 48
4 EXPERIMENTATION AND RESULTS ................................................................. 49
  4.1 LOADING HISTORY .......................................................................................... 49
  4.2 SPECIMEN 1 EXPERIMENT RESULTS ............................................................ 51
  4.3 SPECIMEN 2 EXPERIMENT RESULTS ............................................................ 65
  4.4 SPECIMEN 3 EXPERIMENT RESULTS ............................................................ 80
  4.5 SPECIMEN 4 EXPERIMENT RESULTS ............................................................ 94
5 DISCUSSION OF RESULTS AND CONCLUSIONS ........................................... 108
  5.1 DISCUSSION OF THE VALIDITY OF THE TEST DATA .................................... 108
## LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table 2.1 Data from Monotonic Reinforcing Steel Tests</td>
<td>19</td>
</tr>
<tr>
<td>Table 2.2 Data from Monotonic Concrete Cylinder Tests</td>
<td>27</td>
</tr>
<tr>
<td>Table 2.3 Results from Cyclic Displacement-Control Tests on Concrete Cylinders</td>
<td>32</td>
</tr>
<tr>
<td>Table 2.4 Results from Cyclic Force-Control Tests on Concrete Cylinders</td>
<td>32</td>
</tr>
<tr>
<td>Table 5.1 Comparison of Energy Dissipation and Damping at Ductility 4, Cycle 1</td>
<td>123</td>
</tr>
<tr>
<td>Table 5.2 Specimen 1 Limit States</td>
<td>125</td>
</tr>
<tr>
<td>Table 5.3 Specimen 2 Limit States</td>
<td>125</td>
</tr>
<tr>
<td>Table 5.4 Specimen 3 Limit States</td>
<td>125</td>
</tr>
<tr>
<td>Table 5.5 Specimen 4 Limit States</td>
<td>126</td>
</tr>
</tbody>
</table>
**LIST OF FIGURES**

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>USGS Relative Seismic Intensity Map for North America</td>
<td>4</td>
</tr>
<tr>
<td>1.2</td>
<td>The USDA Map of Mean Annual Low Temperatures for North America</td>
<td>5</td>
</tr>
<tr>
<td>1.3</td>
<td>Impact of Increased Concrete Strength on a Concrete Crushing Failure</td>
<td>8</td>
</tr>
<tr>
<td>1.4</td>
<td>Impact of Increased Concrete Strength on a Steel Rupture Failure</td>
<td>8</td>
</tr>
<tr>
<td>1.5</td>
<td>The Impact of Confinement on the Stress-Strain Behavior of Concrete</td>
<td>9</td>
</tr>
<tr>
<td>1.6</td>
<td>The Impact of Temperature on the Charpy Notch Toughness of Steel</td>
<td>14</td>
</tr>
<tr>
<td>2.1</td>
<td>MTS Universal Testing Machine Prepared for Concrete Cylinder Testing</td>
<td>16</td>
</tr>
<tr>
<td>2.2</td>
<td>MTS Universal Testing Machine Prepared for Reinforcement Testing</td>
<td>17</td>
</tr>
<tr>
<td>2.3</td>
<td>Stress-Strain Responses of Reinforcement at the Strain Rate of 0.01 s(^{-1}) and Various Temperatures</td>
<td>20</td>
</tr>
<tr>
<td>2.4</td>
<td>Stress-Strain Responses of Reinforcement at the Strain Rate of 0.001 s(^{-1}) and Various Temperatures</td>
<td>20</td>
</tr>
<tr>
<td>2.5</td>
<td>Stress-Strain Responses of Reinforcement at Ambient Lab Temperature and Varying Strain Rates</td>
<td>23</td>
</tr>
<tr>
<td>2.6</td>
<td>Stress-Strain Responses of Reinforcement at Low Temperature and Varying Strain Rates</td>
<td>23</td>
</tr>
<tr>
<td>2.7</td>
<td>Picture of Linear Potentiometer Used to Record Displacement for the Concrete Cylinder Tests</td>
<td>25</td>
</tr>
<tr>
<td>2.8</td>
<td>Linear Potentiometer Readings for Test Specimen 10</td>
<td>26</td>
</tr>
<tr>
<td>2.9</td>
<td>Stress-Strain Responses of Concrete at a Strain Rate of 0.0001 s(^{-1})</td>
<td>28</td>
</tr>
<tr>
<td>2.10</td>
<td>Stress-Strain Responses of Concrete at a Strain Rate of 0.001 s(^{-1})</td>
<td>29</td>
</tr>
<tr>
<td>2.11</td>
<td>Stress-Strain Responses of Concrete at 22(^{\circ})C and Varying Strain Rates</td>
<td>29</td>
</tr>
<tr>
<td>2.12</td>
<td>Stress-Strain Responses of Concrete at -20(^{\circ})C and Varying Strain Rates</td>
<td>30</td>
</tr>
<tr>
<td>2.13</td>
<td>Stress-Strain Responses of Concrete at -40(^{\circ})C and Varying Strain Rates</td>
<td>30</td>
</tr>
<tr>
<td>2.14</td>
<td>Theoretical Monotonic Force-Deformation Responses Calculated Using the Material Properties at Various Temperatures</td>
<td>34</td>
</tr>
<tr>
<td>3.1</td>
<td>Test Setup</td>
<td>36</td>
</tr>
<tr>
<td>3.2</td>
<td>Outside View of the Environmental Chamber, Reaction Frame, and Hydraulic Actuator</td>
<td>37</td>
</tr>
<tr>
<td>3.3</td>
<td>The Actuator Extension</td>
<td>38</td>
</tr>
<tr>
<td>3.4</td>
<td>The Test Setup</td>
<td>39</td>
</tr>
<tr>
<td>3.5</td>
<td>The Footing Support</td>
<td>41</td>
</tr>
<tr>
<td>3.6</td>
<td>Location of Thermocouples within the Cross-Section</td>
<td>42</td>
</tr>
<tr>
<td>3.7</td>
<td>Location of the Linear Potentiometers</td>
<td>44</td>
</tr>
<tr>
<td>3.8</td>
<td>Column Design</td>
<td>45</td>
</tr>
<tr>
<td>3.9</td>
<td>Test Specimen Design</td>
<td>47</td>
</tr>
<tr>
<td>4.1</td>
<td>Force Control Portion of the Load History</td>
<td>49</td>
</tr>
<tr>
<td>4.2</td>
<td>Displacement Control Portion of the Loading History</td>
<td>50</td>
</tr>
<tr>
<td>4.3</td>
<td>Longitudinal Steel Temperature for Specimen 1, &quot;TC1&quot;</td>
<td>51</td>
</tr>
<tr>
<td>4.4</td>
<td>Temperature between TC1 and TC3, &quot;TC2&quot;</td>
<td>52</td>
</tr>
<tr>
<td>4.5</td>
<td>Temperature at the Center of the Cross-Section of Specimen 1, &quot;TC3&quot;</td>
<td>52</td>
</tr>
</tbody>
</table>
Figure 4.6 Crack Formation on the Top of Specimen 1 during the Push Segment of Ductility 1.5, Cycle 3 .......................... 54
Figure 4.7 Crack Formation of Specimen 1 during the Push Segment of Ductility 3, Cycle 1 ......................................................... 57
Figure 4.8 Crushing of Concrete Cover at the Bottom of Specimen 1 during the push segment of Ductility 4, Cycle 1 ....................... 58
Figure 4.9 Cracking and Flaking of Concrete on Specimen 1 during the Push Segment of Ductility 5, Cycle 1 .............................. 58
Figure 4.10 Crushing of Concrete Cover and Buckling of Bottom Reinforcement of Specimen 1 during the Push Segment of Ductility 6, Cycle 1 ................................................................. 60
Figure 4.11 Rupture of Bottom Reinforcement of Specimen 1 during the Push Segment of Ductility 6, Cycle 3 .............................. 60
Figure 4.12 Crushing of the Concrete and Rupture of the Top Reinforcement of Specimen 1 during the Push Segment of Ductility 8, Cycle 1 ................................................................. 60
Figure 4.13 Force-Deformation Response of Specimen 1 and Calculated Monotonic Response .................................................. 61
Figure 4.14 Specimen 1 Backbone Responses ................................................................. 62
Figure 4.15 Specimen 1 Displacement Profiles .............................................................. 63
Figure 4.16 Specimen 1 Moment-Curvature Response, 3.6 Inches to 10.8 Inches from the Face of the Footing ............................ 63
Figure 4.17 Specimen 1 Curvature Profiles .................................................................. 64
Figure 4.18 Specimen 2 Temperature While Cooling ..................................................... 66
Figure 4.19 Longitudinal Steel Temperature for Specimen 2, "TC1" .................................................. 67
Figure 4.20 Temperature between TC1 and TC3, "TC2" .............................................. 68
Figure 4.21 Temperature at the Center of the Cross-Section of Specimen 2, "TC3" ... 68
Figure 4.22 Crack Formation on the Top of Specimen 2 during the Push Segment of Ductility 1.5, Cycle 1 ........................................ 70
Figure 4.23 Crack Formation on the Top of Specimen 2 during the Push Segment of Ductility 3, Cycle 1 ........................................ 70
Figure 4.24 Crushing of Concrete Cover on the Bottom of Specimen 2 during the Push Segment of Ductility 4, Cycle 1 .................. 71
Figure 4.25 Crack Formation on the Top of Specimen 2 during the Push Segment of Ductility 5, Cycle 1 ........................................ 71
Figure 4.26 Crushing of Specimen 2 during the Push Segment of Ductility 6, Cycle 1 .... 72
Figure 4.27 Concrete Crushing and Rupture of Reinforcement on the Bottom of Specimen 2 after Testing ....................................... 74
Figure 4.28 Concrete Crushing on the Top of Specimen 2 after Testing ........................... 74
Figure 4.29 Specimen 2 Force-Deformation Response .................................................. 77
Figure 4.30 Specimen 2 Backbone Responses .............................................................. 77
Figure 4.31 Specimen 2 Displacement Profiles .............................................................. 78
Figure 4.32 Specimen 2 Moment-Curvature Response, 0 to 6.1 Inches from the Face of the Footing .................................................. 78
Figure 4.33 Specimen 2 Moment-Curvature Response, 3.8 to 10.7 Inches from the Face of the Footing ............................................... 79
Figure 4.34 Specimen 2 Curvature Profiles ................................................................. 79
Figure 4.35 Temperature of Specimen 3 while Cooling .................................................. 81
Figure 4.36 Ambient Environmental Chamber Temperature during Test 3 .................... 81
Figure 4.37 Longitudinal Steel Temperature for Specimen 3 during Testing, "TC1" ..... 82
Figure 4.38 Temperature between TC1 and TC3, "TC2" ................................................. 82
Figure 4.39 Temperature at the Center of the Cross-Section of Specimen 3 during Testing, "TC3" ........................................................................................................ 83
Figure 4.40 Crack Formation on the Top of Specimen 3 during the Push Segment of Ductility 1.5, Cycle 1 ................................................................................................. 84
Figure 4.41 Crack Formation on the Top of Specimen 3 during the Push Segment of Ductility 3, Cycle 1 ................................................................................................. 84
Figure 4.42 Crushing of Concrete Cover on the Bottom of Specimen 3 during the Push Segment of Ductility 4, Cycle 1 ............................................................................ 85
Figure 4.43 Crack Formation on the Top of Specimen 3 during the Push Segment of Ductility 5, Cycle 1 ................................................................................................. 86
Figure 4.44 Crushing of Concrete Cover on the Bottom of Specimen 3 during the Push Segment of Ductility 6, Cycle 1 ................................................................................................. 86
Figure 4.45 Concrete Crushing and Rupture of Reinforcement on the Top of Specimen 3 after Testing .................................................................................................................. 89
Figure 4.46 Concrete Crushing on the Bottom of Specimen 3 after Testing ..................... 89
Figure 4.47 Specimen 3 Force-Deformation Response ......................................................... 90
Figure 4.48 Specimen 3 Backbone Responses .................................................................... 91
Figure 4.49 Specimen 3 Moment-Curvature Response, 3.8 to 10.6 Inches from the Face of the Footing .................................................................................................................................................. 92
Figure 4.50 Specimen 3 Moment-Curvature Response, 0 to 6.2 Inches from the Face of the Footing .................................................................................................................................................. 92
Figure 4.51 Specimen 3 Curvature Profiles ..................................................................... 93
Figure 4.52 Temperature of Specimen 4 While Cooling .................................................... 95
Figure 4.53 Longitudinal Steel Temperature for Specimen 4, "TC1" ................................. 95
Figure 4.54 Temperature between TC1 and TC3, "TC2" ................................................. 96
Figure 4.55 Temperature at the Center of the Cross-Section of Specimen 4, "TC3" ....... 96
Figure 4.56 Crack Formation on the Bottom of Specimen 4 during the Push Segment of Ductility 1.5, Cycle 1 ................................................................................................. 97
Figure 4.57 Crushing of Concrete Cover on the Top of Specimen 4 during the Push Segment of Ductility 3, Cycle 1 ................................................................................................. 98
Figure 4.58 Crushing of Concrete Cover on the Top of Specimen 4 during the Push Segment of Ductility 5, Cycle 1 ................................................................................................. 99
Figure 4.59 Crushing of Concrete Cover on the Bottom of Specimen 4 during the Push Segment of Ductility 4, Cycle 1 ................................................................................................. 100
Figure 4.60 Crushing of Concrete Cover on the Top of Specimen 4 during the Push Segment of Ductility 6, Cycle 1 ................................................................................................. 100
Figure 4.61 Concrete Crushing and Rupture of Reinforcement on the Top of Specimen 4 after Testing .................................................................................................................. 102
Figure 4.62 Concrete Crushing and Rupture of Reinforcement on the Bottom of Specimen 4 after Testing .................................................................................................................. 102
Figure 4.63 Specimen 4 Force-Deformation Response ....................................................... 103
Figure 4.64 Specimen 4 Backbone Response .................................................................... 104
Figure 4.65 Specimen 4 Moment-Curvature Response, 0 to 6.0 Inches from the Face of the Footing .............................................................. 105
Figure 4.66 Specimen 4 Moment-Curvature Response 3.75 Inches to 10.5 Inches from the Face of the Footing .................................................... 105
Figure 4.67 Specimen 4 Curvature Profiles ................................................................................................................................. 107
Figure 5.1 Specimen 1 Comparison of Displacement Obtained from String Potentiometers with Displacement Calculated from Curvature 110
Figure 5.2 Specimen 2 Comparison of Displacement Obtained from String Potentiometers with Displacement Calculated from Curvature 111
Figure 5.3 Specimen 3 Comparison of Displacement Obtained from String Potentiometers with Displacement Calculated from Curvature 112
Figure 5.4 Specimen 4 Comparison of Displacement Obtained from String Potentiometers with Displacement Calculated from Curvature 113
Figure 5.5 Comparison of First Cycle Backbone Responses for All Four Specimens .................................................................................. 114
Figure 5.6 Comparison of Second Cycle Backbone Responses for All Four Specimens ........................................................................ 115
Figure 5.7 Comparison of Third Cycle Backbone Responses for All Four Specimens 115
Figure 5.8 Specimen Ultimate Strength vs. Temperature ........................................................................................................................ 116
Figure 5.9 Specimen Overstrength vs. Temperature .......................................................................................................................... 116
Figure 5.10 Specimen Drift at Maximum Force vs. Temperature ........................................................................................................ 117
Figure 5.11 Specimen Drift at First Rupture vs. Temperature ........................................................................................................ 117
Figure 5.12 Specimen Drift at 90% of Maximum Force vs. Temperature .......................................................................................... 118
Figure 5.13 Drift at 80% of Maximum Force vs. Temperature ......................................................................................................... 118
Figure 5.14 Comparison of Hysteresis loop Responses during Ductility 4, Cycle 1 for All Specimens ........................................................................ 123
Figure 5.15 Plastic Hinge Length as a Function of Displacement Ductility during the Push Segment of the First Cycle 128
Figure 5.16 Plastic Hinge Length as a Function of Displacement Ductility during the Pull Segment of the First Cycle 129
Figure A1.1 Displacement Control Stress-Strain Response of Concrete Cylinder 1 at a temperature of 22°C and a Strain Rate of 0.0001 s^{-1} ........................................ 134
Figure A1.2 Displacement Control Stress-Strain Response of Concrete Cylinder 4 at a Temperature of 22°C and a Strain Rate of 0.0001 s^{-1} ........................................ 134
Figure A1.3 Displacement Control Stress-Strain Response of Concrete Cylinder 5 at a Temperature of 22°C and a Strain Rate of 0.001 s^{-1} ........................................ 135
Figure A1.4 Force Control Stress-Strain Response of Concrete Cylinder 6 at a Temperature of 22°C and a Strain Rate of 0.001 s^{-1} ........................................ 135
Figure A1.5 Force Control Stress-Strain Response of Concrete Cylinder 7 at a Temperature of 22°C and a Strain Rate of 0.001 s^{-1}, 153 Cycles to Rupture ........ 136
Figure A1.6 Force Control Stress-Strain Response of Concrete Cylinder 8 at a Temperature of 22°C and a Strain Rate of 0.0001 s^{-1} ........................................ 136
Figure A1.7 Displacement Control Stress-Strain Response of Concrete Cylinder 12 at a Temperature between -24°C and -19°C, with a Strain Rate of 0.0001 s^{-1} ............ 137
Figure A1.8 Displacement Control Stress-Strain Response of Concrete Cylinder 13 at a Temperature of -24°C and a Strain Rate of 0.001 s^{-1} ........................................ 137
Figure A1.9 Force Control Stress-Strain Response of Concrete Cylinder 14 between the Temperature of -20°C and 1.5°C, with a Strain Rate of 0.0001 s⁻¹ for 66 Cycles. 138
Figure A1.10 Displacement Control Stress-Strain Response of Concrete Cylinder 14 at a Temperature of -21°C and a Strain Rate of 0.0001 s⁻¹ ........................................... 138
Figure A1.11 Force Control Stress-Strain Response of Concrete Cylinder 15 between the Temperature of -25°C and -10°C, with a Strain Rate of 0.001 s⁻¹ for 400 Cycles. 139
Figure A1.12 Force Control Stress-Strain Response of Concrete Cylinder 15 between the Temperature of -23°C and -7°C, with a Strain Rate of 0.001 s⁻¹ for 400 Cycles ... 139
Figure A1.13 Displacement Control Stress-Strain Response of Concrete Cylinder 15 at a Temperature of -21°C and a Strain Rate of 0.001 s⁻¹ ............................................. 139
Figure A1.14 Force Control Stress-Strain Response of Concrete Cylinder 16 between the Temperature of -23°C and -11°C, with a Strain Rate of 0.001 s⁻¹, 239 Cycles to Rupture ................................................................................................................... 140
Figure A1.15 Displacement Control Stress-Strain Response of Concrete Cylinder 20 at a Temperature between -41°C and -21°C, with a Strain Rate of 0.0001 s⁻¹ .......... 141
Figure A1.16 Displacement Control Stress-Strain Response of Concrete Cylinder 21 at a Temperature of -42°C, with a Strain Rate of 0.001 s⁻¹ .............................................................. 141
Figure A1.17 Force Control Stress-Strain Response of Concrete Cylinder 23 between the Temperature of -39°C and -33°C, with a Strain Rate of 0.001 s⁻¹, 138 Cycles to Rupture ................................................................................................................... 142
Figure A1.18 Force Control Stress-Strain Response of Concrete Cylinder 24 between the Temperature of -39°C and -27°C, with a Strain Rate of 0.001 s⁻¹ for 239 Cycles . 142
Figure A1.19 Displacement Control Stress-Strain Response of Concrete Cylinder 24 at a Temperature of -44°C, with a Strain Rate of 0.001 s⁻¹.............................................................. 143
Figure A2.1 Linear Potentiometer History on the Top of Specimen 1, 3.6 Inches to 10.8 Inches from the Face of the Footing .......................................................... 144
Figure A2.2 Linear Potentiometer History on the Bottom of Specimen 1, 3.6 Inches to 10.6 Inches from the Face of the Footing .......................................................... 144
Figure A2.3 Linear Potentiometer History on the Top of Specimen 1, 10.8 Inches to 17.9 Inches from the Face of the Footing ......................................................... 145
Figure A2.4 Linear Potentiometer History on the Bottom of Specimen 1, 10.6 Inches to 17.4 Inches from the Face of the Footing ......................................................... 145
Figure A2.5 Linear Potentiometer History on the Top of Specimen 1, 17.9 Inches to 24.9 Inches from the Face of the Footing ......................................................... 146
Figure A2.6 Linear Potentiometer History on the Bottom of Specimen 1, 17.4 Inches to 24.1 Inches from the Face of the Footing ......................................................... 146
Figure A2.7 Linear Potentiometer History on the Top of Specimen 1, 24.9 Inches to 31.9 Inches from the Face of the Footing ......................................................... 147
Figure A2.8 Linear Potentiometer History on the Bottom of Specimen 1, 24.1 Inches to 31.0 Inches from the Face of the Footing ......................................................... 147
Figure A2.9 Linear Potentiometer History on the Top of Specimen 1, 31.9 Inches to 38.8 Inches from the Face of the Footing ......................................................... 148
Figure A2.10 Linear Potentiometer History on the Bottom of Specimen 1, 31.0 Inches to 37.6 Inches from the Face of the Footing ......................................................... 148
Figure A2.11 Specimen 1 String Potentiometer History at the Location 37.6 Inches from the Face of the Footing ......................................................... 149
Figure A2.12 Specimen 1 String Potentiometer History at the Location 61.9 Inches from the Face of the Footing ........................................................................................................... 149
Figure A2.13 Specimen 1 String Potentiometer History at the Location 73 Inches from the Face of the Footing ........................................................................................................... 150
Figure A3.1 Linear Potentiometer History on the Top Left (Facing the Footing) of Specimen 2, 0 Inches to 6 Inches from the Face of the Footing ........................................ 151
Figure A3.2 Linear Potentiometer History on the Top Right (Facing the Footing) of Specimen 2, 0 Inches to 5.9 Inches from the Face of the Footing ........................................ 151
Figure A3.3 Linear Potentiometer History on the Bottom Right (Facing the Footing) of Specimen 2, 0 Inches to 6.1 Inches from the Face of the Footing .......................... 152
Figure A3.4 Linear Potentiometer History on the Bottom Left (Facing the Footing) of Specimen 2, 0 Inches to 6.2 Inches from the Face of the Footing ...................................... 152
Figure A3.5 Linear Potentiometer History on the Top of Specimen 2, 3.75 Inches to 10.8 Inches from the Face of the Footing ................................................................. 153
Figure A3.6 Linear Potentiometer History on the Bottom of Specimen 2, 3.9 Inches to 10.6 Inches from the Face of the Footing ................................................................. 153
Figure A3.7 Linear Potentiometer History on the Top of Specimen 2, 10.8 Inches to 17.8 Inches from the Face of the Footing ................................................................. 154
Figure A3.8 Linear Potentiometer History on the Bottom of Specimen 2, 10.6 Inches to 17.4 Inches from the Face of the Footing ................................................................. 154
Figure A3.9 Linear Potentiometer History on the Top of Specimen 2, 17.8 Inches to 24.4 Inches from the Face of the Footing ................................................................. 155
Figure A3.10 Linear Potentiometer History on the Bottom of Specimen 2, 17.4 Inches to 24.1 Inches from the Face of the Footing ................................................................. 155
Figure A3.11 Linear Potentiometer History on the Top of Specimen 2, 24.4 Inches to 31.2 Inches from the Face of the Footing ................................................................. 156
Figure A3.12 Linear Potentiometer History on the Bottom of Specimen 2, 24.1 Inches to 30.8 Inches from the Face of the Footing ................................................................. 156
Figure A3.13 Linear Potentiometer History on the Top of Specimen 2, 31.2 Inches to 37.6 Inches from the Face of the Footing ................................................................. 157
Figure A3.14 Linear Potentiometer History on the Bottom of Specimen 2, 30.8 Inches to 37.5 Inches from the Face of the Footing ................................................................. 157
Figure A3.15 Specimen 2 String Potentiometer History at the Location 37.9 Inches from the face of the footing ................................................................. 158
Figure A3.16 Specimen 2 String Potentiometer History at the Location 58.9 Inches from the Face of the Footing ................................................................. 158
Figure A3.17 Specimen 2 String Potentiometer History at the Location 71.5 Inches from the Face of the Footing ................................................................. 159
Figure A4.1 Linear Potentiometer History on the Top Left (Facing the Footing) of Specimen 3, 0 Inches to 6.2 Inches from the Face of the Footing .............................................. 160
Figure A4.2 Linear Potentiometer History on the Top Right (Facing the Footing) of Specimen 3, 0 Inches to 6.1 Inches from the Face of the Footing .............................................. 160
Figure A4.3 Linear Potentiometer History on the Bottom Right (Facing the Footing) of Specimen 3, 0 Inches to 6 Inches from the Face of the Footing .............................................. 161
Figure A4.4 Linear Potentiometer History on the Bottom Left (Facing the Footing) of Specimen 3, 0 Inches to 6 Inches from the Face of the Footing .............................................. 161
Figure A4.5 Linear Potentiometer History on the Top of Specimen 3, 3.75 Inches to 10.7 Inches from the Face of the Footing .......................................................... 162
Figure A4.6 Linear Potentiometer History on the Bottom of Specimen 3, 3.8 Inches to 10.6 Inches from the Face of the Footing .......................................................... 162
Figure A4.7 Linear Potentiometer History on the Top of Specimen 3, 10.7 Inches to 17.3 Inches from the Face of the Footing .......................................................... 163
Figure A4.8 Linear Potentiometer History on the Bottom of Specimen 3, 10.6 Inches to 17.2 Inches from the Face of the Footing .......................................................... 163
Figure A4.9 Linear Potentiometer History on the Top of Specimen 3, 17.3 Inches to 23.9 Inches from the Face of the Footing .......................................................... 164
Figure A4.10 Linear Potentiometer History on the Bottom of Specimen 3, 17.2 Inches to 23.9 Inches from the Face of the Footing .......................................................... 164
Figure A4.11 Linear Potentiometer History on the Top of Specimen 3, 23.9 Inches to 30.9 Inches from the Face of the Footing .......................................................... 165
Figure A4.12 Linear Potentiometer History on the Bottom of Specimen 3, 23.9 Inches to 30.6 Inches from the Face of the Footing .......................................................... 165
Figure A4.13 Linear Potentiometer History on the Top of Specimen 3, 30.9 Inches to 37.6 Inches from the Face of the Footing .......................................................... 166
Figure A4.14 Linear Potentiometer History on the Bottom of Specimen 3, 30.6 Inches to 37.1 Inches from the Face of the Footing .......................................................... 166
Figure A4.15 Specimen 3 String Potentiometer History at the Location 59.9 Inches from the Face of the Footing .......................................................... 167
Figure A4.16 Specimen 3 String Potentiometer History at the Location 70.5 Inches from the Face of the Footing .......................................................... 167
Figure A5.1 Linear Potentiometer History on the Top Left (Facing the Footing) of Specimen 4, 0 Inches to 6.1 Inches from the Face of the Footing .......................................................... 168
Figure A5.2 Linear Potentiometer History on the Top Right (Facing the Footing) of Specimen 4, 0 Inches to 6.25 Inches from the Face of the Footing .......................................................... 168
Figure A5.3 Linear Potentiometer History on the Bottom Right (Facing the Footing) of Specimen 4, 0 Inches to 6.1 Inches from the Face of the Footing .......................................................... 169
Figure A5.4 Linear Potentiometer History on the Bottom Left (Facing the Footing) of Specimen 4, 0 Inches to 6.25 Inches from the Face of the Footing .......................................................... 169
Figure A5.5 Linear Potentiometer History on the Top of Specimen 4, 3.75 Inches to 10.5 Inches from the Face of the Footing .......................................................... 170
Figure A5.6 Linear Potentiometer History on the Bottom of Specimen 4, 3.7 Inches to 10.4 Inches from the Face of the Footing .......................................................... 170
Figure A5.7 Linear Potentiometer History on the Top of Specimen 4, 10.5 Inches to 17.3 Inches from the Face of the Footing .......................................................... 171
Figure A5.8 Linear Potentiometer History on the Bottom of Specimen 4, 10.4 Inches to 17.0 Inches from the Face of the Footing .......................................................... 171
Figure A5.9 Linear Potentiometer History on the Top of Specimen 4, 17.3 Inches to 23.9 Inches from the Face of the Footing .......................................................... 172
Figure A5.10 Linear Potentiometer History on the Bottom of Specimen 4, 17.0 Inches to 23.6 Inches from the Face of the Footing .......................................................... 172
Figure A5.11 Linear Potentiometer History on the Top of Specimen 4, 23.9 Inches to 30.9 Inches from the Face of the Footing .......................................................... 173
Figure A5.12 Linear Potentiometer History on the Bottom of Specimen 4, 23.6 Inches to 30.1 Inches from the Face of the Footing .............................................................. 173
Figure A5.13 Linear Potentiometer History on the Top of Specimen 4, 30.9 Inches to 37.3 Inches from the Face of the Footing .............................................................. 174
Figure A5.14 Linear Potentiometer History on the Bottom of Specimen 4, 30.1 Inches to 36.9 Inches from the Face of the Footing .............................................................. 174
Figure A5.15 Specimen 4 String Potentiometer History at the Location 59.2 Inches from the Face of the Footing .............................................................. 175
Figure A5.16 Specimen 4 String Potentiometer History at the Location 69.9 Inches from the Face of the Footing .............................................................. 175
1 Introduction

As the study of earthquake engineering has advanced, engineers have determined how to significantly improve structural behavior during a seismic event. The scope of these advances runs the gamut of technology and research, from complex computer analysis to large-scale laboratory testing of structural members. The great irony of such diverse advances is that they have shown that seismic design often times can be simplified to one element of structural performance: ductility. When a structure undergoes significant lateral loading, it will behave in a satisfactory manner as long as it possesses the necessary ductility capacity to meet the demand applied by the earthquake. While a good engineer will utilize a multitude of engineering concepts in the seismic design of a structure, the thread tying them together must be the simple over-arching concept of ductility.

1.1 Ductility

As defined by Paulay and Priestley (1992), ductility is the ability of a structure to undergo significant inelastic deformation without a significant loss of strength or stiffness. At this point, it must be noted that some structures are designed to remain elastic during a seismic event, and therefore they should never reach the inelastic range. Nevertheless, these structures still must be designed for ductile behavior, because when a slightly larger earthquake hits the structure than the maximum considered earthquake, these structures will collapse if they are not designed for sufficient ductility, as the 1994 Kobe earthquake has demonstrated (Priestley et. al. 1996). One cannot design elastically for an infinitely large earthquake, and thus it becomes necessary to design for structural behavior in the nonlinear range. In addition to displacement capacity, another favorable
characteristic of behavior that ductility provides is energy dissipation. This is evidenced by large, wide hysteresis loops on a force-deformation curve, as the area of the loops corresponds to hysteretic damping, and therefore energy dissipation.

1.2 Research Motivation

This research seeks to determine the impact of low temperatures on the seismic behavior of reinforced concrete structural members. At low temperatures, past research has indicated that concrete increases in strength significantly (Lee et. al. 1988), which could cause brittle behavior during an earthquake. In addition, research has indicated that low temperatures cause steel to lose ductility capacity (Erranti and Lee 1986), which could also have a detrimental impact on the member behavior. An illustration of this point can be drawn from the 1940s, when the hulls of several ships sailing in the cold waters of the North Atlantic unexpectedly fractured, causing some of them to sink (Bruneau et al. 1998; Mamlouk and Zaniewski 1999). The following research project seeks to determine if such unfavorable loss of ductility will occur in reinforced concrete members under low temperatures.

1.3 Research Objectives

The most important factors that will be investigated in this research program will be the change in ductility capacity and the change in hysteretic damping of reinforced concrete structural members at low temperatures. In addition, the change in strength will also be monitored to determine the increase in moment capacity, which will be useful to determine the change in shear demand as well as possible changes in the hierarchy of strength. From this data, overstrength factors and steel strain reduction factors can be established for design, if they are necessary.
1.4 Research Overview

In order to accomplish the research objectives, experimental studies on the material properties of reinforcement and concrete cylinders at low temperatures were performed. In addition to the material testing, four identical circular column-type members were subjected to a simulated seismic loading history inside of an environmental chamber under various temperatures. The specimens are not true columns because they will be tested while in a horizontal position, and therefore they will not carry an axial load. However, the specimens are column-type members in their cross-section design, and their response to the experimental regimen is expected to shed light on column behavior at low temperature so that the knowledge gained can be used for design applications. The specimen design will be typical of column designs used in seismic areas, and they will be tested at temperatures of approximately 23°C, -20°C, -30°C, and -40°C.

1.5 Research Significance

The potential geographic area that could be impacted by this research can be thought of as an overlay of a seismic hazard map with an isothermal map. The exact area of impact cannot be identified until a better understanding of the effect of low temperatures on reinforced concrete member behavior is achieved. Figure 1.1 shows the North American portion of the USGS relative seismic intensity map, while Figure 1.2 represents the USDA North American map of mean annual low temperatures showing isotherms of 0°C, -20°C, and -40°C. Although no definitive answer can be given at this time as to the range of areas impacted by the combined effects of seismic excitations and low temperature, the possibility exists that a large portion of North America could be
affected, as seen by the common areas of these two maps. Two locations of particular interest are Alaska and Missouri, which both experience low temperatures and significant seismic activity. The following research program is an attempt to shed light on the issue so that brittle seismic failure can be prevented in those regions where temperatures are low enough for it to occur. Furthermore, those regions that will not face temperatures low enough to cause brittle failure will know at the conclusion of this research that there is no reason for concern in these areas.

Figure 1.1 USGS Relative Seismic Intensity Map for North America (1999)
Figure 1.2 The USDA Map of Mean Annual Low Temperatures for North America (2002)
1.6 Material Behavior

In order to understand the structural behavior of steel reinforced concrete members, one must first come to an understanding of the behavior of the component materials. Extensive testing has been conducted on steel reinforcement and concrete cylinders at room temperature, and their material properties are well established. However, many researchers have noted that relatively little experimentation has been conducted on the behavior of steel and concrete at low temperatures (Filiatrault and Holleran 2001; Lee et al. 1988). This chapter presents a review of the basic properties of steel and concrete, including what is known about low temperature effects. Several other factors will also be considered as they apply to seismic design, including the impact of reversed cyclic loading, high strain rates, confinement, as well as the bond strength between steel and concrete. All of these variables will be evaluated in the light of ductile behavior of reinforced concrete structures. Another factor that could impact the behavior of structural members is the freezing of soil surrounding a structure. This will not be discussed in detail in this paper, but it is another factor that must be kept in mind when discussing structural engineering in cold regions (Sritharan et al. 2004).

1.6.1 Concrete Strength

While the stress-strain behavior of concrete is an important characteristic of its behavior, the fundamental characteristic an engineer will typically specify in a design is the ultimate concrete strength. Typical concrete strengths of 28-day old concrete are determined by factors such as water/cement ratio, admixtures, and the actual ingredients (cement and aggregate) used in the mix. For seismic design, strengths usually range from approximately 3.25 to 6.5 ksi (22.4 to 44.8 MPa), but it is important to note that the
actual 28-day strength for concrete is generally 20-25% higher than the specified strength. Concrete also increases in strength with age, and it is important for an engineer to be aware of this increase in strength. For example, the concrete specified for the footings of the test specimens in this project was 4 ksi (27.6 MPa), but the actual strength was 7 ksi (48.23 MPa) when they were tested after 242 days. This is an increase in strength of 75% due to the fact that concrete is generally stronger than the specified strength, and because of a strength increase due to age. As concrete gains strength, it can be expected to tend toward more brittle behavior, which is not desirable for seismic design (Preistley et. al. 1996).

In some cases, an increase in strength will be beneficial to seismic behavior, while in other cases it will be detrimental. This is demonstrated by Figure 1.3 and Figure 1.4. In both cases, at the ultimate conditions the depth of the neutral axis decreases due to the increase in concrete strength. In the first case, based on a concrete crushing failure, this change in the neutral axis depth is beneficial, because the ultimate curvature is increased, the displacement capacity is increased, and there is greater ductility. In the second case, based on a steel rupture failure, the change in the neutral axis depth is detrimental, because the ultimate curvature is decreased, the displacement capacity is decreased, and the column has less ductility.
Figure 1.3 Impact of Increased Concrete Strength on a Concrete Crushing Failure

Figure 1.4 Impact of Increased Concrete Strength on a Steel Rupture Failure
1.6.2 Concrete Stress-Strain Behavior

The stress-strain behavior of concrete is best considered in light of confinement. Concrete without confinement loaded in flexure will usually crush at an ultimate strain of about 0.003 or 0.004 (ACI 318 2000). However, when the concrete is properly confined, it can sustain a strain of 5 or more times this amount. This is demonstrated in Figure 1.5 (Mander et. al. 1988). While the ultimate stress does increase, the important aspect of behavior is the increase in strain, because this allows a larger rotation capacity of concrete members. For example, a bridge column with confined concrete will sustain a larger core compressive strain than a similar column with unconfined concrete. As a result, the confined column has a larger rotation capacity, which thus increases its ductility capacity.

Figure 1.5 The Impact of Confinement on the Stress-Strain Behavior of Concrete (Mander et. al. 1988)
1.6.3 Steel Monotonic Stress-Strain Behavior

When loaded in a monotonic fashion, the stress-strain relationship of steel is characterized by an initial period of linearly elastic behavior. Following this is a period of yielding, then strain hardening of the steel, and ultimately the fracture of the specimen. The yielding and strain hardening of steel are both desirable characteristics for seismic design because they give ductility to reinforced concrete members. It is important in seismic design to ensure that the yield point is not higher than expected, since capacity design requires the accurate knowledge of member strength. If reinforcement is stronger than expected, a member would develop a plastic hinge later than expected, thus possibly upsetting the intended hierarchy of strength. Another reason that the yield stress of steel must not be too high is that this ensures the proper occurrence of strain hardening, which is beneficial because it spreads plastic behavior over a greater length of a member. For example, when one portion of a bar has yielded and entered strain hardening, the stress will increase in the bar. This causes a longer portion of the bar to yield (which had previously been elastic). In other words, the plastic action in the bar is not concentrated, and therefore strain is spread along the bar. A706 seismic steel is regulated to make sure its yield point stays below 78 ksi (537.4 MPa) to ensure that the approximate yield strength is known and so that the reinforcement behaves with proper strain hardening (Preistley et. al. 1996).

1.6.4 Steel Cyclic Stress-Strain Behavior

As noted above, the inelastic behavior of steel gives a reinforced concrete member its ductility. This depends on the ability of the steel to sustain repeated cycles into the nonlinear range during an earthquake without significant stress degradation. These cycles will produce large hysteresis loops, and the area inside of the loops
represents the energy dissipation abilities of the system. Without entering the inelastic range, the steel stress-strain response will oscillate along a straight line with a slope equal to the modulus of elasticity. Thus, a structure which remains in the elastic range will not dissipate energy as much as an inelastic structure.

When discussing the cyclic behavior of steel, it is also important to mention buckling of rebar. This is a predominant failure mode for well-designed reinforced concrete columns. While the confinement of the concrete core of a column increases the ultimate strain of the concrete, it also prevents the longitudinal steel from buckling. However, after enough cycles in the inelastic range, a column will typically fail by one of two modes of buckling. Either the bar will buckle over a several layers of transverse reinforcement, or it will buckle locally between layers of transverse reinforcement (Priestley et. al. 1996).

1.6.5 Impact of High Strain Rates on Steel and Concrete Behavior

While the data on concrete behavior at high strain rates is not exhaustive, research indicates that a high strain rate has the effect of increasing the ultimate concrete strength, the ultimate concrete strain, as well as the elastic modulus of the concrete (Bischoff and Perry 1991). For steel, research has shown that high strain rates increase the yield and ultimate strengths, while they have no impact on the elastic modulus (Filiatrault and Holleran 2001). However, it has been noted that the impact of high strain rates does not have a significant effect on overall structural behavior, as quasi-static laboratory test results are comparable to shake table test results with high strain rates (Preistley et. al. 1996).
1.6.6 Impact of Low Temperatures on Concrete Behavior (Including Bond Strength)

Research conducted by Lee et al. (1988) has found that concrete strength increases by as much as 54% when its temperature is reduced from 20°C to –30°C. The modulus of elasticity increased by about 25%, while the splitting tensile strength increased nearly 59%. They also found that with a similar temperature reduction the bond strength between steel and concrete increased 55% under monotonic loading, 48% under repeated cyclic loading, and 76% under reversed cyclic loading. Poisson’s ratio increased by 20%, which could possibly have an impact on the confined behavior of concrete, expressly that this could cause the confining steel to rupture prematurely due to the added expansion of the concrete.

The increases in strength that occur at low temperatures are analogous to the use of high strength concrete, which is becoming more and more commonplace. Much research has been conducted on the behavior of members that use high strength concrete, and the results indicate that ductile behavior can occur in columns as long as sufficient transverse reinforcement is provided. High strength concrete columns require greater confinement than normal strength concrete columns, but the necessary confinement can be provided in the form of more tightly spaced transverse reinforcement or higher strength confining steel (Razvi and Saatcioglu 1994; Saatcioglu and Baingo 1999). Less micro-cracking occurs in high strength concrete than in normal strength concrete as the ultimate stress is approached, which means that the high strength concrete will expand less in the lateral direction prior to failure. As a result, the greater confining pressure is needed in high strength concrete (Park 1998). This is where the analogy between high strength concrete and concrete at low temperatures breaks down, since the poisson’s ratio
of concrete at low temperatures increases, as stated above. Therefore, the increase in strength at low temperatures is possibly more favorable than the use of high strength concrete due to the fact that members at low temperature may not need the additional confinement that high strength concrete requires. The possibility exists that the increased expansion caused by low temperature will offset the reduction in micro-cracking and lateral expansion caused by the increase in strength.

Research on high strength concrete has also shown that axial load can significantly affect the behavior of a high strength concrete column, as a higher axial load tends to decrease the ductility of the member (Saatcioglu and Baingo 1999; Azizinamini et al. 1994). All of these factors are pertinent to the discussion of concrete at low temperatures even though the effects of variable confinement and variable axial load will not be investigated in this research. However, this knowledge could help interpret the results of the testing, and it also serves as a guide for future research.

1.6.7 Impact of Low Temperatures on Steel Behavior

Research conducted using the Charpy notch toughness test has shown that steel loses much ductility at low temperatures. The Charpy notch toughness test measures the energy required to fracture a V-notched steel specimen, and the results of tests at various temperatures are shown in Figure 1.6 (Mamlouk and Zaniewski 1999). Some steels will have an S shaped curve that is shifted to the left or right depending on their chemical makeup, but the general shape of the curve will typically stay the same (Eranti and Lee 1986). This graph is a tremendous motivation for this research. If reinforcing steel shows a similar loss of ductility at low temperatures, it would have serious detrimental effects on reinforced concrete behavior.
While the research conducted on steel reinforcement is not exhaustive, Filiatrault and Holleran (2001) have conducted some experiments that are relevant to this research. They tested rebar specimens while varying the temperature and the strain rate. They found that the increase in strain rate (up to 0.1 s$^{-1}$) and the decrease in temperature (down to $-40^\circ$C) increased the yield strength and the ultimate strength of the specimens by about 20% and 10%, respectively. However, their most important finding was that the ultimate tensile strain did not decrease significantly at the high strain rate and low temperature. This shows that the cold temperatures did not decrease the ductility of the reinforcement.

While these findings are important, what remains to be seen is the cyclic behavior of reinforcement. According to the author’s knowledge, no one has conducted experiments on the cyclic behavior of steel reinforcement at low temperatures.

1.7 Conclusion

The preceding literature review has discussed the research that has been conducted on steel and concrete materials at low temperatures. Many agree that there is more research necessary in order to have comprehensive knowledge of the behavior of
these materials at low temperatures. Some of the important findings are that concrete and steel increase in strength at low temperatures. These increases in strength could have the impact of increasing the moment capacity of structural members and thereby possibly increasing the shear demand and offsetting the hierarchy of strength if they are not taken into account.

The previous literature review also showed that the ductility of steel at low temperatures varies depending on the test, and more research needs to be conducted in this area. Charpy V-notch tests indicate a loss of ductility for V-notched specimens, while research conducted by Filiatrault and Holleran (2001) indicates that steel reinforcement does not lose ductility under monotonic loading.
2 Material Testing

The purpose of the material testing program was to determine the material properties so that they can be used to help interpret the behavior of the columns. A total of 24 concrete cylinders were tested having dimensions of 4 inches (101.6 mm) in diameter and 8 inches (203.2 mm) tall, while 15 reinforcing steel specimens were tested using #5 rebar. The steel tested was taken from the same shipment of A706 steel used to reinforce the columns, while the cylinders were cast using the same batch of concrete used to cast the columns. All of the tests were conducted using a 220 kip (978.6 kN) MTS universal testing machine and an MTS Testar hydraulic control system. The universal testing machine is shown in Figure 2.1 for cylinder testing and in Figure 2.2 for reinforcement testing.

![Figure 2.1 MTS Universal Testing Machine Prepared for Concrete Cylinder Testing](image)
2.1 Steel Reinforcement Testing

Of the 15 reinforcement tests, 2 were conducted cyclically and 13 were conducted monotonically under varying temperatures and displacement rates. During the cyclic testing, the MTS universal testing machine was not capable of putting the reinforcement in compression, and the machine buckled during the tests. Therefore, extensive testing could not be conducted, and only the monotonic tests will be discussed in detail here. Strain was recorded using an MTS extensometer with a 2-inch (50.8 mm) gage length, while the distance between the grips of the MTS was 4 inches (101.6 mm). The primary displacement rates used for the monotonic tests were 0.04 in/s (1.016 mm/s) and 0.004 in/s (0.1016 mm/s), which correspond approximately to strain rates of 0.01 s\(^{-1}\) and 0.001 s\(^{-1}\), respectively. One test was also conducted at a displacement rate of 0.0004 in/s. A variety of strain rates were chosen so that the strain rate effects could be evaluated, and
these particular strain rates were chosen because the highest strain rate corresponds to
earthquake type strain rates (Bischoff and Perry 1991).

All of the reinforcement tests were conducted at two approximate temperatures,
the ambient temperature of the lab (about 22°C) and the temperature of dry ice. The low
temperature specimens were placed in a cooler with dry ice until they were ready to test,
and then each individual bar was quickly removed from the cooler and placed into the
grips of the MTS; then the test was conducted as quickly as possible. The temperature
was recorded continually during each test using a thermocouple placed on the bar. The
bars were not insulated, so the temperature did increase during the test. The impact that
this had on the results of the tests will be discussed below.

During the initial tests at low temperatures, the specimens failed at the grip of the
MTS, outside of the extensometer gage length. It is believed that the specimens failed at
this location because the contact of the bar with the grip rapidly increased the temperature
at this portion of the bar. Since the strength of steel decreases with increased temperature
(which the literature review has shown and the results of the tests will also demonstrate),
the reinforcement therefore failed sooner at the grip. In order to overcome this, three
specimens were machined to reduce the diameter of the bar at the center, so that the
failure would occur within the extensometer gage length. These specimens were then
tested and the results were compared to all of the other specimens.

The results of the monotonic experiments are shown in Table 2.1. The
description of each specimen shows the initial temperature condition of the specimen as
well as whether or not the specimen was machined. The data shown includes the yield
stress, the ultimate stress, the strain at the ultimate stress, as well as the fracture strain.
For the low temperature specimens, the temperature at each significant event is recorded in the table next to the data.

Table 2.1 Data from Monotonic Reinforcing Steel Tests (1 ksi = 6.89 MPa)

<table>
<thead>
<tr>
<th>Specimen Description</th>
<th>Strain Rate s⁻¹</th>
<th>Yield Stress (ksi)</th>
<th>Ultimate Stress (ksi)</th>
<th>Strain at Ultimate Stress</th>
<th>Fracture Strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>lab temperature</td>
<td>0.01</td>
<td>75.5</td>
<td>102.78</td>
<td>0.1132</td>
<td>0.2318</td>
</tr>
<tr>
<td>lab temperature</td>
<td>0.01</td>
<td>75.4</td>
<td>102.48</td>
<td>0.1107</td>
<td>0.2432</td>
</tr>
<tr>
<td>dry ice, machined bar</td>
<td>0.01</td>
<td>---</td>
<td>113.16 (-18.2°C)</td>
<td>0.0999</td>
<td>0.2107 (-15.4°C)</td>
</tr>
<tr>
<td>dry ice, machined bar</td>
<td>0.01</td>
<td>87.1 (-33.2°C)</td>
<td>114.63 (-30.3°C)</td>
<td>0.1062</td>
<td>0.2172 (-25.3°C)</td>
</tr>
<tr>
<td>dry ice</td>
<td>0.01</td>
<td>77.2 (-19.8°C)</td>
<td>100.60 (-15.5°C)</td>
<td>0.0767</td>
<td>---</td>
</tr>
<tr>
<td>dry ice</td>
<td>0.01</td>
<td>81.8 (-22.3°C)</td>
<td>106.09 (-16.4°C)</td>
<td>0.0755</td>
<td>---</td>
</tr>
<tr>
<td>lab temperature</td>
<td>0.001</td>
<td>73.8</td>
<td>101.23</td>
<td>0.1179</td>
<td>0.2352</td>
</tr>
<tr>
<td>lab temperature</td>
<td>0.001</td>
<td>73.2</td>
<td>100.38</td>
<td>0.1082</td>
<td>0.2372</td>
</tr>
<tr>
<td>dry ice, machined bar</td>
<td>0.001</td>
<td>79.0 (-21.5°C)</td>
<td>109.26 (-1.7°C)</td>
<td>0.1032</td>
<td>0.2354 (18.2°C)</td>
</tr>
<tr>
<td>dry ice</td>
<td>0.001</td>
<td>78.0 (-18.1°C)</td>
<td>104.09 (11.4°C)</td>
<td>0.0938</td>
<td>---</td>
</tr>
<tr>
<td>dry ice</td>
<td>0.001</td>
<td>73.3 (-20.6°C)</td>
<td>97.76 (13.9°C)</td>
<td>0.1074</td>
<td>---</td>
</tr>
<tr>
<td>dry ice</td>
<td>0.001</td>
<td>73.3 (-10.9°C)</td>
<td>97.94 (12.5°C)</td>
<td>0.0990</td>
<td>---</td>
</tr>
<tr>
<td>lab temperature</td>
<td>0.0001</td>
<td>68.9</td>
<td>95.75</td>
<td>0.1195</td>
<td>0.2383</td>
</tr>
</tbody>
</table>

Note: Temperature at the point of interest is indicated in parentheses.

Figure 2.3 and Figure 2.4 show graphically what is given in Table 2.1. Figure 2.3 shows all of the stress-strain responses at the strain rate of 0.01 s⁻¹. Since the temperature of the specimens varied during each test, the specimen temperatures are shown on the graph at yielding, ultimate strength, and fracture, with arrows pointing to each point. Figure 2.4 shows the stress-strain responses at the strain rate of 0.001 s⁻¹, and the temperatures are shown in the same manner as in Figure 2.3. The solid lines showing the low temperature, un-machined bars in Figure 2.3 and Figure 2.4 drop dramatically because the specimens underwent necking and fracture outside of the extensometer. As the load drops while the specimen necks outside of the extensometer, the nominal stress decreases within the gage length, which results in the drastic elastic unloading shown in the figure. Even though these specimens do not show the complete stress-strain behavior, they accurately show the behavior up to necking.
Figure 2.3 Stress-Strain Responses of Reinforcement at the Strain Rate of 0.01 s\(^{-1}\) and Various Temperatures (1 ksi = 6.89 MPa)

Figure 2.4 Stress-Strain Responses of Reinforcement at the Strain Rate of 0.001 s\(^{-1}\) and Various Temperatures (1 ksi = 6.89 MPa)
One can make several observations from Table 2.1, Figure 2.3, and Figure 2.4, including the fact that the yield stress and the ultimate stress both increase at low temperatures, as discussed in the literature review. Another observation is that the onset of yielding is less defined for the low temperature specimens. This can be attributed to the fact that the temperature varied across the cross-section of the bar, and therefore the yield stress varied within the cross-section. The outside of the bar increased in temperature during the test because it was exposed to the warm air of the lab, while the center of the bar was cooler. As a result, at the center of the cross-section, the onset of yielding occurred later than the outer portion of the cross-section, and thus the yield point is less defined than if the temperature of the cross-section were uniform.

What these graphs demonstrate is that the reinforcement behaved in a manner that could have detrimental effects during an earthquake. Three of the low temperature specimens yielded at stresses greater than the acceptable limit of 78 ksi (537.4 MPa) established for A706 steel (Priestley et. al. 1996). This could upset the hierarchy of strength if it is not taken into account during the design. There is also a noticeable decrease in the fracture strain at low temperature, as observed in the machined bars that were tested at low temperature. At a strain rate of 0.01 s\(^{-1}\), the decrease in fracture strain at the low temperatures is 9.9%, while the decrease in the strain at ultimate stress is 7.9%. These changes are less dramatic at a strain rate of 0.001 s\(^{-1}\) due to the fact that the bar warmed up during the test because of the slower strain rate. This decrease in fracture strain at low temperature contradicts the findings of Filiatrault and Holleran (2001), who found that there was no significant decrease in fracture strain at low temperatures. The observed decrease in ductility appears to be consistent with the results of the Charpy V-
notch test, although the reductions in ductility under monotonic loading are perhaps not as dramatic as the loss of ductility under the impact testing. The results of these reinforcement tests suggest an increase in strength and a decrease in ductility at low temperatures for the reinforced concrete member experiments.

While the temperature of the specimen impacted its response, the strain rate also affected the response in a similar manner. As the strain rate increased, the yield stress and ultimate stress increased, as can be seen in Figure 2.5, which shows three specimens tested at ambient lab temperature at 3 different strain rates. Figure 2.6 shows the effect of strain rate on the material response at low temperature. The difference in ultimate strain can be attributed to the increase in temperature during the test of the specimen at the slower strain rate, as mentioned above.
Figure 2.5 Stress-Strain Responses of Reinforcement at Ambient Lab Temperature and Varying Strain Rates (1 ksi = 6.89 MPa)

Figure 2.6 Stress-Strain Responses of Reinforcement at Low Temperature and Varying Strain Rates (1 ksi = 6.89 MPa)
2.2 **Concrete Cylinder Tests**

Of the 24 concrete cylinder tests, 8 were tested at ambient laboratory temperature, 9 were tested at a temperature of approximately –20°C, and 7 were tested at approximately –40°C. All of the tests were conducted within a one-week period one year after the columns were cast, and the specified concrete strength at casting was 4 ksi (27.6 MPa). The low temperature specimens were placed inside of the NCSU environmental chamber (more information on the environmental chamber will follow in section 3.1) in order to cool them to the appropriate temperature, and fiberglass insulation was wrapped around them with duct tape to keep them cold during each test. A thermocouple was placed in the center of each cylinder tested at low temperature during casting so that the temperature could be monitored. Once each specimen was ready for testing, it was removed from the environmental chamber and placed in the MTS universal testing machine, and the test was conducted.

Due to the fact that some of the specimens had thermocouples extending from them, the top and bottom of the specimens could not be ground to get a smooth contact surface for testing. Therefore, all of the specimens were capped with a thin layer of hydrostone that served the purpose of creating a smooth, level surface onto which the MTS could apply the load. Displacement was recorded with 4 one-inch (25.4 mm) linear potentiometers that were placed on the bottom of the compression plate of the MTS and held in place by a magnetic base. Figure 2.7 shows one of the four potentiometers. These 4 potentiometers were averaged to determine displacement, which was then divided by the total height of each specimen in order to get strain. Figure 2.8, which shows the potentiometer readings for cylinder specimen 10, helps to demonstrate the validity of taking the average of the 4 potentiometers to get strain. All 4 of the
potentiometer readings are within 0.01 of an inch of each other for the entire test, and all of the other tests were similar.

Figure 2.7 Picture of Linear Potentiometer Used to Record Displacement for the Concrete Cylinder Tests
Of the 24 specimens, 10 were tested monotonically at a strain rate of either 0.001 \( \text{s}^{-1} \) or 0.0001 \( \text{s}^{-1} \), and the results are presented in Table 2.2. This table contains the strain rate of each specimen, the initial testing temperature, and the ultimate stress. As one can see, as the temperature decreases, there is a clear increase in concrete strength. In addition to the strength values, this table also shows the strain at \( f'c \), the strain at 0.5\( f'c \) on the degrading portion of the stress-strain curve, and the strain at 0.2\( f'c \) on the degrading portion of the stress-strain curve. These strain values are compared to show the strain capacity of the concrete at various temperatures. Notice that the specimens tested at \(-40^\circ\text{C}\) have a much higher strain at the maximum stress than the other specimens, while the strain increase at 0.5\( f'c \) and 0.2\( f'c \) is very small compared to the strain at \( f'c \). This indicates a brittle failure in the specimens tested at \(-40^\circ\text{C}\)—in fact, some of the specimens failed in a violent manner before degrading to 0.5\( f'c \). The strain...
values given with an asterisk indicate the specimens that failed abruptly, and the values
given are the last value of strain recorded before the abrupt failure. The places in the
table where a dashed line is given for the strain values indicate where the test was
stopped before it degraded to these values of stress.

Table 2.2 Data from Monotonic Concrete Cylinder Tests (1 ksi = 6.89 MPa)

| Cylinder ID Number | Strain Rate s\(^{-1}\) | Temperature °C | f'c ksi | strain at 0.5f'c strain at 0.2f'c strain at 0.5f'c after softening strain at 0.2f'c after softening |
|--------------------|------------------------|----------------|--------|------------------|------------------|------------------|------------------|
| 1                  | 0.0001                 | 22             | 3.51   | 0.0046           | ---              | ---              |
| 2                  | 0.0001                 | 22             | 5.72   | 0.0057 0.0064    | 0.0083           |
| 3                  | 0.001                  | 22             | 5.67   | 0.0051 0.0063    | 0.0096           |
| 9                  | 0.0001                 | -23            | 5.98   | 0.0053 0.0065    | ---              |
| 10                 | 0.0001                 | -23            | 6.94   | 0.0050 0.0062    | 0.0072           |
| 11                 | 0.001                  | -24            | 6.65   | 0.0055 0.0065    | 0.0091           |
| 22                 | 0.0001                 | -21            | 5.26   | 0.0055 0.0079    | ---              |
| 17                 | 0.0001                 | -41            | 9.43   | 0.0069 0.0075*   | ---              |
| 18                 | 0.0001                 | -40            | 7.75   | 0.0064 0.0078    | 0.0082           |
| 19                 | 0.001                  | -40            | 9.69   | 0.0069 0.0076*   | ---              |

*A Abrupt failure before degrading to 0.5f’c

The following figures graphically represent the data that are presented in Table 2.2. Figure 2.9 shows specimens 2, 10, and 17, which were tested at the three different temperatures under the same strain rate, 0.0001 s\(^{-1}\). They demonstrate the increase in strength at low temperatures and the differences in strain that were discussed in the preceding paragraph. Moreover, this graph shows the increase in modulus of elasticity as the temperature decreases. Figure 2.10 demonstrates these same phenomena with specimens 3, 11, and 19, which were tested at three different temperatures but the same strain rate of 0.001. Figure 2.11 shows specimens at the same temperature, 22°C, but two different strain rates. The graphs shown are very similar, and the specimen tested at the slower strain rate is actually a little stronger than the specimen tested at the faster strain...
rate. This is contrary to what one would expect, and it can be attributed to the variable nature of concrete. Figure 2.12 shows specimens tested at \(-20^\circ\)C under two different strain rates, and one can observe a slight increase in strength at the faster strain rate over the average strength of the specimens tested at the slower strain rate. Figure 2.13 shows specimens tested at \(-40^\circ\)C under two different strain rates, and the specimen with the faster strain rate has a higher strength.

![Stress-Strain Responses](image)

Figure 2.9 Stress-Strain Responses of Concrete at a Strain Rate of 0.0001 s\(^{-1}\) (1 ksi = 6.89 MPa)
Figure 2.10 Stress-Strain Responses of Concrete at a Strain Rate of 0.001 s\(^{-1}\) (1 ksi = 6.89 MPa)

Figure 2.11 Stress-Strain Responses of Concrete at 22\(^{\circ}\)C and Varying Strain Rates (1 ksi = 6.89 MPa)
Figure 2.12 Stress-Strain Responses of Concrete at -20°C and Varying Strain Rates (1 ksi = 6.89 MPa)

Figure 2.13 Stress-Strain Responses of Concrete at -40°C and Varying Strain Rates (1 ksi = 6.89 MPa)
In addition to the monotonic tests, six specimens were tested under displacement control cyclic loading with increasing amplitude. Each specimen was loaded to a target compressive strain of 0.0005, and then the specimen was unloaded to zero strain. Then the specimen was loaded to 0.001 compressive strain and back to zero, and so on until the specimen failed. The target strains in the incremental steps were 0.0005, 0.001, 0.002, 0.003, etc. The results of these tests are presented in Table 2.3, and the same data are given that were presented in Table 2.2. One can observe the same trends that were present in the monotonic tests. Although the responses of these specimens are not shown graphically in the body of this paper, they are given in Appendix A-1.

The remaining specimens were tested in force control in a cyclic manner. Each specimen was given a target load, and the specimen was cycled between this load and back to 1 kip (4.45 kN) until failure. For example, a specimen was loaded to 53 kips (235.9 kN), or a stress of approximately 4.2 ksi (28.9 MPa; the actual stress varied slightly depending on the exact area of the cylinder). Then, the load was reduced to 1 kip; the load was then brought back to 53 kips, and so on until the specimen failed. The results of these tests are presented in Table 2.4, and the responses are shown in the appendix. Specimens 6, 7, 14, and 15 were all cycled using the maximum load of 53 kips, and this load was chosen because it is approximately 75% of the ultimate failure load for the cylinders at 20\(^\circ\)C. Specimens 8, 16, and 24 were cycled at a load of 60 kips (267 kN; approximately 4.85 ksi, or 33.4 MPa) because this is approximately 85% of the ultimate failure load for the cylinders at 20\(^\circ\)C. Specimen 23 was cycled at a force of 88 kips (391.6 kN), or 7.18 ksi (49.5 MPa), because this corresponded to approximately 75% of the ultimate stress of the concrete at –40\(^\circ\)C. Some of the low temperature specimens
were not cycled to failure because of the fact that as time progressed during the test, the temperature of the specimen began to increase. Therefore, some of the specimens were tested monotonically after a certain number of cycles. The basic procedure operated in this manner: the original cyclic test was conducted, but as the test went on, the temperature increased; the test was stopped and the specimen was placed in the environmental chamber to return it to its initial temperature; and then the monotonic test was conducted.

Table 2.3 Results from Cyclic Displacement-Control Tests on Concrete Cylinders (1 ksi = 6.89 MPa)

<table>
<thead>
<tr>
<th>Cylinder ID Number</th>
<th>Strain Rate s⁻¹</th>
<th>Temperature °C</th>
<th>f'c ksi</th>
<th>strain at f'c</th>
<th>strain at 0.5f'c after softening</th>
<th>strain at 0.2f'c after softening</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>0.0001</td>
<td>22</td>
<td>4.23</td>
<td>0.0040</td>
<td>0.0058</td>
<td>---</td>
</tr>
<tr>
<td>5</td>
<td>0.001</td>
<td>22</td>
<td>6.25</td>
<td>0.0051</td>
<td>0.0061</td>
<td>0.0075</td>
</tr>
<tr>
<td>12</td>
<td>0.0001</td>
<td>-24 to -19</td>
<td>5.33</td>
<td>0.0048</td>
<td>0.0061</td>
<td>---</td>
</tr>
<tr>
<td>13</td>
<td>0.001</td>
<td>-24</td>
<td>6.24</td>
<td>0.0049</td>
<td>0.0061</td>
<td>---</td>
</tr>
<tr>
<td>20</td>
<td>0.0001</td>
<td>-41 to -21</td>
<td>5.51</td>
<td>0.0075</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>21</td>
<td>0.001</td>
<td>-42</td>
<td>9.33</td>
<td>0.0070</td>
<td>0.0074*</td>
<td>---</td>
</tr>
</tbody>
</table>

*Abrupt failure before degrading to 0.5f'c

Table 2.4 Results from Cyclic Force-Control Tests on Concrete Cylinder (1 ksi = 6.89 MPa)

<table>
<thead>
<tr>
<th>Cylinder ID Number</th>
<th>Strain Rate s⁻¹</th>
<th>Initial Temperature °C</th>
<th>Cycling stress ksi</th>
<th># cycles at cycling stress</th>
<th>Monotonic f'c ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>0.001</td>
<td>22</td>
<td>4.18</td>
<td>3</td>
<td>---</td>
</tr>
<tr>
<td>7</td>
<td>0.001</td>
<td>22</td>
<td>4.28</td>
<td>153</td>
<td>---</td>
</tr>
<tr>
<td>8</td>
<td>0.0001</td>
<td>22</td>
<td>4.69</td>
<td>1</td>
<td>---</td>
</tr>
<tr>
<td>14</td>
<td>0.0001</td>
<td>-20</td>
<td>4.26</td>
<td>66</td>
<td>5.97</td>
</tr>
<tr>
<td>15</td>
<td>0.001</td>
<td>-20</td>
<td>4.3</td>
<td>800</td>
<td>7.77</td>
</tr>
<tr>
<td>16</td>
<td>0.001</td>
<td>-20</td>
<td>4.85</td>
<td>239</td>
<td>---</td>
</tr>
<tr>
<td>23</td>
<td>0.001</td>
<td>-40</td>
<td>7.18</td>
<td>138</td>
<td>---</td>
</tr>
<tr>
<td>24</td>
<td>0.001</td>
<td>-40</td>
<td>4.85</td>
<td>239</td>
<td>10.82</td>
</tr>
</tbody>
</table>
2.3 Conclusions

The preceding material research has reaffirmed much of what was stated in the literature review. For steel, an increase in the strain rate and a decrease in the temperature have the impact of increasing the yield stress and ultimate stress without impacting the modulus of elasticity. In regard to concrete, low temperatures cause a significant increase in strength, and the modulus of elasticity also increases slightly. A loss in ductility can be seen in both materials, as the steel specimens suffered a reduction in the ultimate strain and the concrete specimens at −40°C failed abruptly without a softening portion of the stress-strain curve in most cases.

All of the numerical data gathered from the material testing were used to determine theoretical moment-curvature responses for the specimens, and from there the force-deformation responses were calculated. The theoretical responses were developed using the following material properties: 70 ksi (482.3 MPa) steel and 5.5 ksi (37.9 MPa) concrete at 20°C; 78 ksi (537.4 MPa) steel and 7 ksi (48.2 MPa) concrete at −20°C; and 87 ksi (599.4 MPa) steel and 9.5 ksi (65.5 MPa) concrete at −40°C. The theoretical responses were derived using the computer program developed by King et al. (1986) with the material model developed by Mander (1988). As will be shown later, the theoretical responses for the columns overestimate the actual experimental strength of the columns (see the force deformation curves for the specimens, Figure 4.13, Figure 4.29, Figure 4.47, and Figure 4.63). This is due in part to the fact that the Mander model does not exactly predict the behavior of the steel that was used in the columns. The ultimate stress of steel in the Mander model is 1.5fy, whereas the material tests suggest that the ultimate stress for the steel used during experimentation was actually between 1.3fy and 1.4fy. In spite of this difference, it was not deemed necessary to replace the Mander model with
the actual material properties in a theoretical analysis. Any differences between the theoretical predictions and the actual experimental results simply needed to be taken into account. The theoretical force deformation monotonic responses are shown in Figure 2.14 calculated using the material properties at 23\(^0\)C, -20\(^0\)C, and -40\(^0\)C. These responses demonstrate that, in theory, the strength at the member level increases as the temperature decreases.

Figure 2.14 Theoretical Monotonic Force-Deformation Responses Calculated Using the Material Properties at Various Temperatures (1 kip = 4.45 kN; 1 in = 25.4 mm)
3 Test Setup, Instrumentation, and Test Specimen Design

This chapter contains information on the test setup, the instrumentation, and the test specimen design. The three most important aspects of the test setup were the environmental chamber, the actuator extension, and the support for the footing.

3.1 Environmental Chamber

In order to accommodate the need for temperatures down to –40°C, the tests were conducted with the specimens inside of a Thermotron environmental chamber. The environmental chamber is a model WP-1512-CHM-25-25 with a temperature range of –68°C to 85°C and having dimensions of 24’3” long (7.39 m), 6’10” tall (2.08 m), and 6’10” wide. The specimens were placed inside the chamber; they were prepared for the testing regimen; and then they were allowed to cool until they reached the desired temperature. Once the specimens arrived at the proper temperature, the tests were conducted.

The dimensions of the chamber made it necessary that the specimens be tested while lying in a horizontal position rather than standing vertically erect, as Figure 3.1 shows. The load was applied vertically in the absence of any axial load, and therefore the members cannot literally be called columns. However, at times they will be referred to as columns because this is the typical application of the members. The environmental chamber is shown in Figure 3.2. Notice that there is a loading frame that straddles the chamber, and a 110 kip (489.5 kN) hydraulic actuator (MTS model 244.41) extends down from the frame. An MTS load cell, model 661.23E-01, was attached to the actuator in order to record the force response.
Figure 3.1 Test Setup

Test Setup
(loading frame not shown for clarity)
In the roof of the chamber was a hole six inches (152.4 mm) in diameter that allowed for an actuator extension to protrude into the chamber from the outside. During the testing of the first specimen, the hole proved to be too small to allow for rotation of the actuator extension, so the hole was enlarged to 12 inches (304.8 mm) in diameter. More will be said about this in the review of the experiments.

3.2 Actuator Extension

Due to the fact that the actuator could not properly operate at low temperatures inside the chamber, accommodations had to be made for the actuator to apply loading from outside of the chamber. As a result, an actuator extension was fabricated to screw into the actuator on one end and to clamp the end of the column on the other. The actuator extension is shown in Figure 3.3, and it consisted of two major components. The first component is the long tube section that screwed into the actuator and extended down through the roof of the chamber (hereafter this will be referred to as the “tube
extension”), while the second component was a collar that fit snugly around the column (hereafter referred to as the “collar”). The tube extension connected to the collar with a pin that was 1.5 inches (38.1 mm) in diameter, and the two collar pieces were held snugly around the column by four 1-inch (25.4 mm) diameter pieces of threaded rod. This setup is shown in Figure 3.4. The tip of the tube extension was a 5.5 inch (139.7 mm) long, 2-inch (50.8 mm) diameter piece of threaded rod (12 threads per inch) that screwed into the load cell of the actuator. This was welded to four triangular flares and a flat plate, while the flat plate was welded to a 49-inch long tube section. The tube extension had a 4-inch (101.6 mm) inside diameter with a half-inch (12.7 mm) wall thickness. A short 5-inch (127 mm) inside diameter steel tube sleeve was placed overtop of the long tube at the opposite end from the threaded rod, and a hole was drilled through the two tubes to accommodate the pin. The sleeve was placed on the tube extension to increase the bearing capacity of the hole in the tube section at the location of the pin.

Figure 3.3 The Actuator Extension
The collar of the actuator extension consisted of two pieces that mirrored each other. Each piece had three parallel plates that were 24 inches by 9 inches by 1/2 inch (610 X 229 X 12.7 mm) thick. The plates were cut out with an 18.75-inch (476 mm) diameter to conform to the column, and a material called Lexan was placed between the plates and the bearing surface on the column. The three plates were spaced 2 inches (50.8 mm) apart on center, and they welded to a top plate that was 24 inches by 6 inches by 1/2 inch (610 X 152 X 12.7 mm) thick, having four holes drilled to accommodate the threaded rod. Perpendicular to the three plates were two 1.5-inch by 6-inch by 3/8-inch (38.1 X 152 X 9.5 mm) thick plates that were welded to the collar, and four pieces of wood cut from 4x4 sections were placed between the three parallel plates to increase their stability. On top of the collar, two 6 inch by 6 inch by 1 inch (152 X 152 X 25.4 mm) thick plates extended so that the collar could connect to the tube extension.
3.3 The Footing Support

The footing of the column rested on a steel base plate that had 4 squat legs, with a layer of hydrostone in between the footing and the plate, as shown in Figure 3.5. The total height of the base plate with its legs was 12 inches (305 mm). The legs of the base plate then rested on 4 22-inch (559 mm) long steel tube sections (each tube section had a 5-inch {127 mm} inside diameter with 1/4 inch {6.4 mm} wall thickness.) that extended through the floor of the environmental chamber all the way down to the strong floor of the lab. The base plate could not rest on the floor of the environmental chamber as the floor was not designed to sustain such loads, thus necessitating the piers which rested on the strong floor. Expanding foam was applied between the tube sections and the floor of the environmental chamber in order to provide insulation and to stabilize the legs during the pre-stressing operation. Post-tensioning steel extended through the footings all the way down through the strong floor in order to secure the footing for loading. The post-tensioning consisted of four Dywidag post-tensioning bars with a diameter of 1-3/8 inches (35 mm), and they were stressed to approximately 45 kips (200 kN) per bar. In summary, a drawing of the test setup is shown in Figure 3.1, showing all of the major components of the setup.
Figure 3.5 The Footing Support
3.4 Instrumentation

The instrumentation used during the testing consisted of three thermocouples to monitor the temperature of the column, three string potentiometers used to measure the displacement profile, and 14 linear potentiometers used to determine the curvature profile. All of the instrumentation is shown in Figure 3.1. The three thermocouples were cast inside of the column at a distance of three feet from the footing, with one thermocouple located on the longitudinal steel. The other two thermocouples were placed on a #3 reinforcing bar, one at the center of the column and the other halfway between the center and the surface of the column. The location of the thermocouples in the cross-section is shown in Figure 3.6, and hereafter they will be referred to as TC1, TC2, and TC3, as designated in the figure. Another thermocouple, designated TC4, was located in the ambience of the environmental chamber.

![Figure 3.6 Location of Thermocouples within the Cross-Section](image)

Two of the three string potentiometers were located on opposite sides of the actuator extension, and one was placed at the end of the last gage of the linear potentiometers. The string potentiometers used were Patriot sensors, and they ranged from 15 to 25 inches (381 to 635 mm) in length. They were extended using a leader, and
then attached to the specimen by an aluminum angle that was screwed into a small block of plywood that in turn was glued to the surface of the column.

Ten Duncan 6510 1-inch (25.4 mm) linear potentiometers were aligned on the top and the bottom of the column consecutively, with a gage length of approximately 6.75 inches (171.5 mm). The potentiometers were screwed onto 1/4 inch threaded studs that were glued into the column with epoxy, and the landing point for each potentiometer was a small aluminum angle that was also attached to the threaded stud. This configuration is shown in Figure 3.7. The bottom of the first gage was placed at about 3.6 inches (91.4 mm) from the face of the footing. They were not placed at the face because the footing was small, and the interface between the footing and the base plate would not allow for a potentiometer placement. Instead, four Duncan 6515 1.5-inch (38.1 mm) potentiometers were placed on the sides of the column landing on the footing, two on top and two on the bottom. These four potentiometers were used to measure the curvature at the very base of the column for the final three test specimens, and their configuration (on the topside of the specimen) is shown in Figure 3.7. The small drawing in the corner of the photo shows the view of the photo in relation to the entire specimen. These potentiometers were not used during the first test, but only for the low temperature specimens.

The data acquisition system used during the testing was an Optim Megadac 3415AC, and TCS for Windows was used to record the data at 1 scan per second. 808FB-1 cards were used to read the string potentiometers and the curvature potentiometers with an excitation of 10 volts and a gain of 1. The thermocouples were wired using an 816TC card with a gain of 100.
Figure 3.7 Location of the Linear Potentiometers
3.5 Test Specimen Design

The test specimens used for the experimentation consisted of four identical columns with their respective footings. The columns were 18 inches (457 mm) in diameter and 72 inches (1829 mm) long, while the footings were 42 inches by 42 inches (1067 mm), with a depth of 20 inches (508 mm). Due to the fact that the columns were horizontal during the application of the loading, the columns extended out of the 20-inch face, as shown in Figure 3.8. The complete design of the test specimen is shown in Figure 3.9.

![Figure 3.8 Column Design (1 in = 25.4 mm)](image)

3.6 Column Design

The columns were designed to be lightly reinforced, with a 0.97% reinforcement ratio. The light reinforcement is typical of seismic design because a smaller reinforcement ratio will usually allow a larger ductility capacity. The reinforcement consisted of 8 #5 bars equally spaced throughout the column. The bars were 8 feet (2440 mm) long with an 18-inch (457 mm) bend, and they were all A706 seismic grade steel. The bends of the two bars on top and bottom of the specimen were turned inward towards
each other, while the other bends of the other six bars were turned outward into the side of the footing. Cover for the bars inside the column was 1 inch (25.4 mm), while the transverse reinforcement consisted of a #3 spiral with 2.5 inch (63.5 mm) spacing. This resulted in a transverse steel ratio of 1.1%. The transverse reinforcement was tied from the end of the column all the way down into the footing to ensure proper confinement in the footing.
Figure 3.9 Test Specimen Design (1 in = 25.4 mm)
3.7 **Footing Design**

In order to understand the footing design, it is important to understand the constraints of the specimen design. Due to the fact that the tests were conducted inside of the environmental chamber, the specimens had to be moved in and out with a forklift. As a result, the specimens had to weigh less than 4.7 kips (20.9 kN), since this is the capacity of the forklift at the NC State Constructed Facilities Lab. This is the main reason why the footing was so small.

The primary reinforcement of the footing consisted of 18 #5 bars running parallel to the column. Ten of the reinforcing bars were on the topside of the footing, while eight of the bars were on the bottom. Each of the bars was 40 inches (1016 mm) long and had a bend on each end of 16 inches (406 mm). The bottom of the footing had an additional four #3 bars at the outside edges (two 28-inch (711 mm) bars were lap spliced on each side). In the orthogonal direction were 16 bars running perpendicular to the column, with 8 each on the top and bottom. The bars were #5 with a length of 40 inches (1016 mm) and a bend of 18 inches (457 mm).

Inside of the footing were four PVC tubes that allowed for the post-tensioning of the specimen to the strong floor. The tubes were two inches (50.8 mm) in diameter and were spaced in a square of 36 inches by 36 inches (914 mm) on center, which matched the pattern of holes in the strong floor. The strength of the concrete in the footings was tested using three 4 inch (101.6 mm) diameter by 8 inch (203.2 mm) tall cylinders, and the strength of the cylinders was determined to be 7382 psi (50.9 MPa), 7244 psi (49.9 MPa), and 6620 psi (45.6 MPa), respectively.
4 Experimentation and Results

This chapter presents the results of the four specimen experiments. Each of the experiments is reviewed in detail, and each column is evaluated in comparison with the other columns.

4.1 Loading History

The load history of the testing routine is shown in Figure 4.1 and Figure 4.2. The first four cycles of each test were in force control (Figure 4.1), and this brought the specimen up to the first yield point at room temperature. The specimen was cycled at amplitudes of 1/4 of first yield, 1/2 of first yield, 3/4 of first yield, and finally at first yield. After this point, the loading was transferred to displacement control (Figure 4.2).

![Figure 4.1 Force Control Portion of the Load History (1 k = 4.45 kN)](image-url)
The specimens were cycled three times at each amplitude of displacement, beginning at the equivalent yield displacement (ductility 1). The next displacement was a ductility of 1.5, followed by ductility 2, 3, 4, 5, 6, and 8 in consecutive order until the failure of the specimen. Some slight variations in the loading occurred in test specimen 1, but this will be discussed in detail below. The test was conducted such that after each cycle was completed, the specimen was held at the target force or displacement, and qualitative observations of the specimen were made. An additional note is that the specimens had a dead load moment of 5.0 kip-feet (6.8 kN-m) at the base due to the cantilever weight of the column, but the load history does not take this into account by placing a larger applied force in the pull (upward) direction than the push (downward) direction. The applied forces in the two directions are the same.
4.2 Specimen 1 Experiment Results

Specimen 1 was tested at the ambient laboratory temperature of about $23^\circ$C on March 3, 2004. Figure 4.3, Figure 4.4, and Figure 4.5 show the temperature readings inside the column during the test as a function of column displacement.

The test began in force control, and the first amplitude of the loading was 2.73 kips (12.1 kN) in the push direction (downward). This corresponded to one-fourth of the theoretical first yield point. At this point there was a tremendous degree of noise in the system, so the specimen was unloaded and the actuator was tuned so as to remove the noise. After tuning the actuator, the force deformation curve was much smoother, and the test continued by returning to 2.73 kips (12.1 kN) in the push direction. The specimen was returned to a force of zero, and then a load of 2.73 kips (12.1 kN) was placed in the pull direction (upward). At this point the previously stated load history of Figure 4.1 was resumed.

Figure 4.3 Longitudinal Steel Temperature for Specimen 1, "TC1"
Figure 4.4 Temperature between TC1 and TC3, "TC2"

Figure 4.5 Temperature at the Center of the Cross-Section of Specimen 1, "TC3"
At each amplitude of loading, the specimen was held in place so that it could be visually examined. The first hairline cracks appeared on the specimen at a force of approximately 5.5 kips (24.5 kN), located on the top of the specimen between 7 and 17 inches (178 and 432 mm) from the face of the footing. At the next load step, 3/4 of first yield, a crack formed between the face of the footing and the column. More cracks appeared up to a distance of 39 inches (990.6 mm) from the footing, and they were spaced at about 6 to 8 inches (152 to 203 mm) apart. At the next cycle, corresponding to the first yield force, the first hairline crack in the footing appeared, about 2.5 inches (63.5 mm) from the face of the footing.

At this point the force control portion of the load history was completed, and the displacement control portion of the load history began. The actual column displacement at the first yield force was 0.34 inches (8.6 mm; this corresponds to the first yield displacement), so the equivalent yield displacement was calculated using the equation

\[ \Delta_y = M_i(\Delta_y')/M_y' \]

Where \( M_i \) is the ideal moment capacity of the section, \( M_y' \) is the first yield moment of the section, and \( \Delta_y' \) is the first yield displacement. The equivalent yield displacement was determined to be 0.64 inches (16 mm), corresponding to a ductility of 1. This displacement was used to calculate the remaining levels of ductility for the load history.

After reaching the first loading amplitude in displacement control, we tuned the actuator again due to the static in the system. The cracks in the member extended up 48 inches from the footing, and more cracks formed in the footing. At the first cycle of
ductility 1.5, the first cracked approximately 7 inches (178 mm) from the footing opened up to a width of approximately 1 mm. By the third cycle, an even larger crack had formed at the footing face, as shown in Figure 4.6. This gave us a good indication that the footing would not fail (even though at this point there was cracking in the footing, as one can see in the photo), but that the plastic hinge would form at the column base. Prior to the test there was concern that failure would occur in the footing due to the fact that the footing is so small. However, these observations gave reasonable indications that the plastic hinge would form at the base of the column as desired. The drawing in the corner of the photo shows the view of the photo in relation to the entire specimen.

![Figure 4.6 Crack Formation on the Top of Specimen 1 during the Push Segment of Ductility 1.5, Cycle 3](image-url)
On the third cycle of ductility 2, two problems arose that forced the stopping of the test. Originally, there was a 3/8-inch (9.5 mm) neoprene layer between the collar of the actuator extension and the specimen, but the actuator extension sliced it through on one side. In addition, the actuator extension began to bear against the roof of the environmental chamber due to the rotation at the pin connection of the actuator extension. In order to solve these problems, a larger hole was cut in the roof of the environmental chamber, and Lexan was placed between the collar and the specimen in place of the neoprene.

After resolving these issues, testing of the first specimen was resumed on March 9, 2004. The loading frame was moved slightly to better accommodate the actuator extension in the hole of the roof of the chamber, which shortened the moment arm of the column. Originally the centerline of the actuator extension applied the load with a moment arm of 5.62 feet (67.44 inches; 1.713 m), while during the second portion of testing the moment arm was 5.417 feet (65 inches; 1.651 m). The moment arm of the column was kept constant at 65 inches for the rest of the experiments so that a direct comparison could be made more easily among the columns.

At the end of the testing on March 3, the column had a displacement of 0.35 inches (8.9 mm). When conducting the experimentation on March 9, this was not taken into account, so the remainder of the test was offset by 0.35 inches. This means that in the push direction, the actual displacements are 0.35 inches larger than the ductility levels stated in the load history. In the pull direction, the displacements are 0.35 inches smaller than the ductility levels stated in the load history. In the following description of the test,
whenever the displacements are given, these are the actual absolute displacements from
the origin of the column at the beginning of testing on March 3.

The first load step for the second round of testing was in the pull direction to a
displacement of 0.73 inches (18.5 mm) and a force of 13 kips (57.9 kN). This was a little
over half way to the third cycle of ductility 2. At this point everything was working well,
so we concluded the loading at ductility 2 and moved on to ductility 3. Figure 4.7 shows
the development of the cracking on the topside of the column at the first cycle of ductility
3. Some slight surface crushing occurred on the bottom of the column, which gradually
increased with the progression of the cycles. The level of crushing on the bottom is
shown at the first cycle of ductility 4 in Figure 4.8.

During the cyclic loading segments of ductility 4, new cracks formed up to 54
inches (1372 mm) from the footing, and slight crushing began to occur on the topside of
the column. The crack widths near the base of the column reached 1/8 of an inch (3.2
mm). Figure 4.9 shows the development of cracking at the first cycle of ductility 5,
which proceeded to become significant crushing by the third cycle. The rebar became
exposed, and the crushing extended from the footing face out 4.5 inches (114 mm). On
the first cycle of ductility 6, significant buckling occurred in the bottom bar of the
longitudinal reinforcement (shown in Figure 4.10), and very slight buckling also occurred
in the top bar. On the second cycle, the core concrete began to crush, such that the entire
circumference of the long steel on the bottom of the column was exposed. In addition,
the first crushing began to occur in the footing on the top of the specimen. The crushing
extended about 5 inches (127 mm) in both directions from the interface of the column
and the footing, although the crushing in the column was much deeper and more
significant. The crushing in the footing was only spalling of the cover, without significant crushing in the core.

Figure 4.7 Crack Formation of Specimen 1 during the Push Segment of Ductility 3, Cycle 1

Figure 4.8 Crushing of Concrete Cover at the Bottom of Specimen 1 during the push segment of Ductility 4, Cycle 1
Figure 4.9 Cracking and Flaking of Concrete on Specimen 1 during the Push Segment of Ductility 5, Cycle 1

Figure 4.10 Crushing of Concrete Cover and Buckling of Bottom Reinforcement of Specimen 1 during the Push Segment of Ductility 6, Cycle 1
On the final cycle at ductility 6 in the pull direction, the bottom bar ruptured (Figure 4.11), while the top bar ruptured on the return cycle, heading to the first cycle of ductility 8 in the push direction (Figure 4.12). The column was cycled one time at ductility 8, but by this time, the stiffness of the column degraded significantly, so the load was brought to zero and the test was concluded.

The force-deformation response for test specimen 1 is given in Figure 4.13, and the calculated monotonic response is compared with it. Remember that the calculated response overestimates the strength because of the difference between the material properties of the model and the actual material properties of the specimens, as mentioned during the discussion of the material tests in chapter 2. The cross-section of the specimen is shown in quadrant 2 of Figure 4.13, and the bars of reinforcement that ruptured are crossed out with an x. These bars are labeled consecutively in the order in which they ruptured. The bottom bar rupture is indicated in Figure 4.13 when a significant decrease in stiffness in the curve occurs during the third pull cycle of ductility 6. The largest displacement that the column had undergone up to this point was 3.62 inches (92 mm) in the pull direction and 4.14 inches (105 mm) in the push direction. The drop in strength of the curve in the first quadrant marks the point where the top bar ruptured just prior to arriving at ductility 8. The maximum force that the column sustained in the push direction was 23.08 kips (102.7 kN) during the first cycle of ductility 3, while the maximum force the column sustained in the pull direction was 22.6 kips (100.6 kN) during the first cycle of ductility 6. This means that the average ultimate strength of the column was about 22.84 kips (101.6 kN). In addition to strength behavior, the force-
deformation graph shows hysteresis loops with good damping characteristics, and more quantitative observations will be made about that in the final chapter.

Figure 4.11 Rupture of Bottom Reinforcement of Specimen 1 during the Push Segment of Ductility 6, Cycle 3

Figure 4.12 Crushing of the Concrete and Rupture of the Top Reinforcement of Specimen 1 during the Push Segment of Ductility 8, Cycle 1
Figure 4.13 Force-Deformation Response of Specimen 1 and Calculated Monotonic Response (1 kip = 4.45 kN; 1 in = 25.4 mm)

Figure 4.14 shows the specimen 1 backbone curves of the maximum force on the first, second, and third cycles of response (the point on the third cycle in the pull direction that looks out of place was the abbreviated ductility 2, cycle 3 displacement at the beginning of the second day of testing). This figure shows that there is some slight strength degradation on the second and third cycles, which is to be expected. The degradation in the push direction is greater than the degradation in the pull direction because of the offset of the loading. In the pull direction, the column did not undergo as great of a demand, and therefore the strength did not degrade as much. Figure 4.15 gives the displacement profile of the column at each level of ductility during the test. Figure 4.16 shows the moment-curvature of specimen 1 at a section 3.6 inches (91 mm) to 10.8 inches (274 mm) from the footing, and the theoretical moment curvature envelope is also
displayed. This data shows the test through ductility 5 cycle 3, as the data is no longer valid after this point because the landing angle of the linear potentiometer fell off of the column due to crushing. Figure 4.17 shows the curvature profile of specimen 1 during the first cycle of each displacement, and again, the bottom point is not shown after ductility 5 because the data points are not valid. More will be said about each of these graphs in the following sections as they are compared to the corresponding graphs of the other columns.

![Figure 4.14 Specimen 1 Backbone Responses (1 kip = 4.45 kN; 1 in = 25.4 mm)](image-url)

Figure 4.14 Specimen 1 Backbone Responses (1 kip = 4.45 kN; 1 in = 25.4 mm)
Figure 4.15 Specimen 1 Displacement Profiles (1 in = 25.4 mm)

Figure 4.16 Specimen 1 Moment-Curvature Response, 3.6 Inches to 10.8 Inches from the Face of the Footing (1 kft = 1.36 kNm; 1 in = 25.4 mm)
Figure 4.17 Specimen 1 Curvature Profiles (1 in = 25.4 mm)
4.3 *Specimen 2 Experiment Results*

Specimen 2 was placed in the environmental chamber on March 11, 2004. After the instrumentation was placed on the specimen, the calibration at low temperatures needed to be validated. The environmental chamber was set to $-23^\circ C$ on March 19, and the specimen was allowed to cool. After about four hours at this rate, the thermocouple on the longitudinal steel had reached $0^\circ C$, while the ambient temperature inside the chamber was still at $-23^\circ C$. This assured us that the instrumentation was at least at $0^\circ C$, and probably very close to the target temperature for the second test. At this point, the calibration of the string potentiometers were checked by simply taking them off of the specimen and letting them retract all the way to the ground. The calibration of the linear potentiometers was checked by placing a steel block between the pot and its landing point. In both cases (for the string potentiometers and the linear potentiometers), the same measurements were taken before the specimen was cooled and after the specimen was cooled, and the data were compared. In each case the values were within 1% of each other, demonstrating that the calibration was still effective at the cold temperature. Thus, no re-calibration at the cold temperatures was necessary.

On the morning of March 23, the environmental chamber again was set to about $-23^\circ C$ to cool the specimen. Data points were collected once every minute to show the pattern of the cooling of the column, given in Figure 4.18. As a review, TC1 was located on the longitudinal steel, TC3 was located in the center of the column, and TC2 was located directly in between TC1 and TC3. TC4 recorded the ambient temperature of the environmental chamber. The chamber was turned off that evening with the outside thermocouple reading $-22^\circ C$ and the center thermocouple reading $-20^\circ C$. By the next
morning the specimen was reading about \(-17^0\text{C}\) on the outside thermocouple and the core had a temperature of about \(-18^0\text{C}\). The chamber was turned on again and the specimen was allowed to cool for a couple of hours until it returned to \(-20^0\text{C}\), and then the testing began. Figure 4.19, Figure 4.20, and Figure 4.21 show the temperature of the column during the test as a function of column displacement.

Figure 4.18 Specimen 2 Temperature While Cooling
The specimen was loaded using the same loading history as shown in Figure 4.1 and Figure 4.2 even though the first yield displacement for column 2 was different from column 1. Therefore, when the various levels of ductility are referred to for all of the low temperature specimens, it is crucial to keep in mind that these are the ductility levels of the ambient temperature specimen, not the low temperature specimen. The same load history was used for the low temperature specimens so that a direct comparison could be made between all of the specimens. The first yield displacement for the control specimen was 0.34 inches (8.6 mm) at a force of 10.92 kips (48.6 kN), while the displacement for the second specimen at the same force was 0.19 inches (4.8 mm), which shows an increase in stiffness of the specimen at the low temperature.

Figure 4.19 Longitudinal Steel Temperature for Specimen 2, "TC1" (1 in = 25.4 mm)
Figure 4.20 Temperature between TC1 and TC3, "TC2" (1 in = 25.4 mm)

Figure 4.21 Temperature at the Center of the Cross-Section of Specimen 2, "TC3" (1 in = 25.4 mm)
During the testing of the second specimen fewer observations were made due to the fact that the specimen remained subject to the low temperature in the environmental chamber. This prevented the marking of cracks and the taking of pictures, but there were still two webcams inside of the chamber that were placed in order to view the plastic hinge region of the specimen from the top and the bottom. The top webcam looked through a hole in the roof of the chamber down onto the plastic hinge, while the bottom webcam looked up at the plastic hinge from the floor of the chamber. The photos shown were taken using the webcams at the same points of testing (in most cases) as the photos of specimen 1, and direct comparison can be made between them. The first picture was taken at during the push segment of ductility 1.5, cycle 1, shown in Figure 4.22. By comparing this picture with Figure 4.6, it can be seen that the cracking was similar at this level of ductility for the two columns. In both cases a significant crack formed approximately seven inches from the face of the footing. The next picture, Figure 4.23, shows the push segment of ductility 3, cycle 1, and again, similar behavior is exhibited by the two columns. A large crack can be seen at the interface between the column and the footing on both specimens. The onset of crushing for specimen 1 began at ductility 3 cycle 1 in the push direction, while crushing in specimen 2 began at ductility 4 cycle 1 in the push direction. However, due to the offset of the load history during the first test, it can be concluded that crushing began at similar levels for both columns. The similarity in crushing can be seen by observing Figure 4.24, which shows the bottom of specimen 2 during the push segment of ductility 4 cycle 1, and comparing it with Figure 4.8. By ductility 5, cycle 1, several large cracks had formed in column 2, and these are shown in Figure 4.25 (compare to Figure 4.9). Figure 4.26 shows specimen 2 at ductility 6,
Figure 4.22 Crack Formation on the Top of Specimen 2 during the Push Segment of Ductility 1.5, Cycle 1

Figure 4.23 Crack Formation on the Top of Specimen 2 during the Push Segment of Ductility 3, Cycle 1
cycle 1, which can be compared to Figure 4.11. On the first cycle of ductility 6 in the pull direction, the bottom bar of column 2 ruptured, a failure that occurred sooner than the failure of specimen 1. Before the bar ruptured, the largest displacements that the member had faced were 3.22 inches (82 mm) in the pull direction and 3.79 inches (96 mm) in the push direction. By comparison, the same bar of column 1 ruptured after undergoing displacements of 3.62 inches (92 mm) in the pull direction and 4.14 inches (105 mm) in the push direction. This corresponds to a reduction in displacement capacity of the column at the colder temperature.

Another bar on the bottom of the specimen ruptured on the first cycle of ductility 8 in the pull direction, which is different behavior from the first specimen, in which the top bar fractured after the fracture of the bottom bar. It is believed that the reason for this
Figure 4.25 Crack Formation on the Top of Specimen 2 during the Push Segment of Ductility 5, Cycle 1

Figure 4.26 Crushing of Specimen 2 during the Push Segment of Ductility 6, Cycle 1
difference is that the reinforcing cage of column 2 was slightly off center in the column, causing greater demand to be placed on the lower bars. On the second cycle of ductility 8, another bar on the bottom of the column ruptured, so that all three of the bottom bars failed during the test. The final condition of the test specimen is shown in Figure 4.27 and Figure 4.28, and there are a few more differences to note in comparison with test specimen 1. Specimen 1 suffered concrete crushing in the footing, while specimen 2 had no concrete crushing in the footing. This can be explained by the fact that the load history was offset for column 1, causing larger tensile strains in the top of the footing and therefore causing it to break up more easily in compression. It is also noteworthy that the cracking is significant in the footing of specimen 2, and appears to be very near to the onset of crushing. The difference in behavior between the specimens is significant, but it is not so great that it cannot be accounted for.

In addition to the variances in footing crushing, there are also some differences in column crushing that can be observed from the photographs. The extent of crushing in the column area for specimen 2 was about 12 inches (305 mm), while the extent of crushing in the column area of test specimen 1 was only about 6 inches (152 mm). The extent of crushing was close to the same for both specimens, or about 12 inches. In specimen 1 the crushing was split almost evenly between the footing and the column, while in specimen 2 it was entirely in the column.
Figure 4.27 Concrete Crushing and Rupture of Reinforcement on the Bottom of Specimen 2 after Testing

Figure 4.28 Concrete Crushing on the Top of Specimen 2 after Testing
The force-deformation response for specimen 2 is shown in Figure 4.29, and it can be compared with the same response for specimen 1. One can observe the three instances of bar rupture in the third quadrant of the graph where there is a dramatic decrease in strength of the specimen at each point of rupture. The ultimate strength for column 2 was 23.38 kips (104 kN) in the push direction and 25.89 kips (115.2 kN) in the pull direction, giving the average ultimate strength of 24.64 kips (109.7 kN). The specimen was stronger in the pull direction due to the fact that the self-weight of the column caused an additional moment in the push direction, while the dead load helped resist the load of the actuator in the pull direction. Specimen 2 had a strength increase of 7.9% over the specimen tested at laboratory ambient temperature, meaning that the cold temperature did have the result of increasing the strength of the member. In addition, the displacement capacity was reduced from 3.62 inches (92 mm) to 3.22 inches (82 mm) in the pull direction and from 4.14 inches (105 mm) to 3.79 inches (96 mm) in the push direction. The hysteretic damping is similar for the two specimens, and this will be discussed in greater detail in the final chapter.

Figure 4.30 shows the backbone curves for each cycle of response for specimen 2, and they show comparable degradation in stiffness to specimen 1. Figure 4.31 demonstrates the displacement profile of specimen 2, which is similar to the displacement profile for specimen 1. Figure 4.32 and Figure 4.33 show the moment-curvature responses of specimen 2. Figure 4.33 shows the curvature between 3.8 inches (97 mm) and 10.7 inches (272 mm) from the base of the footing, and this can be compared directly to the moment-curvature response of specimen 1. The graph shows the data through ductility 4 cycle 3, as the data beyond this point is not valid. Figure 4.32 shows the
curvature of the first 6.1 inches (155 mm) of the column. This data was not recorded for specimen 1, but it will be compared to the test data of the remaining specimens. This graph shows the data through the second cycle of ductility 6. Figure 4.34 shows the column curvature profile, but remember that the bottom row of points was calculated from the additional linear potentiometers for the second test, so these points cannot be compared to specimen 1 since this data does not exist for that specimen. One can notice that the curvature at the base of the specimen gives an asymmetric profile in the opposite directions of loading. This is caused by the asymmetry in the method that the footing resists the loading of the actuator. In the push direction (the right side of the curvature profile), the footing transmits the load to the base plate, and therefore the curvature is concentrated at the base of the column while there is little curvature in the footing. In the pull direction, the footing presses against the post-tensioning washers, which are very small compared to the base plate. Therefore, the curvature in the footing was greater in the pull direction, as evidenced by the fact that the cracking and crushing on the top of the footing was greater than the cracking and crushing on the bottom for each specimen. Since the curvature was greater in the footing, the plasticity was spread along a greater portion of the specimen, so that the curvature readings at the base of the column are smaller in the pull direction than in the push direction.

Much of the data in these graphs will be elaborated upon later in this paper so that all four specimens can be compared to one another. In summary, test specimen two demonstrates that low temperatures have the impact of increasing the strength while decreasing the ductility of reinforced concrete members.
Figure 4.29 Specimen 2 Force-Deformation Response (1 kip = 4.45 kN; 1 in = 25.4 mm)

Figure 4.30 Specimen 2 Backbone Response s (1 kip = 4.45 kN; 1 in = 25.4 mm)
Figure 4.31 Specimen 2 Displacement Profiles (1 in = 25.4 mm)

Figure 4.32 Specimen 2 Moment-Curvature Response, 0 to 6.1 Inches from the Face of the Footing
(1 kft = 1.36 kNm; 1 in = 25.4 mm)
Figure 4.33 Specimen 2 Moment-Curvature Response, 3.8 to 10.7 Inches from the Face of the Footing
(1 kft = 1.36 kNm; 1 in = 25.4 mm)

Figure 4.34 Specimen 2 Curvature Profiles (1 in = 25.4 mm)
4.4 Specimen 3 Experiment Results

Column 3 was tested on Thursday, June 3, 2004. The environmental chamber was started at about 10 am on June 2 to cool the column, and the specimen reached –39° C by the evening of June 3. Figure 4.35 shows the cooling pattern for the column during this time, while Figure 4.36 shows the ambient temperature of the environmental chamber during the test. Figure 4.37, Figure 4.38, and Figure 4.39 show the temperature inside the column during the test. As the graphs indicate, the environmental chamber and the specimen grew warmer during the test due to some technical difficulties with the chamber. The warmest temperature that TC1 experienced was –33° C, which was located on the reinforcement. However, this thermocouple was embedded in the concrete away from the plastic hinge region, so it was never exposed to the air of the chamber as the plastic hinge was after spalling. Due to the fact that the temperature of the air inside the chamber reached a temperature of -25° C, one can assume that the temperature of the plastic hinge was somewhere between –33° C and –25° C at the end of the test. Assuming that the plastic hinge was –33° C at the end of the test would not be valid due to the fact that it was more exposed than TC1 because of concrete crushing.
Figure 4.35 Temperature of Specimen 3 while Cooling

Figure 4.36 Ambient Environmental Chamber Temperature during Test 3 (1 in = 25.4 mm)
Figure 4.37 Longitudinal Steel Temperature for Specimen 3 during Testing, "TC1" (1 in = 25.4 mm)

Figure 4.38 Temperature between TC1 and TC3, "TC2" (1 in = 25.4 mm)
As with test specimen 2, no observations could be made during the test except what could be seen by the webcams. The same load history that was applied to specimen 2 was also applied to specimen 3, as described in section 4.1. At the first yield force, specimen 3 reached a displacement of 0.16 inches (4.1 mm), compared to 0.34 inches (8.6 mm) for specimen 1 and 0.19 inches (4.8 mm) for specimen 2. This demonstrates an increase in stiffness at the lower temperature. The first cracks became visible through the webcam on the first cycle of ductility 1, while on specimen 2 the first cracks were not noticed until ductility 1.5. Figure 4.40 shows the top of the specimen during the push segment of ductility 1.5, cycle 1, and this can be compared to the photographs of specimens 1 and 2. All three specimens exhibited a major crack that opened up about seven inches from the face of the footing. The growth of this crack at ductility 3, cycle 1
in the push direction can be seen in Figure 4.41, while the onset of crushing can be observed on the bottom of the specimen in Figure 4.42, which shows the specimen at ductility 4, cycle 1. Crushing began at similar displacements for all three specimens.

Figure 4.40 Crack Formation on the Top of Specimen 3 during the Push Segment of Ductility 1.5, Cycle 1

Figure 4.41 Crack Formation on the Top of Specimen 3 during the Push Segment of Ductility 3, Cycle 1
The extent of cracking can be seen on the top of the specimen in Figure 4.43, where the specimen is at the first cycle of ductility 5. The cracking does appear to be less than the cracking of specimen 1 and specimen 2, as it has only two major cracks with slight cracking in the footing. This is significantly less than the cracking that was observed during the first two tests. By ductility 6 cycle 1, however, the crushing that occurred in specimen 3 was just as much as what was observed in the first two specimens, and this can be seen in Figure 4.44. The bottom bar of steel ruptured in the pull direction on the first cycle of ductility 6, which is the same point at which the same bar ruptured in column 2. The largest displacements that the specimen had undergone up to that point were 3.79 inches (96 mm) in the push direction and 3.13 inches (80 mm) in the pull direction. This is almost identical to specimen 2, while it is a reduction from specimen 1, which had undergone displacements of 4.14 inches (105 mm) in the push direction and 3.62 inches (92 mm) in the pull direction.
Figure 4.43 Crack Formation on the Top of Specimen 3 during the Push Segment of Ductility 5, Cycle 1

Figure 4.44 Crushing of Concrete Cover on the Bottom of Specimen 3 during the Push Segment of Ductility 6, Cycle 1
In addition to the rupture of the first bottom bar, a second bottom bar ruptured on the second cycle of ductility 6 in the pull direction, which occurred sooner than the rupture of the same bar in specimen 2. This demonstrates a decrease in the ductility of the specimen at the colder temperature. The largest displacements that specimen 3 faced prior to the second rupture were 3.79 inches (96 mm) in the push direction and 3.84 inches (98 mm) in the pull direction. In contrast, specimen 2 had faced displacements of 5.08 inches (129 mm) in the push direction and 3.79 inches (96 mm) in the pull direction. In the push direction of ductility 6, cycle 3, for specimen 3, the top bar of the specimen fractured. This bar did not fail for specimen 2, and it did not fail in specimen 1 until ductility 8. After the third cycle of ductility 6 in the push direction, the test was concluded.

The final condition of the specimen can be seen in Figure 4.45, which shows the top of the specimen, and Figure 4.46, which shows the bottom of the specimen. As can be seen in Figure 4.45, crushing occurred in the footing that is similar to that which occurred in specimen 1. The extent of crushing in the footing was about 3.5 inches (89 mm) from the interface of the specimen and the footing, while the crushing extended up the column about 11.5 inches (292 mm) on the top of the specimen and 10.5 inches (267 mm) on the bottom of the specimen. One note is that the crushing inside the core of the column is not as significant as the crushing inside the core that specimens 1 and 2 experienced. This can be explained by the variance of the temperature through the cross-section during the test. As the test progressed, the temperature of the core was colder than the temperature of the cover concrete because of the increasing temperature in the environmental chamber. Since the strength of concrete increases with temperature (as the
literature review and the material experimentation in this study have demonstrated), the
colder (and stronger) concrete in the core suffered less damage than it would have if the
cross-section had a uniform temperature.

Figure 4.47 shows the force deformation response for test specimen 3. The
theoretical force-deformation response of a specimen at -40°C is shown with the actual
force-deformation response since the original temperature of the specimen was -40°C. A
different force-deformation response was not created for this specimen since there is no
way to model the variations in temperature (and therefore behavior) of the specimen
during the test. The rupture of the two bottom bars can be seen in the third quadrant
where there is a tremendous decrease in strength, while the rupture of the top bar can be
seen in the second quadrant, at the point where there is a slight decrease in strength. The
ultimate strength of the specimen in the push direction was 24.94 kips (111 kN), while
the ultimate strength in the pull direction was 26.53 kips (118 kN). This resulted in an
average ultimate strength of 25.74 kips (114.5 kN), which is an increase over specimen 1
of 12.7 percent and an increase over specimen 2 of 4.5 percent. Like specimens 1 and 2,
specimen 3 has good damping properties.
Figure 4.45 Concrete Crushing and Rupture of Reinforcement on the Top of Specimen 3 after Testing

Figure 4.46 Concrete Crushing on the Bottom of Specimen 3 after Testing
Figure 4.47 Specimen 3 Force-Deformation Response (1 kip = 4.45 kN; 1 in = 25.4 mm)

Figure 4.48 shows the backbone curves for all three cycles of each displacement level, and the degrading strength on successive cycles can be clearly seen. The displacement profiles are not given for specimen 3 because of the malfunction of the string potentiometer located at the center of the specimen during the test. Since the force-deformation response is calculated from the other two string potentiometers, it would be unnecessary to show them in profile. Figure 4.49 shows the moment-curvature response for the section of the column 3.8 inches (97 mm) through 10.6 inches (269 mm) from the face of the footing. The graph shows the data through the third cycle of ductility 5 in the push direction. The moment-curvature data of the first 6.2 inches (157 mm) of the column is given in Figure 4.50, and this graph shows the data through ductility 5, cycle 3. The curvature profile is given in Figure 4.51. One may notice that the bottom row of points on the curvature profile is offset to the left, which is the opposite offset of the test
specimen 2 curvature profile. This can be explained by observing the specimen 3 curvature at ductility 3, which is almost the same as the curvature value at ductility 2. It is evident that a problem occurred with the instrumentation at this point that was not detected during the test. Therefore, it is believed that the explanation given for the offset of the curvature profile of specimen 2 is still valid. This phenomenon will be discussed again in the final chapter of this paper in the context of the presentation of the displacements calculated from the curvature profile. In conclusion, test specimen 3 maintained the trend that test specimen 2 established, namely the trend of increased strength and reduced ductility as the temperature decreases.

Figure 4.48 Specimen 3 Backbone Responses (1 kip = 4.45 kN; 1 in = 25.4 mm)
Figure 4.49 Specimen 3 Moment-Curvature Response, 3.8 to 10.6 Inches from the Face of the Footing
(1 kft = 1.36 kNm; 1 in = 25.4 mm)

Figure 4.50 Specimen 3 Moment-Curvature Response, 0 to 6.2 Inches from the Face of the Footing
(1 kft = 1.36 kNm; 1 in = 25.4 mm)
Figure 4.51 Specimen 3 Curvature Profiles (1 in = 25.4 mm)
4.5 Specimen 4 Experiment Results

Specimen 4 was tested on June 24, 2004, at a temperature of \(-40^\circ\)C, and unlike specimen 3, the temperature remained constant for the duration of the test. The cooling of the specimen can be seen in Figure 4.52, which shows that it required about 24 hours to completely cool the specimen. Figure 4.53, Figure 4.54, and Figure 4.55 show the temperature of the column during the test, demonstrating the consistency in temperature.

Specimen 4 was tested using the same regimen that is described in section 4.1. At the first yield force (of the ambient temperature specimen) of 10.92 kips (277 kN), the column achieved a displacement of 0.13 inches (3.3 mm). At low temperatures, the columns exhibited a gradual decrease in displacement at this force as the temperature decreased. The webcams were again used to make observations, but the top webcam was out of commission due to the fact that ice formed on the lens of the camera, blurring its vision.

In place of the top webcam, pictures were taken at key points during the test by removing the insulation in the hole of the chamber above the plastic hinge. The first cracks in the specimen became visible on the first cycle of ductility 1 in the pull direction. Figure 4.56 shows the beginning of cracking on the bottom of the column during the push segment ductility 1.5 cycle 1, where there is a crack about 8 inches (203 mm) from the face of the footing and one very close to the footing. At this point in the procedure, the column exhibited similar behavior to the other columns.
Figure 4.52 Temperature of Specimen 4 While Cooling

Figure 4.53 Longitudinal Steel Temperature for Specimen 4, "TC1" (1 in = 25.4 mm)
Figure 4.54 Temperature between TC1 and TC3, "TC2" (1 in = 25.4 mm)

Figure 4.55 Temperature at the Center of the Cross-Section of Specimen 4, "TC3" (1 in = 25.4 mm)
The status of the column at ductility 3, cycle 1 is shown in Figure 4.57, where there is a large crack visible between the interface of the cantilever member and the footing. In this photo there are also the first appearances of crushing in the specimen near the right linear potentiometer, while the cracks that appear 8 to 10 inches (203 to 254 mm) from the footing face are less significant than the cracks that occurred in the same location of the other specimens (compare to Figure 4.7, Figure 4.23, and Figure 4.41). These phenomena will be important during the discussion of the curvature profile later in this section. From the pictures, the plastic curvature appears to be more highly concentrated close to the base of the cantilever member, more so than in the other members; the curvature profile will bear witness to this. One can see from Figure 4.58 that the crushing is well advanced by the first cycle of ductility 5, and the crushing of specimen 4 exceeds the crushing experienced by any of the other specimens at this stage of the test. In addition, as noted previously, the cracks in specimen 4 approximately 8 to 10 inches (203 to 254 mm) from the footing are significantly smaller that the cracks in the same location of the other specimens. Again, this indicates a reduction in the extent
of plasticity. The shortening in the extent of plasticity can be attributed to the brittle nature of the concrete at –40°C. Remember from the material testing regimen that the concrete cylinders failed abruptly with little to no degrading strength on the stress-strain curve. This behavior in specimen 4 prevented the spread of the crushing along the length of the member. In other words, once the onset of crushing began, the concrete completely failed in a concentrated location.

Figure 4.57 Crushing of Concrete Cover on the Top of Specimen 4 during the Push Segment of Ductility 3, Cycle 1
On the bottom of the specimen, the crushing began on the third cycle of ductility 3, and this crushing can be seen in Figure 4.59, which shows the specimen at ductility 4, cycle 1. By comparing this figure to the photos of the other 3 specimens, one can observe that all four specimens suffered the slight onset of crushing at this point. Figure 4.60 shows the development of that crushing on the first cycle of ductility 6, where one can see that there is significant crushing on specimen 4. This appears to be similar to the crushing of the other specimens.
The first rupture of bottom reinforcement occurred during the pull segment of the second cycle of ductility 5. The bottom-most bar ruptured first in all four specimens. However, specimen four had significantly less ductility than the other three specimens. The largest displacements that it had undergone before rupture were 3.16 inches (80 mm) in the push direction and 3.16 inches in the push direction. This is a sooner failure than that which occurred in the other three specimens, demonstrating a reduction in ductility at the lower temperature. Specimens 2 and 3 both suffered rupture on the first cycle of
ductility 6 after undergoing displacements of approximately 3.79 inches (96 mm) in the push direction and 3.15 inches (80 mm) in the pull direction. In other words, in the direction of failure (pull), the three low temperature specimens had all undergone the same displacement, although specimen 4 underwent fewer cycles at that displacement.

The top bar of specimen 4 failed on the way to the first cycle of ductility 6 (push direction), while another bottom bar failed on the return to the pull direction. After this, the test was concluded. The final condition of the specimen can be seen in Figure 4.61 and Figure 4.62, which show the top and bottom of the specimen after the test. The extent of crushing on the top was 10.5 inches (267 mm) from the face of the footing, while the extent of crushing on the bottom was 8.5 inches (216 mm). In addition, there was no crushing in the footing, and the cracking that did occur in the footing of specimen 4 is significantly less than that which occurred in specimen 2, which also suffered no crushing in the footing. Of all of the specimens, specimen 4 suffered the least damage in the footing while suffering the greatest damage to the core of the column. By comparing the crushing seen in Figure 4.61 to that of the other specimens, one can definitely see that the damage to the core of specimen 4 is the most severe. The severe damage to the core combined with the low damage to the footing can be attributed to the brittleness of the concrete at low temperature.
Figure 4.61 Concrete Crushing and Rupture of Reinforcement on the Top of Specimen 4 after Testing

Figure 4.62 Concrete Crushing and Rupture of Reinforcement on the Bottom of Specimen 4 after Testing
The force-deformation response of specimen 4 is shown in Figure 4.63. The ultimate strength for the specimen was 24.82 kips (110.4 kN) in the push direction and 26.79 kips (119.2 kN) in the pull direction, which corresponds to an average ultimate strength of 25.81 kips (114.9 kN). This is a slight increase over specimen 3, while it is an increase of 13% over specimen 1 and an increase of 4.7% over specimen 2. As one can see, the hysteresis loops show good energy dissipation characteristics, which will be quantified in the analysis section. Figure 4.64 shows the backbone curves for specimen 4, and notice the strength degradation that is evident in the push direction (positive force, positive displacement). At ductility 4, cycle 1, the strength of the column is 24.82 kips (110.4 kN), while at ductility 5, cycle 1, the strength is significantly less, 21.67 kips (96.4 kN). This is prior to any rupture of the reinforcement, yet there is significant strength degradation due to the brittle failure of the concrete.
Figure 4.65 shows the curvature of the base of the cantilever member, from the face of the footing to a distance six inches from the face. One can see the location of the first rupture of the reinforcement in the fourth quadrant where the strength and stiffness degrades significantly as the curve approaches the second cycle of ductility 5. In addition, the second and third ruptures are evident by the dramatic drop in strength at the respective locations. Figure 4.66 shows the curvature of the column at the section located from 3.75 inches (95 mm) to 10.5 inches (267 mm) away from the face of the footing. Notice the contrast between this graph and the previous graph, as well as the contrast between this graph and the respective graphs of the other specimens. This graph shows very small curvature readings compared to the previous graph and to the same graphs of the other specimens. The reason for this was discussed previously: the brittle behavior of the concrete at this temperature caused the extent of plasticity to reduce. The curvature
Figure 4.65 Specimen 4 Moment-Curvature Response, 0 to 6.0 Inches from the Face of the Footing (1 kft = 1.36 kNm; 1 in = 25.4 mm)

Figure 4.66 Specimen 4 Moment-Curvature Response 3.75 Inches to 10.5 Inches from the Face of the Footing (1 kft = 1.36 kNm; 1 in = 25.4 mm)
profile is shown in Figure 4.67, and it demonstrates this as well. The plasticity of the points nearest to the footing far exceeds the plasticity of the next points away from the footing. The curvature profiles of the other specimens do not show this sharp distinction, but all have a more gradual transition from the plastic hinge region to the elastic region moving up the column.

The testing of the four specimens has given insight into the behavior of reinforced concrete members at low temperatures. These tests have demonstrated that reinforced concrete undergoes a gradual increase in strength coupled with a reduction in displacement capacity as the temperature decreases. In addition, the test at \(-40^\circ C\) revealed that the brittle behavior of the concrete resulted in a reduction in the extent of plasticity and a concentration of damage. The final chapter discussing the results and conclusions will attempt to tie all of the testing together, including the material testing, while making quantified observations on the reliability of the data and the limit states of the specimens.
Figure 4.67 Specimen 4 Curvature Profiles (1 in = 25.4 mm)
5 Discussion of Results and Conclusions

The following chapter presents the results and conclusions of this research program. The first portion of the chapter presents graphs that help establish the validity of the recorded data. These graphs present the displacement determined from the string potentiometers on the y-axis, while the displacement integrated from the curvature profile is on the x-axis. Afterwards, the backbone curves of all four specimens are shown on the same graph, which will aid in the comparison of the four specimens. Other graphs are then presented, some of which compare the ultimate strengths of the specimens while others compare the drift capacity of the specimens. These graphs will help demonstrate the increase in strength and loss of ductility of the specimens as temperature decreases, and they will be used to discuss the possible need of an overstrength factor for low temperatures. Following the discussion of strength, the energy dissipation and damping characteristics will be discussed in conjunction with the limit states of the columns. The plastic hinge lengths will then be presented in a graph as a function of displacement ductility. Finally, the areas for future research will be addressed at the close of this chapter.

5.1 Discussion of the validity of the test data

In any research endeavor, it is always beneficial to be able to validate the data that has been recorded. The best way to validate the data for this research project was to compare the displacement that was recorded by the string potentiometers to the curvature recorded by the linear potentiometers placed on the column. This was done by taking the first moment of the area of the curvature profile about the location of the applied load, which should give the displacement of the column. Therefore, the displacement that was
measured directly using string potentiometers can be compared to the displacement calculated from the curvature profile, and theoretically they should be exactly the same (Note: a string potentiometer was not placed directly at the location of the applied load, but two string potentiometers were placed on opposite sides of the actuator extension collar, and a linear interpolation was made between them to determine the displacement at the location of the applied load).

The first graph comparing the two displacements is given in Figure 5.1, which shows the displacement of specimen 1. The displacements are calculated from the data collected at the first cycle of each level of ductility. For test specimen 1, remember that the curvature at the base of the column was not recorded due to the inability to place a linear potentiometer on the bottom side of the column at the base due to the footing base plate. The idea of placing the linear potentiometers on the sides of the column was not conceived until after specimen 1 was already tested, so only the last three specimens have the full curvature profile. Therefore, it is difficult to gain much information from Figure 5.1. However, the displacement calculated from the curvature is less than the displacement obtained from the linear potentiometers, which is expected due to the fact that the curvature profile for specimen 1 is not complete.
The graph comparing the displacements for test specimen 2 is shown in Figure 5.2, and this graph is closer to what one would expect. One can see at the end of the test that the data is no longer valid because the damage to the specimen became so severe that the linear potentiometers placed on the column could not remain on the specimen and still make valid readings. However, just prior to this, in the push direction at displacement ductility 5, the displacement obtained from the string potentiometers is 3.14 in (80 mm), while the displacement calculated from the curvature potentiometers is 2.58 in (66 mm). The explanation of this discrepancy has already been discussed, namely the fact that there is curvature in the footing due to the footing’s small size. In the pull direction, the last valid displacement obtained from the string potentiometers was at displacement ductility 5, where the displacement obtained from the string potentiometers is -3.21 in (82 mm),
while the displacement integrated from the curvature profile was -2.31 in (59 mm). The reason that the discrepancy is larger in the pull direction than the push direction has also already been explained: it is the result of the fact that the curvature in the footing is greater in the pull direction than in the push direction due to the asymmetry of the way in which the footing, the base plate, and the post-tensioning resist the applied load.

Figure 5.2 Specimen 2 Comparison of Displacement Obtained from String Potentiometers with Displacement Calculated from Curvature (1 in = 25.4 mm)

The next graph, Figure 5.3, shows the displacements of test specimen 3, and one can see at the location of the arrow that there is a kink in the line. If one looks back to the curvature profiles for specimen 3 in Figure 4.51, it will show that the curvature at the base of the specimen for ductility 2 (0.0014 in\(^{-1}\); 5.51e-5 mm\(^{-1}\)) was almost virtually the same as the curvature at ductility 3 (0.0015 in\(^{-1}\); 5.91e-5 mm\(^{-1}\)). It has not been determined exactly what happened, but it is clear that the curvature data beyond this point
is not valid unless there was a sharp increase in curvature of the footing at that point in the test.

![Graph showing comparison of displacement obtained from string potentiometers with displacement calculated from curvature.](image)

**Figure 5.3 Specimen 3 Comparison of Displacement Obtained from String Potentiometers with Displacement Calculated from Curvature**

Figure 5.4 shows the displacements of test specimen 4, and it appears to be similar to the displacement graph for test specimen 2. The curvature in the footing for test specimen 4 is greater in the pull direction of loading than in the push, and eventually as the test progressed, the damage became so significant that it became incapable of making accurate curvature readings at the base of the specimen.

Perhaps one would think that all of the caveats are cause to doubt the validity and usefulness of this testing program. However, the logistical difficulty of conducting these experiments must be kept in mind. Coming up with a workable test setup and placing the test specimens into the environmental chamber were difficult to perform. The specimens had to be placed on their side due to the dimensions of the environmental chamber, while
the resources of the Constructed Facilities Lab necessitated the use of the base plate and determined the size of the test specimens. The instrumentation worked when subjected to the low temperatures, but these conditions were certainly not ideal for their operation. Despite the imperfection in the results, it is believed that the testing regimen has fulfilled its intended purpose; that is, to shed light on the behavior of reinforced concrete at low temperatures.

5.2 **Strength and ductility capacity**

This section presents the specimen backbone curves in a slightly different format than they were presented in the test summaries. The first cycle backbone responses of all of the specimens are shown together in Figure 5.5; Figure 5.6 shows the second cycle response of the specimens, while Figure 5.7 shows the third cycle response. In addition, Figure 5.8 shows the maximum strength of each specimen on the y-axis with the
corresponding temperature of the specimen on the x-axis, while Figure 5.9 shows the specimen overstrength as a function of temperature. The specimen overstrengths are simply the strengths of the specimens normalized to the strength of the specimen tested at 23°C. For all of the specimens, the strength given refers to the average of the maximum strength in each direction.

Figure 5.10, Figure 5.11, Figure 5.12, and Figure 5.13 all plot the percent drift of the specimens against temperature at various limit states. The drift at ultimate strength is shown in Figure 5.10; the drift at first rupture of reinforcement is shown in Figure 5.11; the drift at 90% of maximum force is shown in Figure 5.12; and the drift at 80% of maximum force is shown in Figure 5.13. The drift given refers to the average drift in both directions at the limit state under consideration.

![Graph](image-url)

*Figure 5.5 Comparison of First Cycle Backbone Responses for All Four Specimens (1 kip = 4.45 kN; 1 in = 25.4 mm)*
Figure 5.6 Comparison of Second Cycle Backbone Responses for All Four Specimens (1 kip = 4.45 kN; 1 in = 25.4 mm)

Figure 5.7 Comparison of Third Cycle Backbone Responses for All Four Specimens (1 kip = 4.45 kN; 1 in = 25.4 mm)
Figure 5.8 Specimen Ultimate Strength vs. Temperature (1 kip = 4.45 kN)

Figure 5.9 Specimen Overstrength vs. Temperature
Figure 5.10 Specimen Drift at Maximum Force vs. Temperature

Figure 5.11 Specimen Drift at First Rupture vs. Temperature
Figure 5.12 Specimen Drift at 90% of Maximum Force vs. Temperature

Figure 5.13 Drift at 80% of Maximum Force vs. Temperature
While viewing all of these graphs, it is important to think about how test specimen 3 fits into the picture. The critical point to address is exactly what temperature should be assigned to this specimen. In order to determine this, the logic that was followed began like this: the maximum strength of the specimen occurred at displacement ductility 4, cycle 1. At this point, crushing had just begun, but no reinforcement was exposed. It is therefore concluded that the thermocouple readings for the specimen three feet from the face of the footing were still valid for the plastic hinge region. Those temperature readings were \(-35.4^\circ C\) for TC1, \(-37.6^\circ C\) for TC2, and \(-38.2^\circ C\) for TC3. TC4, which was located in the atmosphere of the chamber, was reading \(-28.4^\circ C\) at ductility 4, cycle 1. As a result, the approximate temperature of the specimen was chosen to be \(-37^\circ C\) since it is roughly the average temperature for the cross-section of the specimen.

The reason that specimen 3 and specimen 4 are so similar in strength, as shown in Figure 5.8 and Figure 5.9, is that they were so close in temperature at the ultimate strength of each specimen. Towards the end of the test, as the plastic hinge suffered greater crushing and therefore greater exposure to the warmer temperature of the chamber, specimen 3 began to behave more like specimen 2, although specimen 3 still had less displacement capacity than specimen 2 because it was colder for the duration of the test. In Figure 5.11, Figure 5.12, and Figure 5.13, the temperature of specimen 3 is given as \(-29^\circ C\) because this is the average of the reinforcement temperature (TC1, \(-33.2^\circ C\)) and the ambient chamber temperature (TC4, \(-25.1^\circ C\)) at the first rupture of the reinforcement. There is no way to know exactly what the temperature of the plastic hinge
was at these limit states, but it can be assumed that it was somewhere between the
temperatures of TC1 and TC4.

The results of these four experiments clearly indicate that reinforced concrete
members increase in strength and lose displacement capacity as temperature decreases.
The increase in strength is moderate at best, ranging from 7.9% at -20\(^{\circ}\)C to 13% at -40\(^{\circ}\)C;
nevertheless, it is real. At this time, it is good to remember the results of the material
tests. At a strain rate of 0.01s\(^{-1}\), the ultimate stress of reinforcement increased 11% as the
temperature was decreased from ambient lab temperature to the temperature of dry ice.
The strength of the concrete cylinders also increased as the temperature decreased, and
these both correspond well with the increase in strength of the reinforced concrete
specimens.

The reduction in displacement capacity at low temperatures, like the corresponding
increase in strength, is modest. It is important to remember that even the specimen tested
at -40\(^{\circ}\)C was displaced to a drift of 4.86% prior to rupture (Figure 5.11), showing that this
was still a ductile specimen. In relation to the other specimens, however, one can see a
distinct reduction in the drift capacity with decreasing temperature. The explanation for
this is derived from the material behavior observed during the material tests. At a strain
rate of 0.01 s\(^{-1}\), the strain at fracture of the reinforcement at low temperature was 90% of
the fracture strain at ambient lab temperature, which is a significant reduction in capacity.
As for concrete, the brittle behavior observed at low temperature when the cylinders were
tested exhibited itself during the specimen tests, in particular at the temperature of -40\(^{\circ}\)C.

It is believed that these experiments indicate a possible need of an overstrength
factor for low temperatures. While this factor could be in the neighborhood of 1.1, it is
premature to say conclusively whether the factor is necessary and what that factor should be. It is too early to be conclusive because there are too many unknown areas of reinforced concrete behavior at low temperatures. One of these areas is the flexural behavior of members under an axial load. Remember from the literature review that increasing the axial load played a large role in the loss of displacement capacity of columns constructed using high strength concrete. There is no way to know for sure what the impact of an axial load will be on low temperature specimens yet, so it is probably best to withhold judgment on any overstrength factors at the moment. However, based on this research and other research of high strength concrete, it is believed that the impact of an axial load on low temperature specimens is worth investigating. It would also be beneficial to look into the impact of confinement at low temperatures, since past research investigating high strength concrete has demonstrated that columns with higher axial load require greater confinement (Razvi and Saatciuglu 1994; Saatcioglu and Baingo 1999; Azizinamini et al. 1994). Perhaps the addition of steel fibers into the concrete mix could increase the ductility of the concrete and prevent brittle crushing that occurred in specimen 4 at -40°C (Foster, 2001). Other areas for future research of reinforced concrete at low temperatures include the shear behavior of RC members, as well as the behavior of highly reinforced sections. At the material level, it would be beneficial to investigate the cyclic behavior of reinforcement at low temperatures. All of this knowledge would benefit in the design of reinforced concrete to withstand the combination of seismic loads and low temperatures.
5.3 **Limit States**

During the course of the reports on the four specimen tests, the energy dissipation characteristics have been commented upon in a qualitative manner. Figure 5.14 gives a direct qualitative comparison of the energy dissipation characteristics of the four test specimens. This graph shows the force-deformation response for the first complete cycle of ductility 4, and one can see that there is great similarity between the four specimens. The dissipated energy for these hysteresis loops was calculated by determining the area inside each loop using the formula

\[
\text{Dissipated Energy} = \sum \frac{1}{2} (F_n + F_{n+1})[(d_{n+1} - d_n)]
\]

where \((d_n, F_n)\) refers to a point on the force-deformation curve. After determining the area inside the curve \((A_1)\), the damping was calculated according to the Jacobsen (1930) approach using the formula

\[
\% \text{ Damping} = 100 \times \left(\frac{2}{\pi}\right) \times \left(\frac{A_1}{A_2}\right)
\]

where \(A_2\) is the area of a rigid, perfectly-plastic member with the same maximum strength and the same maximum displacement in each direction as the actual member. The results of these calculations are presented in Table 5.1, and one can see that the colder specimens dissipated more energy because they possessed greater strength, but there is very little difference in damping during a single cycle between the four specimens.
Figure 5.14 Comparison of Hysteresis loop Responses during Ductility 4, Cycle 1 for All Specimens (1 kip = 4.45 kN; 1 in = 25.4 mm)

Table 5.1 Comparison of Energy Dissipation and Damping at Ductility 4, Cycle 1 (1 k-in = 0.113 kN-m)

<table>
<thead>
<tr>
<th>specimen</th>
<th>Energy Dissipated k-in</th>
<th>Damping %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>80.91</td>
<td>23.13</td>
</tr>
<tr>
<td>2</td>
<td>90.48</td>
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<tr>
<td>3</td>
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<td>23.14</td>
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<tr>
<td>4</td>
<td>96.91</td>
<td>24.03</td>
</tr>
</tbody>
</table>

The dissipated energy was also calculated at various limit states for the four specimens, and this is shown in Table 5.2 for specimen 1, Table 5.3 for specimen 2, Table 5.4 for specimen 3, and Table 5.5 for specimen 4. In addition to the dissipated energy, the force, displacement, curvature, concrete compressive strain, and steel tensile strain are all shown for each limit state. The various limit states given are cracking, first yield, equivalent yield, crushing, ultimate strength, rupture, approximately 90% of
ultimate strength, and approximately 70% of ultimate strength. The concrete
compressive strain was calculated using the linear potentiometers placed at the base of
the columns. Remember that there were two potentiometers each on the top-side and on
the bottom-side of the specimen (see Figure 3.7), and these potentiometers were offset
and placed at the location of the extreme surface of the column. Therefore, the concrete
compressive strain could be calculated by simply taking the average of the two
potentiometers on the compression side of the specimen, and dividing that by the gage
length of the potentiometer. The steel tensile strain was calculated using the formula

\[
e_{st} = e_{ct} - (abs(e_{ct}) + abs(e_{cc})) \times (1.3125/D)
\]

where \(e_{st}\) is the steel tensile strain, \(e_{ct}\) refers to the extreme fiber concrete tensile strain,
\(e_{cc}\) is the concrete compressive strain, and the abbreviation abs means that the absolute
value of the strain was taken. The value of 1.3125 is the effective cover to the center of
the reinforcement in inches (33.3 mm), while D refers to the distance between the
potentiometers on opposite sides of the specimen. Due to the fact that these
potentiometers were not placed on specimen 1, the curvature and strain data are not given
for this specimen.
Table 5.2 Specimen 1 Limit States (1 kip = 4.45 kN; 1 in = 25.4 mm; 1 k-in = 0.113 kN-m)

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Force kips</th>
<th>Displacement in</th>
<th>Dissipated Energy k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking</td>
<td>4.13</td>
<td>0.06</td>
<td>0.2</td>
</tr>
<tr>
<td>First Yield</td>
<td>11.27</td>
<td>0.37</td>
<td>4.3</td>
</tr>
<tr>
<td>Equivalent Yield</td>
<td>16.02</td>
<td>0.61</td>
<td>10.8</td>
</tr>
<tr>
<td>Crushing</td>
<td>23.08</td>
<td>2.20</td>
<td>122.9</td>
</tr>
<tr>
<td>Ultimate Strength</td>
<td>23.08</td>
<td>2.20</td>
<td>122.9</td>
</tr>
<tr>
<td>Rupture</td>
<td>-8.77</td>
<td>1.17</td>
<td>1135.8</td>
</tr>
<tr>
<td>92.4% Fmax</td>
<td>21.33</td>
<td>4.10</td>
<td>865.6</td>
</tr>
<tr>
<td>78% Fmax</td>
<td>18.00</td>
<td>4.14</td>
<td>1132.8</td>
</tr>
</tbody>
</table>

Note: a negative sign indicates the pull direction.

Table 5.3 Specimen 2 Limit States (1 kip = 4.45 kN; 1 in = 25.4 mm; 1 k-in = 0.113 kN-m)

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Force kips</th>
<th>Displacement in</th>
<th>Curvature 1/in</th>
<th>Concrete Compression Strain $\varepsilon_c$</th>
<th>Steel Tension Strain $\varepsilon_s$</th>
<th>Dissipated Energy k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking</td>
<td>1.93</td>
<td>0.02</td>
<td>0.0000</td>
<td>-0.0001</td>
<td>-0.0001</td>
<td>0.0</td>
</tr>
<tr>
<td>First Yield</td>
<td>11.15</td>
<td>0.27</td>
<td>0.0003</td>
<td>-0.0016</td>
<td>0.0030</td>
<td>2.9</td>
</tr>
<tr>
<td>Equivalent Yield</td>
<td>19.07</td>
<td>0.63</td>
<td>0.0007</td>
<td>-0.0031</td>
<td>0.0089</td>
<td>9.9</td>
</tr>
<tr>
<td>Crushing</td>
<td>23.14</td>
<td>2.51</td>
<td>0.0043</td>
<td>-0.0059</td>
<td>0.0655</td>
<td>340.2</td>
</tr>
<tr>
<td>Ultimate Strength</td>
<td>-25.89</td>
<td>-2.52</td>
<td>-0.0041</td>
<td>-0.0046</td>
<td>0.0637</td>
<td>392.3</td>
</tr>
<tr>
<td>92.4% Fmax</td>
<td>-23.93</td>
<td>-3.22</td>
<td>-0.0041</td>
<td>-0.0020</td>
<td>0.0658</td>
<td>932.6</td>
</tr>
<tr>
<td>Rupture</td>
<td>-19.99</td>
<td>-2.27</td>
<td>-0.0016</td>
<td>0.0127</td>
<td>0.0380</td>
<td>1049.2</td>
</tr>
<tr>
<td>71.4% Fmax</td>
<td>-18.48</td>
<td>-3.88</td>
<td>-0.0051</td>
<td>0.0893</td>
<td>-0.0092</td>
<td>1315.6</td>
</tr>
</tbody>
</table>

Note: a negative sign indicates the pull direction.

Table 5.4 Specimen 3 Limit States (1 kip = 4.45 kN; 1 in = 25.4 mm; 1 k-in = 0.113 kN-m)

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Force kips</th>
<th>Displacement in</th>
<th>Curvature 1/in</th>
<th>Concrete Compression Strain $\varepsilon_c$</th>
<th>Steel Tension Strain $\varepsilon_s$</th>
<th>Dissipated Energy k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking</td>
<td>1.03</td>
<td>0.00</td>
<td>0.0000</td>
<td>0.0000</td>
<td>0.0001</td>
<td>0.0</td>
</tr>
<tr>
<td>First Yield</td>
<td>10.99</td>
<td>0.16</td>
<td>0.0002</td>
<td>-0.0010</td>
<td>0.0019</td>
<td>1.9</td>
</tr>
<tr>
<td>Equivalent Yield</td>
<td>21.20</td>
<td>0.61</td>
<td>0.0006</td>
<td>-0.0027</td>
<td>0.0075</td>
<td>10.6</td>
</tr>
<tr>
<td>Crushing</td>
<td>24.94</td>
<td>2.50</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>371.7</td>
</tr>
<tr>
<td>Ultimate Strength</td>
<td>-26.53</td>
<td>-2.49</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>424.8</td>
</tr>
<tr>
<td>92.5% Fmax</td>
<td>-24.15</td>
<td>-3.12</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>966.8</td>
</tr>
<tr>
<td>Rupture</td>
<td>-12.23</td>
<td>-1.49</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>1067.8</td>
</tr>
<tr>
<td>73% Fmax</td>
<td>-19.02</td>
<td>-3.78</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>1107.3</td>
</tr>
</tbody>
</table>

Note: a negative sign indicates the pull direction.
Table 5.5 Specimen 4 Limit States (1 kip = 4.45 kN; 1 in = 25.4 mm; 1 k-in = 0.113 kN-m)

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Force kips</th>
<th>Displacement in</th>
<th>Curvature 1/in</th>
<th>Concrete Compression Strain $\varepsilon_c$</th>
<th>Steel Tension Strain $\varepsilon_s$</th>
<th>Dissipated Energy k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking</td>
<td>1.15</td>
<td>0.00</td>
<td>0.0000</td>
<td>-0.0001</td>
<td>0.0001</td>
<td>0.0</td>
</tr>
<tr>
<td>First Yield</td>
<td>10.93</td>
<td>0.13</td>
<td>0.0002</td>
<td>-0.0009</td>
<td>0.0018</td>
<td>1.6</td>
</tr>
<tr>
<td>Equivalent Yield</td>
<td>21.69</td>
<td>0.62</td>
<td>0.0007</td>
<td>-0.0028</td>
<td>0.0096</td>
<td>10.8</td>
</tr>
<tr>
<td>Crushing</td>
<td>24.61</td>
<td>1.79</td>
<td>0.0029</td>
<td>-0.0063</td>
<td>0.0417</td>
<td>203.5</td>
</tr>
<tr>
<td>Ultimate Strength</td>
<td>-26.79</td>
<td>-1.93</td>
<td>-0.0022</td>
<td>-0.0021</td>
<td>0.0354</td>
<td>242.4</td>
</tr>
<tr>
<td>90.5% Fmax</td>
<td>-24.24</td>
<td>-2.51</td>
<td>-0.0024</td>
<td>0.0056</td>
<td>0.0451</td>
<td>641.2</td>
</tr>
<tr>
<td>Rupture</td>
<td>-5.82</td>
<td>1.43</td>
<td>0.0041</td>
<td>0.0689</td>
<td>-0.0100</td>
<td>807.4</td>
</tr>
<tr>
<td>69.5% Fmax</td>
<td>-18.63</td>
<td>-3.16</td>
<td>-0.0044</td>
<td>0.0801</td>
<td>-0.0059</td>
<td>864.3</td>
</tr>
</tbody>
</table>

What these tables show is reasonable compatibility with the material tests. For test specimen 2, the concrete compression strain given at the crushing limit state is 0.0059, while the concrete cylinder specimens tested at –20°C had an ultimate strain of approximately 0.0049 and a strain at 0.5$f_c$ on the softening portion of the stress-strain response of approximately 0.0061 (see Table 2.3). For specimen 4, the concrete strain at crushing was 0.0063, which was higher than that of specimen 2. The crushing strain of specimen 4 was lower than the ultimate strain of the cylinders tested at –40°C, but the trend established during the material tests of higher strain at crushing for the lower temperature specimens held true. The strain at crushing for test specimen 3 could not be determined due to the unreliability of the data beyond ductility 2.

In addition to the calculations for concrete strain, the steel strain at the first yield force for test specimen 2 was calculated to be 0.003. The reinforcement tested at low temperatures did not have a well defined yield plateau, so it is difficult to say exactly what the yield strain was for these tests. At ambient temperature, the yield strain was 0.0026, while the low temperature specimens approximately had this yield strain as well. Specimens 3 and 4 had strains of 0.0019 and 0.0018 at the first yield force, which makes
sense because they were stiffer than specimen 2 and the reinforcement had not yielded yet. Due to the increase in stiffness, the strain was spread more along the member.

It has already been noted that the lower the temperature of the specimen, the more energy that the specimen dissipated for a single hysteresis loop. However, one can see from the summary tables that specimen 1 dissipated the most energy of all the specimens prior to the first rupture of the reinforcement. This is due to the fact that specimen 1 had the largest displacement capacity, and therefore underwent the most number of displacement cycles prior to rupture of the reinforcement. Specimen 4, on the other hand, dissipated the lowest quantity of energy prior to rupture because of its low displacement capacity.

5.4 Plastic Hinge Length

The discussion of the extent of plasticity was a major factor in the review of test specimen 4, and this section presents a quantitative assessment of the plastic hinge lengths for specimens 2 and 4. Only specimens 2 and 4 will be discussed because the data is unavailable for specimen 1, and the data for specimen 3 is invalid. Figure 5.15 shows the plastic hinge length of the specimens in the push direction as a function of displacement ductility, while Figure 5.16 shows the same relationship in the pull direction. The plastic hinge length was determined using the relationship (Priestley, Seible, and Calvi 1996)

\[ L_p = \Delta_p / (\phi_p * L) \]

in which \( L_p \) is the plastic hinge length, \( \Delta_p \) is the plastic displacement at a given displacement ductility, \( \phi_p \) is the plastic curvature, and \( L \) is the length from the face of the
footing to the location of the applied load. The value for $\Delta_p$ was determined by subtracting the equivalent yield displacement from the displacement at the given displacement ductility, while $\phi_p$ was calculated by subtracting the equivalent yield curvature from the curvature at the given ductility.

![Figure 5.15 Plastic Hinge Length as a Function of Displacement Ductility during the Push Segment of the First Cycle (1 in = 25.4 mm)](image-url)
Figure 5.16 Plastic Hinge Length as a Function of Displacement Ductility during the Pull Segment of the First Cycle (1 in = 25.4 mm)

In general, the plastic hinge lengths calculated in the push direction are shorter than the plastic hinge lengths that were calculated in the pull direction. This agrees with what has been stated previously, that the plastic curvature in the footing is greatest in the pull direction. The plastic hinge lengths in the push direction for the two specimens are very close to the theoretical plastic hinge length of 6.1 in (155 mm). In the pull direction, the plastic hinge length for specimen 4 increases gradually up to ductility 4 and then decreases gradually until the end of the test. A possible explanation of this is that the specimen was so stiff that the curvature was spread along the member and into the footing until ductility 4. Then, as the abrupt crushing of the brittle concrete occurred, the damage localized, drastically shortening the extent of plasticity. This explanation is consistent with what has been discussed previously about specimen 4. The photographs taken at the end of the test showed severe damage to a concentrated portion of the
column, while the curvature profile of specimen 4 demonstrated that the curvature of the column approximately 4 to 10 inches (102 to 254 mm) from the footing was much lower than that of specimen 2.

5.5 Summary

The preceding research has investigated the behavior of steel, concrete, and reinforced concrete members at low temperatures. Reinforcing steel was tested in direct tension at the temperature of dry ice and compared to the same reinforcement tested at the ambient temperature of the lab (23°C). The results of these tests, at a strain rate of 0.01 s⁻¹, showed that the steel increased in yield strength by 15% and in ultimate strength by 11% at the lower temperature. The same tests demonstrated that the steel had a reduction in fracture strain of 10% at the lower temperature. Tests were also conducted at different strain rates, and these tests showed that steel increases in yield strength and ultimate strength with increasing strain rate, while the fracture strain remained unchanged.

Concrete cylinders were tested in direct compression and cyclic compression at temperatures of 23°C, -20°C, and -40°C. These tests demonstrated that concrete increases in strength, modulus of elasticity, and strain at maximum stress as the temperature decreases. At -40°C, the concrete experienced a brittle failure with little softening on the stress-strain response after reaching f’c.

Four reinforced concrete “column-type” members were tested under simulated earthquake loading at temperatures of 23°C, -20°C, -30°C (approximately), and -40°C. The reinforced concrete members gained flexural strength of 7.9% at -20°C and 13% at -40°C compared to the specimen tested at 23°C. The specimens also lost displacement
capacity with decreasing temperature: at 23°C, the drift prior to the first rupture of
reinforcement was 5.97%; at -20°C, it was 5.39%; at -30°C, it was 5.32%; and at -40°C, it
was 4.86%. At -40°C, the reinforced concrete specimen exhibited a shortening in the
extent of plasticity, while the hysteretic damping characteristics of the specimens were
not significantly affected by the temperature.

There is a need for future research of reinforced concrete at low temperatures to
investigate its shear behavior, its flexural behavior under an axial load combined with the
impact of confinement, the behavior of highly reinforced sections, as well as the cyclic
behavior of reinforcement. Knowledge of these areas will help engineers design
reinforced concrete structures for appropriate seismic behavior at low temperatures.
References


Figure A1.1 Displacement Control Stress-Strain Response of Concrete Cylinder 1 at a temperature of 22°C and a Strain Rate of 0.0001 s⁻¹ (1 ksi = 6.89 MPa)

Figure A1.2 Displacement Control Stress-Strain Response of Concrete Cylinder 4 at a Temperature of 22°C and a Strain Rate of 0.0001 s⁻¹ (1 ksi = 6.89 MPa)
Figure A1.3 Displacement Control Stress-Strain Response of Concrete Cylinder 5 at a Temperature of 22°C and a Strain Rate of 0.001 s\(^{-1}\) (1 ksi = 6.89 MPa)

Figure A1.4 Force Control Stress-Strain Response of Concrete Cylinder 6 at a Temperature of 22°C and a Strain Rate of 0.001 s\(^{-1}\) (1 ksi = 6.89 MPa)
Figure A1.5 Force Control Stress-Strain Response of Concrete Cylinder 7 at a Temperature of $22^\circ$C and a Strain Rate of 0.001 s$^{-1}$, 153 Cycles to Rupture (1 ksi = 6.89 MPa)

Figure A1.6 Force Control Stress-Strain Response of Concrete Cylinder 8 at a Temperature of $22^\circ$C and a Strain Rate of 0.0001 s$^{-1}$ (1 ksi = 6.89 MPa)
Figure A1.7 Displacement Control Stress-Strain Response of Concrete Cylinder 12 at a Temperature between -24°C and -19°C, with a Strain Rate of 0.0001 s⁻¹ (1 ksi = 6.89 MPa)

Figure A1.8 Displacement Control Stress-Strain Response of Concrete Cylinder 13 at a Temperature of -24°C and a Strain Rate of 0.001 s⁻¹ (1 ksi = 6.89 MPa)
Figure A1.9 Force Control Stress-Strain Response of Concrete Cylinder 14 between the Temperature of -20°C and 1.5°C, with a Strain Rate of 0.0001 s⁻¹ for 66 Cycles (1 ksi = 6.89 MPa)

Figure A1.10 Displacement Control Stress-Strain Response of Concrete Cylinder 14 at a Temperature of -21°C and a Strain Rate of 0.0001 s⁻¹ (1 ksi = 6.89 MPa)
Figure A1.11 Force Control Stress-Strain Response of Concrete Cylinder 15 between the Temperature of -25°C and -10°C, with a Strain Rate of 0.001 s\(^{-1}\) for 400 Cycles (1 ksi = 6.89 MPa)

Figure A1.12 Force Control Stress-Strain Response of Concrete Cylinder 15 between the Temperature of -23°C and -7°C, with a Strain Rate of 0.001 s\(^{-1}\) for 400 Cycles (1 ksi = 6.89 MPa)
Figure A1.13 Displacement Control Stress-Strain Response of Concrete Cylinder 15 at a Temperature of -21°C and a Strain Rate of 0.001 s\(^{-1}\) (1 ksi = 6.89 MPa)

Figure A1.14 Force Control Stress-Strain Response of Concrete Cylinder 16 between the Temperature of -23°C and -11°C, with a Strain Rate of 0.001 s\(^{-1}\), 239 Cycles to Rupture (1 ksi = 6.89 MPa)
Figure A1.15 Displacement Control Stress-Strain Response of Concrete Cylinder 20 at a Temperature between -41°C and -21°C, with a Strain Rate of 0.0001 s⁻¹ (1 ksi = 6.89 MPa)

Figure A1.16 Displacement Control Stress-Strain Response of Concrete Cylinder 21 at a Temperature of -42°C, with a Strain Rate of 0.001 s⁻¹ (1 ksi = 6.89 MPa)
Figure A1.17 Force Control Stress-Strain Response of Concrete Cylinder 23 between the Temperature of -39°C and -33°C, with a Strain Rate of 0.001 s^{-1}, 138 Cycles to Rupture (1 ksi = 6.89 MPa)

Figure A1.18 Force Control Stress-Strain Response of Concrete Cylinder 24 between the Temperature of -39°C and -27°C, with a Strain Rate of 0.001 s^{-1} for 239 Cycles (1 ksi = 6.89 MPa)
Figure A1.19 Displacement Control Stress-Strain Response of Concrete Cylinder 24 at a Temperature of -44°C, with a Strain Rate of 0.001 s\(^{-1}\) (1 ksi = 6.89 MPa)
Appendix A-2

Figure A2.1 Linear Potentiometer History on the Top of Specimen 1, 3.6 Inches to 10.8 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A2.2 Linear Potentiometer History on the Bottom of Specimen 1, 3.6 Inches to 10.6 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A2.3 Linear Potentiometer History on the Top of Specimen 1, 10.8 Inches to 17.9 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A2.4 Linear Potentiometer History on the Bottom of Specimen 1, 10.6 Inches to 17.4 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A2.5 Linear Potentiometer History on the Top of Specimen 1, 17.9 Inches to 24.9 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A2.6 Linear Potentiometer History on the Bottom of Specimen 1, 17.4 Inches to 24.1 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A2.7 Linear Potentiometer History on the Top of Specimen 1, 24.9 Inches to 31.9 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A2.8 Linear Potentiometer History on the Bottom of Specimen 1, 24.1 Inches to 31.0 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A2.9 Linear Potentiometer History on the Top of Specimen 1, 31.9 Inches to 38.8 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A2.10 Linear Potentiometer History on the Bottom of Specimen 1, 31.0 Inches to 37.6 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A2.11 Specimen 1 String Potentiometer History at the Location 37.6 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A2.12 Specimen 1 String Potentiometer History at the Location 61.9 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A2.13 Specimen 1 String Potentiometer History at the Location 73 Inches from the Face of the Footing (1 in = 25.4 mm)
Appendix A-3

Figure A3.1 Linear Potentiometer History on the Top Left (Facing the Footing) of Specimen 2, 0 Inches to 6 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A3.2 Linear Potentiometer History on the Top Right (Facing the Footing) of Specimen 2, 0 Inches to 5.9 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A3.3 Linear Potentiometer History on the Bottom Right (Facing the Footing) of Specimen 2, 0 Inches to 6.1 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A3.4 Linear Potentiometer History on the Bottom Left (Facing the Footing) of Specimen 2, 0 Inches to 6.2 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A3.5 Linear Potentiometer History on the Top of Specimen 2, 3.75 Inches to 10.8 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A3.6 Linear Potentiometer History on the Bottom of Specimen 2, 3.9 Inches to 10.6 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A3.7 Linear Potentiometer History on the Top of Specimen 2, 10.8 Inches to 17.8 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A3.8 Linear Potentiometer History on the Bottom of Specimen 2, 10.6 Inches to 17.4 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A3.9 Linear Potentiometer History on the Top of Specimen 2, 17.8 Inches to 24.4 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A3.10 Linear Potentiometer History on the Bottom of Specimen 2, 17.4 Inches to 24.1 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A3.11 Linear Potentiometer History on the Top of Specimen 2, 24.4 Inches to 31.2 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A3.12 Linear Potentiometer History on the Bottom of Specimen 2, 24.1 Inches to 30.8 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A3.13 Linear Potentiometer History on the Top of Specimen 2, 31.2 Inches to 37.6 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A3.14 Linear Potentiometer History on the Bottom of Specimen 2, 30.8 Inches to 37.5 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A3.15 Specimen 2 String Potentiometer History at the Location 37.9 Inches from the face of the footing (1 in = 25.4 mm)

Figure A3.16 Specimen 2 String Potentiometer History at the Location 58.9 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A3.17 Specimen 2 String Potentiometer History at the Location 71.5 Inches from the Face of the Footing (1 in = 25.4 mm)
Appendix A-4

Figure A4.1 Linear Potentiometer History on the Top Left (Facing the Footing) of Specimen 3, 0 Inches to 6.2 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A4.2 Linear Potentiometer History on the Top Right (Facing the Footing) of Specimen 3, 0 Inches to 6.1 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A4.3 Linear Potentiometer History on the Bottom Right (Facing the Footing) of Specimen 3, 0 Inches to 6 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A4.4 Linear Potentiometer History on the Bottom Left (Facing the Footing) of Specimen 3, 0 Inches to 6 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A4.5 Linear Potentiometer History on the Top of Specimen 3, 3.75 Inches to 10.7 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A4.6 Linear Potentiometer History on the Bottom of Specimen 3, 3.8 Inches to 10.6 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A4.7 Linear Potentiometer History on the Top of Specimen 3, 10.7 Inches to 17.3 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A4.8 Linear Potentiometer History on the Bottom of Specimen 3, 10.6 Inches to 17.2 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A4.9 Linear Potentiometer History on the Top of Specimen 3, 17.3 Inches to 23.9 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A4.10 Linear Potentiometer History on the Bottom of Specimen 3, 17.2 Inches to 23.9 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A4.11 Linear Potentiometer History on the Top of Specimen 3, 23.9 Inches to 30.9 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A4.12 Linear Potentiometer History on the Bottom of Specimen 3, 23.9 Inches to 30.6 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A4.13 Linear Potentiometer History on the Top of Specimen 3, 30.9 Inches to 37.6 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A4.14 Linear Potentiometer History on the Bottom of Specimen 3, 30.6 Inches to 37.1 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A4.15 Specimen 3 String Potentiometer History at the Location 59.9 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A4.16 Specimen 3 String Potentiometer History at the Location 70.5 Inches from the Face of the Footing (1 in = 25.4 mm)
Appendix A-5

Figure A5.1 Linear Potentiometer History on the Top Left (Facing the Footing) of Specimen 4, 0 Inches to 6.1 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A5.2 Linear Potentiometer History on the Top Right (Facing the Footing) of Specimen 4, 0 Inches to 6.25 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A5.3 Linear Potentiometer History on the Bottom Right (Facing the Footing) of Specimen 4, 0 Inches to 6.1 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A5.4 Linear Potentiometer History on the Bottom Left (Facing the Footing) of Specimen 4, 0 Inches to 6.25 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A5.5 Linear Potentiometer History on the Top of Specimen 4, 3.75 Inches to 10.5 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A5.6 Linear Potentiometer History on the Bottom of Specimen 4, 3.7 Inches to 10.4 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A5.7 Linear Potentiometer History on the Top of Specimen 4, 10.5 Inches to 17.3 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A5.8 Linear Potentiometer History on the Bottom of Specimen 4, 10.4 Inches to 17.0 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A5.9 Linear Potentiometer History on the Top of Specimen 4, 17.3 Inches to 23.9 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A5.10 Linear Potentiometer History on the Bottom of Specimen 4, 17.0 Inches to 23.6 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A5.11 Linear Potentiometer History on the Top of Specimen 4, 23.9 Inches to 30.9 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A5.12 Linear Potentiometer History on the Bottom of Specimen 4, 23.6 Inches to 30.1 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A5.13 Linear Potentiometer History on the Top of Specimen 4, 30.9 Inches to 37.3 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A5.14 Linear Potentiometer History on the Bottom of Specimen 4, 30.1 Inches to 36.9 Inches from the Face of the Footing (1 in = 25.4 mm)
Figure A5.15 Specimen 4 String Potentiometer History at the Location 59.2 Inches from the Face of the Footing (1 in = 25.4 mm)

Figure A5.16 Specimen 4 String Potentiometer History at the Location 69.9 Inches from the Face of the Footing (1 in = 25.4 mm)