ABSTRACT

STANFORD, KIRK ALAN. Strengthening of Steel Structures with High Modulus Carbon Fiber Reinforced Polymers (CFRP) Materials: Bond and Development Length Study. (Under the direction of Dr. Sami Rizkalla).

Cost-effective solutions for the rehabilitation and strengthening of steel structures, such as steel bridges and steel monopole towers used for cellular phone antennas, are greatly needed by government transportation departments and industry. Rehabilitation is often required due to loss of cross-section from corrosion and/or changes of the demand or use of a structure. Current techniques for strengthening steel structures have several drawbacks including requiring heavy equipment for installation, their fatigue performance, in addition to the need for ongoing maintenance due to continued corrosion attack. The current research program proposed the use of a new high modulus carbon fiber reinforced polymer (CFRP) for strengthening of steel structures. This program includes extensive research to select the resin for wet lay-up of carbon fiber sheets and the adhesives for bonding of pre-cured laminate strips. The bond behavior of FRP materials to steel structures is quite different from that of concrete structures. Preliminary test results showed the occurrence of very high bond stresses for most strengthening applications due to the amount of strengthening required for developing the material for steel structures and bridges.
Strengthening of Steel Structures with High Modulus Carbon Fiber Reinforced Polymers (CFRP) Materials: Bond and Development Length Study

by
Kirk Alan Stanford

A thesis submitted to the Graduate Faculty of
North Carolina State University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Civil Engineering

Raleigh, North Carolina

2009

APPROVED BY:

________________________________     _________________________
Dr. James Nau                                                                Dr . Emmett Sumner

________________________________
Dr. Sami Rizkalla
Chair of Advisory Committee
BIOGRAPHY

Kirk Alan Stanford received his Bachelor of Science degree in Civil Engineering from North Carolina State University in 2001. After graduating with honors, he continued to pursue a Master of Science degree under the guidance of Dr. Sami Rizkalla. Upon completion of his research in 2004, Mr. Stanford practiced as an Engineering Intern at a local firm in Greensboro, North Carolina performing structural investigations and analysis. Mr. Stanford is now married and a proud father of a young son. He would like to thank all who helped make this day possible.
ACKNOWLEDGEMENTS

The author would like to acknowledge the support and guidance of Dr. Sami Rizkalla, without whom this work would not be possible. Thank you for your patience and instruction throughout this experience. Direction provided by the examining committee of Dr. James Nau and Dr. Emmett Sumner are also greatly appreciated. The oral defense will be something I will always remember.

The author would like to also thank Mitsubishi Chemical America, Inc. and Mr. Akira Nakagoshi for their generous financial support and donation of the carbon fiber materials for this research. I hope that this research will be a platform from which new opportunities are created and explored for their carbon fiber materials.

The author would finally like to acknowledge the support of the faculty and students at the Constructed Facilities Laboratory who helped me complete this research. The technicians of Mr. Jerry Atkinson and Mr. William Dunleavy were always helpful and the support of my research team, Mr. Mina Dawood, Mr. Bryan Lanier, and Mr. David Schnerch was invaluable. This research experience with you was the most rewarding part.
# Table of Contents

List of Tables .......................................................................................................................... ix
List of Figures ............................................................................................................................. xi

1 Introduction .............................................................................................................................. 1

1.1 General ............................................................................................................................... 1
1.2 Research Objective ............................................................................................................. 2
1.3 Research Scope .................................................................................................................. 3
1.4 Overall Research Program ............................................................................................... 3
1.5 Thesis Contents .................................................................................................................. 6

2 Background ............................................................................................................................ 8

2.1 Introduction ......................................................................................................................... 8
2.2 Flexural Behavior of Steel Members Strengthened with FRP Materials .................. 12

2.2.1 Strengthening of Undamaged Wide-Flanged Steel Beams ....................................... 15
2.2.2 Strengthening of Steel-Concrete Composite Girders ............................................... 18
2.2.3 Rehabilitation of Corroded Steel Girders .................................................................. 20
2.2.4 Rehabilitation of Notched Steel and Cast Iron Beams ............................................. 21
2.2.5 Strengthening of Tubular Members ............................................................................ 24
2.2.6 Strengthening of Cast Iron Beams .......................................................................... 26
2.2.7 Use of Prestressed CFRP Strengthening Systems ................................................... 28

2.3 Field Applications ......................................................................................................... 30

2.3.1 Steel and Cast Iron Bridge Strengthening with CFRP ............................................ 30
2.3.2 Strengthening Applications Using High Modulus CFRP ...................................... 33

2.4 Bond Characteristics ................................................................................................... 34

2.4.1 Mechanics of Adhesion .............................................................................................. 35
2.4.2 Adhesion Selection ...................................................................................................... 35
2.4.3 Surface Preparation for Steel and CFRP .................................................................. 38
2.4.4 Silane Adhesion Promotors ....................................................................................... 40
3.5.2 Test Program ........................................................................................................ 77
3.5.3 Test Procedure and Instrumentation ................................................................. 80
3.6 PHASE 1(B) ADHESIVE SELECTION FOR BONDED LAMINATE STRIPS .......... 83
  3.6.1 Test Specimens .............................................................................................. 83
  3.6.2 Test Program ................................................................................................. 83
  3.6.3 Test Procedure and Instrumentation .............................................................. 84
3.7 PHASE 2(B) DEVELOPMENT LENGTH OF CFRP STRIPS ......................... 87
  3.7.1 Test Specimens .............................................................................................. 87
  3.7.2 Test Program ................................................................................................. 87
  3.7.3 Test Procedure and Instrumentation .............................................................. 88
4 EXPERIMENTAL RESULTS ................................................................................... 89
  4.1 PHASE 1(A) RESIN SELECTION FOR WET LAY-UP OF CFRP SHEETS ........ 89
    4.1.1 Introduction ................................................................................................. 89
    4.1.2 Room Temperature Cured Resins ............................................................. 91
    4.1.3 Room Temperature Cured Resins with Wetting Agent ......................... 95
    4.1.4 Room Temperature Cured Hybrid Resins .............................................. 97
    4.1.5 Heat Cured Resins .................................................................................. 100
    4.1.6 Heat Cured Prepreg Resins .................................................................. 102
    4.1.7 Summary ............................................................................................... 105
  4.2 PILOT TESTS SPECIMEN DEVELOPMENT FOR ADHESIVE SELECTION .... 107
    4.2.1 Introduction ............................................................................................. 107
    4.2.2 HSS Beam Tests ..................................................................................... 107
    4.2.3 W4 x 13 Beam Tests .............................................................................. 110
    4.2.4 Summary ............................................................................................... 111
    4.2.5 Final Test Specimen ............................................................................. 112
  4.3 PHASE 1(B) ADHESIVE SELECTION FOR BONDED LAMINATE STRIPS ... 112
    4.3.1 Introduction ............................................................................................. 112
    4.3.2 Overall Test Results ............................................................................... 117
    4.3.3 Test Results of SP Systems Spabond 345 Epoxy Adhesive ................. 121
4.3.4 Test Results of Weld-On SS620 Acrylic Adhesive..................................... 123
4.3.5 Test Results of Vantico Araldite 2015 Epoxy Adhesive............................. 125
4.3.6 Test Results of Jeffco 121 Epoxy Adhesive............................................. 127
4.3.7 Test Results of Fyfe Tyfo MB2 Epoxy Adhesive ....................................... 129
4.3.8 Test Results of Sika Sikadur 30 Epoxy Adhesive....................................... 130
4.3.9 Costs ............................................................................................................ 131
4.3.10 Summary ................................................................................................... 132

4.4 **Phase 2(a) Development Length of CFRP Sheets** ............................................ 133
   4.4.1 Introduction ................................................................................................. 133
   4.4.2 One Ply CFRP Sheet Tests ........................................................................ 134
   4.4.3 Two Ply CFRP Sheet Tests ......................................................................... 138
   4.4.4 Summary ..................................................................................................... 142

4.5 **Phase 2(b) Development Length of CFRP Strips** ............................................. 143
   4.5.1 Introduction ................................................................................................. 143
   4.5.2 One Ply CFRP Laminate Strip Tests .......................................................... 144
   4.5.3 Two Ply CFRP Laminate Strip Tests .......................................................... 146
   4.5.4 Summary ..................................................................................................... 150

4.6 **Bond Behavior** ............................................................................................. 150
   4.6.1 Introduction ................................................................................................. 150
   4.6.2 Measured Strain and Shear Stress for CFRP Fiber Sheets ......................... 153
   4.6.3 Measured Strain and Shear Stress for CFRP Laminate Strips .................... 161

4.7 **Shear Lag Effect** ......................................................................................... 170
4.8 **Nonlinear Strain Measurements** .................................................................. 177
4.9 **Welded Steel Plate Effect** ............................................................................. 178

5 **Conclusions** ..................................................................................................... 182
   5.1 Saturation Resin Selection for the Wet Lay-Up of CFRP Dry Fiber Sheets 182
   5.2 Adhesive Selection for Bonding of CFRP Laminate Strips ............................. 183
   5.3 Development Length Study and Two Ply Applications for Both CFRP Materials ......................................................................................................................... 183
5.4 Bond Characteristics ........................................................................................................ 184
5.5 Shear Lag Effect .................................................................................................................. 184
5.6 Nonlinear Strain Measurements ......................................................................................... 185
5.7 Welded Steel Plate Effect .................................................................................................... 185
5.8 Future Work ......................................................................................................................... 185
6 References ............................................................................................................................. 187
7 Appendix ................................................................................................................................ 194
LIST OF TABLES

Table 3-1   Scheme of the experimental program 57
Table 3-2   Mechanical properties of unidirectional fiber sheets and CFRP laminate strips 59
Table 3-3   Properties of EPON Resin 9310 59
Table 3-4   Properties of steel substrate 62
Table 3-5   Room Temperature tensile properties of saturant resins used for bonding unidirectional carbon fiber sheets to steel 63
Table 3-6   Room Temperature tensile properties of adhesives used for bonding laminate strips to steel 64
Table 3-7   Test program of resin selection for wet lay-up of unidirectional carbon fiber sheets 67
Table 3-8   Prepreg resin selection test program 67
Table 3-9   Development lengths tested for one and two plies of the carbon fiber sheets 79
Table 3-10  Test program of adhesive selection for laminate strips 83
Table 3-11  Development lengths tested for one and two plies of CFRP laminate strips 87

Table 4-1   Average shear strength and longitudinal strain at peak stress values for wet lay-up resins cured at room temperature 93
Table 4-2   Average shear strength and longitudinal strain at peak stress values for wet lay-up resins with a wetting agent 97
Table 4-3   Average shear strength and longitudinal strain at peak stress values for wet lay-up of hybrid resin combinations 99
Table 4-4   Average shear strength and longitudinal strain at peak stress values for wet lay-up of heat cured resins 100
Table 4-5   Average shear strength and longitudinal strain at peak stress values for wet lay-up prepreg resins heat cured 103
Table 4-6   Test results for HSS beam pilot tests 109
Table 4-7   Test results for W4 x 13 beam pilot tests 110
Table 4-8   CFRP strip strain at rupture/debonding for tested adhesives and development lengths with one ply of CFRP strips 118
Table 4-9   Averages of determined stiffness and strength increases for the series of adhesives tested at each development length 120
Table 4-10  One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the SP Systems Spabond 345 adhesive tests 122
| Table 4-11 | One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the Weld-On SS620 adhesive tests | 124 |
| Table 4-12 | One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the Vantico Araldite 2015 adhesive tests | 126 |
| Table 4-13 | One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the Jeffco 121 adhesive tests | 128 |
| Table 4-14 | One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the Fyfe Tyfo MB2 adhesive tests | 129 |
| Table 4-15 | One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the Sika Sikadur 30 adhesive tests | 131 |
| Table 4-16 | 2004 Costs comparison for adhesive bonding CFRP laminate strips to steel | 132 |
| Table 4-17 | One ply CFRP sheet test results for 51 mm lengths | 135 |
| Table 4-18 | One ply CFRP sheet stiffness and strengths for 51 mm development lengths | 136 |
| Table 4-19 | Two ply CFRP sheet ultimate shear, strain, and load | 139 |
| Table 4-20 | Two ply CFRP sheet stiffness and strengths | 139 |
| Table 4-21 | Maximum shear stress and failure mode for beams strengthened with CFRP laminate strips using different development lengths | 146 |
| Table 4-22 | Two ply development length test results | 147 |
| Table 4-23 | Table 4-23 Maximum shear stress (MPa) and failure mode for beams strengthened by wet lay-up of CFRP sheets using different development lengths | 154 |
| Table 4-24 | Maximum shear stress (MPa) and failure mode for beams strengthened by adhesive bonding of CFRP strips using different development lengths | 161 |
# LIST OF FIGURES

| Figure 3-1 | Comparison of tensile stress and strain properties of several FRP materials to steel | 58 |
| Figure 3-2 | Tensile stress and strain for steel coupons taken from the flanges of the test beams | 60 |
| Figure 3-3 | Tensile stress and strain for steel coupons taken from the webs of the test beams | 61 |
| Figure 3-4 | Tensile stress and strain for coupons taken from the steel plates | 61 |
| Figure 3-5 | Test specimens for resin selection of CFRP sheets | 65 |
| Figure 3-6 | Fixtures for fabrication of double lap shear coupons | 68 |
| Figure 3-7 | Fabrication of the opposite side of the double lap shear coupons | 68 |
| Figure 3-8 | Test instrumentation and setup of double lap shear coupons | 69 |
| Figure 3-9 | HSS cross-sections for pilot beam tests | 71 |
| Figure 3-10 | Instrumentation at mid-span for HSS beam specimens | 73 |
| Figure 3-11 | Test setup and instrumentation for W4 x 13 pilot tests | 75 |
| Figure 3-12 | Test specimen cross-section for development length study of bonded CFRP sheets | 78 |
| Figure 3-13 | Weld configurations along compression flange of SLB specimen | 79 |
| Figure 3-14 | Bond process for CFRP sheets | 80 |
| Figure 3-15 | Test setup for development length of bonded carbon fiber sheets | 81 |
| Figure 3-16 | Test setup and instrumentation for development length tests | 82 |
| Figure 3-17 | Strain gauge locations along the carbon fiber sheet | 82 |
| Figure 3-18 | CFRP laminate strip application | 85 |
| Figure 3-19 | Cross-section of lateral supports | 86 |
| Figure 3-20 | Lateral bracing at the supports | 86 |

<p>| Figure 4-1 | Average shear strength and longitudinal strain at peak stress for wet lay-up resins cured at room temperature | 92 |
| Figure 4-2 | Stress-strain behavior of double lap-shear coupons using different resins | 94 |
| Figure 4-3 | Adhesive failure of typical 3M DP810 double lap shear coupon | 95 |
| Figure 4-4 | Comparison of average shear strength and longitudinal strain at peak stress for wet lay-up resins with a wetting agent | 96 |
| Figure 4-5 | Average shear strength and longitudinal strain at peak stress for wet lap-up of hybrid resin combinations | 98 |
| Figure 4-6 | Debonding failure of typical Jeffco-Sika Sikadure 300 double lap shear coupons | 99 |
| Figure 4-7 | Average shear strength and longitudinal strain at peak stress for wet lap-up of heat cured resins | 101 |
| Figure 4-8 | Rupture failure of typical coupon using SP Systems Ampreg 22 heat cured resin | 102 |
| Figure 4-9 | Failure mode of typical Sika Sikadur 330-Prepreg 2A test specimen | 103 |
| Figure 4-10 | Average shear strength and longitudinal strain at peak stress for wet lap-up of heat cured prepreg resins | 104 |
| Figure 4-11 | Test setup and debonding failure of CFRP sheets on the 6.4 mm HSS specimen | 109 |
| Figure 4-12 | Wrapping the ends of the CFRP sheets | 110 |
| Figure 4-13 | Typical applied load verses mid-span displacement of beam strengthened with one ply thickness CFRP laminate strip | 116 |
| Figure 4-14 | Beam strengthened with CFRP strips having a 203 mm development length, and bonded with Tyfo MB2 adhesive just before unloading | 117 |
| Figure 4-15 | Ultimate strains for bonded CFRP strips with different adhesives and at different development lengths | 119 |
| Figure 4-16 | Load displacement response curves for beams strengthened with different adhesives at a 102 mm development length | 121 |
| Figure 4-17 | Failure of 102 mm CFRP laminate strip using SP Systems Spabond 345 | 122 |
| Figure 4-18 | Failure of 76 mm CFRP laminate strip using SP Systems Spabond 345 | 123 |
| Figure 4-19 | Failure of 76 mm CFRP laminate strip using Weld-On SS620 | 124 |
| Figure 4-20 | Debonding failure of 51 mm CFRP laminate strip using Weld-On SS620 | 125 |
| Figure 4-21 | Failure of 152 mm (6 inch) CFRP laminate strip using Vantico Araldite 2015 | 126 |
| Figure 4-22 | Failure of 152 mm (6 inch) CFRP laminate strip using Jeffco 121 | 127 |
| Figure 4-23 | Varying validation test results for Jeffco 121 adhesive | 128 |
| Figure 4-24 | Failure of 102 mm (4 inch) CFRP laminate strip using Fyfe Tyfo MB2 adhesive | 130 |
| Figure 4-25 | Failure of 203 mm CFRP laminate strip using Sika Sikadur 30 | 131 |
| Figure 4-26 | Three varying development lengths for the Sika Sikadure 330 resin | 133 |
| Figure 4-27 | Applied load verses mid-span displacement for beams strengthened with one ply of CFRP sheet 51 mm in length using (MB) Degussa MBrace Saturant and (S) Sika Sikadur 330 | 137 |</p>
<table>
<thead>
<tr>
<th>Figure 4-28</th>
<th>Failure of 51 mm CFRP sheet using Degussa MBrace Saturant resin</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure 4-29</td>
<td>Failure of 51 mm CFRP sheet using Sika Sikadur 330 resin</td>
</tr>
<tr>
<td>Figure 4-30</td>
<td>Maximum CFRP strains for beams strengthened with two plys of CFRP sheets using different development lengths</td>
</tr>
<tr>
<td>Figure 4-31</td>
<td>Failure of 102 mm CFRP sheet using Degussa MBrace Saturant resin</td>
</tr>
<tr>
<td>Figure 4-32</td>
<td>Failure of 76 mm CFRP sheet using Sika Sikadur 330 resin</td>
</tr>
<tr>
<td>Figure 4-33</td>
<td>CFRP strip strain at rupture/debonding for four out of six tested adhesives using different development lengths</td>
</tr>
<tr>
<td>Figure 4-34</td>
<td>Load displacement response of both beams strengthened with two plys of the CFRP strips using different adhesives and development lengths</td>
</tr>
<tr>
<td>Figure 4-35</td>
<td>Rupture failure of the double ply CFRP strip using SP Systems Spabond 345 adhesive at 203 mm development length</td>
</tr>
<tr>
<td>Figure 4-36</td>
<td>Debonding failure of the double ply CFRP strip using Jeffco 121 adhesive at 254 mm development length</td>
</tr>
<tr>
<td>Figure 4-37</td>
<td>Measured strain profile along one ply CFRP sheet with development length of 51 mm using Degussa MBrace Saturant resin</td>
</tr>
<tr>
<td>Figure 4-38</td>
<td>Measured strain profile along two ply CFRP sheet with development length of 76 mm using Degussa MBrace Saturant resin</td>
</tr>
<tr>
<td>Figure 4-39</td>
<td>Shear stress profile for beam strengthened with one ply of CFRP sheets using Degussa MBrace Saturant resin and 51 mm development length at different load stages</td>
</tr>
<tr>
<td>Figure 4-40</td>
<td>Average shear stress at different distances from midspan to the end of the CFRP sheet for the test using Degussa MBrace Saturant resin and 51 mm development length</td>
</tr>
<tr>
<td>Figure 4-41</td>
<td>Average shear stress at different distances from midspan to the end of the CFRP sheet for the test using Sika Sikadur 330 resin and 51 mm development length</td>
</tr>
<tr>
<td>Figure 4-42</td>
<td>Shear stress profile for beam strengthened with two ply of CFRP sheets using Degussa MBrace Saturant resin and 102 mm development length at different load stages</td>
</tr>
<tr>
<td>Figure 4-43</td>
<td>Average shear stress at different distances from midspan to the end of the two ply CFRP sheets for the test using MBrace Saturant resin and 102 mm development length</td>
</tr>
<tr>
<td>Figure 4-44</td>
<td>Shear stress profile for beam strengthened with two ply of CFRP sheets using Sika Sikadur 330 resin and 102 mm development length at different load stages</td>
</tr>
<tr>
<td>Figure 4-45</td>
<td>Maximum shear stress profiles for two beams strengthened with two plys of CFRP sheets and 102 mm development length for different resins</td>
</tr>
<tr>
<td>Figure 4-46</td>
<td>Average shear stress at different distances from midspan to the end of the CFRP strip for the test using Jeffco 121 adhesive and 102 mm development length</td>
</tr>
<tr>
<td>-------------</td>
<td>-------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Figure 4-47</td>
<td>Average shear stress at different distances from midspan to the end of the CFRP strip for the test using SP Spabond 345 adhesive and 102 mm development length</td>
</tr>
<tr>
<td>Figure 4-48</td>
<td>Average shear stress at different distances from midspan to the end of the CFRP strip for the test using Weld-On SS620 adhesive and 102 mm development length</td>
</tr>
<tr>
<td>Figure 4-49</td>
<td>Shear stress profile comparisons for three beams strengthened with one ply of CFRP strip and 102 mm development length for different resins</td>
</tr>
<tr>
<td>Figure 4-50</td>
<td>Average shear stress at different distances from midspan to the end of the CFRP strip for the test using Weld-On SS620 adhesive and 76 mm development length</td>
</tr>
<tr>
<td>Figure 4-51</td>
<td>Shear stress profiles for beams strengthened with one ply of CFRP strip using Weld-On SS620 adhesive and different development lengths</td>
</tr>
<tr>
<td>Figure 4-52</td>
<td>Maximum shear stress profile comparisons for beams strengthened with one ply of CFRP strip using Weld-On SS620 adhesive and different development lengths</td>
</tr>
<tr>
<td>Figure 4-53</td>
<td>Average shear stress at different distances from midspan to the end of the two ply CFRP strip for the test using Jeffco 121 adhesive and 254 mm development length</td>
</tr>
<tr>
<td>Figure 4-54</td>
<td>Average shear stress at different distances from midspan to the end of the two ply CFRP strip for the test using SP Spabond 345 adhesive and 203 mm development length</td>
</tr>
<tr>
<td>Figure 4-55</td>
<td>Shear stress profile comparison for beams strengthened with two ply of CFRP sheets using Sika Sikadur 330 resin and with one ply of CFRP strip using Weld-On SS620 adhesive at the same 102 mm development length</td>
</tr>
<tr>
<td>Figure 4-56</td>
<td>Strains at mid-span v. beam depth for specimen bonded with 152 mm development length using Vantico Araldite 2015</td>
</tr>
<tr>
<td>Figure 4-57</td>
<td>Strains at mid-span v. beam depth for specimen bonded with 102 mm development length using Jeffco 121 adhesive</td>
</tr>
<tr>
<td>Figure 4-58</td>
<td>Strains at mid-span v. beam depth for unstrengthened beam until load at stop of test</td>
</tr>
<tr>
<td>Figure 4-59</td>
<td>Strain diagram for unstrengthened beam until load at stop of test: 0.2% Yield Region</td>
</tr>
<tr>
<td>Figure 4-60</td>
<td>Measured strain across the bottom half of the tension flange of the unstrengthened beam</td>
</tr>
<tr>
<td>Figure 4-61</td>
<td>Applied Moment vs. Measured Strain at Mid-span of Control Beam</td>
</tr>
<tr>
<td>Figure 4-62</td>
<td>Neutral axis depths for steel beam configurations</td>
</tr>
<tr>
<td>Figure 4-63</td>
<td>Neutral axis depth of Weld-On SS620 test beams at 102 mm development length</td>
</tr>
<tr>
<td>Figure 4-64</td>
<td>Neutral axis depth of SP Spabond 345 test beams at 102 mm development length</td>
</tr>
<tr>
<td>Figure 4-65</td>
<td>Neutral axis depth of Jeffco 121 test beams at 102 mm development length</td>
</tr>
<tr>
<td>Figure 7-1</td>
<td>Test Results of Degussa MBraze Saturant Epoxy Resin</td>
</tr>
<tr>
<td>Figure 7-2</td>
<td>Test Results of Sika Sikadur 330 Epoxy Resin</td>
</tr>
<tr>
<td>Figure 7-3</td>
<td>Test Results of 3M DP810 Acrylic Resin</td>
</tr>
<tr>
<td>Figure 7-4</td>
<td>Test Results of 3M DP460 Epoxy Resin</td>
</tr>
<tr>
<td>Figure 7-5</td>
<td>Test Results of SP Systems Ampreg 22 (fast hardener) Epoxy Resin</td>
</tr>
<tr>
<td>Figure 7-6</td>
<td>Test Results of SP Systems Ampreg 22 (slow hardener) Epoxy Resin</td>
</tr>
<tr>
<td>Figure 7-7</td>
<td>Test Results of Resinlab EP1246 Acrylic/Epoxy Resin</td>
</tr>
<tr>
<td>Figure 7-8</td>
<td>Test Results of Sika Sikadur 300 Epoxy Resin</td>
</tr>
<tr>
<td>Figure 7-9</td>
<td>Test Results of Jeffco 121 Epoxy Resin</td>
</tr>
<tr>
<td>Figure 7-10</td>
<td>Test Results of Reichhold Atprime 2 Urethane Resin</td>
</tr>
<tr>
<td>Figure 7-11</td>
<td>Test Results of Sika Sikadur 330 Epoxy Resin with Wetting Agent</td>
</tr>
<tr>
<td>Figure 7-12</td>
<td>Test Results of 3M DP460 Epoxy Resin with Wetting Agent</td>
</tr>
<tr>
<td>Figure 7-13</td>
<td>Test Results of 3M DP810 Acrylic Resin with Wetting Agent</td>
</tr>
<tr>
<td>Figure 7-14</td>
<td>Test Results of Jeffco 121 – Sika Sikadur 330 Hybrid Resin</td>
</tr>
<tr>
<td>Figure 7-15</td>
<td>Test Results of Degussa MBraze Primer – Saturant Hybrid Resin</td>
</tr>
<tr>
<td>Figure 7-16</td>
<td>Test Results of Sika Sikadur 330 – 300 Hybrid Resin</td>
</tr>
<tr>
<td>Figure 7-17</td>
<td>Test Results of Jeffco 121 – Sika Sikadur 300 Hybrid Resin</td>
</tr>
<tr>
<td>Figure 7-18</td>
<td>Test Results of Sika Sikadur 330 Heat Cured Epoxy Resin</td>
</tr>
<tr>
<td>Figure 7-19</td>
<td>Test Results of SP Systems Ampreg 22 Heat Cured Resin</td>
</tr>
<tr>
<td>Figure 7-20</td>
<td>Load Displacement Response for 51 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-21</td>
<td>Load Displacement Response for 76 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-22</td>
<td>Load Displacement Response for 102 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-23</td>
<td>Load Displacement Response for Weld-On SS620 adhesive replica beam tests</td>
</tr>
<tr>
<td>Figure 7-24</td>
<td>Load Displacement Response for SP Spabond 345 adhesive replica beam tests</td>
</tr>
<tr>
<td>Figure 7-25</td>
<td>Load Displacement Response for Jeffco 121 adhesive replica beam tests</td>
</tr>
<tr>
<td>Figure 7-26</td>
<td>Load Displacement Response for 127 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-27</td>
<td>Load Displacement Response for 152 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-28</td>
<td>Load Displacement Response for 203 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-29</td>
<td>Neutral axis depth for 51 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-30</td>
<td>Neutral axis depth for 76 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-31</td>
<td>Neutral axis depth for 102 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-32</td>
<td>Neutral axis depth for 127 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-33</td>
<td>Neutral axis depth for 152 mm development length beam tests</td>
</tr>
<tr>
<td>Figure 7-34</td>
<td>Neutral axis depth for 203 mm development length beam tests</td>
</tr>
</tbody>
</table>
1 INTRODUCTION

1.1 GENERAL

Innovative and cost effective methods are required for the strengthening and rehabilitation of steel structures that are deficient due to the demand to increase the specified load and/or deterioration as a result of corrosion. A considerable amount of research has established the successful use of carbon fiber reinforced polymer (CFRP) materials for strengthening concrete structures. With the introduction of new high-modulus CFRP materials, the possibility for providing a solution to the ongoing problem of infrastructure deterioration may be extended to steel structures as well. While the transfer of the technology from one material to another may initially seem straightforward, it is complicated by several potential problems. More strengthening material is need to achieve a significant strength increase since steel is much stronger than concrete, especially in tension. As more strengthening material is added, bond stresses become more critical than for concrete structures since debonding failures do not occur in the substrate as in concrete structures. Effective use of the material requires careful surface preparation, adhesive selection and development length or else the material may be used less effectively due to high peeling stresses at the ends of the CFRP and potential shear lag effects. Further complications may also arise due to the potential for galvanic corrosion between the carbon and steel materials. Despite these challenges, CFRP materials have shown to reduce construction costs and also indirect costs, such as the disruption to the public and the environmental costs of disposal and replacement of older structures. The properties of CFRP also make it a viable alternative to conventional methods of strengthening since it is light weight, has high fatigue strength, and is resistant to corrosion. These low-cost and application benefits successfully tested of CFRP strengthening systems make this option a practical answer to offset the high demand by industry for a low-cost means of rehabilitating and strengthening steel structures. Since
many of the structures built in the post-World War II era are already past their design life, the inventory of deteriorated steel structures in need of rehabilitation can only be expected to increase (Tang and Hooks, 2001).

1.2 Research Objective

While FRP materials have been successfully used for flexural strengthening, shear strengthening and ductility enhancement of concrete bridge structures, there are an even greater number of steel structures in need of strengthening. In particular, two end users were identified in the preliminary stages of the research program that have an immediate need for strengthening and rehabilitation systems. These are the telecommunications industry and departments of transportation. Due to the increasing number of cellular phone users and the demand for improved service, the industry has required cellular phone companies to increase the number of antennas on monopole towers. This trend has been exasperated by the reluctance of communities to allow new monopoles to be built. The addition of new antennas increases the wind load acting on the monopole, requiring additional strengthening to match this demand. Existing techniques for strengthening monopoles with steel collars or with an additional lattice structure are costly and negatively affect the visual appearance of the structure. Transportation departments, who are facing tightening budgets, are also demanding strengthening and rehabilitation systems for steel bridges. The strengthening system must be cost effective and should not cause major interruption of traffic.

As such, the objective of this research was to first determine the fundamental requirements for strengthening steel structures, such as steel monopole towers and steel-concrete composite girders, with high modulus carbon fiber reinforced polymer (HMCFRP) materials. This main objective was specifically targeted at determining the most effective resin and adhesive for CFRP sheets and laminate strips respectively as well as their respective required development lengths for developing their full capacity on the steel structure.
1. INTRODUCTION

1.3 RESEARCH SCOPE

The most suitable saturating resins and bonding adhesives that could achieve full utilization of the material when bonded to the steel surface was determined through different tests methods. Both unidirectional dry fiber sheets applied through a wet lay-up process and adhesive bonded pultruded laminate strips were considered. Lap bond shear tests and four-point loaded flexural members were the selected methods for determining bond stresses and required development lengths used as selection criteria for the system. From these tests, a strengthening system using both types of high modulus CFRP was developed and proposed. This foundation research was later continued within the overall research program to examine further behaviors of this developed high modulus CFRP strengthening system on large scale members and testing of different parameters, including varying joint configurations, anchorage, fatigue and durability. A brief outline of the overall research program is introduced in section 1.4.

1.4 OVERALL RESEARCH PROGRAM

The development of the system for strengthening steel structures with high modulus CFRP materials was divided into experimental and analytical phases over a six year period. The experimental program was conducted in five phases. In the first phase suitable resins and adhesives were determined for use with the high modulus CFRP materials to determine a proposed strengthening system. After two years of testing, large scale members were tested to study the effectiveness of various strengthening configurations in Phase 2. The behavior of large-scale monopole towers strengthened with FRP materials and steel-concrete composite beams that were strengthened with unstressed and pre-stressed FRP materials were reviewed. Overloading and fatigue behavior of the strengthening system on steel-concrete composite beams were next examined in the third phase. A detailed study of the
1. INTRODUCTION

bond characteristics was conducted in the fourth phase; while the durability of the proposed system was evaluated in the fifth phase.

The first series of tests, which comprise the research presented in this thesis paper, determined the most suitable resins for the application of CFRP sheets by the process of wet lay-up as well as the most suitable adhesives for adhesive bonding of CFRP laminate strips. A double-lap shear specimen was used in the first part of this phase for the resin selection using the CFRP sheets. The selection of an adhesive for use with the CFRP strips, which have much greater flexural stiffness, employed a singly symmetric flexural specimen that was similar in configuration to a steel-concrete composite beam predominant of bridge structures. This type of specimen was also used to investigate the development length and bond parameters that were used to validate the results of an analytical model developed later in Phase 4.

With an understanding of these parameters, the remaining phases of the research program were completed by my colleagues Mina Dawood, Bryan Lanier, and David Schnerch. These remaining phases involved the strengthening of two types of structures commonly in use today, steel monopole towers and steel-concrete composite beams, during Phases 2 and 3, and a detailed study of the bond characteristics and durability of the system through double lap shear coupons and small test beams in Phases 4 and 5.

In Phase 2, three large-scale monopole towers were strengthened to provide strength increases in the service range (Lanier, 2005). An additional four large-scale steel-concrete composite beams were fabricated and strengthened with different configurations to provide increases in their stiffness and ultimate strength (Schnerch, 2005). A pre-stressing system was also investigated as a means to reduce the cost of the system by more effectively utilizing the CFRP material and to preserve the ductility of the structure.
Six small-scale steel-concrete composite beams were tested under fatigue and overloading conditions in the third phase to study the behavior of the strengthening system under severe loading conditions (Dawood, 2005). Behavior of the strengthened beams resulted in excellent serviceable condition after exhibiting significant residual deflections and sustained up to three million load cycles at simulated an increased live load level without showing any indications of degradation or failure.

In the fourth phase of the research, a detailed experimental program was conducted to study the bond behavior of the proposed strengthening system (Schnerch and Dawood, 2005). The different application details studied included the shape of the CFRP strip end, the presence of additional mechanical anchorage by either a transverse CFRP wrap or steel clamp, and the total length of the splice plate using finite lengths of CFRP strips that are practical for shipping. The double-lap shear coupon tests suggest that for some plate end configurations the presence of a steel clamp near the plate ends could further increase the joint capacity by up to 80%. However, results of the beam tests indicated that both types of mechanical anchorage near the CFRP end, the transverse CFRP wrap and the steel clamp, did not increase the joint strength. Beam tests without the anchorage indicated that use of proper detailing of a reverse tapered plate end for the CFRP strip can be used to enhance the serviceability of a long-span steel beam even if a splice joint is located at a location of relatively high moment.

The fifth and final phase studied the environmental durability of the proposed strengthening system (Dawood, 2007). A total of 52 CFRP-to-steel double lap shear coupons were tested using two different methods of enhancing the durability of the system. These methods included the separate and combination of an additional glass fiber insulating layer between the steel and the carbon fiber and the pretreatment of the steel surface with a silane coupling agent to enhance the resistance of the steel-adhesive bond line to ingress of moisture. The effect of sustained load was also considered. Results from these tests showed that the
The presence of the silane adhesion promoter greatly enhanced the durability of the strengthening system. Further while, the presence of the glass fiber increased the initial strength of the bonded joint; however, it did not enhance the durability.

The development of analytical methods for the designer was also a priority. As such a model was used to predict the inelastic behavior of strengthened steel structures and compared to the findings of the experimental program. Another model was also used to confirm the bond stresses and to allow the strengthening system to be applied to other structural configurations. The effects of different parameters in influencing the behavior of strengthened steel structures were also shown (Dawood, 2005). Based on the research findings flexural design guidelines were also presented.

1.5 THESIS CONTENTS

The contents of this thesis are as follows:

Chapter 2 is a review of previous work in the area of strengthening metallic structures with FRP material. This focused review included the flexural behavior of steel structures strengthened with standard modulus CFRP materials as well as high modulus CFRP. An examination of previous testing and the investigation of bond performance with different adhesive and adherend properties, as well as geometrical consideration of the joint and its long-term durability were reviewed.

Chapter 3 describes the details of the experimental program including the geometric and material properties of the test specimens, the testing configuration and the instrumentation used to measure various parameters.

Chapter 4 presents the experimental results of each series of tests conducted for determining the saturant resins and bonded adhesives used for further studies. In addition to describing
1. INTRODUCTION

the behavior of the strengthening schemes, their overall performance and associated failure modes are discussed.

Chapter 5 evaluates the behavior of a model used for predicting the performance of high modulus CFRP strengthening systems for steel-concrete composite beams. Comparisons of this model to experimental results were presented.

Finally, Chapter 6 summarizes the research conducted, presents conclusions and suggests the direction for future research.
2 BACKGROUND

2.1 INTRODUCTION

Although fiber reinforced polymer (FRP) materials have been used for the repair and strengthening of metallic structures in the aerospace, marine, automotive and manufacturing industries for many years, the use of FRP materials for the repair and strengthening of steel structures in the civil engineering industry has only recently been researched and applied. One reason for this recent investigation is the emergent market for repair of deficient infrastructure, especially within the transportation industry. According to McKenna and Erki (1994), approximately 40% of the bridges in North America are deficient to such a degree that they require some form of rehabilitation or replacement. In 2001, Tang and Hooks (2001) reported that of the 583,000 bridges in the USA, a total of 173,000 were functionally obsolete or structurally deficient.

Another reason is the need for strengthening existing structures to meet present and future load requirements incorporating the increased load demands (ie-traffic for highway bridges) for structures coming to an end of their design life or to strengthen members that have been damaged accidentally and by prolonged deterioration over time resulting in a reduction in structural capacity. Steel that has damage may require additional reinforcement to improve the flexural strength and cyclic fatigue performance and to reinforce local stress concentrations such as cracks or buckling within the web section of the member.

Major causes for deterioration in steel structures are attributed to the age of the structure, lack of proper maintenance, and prolonged deterioration due to environmental attack, such as extensive use of deicing salts in the winter. For steel structures, deficiencies caused by prolonged corrosion damage typically result in cross-sectional losses. Corrosion damage can
be widespread to weaken an entire structural member and/or come in the form of localized pits and holes, causing stress concentrations that result in crack initiation (Karbhari and Shulley, 1995). Corrosion can also reduce the flexural strength of a member, cause eccentricities in loading of the structure, cause web buckling or crippling and reduce the fatigue resistance of the member.

A third reason is the need for improved long term performance of repairs equal to or better than the conventional repair methods for steel. Conventional methods of flexural repair of steel structures involve post tensioning and welding or bolting steel plates to the tension flange of the member. Although proven, these methods can result in significant induced residual stresses from welding or local stress raisers and loss of cross-section from drilling holes. The attachment of additional steel plates can significantly increase the dead load of a structure, limiting the amount by which live load can be increased. Also, continued corrosion and damage of fatigue sensitive welded details due to repeated forms of loading, such as traffic loads on a bridge, can substantially reduce the long term benefits of conventional repairs.

There are also many advantages in favor of using CFRP materials for repair and rehabilitation of bridges and structures. Installation of CFRP typically requires very little effort in staging and lifting the material in place during installation. CFRP materials add minimal dead weight to a structure due to the high strength to weight ratio and there is typically little visual impact on the structure, such that good aesthetics can be maintained with little loss of bridge clearance. Due to the ease of application, disruption of traffic during construction may also be reduced or eliminated. Application of bonded FRP material also results in reduced stress-concentrations as compared to mechanical fastening and does not generate thermal induced residual stresses and heat-affected areas in the metal as welding (Grabovac et al., 1991).
Cost savings may also be realized despite the high material costs associated with FRP materials. These savings can come through reduced labor requirements for staging and lifting material during installation or can be evaluated for long term reduction in maintenance costs typically associated with standard repairs that are subject to corrosion. Example reports of these cost savings were reported by Moy et. al. (2001) when the project overall costs were reduced due to installation savings in the strengthening of tunnel supports for the London underground railway system with CFRP materials. In this project, the difficult access and the impossibility of a lengthy service shut down led to a short-term cost competitive for CFRP materials. Long-term cost benefits were even more favorable for Gillespie et.al. (1996b) due to the expected durability of the CFRP materials used. A cost analysis comparing the cost of rehabilitation with CFRP to the cost of replacement of a bridge with corroded steel girders showed that rehabilitation with CFRP was only 28 percent of the total cost of replacement, with most of the cost savings associated with the fact that there is no need to replace the concrete deck in the case of rehabilitation. From these and other examples, the high material costs of the CFRP material has not shown to significantly increase total costs since material costs are often a small portion of the overall project budget.

CFRP applications to steel have also learned from the widely established use of FRP materials for the strengthening of concrete structures in flexure and shear, as well as ductility for concrete columns. CFRP sheets may be applied on concrete surfaces by a wet lay-up process, building up the number of layers or plys necessary according to the strengthening requirements. Alternately, unidirectional CFRP strips can be manufactured to a desired width and thickness using a pultrusion process. For concrete structures, these strips can be directly bonded to the surface, or a groove may be cut into the concrete to allow for bonding of the strips within the groove using a near-surface mounted technique. For strengthening steel structures, near-surface mounting would be difficult to achieve and is nevertheless, not
necessary since bond failures will not occur within the substrate, as is often the case for FRP materials bonded to concrete surfaces.

It has been noted that the CFRP strips currently used to strengthen concrete structures are typically 4 mm in thickness (Hill, 2000). To achieve a reasonable level of strengthening for a steel beam, these same strips would have to be approximately 20 mm in thickness. Higher modulus materials have been recommended to reduce the thickness of the CFRP strips required for steel applications. However, most of the research to date on strengthening steel structures has been focused on the use of standard modulus CFRP materials, with a fiber tensile modulus of around 240 MPa that is approximately three-quarters that of structural steel (Bassetti et. al., 1998). Significant increases in yield strength and ultimate capacity of steel and steel-concrete composite beams were achieved when using standard-conventional modulus CFRP strengthening systems (Mertz and Gillespie, 1996; Sen et. al. 2001; Tavakkolizadeh and Saadatmanesh, 2003c; Al-Saidy et. al., 2004), although, a large amount of CFRP material or several layers of CFRP may be required to achieve an adequate increase of the elastic stiffness (Colombi and Poggi, 2006a).

Research by Tavakkolizadeh and Saadatmanesh, (2003c) demonstrated that increasing the number of CRFP layers reduces the utilization of the CFRP. In order to reduce the amount of CFRP needed to achieve a given stiffness enhancement, or to more efficiently use standard modulus CFRP materials, Schnerch et. al. (2006) recommends to pre-stress CFRP strips before bonding to the steel. With epoxy applied to the pre-stressed strip, the stress is maintained in the strip until the epoxy is fully cured. Once the epoxy is cured, the stress may be released. While, bonding of unstressed CFRP strips reduces the extra stresses due to live loads placed on a structure, bonding of pre-stressed strips also relieves existing dead-load stresses.
Recently, high modulus and ultra high modulus carbon fibers have been developed with a modulus of elasticity three times that of conventional steel (Mitsubishi, n.d.). These high modulus carbon fibers are produced as dry fiber tow sheets that can be impregnated with a saturating resin on-site using a wet lay-up technique. For applications requiring a higher degree of strengthening, these high modulus carbon fibers can also be pultruded into a precured laminate strip which can be subsequently bonded to the surface of the structure using a structural adhesive. The modulus of elasticity of these fabricated unidirectional composite laminates is approximately twice that of structural steel. Due to its high modulus properties, less material is needed than would be required if standard modulus CFRP were used. This was shown in the research conducted in this thesis, as well as (Lanier, 2004; Schnerch, 2005; and Dawood, 2005, 2007).

2.2 FLEXURAL BEHAVIOR OF STEEL MEMBERS STRENGTHENED WITH FRP MATERIALS

Various research approaches have been conducted in the study of FRP composite repair systems for steel wide flange beams. Some have considered repair of naturally deteriorated girders that represent expected retrofit field application. Others have considered simulating damage to the beam by notching part or the entire tension flange. This method however, creates high stress concentrations at the location of the notch, which does not accurately represent the effect of widespread deterioration. Still others have selected to strengthening only undamaged steel beams or steel-concrete composite beams that are representative of typical construction. This review considers all of the most up-to-date research available for the use of FRP materials in strengthening and repairing steel flexural members. Further research on bonded lap joints, fatigue strength, environmental durability, analysis and modeling, and design considerations have been discussed in detail by my colleagues Schnerch (2005) and Dawood (2005, 2007).
A good start to quickly understand the research has been done related to the use of FRP materials for strengthening metallic structures is to review several of the most recent published review papers that summarize this research. Some of the best reviews were described by Hollaway and Cadei in 2002. In their review they discussed the different techniques for installation of FRP materials on metallic structures. They focused on surface preparation techniques, material selection and installation requirements, while briefly presenting environmental durability issues. A number of specific field applications, primarily in the United Kingdom, were also presented to illustrate the ease and effectiveness of using FRP materials for rehabilitation of metallic structures.

In 2003, Shaat et al. presented a review of the research related to the flexural behavior of steel members strengthened with FRP materials. In every presented finding, the use of the FRP enhanced the flexural strength and stiffness of the steel members loaded under flexure. Further research on the bond behavior and environmental durability of CFRP strengthened steel structures, the use of high-modulus CFRP materials for strengthening steel-concrete composite girders and the use of FRP for strengthening compression members was also identified.

In 2004, Buyukozturk et al. presented the research related to debonding of FRP materials from steel surfaces. Four possible debonding mechanisms were identified. These mechanisms were adhesive failure at the steel-adhesive interface, cohesive failure within the adhesive layer, cohesive failure at the adhesive-FRP interface and delamination within the FRP material itself. They also reviewed two main analytical approaches which are commonly used to evaluate debonding of FRP materials, stress-based approaches and fracture mechanics-based approaches. Further research needs included the additional development of debonding models that take fatigue loading into account, continued experimental research to validate the current analytical models, environmental durability studies of bonded strengthening systems, non-destructive analysis methods to evaluate bond
integrity and the development of suitable failure criteria to predict debonding of FRP materials.

Also in 2004 Cadai, et. al. presented one of the first design guidelines for strengthening metallic structures for the civil engineering industry. Design and detailing of FRP materials to avoid debonding failures were presented, as well as sectional analysis to determine the quantity of FRP material required to generate a required strengthening level. Additional methods for strengthening brittle structures, such as historic cast iron bridges, and steel-concrete composite girders were proposed as well. Also, detailed analysis accounting for axial loads, thermal effects, continual loading, and prestressing of composite materials are reviewed.

In 2006, Schnerch et., al., presented test results showing the importance of proper detailing of the ends of the FRP strips for strengthening flexural steel members. Large scale steel monopoles and steel-concrete composite bridge girders were tested to evaluate intermediate and high modulus CFRP strengthening systems. Prestressing CFRP plates prior to bonding were also investigated. Current methods for determining bond stresses and their use for the design of FRP strengthening system for steel structures were presented. Shear stress distributions determined in the experimental program were compared to analytical models using a stress-based approach. Adhesive selection, surface preparation methods and means of preventing galvanic corrosion were also discussed.

In 2007, Zhao and Zhang identified the significant developments that have been recently made on the understanding of the bond behavior of FRP materials bonded to steel surfaces. They discussed the recent developments related to strengthening of tubular steel structures and repair of cracked steel structures with FRP materials. They also identified future research needs related to the development of bond-slip relationships for FRP materials.
bonded to steel surfaces, the development of fatigue crack propagation models, and further study on the lateral-torsional stability of FRP strengthened steel flexural members.

In 2008, Dawood and Rizkalla, summarized various aspects which have been considered to develop high modulus (HM) CFRP strengthening systems for steel flexural members. The system development included the adhesive selection, large-scale verification, fatigue and overloading behavior, bond characteristics and environmental durability of a proposed HM CFRP system. Flexural design guidelines were also presented.

2.2.1 Strengthening of Undamaged Wide-Flanged Steel Beams

Research into strengthening plain wide-flanged steel beams with CFRP materials applied to the tension flange has shown increases in the yield and ultimate strength and post-elastic stiffness of typical wide-flanged beams by reducing the strain in the tension flange. These benefits are highest under ultimate strength tests when the compression flange of the beam is stabilized with an addition of a bolted or welded steel plate. The strain profiles of strengthened wide-flanged beams with CFRP result with the neutral axis towards the strengthening material (Liu et al., 2001). Because the lever arm between the neutral axis and the CFRP strengthening material is small, the CFRP is not effectively utilized and may never reach its failure strain until after much of the remaining section is in compression. Continued yielding of the compression side section can make the lever arm even smaller. Thus, near ultimate loads, the stability of the compression flange becomes the dominant factor in controlling the behavior.

In an attempt to make more efficient use of the CFRP materials bonded to the tension flange of wide-flanged beams, Edberg et. al. (1996) tried to increase the distance between the bottom flange and the location of the CFRP strip by separating the two materials with an aluminum honeycomb foam core material. This scheme, out of a total of five different rehabilitation configurations of bonded CFRP plates and bonded pultruded glass fiber
reinforced polymer (GFRP) channels, was part of an initial study in the flexural rehabilitation of steel beams. Ten 1524 mm (60 inches) long W200 x 15 (W8x10) steel beams were tested. The beams were loaded under four-point bending. Testing first was conducted within the elastic range of the members for five cycles of loading. Increases in stiffness compared to the control beam resulted to up to 30 percent for the beam strengthened with CFRP strips bonded to the aluminum honeycomb that separated the CFRP material from the tension flange. These same beams were next loaded up to ultimate strength. Before they were loaded, the beams had an additional steel plate adhesively bonded and bolted to the compression flange to provide lateral stability and to raise the neutral axis depth that would imitate that of a girder acting compositely with a concrete deck. Results yielded an increase between 37 percent (GFRP) and 71 percent (CFRP) in yield strength for the various configurations studied. Most of the strengthening configurations failed in the ultimate strength tests by debonding of the composite reinforcement starting at their ends due to the large shear forces and curvatures present at the composite termination. Tapering of the composite over a sufficient length was proposed to prevent debonding failure in the future.

Four similar configured tests were conducted on the same W200 x 15 (W8x10) steel girders by Ammar (1996). These beams were strengthened directly with unidirectional CFRP strips that were 5.33 mm (0.21 inches) in thickness and 101.6 mm (4 inches) in width, with a 51 percent fiber volume fraction. This represented a reinforcement ratio of 14.5 percent based on the ratio of the cross-sectional area of the fiber, accounting for the fiber volume fraction, and the steel. The elastic modulus of these strips was 112 GPa, with an ultimate strain of 9 millistrain. Lateral buckling was prevented by bolting a 25.4 mm (1 inch) thick and 101.6 mm (4 inches) wide steel plate to the compression flange. Surface preparation of the steel consisted of sandblasting. The CFRP strips were sanded using 150-grit sandpaper. The bonding strips were adhered with epoxy and clamped during curing, with a clamp spacing of 300 mm (11 - 13/16 inches). The beams were loaded under four-point bending,
2. BACKGROUND

with a span of 1372 mm (54 inches) and a constant moment region of 203 mm (8 inches).
Stiffness increases of 23.3 percent and a 56 percent increase in the yield load was achieved
before debonding of the strips at 6 millistrain.

In 2002, Moy and Nikoukar investigated the effect of loading 1200 mm (47 inches) long 127
x 76 UB13 steel beams during the adhesive curing process to determine the resulting loss of
stiffness. The beams were strengthened with 7.6 mm thick high-modulus CFRP strips with a
tensile modulus of 310 GPa on the tension flange. Because the beams were not stabilized on
the compression side, strain distributions across the depth of the section were recorded.
During the loading process the measured strain distribution remained linear as the
compression steel yielded first. Cycling of the load just past yield throughout the curing
process reduced the flexural stiffness of the beams. It was found that if the maximum
adhesive stress during curing was limited to 1.0 MPa, the flexural stiffness was not reduced
and achieved results similar to a strengthened beam that had not been loaded during curing.

In 2003, Abushagur studied symmetric cross sections of steel W150 x 37 beams strengthened
with 19 mm (3/4 inch) thick GFRP plates bonded to the top and bottom flanges. The
reinforcement ratio was 29 percent, based on an assumed fiber volume fraction of 24 percent.
The results of the tests indicated that an initial stiffness increase of 15 percent was achievable
and that the yield load could increase 23 percent. The ultimate strength was increase by 78
percent before failure of the bonded GFRP plates by delamination between the plys on the
tension side of the beam. Despite the fact that the beams were heavily reinforced, little
improvement in the stiffness before yield of the beam was noted due to the low elastic
modulus of this material.

Patnaik and Bauer (2004) performed strengthening of three 3.15 m (12 ft) long built-up wide
flange beams in four-point bending. Two of the beams were strengthened by adhesively
bonding full length standard modulus CFRP laminate strips across the bottom flange of the
beams. An ultimate strength increase of 14 percent was recorded for the flexural specimens, with both beams failing by lateral-torsional buckling. No debonding was noted until after rupture of the CFRP strips.

2.2.2 Strengthening of Steel-Concrete Composite Girders

Earlier lessons learned from strengthening undamaged wide-flanged steel beams led more researchers to study the application of bonded CFRP strips as a repair and strengthening system for steel-concrete composite beams typical in building and bridge construction. The efficiency of this structural system, making use of the steel in tension and the concrete deck in compression, while also providing lateral support to the steel beam, provides better utilization of the bonded FRP strips. Although little enhancement to the stiffness of the steel-concrete composite beams has been shown when strengthened with CFRP materials, the ultimate strength of these types of beams has been greatly improved.

Some of the earliest research found in strengthening steel-concrete composite girders was from Tavakkolizadeh (2001). To investigate the effectiveness of the epoxy bonded CFRP sheets in repair and retrofit of composite girders, a total of six large-scale steel-concrete composite girders made of W 14 x 30 A36 steel beam and 7.5 cm. thick by 91 cm wide concrete slab were prepared and tested. Three different numbers of CFRP layers and three different damage levels in range were considered. The retrofitting test results showed that epoxy bonded CFRP sheet increased the ultimate load carrying capacity of composite girders and the behavior can be conservatively predicted by traditional methods. The repair test results also showed that epoxy bonded CFRP sheet could restore the ultimate load carrying capacity and stiffness of damaged steel-concrete composite girders. The ultimate capacity of the repaired beam was predicted by traditional methods of analysis of steel-concrete composite beams.
In a later study, Tavakkolizadeh and Saadatmanesh (2003a) studied the behavior of bonded CFRP laminate strips to steel-concrete composite beams. Three undamaged composite girders were tested, using one, three, or five plys of CFRP strips bonded to the outside of the tension flange. Considerable ultimate strength increases and insignificant elastic stiffness increases were observed. To potentially increase the elastic stiffness, strengthening with an increased number of plys of CFRP strips was evaluated. Utilization efficiency was shown to decrease as the number of plys was increased. No debonding was observed, despite the lack of mechanical anchorage, except for one specimen that was fabricated with improperly mixed epoxy that did not fully cure.

Two sets of steel-concrete composite beams loaded past the yield strength of the tension flange to represent damage that may be typical of severe overloading conditions were strengthened with externally bonded CFRP laminates and tested by Sen et al. (2001). The 6.1 m long girders consisted of W200 x 36 steel beams and a 710 mm wide and 114 mm thick concrete slab. Surface preparation was by sandblasting and cleaning with acetone. The CFRP strip was clamped to the surface of the beam with a wooded board in between to distribute the pressure. Both 2 mm thick and 5 mm thick laminates were used for the strengthening schemes. Ultimate and yield strength increases were effectively increased, however stiffness increases were small particularly for the thinner of the two types of CFRP laminate strips studied. This was due to modulus of the CFRP material used. It was noted that even for these specimens, the increase in the elastic region of the strengthened members might allow service load increases. The strain profiles for the beams with the 2 mm thick CFRP strips showed the strain profile is relatively linear until yielding. After yielding, the strain distribution became non-linear with the CFRP strip straining significantly more than the steel that it was bonded to. This was not the case for the 5 mm thick strips that either maintained the linear profile, or had a lower strain in the FRP relative to the steel tension flange.
Intermediate and high modulus CFRP materials were proven effective in increasing the elastic stiffness and ultimate strength of two large scale steel-concrete composite beams tested by Schnerch, (2005). Increases in the elastic stiffness by up to 10 percent and ultimate strength by up to 16 percent for the tested beams was determined for the intermediate modulus CFRP strengthening system; while increases in the elastic stiffness of 36 percent and ultimate strength of 45 percent were obtained for the high modulus CFRP material. It was also demonstrated that pre-stressing CFRP laminate plates prior to bonding to the steel surface could increase the efficiency of the strengthening system.

In a partner study by Dawood et. al. (2007), overloading and fatigue behavior of steel-concrete composite beams strengthened with high modulus CFRP was investigated. Of the six strengthened beams that were overloaded, all resulted in reduced residual deflections while increasing the elastic stiffness, yield load, and ultimate strength. The strengthened beams loaded under fatigue loads were capable of sustaining 3 million loading cycles with a simulated increase in live load of 20 percent without degradation of the measured elastic stiffness. The author compared his work with other studies and suggested that the presence of the CFRP materials can maintain a strengthened beam in good serviceable condition even after experiencing severe overloading conditions and fatigue loads.

### 2.2.3 Rehabilitation of Corroded Steel Girders

Another application of bonded FRP materials is the structural rehabilitation of corroded steel beams that have been removed from an existing structure, such as bridge girders. For beams and bridge girders with natural damage of the tension flange, rehabilitation with the CFRP typically has restored the lost strength of the beams to levels comparable to those of undamaged girders. This was shown by Gillespie et al. (1996a) when four severely corroded steel bridge girders were removed from a bridge that was to be demolished. The cross-section loss due to the corrosion was uniform along the length and concentrated around the tension
2. BACKGROUND

flange and web of each girder. Stiffness losses tested before the rehabilitation ranged from 13 percent to 32 percent from all four beams. Two of the girders were rehabilitated using CFRP strips bonded to the top and bottom surfaces of the tension flange. The elastic stiffness of both chosen beams was 62 percent and 87 percent before rehabilitation. After rehabilitation, elastic stiffness was restored to 83 percent and 97 percent of the original girders, respectively. Similarly, the plastic moment capacities of these two beams were increased to 85 percent and 113 percent. The difference between the two results was accounted for the difference in the amount of corrosion between the two girders. Ultimate strength of the girders was reached when buckling of the compression flange occurred. The CFRP strips were not fully utilized, but did significantly shift the neutral axis toward the tension flange.

2.2.4 Rehabilitation of Notched Steel and Cast Iron Beams

Most often it is not possible to obtain corroded girders from existing structures; it is more convenient to simulate the corrosion damage in some form. Thus, for nearly all of the research completed to date, the use of partial-depth or partial-width notches, have been used to reduce the cross-section of the beam, simulating corrosion damage. Notching of beams before strengthening also maintains a typical strain profile through the cross-section, with the neutral axis occurring near its centroid. This allows a greater lever arm between the neutral axis and the applied CFRP material, making its use more effective.

The first such study was conducted by Barnes et al. (1999) in testing cast iron beams that were notched to simulate the presence of corrosion. Approximately 70 percent of the tensile flange of the cast iron was notched. The notched beam before rehabilitation was found to carry 33 percent of the load of the undamaged beam. After rehabilitation using high modulus CFRP strips, the capacity of the beam was restored to 96 percent of its original capacity using reinforcement equivalent to 20 percent of the original tension flange area. Failure was
by rupture of the CFRP strips at their ultimate elongation, with no evidence of an adhesive bond failure.

Testing of one undamaged beam, one notched beam, one notched beam with a full-length bonded CFRP strip and one notched beam with a quarter length bonded CFRP strip were conducted by Liu et al. (2001). As compared to the other studies of notched beams, a particularly severe notch was created, likely resulting in the premature debonding failure of the CFRP strips. Failure modes for the undamaged beam and for the notched beam were by lateral torsional buckling. Failure modes for the two strengthened beams were by debonding of the CFRP strips. Although, the strength of the original section was not completely restored, an increase in the strength and stiffness was achieved in comparison to the notched beam without strengthening.

Similar results were found for six S130 x 15 beams that were damaged using partial-depth notches across the bottom of the tension flange (Tavakkolizadeh and Saadatmanesh, 2001a). Notches were generated using a band saw, with three notches cut to a depth of 3.2 mm and three notches cut to a depth of 6.4 mm across the tension flange. These notch depths corresponded to 40 percent and 80 percent cross-section loss of the tension flange, respectively. Three different lengths of CFRP strip were used to rehabilitate each beam. The stiffness of the shallow notched beam rehabilitated with the longest strip exceeded the stiffness of the intact section, whereas for the other tests the stiffness of the rehabilitated beams was not less than 93 percent of the intact section. Similar performance was noted for the ultimate load. In all of the tests of the beams with the deep notch, failure was by debonding of the patch. For the beams with the shallow notches, the tests had to be stopped due to lateral-torsional instability before the bonded CFRP material completely debonded. However, some debonding was observed at the ends of the CFRP strips.
Rehabilitation of steel-concrete composite beams was also investigated (Tavakkolizadeh and Saadatmanesh, 2003c). Unlike the previous tests, the notches were cut from the sides of the tension flange, rather than from the bottom. The notches represented 25, 50 and 100 percent losses of the tension flange. The beams were strengthened with one, three, and five plys of CFRP, respectively. Failure of the one ply strip was by rupture of the fibers, while debonding progressed from mid-span resulting from the stress-concentration generated by the notch for both additional ply repair methods. In all cases, the elastic stiffness of the original beam was nearly restored by the rehabilitation. This finding differs from the findings of a similar study by Lee et al. (2005). The rehabilitated girders in his study were able to match the strength of the undamaged girder but not the initial elastic stiffness. Ten scaled steel-concrete composite beams were tested intact, with part of the tension flange removed and rehabilitated with CFRP strips.

Al-Saidy et al. (2004) also performed tests on steel-concrete composite beam that had a portion of the flange removed to simulate corrosion damage. Shear stresses in the elastic range were highest at the ends of the bonded CFRP plate. After yielding of the steel, higher shear stresses were noted near the quarter points rather than at the ends of the CFRP. However, for these beams shear stresses determined from the strains were less than 1.2 MPa.

In all of the previous studies, the method of reducing the cross-sectional area of the steel was by some form of cut. This type of damage may be considered to be particularly severe compared to the usual overall loss of cross-sectional area resulting from corrosion. Furthermore, a cut results in a different adhesive shear stress distribution, making debonding more likely than may be the case for a generally corroded beam. Correspondingly, for nearly all of these tests, some form of debonding was evident even in the full-length rehabilitation used by Liu et al. (2001).
A different approach for simulating corrosion damage was performed by Photiou et al. (2004) for modified steel HSS sections. Each beam had 50 percent of the wall thickness removed on the tension side by a milling process. Four beams were rehabilitated using two different rehabilitation processes with both standard and high modulus CFRP material. Two of these beams used a prepreg and vacuum bag system utilizing a U-shaped hybrid system. The other beams were strengthened with high modulus CFRP material bonded to the soffit of the beam only. For the first beam strengthened using the high modulus CFRP material and U-shaped strengthening configuration, the original strength of the beam was restored, with failure occurring due to rupture of the CFRP material at its ultimate tensile stress. By comparison, the beam strengthened with the standard modulus CFRP material and U-shaped strengthening configuration, reached a higher deflection and a similar strength due to additional thickness of material being used. The beam strengthened with the high modulus material on the soffit alone resulted in the load-deflection behavior similar to the first beam and resulted in rupture of the CFRP strips followed by debonding from the rest of the beam. In all cases, the full plastic strength of the beam was restored.

2.2.5 Strengthening of Tubular Members

There are a few researchers that have investigated the use of CFRP for retrofitting hollow steel structures and tubular thin walled members. These investigations have focused on studying bond, flexural and axial compression behavior of the strengthened members. Researchers that have studied bond and flexural behavior are presented in this section.

Vatovec et. al. (2002) tested a series of square hollow steel sections strengthened with varying configurations of CFRP strips bonded to the tension and compression flanges of the tube. The tubular members were 152 mm in depth with a span of 3048 mm and were partially filled with concrete to prevent buckling of the compression flange. The reported increases of strength varied from 6 to 26 percent depending on the configuration and number
2. BACKGROUND

Failure of the beams generally occurred due to premature bonding on the tension flange, and debonding and buckling of the CFRP in compression. No meaningful difference in stiffness between the unstrengthened tubes and strengthened tubes was observed.

Photiou et al. (2005, 2006b) tested four 80 mm by 120 mm rectangular hollow steel sections that were strengthened using hybrid configurations of double ply pre-impregnated unidirectional CFRP laminate strips and triple ply pre-impregnated 45°± GRFP sheets. All beams had the same hybrid lay-up of both CFRP and GFRP composites, but for each of the configurations either a high modulus or an ultra high modulus CFRP was used. Two of the beams were strengthened with hybrid U-shaped pre-impregnated wraps that were bonded to the tension flange and up both sides of the web of the beams. The other two were strengthened with a hybrid prepreg composite plate on the soffit side only. The tubular members spanned 1800 mm and were loaded in four point bending. Beams strengthened with the high modulus CFRP reached higher ultimate loads and had a more ductile response than the beams strengthened with the ultra high modulus CFRP. The U-shaped wrapping scheme helped to confine the rupture failure of the hybrid composite to a localized region and provided residual composite action of the materials to the steel beams, sustaining loads after failure occurred. By contrast, beams bonded with the CFRP plate alone failed by complete debonding.

Lanier (2005) tested three 12-sided hollow steel monopole towers scaled and tapered to represent those used by the telecommunications industry. The towers were strengthened with partial lengths of high modulus and intermediate modulus CFRP laminate strips and sheets. The strengthening materials was bonded on the top three and bottom three faces of each tower and layered at the base. All were tested in a cantilever configuration with a point load applied to the tip to simulate the effects of wind. Stiffness increases up to 39 percent as measured by the tip deflection were reported, while strength increases were insignificant due
to the relatively low failure strain of the CFRP materials. Buckling of the compression faces near the base followed by rupture or debonding of the tension composites and crushing of the compression composites were the modes of failure for all three towers.

Bond behavior of high modulus CFRP bonded to hollow tubular steel sections was investigated by Fawzia (2007) from six tests consisting of two lengths of very high strength steel tubes butt joined together with five layers of high modulus CFRP sheets around the outer surfaces of the tubes. Bond overlap lengths ranging from 40 mm to 85 mm were considered. Additional steel plate double lap shear coupons fabricated with the same CFRP materials and bond lengths were constructed for comparison. Tests indicated that the curvature of the steel tubes did not have any effect on the bond behavior of the CFRP as anticipated. Considerable shear lag effects were also noted when strains on the outer CFRP layer were measured approximately 50 percent lower than the measured strains at the inner most CFRP layer. This behavior indicated that the wet lay-up procedure by hand for multiple layers has further needs for research study.

Seica and Packer (2007) developed a CFRP strengthening system for circular hollow steel members used in underwater applications. Four members were strengthened and allowed to cure underwater, while two members were strengthened and room temperature cured in air for comparison. Flexure tests of the beams indicated that conventional methods of curing provide a greater amount of flexural strength and ductility than curing underwater. The beams cured underwater did however provide moderate increases in strength and ductility compared to the unstrengthened member.

2.2.6 Strengthening of Cast Iron Beams

The earliest metallic structures, particularly in Europe, were largely constructed of cast and wrought irons (Hollaway and Cadei, 2002). In recent years, FRP materials have also been introduced for the repair of these historical structures. Since strengthening of cast iron beams
with FRP materials is potentially more advantageous than steel beams due to its brittle behavior, the behavior of FRP composites bonded to cast iron was also briefly reviewed to determine the lessons learned from recent research and field application repairs.

In 1998 Moriarty published tests on standard and high modulus CFRP strengthened cast iron beams using a cast iron formulation that was reproduced to simulate historical cast iron used in the 1860s. The high modulus CFRP strips bonded to the tension side of the beams were 1.7 mm and 3.4 mm in thickness. Ultimate strength increases of these two strengthening configurations were 66 and 107 percent, respectively, when compared to the unstrengthened control beam. Similar tests using the standard modulus CFRP strips showed no increase in strength. Improvement in the initial flexural stiffness and an increase in deflection at failure were also noted.

Rostásy et al. (2005) has also conducted tests of cast iron beams that were modeled from those used in a historic building. Some of these modeled beams were artificially notched to simulate the effect of internal defects with the historical cast iron. Two types of CFRP strips were used with a tensile modulus of either 170 GPa or 276 GPa. The beams tested without a notch resulted in an ultimate strength increase of 22 percent using the lower modulus CFRP strips and 66 percent increase using the thicker higher modulus CFRP strips. Tests also showed that the strain distribution remained linear across the depth of the beams and that the strengthening effect of the CFRP strips was even more pronounced for the notched specimens.

Moy et. al. (2004) investigated the use of externally bonded CFRP for the strengthening of wrought iron structures. Their research showed that application of CFRP to both the tension and compression flange of wrought iron beams provided a much more effective strengthening system than bonding to the tension side only. CFRP was also noted to help prevent possible delamination of the wrought iron in the compression zone.
Other research that received much public attention was from both Hill et. al. (1999) and Leonard (2002) for the extensive experimental and numerical investigations on the use of resin infusion of CFRP materials for the retrofit of overstressed cast iron compression struts located within the London Underground system. In both projects, the application of the externally bonded CFRP effectively reduced stresses in the overstressed members.

2.2.7 Use of Prestressed CFRP Strengthening Systems

Prestressing CFRP materials is another way to effectively utilize bonded CFRP systems. While the limiting strain for steel structures is normally its yield strain, many types of FRP materials have significantly higher failure strains. This is especially true for lower modulus CFRP materials and all GFRP materials. Thus, certain metallic structures, such as cast iron, that have a low strain to failure results in inefficient use of the CFRP material unless it is prestressed (Hollaway and Cadei, 2002). Prestressing CFRP materials can also significantly increase the stiffness and strength capacities of steel members under service loads when enough prestressing forces are applied such that the CFRP fails at yield strain of the steel (Schnerch, 2005).

The Hythe Bridge, in Oxford, England is one cast iron bridge that was successfully strengthened with prestressed CFRP strips (Luke, 2001A). A proprietary technique was used to prestress the strips and jack them to the required strain before being bonded to the structure. The stressed CFRP strips were then secured to the main structure by adhesive bond and by both an anchorage system using high strength bolts and an end tab, which was also used to stress the strip.

This technique was further investigated for cast iron beams that were taken from a demolished structure and tested by Luke (2001a). These beams were prestressed up to 25 tonnes with CFRP strips using a hydraulic jack reacting against a temporary jacking frame fixed to the anchorage and cycled without any sign of failure. The prestress force was
designed to remove all tensile stress from the cast iron beams under the full 40 tonne weight limit of the bridge, since cast iron is brittle in tension.

In 2005, Schnerch investigated the effectiveness of pre-stressing high modulus CFRP strips on a steel-concrete composite girder. The test beam consisted of a W310x45 steel section acting in composite action with an 840x100 mm reinforced concrete deck slab. The applied CFRP material, which had an elastic modulus of 460 GPa and an ultimate strain of only 0.4 percent, was pre-stressed only to a stress level corresponding to 18 percent of the ultimate strength of the CFRP. After strengthening, the beam was loaded monotonically to failure using a four-point bending configuration with a total span of 6400 mm and a 1000 mm constant moment region. Because the pre-stressed beam was designed primarily to increase the stiffness, without increasing the flexural capacity of the section, a stiffness increase of 31 percent was only reported. Installation of the pre-stressed CFRP strip improved the serviceability of the member while maintaining the full ductility of the original section.

A different steel beam strengthening technique using tensioned CFRP materials was completed by Lee et al. (2005). Rather than by bonding CFRP strips, this technique involved the post-tensioning of CFRP bars that were external to the structure. In this case the CFRP bars were embedded in steel tube anchors with threaded ends. These tubes were then connected to the steel beam by means of stiffened angles that were bolted near the top surface of the tension flange. The total force applied to the CFRP bars was 53 kN, or 36 percent of the ultimate tensile strength of the bars. Although load testing showed negligible strain reductions compared to load testing completed before the strengthening, the dead and live load moments were reduced by 3-5 percent allowing additional live load capacity. The post-tensioning was removed from the bridge after two years and showed relatively small losses, expected to be due to slippage of the anchorage.
2.3 Field Applications

FRP materials were first used for repair and strengthening of metallic structures for aerospace and mechanical engineering applications where the high cost of the material was not a significant drawback. After many years of use, there recently have been several field applications where CFRP materials have been used to strengthen steel beams, cast iron bridge members and historic significant structures. Many field applications are from Europe due to the number of historic structures and bridges still in use, although recent applications have been performed in the United States. Found examples of several successful CFRP applications are reported.

2.3.1 Steel and Cast Iron Bridge Strengthening with CFRP

Several steel bridges in the United Kingdom were strengthened using CFRP laminate strips. Luke (2001B) represented the Slattocks Canal Bridge that consisted of rolled steel joists spanning 7.62 meters and supporting a reinforced concrete deck. Each steel joist was strengthened using 100 mm wide and 8mm thick CFRP laminate strips bonded to the tension flange. A London Underground steel bridge at Acton in West London was also strengthened with CFRP strips to reduce the live load stresses by 25 percent (Moy and Nikoukar, 2002). Since the bridge carried insignificant dead load and experienced cyclic loading from train traffic only, the reduction in live load stresses was expected to have a significant beneficial effect on the fatigue life of the bridge.

Hollaway and Cadai (2002) presented several successful CFRP applications to cast iron bridges in the United Kingdom (UK). The Tickfor Bridge in Newport Pagnell was strengthened with CFRP sheets using a wet lay-up process that was particularly suited for strengthening the curved surfaces of the historic beams. The King Street Railway Bridge in Mold was strengthened with CFRP laminate strips that helped strengthen six cast iron girders.
2. BACKGROUND

to allow 40 tonne vehicles to use the bridge. Unlike most CFRP strips that carry only the live load applied to the beam after strengthening, these CFRP strips also carried a portion of the dead load of the bridge. This was accomplished when the temporary supporting struts, used to relieve a portion of the bridge load during the strengthening process, were removed after the installation resulting in partial prestressing of the CFRP strips.

At least five bridges have been strengthened with CFRP in the United States as demonstration projects used primarily to monitor the long term performance and durability of the applied CFRP material. Two bridges in Delaware, Bridge 1-704, which carries southbound I-95 traffic over Christina Creek, (Miller et al., 2001) and the Ashland Bridge (Chacon et. al., 2004) were strengthened using the same strengthening system consisting of 5.25 mm thick CFRP strips. Both steel-concrete composite bridges did not require any particular amount of strengthening. For Bridge 1-704, one single girder was strengthened; the presence of the CFRP material was found to reduce the measured strain in the tension flanges of the steel girder by 15 percent and increase the stiffness of the girder by 12 percent. These results were determined from vehicle load tests performed before and after the strengthening. For the Ashland Bridge, two girders were selected for retrofit and a new concrete deck was cast to act in complete composite action with the steel girders after the CFRP system was installed. As a result, the CFRP strengthening system contributed to the support of the concrete deck dead load.

The Sauvia Island Bridge in Washington (Mosallam, 2005) used a proprietary CFRP strengthening system developed from an experimental program started by Edberg et. al. (1996) to strengthen the entire bridge. The strengthening system consisted of a CFRP laminate strip bonded to an aluminum honeycomb core. The purpose of the aluminum core was to increase the distance of the CFRP strip from the neutral axis of the steel beam, thereby increasing the stiffness of the member. Installation was successfully completed in only 5 hours. Monitoring of attached strain gauges on the steel and the CFRP, as well as
2. BACKGROUND

performance of pull-off tests of samples placed at several different locations along the bridge is underway.

Four girders of a three span bridge over Walnut Creek in Pottawattamie County in Iowa were strengthened with conventional CFRP laminate strips by Phares et al., (2003). Only the positive moment regions of this bridge were strengthened. The CFRP strips were bonded to both the bottom and top surfaces of the tension flanges at several locations. Different strengthening configurations were applied with the intent of examining the long term durability, including the effect of strengthening on the top side of the tension flange on an edge girder exposed directly to environmental exposure. A primer was applied to the steel immediately after sand blasting of the steel surface to prevent direct contact between the CFRP and the steel, reducing the potential for galvanic corrosion and to promote a more durable adhesive bond. Load tests of the girders before and after the CFRP strengthening revealed similar measured strains in the steel tension flange of the girders. Analysis also showed that stiffness of the bridge girders could modestly be increased by 1.2 percent per ply of CFRP strips (Wipf et al., 2005). This was likely due to the relatively small amount of CFRP material installed and the low modulus of the CFRP selected.

The first bridge in the United States that was strengthened with unbonded external post-tensioning CFRP tendons was a bridge over Willow Creek, west of Bayard, Iowa (Phares et al., 2003; Lee et. al., 2005). The CFRP tendons were mechanically anchored and connected to the web of the bridge girders using stiffener angles and high strength bolts. The existing steel girders of the bridge supported a non-composite reinforced concrete deck. Load testing of the bridge after strengthening showed that the bridge stiffness essentially was unchanged, but the load capacity of the bridge was increased due to an induced negative moment equal to approximately 3-5 percent of the live load moments. Additional load tests after two years of service indicated essentially no change in behavior. When the post tensioning CFRP tendons
were removed to evaluate the post tensioning losses, the average measured loss was only 5 percent of the applied post tensioning force.

The first metallic structure in the world strengthened with prestressed CFRP laminates was the cast iron Hythe Bridge in Oxford, England (Luke, 2001a). The bridge, originally constructed in 1874, consisted of eight inverted tee-section cast iron beams that spanned 7.8 meters, with brick jack arches and infill material between the flanges and the deck. Assessment showed that it was capable of supporting 7.5 tonnes, but required strengthening to 40 tonnes to continue its use in one of the busiest sections of the city. Since the bridge could not be closed due to its essential use, the installation of prestressed CFRP was selected. The success of the prestressed CFRP installation permitted the bridge to be free from tensile stresses during normal traffic loading.

2.3.2 Strengthening Applications Using High Modulus CFRP

There have been several field applications demonstrating the use of high modulus CFRP materials particularly suited for strengthening steel structures. One example was found from Hill et al. (1999) who showed the application of high modulus materials as an effective repair to cruciform cast iron struts for a vent shaft in the London Underground. High modulus fibers were selected because the struts would partially be loaded during the strengthening process, resulting in a relatively small remaining strain capacity. The strength capacity under service loads governed the design, and it was determined that the high modulus fibers could contribute much more to the strength of the members than if lower modulus fibers were used.

High modulus materials were also used to repair a steel tank 27 meters in height and 1.9 meters in diameter that was part of an industrial plant (Nakagoshi et. al., 2000). Corrosion of the steel led to a loss in cross section of up to 50 percent. The high modulus CFRP materials were used to reinforce the top 4 meters of the tank, where most of the
2. BACKGROUND

corrosion damage was observed, and the bottom 6 meters of the tank, to prevent buckling as a result of seismic events.

2.4 BOND CHARACTERISTICS

The advantages of installing FRP composite materials to steel through adhesive bond on the steel surface in lieu of bolting or other mechanical methods are that adhesive bonds can more uniformly transfer stress along the length of the composite and result in lower stress concentrations. The anisotropic nature of most CFRP materials would also preclude bolting as a connection method, since the strength of these materials perpendicular to the fiber direction is relatively low. High local stress concentrations around bolts have a tendency to split the fibers.

A high degree of performance is necessary from the bond between the CFRP materials and steel in order to fully utilize the applied strengthening material. The basic requirements for good bond are direct contact between the adhesive and the steel and CFRP substrates, and the removal of weak layers or contamination at the interface (Hutchinson, 1987). It is difficult to verify the quality of the bond after installation, thus a meticulous approach is needed to remove local defects that can result in complete debonding of the CFRP material when the bond is stressed.

The durability of an adhesive joint will depend mainly upon the exposure of the bond to extreme temperatures and/or moisture. The thickness of the joint must also be considered in order to prevent any possible galvanic corrosion to occur between the anodic steel material and the cathode CFRP bonding material.
2. BACKGROUND

2.4.1 Mechanics of Adhesion

Adhesion in general can be explained as four possible mechanisms: absorption, mechanical interlocking, diffusion and electrostatic attraction (Mays and Hutchinson, 1992). The most favored between FRP materials and steel is the absorption mechanism, with mechanical interlocking also playing an important role. When the adhesive and substrate are in direct contact, the molecules of the adhesive can be physically adsorbed onto the surface of the substrate through van der Waal’s forces. Bonds that rely only on these van der Waal’s forces will nearly always result in desorption of the adhesive from the steel substrate when water is introduced at the interface in sufficient quantities (Gettings and Kinloch, 1977). A more environmentally stable bond requires the presence of primary bonds, since water molecules cannot easily break these bonds at the interface. Primary bonds can be achieved by the use of silane primers in adhering steel interfaces, where the silane primer forms primary bonds with the steel substrate resulting in a much more environmentally stable bond.

The adhesive and substrate become mechanical interlocked when the applied liquid adhesive fills into pores of an irregular surface and hardens. A rough surface improves energy dissipation and helps assure that any cracks formed near the adhesive-substrate interface will change direction as a result of the irregular surface, diverting any cracks into the body of the adhesive. Additionally, rough surfaces have a larger surface area than a smooth one to allow for more absorption of the adhesive. Too much roughness may be detrimental, since large surface irregularities create interfacial stress concentrations. Deep voids in the surface are also likely sources for trapped air if proper wetting is not achieved. Optimum surface roughness or profile will vary from one adhesive to another (Sykes, 1982).

2.4.2 Adhesion Selection

There are certain requirements adhesives should meet for effectively bonding FRP composite materials to steel. These requirements depend on the type of composite, the application
2. BACKGROUND

process, and the environment of its application. Adhesives on the market today have been
developed to meet certain requirements demanded by the industry that uses them. The most
important are highlighted in this section. It is from these requirements and research findings
that the adhesives used in the experimental program were chosen.

To optimally utilize the composite material used to strengthen or rehabilitate a steel member,
the adhesive must first have adequate bond strength for bonding the composite to the steel
(Gillespie Jr., et al., 1996A). Failure of a strengthened or rehabilitated steel member should
not be by premature debonding of the composite material; instead, the system should expect
good bond strength within the adhesive layer so that it is governed by the ultimate stress
capacity of the FRP composite. Surface preparation plays an important role for this not to
occur. The type of FRP composite material used to bond to steel and the type of steel
substrate will help dictate the selection of the adhesive for the system. For example, tests by
Nakazawa (1994) have shown that galvanized steel typically does not respond as well as
standard cold-rolled steel when certain adhesives are applied to it.

Secondly, the adhesive must be sufficiently durable in the environment of its application for
the steel member to match the life expectancy of the structure. For bridges this life
expectancy is typically 75 years (Gillespie Jr., et al., 1996A). Various test methods are
implemented to determine the durability of the adhesive in extreme temperature and
corrosive environments. The most common are the wedge test and lap-shear tests. Adhesion
promoters, such as silanes, can also be applied to the steel surface following the surface
treatment for applications that require long term durability. These agents are designed
specifically to enhance bonding between the surface oxides of metals and polymers.

Thirdly, the adhesive application process should be easy to utilize in field conditions. To
provide an adhesive that can facilitate this process, the glass transition temperature (Tg), pot
life of the adhesive after mixing, and the cure cycle for the adhesive should all be considered
before application. If an adhesive is used beyond its glass transition temperature, creep effects may become significant. Rajagopalan et al. (1996) recommends a glass transition temperature of at least 60°C for infrastructure applications. Generally it is desired that the entire bonding process, from the start of mixing the adhesive components to the clamping of the joint be completed within the pot life of the adhesive. Working an adhesive beyond its pot life may affect the bond strength, since a portion of the adhesive has completed the reaction process beyond the pot life. The pot life and rate of cure are also important to determining the amount of material that can be applied at one time. Low pot life time is admissible for small scale operations, though adhesive with infinite pot life, such as single component epoxies, are a must for large scale structures (Gillespie Jr., et al., 1996a).

Prevention of the formation of galvanic couples is also an important parameter in selecting an appropriate resin/adhesive system (Rajagopalan et al., 1996). Although this is described more fully in Section 2.4.6, adhesives should provide an electrical barrier between the steel and carbon fiber to reduce the rate of galvanic corrosion of the steel. According to the research completed by several researchers, amine-epoxy adhesives have shown to help prevent galvanic corrosion between carbon and steel (Tavakkolizadeh and Saadatmanesh 2001). Also, since amine-epoxy adhesives are known for their high strength and stiffness properties, most resins and adhesives available for bonding steel and CFRP materials are amine-epoxy.

Different types of adhesives have been used to bond CFRP to steel, but generally room-temperature cured epoxies have been chosen due to their superior performance and ease of use. Components that make up epoxies include the resin, flexibilizers, tougheners, fillers and hardeners (Mays and Hutchinson, 1992). The role of the flexibilizers is to improve the impact resistance and peel strength of the adhesive. Tougheners absorb fracture energy and are especially important in considering the fatigue behavior of the joint. Fillers may be used to reduce the cost of an adhesive or to improve the gap filling capability of an adhesive. The
hardener controls the pot-life or workability time of the adhesive and is typically considered the most important component of the adhesive.

2.4.3 Surface Preparation for Steel and CFRP

Surface preparation for steel is needed to enhance the formation of chemical bonds between the steel surface and the adhesive. Most surface treatment involves cleaning, followed by removal of weak layers and then re-cleaning (Mays and Hutchinson, 1992). Thus, an important first step is the removal of surface dust, grease, and other contaminants. This can be achieved by wiping or brushing the surface with an appropriate solvent. To prevent recontamination of the surface after the solvent evaporates, sufficient amounts of solvent should be used. Additionally, solvents should never be reused (Cadai et al., 2004).

Weak layers on the steel surface, such as paint, mill scale or some other corrosion product, should next be removed to expose the competent material beneath them. These layers are usually very thin and loose, and can cause premature bond failure unless removed. From all of the research, the most effective means for removal of these weak layers and that achieved the highest bond strengths was grit blasting. Grit blasting creates a chemically active steel surface that promotes the chemical bond and mechanical interlock of the adhesive to the steel adherend (Sykes, 1982, Holloway and Cadai, 2002). Grit blasting is also preferred over mechanical abrasion as abrasion tends to trap contaminants near the surface of the material (Cadei, et. al., 2004). Scnerch (2005) discovered that the composition and size of the grit must also be compatible with the applied adhesive used with the strengthening material, as different grit particles vary the surface profile required for best bond with the adhesive. Also, any grit that is to be recycled during the blasting process should be degreased with a suitable solvent to prevent contamination of the grit.

The final step before application of the CFRP composite material to the prepared steel is to re-clean the steel surface to remove any fine abrasive dust that may remain embedded on the
steel surface. Holloway and Cadei (2002) assert that this dust should be removed by either dry wipe or by a vacuum head with brushes. Solvent wiping should be avoided, since wiping only partially removes the dust and potentially redistributes the dust along the surface. However, several different studies have shown that solvents may be used to clean the surface after grit blasting without resulting in poor bond performance (Dodiuk and Kenig, 1988, El Damatty et. al., 2003, Tavakkolizadeh and Saadatmanesh, 2003C, Al Saidy, 2004, Lanier, 2004, Photiou et. al., 2004). Solvents should also be applied in excess to remove the dust from the surface without getting redeposited after the solvent evaporates.

After the surface preparation of the steel by grit blasting and re-cleaning, it is important to install the initial adhesive or primer application within the shortest time possible to minimize the possibility of recontamination or oxidation of the steel surface (Allen et. al, 1988). Too long of a time after the surface preparation can result in adhesive failure between the steel substrate and the adhesive, as shown by Matta et al. (2004), when performing fatigue tests on cracked and uncracked steel specimens repaired with CFRP materials after three days from the steel surface preparation. Most studies applied the composite strengthening material immediately after surface preparation of the steel.

Although not practical for rehabilitation and structural applications on site, chemical surface treatments are also available for preparing the steel surface (Holloway and Cadei, 2002). One example is the use of phosphate treatments widely used in preparing steel for painting. According to a study completed by Sykes (1982), the use of phosphate treatments may increase bond strength and prevent debonding and corrosion at breaks within the coating.

Surface preparation for the CFRP materials depends upon how the materials are fabricated and shipped. Pultruded laminate strips are often fabricated with a sacrificial peel ply layer on one or both sides of the strip. The peel-ply is a sacrificial layer of glass fiber and polymer material that may be removed immediately prior to bonding to reveal a clean and textured
2. BACKGROUND

surface of the CFRP. If a peel ply is not available, the surface of the strips should be lightly abraded and cleaned with an appropriate solvent. Holloway and Cadei (2002) recommend abrading the pultruded strips on the side to be bonded with sandpaper and then clean the surface with a solvent. West (2001) recommends abrasion by bead blasting. Abrading too much may damage the fibers within the strip. No surface preparation is necessary if dry fiber sheets are applied by wet lay-up in situ on the structure (Cadei et. al., 2004). Inspection is recommended of the dry fiber sheets and pultruded laminate strips prior to installation for any signs of contaminants that may affect the bond performance and damage, such as splitting, warping, or fiber breakage that may have occurred during the manufacturing, shipping, or handling of the materials.

2.4.4 Silane Adhesion Promoters

Adhesion promoters have been found to increase the durability and bond performance of a joint by resisting water infusion at the bond interface. Among the studies that used adhesion promoters with their adhesives, the most common and most effective was silanes. Adhesion promoters, such as silanes, have been shown to increase the durability of steel-epoxy bonds without affecting the initial bond strength (McKnight et al., 1994). Silanes are hybrids of silica and organic materials related to resins that have been shown to increase the environmental failure resistance of aluminum and steel to epoxy joints (Hutchinson, 1987). Hashim (1999) notes that silane primers can be used to inhibit corrosion and promote adhesion. Allan et al. (1988) reports that silane can also be used on grit blasted aluminum surfaces to considerably increase the durability of the interface. As such, they have been used in field applications such as the strengthening of bridge 1-704, which carries southbound traffic on Interstate 95 in Delaware (Miller et al., 2001).

Silanes adhesion promoters are noted also to greatly reduce the variability of bond performance, while protecting the freshly prepared surface from damage, exposure to
environmental conditions and contamination (Hutchinson, 1987). Application of a 5 percent solution of silane primer applied to grit blasted surfaces results in a water-stable interface. Some adhesives incorporated silane in their formulation, but naval application experience has shown this to be less effective than a separate silane layer (Allan et al, 1988). Gettings and Kinloch (1977) found that durability was improved only when there was evidence of primary bonding between the polysiloxane primer and the steel surface.

2.4.5 Installation and Cure Methods

Proper installation of the CFRP has been fully explained and laid out by Schnerch (2005). From his review and the review of several authors, the installation method most suited for bonding CFRP to steel is to first prepare the substrates that are to be bonded. When environmental conditions allow, the surface of the base metal and CFRP materials should be prepared as described in section 2.1.3. The adhesive should then be thoroughly mixed, applied to the surface of the CFRP strip and clamped within the pot life of the epoxy. The adhesive may be applied to the CFRP surface only, but for highly irregular surfaces it is recommended to apply the epoxy to the CFRP and the steel surface to minimize the formation of air voids within the adhesive layer. The thickness of the bond line can be controlled by the use of a plastic trowel with a ‘V’ shaped notch or be mixing a small amount of glass beads into the adhesive. To prevent sagging of the CFRP plates a temporary clamping system should be installed until the adhesive has cured sufficiently, typically within 24 hours.

The cure time determines when a joint may be subjected to its full design loading. Many adhesives on the market have accelerated and room temperature curing cycles available to industry needs. Although additional heat applied to the adhesive bond between the composite and steel aids in accelerating the curing process, room temperature cure is most common. How fast adhesives cure at room temperature depends upon the ambient
2. BACKGROUND

temperatures the composite system is exposed to. Allen et al. (1982) notes that cure time is halved for each 8°C rise in temperature or doubled for each 8°C reduction in temperature. Below a certain temperature the epoxy may never set. For most epoxies this temperature is just above freezing (0°C).

For field conditions requiring other methods of curing the adhesive, such as in cold weather conditions, several heating methods are available to speed the curing process and create a good bond. The most common techniques include heating blankets and induction heaters. This was demonstrated for rehabilitating two corroded steel girders with carbon fiber pultruded strips by Gillespie Jr., et al. (1996a, 1996b). The techniques included both heating blankets and induction heaters to elevate the temperature in the steel member at discrete locations. The elevated temperatures were monitored by using infra-red equipment. The cure time of the two-part epoxy Ciba-Geigy AV8113 adhesive chosen was 16 hours at room temperature; through the application of the heating techniques, the cure time of the adhesive could be reduced to 20 minutes.

2.4.6 Durability of the Bond

Environmental exposure can inversely affect the performance of the bond in two primary ways. First, galvanic corrosion between the applied CFRP and the steel can occur when the two adherends are in direct contact. Second, the mechanical properties of the adhesive can be weakened under prolonged exposure to moisture at the bond interface.

For galvanic corrosion to occur, two electrically dissimilar materials must be electrically connected in the presence of an electrolyte. This electrolytic solution may be generated by the presence of water with a salt, fertilizer, acid or a combustion product. If these conditions are met, current will flow through the electrolyte from the anodic metal to the cathodic material, causing the anodic metal to deteriorate. This has been observed when CFRP is in direct contact with steel causing accelerated corrosion of the steel, leading to loss of bond
strength. In general, galvanic corrosion is prevented when the flow of corrosion currents is stopped or nonexistent. Of the considerable research that has been focused on the prevention of galvanic corrosion, the most common observation is that a thin layer of adhesive can provide enough of an electrical barrier between the steel and carbon fiber to reduce the rate of galvanic corrosion of the steel. Increasing the thickness of the adhesive layer can further reduce the corrosion rate (Tavakkolizadeh and Saadatmanesh, 2001a). Insulating the two dissimilar materials in the adhesive layer to reduce the likelihood of galvanic corrosion has also been accomplished by coating the steel surface with a water resistant sealant or epoxy (Evans and Rance, 1958) and by installing a glass fiber scrim layer between the carbon material and the steel (Sloan and Talbot, 1992; West, 2001). The introduction of GFRP material, however, may be less durable than the adhesive on its own (Schnerch, 2005).

In addition to galvanic corrosion, accelerated deterioration of the adhesive bond itself can occur under long term exposure to moisture, especially in conjunction with salts resulting from deicing of roadways or ocean spray. Moisture can enter an adhesive bond by diffusion through the adhesive or one of the adherends by capillary action along cracks in the adhesive or by wicking along the interface. Once in the joint, water can cause reversible or permanent damage to the adhesive and may influence the behavior of the joint due to stiffness change of the resin resulting from the exposure (Karbhari and Shulley, 1995). The polar molecules which give an adhesive its adhesive properties also make the adhesive inherently hydrophilic (Hutchinson, 1987 and Hand et al. 1991). Adhesives become plasticized by water absorption, greatly affecting their mechanical properties. Findings from several investigations have generally found that the use of silane primers with thermosetting epoxy adhesives can improve the long term durability of adhesive joints under various environmental conditions (Mertz and Gillespie, 1996; West, 2001).

Thermal effects may also degrade the bond performance of steel to FRP joints either alone or in combination with water (Hollaway and Cadei, 2002). Freezing temperatures may cause
either the adhesive or the CFRP to crack or harden. Combined with freeze-thaw cycling, the stresses induced by this type of thermal loading can result in debonding. Conversely, high temperatures may result in softening of the adhesive to an extent that viscoelastic effects become significant. Hollaway and Cadei (2002) recommend that the composite material and the adhesive have a glass transition temperature of at least 30° C above the maximum design temperature.

2.5 Bond Behavior of CFRP to Steel Under Flexural Loading

Applied loads to steel members under flexure are transferred to adhesively bonded CFRP materials through the adhesive joint by shear. The joint’s ability to transfer loads depends upon the cohesive strength of the adhesive and the degree of adhesion to the bonding surface (Hutchinson, 1987). For infinitely stiff adherends, the shear stress and strain are constant throughout the bonded interface. However, if the adherends have some degree of elasticity, then the stress and strain in the bond line change, with the interfacial shear stresses maximized at the joint ends and minimized at the middle of the joint (Price and Moulds, 1991). Lower bond stresses have been observed when longer bonded CFRP plates have been applied with a small distance between the end of the plate and the support. Smaller adhesive bond lines, increasing the adhesive stiffness, stiffer FRP strips, and thicker FRP strips have resulted in higher bond stresses. Failure occurs when peak interfacial shear stress exceed the shear strength of the adhesive, when peel stresses normal to the bond develop due to eccentricities in the joint and exceed the adhesion strength of any bonding substrate, or when the rupture strength of the applied CFRP material is exceeded.

2.5.1 Failure Modes

There are basically two modes of failure for externally bonded FRP materials to steel surfaces, debonding from the steel adherend or rupture of the FRP material. Debonding
mechanisms are best described by Buyukozturk et al. (2004) when he presented the research related to debonding of FRP materials from steel surfaces. Four possible debonding mechanisms were identified. These mechanisms were adhesive failure at the steel-adhesive interface, cohesive failure within the adhesive layer, cohesive failure at the adhesive-FRP interface and delamination within the FRP material itself.

Adhesive failures at the interface of the adhesive and the steel substrate can occur when the adhesion forces between the two substrates is exceeded. This typically is the result from poor surface preparation of the steel. Improvements to the adhesive strength or surface area will show little or no effect if the failure mechanism is adhesive.

Cohesive failure within the adhesive occurs when the peak interfacial shear stresses of the joint exceed the shear strength of the adhesive. This type of failure mechanism can generally be prevented when the strength of the adhesive is improved. Increases to the surface area can lower bond stresses and also strengthen the bonded joint. Significant improvements are observable when the width of the bonded joint is increased rather than extending the length of the joint along the direction of the applied load.

Cohesive failure at the adhesive-FRP interface typically occurs when critical bond stresses form from material discontinuities associated with cracks. This type of failure can be prevented by proper inspection and cleaning of the materials of any local defects. Defects in the adhesive at the end of a strengthening plate are particularly detrimental. This zone is most highly stressed, and it is also exposed to environmental attack and other forms of damage that can cause peak stresses to grow as an end defect propagates into the length of the bonded plate. The stability of a crack propagating in the end of the adhesive should be assessed, particularly where the strengthening has been tapered to reduce the stress concentration.
As noted by Buyukozturk et al. (2004), once the critical stress in the adhesive is reached, debonding may result from the most energetically favorable crack propagating through the steel/adhesive interface, the CFRP/adhesive interface, the adhesive or within the CFRP material. The potential of debonding failures can be decreased when improvements are made during the surface preparation process or in the compatibility between the adhesive and substrate materials and when the applied strengthening material is extended as close to the supports as possible. Debonding failures are typically unexpected and difficult to predict.

Failure by rupture of the CFRP material best utilizes the applied CFRP composite assuring that sufficient adhesion and adhesive strength was achieved for the strengthening system. This failure occurs when the maximum strain within the CFRP is reached, usually within the maximum moment region of the steel beam unless local stresses arise within the CFRP material due to discontinuities. While favorable for experimental studies, rupture of the CFRP material should never be achieved for in situ rehabilitation and strengthening projects. An allowable live load level can be designed to prevent this mode of failure.

In comparison, failures for FRP-strengthened reinforced concrete members may occur by flexural failures of critical sections, such as FRP rupture and crushing of compressive concrete, or by debonding of the FRP plate from the reinforced concrete beams. The latter type occurs with a loss in the composite action between the bonded FRP and the reinforced concrete member. Like for steel, debonding in FRP strengthened reinforced concrete members occurs in regions of high stress concentrations, which are often associated with material discontinuities and with the presence of cracks. Propagation path of debonding initiated from stress concentrations depends on the elastic and strength properties of the repair and substrate materials as well as their interface fracture properties. For concrete, a majority of the debonding failures has been reported to take place in the concrete substrate. Nevertheless, depending on the geometric and material properties and mechanical and environmental effects to which the interface region is subjected to, other debonding
mechanisms can also occur. Experimental results have shown that FRP debonding is a highly complex phenomenon that can involve failure propagation within the concrete substrate, within the adhesive, within the FRP laminate, and the interfaces of these layers (Büyüköztürk et al, 1999a; Smith and Teng, 2001; Büyüköztürk et al, 2003; Büyüköztürk et al, 2004)

2.5.2 Ways to Improve Bond

Various studies have been performed to optimize the adhesive joint strength of the bonded CFRP material and to reduce the likelihood of failure by debonding. For joints loaded under flexure, it is most beneficial to reduce the shear and peel stress concentrations that are generally highest at the ends of the adhesive joint. Techniques to reduce the stress concentration include detailing of the ends of the adherends and mechanical anchorage of the CFRP composite.

The various detailing of the adherend ends which have been studied include, providing a spew fillet of excess adhesive at the ends of the joint, tapering the edge of the CFRP composite material, increasing the thickness of the adhesive, or reducing the elastic mismatch between two different adherends with perforations. According to Price and Moulds (1991) and Hildebrand (1994), the most effective may be the reverse taper with an adhesive fillet at the end of the composite strip. This agrees with the research review completed by Schnerch (2005) and Dawood (2005). Installation for CFRP laminate strips requires that the shorter face of the tapered end of the CFRP strip is placed on the flange of the steel beam, followed by filling the tapered gap with adhesive during the bonding operation. For repairs consisting of multiple plys of a thinner CFRP material, like sheets, the interlaminar shear and peel stresses can be reduced by reverse tapering of subsequent plys of material. This was validated by Ong and Shen (1992) as each ply was reduced in length so that the longest layer was on the outside and progressively thinner layers were towards the steel flange surface.
2. BACKGROUND

Other researchers have implemented mechanical anchorage of the joint end as a method in assisting in stress transfer and prevention of premature debonding. Sen and Libby (1994) used a combination of bolts and specially designed clamps, Sen et al. (2001) tried bolting and clamping separately, and Liu et al. (2001) recommended using glass fiber wraps around the bonded composite and the flange of the beam. While successful in all applications, it was noted that caution should be exercised when bolting CFRP materials to prevent longitudinal splitting of unidirectional CFRP products.

2.5.3 Development Length

Very little research has been conducted to determine the development length of CFRP sheets and strips bonded to steel. In 2001, Miller et al. determined where force transfer occurs within the bond of CFRP to steel from tension tests performed with steel plates reinforced on each side with CFRP laminate strips. Test results and an analytical model, which was validated by strain gauges on one sample specimen, indicated that 98 percent of the force transfer occurs within 100 mm of the end of the CFRP strips. With similar materials, Lam et al. (2004) concluded that increasing the overall bond length from 100 mm to 300 mm for a series of steel-CFRP double lap shear coupons did not significantly increase the bond strength of the joint. This was due to the presence of significant bond stress concentrations near the end of the CFRP and steel materials. Increasing the bond length did however increase the maximum ductility of the joints.

In 2004, Jiao and Zhao determined the bond length of CFRP sheets bonded to very high strength (VHS) steel tubes. The steel for these tests had yield strength of 1350 MPa and an ultimate strength of 1500 MPa. The experimental study compared two steel tubes that were joined by use of CFRP material alone or a combination of the CFRP material and welding. Tests results showed that the CFRP material could strengthen the butt-welded tubes to
restore the full yield capacity. A development of 75 mm was found sufficient to achieve full strength increase when four plys of the CFRP material was installed.

In 2005, Nozaka et. al. published the first study on the bond length of CFRP strips applied to a flexural member. The experiment studied the repair of cracked steel girders. The test configuration consisted of a CFRP strip bonded to two steel plates that were subsequently bolted to a steel girder, which had a specially fabricated notch in the tension flange and web at midspan. A total of 27 tests were conducted using two different types of CFRP plates and five different adhesives. Under cyclic loading, increases to the ductility of the adhesive ensured high maximum strains achieved in the CFRP strip. The adhesives yielded very rapidly as the bonded CFRP strip was loaded, resulting in failure by debonding for each test. The results further indicated that increasing the number of layers of the CFRP joint increased the tension strength of the bonded joint and reduced the maximum measured strain in the CFRP prior to the debonding. Heat curing was also found to have insignificant effect on the bond. The development length was defined as the shortest length that maximizes the load transferred to the CFRP strip and for the adhesives tested; this development length was found to be less than 203 mm. Increases to the bonded length beyond the effective bond length did not result in any significant increase of the tension strength of the bonded joint.

In 2006, Lenwari et al. studied analysis predictions of development length by testing seven steel beams strengthened with conventional modulus CFRP laminate strips using three different lengths. The different strip lengths considered were 500 mm, 650 mm and 1200 mm. Both of the shorter lengths failed by debonding at the same applied moment, while the 1200mm length failed by rupture of the CFRP strip. Measured strains a distance of approximately 100 mm from the end of the CFRP was required to achieve conformance with the predicted strains obtained from an elastic-plastic section analysis. The measured 100 mm length was also required to conform to the predicted adhesive shear stresses obtained from a stress-based analysis, which uses differential equilibrium and compatibility to predict the
shear stresses in the adhesive layer. For the stress-based analysis, this 100 mm length was found independent of the length of the CFRP.

A similar study was conducted by Deng and Lee (2007a), who tested a series of simply supported steel beams that were strengthened with CFRP laminate strips of different lengths and thicknesses. Test results demonstrated that increasing the length of the bonded CFRP strip closer to a region of lower moment near the support helped to increase the debonding failure load of the CFRP composite system and lower the shear and normal peeling stresses within the adhesive. Increases in the CFRP thickness decreased the debonding failure load, resulting in higher bond stresses at the CFRP strip end. From the test data, the authors established a fracture mechanics based analytical model that could predict the magnitude of the maximum adhesive principal stress experienced near the end of the CFRP strip as a function of the applied moment at the same location. They further proposed that debonding failure of the CFRP strips would occur when the principal stress reached a limiting value.

2.5.4 Analysis of Bond Stresses

Analysis of steel beams strengthened externally with bonded FRP composite materials can be understood by analyzing both the adhesive joint and the applied composite in two procedures. The adhesive shear and normal peeling stresses is determined from differential equations of equilibrium that includes shear lag. Composite materials are analyzed using a moment-curvature analysis, which typically neglects the effect of shear lag between the steel and the FRP strengthening material.

The differential equations of equilibrium that are used to determine the adhesive bond stresses employ a shear lag model approach. In a shear lag model, stresses within the adhesive are transferred between the steel and composite material adherends by shear deformation of the adhesive layer. These stresses are generally assumed constant through the adhesive thickness to simplify the equilibrium equations. Bond stresses can then be
calculated along the entire length of the adhesive joint as a function of the geometric and material properties of the adhesive and the adherends. Compatibility is enforced at the interface between each of the adherends and the adhesive. Boundary conditions are then used to solve the differential equations of equilibrium.

There are fortunately a number of closed form solutions that have been developed and published for different adhesive joint configurations. In externally bonded joints, Albat and Romilly (1999) presented a solution that considers shear lag by the use of a correction factor. This was later verified experimentally by Miller (2000). Several other researchers presented solutions for cover plated beams (Taljsten, 1997; Smith and Teng, 2000; Cadei and Stratford, 2004). Published solutions typically predict very high shear and peel stresses at the termination of the joint that rapidly deteriorate away from the joint end.

For published solutions and within the differential equations of equilibrium, the average shear stress in the adhesive is typically used by dividing the applied force by the area of the adhesive. It is the opinion of Cadei et. al. (2004) that a thorough analysis or design of an adhesive joint is insufficient when it considers only the average shear stress within the joint. Principal stress at joint terminations should be determined from maximum shear and peeling stresses calculated using an appropriate bond model. Therefore, Cadei et. al. (2004) highly recommends the lap-joint models developed by Hart-Smith. His models were investigated thoroughly by Carpenter (1991) by using finite element to study various simplifying assumptions used in lap-joint theories. Carpenter’s study showed that simplifying assumptions did not affect the peak shear stresses and that only certain assumptions presented errors for the maximum normal peeling stresses.

Analysis of the CFRP composite strengthening material using moment-curvature generally assumes a linear strain profile throughout the depth of the section. This assumption requires that there is perfect bond between the steel and the composite adherend and that the adhesive
has sufficient strength to transfer the applied stresses from the steel beam to the bonded strengthening material. Shear lag effects through the adhesive joint are not considered. The moment curvature relationship is determined by using a stepwise increment of the strain at the top surface of the beam and iterating the neutral axis depth of the strengthened cross section from equilibrium. Non-linear characteristics of the steel and concrete materials should be incorporated. The load-deflection behavior of the strengthened steel beam can then be calculated by integration of the curvature after the complete moment curvature relationship of the beam is established.

While most researchers use the moment-curvature analysis, other researchers have used finite element methods to predict the behavior of steel girders strengthened with CFRP materials (Sen and Liby, 1994; Mossallam, 2005). This method has not yet become widespread.

2.5.5 Shear Lag Effect

Shear lag effects on the overall behavior of a strengthened beam have been discussed by several researchers. Shear lag is an essential aspect of determining bond stresses within adhesive joints when using differential equilibrium equations. However, shear lag effects have been neglected when analyzing a section of the strengthened beam using moment-curvature relationships. For this reason, the concept that plane sections remain plane is questioned when steel beams are strengthened with externally bonded composites.

One of the first experiments identifying possible shear lag effects for externally bonded CFRP to steel loaded under flexure was Sen and Liby (1994). Dramatic non-linearity of the strain profile between the tension flange of the steel beam and the bonded composite material was observed in several test beams. For several cases, the strains in the CFRP measured at mid-span and at the quarter-spans of the beams were found substantially lower than the strain measured in the steel tension flange, which are consistent with shear lag assumptions. However, other tests resulted in CFRP strains much higher than strains measured in the
tension flange of the steel or the strains profiles were essentially linear. No trend in the results could be identified and the authors concluded that the clamps and bolts provided at the ends of the bonded composites may have possibly affected the strain at the level of the CFRP.

Ono et. al. (2001) also observed a noticeable shear lag effect for a small scaled steel beam consisting of two 40 mm thick by 100 mm wide steel plates bonded together by a 1mm thick epoxy layer. The beam was loaded in three-point bending with a span of 300 mm and with the joint oriented in the horizontal plane. No noticeable shear lag effects were observed for a second beam tested after a 0.22 mm thick layer of unidirectional carbon fiber sheet was placed between the two plates during the bonding process.

Several other researchers attributed shear lag effects in their test results. Tavakkolizadeh and Sasdatmensh (2003c) proposed shear lag as a cause for why experimental tests resulted in lower stiffness than predicted for the strengthened test beams using moment-curvature analysis. Mosallam (2005) identified slight shear lag through the thickness of the aluminum honeycomb core of a composite panel when test beams were loaded at higher load levels. Nozaka et. al. (2005) questioned shear lag effects through the thickness of bonded CFRP laminate strips located near a crack within the tension flange of the steel beam. Localized variations in stress due to the presence of the crack were also proposed. Shear lag effects within layered wet lay-up CFRP sheets around butt joined hollow tubular steel sections were also reported by Fawzia (2007).

Other studies have shown through experimental correlation with predicted behavior using moment-curvature analysis that shear lag effects were very minimal and inconclusive as a cause (Al-Saidy, 2004 using standard modulus CFRP; Schnerch, 2005 and Dawood 2007 using high modulus CFRP). The slight discontinuities of the strain profiles observed were stated to likely be caused by the effect of residual stresses that formed in the steel beams in
2. BACKGROUND

the manufacturing process or by the effect of localized instability and/or possible lateral movement of the tension flange of the steel beams during testing.

There are some researchers that believe that the assumption of a linear strain profile used in the moment-curvature analysis is a reasonably accurate prediction of the behavior of retrofitted steel members. Oehlers et. al. (1997) suggested this when he compared the interaction between a concrete slab and a steel beam using standard mechanical shear connectors used in typical construction to a chemical bonded interaction between two adherends. The interaction between the concrete slab and the steel beam was demonstrated analytically to reduce the capacity of a section when full interaction was not obtained. This same partial interaction result can also be applied to the interfaces of two adherends, although in chemically bonded interfaces the stiffness of the attachment tends to infinity resulting much closer to a full interaction condition.

Moy and Nikoukar (2002) also determined after linear strain profiles of strengthened test beams were experimentally observed that plane sections remain plane was a valid assumption. This was presented after several tests and models demonstrated that for adhesives with a modulus of elasticity of less than 10 MPa (1450 psi), the effective moment of inertia of the strengthened section and thus the stiffness of the strengthened beam, could be reduced significantly and still have a linear strain profile.
3  EXPERIMENTAL PROGRAM

3.1 INTRODUCTION

The objective of the experimental program is to develop a system to increase the stiffness and strength of steel structures and bridges. While previous work has shown strength increases were possible for steel structures reinforced with conventional CFRP materials, little stiffness increase has been shown. In order to provide stiffness increases to steel structures while minimizing the thickness of the applied CFRP material, a new type of high-modulus carbon fiber is used.

This high modulus carbon fiber material is composed of coal tar pitch based carbon fiber that is manufactured in the form of unidirectional fiber sheets and pultruded laminate strips. The sheets typically have a width of 330 mm and are suitable when a wet lay-up process is necessary to conform to the exact surface configuration of the structure. The laminate strips are expected to be more suitable for field applications where a greater degree of strengthening is required and flat uniform surfaces are available for bonding.

Externally bonding of the new high modulus CFRP material to steel required knowledge and testing of adhesives, surface preparation, and length of CFRP needed to create the most effective bond between the two substrates. This bond characterization testing and development were completed in two test phases.

The first testing phase was the selection of the most effective bonding adhesive and surface preparation needed for bonding the CFRP material to steel. Knowledge of the best surface preparation for bonding the two materials together were determined from a detailed literature review of research and testing completed by several research institutions and universities. Description of this surface preparation was included in Section 2.4 of the thesis.
3. EXPERIMENTAL PROGRAM

Determination of the most effective adhesives for bonding the CFRP material to steel was completed in a testing program where several suitable saturating resins and structural adhesives were tested for the wet lay-up of unidirectional fiber sheets and bonding of the laminate strips respectively. This testing program consisted of 178 double lap shear coupon tests for the wet lay-up of the unidirectional fiber sheets and 21 scaled steel beam tests for the bonding of the laminate strips. Ten different saturating resins and six structural adhesives were initially selected.

The second testing phase was the determination of development length required to fully utilize the strength of the CFRP material. This phase overlapped with the adhesive selection phase for bonded laminate strips and continued for both CFRP applications. Development lengths ranging from 51 mm (2 inches) to 203 mm (8 inches) were tested. The specimens used for this study were the same scaled beam specimens that were used in the adhesive selection. Five resins and adhesives from the first phase were considered.

The overall scheme of test specimens and CFRP used in the experimental program is shown in Table 3-1. Because the focus of the experimental program was to examine the bond of both CFRP applications, the report of the experimental investigation was divided into unidirectional fiber sheet tests and laminate strip tests respectively.

Scaled steel beams were selected as test specimens for the bonded CFRP laminate strip and the development length tests instead of double lap shear coupons, as completed in Phase 1 for the unidirectional fiber sheets because the beams created a better representation of bond stresses for flexural strengthening applications. Curvature of beams under the effect of flexural loading induces normal and shearing stresses at the interface of the adhesive layer and the CFRP laminate strip. These stresses are representative of actual field applications compared to shear and peeling stresses resulting from a double lap shear test.
3. EXPERIMENTAL PROGRAM

Table 3-1 Scheme of the experimental program

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Resin Selection for Unidirectional Fiber Sheets</th>
<th>Adhesive Selection for Laminate Strips</th>
<th>Development Length Study for Sheets and Strips</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Double lap-shear coupons</td>
<td>Scaled steel beams</td>
<td>Scaled steel beams</td>
</tr>
</tbody>
</table>

| Application Method | Unidirectional fiber sheets | CFRP laminate strips | Both application methods |

3.2 MATERIAL PROPERTIES

3.2.1 High Modulus CFRP Materials

The CFRP materials used in this experimental program exhibit a high tensile modulus and a low tensile strain in comparison to other FRP materials manufactured in the market. These tensile properties in theory can provide significant stiffness and strength to steel. A comparison of these tensile properties to other types of FRP materials is shown in Figure 3-1.
Figure 3-1 Comparison of tensile stress and strain properties of several FRP materials to steel

The mechanical properties of the high modulus CFRP materials used in the experimental program are provided in Table 3-2. This type of fiber has traditionally been limited to the aerospace industry due to their exceptional mechanical properties and cost. More recently, industrial grades of these fibers have been developed leading to the identification of non-aerospace applications where their higher modulus is an advantage. The unidirectional fiber sheets were packaged as 50-meter long rolls, with a width of 332 mm (13.0 inches) and an effective thickness of 0.19 mm (0.0075 inches). The laminate strips were fabricated in 35.8 mm (1.41 inches) wide strips with a thickness of 1.42 mm (0.0560 inches). Fiber volume fraction was 55% for all laminate strips.
3. EXPERIMENTAL PROGRAM

Table 3-2 Mechanical properties of unidirectional fiber sheets and CFRP laminate strips

<table>
<thead>
<tr>
<th></th>
<th>Unidirectional Fiber Sheets</th>
<th>CFRP Laminate Strips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Modulus</td>
<td>640 GPa</td>
<td>340 GPa</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>2448 MPa</td>
<td>1190 MPa</td>
</tr>
<tr>
<td>Ultimate Elongation</td>
<td>0.4 %</td>
<td>0.33 %</td>
</tr>
<tr>
<td>Fiber Volume Fraction</td>
<td>-</td>
<td>55 %</td>
</tr>
</tbody>
</table>

Properties of the laminate strips were about 55% lower than the unidirectional fiber sheets due to the effect of the fiber volume fraction. The resin used in manufacturing the laminate strips was EPON Resin 9310. It is based on a bisphenol-A epoxy resin and was manufactured by Resolution Performance Products. The properties of the EPON 9310 resin are given in Table 3-3.

Table 3-3 Properties of EPON Resin 9310

<table>
<thead>
<tr>
<th>Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Modulus</td>
<td>3103 MPa</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>76 MPa</td>
</tr>
<tr>
<td>Ultimate Elongation</td>
<td>4.0 %</td>
</tr>
</tbody>
</table>

3.2.2 Steel Substrates

The steel substrates selected for the experimental program exhibited an elastic-plastic bilinear stress-strain relationship. The first steel substrate used in testing was the steel plates used for the double lap-shear coupon tests in Phase 1 of the experimental program. Although no tests were performed to prove this relationship, the steel was known to be a grade A36 mild steel, which commonly exhibits this behavior.
The second steel substrate used for testing were the scaled beams for bonded laminate strips in Phase 1, and the development length tests in Phase 2 of the experimental program. Coupons taken from the flanges and web sections of several beams were tested according to ASTM A370 to prove this behavior. Steel plates that were used for these beam tests were also tested. Figures 3-2, 3-3 and 3-4 show the tensile stress and strain curves for the flange, web, and steel plate coupon tests respectively. The averaged elastic modulus and the yield strength for all coupons are shown in Table 3-4. The 0.2% offset method was used to determine the yield strength of each steel coupon.

Figure 3-2 Tensile stress and strain for steel coupons taken from the flanges of the test beams
3. EXPERIMENTAL PROGRAM

Figure 3-3 Tensile stress and strain for steel coupons taken from the webs of the test beams

Figure 3-4 Tensile stress and strain for coupons taken from the steel plates
Table 3-4 Properties of steel substrate

<table>
<thead>
<tr>
<th></th>
<th>Flange</th>
<th>Web</th>
<th>Plate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Modulus</td>
<td>216 GPa</td>
<td>213 GPa</td>
<td>214 GPa</td>
</tr>
<tr>
<td>Minimum Yield Strength</td>
<td>564 MPa</td>
<td>585 MPa</td>
<td>346 MPa</td>
</tr>
<tr>
<td>Elongation at Yield</td>
<td>0.261 %</td>
<td>0.275 %</td>
<td>0.162 %</td>
</tr>
</tbody>
</table>

3.2.3 Adhesives and Resins

According to the research completed by the University of Delaware (Bourban et al. 1994, McKnight et al. 1994) and Nippon Steel Corporation in Chiba, Japan (Nakazawa 1994), epoxy resins have been found to provide effective bond of the CFRP to steel surfaces. Reports indicated also that they have even helped prevent galvanic corrosion between carbon and steel (Tavakkolizadeh and Saadatmanesh 2001). Also, amine-epoxy adhesives are known for their high strength and stiffness properties, which aid the transfer of forces from the steel to the high modulus CFRP. Therefore, the resins and adhesives used to bond the steel and the CFRP material were mostly amine-epoxy. Other resins, such as acrylics, urethanes, and the blend combination of an acrylic-epoxy, were selected as comparisons in the testing program. The tensile properties of each resin and adhesive are shown in Tables 3-5 and 3-6 respectively. For the prepreg process, three different prepreg resins developed by Reichold, Inc. were each evaluated with three different bonding resins, resulted in nine different trials. Those resins included a thickenable terathalic, a urethane modified vinylester, and a high temperature sheet molding compound.
### Table 3-5 Room Temperature tensile properties of saturant resins used for bonding unidirectional carbon fiber sheets to steel

<table>
<thead>
<tr>
<th>Resin</th>
<th>Designation</th>
<th>Resin Type</th>
<th>Tensile Modulus (MPa)</th>
<th>Tensile Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3M DP460</td>
<td>DP460</td>
<td>Amine-epoxy</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>3M DP810</td>
<td>DP810</td>
<td>Acrylic</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Degussa MBrace Saturant</td>
<td>MSAT</td>
<td>Amine-epoxy</td>
<td>3034</td>
<td>3.50</td>
</tr>
<tr>
<td>Jeffco 121</td>
<td>J121</td>
<td>Amine-epoxy</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Reichhold Atprime 2</td>
<td>ATP2</td>
<td>Urethane</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Resinlab EP1246</td>
<td>EP1246</td>
<td>Acrylic-epoxy</td>
<td>2256</td>
<td>2.75</td>
</tr>
<tr>
<td>Sika Sikadur 300</td>
<td>S300</td>
<td>Amine-epoxy</td>
<td>1724</td>
<td>3.00</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td>S330</td>
<td>Amine-epoxy</td>
<td>2000</td>
<td>1.50</td>
</tr>
<tr>
<td>SP Systems Ampreg 22</td>
<td>AM22F</td>
<td>Amine-epoxy</td>
<td>3780</td>
<td>3.00</td>
</tr>
<tr>
<td>(fast hardener)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SP Systems Ampreg 22</td>
<td>AM22S</td>
<td>Amine-epoxy</td>
<td>3890</td>
<td>3.40</td>
</tr>
<tr>
<td>(slow hardener)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prepreg 1A</td>
<td>R.1A</td>
<td>Thickenable terathalic</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Prepreg 2A</td>
<td>R.2A</td>
<td>Urethane modified vinylster</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Prepreg 5A</td>
<td>R.5A</td>
<td>High temperature sheet molding compound</td>
<td>*</td>
<td>*</td>
</tr>
</tbody>
</table>

(i) * Product information not available
Table 3-6 Room Temperature tensile properties of adhesives used for bonding laminate strips to steel

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>Designation</th>
<th>Adhesive Type</th>
<th>Tensile Modulus (MPa)</th>
<th>Tensile Elongation(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fyfe Tyfo MB2</td>
<td>FMB2</td>
<td>Amine-epoxy</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Jeffco 121</td>
<td>J121</td>
<td>Amine-epoxy</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Sika Sikadur 30</td>
<td>S30</td>
<td>Amine-epoxy</td>
<td>4480</td>
<td>1.00</td>
</tr>
<tr>
<td>SP Systems Spabond 345</td>
<td>SP345</td>
<td>Amine-epoxy</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Vantico Araldite 2015</td>
<td>VA2015</td>
<td>Amine-epoxy</td>
<td>2000</td>
<td>3.00</td>
</tr>
<tr>
<td>Weld-On SS620</td>
<td>SS620</td>
<td>Acrylic</td>
<td>620</td>
<td>120.00</td>
</tr>
</tbody>
</table>

* Product information not available

3.3 PHASE 1(a) RESIN SELECTION FOR WET LAY-UP OF CFRP SHEETS

3.3.1 Test Specimens

The test specimens used for selecting the most effective resin for bonding unidirectional carbon fiber sheets to steel were double lap shear coupons. The coupons consisted of two 25.4 mm (1.0 inch) wide A36 steel plates joined together with five tows of the unidirectional carbon fiber sheet on both sides. The fiber sheets overlapped the steel plates 25.4 mm (1.0 inch) on each plate. A 1.9 mm (0.075 inch) gap separated the two steel plates to minimize the effect of end to end bonding. Grip areas were marked 31.8 mm (1.25 inch) from each end of the steel plates to designate the clamped area for the hydraulic grips of the test machine. The geometry and dimensions of the test specimen are shown in Figure 3-6. The geometry of the specimens was determined based on ASTM D 3528-96 specification for testing materials in double lap shear.
3. EXPERIMENTAL PROGRAM

3.3.2 Test Program

The test program for evaluation of the wet lay-up specimens included several different parameters as indicated in Table 3-7. Trials using ten different resins were cured at room temperature for one week. Seven of the resins used were amine-epoxy resins, 3M DP-810 was an acrylic resin, Resinlab EP1246 was an acrylic-epoxy blended resin and Reichhold ATP2 was a urethane resin. Certain resins from these room temperature cured series were also tested several times using additional variables to observe their possible improvements as bonding agents. These variables included different cure temperatures, use of a wetting agent and resin hybridization. Two resins were cured at an elevated temperature of 50°C for sixteen hours and then removed from the oven to cure at room temperature for seven days. Three resins were also evaluated with a wetting agent provided by BYK Chemie Inc., BYK-A560 to improve the saturation of the fibers. Two different doses of the wetting agent were
used, a low dose that was 0.5 percent of the total resin weight and a high dose that was 1.0 percent of the total resin weight. Three other resins were bonded as a hybrid application; one was used to saturate the carbon fibers, while the other was used to bond the saturated fibers to the steel. Typically, six replicates were used for each trial. More detail about the behavior of the additional variables is discussed in the test results section. A total of 156 double lap shear coupons were tested.

For the prepreg process, two resins were used for each test specimen. The prepreg resin was used to saturate the fibers and remain partially cured during the bonding process. Reichhold, Inc. developed all of the prepreg resins used in the experimental program. A second, bonding resin was used to bond the prepreg to the surface of the steel. Once the prepreg was bonded, the specimens were placed into an oven to cure. Specimens using the thickenable terathalic prepreg resin or the urethane modified vinylester prepreg resin were cured at 80 °C for 10 minutes. The remaining specimens using the high temperature sheet-molding compound were cured at 135 °C for 10 minutes. The test program for the prepreg specimens together with their bonding resins is given in Table 3-8.
3. EXPERIMENTAL PROGRAM

Table 3-7 Test program of resin selection for wet lay-up of unidirectional carbon fiber sheets

<table>
<thead>
<tr>
<th>Resin</th>
<th>Cure Temperatures</th>
<th>Additional Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>3M – DP 460</td>
<td>RM</td>
<td></td>
</tr>
<tr>
<td>3M – DP 810</td>
<td>RM</td>
<td>✓</td>
</tr>
<tr>
<td>Degussa MBrace Saturant</td>
<td>RM</td>
<td>with Degussa MBrace Primer</td>
</tr>
<tr>
<td>Jeffco 121</td>
<td>RM</td>
<td>with Sika Sikadur 300, 330</td>
</tr>
<tr>
<td>Reichhold Atprime 2</td>
<td>RM</td>
<td></td>
</tr>
<tr>
<td>Resinlab EP1246</td>
<td>RM</td>
<td></td>
</tr>
<tr>
<td>Sika Sikadur 300</td>
<td>RM, 50°C</td>
<td>with Jeffco 121, Sika Sikadur 330</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td>RM</td>
<td>with Jeffco 121, Sika Sikadur 300</td>
</tr>
<tr>
<td>SP Systems Ampreg 22 (fast hardener)</td>
<td>RM</td>
<td></td>
</tr>
<tr>
<td>SP Systems Ampreg 22 (slow hardener)</td>
<td>RM, 50°C</td>
<td></td>
</tr>
</tbody>
</table>

*RM – Room Temperature Curing

Table 3-8 Prepreg resin selection test program

<table>
<thead>
<tr>
<th>Prepreg resin</th>
<th>Bonding resin</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickenable terathalic resin</td>
<td>Atprime 2</td>
</tr>
<tr>
<td>Urethane modified vinylester resin</td>
<td>Atprime 2</td>
</tr>
<tr>
<td>High temperature sheet molding compound</td>
<td>Atprime 2</td>
</tr>
</tbody>
</table>

3.3.3 Test Procedure and Instrumentation

The double lap shear coupons were prepared and fabricated for each resin series in Phase 1(a). The unidirectional fiber sheets were cut into 52.7 mm (2.1 inch) lengths and 5
tows of the sheet in width, or approximately 25.4 mm (1.0 inch). The bonded surface on each steel plate was 25.4 mm (1.0 inch) for both sides. The steel surface was then prepared by sanding the bonding area with 80-grit sandpaper to remove all surface mill scale. Sykes (1982) suggested that for the preparation of small areas, this sanding with emery paper removes surface films and provides good mechanical keying suitable for adhesive bonding to steel. Immediately prior to resin application, the steel was cleaned with acetone. The steel plates were then clamped in a fixture to maintain their alignment during the strengthening process.

For the strengthening process, a uniform coating of resin was applied to one side of the steel plates in the bonded region. Five tows of the unidirectional, high modulus carbon fiber were applied across the steel plates, with 25.4 mm of overlap on each of the steel plates. The sheets were pressed into the resin on the steel plates, and additional resin was placed as an overcoat on the sheets. A 1.9 mm gap was left between the steel plates to minimize the effect of end-to-end bonding as shown in Figure 3-6. Once the resin had achieved sufficient working strength, the same strengthening procedure was completed on the reverse side as shown in Figure 3-7.
3. EXPERIMENTAL PROGRAM

Figure 3-7. The test specimens were then marked 31.8 mm from their ends to indicate the area to be clamped within the hydraulic grips of the test machine. Specimens were cured at least seven days prior to testing.

A similar procedure was followed for the specimens fabricated using the prepreg process. For these specimens, a prepreg was first made using a prepreg resin and the carbon fiber. Once the prepreg had partially cured, it was cut to the same dimensions as in the resin selection phase and bonded to the surface of the steel using an adhesive. The test specimens using the prepreg required heat curing in order to set the prepreg resin.

The wooden fixtures fabricated to align and clamp the steel plates during the application and curing processes were constructed by bonding three 1 mm thick carbon fiber strips onto a wooden block, forming three panels on each block. Before application of the resin, a thin plastic sheet, 0.08 mm in thickness, was placed into the panels to prevent the resin from bonding to the wood. The ends of the steel plates were clamped down after the plates were aligned and positioned for fabrication. Masking tape was also used to control the resin from overflowing onto the steel plates and to control the ends of the bond.

The coupons were loaded in tension at the displacement rate of 0.12 mm/min in a universal closed-loop testing machine with hydraulic grips, as shown in Figure 3-8. The specimens were instrumented with strain gauge type displacement transducers to monitor the longitudinal strain across
the bonded region. Two displacement transducers were positioned on each side of the specimen over the bonded region; each had a gauge length of 100 mm. The ultimate shear strength and strain at peak stress was measured for each test.

3.4 PILOT TESTS SPECIMEN DEVELOPMENT

3.4.1 Introduction

Several pilot tests were evaluated before coming up with a test specimen for determining adhesives for bonded laminate strips in Phase 1 and development length of CFRP sheets and laminate strips in Phase 2. These trial specimens were cold-formed grade A572 square Hollow Structural Section (HSS) beams and hot-rolled grade A36 W4 x 13 wide flanged beams. The cold-formed HSS beams had steel properties that matched the steel properties of three monopoles that were tested for the third phase of the experimental program, and exhibited a nonlinear stress strain relationship. The hot-rolled wide flanged beams were made of mild steel and exhibited a bilinear stress strain relationship. The first specimens that will be discussed are the square HSS beams.

3.4.2 HSS Test Specimens

The first trial specimens that were tested were six HSS beams 63.5 mm wide by 63.5 mm in height by 914 mm in length (2.5 x 2.5 x 36 inches). Two different wall thicknesses were considered, a 6.4 mm (1/4 inches) thickness and a 4.8 mm (3/16 inches) thickness. A drawing of the HSS cross sections is shown in Figure 3-9.
3. EXPERIMENTAL PROGRAM

3.4.3 HSS Test Program

A series of three beam tests were completed for each wall thickness; one beam was strengthened with a 38.0 mm (1.5 inch) wide by 508 mm (20 inches) long bonded CFRP laminate strip, another was strengthened with two layers of 45.0 mm (1.8 inch) wide by 508 mm (20 inch) long unidirectional fiber sheets, and the third was unstrengthened for use as a control specimen. The beam strengthened with the bonded carbon fiber sheet used the saturant resin of Degussa MBrace Saturant. The beam strengthened with the laminate strip was bonded with the Jeffco 121 adhesive.

3.4.4 Test Procedure and Instrumentation

The best preparation for steel was found to be by sandblasting the steel surface and then cleaning it with a solvent, such as acetone. Thus, all HSS beams were sandblasted first, and then cleaned with acetone. The CFRP laminates were then prepared by lightly roughing the bonding surface with 120 grit sandpaper, rinsing in water, and then cleaning with methanol. Bonding of the fiber sheets required no surface preparation.
To minimize oxidation of the steel, bonding of the CFRP material occurred immediately after surface preparation. For the bonding of the laminate strips, a thin layer of adhesive was first spread out onto the steel surface by tapering the adhesive from the middle of the bond area so that when the cleaned CFRP strip was applied to the adhesive layer, air voids could be pushed away from the center of the bond to the exterior of the laminate strip. Pressure was applied to the laminate strip by a roller until a uniform thickness of adhesive was determined. The bonding process for the CFRP sheets followed the procedure of the resin selection, where a thin layer of resin was first applied to the steel surface, followed by application of the CFRP sheet, and then an additional thin resin layer to the top surface of the CFRP sheet to allow for complete saturation. All strengthened specimens were allowed to cure at room temperature seven days before testing.

The ideal test setup for the beams in terms of ease of setup and analysis was to load the beams in flexure simply supported, on top of roller supports placed at each end. A combination of three and four point loading was attempted in these tests to determine the preferred loading for the remaining tests in the experimental program. The 6.4 mm (1/4 inch) HSS beams were loaded in four point flexural loading, while the 4.8 mm (3/16 inch) HSS beams were loaded in three point flexural loading.
Instrumentation for the HSS beams consisted of linear voltage and strain gauge type displacement transducers, electric resistance foil strain gauges, and load cells that were used to measure displacements, strain, and load respectively for each beam. Three locations were selected to measure displacement using the linear voltage displacement transducers; these locations were at mid-span and at the supports. Three locations were also selected for the strain gauge type displacement transducers. All gauges were placed strategically at mid-span of the beam; two were placed on the front and back side of the HSS beam, while the third was placed on the bottom tension side, as shown in Figure 3-10. They were used to measure strain over a given gauge length and were positioned in such a way so that a strain profile could be determined at mid-span. The strain gauge type displacement transducers measured displacement over a given gauge length, thus strain was converted by dividing the measured displacement by the gauge length of the instrument, which was 100 mm (4 inches). These same strain gauge type displacement transducers were useful for the double lap-shear coupons tested in the resin selection phase, and thus were thought to be useful for the beam specimens as well. It was later determined that these transducers were not accurate in their measurements due to twisting of the gauges caused by the curvature the beams underwent during loading.

The electric resistance foil strain gauges measured strain at mid-span. Placement of these gauges were on the top compression and bottom tension sides of the 6.4 mm (1/4 inch) thick
HSS beams and 12.5 mm (1/2 inch) from the top on one side and on the bottom tension side for the 4.8 mm (3/16 inch) thick HSS specimens. The strain measurements from these gauges were reliable.

The final instrument used in the pilot tests was load cells that were placed under each beam support to capture the entire load on the beam. Each load cell was calibrated and had a higher resolution than that of the test machine. A higher resolution was not necessarily needed for testing purposes, but the resolution was helpful in determining any significant load changes if placement of the loading block(s) were not spaced evenly between the supports. Several different configurations were tried before a final position was determined.

3.4.5 Wide Flanged Beam Test Specimens

W4 x 13 test specimens were next selected as a comparison to the cold-formed HSS specimens. Unlike the HSS beams, these beams were hot rolled and thus exhibited a bi-linear stress strain relationship as compared to the nonlinear stress strain relationship of the HSS beams. One beam was discovered and used for three separate tests by cutting the beam into two 914 mm (36 inch) long sections.

3.4.6 Wide Flanged Beam Test Program

Three tests comprised the test program for the W4 x 13 beams. Like the HSS beam tests, one unstrengthened control test was desired as a comparison for each bonded CFRP strengthening system. To alleviate the challenge of testing three tests on two W4 x 13 specimens, one unstrengthened specimen was loaded up to 60% of its yield strength and unloaded. That same beam was later strengthened with one of the strengthening methods. Comparisons were then made to the control test by using a model to extrapolate the test date of the unstrengthened beam.
3. EXPERIMENTAL PROGRAM

3.4.7 Test Procedure and Instrumentation

The same procedures for bonding and preparing the steel surface and the CFRP material used in the HSS pilot tests were also adopted for these specimens. One of the beam sections had a 76.0 mm (3.0 inch) wide by 508 mm (20 inch) long section of the CFRP laminate strip bonded to the tension flange using the available adhesive of Sika Sikadur 30. The other had a 83.0 mm (3.3 inch) wide by 508 mm (20 inch) long section of the CFRP sheet bonded to its tension flange using Sika Sikadur 330.

Both beams were placed in the same test setup that was used for the HSS beams. Displacement was measured at mid-span and at the roller supports with three linear voltage displacement transducers, and strain was measured at mid-span using two electric resistance foil strain gauges. The locations of the strain gauges were at the inside of the compression and tension flanges. Strain gauge type displacement transducers were not used for these tests due to their inaccurate measurements discovered from the HSS pilot tests.

A photo of the test setup with instrumentation for the W4 x 13 beam tests is shown in Figure 3-11. The two load cells, placed under the roller supports and a steel block, rested on a flat surface that prevented movement during loading.

Figure 3-11 Test setup and instrumentation for W4 x 13 pilot tests
3. EXPERIMENTAL PROGRAM

3.4.8 Summary

Testing of the pilot beams resulted in the full development of the test setup. Four-point loading was chosen as the preferred loading method because it provided a region of zero shear and maximum strain that could easily be measured. Foil type strain gauges were accurate and preferred over strain gauge type displacement transducers. Test setup was easy and fast to set up.

Test results from these pilot tests revealed that the hot rolled W4 x 13 sections exhibited more post yield strengthening than the cold formed HSS beams. Because of the results, a decision was made to use hot rolled steel sections for the rest of the experimental program. Further consideration of the pilot test also showed that behavior of the hot rolled steel sections tested could further be improved if the sections simulated steel girders in the field which acted compositely with a concrete slab or deck. Investigation from the literature review showed that one of the major applications for this high modulus CFRP material was for girders that already were compositely cast with concrete, such as the many steel bridge girders that were in need of strengthening and rehabilitation.

The search and design for a scaled model of common steel-concrete composite girders was then underway. After determining a test program and costs, the final decision came to using scaled wide flanged steel beams called Super Light Beams (SLB). The beams were perfectly scaled models of the girders commonly used in bridges, and because of its size, they were easy to fabricate the composite concrete action by welding a small steel plate to its compression flange. Few trial tests were conducted to approve its use as the test beam of choice. Further details of the beam are described in the next section.
3. EXPERIMENTAL PROGRAM

3.5 PHASE 2(a) DEVELOPMENT LENGTH OF CFRP SHEETS

3.5.1 Test Specimens

The test specimens used for the development length of bonded unidirectional carbon fiber sheets simulated the strain profile of a wide flange steel beam that acts compositely with a concrete deck. Super Light Beams, SLB100 x 4.8, were used, as shown in Figure 3-12, with an additional 6.4 mm (1/4 inch) thick steel plate that was stitch welded to the compression flange along its length to simulate the concrete deck and consequently shift the neutral axis towards the compression flange approximately 10 mm higher than center of the steel beam alone. The weld used was an E70 4.8 mm (3/16 inch) fillet weld. It was stitch welded to reduce the potential of residual stresses forming in the flange during the welding process. Configurations of the weld along the entire length of the beam are shown in Figure 3-13.

After the compression flange was strengthened with the welded steel plate, the tension flange was strengthened with the unidirectional carbon fiber sheet that nearly covered the entire bottom of the tension flange. To ensure consistency among the specimens tested, an 11-tow width was used for all tests using these CFRP sheets.

3.5.2 Test Program

The test program consisted of ten tests that were designed to determine the development length for the unidirectional fiber sheets and to study the relationship of CFRP thickness to the development length. Four of the ten tests were beams strengthened with one ply of the unidirectional carbon fiber sheet; the other six tests were beams strengthened with two plies of the carbon fiber sheet. The relationship of the thickness to the development length was assumed to be a linear relationship.

Two resins were selected in this study based on the results from the resin selection; the resins were Degussa MBrace Saturant and Sika Sikadur 330. The development lengths for the one
3. EXPERIMENTAL PROGRAM

and two-ply strengthening tests are shown in Table 3-9. Tests were duplicated for the one-ply tests, while one test was completed for each development length selected in the two-ply test program.

Figure 3-12 Test specimen cross-section for development length study of bonded CFRP sheets
3. EXPERIMENTAL PROGRAM

Top View of Compression Flange

Compression flange of specimen

A36 Steel plate

E70 4.8 mm fillet weld

165 mm

241 mm

305 mm

375 mm

432 mm

Figure 3-13 Weld configurations along compression flange of SLB specimen

Table 3-9 Development lengths tested for one and two plies of the carbon fiber sheets

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>One ply</th>
<th>Two plies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degussa MBrace Saturant</td>
<td>51 mm</td>
<td>51 mm</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td>51 mm</td>
<td>51 mm</td>
</tr>
</tbody>
</table>
3.5.3 Test Procedure and Instrumentation

The steel surface was prepared for strengthening by sandblasting, followed by cleaning with acetone. The unidirectional sheets do not require surface preparation before bonding. They were cut to an eleven-tow width of 53.7 mm (2.1 inches), which almost covered the entire face of the tension flange. The strengthening process immediately followed the surface preparation to minimize possible oxidation of the steel surface. The bonding process consisted of preparing the resin according to the specifications of the manufacturer, applying a thin layer to the steel surface, and aligning the fiber sheet onto the resin layer. An additional layer of resin was applied to the top of the sheets for complete saturation of the fibers, as shown in Figure 3-14. For ease and repetition, the entire bonding process was done with the SLB tension flange facing up. Testing of the strengthened beams occurred after the resin cured at room temperature for at least seven days.

The beams were simply supported and loaded under four-point loading using a spherically seated bearing block, as shown in Figures 3-15 and 3-16. A lateral bracing system was
provided by supporting the top flange of the beam over the supports with two angles that were fixed to each support. Section 3.6 describes the lateral supports in more detail. Load was applied at a constant displacement rate of 0.75 mm/min.

Displacement and strain were measured at the mid-span of the beam. Displacement was measured using two linear voltage displacement transducers, as shown in Figure 3-16. Strain was measured using electric resistance foil strain gauges bonded on the bottom surface of the compression flange, top surface of the tension flange, and outer surface of the tension flange for the unstrengthened specimen and outer surface of the CFRP sheet for the strengthened beams. The strain gauge locations are shown in Figure 3-17.
Some of the specimens had additional strain gauges bonded to the carbon fiber surface along one half of the length from mid-span to determine the strain profile of the carbon fiber sheet during loading. The gauges were bonded as close together as possible; about 25 mm (1 inch) apart, near the end of the fiber sheet until the mid-span of the test specimen to capture the strain where the bond stresses were highest. Typical instrumentation of the strain gauges for the strain profile is shown in Figure 3-17.

![Figure 3-16 Test setup and instrumentation for development length tests](image)

![Figure 3-17 Strain gauge locations along the carbon fiber sheet](image)
3.6 PHASE 1(B) ADHESIVE SELECTION FOR BONDED LAMINATE STRIPS

3.6.1 Test Specimens

The same SLB 100 x 4.8 specimens, used for the development length study for bonding the unidirectional carbon fiber sheets, were used in the adhesive selection for bonded high modulus laminate strips. Similar to the unidirectional carbon fiber sheets, CFRP laminate strips were bonded to the tension flange of the test specimen. The width and thickness of each laminate strip was 35.8 mm (1.4 inches) and 1.42 mm (0.056 inches) respectively.

3.6.2 Test Program

Six adhesives were evaluated to determine the most suitable properties for bonding the laminate strip to steel. Development lengths varied from 51 mm (2 inches) to 203 mm (8 inches) in this investigation. The purpose of changing the development lengths was to study development length as well as to disaffiliate the adhesive performance. The change in the bond length of the strip resulted in higher average bond stresses making the adhesive performance more critical. Table 3-10 shows the test program used for the adhesive selection. All adhesives were first tested at a development length of 203 mm (8 inches); additional development lengths were tested until all adhesives failed in debonding of the CFRP laminate strip. The adhesives evaluated for this study were: Fyfe Tyfo MB2, Jeffco 121, Sika Sikadur 30, SP Spabond 345, Vantico Araldite 2015, and Weld-On SS620.
Table 3-10 Test program of adhesive selection for laminate strips

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>Development Lengths (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>203 mm</td>
</tr>
<tr>
<td>Sika Sikadur 30</td>
<td>✔</td>
</tr>
<tr>
<td>Fyfe Tyfo MB2</td>
<td>✔</td>
</tr>
<tr>
<td>Jeffco 121</td>
<td>✔</td>
</tr>
<tr>
<td>Vantico Araldite 2015</td>
<td>✔</td>
</tr>
<tr>
<td>SP Systems Spabond 345</td>
<td>✔</td>
</tr>
<tr>
<td>Weld-On SS620</td>
<td>✔</td>
</tr>
</tbody>
</table>

The adhesives with the highest strength and stiffness in comparison to the unstrengthened control specimen and the maximum ultimate CFRP strain causing failure of the CFRP laminate strips by rupture were repeated to determine the accurate development length.

3.6.3 Test Procedure and Instrumentation

The steel surface was prepared for strengthening by sandblasting, followed by cleaning with acetone. The surface of the laminate strips were prepared by first roughening the surface with 120-grit sandpaper, then rinsing it in water, and finally cleaning it with methanol. To minimize possible oxidation of the steel surface, the strengthening process immediately followed the surface preparation. Test specimens were strengthened with the tension flange facing up for ease in the bonding process and for quick repetition, since several beams were strengthened at the same time. Bonding the CFRP laminate strips to the tension flange of the test specimen consisted of preparing the adhesive according to the manufacturer’s specifications, applying a thin layer to the steel surface, and then applying the cleaned strip to the adhesive layer, as shown in Figure 3-18. The adhesive layer was tapered from the middle of the bond area so that air voids could be pushed away from the center of the bond to the exterior of the laminate strip, and also to make sure enough adhesive was available for the entire bonded area. The cleaned strip was bonded by applying pressure with a roller until a
uniform adhesive thickness was achieved. Thickness of the adhesive was determined from measurement of the steel tension flange, adhesive, and CFRP strip and subtracting the flange and strip thickness. Testing of the strengthened beams occurred after the adhesive cured at room temperature for at least seven days.

The test setup of the beams was similar to the ones tested in Phase 2(a). The beams were simply supported and loaded under four-point loading using a spherically seated bearing block. Lateral bracing was provided by fixed steel angles supporting the top flange of the beam over the supports. Details of the supports and lateral bracing are shown in Figures 3-19 and 3-20. Load was applied at the constant displacement rate of 0.75 mm/min.

Figure 3-18 CFRP laminate strip application
3. EXPERIMENTAL PROGRAM

Figure 3-19 Cross-section of lateral supports

Figure 3-20 Lateral bracing at the supports
3. EXPERIMENTAL PROGRAM

3.7 PHASE 2(b) DEVELOPMENT LENGTH OF CFRP STRIPS

3.7.1 Test Specimens

The test specimens used in Phase 1(b) were used to determine the development length of bonded CFRP laminate strips.

3.7.2 Test Program

The test program consisted of nine tests that were designed to confirm the results from the adhesive selection for the precured laminate strips, and to also study the relationship of CFRP thickness to the development length. Three adhesives were selected in this study based on the results from the adhesive selection. Those adhesives were Jeffco 121, SP Systems Spabond 345, and Weld-On SS620. Again, one and two plies of the high modulus CFRP were bonded to the tension flange of the test specimens. The development lengths tested for each series of tests are shown in Table 3-11. The results from the beams strengthened with one ply of the laminate strips were compared to the tests done in the adhesive selection.

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>One ply</th>
<th>Two plies</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP Systems Spabond 345</td>
<td>76 mm</td>
<td>102 mm</td>
</tr>
<tr>
<td>Weld-On SS620</td>
<td>76 mm</td>
<td>102 mm</td>
</tr>
<tr>
<td>Jeffco 121</td>
<td>102 mm</td>
<td>127 mm</td>
</tr>
</tbody>
</table>

Table 3-11 Development lengths tested for one and two plies of CFRP laminate strips
3. EXPERIMENTAL PROGRAM

3.7.3 Test Procedure and Instrumentation

Preparation for the specimens followed the procedure used for the adhesive selection in Phase 1(b). Sandblasting and acetone cleaning prepared the steel surface, while the surface of the laminate strips were roughened with 120 grit sandpaper, rinsed in water, and cleaned with methanol. Again, the strengthening process immediately followed the surface preparation to ensure minimal oxidation of the steel.

Bonding the high modulus CFRP laminate strips to the tension flange of the SLB was identical to what was done in the adhesive selection phase. The specimens were cured at room temperature for at least seven days before testing.

The setup for the beams was exactly the same as the setup for the adhesive selection and development length tests for bonded CFRP sheets. The beams were simply supported and laterally braced at the end supports. A displacement rate of 0.75 mm/min was again used as the loading rate. Strain and displacement were measured at mid-span using electric resistance foil strain gauges and two linear voltage displacement transducers. The only addition to the instrumentation for these tests was that additional strain gauges were bonded to the CFRP surface along one half of the length from mid-span to determine the strain profile of the CFRP laminate strip during loading. Strain gauge locations along the CFRP strip for these tests were similar to the placements done in the development length study for bonded carbon fiber sheets.
4 EXPERIMENTAL RESULTS

The main objective of the experimental program is to select the most effective resins and adhesives for high modulus CFRP sheets and laminate respectively. Based on the selected resin and adhesive, the development length required to develop the full strength of the CFRP sheets and laminate were determined. Based on these findings, the proposed high modulus CFRP sheets and laminates were used for strengthening large scaled steel monopole structures by Lanier (2005) and for steel-concrete composite beams typical of bridge structures by Schnerch and Dawood (2005, 2007). The program consisted of two phases. The first phase included selecting the resins and adhesives for the high modulus CFRP sheets and laminate strips proposed for strengthening steel structures. Selection of the saturating resins for wet lay-up of CFRP sheets by a prepreg process were based on the behavior of double lap shear coupons. Ten different resins for the wet lay-up process and three different prepreg resins for the prepreg process were evaluated. Adhesives for bonding pultruded CFRP laminate strips were selected based on their ability to fully utilize the CFRP material at short development lengths using small flexural specimens. Six different adhesives were evaluated based on using variable development lengths. In the second phase, the development lengths of selected resins for one and two plys of CFRP sheets and selected adhesives for one and two plys of CFRP laminate strips were evaluated. Bond behavior was also evaluated for both types of CFRP materials during this phase.

4.1 PHASE 1(A) RESIN SELECTION FOR WET LAY-UP OF CFRP SHEETS

4.1.1 Introduction

Resin selection for the wet lay-up process was determined through testing a series of double lap shear coupons using ten different resins. These resins were selected based on their
properties in bonding unidirectional carbon fiber sheets to steel. Of these selected ten resins, seven were amine-epoxy resins, 3M DP810 was an acrylic resin, Resinlab EP1246 was an acrylic-epoxy blend and Reichhold Atprime2 was a urethane resin. After testing these ten resins cured at room temperature, additional variables were introduced to determine possible bond strength improvements. The variables studied included use of a wetting agent, hybrid combinations of resins, and different curing temperatures, resulting in a total of thirteen different trials.

Three different resins were also studied for the prepreg process whereby saturation of the CFRP sheets was completed before bonding to the steel. During application, the prepreg resin remained in a partially cured state and a bonding resin was used to apply the prepreg sheets to the steel. The double lap shear coupons were then cured in an oven for a prescribed amount of time and at a certain temperature. Nine different trials were tested.

Shear stress strain curves were determined for all double lap-shear tension tests. The resins that achieved high shear strength, high strain at peak stress, and good bond between the steel and the bonded carbon fiber sheets were characterized to be the best performing resins in this selection study. A summary of the results and findings for the saturating and prepreg resin selection phase are presented. Since a total of 186 double lap-shear coupons were tested, the results are described according to the following categories:

- Room Temperature Cured Resins
- Room Temperature Cured Resins with Wetting Agent
- Room Temperature Cured Hybrid Resins
- Heat Cured Resins
- Heat Cured Prepreg Resins
4.1.2 Room Temperature Cured Resins

Ten different resins were evaluated using double lap-shear coupons fabricated and cured at room temperature. The resins selected for further study were based on the shear stress results of these double-lap shear tests. Average shear stress was calculated from measurements of the applied load and the bonded area on each side of the steel plate. Longitudinal strain was also determined from measured strains over a 100 mm (3.94 inch) gauge length crossing the bonded region using strain gauge displacement transducers that were attached to each side of the specimens. Results for the series of tests with the associating number of specimens tested are shown in Figure 4-1 and also given in Table 4-1. The resins that achieved highest shear strength of approximately 12 MPa were Degussa MBrace Saturant and Sika Sikadur 330. Other resins that performed well were the 3M resins, 3M DP460 and 3M DP810, and the SP Systems Ampreg 22 resin with both fast and slow hardeners. The resins that did not achieve adequate high shear stress were Jeffco 121, Reichhold Atprime 2, and Sika Sikadur 300.
Figure 4-1 Average shear strength and longitudinal strain at peak stress for wet lay-up resins cured at room temperature

Strain at peak stress slightly above 0.10 percent was common among the tests. Two resins achieved very high strain; they were 3M DP810 at 0.202 percent and Reichhold Atprime 2 at 0.309 percent. This could be accredited to the debonding failure mode by pullout of the fibers, indicating incomplete wetting of the fibers. Resins such as 3M DP460, Resinlab EP1246, SP Ampreg 22 (fast hardener) and SP Ampreg 22 (slow hardener) resulted in similar shear stress and strain values as shown in Table 4-1.
**Table 4-1 Average shear strength and longitudinal strain at peak stress values for wet lay-up resins cured at room temperature**

<table>
<thead>
<tr>
<th>Resin</th>
<th>No. of Tests</th>
<th>Shear Strength (MPa)</th>
<th>Strain at Peak Stress (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Average Shear</td>
<td>Standard Deviation</td>
</tr>
<tr>
<td>Degussa MBrace Saturant</td>
<td>5</td>
<td>12.33*</td>
<td>1.82</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td>6</td>
<td>12.06*</td>
<td>0.84</td>
</tr>
<tr>
<td>3M DP810</td>
<td>5</td>
<td>10.75</td>
<td>1.03</td>
</tr>
<tr>
<td>3M DP460</td>
<td>6</td>
<td>10.55</td>
<td>0.91</td>
</tr>
<tr>
<td>SP Systems Ampreg 22 (fast hardener)</td>
<td>5</td>
<td>10.16</td>
<td>0.20</td>
</tr>
<tr>
<td>SP Systems Ampreg 22 (slow hardener)</td>
<td>5</td>
<td>9.97</td>
<td>2.05</td>
</tr>
<tr>
<td>Resinlab EP1246</td>
<td>6</td>
<td>9.41</td>
<td>2.28</td>
</tr>
<tr>
<td>Sika Sikadur 300</td>
<td>6</td>
<td>7.09</td>
<td>0.27</td>
</tr>
<tr>
<td>Jeffco 121</td>
<td>6</td>
<td>6.22</td>
<td>0.78</td>
</tr>
<tr>
<td>Reichhold Atprime 2</td>
<td>3</td>
<td>2.76</td>
<td>0.11</td>
</tr>
</tbody>
</table>

*lower bound of shear strength since specimens failed by rupture of the CFRP instead of by debonding*

Most coupons exhibited linear stress strain behavior to failure, however, coupons that failed due to a debonding adhesive failure between the fibers and the adhesive exhibited nonlinear behavior. Coupon failures were a combination of rupture of the CFRP, adhesive pullout of the fibers, or complete debonding of the material. A comparison of stress strain curves for resins with different failure types is shown in Figure 4-2.
The most common mode of failure for the double lap-shear coupons were rupture of the carbon fiber material. This failure mode indicates full utilization of the fibers within the unidirectional sheets. Of all six of the ten resins that failed by rupture, complete saturating of the fibers was achieved with the exception of Sika Sikadure 330. During this series of tests, some pullout was observed before fiber rupture, indicating incomplete fiber wetting. For this reason, this resin was later tested in conjunction with a wetting agent, in an attempt to improve its performance.

Three of the remaining four resins resulted in poor saturation of the carbon fibers, causing fiber pullout within the resin layer, as shown in the light areas surrounding the fiber tows in Figure 4-3 for a typical 3M DP810 resin. The fiber tows that remain bonded to the steel strips are separated between the two pulled fiber tows on the top steel strip and three on the bottom strip. Examination of this specimen after testing revealed individual fibers within the
tows were not fully saturated with resin. The stress-strain behavior for these specimens showed significant elongation before failure due to this failure mode.

The Sika Sikadur 300 resin failed by debonding from the steel surface for all specimens tested even though full fiber saturation of the CFRP sheets was observed. Sikadur 300 is used mainly for bonding and saturating carbon fiber applications on porous concrete surface, and may not have bonded well to steel. For this reason, this resin was selected for further study as a saturating resin only for hybrid resin techniques.

4.1.3 Room Temperature Cured Resins with Wetting Agent

Due to incomplete fiber saturation and the presence of air voids observed in a few of the specimens cured at room temperature, a wetting agent was applied to improve saturation of the carbon fibers and consequently the bond strength. Three resins were selected for this application; they were Sika Sikadur 330, 3M DP810, and 3M DP460. Two doses of wetting agent were applied; the doses were 0.5 percent and 1.0 percent of the total mass of the applied resin, as recommended by the manufacturer. The number of specimens used for each resin is given in parenthesis in Figure 4.4.
Results from the coupon tests revealed no significant improvements in performance by using the wetting agent. Shear strength and strain differences were small, as shown in Figure 4-4 and Table 4-2. For two types of resin, the strength difference was within one standard deviation, while for the DP-810 resin, the shear strength clearly decreased due to the presence of the wetting agent. Due to the modest results, no further testing was completed.

Figure 4-4 Comparison of average shear strength and longitudinal strain at peak stress for wet lay-up resins with a wetting agent

Visual observations revealed no significant difference in the failure modes among the specimens tested without the wetting agent, or with either of the two doses of wetting agent.
4. EXPERIMENTAL RESULTS

Both the Sika Sikadur 330 resin and the 3M DP460 resin failed by rupture of the fiber; failure of the 3M DP810 resin was by fiber pullout.

Table 4-2 Average shear strength and longitudinal strain at peak stress values for wet lay-up resins with a wetting agent

<table>
<thead>
<tr>
<th>Resin</th>
<th>No. of Tests</th>
<th>Shear Strength (MPa)</th>
<th>Average Shear</th>
<th>Standard Deviation</th>
<th>Strain at Peak Stress (%)</th>
<th>Average Strain</th>
<th>Standard Deviation</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sika Sikadur 330-WA0.5%</td>
<td>3</td>
<td>10.66</td>
<td>1.28</td>
<td></td>
<td>0.101</td>
<td>0.01</td>
<td>rupture</td>
<td></td>
</tr>
<tr>
<td>Sika Sikadur 330-WA1.0%</td>
<td>3</td>
<td>12.42</td>
<td>1.78</td>
<td></td>
<td>0.123</td>
<td>0.01</td>
<td>rupture</td>
<td></td>
</tr>
<tr>
<td>3M DP460-WA0.5%</td>
<td>3</td>
<td>10.39</td>
<td>1.73</td>
<td></td>
<td>0.113</td>
<td>0.02</td>
<td>rupture</td>
<td></td>
</tr>
<tr>
<td>3M DP460-WA1.0%</td>
<td>3</td>
<td>11.46</td>
<td>0.55</td>
<td></td>
<td>0.124</td>
<td>0.01</td>
<td>rupture</td>
<td></td>
</tr>
<tr>
<td>3M DP810-WA0.5%</td>
<td>3</td>
<td>9.39</td>
<td>0.60</td>
<td></td>
<td>0.192</td>
<td>0.01</td>
<td>adhesive</td>
<td></td>
</tr>
<tr>
<td>3M DP810-WA1.0%</td>
<td>3</td>
<td>9.07</td>
<td>0.52</td>
<td></td>
<td>0.209</td>
<td>0.01</td>
<td>adhesive</td>
<td></td>
</tr>
</tbody>
</table>

4.1.4 Room Temperature Cured Hybrid Resins

A hybridization technique was investigated for the four resins that were observed to perform well in both wetting and saturating the fiber tows of the CFRP sheet, or bonding to steel. Those four resins were Degussa MBrace Saturant, Jeffco 121, Sika Sikadur 300, and Sika Sikadur 330. Procedure for fabricating the specimens was a first coating of the bonding resin onto the prepared steel surface followed quickly with the CFRP sheet that was saturated with the wetting resin. The technique required that both resins be applied wet-on-wet so that the bond between the two resins would be chemically active during the curing process. One exception to this procedure was the hybrid using the Degussa MBrace Primer and Saturant; the manufacturer recommended that the Primer first set before application of the CFRP sheet wetted with the saturating resin.
Compatibility of the resins became an important aspect of the success of the hybrid resin test specimens. This was evidenced by the varying ultimate shear strength and elongation results. Comparisons were made of the hybrid combinations to the original room temperature cured resin tests alone to determine the effectiveness of these hybrid resins. Figure 4-5 show these comparisons for each test series.

![Figure 4-5 Average shear strength and longitudinal strain at peak stress for wet lap-up of hybrid resin combinations](image_url)

Out of the four hybrid resin combinations, only the Jeffco 121 hybrids resulted in small improvements in ultimate average shear strength and longitudinal strain at peak stress. The
Sika Sikadur 300 and 330 Hybrid was a balance of the high and low shear and strain values demonstrated by the two resin performances. Its performance appeared to be limited by the performance of the Sika Sikadur 300 resin. The Degussa MBrace Primer and Saturant Hybrid resin experienced a small decrease in its ultimate shear strength and strain compared to the performance of the Degussa MBrace Saturant alone.

Table 4-3 Average shear strength and longitudinal strain at peak stress values for wet lap-up of hybrid resin combinations

<table>
<thead>
<tr>
<th>Resin</th>
<th>No. of Tests</th>
<th>Shear Strength (MPa)</th>
<th>Strain at Peak Stress (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jeffco 121-</td>
<td>6</td>
<td>8.60</td>
<td>0.267</td>
</tr>
<tr>
<td>Sika Sikadur 300</td>
<td></td>
<td>0.88</td>
<td>0.28</td>
</tr>
<tr>
<td>Jeffco 121-</td>
<td>6</td>
<td>12.55</td>
<td>0.174</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td></td>
<td>1.72</td>
<td>0.33</td>
</tr>
<tr>
<td>Sika Sikadur 300-</td>
<td>4</td>
<td>9.14</td>
<td>0.093</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td></td>
<td>1.53</td>
<td>0.12</td>
</tr>
<tr>
<td>Degussa MBrace Primer-Saturant</td>
<td>6</td>
<td>10.95</td>
<td>0.123</td>
</tr>
</tbody>
</table>

Of all the tests, the hybrid resin combination of the Jeffco 121 and the Sika Sikadur 300 resins achieved a considerably high longitudinal strain at peak stress. This elongation was much higher than expected since previous test results for both the Jeffco 121 and the Sika Sikadur 300 resins were 0.112% and 0.075% respectively. A plausible reason for this unexpected strain behavior could be the increased adhesive thickness. Another reason could be that the two resins reacted with one another, and thus changed the properties of the Jeffco 121 resin,
which was bonded to the steel surface. A noticeable difference in the failure mode for these coupons also supports this possibility, as shown in Figure 4-6. A summary of the test results is shown in Table 4-3.

4.1.5 Heat Cured Resins

Heat curing of specimens was performed to determine if bond strength could be improved for the CFRP sheet strengthening system. Two resins were selected for this study; they were Sika Sikadur 330 and SP Systems Ampreg 22. Both series of coupons were heated at an elevated temperature of 50°C for sixteen hours and then removed from the oven to cure at room temperature for seven days. Results from these tests show, as shown in Table 4-4.

Table 4-4: Average shear strength and longitudinal strain at peak stress values for wet lap-up of heat cured resins

<table>
<thead>
<tr>
<th>Resin</th>
<th>No. of Tests</th>
<th>Shear Strength (MPa)</th>
<th>Strain at Peak Stress (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sika Sikadur 330 Heat Cured</td>
<td>6</td>
<td>11.10</td>
<td>0.111</td>
</tr>
<tr>
<td>SP Systems Ampreg 22 Heat Cured (slow hardener)</td>
<td>6</td>
<td>9.10</td>
<td>0.097</td>
</tr>
</tbody>
</table>
As shown in Figure 4-7, the effect of heat curing the double lap-shear coupons with the Sika Sikadur 330 and SP Systems Ampreg 22 resins weakened the shear strength and elongation for all tests. This result indicates that the heating of the steel also could have an adverse effect on the bond during the curing process. The steel plates of the coupons were small in size, thus it was possible they experienced a temperature increase at the same rate as the resin. Much larger sections of steel, such as steel girders, would take longer to heat up within the 16 hour allotted time for the heat curing. Heat curing methods are commercial today for FRP repair and strengthening techniques, thus the size of the coupons is thought to also be a cause for the decrease in shear strength and strain.
Failure of the heat cured coupons was by rupture of the CFRP sheets between the two steel plates. A linear stress strain relationship was exhibited in the stress strain curves for both resins. An example of one is shown in Figure 4-8.

4.1.6 Heat Cured Prepreg Resins

Proprietary prepreg saturating resins developed by Reichold, Inc. were also tested to determine the performance of these resins with the high modulus CFRP sheets. Three saturating resins were tested in conjunction with two bonding resins previously tested. Once the prepreg specimens were bonded, the specimens were placed into an oven to cure. Results of these three saturating prepreg resins are listed in Table 4-5 and shown in Figure 4-10.

Figure 4-8 Rupture failure of typical coupon using SP Systems Ampreg 22 heat cured resin
Table 4-5 Average shear strength and longitudinal strain at peak stress values for wet lay-up prepreg resins heat cured

<table>
<thead>
<tr>
<th>Resin</th>
<th>No. of Tests</th>
<th>Average Shear Strength (MPa)</th>
<th>Standard Deviation</th>
<th>Average Strain (%)</th>
<th>Standard Deviation</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prepreg 1A</td>
<td>5</td>
<td>3.26</td>
<td>1.46</td>
<td>0.054</td>
<td>0.20</td>
<td>rupture</td>
</tr>
<tr>
<td>Prepreg 2A</td>
<td>3</td>
<td>2.23</td>
<td>0.40</td>
<td>0.152</td>
<td>0.17</td>
<td>rupture</td>
</tr>
<tr>
<td>Prepreg 5A</td>
<td>4</td>
<td>2.15</td>
<td>0.90</td>
<td>0.102</td>
<td>0.85</td>
<td>rupture</td>
</tr>
<tr>
<td>Jeffco 121-Prepreg 1A</td>
<td>3</td>
<td>1.00</td>
<td>0.16</td>
<td>0.440</td>
<td>0.84</td>
<td>adhesive</td>
</tr>
<tr>
<td>Jeffco 121-Prepreg 2A</td>
<td>3</td>
<td>1.28</td>
<td>0.07</td>
<td>0.636</td>
<td>1.24</td>
<td>adhesive</td>
</tr>
<tr>
<td>Jeffco 121-Prepreg 5A</td>
<td>3</td>
<td>5.16</td>
<td>0.74</td>
<td>0.236</td>
<td>1.16</td>
<td>adhesive</td>
</tr>
<tr>
<td>Sika Sikadur 330-Prepreg 1A</td>
<td>3</td>
<td>4.45</td>
<td>1.08</td>
<td>0.127</td>
<td>0.62</td>
<td>adhesive</td>
</tr>
<tr>
<td>Sika Sikadur 330-Prepreg 2A</td>
<td>3</td>
<td>3.84</td>
<td>0.25</td>
<td>0.078</td>
<td>0.22</td>
<td>adhesive</td>
</tr>
<tr>
<td>Sika Sikadur 330-Prepreg 5A</td>
<td>3</td>
<td>3.02</td>
<td>0.66</td>
<td>0.075</td>
<td>0.20</td>
<td>adhesive</td>
</tr>
</tbody>
</table>

From Table 4-5, it can be seen that the shear strengths were much lower than for the specimens using the room temperature cured wet lay-up saturating resins. There are two possible reasons why the prepreg resins did not perform as expected with the HM fiber. The requirement for an elevated cure temperature would have induced significant thermal stresses upon cooling. However, this does not explain the very high strain to failure compared to some of the wet lay-up resins. From observations of the failed specimens, it seemed that at least some of the prepreg resins had an adverse affect on the curing of the bonding resin, as shown in Figure 4-9. For many of these specimens the failure

Figure 4-9 Failure mode of typical Sika Sikadur 330-Prepreg 2A test specimen
was adhesive between the bonding resin and the steel, while the sudden transfer of load led to rupture of the CFRP bonded to the other side. None of the tests using the Sika Sikadur 330 resin alone exhibited this type of adhesive failure, indicating that there may have been some kind of interaction between the prepreg resin and the bonding resin. Based on the results of the prepreg resin tests, these resins were discontinued from further study.

![Average shear strength and longitudinal strain at peak stress for wet lap-up of heat cured prepreg resins](image)

Figure 4-10 Average shear strength and longitudinal strain at peak stress for wet lap-up of heat cured prepreg resins
4. EXPERIMENTAL RESULTS

4.1.7 Summary

The most suitable resins for bonding the unidirectional carbon fiber sheets to steel were determined based on the higher shear strength, strain that can be achieved, and failure mode from each series of tests. Resins that bonded the CFRP sheets to the steel the best were those whom achieved the highest average shear stress and strain, and failed the CFRP sheet by rupture. Two resins that consistently demonstrated these highest values among all resins when tested at room temperature and with the additional variables were Degussa MBrace Saturant and Sika Sikadur 330.

Different variables, such as the inclusion of a wetting agent, hybrid combinations of resins, and heat curing methods, were included in this resin selection for optimizing the performance of the individual resins cured and tested at room temperature. These additional variables showed little improvement to the results from the room temperature cured resin tests. For some tests, these variables weakened the ultimate shear strength and elongation behavior. The small improvements that were achieved occurred for certain resins and not for the group overall.

For the application of the wetting agent, only one resin, 3M DP460, showed small improvement with its addition. The others did not improve and in fact experienced a decrease in bond strength. The reason why the wetting agent reacted well with one resin and not the others is unknown, though a relationship of the dose amounts to bond strength was determined to affect the behavior of the resins. This relationship showed that doubling the recommended dose amount of 0.5 percent to 1.0 percent improved test results marginally well, though the margin of improvement depended upon the resin.

Combining resins to form hybrid resin systems was thought to utilize the properties of each combined resin to form a superior bond capacity. Half of the four hybrid resins tested did
show improvements in bond strength and elongation, but the other half did not. Of the several combinations, only one, the Jeffco 121 – Sika Sikadur 330 hybrid, was competitive for use in the strengthening system. Its shear strength and elongation were higher than most tests with a strength value of 12.55 MPa and an elongation value of 0.172 percent. No formative pattern or relationship was determined from these tests since the behavior of each series of test varied depending upon the resins selected for the hybrid combinations.

In comparison to the room temperature cured double lap-shear coupons, tests for the heat cured specimens resulted in loss of strength and elongation for both the Sika Sikadur 330 resin and the SP Systems Ampreg 22 resin. This loss of bond strength was thought to have been affected by the temperature increase of the bonded steel plates, which may have affected bond conditions during the curing process. The assumption that the steel plates had similar temperature increases to that of the resins was not proven.

Prepreg resins did not perform well within this resin selection study. Adverse effects on the curing of the bonding resins was observed by the high strain behavior of the Jeffco 121 bonding resin and the adhesive failure mode exhibited by the Sika Sikadur 330 resin. The very low shear strengths achieved in comparison to the room temperature cured resins used for wet lay-up of the CFRP sheets was the main reason why these resins were discontinued from further study.

After all completed testing, a selection of the most suitable resin(s) for bonding the high modulus unidirectional fiber sheets to steel was made. Final results showed three resins that stood out with the highest bond strengths and high elongation strains; these resins were the room temperature cured resins of Degussa MBrace Saturant and Sika Sikadur 330, and the hybrid resin mixture of Jeffco 121 – Sika Sikadur 330. Due to the close performance of these resins with shear strengths of 12.33 MPa, 12.06 MPa, and 12.55 MPa, respectively, selection
came down to practicality and consideration for the next phase in the experimental program, which was the study of development length for the fiber sheets in Phase 2.

The addition of a two layer CFRP sheet application was adopted into the Phase 2 testing program, and was found to be the deciding factor for the resin selection. The hybrid resin system was thought to cause more unknowns in the determination of the development length if applied as a two layer CFRP sheet application since behavior of the CFRP sheets would be a function of two resins instead of one. Thus for comparison purposes, the room temperature cured resins of Degussa MBace Saturant and Sika Sikadur 330 were selected as the most suitable resins for the study of the CFRP sheets in Phase 2 of the experimental program.

4.2 PILOT TESTS SPECIMEN DEVELOPMENT FOR ADHESIVE SELECTION

4.2.1 Introduction

Different types of specimens were considered for determining the most effective adhesive for bonding CFRP laminate strips to the tension side of steel flexural members. Test specimens that simulated typical bond stresses that occur in field applications were selected. The selected was different from the double lap-shear coupons used for the resin selection and was derived to accommodate the high tensile stresses of the laminate strips. These specimens were also used to determine development length for bonded CFRP sheets and laminate strips. Selection of the test specimen took several trial tests using both the CFRP laminate strips and the unidirectional CFRP sheets. The selected specimen was a W-shape steel beam representative of practical beams and girders used in field applications.

4.2.2 HSS Beam Tests

The first specimens tested in the search were square cold formed Hollow Structural Section (HSS) beams. These beams were comprised of the same A572 grade 50 steel that made up
several large scale monopole towers being tested in the lab for the same project. Two different wall thicknesses were considered, a 6.4 mm (1/4 inches) thickness and a 4.8 mm (3/16 inches) thickness. Three beams were tested for each thickness; one was strengthened with a 38.0 mm (1.5 inch) wide by 508 mm (20 inch) long bonded CFRP laminate strip, another was strengthened with two layers of a 45.0 mm (1.8 inch) wide by 508 mm (20 inch) long unidirectional CFRP sheet, and the third was unstrengthened for use as a control specimen. Evaluation of the beams comprised of several criteria, such as the failure mode of the CFRP system, the measured strain at failure of the CFRP, stiffness of the strengthened beam and the reserved load capacity at failure. Comparisons were also made to the unstrengthened control beam to determine the effectiveness of each CFRP system.

Two different loading techniques were initially considered in the testing program. One series of three beams were loaded under three-point loading, while the other series of three beams were loaded under four-point loading. All beams were simply supported, supported at each end with roller supports.

Results for all HSS beams are shown in Table 4-6. No significant advantages were determined from each series of tests. Increases of the stiffness in comparison to the unstrengthened control beam were about 10 percent for the 2 layers of the unidirectional CFRP sheets, while one layer of the CFRP laminate strip achieved stiffness increases averaging 18 percent. The reserved load capacity at failure of each beam was averaged to 9 percent and 16 percent for the CFRP sheets and laminate strips respectively.
Table 4-6 Test results for HSS beam pilot tests

<table>
<thead>
<tr>
<th>Thickness (mm)</th>
<th>Flexural Loading</th>
<th>CFRP System</th>
<th>Peak Strain (%)</th>
<th>Failure Load (kN)</th>
<th>Stiffness Increase (%)</th>
<th>Reserved Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.8</td>
<td>3 point</td>
<td>2 sheets</td>
<td>0.287</td>
<td>47.9</td>
<td>10.0</td>
<td>10.0</td>
<td>rupture</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 strip</td>
<td>0.287</td>
<td>56.3</td>
<td>19.0</td>
<td>17.0</td>
<td>rupture</td>
</tr>
<tr>
<td>6.4</td>
<td>4 point</td>
<td>2 sheets</td>
<td>0.259</td>
<td>66.7</td>
<td>10.0</td>
<td>7.0</td>
<td>debonding</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 strip</td>
<td>0.304</td>
<td>81.8</td>
<td>17.0</td>
<td>14.0</td>
<td>rupture</td>
</tr>
</tbody>
</table>

Figure 4-11 Test setup and debonding failure of CFRP sheets on the 6.4 mm HSS specimen

Only one beam out of this series of tests resulted in an adhesive failure mode by debonding of the CFRP material from the steel surface; this beam was the 6.4 mm (1/4 inch) thick HSS specimen strengthened with two layers of the CFRP sheets as shown in Figure 4-11. To prevent this type of failure mode from occurring in the 4.8 mm (3/16 inch) thick HSS beam test, the ends of the CFRP sheets were wrapped transversely around the section. Wrapping of the sheets at the two ends prevented the CFRP from debonding from the steel surface and...
also increased the ultimate strain in the CFRP sheets. A close up of one of the wrapped ends of the 4.8 mm (3/16 inch) thick HSS beam is shown in Figure 4-12.

**4.2.3 W4 x 13 Beam Tests**

The second test specimen considered in this experimental phase was W4 x 13 hot rolled steel. Two beams were considered in this pilot study. The first test conducted was loading of an unstrengthened beam up to 60% of its yield strength, then unloaded and strengthened with one 76.0 mm (3.0 inch) wide by 508 mm (20 inch) long section of the CFRP laminate strip. The second beam was strengthened with two layers of the 83.0 mm (3.3 inch) wide by 508 mm (20 inch) long section of the CFRP sheet bonded to the tension flange. Both beams were also supported by roller supports and loaded at mid-span, therefore the beam was tested under three-point loading. Results for these tests are shown in Table 4-7. Strain at failure, stiffness, reserved capacity at failure, and the failure modes were recorded.

<table>
<thead>
<tr>
<th>CFRP System</th>
<th>Strain at Failure (%)</th>
<th>Failure Load (kN)</th>
<th>Stiffness Increase (%)</th>
<th>Reserved Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 sheets</td>
<td>0.344</td>
<td>211.1</td>
<td>10.0</td>
<td>24.0</td>
<td>rupture</td>
</tr>
<tr>
<td>1 strip</td>
<td>0.303</td>
<td>224.0</td>
<td>8.0</td>
<td>19.0</td>
<td>rupture</td>
</tr>
</tbody>
</table>

Results for the W4 x 13 beams showed increases in stiffness and reserved strength capacities for the amount of CFRP material used for these tests. The strengthened beams experienced
stiffness increases of about 10 percent, had reserved strength capacity of 24 percent and 19 percent for both the CFRP sheets and laminate strip respectively, and both beams fully utilized the CFRP material by causing the material to fail by rupture.

4.2.4 Summary

In comparison to the HSS beam tests, the W4 x 13 beams achieved higher strains in the CFRP material and higher post yield reserved strength capacities at failure. This was clearly observed by the measured increase of the flexural strength capacity for the unidirectional CFRP sheet beam, which was more than double the strength gained for the HSS beams strengthened with two layers of the same CFRP sheet. Although amounts of CFRP material used for each series of beam tests could have affected the results, the differences observed may be explained by a more influencing factor. This influencing factor was the stress strain behavior of the steel used in the two types of specimens.

The square HSS beams had a nonlinear stress strain relationship; the specimens were cold-formed and welded together in their manufacturing process. The W4 x 13 beams were hot rolled and exhibited a bilinear stress strain relationship. This difference in the stress strain relationship caused the hot rolled W4 x 13 beams to achieve higher post yield strength and stiffness as compared to the nonlinear stress strain relationship, which caused the steel to strain harden immediately after yielding.

Also worth mentioning is the fact that the HSS steel sections were welded together at one seam along the length of the beam. The location of this weld was not taken into consideration during the strengthening and testing process. Only after testing and analysis was this fact discovered. Variances in the strengths of the beams were possible due to the unknown contribution the welded sections had on the stress carrying capacity. Welded sections provide more strength than the surrounding steel sections due to the higher grade
4. EXPERIMENTAL RESULTS

steel used for the weld. Higher strength steel is used at weld locations for prevention of steel separation.

4.2.5 Final Test Specimen

Based on the results of the two different steel sections selected for the specimen development pilot tests, the bi-linear behavior of hot rolled mild steel was the preferred steel section for further investigation of bonding the high modulus CFRP material to steel. The W4 x 13 specimen used for the pilot tests was a heavy and thick flanged beam thought too large for the scope of testing considered for understanding bond stresses and development lengths. A smaller and lighter scaled beam that could effectively utilize the CFRP material without the unnecessary weight and size was more suited for the scope of research, and thus a search was made to uncover a steel section that could resemble typical steel girders used in the field, such as bridge girders, while still utilizing the bi-linear stress strain relationship of hot rolled mild steel.

The selected beam was a Super Light Beam (SLB) and a steel plate was welded to its compression flange to simulate a typical concrete deck acting in composite action for larger scaled bridge girders. The beam specimen is described in detail in Section 3.5.1 and the test results from the use of this test specimen are described in the following sections of this chapter.

4.3 Phase 1(b) Adhesive Selection for Bonded Laminate Strips

4.3.1 Introduction

This program is designed to select the most effective adhesives for CFRP laminates strips proposed for use to strengthen steel structures. Considering the commercially available adhesives in the market, the program included the following six adhesives: Fyfe Tyfo MB2,
Jeffco 121, Sika Sikadur 30, SP Systems Spabond 345, Vantico Araldite 2015, and Weld-On SS620. Of these resins, only the Weld-On SS620 adhesive was acrylic; the rest were amine-epoxy structural adhesives.

Flexural tests were thought to be useful for determining the adhesive strengths of all the adhesives selected for this study due to the expectation that the CFRP strips used would generate significant normal or peeling stresses due to their greater thickness than the CFRP sheets used in the resin selection phase. It was also desired to conduct the bond evaluation process using similar specimen configurations to those of typical large scale members. This configuration would result in interfacial bond stresses that would be similar in magnitude and proportion not only for the large scale specimens that were to be later studied, but also for targeted types of structures to be strengthened with CFRP materials in the field.

Load behavior of the selected W-shape SLB with the welded plate in the compression flange resulted in yielding of the tension flange well before the compression flange due to the shift of the neutral axis caused by the additional steel plate. Also, bonded CFRP strips could be fully utilized due to the larger moment arm acting between the neutral axis and the centroid of the CFRP material.

Different lengths of the CFRP laminate strip were considered to study the adhesive’s behavior. The change in the bond length of the strip resulted in higher average bond stresses consequently making the adhesive performance more critical. Adoption of the different lengths also changed the test program to study development length as well, since the lengths of the CFRP strip could be a function of the development length. The development length was defined as the distance from the loading point to the edge of the CFRP strip due to the symmetric configuration of the four point bending test used. Thus, information for both the adhesive performance and the development lengths of the laminate strips were determined.
Failure of the bonded CFRP strips to the tension flange was due to either debonding or achieving its tensile capacity and rupture. The adhesives that resulted in debonding of the CFRP strip were eliminated from the evaluation. Testing for those adhesives that achieved rupture of the strips was continued with a shorter development length. As the development length was reduced, the adhesive was subjected to higher shear and normal stresses, increasing the possibility of a debonding failure. This process was continued until further reduction in the development length would result in debonding at a lower strain.

The thickness of the CFRP laminate strips were 1.42 mm. This thickness was equivalent to four layers of the CFRP sheet strengthening system. At the start of testing, the development length study of the CFRP sheets was not underway. The initial development length used in the testing program was 203 mm (8 inches) for one ply of the CFRP laminate strips using the selected adhesives. All strips were bonded to the tension flange of the beam specimens. Adhesives were allowed to cure 7 days before testing the simply supported beams under four-point flexural loading.

The criteria for determining the most suitable adhesive for bonding the high modulus CFRP laminate strips to steel were the failure modes, whether the CFRP strips failed by debonding or by rupturing of the fibers, ultimate strain of the laminate strip at failure, stiffness and strength comparisons to an unstrengthened control beam. The first criteria of failure modes were the first assessment that determined each adhesive’s performance. Though the failures of the laminate strip were dependent upon the applied lengths of the laminate strip, they helped determine the strength of the adhesive in resisting shear and normal stresses induced by the beam loaded in flexure. Shear stresses within the adhesive are highest at the bonded ends of the strip and are affected by the length of the applied CFRP material Garden et al. (1998). Thus, the change in the bond length of the strip resulted in higher average bond stresses at the laminate strip ends making the adhesive performance a decisive aspect in determining the most suitable bonding adhesive.
The second criteria that was used for selecting the adhesive for the CFRP laminate strip strengthening system was the ultimate strain at failure on the strip. The CFRP laminate strip strain of 0.330 percent, which was an average value determined from tension tests completed by the CFRP manufacturer, was the target strain the adhesives were to help the laminate strips achieve. An accurate measurement of the strain on the bonded strips was carried out by foil type electric resistance strain gauges placed at the maximum moment location for each beam.

Stiffness and strength comparisons to an unstrengthened beam were the last evaluation process used in determining the most effective adhesive. Stiffness of the test specimens was determined as the slope of the applied load verses mid-span displacement curves, while the strengths were comparisons in load at specific locations thought useful for practicing engineers. The first strength location was the capacity of the beam under service load conditions based on deflection criteria used in LRFD design, which for our study was L/360. The letter L stands for the entire unbraced length of the beam. The second strength location was based on measured strain at the bottom tension flange that corresponded to sixty percent of the yield strength of the small scaled steel beam; this corresponded well with ASD design. The third and final location was at failure of the CFRP, the location where the applied load dropped suddenly.

A typical applied load verses mid-span displacement curve, from which stiffness and strengths were determined, is shown in Figure 4-13. The unstrengthened control specimen is shown as a thin line and is marked with an arrow. Performance of the strengthened beams is shown with thicker lines above the control line with failure of the CFRP strip at the location of sudden load drop. Increases in strengths are shown by arrows. The load deflection response is linear until yielding of the steel. At the top of the strengthened beam curve, the response becomes slightly nonlinear due to the loss of stiffness due to yielding of the beam within the constant moment region. The amount of strength and stiffness increase in
comparison to the control beam depends upon the effectiveness of the bond or the rupture of the CFRP material. Beams that fail by debonding typically do not have higher strength increases than beams that fail by rupture of the CFRP. Likewise, beams that fail by rupture of the CFRP usually achieve higher deflections at failure than those that fail by debonding.

![Graph showing applied load versus mid-span displacement of beam strengthened with one ply thickness CFRP laminate strip](image)

*Figure 4-13 Typical applied load verses mid-span displacement of beam strengthened with one ply thickness CFRP laminate strip*

After failure of the CFRP by rupture or debonding, the strengthened beams revert to its behavior before strengthening. This usually occurs when the bonded strip is completely detached from the specimen from a debonding failure or debonded only surrounding the highest moment region at midspan when the CFRP fails by rupture. After the load drop due to loss of composite action with the CFRP strip, the stiffness of the beam is greatly reduced and tends toward its plastic strength. Testing was continued in most cases to a deflection of...
15 mm. Beyond this deflection, the specimens became unstable due to lateral-torsional buckling of the beam or by local buckling within the load points of the additional steel plate that was welded to the compression flange. In either case, testing was stopped before the beams became unstable. Figure 4-14 shows the beam strengthened with CFRP strips having a 203 mm development length, and bonded with Tyfo MB2 adhesive just before unloading.

![Image of beam strengthened with CFRP strips](image)

*Figure 4-14 Beam strengthened with CFRP strips having a 203 mm development length, and bonded with Tyfo MB2 adhesive just before unloading*

### 4.3.2 Overall Test Results

In determining the most suitable adhesives for bonding the CFRP strips to steel, the CFRP strip strain at failure in conjunction with observation of the failure mode provided the best indication of which adhesives were able to fully utilize the CFRP material at the shortest development lengths. Table 4-8 summarizes the results of the adhesive selection. The two adhesives selected for further study in the development length phase were Weld-On SS620 and SP Systems Spabond 345 adhesives. Both adhesives were found to achieve highest strains by rupture of the CFRP strip at the shortest development lengths, while the remaining
adhesives achieved their highest strains at other lengths. This is shown graphically in Figure 4-15. The manufacturer target strain for the pultruded laminate strips is marked as a line across the graph.

Table 4-8 CFRP strip strain at rupture/debonding for tested adhesives and development lengths with one ply of CFRP strips

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>Development Length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>203 mm</td>
</tr>
<tr>
<td>Weld-On SS620</td>
<td>0.308</td>
</tr>
<tr>
<td></td>
<td>rupture</td>
</tr>
<tr>
<td>SP Spabond 345</td>
<td>0.288</td>
</tr>
<tr>
<td></td>
<td>rupture</td>
</tr>
<tr>
<td>Vantico Araldite 2015</td>
<td>0.309</td>
</tr>
<tr>
<td></td>
<td>rupture</td>
</tr>
<tr>
<td>Jeffco 121</td>
<td>0.298</td>
</tr>
<tr>
<td></td>
<td>rupture</td>
</tr>
<tr>
<td>Fyfe Tyfo MB2</td>
<td>0.347</td>
</tr>
<tr>
<td></td>
<td>rupture</td>
</tr>
<tr>
<td>Sika Sikadur 30</td>
<td>0.281</td>
</tr>
<tr>
<td></td>
<td>debond</td>
</tr>
</tbody>
</table>

*Note underlined values are the average of two test results*
In addition to observing ultimate failure strains and failure modes with the bonded CFRP strip for each adhesive, a review of the stiffness and strength increases of each adhesive was also performed. The trend for the entire test program using one ply CFRP strips showed that higher stiffness and strengths could be achieved when more material is applied in longer lengths. This is observed from averages taken from each series of development length tests as given in Table 4-9. The exceptions to this trend are the 51 mm development length tests and the 127 mm development length tests using the Jeffco 121 adhesive that have averages...
taken from only two tests in the series. Overall, the beams strengthened with the high modulus CFRP strip averaged a 14.1 percent increase in stiffness over the control specimen, around 15 percent increase for the service strengths and a 23.7 percent increase in the reserved capacity at failure of the material by either rupture or debonding.

Table 4-9 Averages of determined stiffness and strength increases for the series of adhesives tested at each development length

<table>
<thead>
<tr>
<th>L_d (mm)</th>
<th>No. of Tests</th>
<th>Strain at Failure (%)</th>
<th>Stiffness Increase (%)</th>
<th>Service Capacity (%) - L/360</th>
<th>Service Capacity (%) - 60% Fy</th>
<th>Reserved Capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>51</td>
<td>2</td>
<td>0.221</td>
<td>11.7</td>
<td>12.3</td>
<td>12.2</td>
<td>10.6</td>
</tr>
<tr>
<td>76</td>
<td>5</td>
<td>0.275</td>
<td>10.1</td>
<td>11.1</td>
<td>11.0</td>
<td>19.7</td>
</tr>
<tr>
<td>102</td>
<td>8</td>
<td>0.279</td>
<td>12.8</td>
<td>13.5</td>
<td>13.5</td>
<td>22.6</td>
</tr>
<tr>
<td>127*</td>
<td>2</td>
<td>0.266</td>
<td>13.6</td>
<td>13.7</td>
<td>13.7</td>
<td>21.6</td>
</tr>
<tr>
<td>152</td>
<td>5</td>
<td>0.304</td>
<td>15.4</td>
<td>15.9</td>
<td>15.6</td>
<td>27.9</td>
</tr>
<tr>
<td>203</td>
<td>6</td>
<td>0.305</td>
<td>18.3</td>
<td>18.9</td>
<td>18.8</td>
<td>29.4</td>
</tr>
</tbody>
</table>

* Jeffco 121 adhesive was the only adhesive tested at the 127 mm development length

As a general rule, each adhesive achieved higher than the averages within the range of their proposed development length. Strong performers throughout all of the tests were the SP Systems Spabond 345 adhesive and the Weld-On SS620 adhesive; this can be seen from the comparison graph for the beams strengthened with the 102 mm development length using five different adhesives as shown in Figure 4-16. These adhesives achieved the highest ultimate capacities within this series of tests, as shown by the performance ranking list in the legend. The thicker lines in the graph represent those adhesives that failed by debonding and thus achieved lower ultimate CFRP strains and strength capacities. The thin lines represent those beams that failed by rupture of the CFRP material and thus achieved higher strains and higher ultimate capacities. The behavior of these two adhesives at such a short development length was another reason why these two were selected for further study.
4. EXPERIMENTAL RESULTS

4.3.3 Test Results of SP Systems Spabond 345 Epoxy Adhesive

The top performer in the adhesive selection tests was the SP Systems Spabond 345 epoxy adhesive. In most cases, it achieved highest stiffness and strength among adhesives that attained the same development length. It also averaged a rupture strain of 0.30 percent within the CFRP laminate strip, which was close to the 0.330 percent strain expected for the unbonded laminate strip and was slightly above the average strain measured for all adhesives. Several different development lengths were tested before the adhesive showed much decline in strength, stiffness and strain. The most predominant failure mode was rupture of the

Figure 4-16 Load displacement response curves for beams strengthened with different adhesives at a 102 mm development length
laminate strips; failure by debonding did not occur until the 76 mm (3 inch) development length was tested. All test results are given in Table 4-10.

Table 4-10 One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the SP Systems Spabond 345 adhesive tests

<table>
<thead>
<tr>
<th>$L_0$ (mm)</th>
<th>Strain at Failure (%)</th>
<th>Stiffness Increase (%)</th>
<th>Service Capacity (%) - $L/360$</th>
<th>Service Capacity (%) - 60% $F_y$</th>
<th>Reserved Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>203</td>
<td>0.288</td>
<td>19.2</td>
<td>20.4</td>
<td>20.3</td>
<td>30.5</td>
<td>rupture</td>
</tr>
<tr>
<td>152</td>
<td>0.294</td>
<td>17.8</td>
<td>18.1</td>
<td>18.0</td>
<td>28.5</td>
<td>rupture</td>
</tr>
<tr>
<td>102</td>
<td>0.311</td>
<td>13.1</td>
<td>13.8</td>
<td>13.7</td>
<td>24.5</td>
<td>rupture</td>
</tr>
<tr>
<td>76</td>
<td>0.243</td>
<td>9.2</td>
<td>10.1</td>
<td>10.0</td>
<td>14.9</td>
<td>debonding</td>
</tr>
<tr>
<td>51</td>
<td>0.183</td>
<td>12.9</td>
<td>13.7</td>
<td>13.7</td>
<td>14.8</td>
<td>debonding</td>
</tr>
</tbody>
</table>

*Note that underlined values are the average result of two tests

The stiffness and strengths determined for the SP Systems Spabond 345 adhesive were among the highest of all adhesives. The average measured stiffness before failure by rupture or debonding was 14.5 percent, while the average strength increases were 15.2 percent, 15.2 percent and 24.6 percent respectively for each category listed in Table 4-10.

![Figure 4-17 Failure of 102 mm CFRP laminate strip using SP Systems Spabond 345](image)

Also noted was the inter-laminar shear failure that occurred for all tests. This shearing within the laminate strip caused a thin layer of the strip to remain bonded to the adhesive, while the top portion either ruptured or debonded from the steel surface. Figure 4-17 provides one example of the typical rupture
failures that occurred for the 203-102 mm (8-4 inch) development length tests. In the figure a portion of the laminate strip remains bonded to the adhesive.

A closer look of this phenomenon is shown in the debonding failure of the 76 mm (3 inch) development length CFRP laminate strip, shown in Figure 4-18. Again, a thin layer of the laminate strip remained bonded to the epoxy adhesive, indicating good bond was achieved between the CFRP laminate strip and the adhesive. No debonding of the adhesive from the steel surface was observed as well, which was another good indication that this adhesive bonds well to steel surfaces and can withstand the shear and normal stresses caused by the flexural loading of the beam specimens. The adhesive was never the cause of failure for all tests.

4.3.4 Test Results of Weld-On SS620 Acrylic Adhesive

Placing a very close second overall in this adhesive selection was the Weld-On SS620 acrylic adhesive. It was an effective adhesive in utilizing the CFRP material because the CFRP laminate strips failed by rupture for five out of the six tests completed. One example of this failure mode is shown Figure 4-19 for the 76 mm (3 inch) development length test. Bond between the steel and the adhesive showed no separation for all tests except for the 51 mm (2 inch) development length test. For this test, the CFRP laminate strip removed only a small
portion of the adhesive from the steel surface at one end of the strip. A photo of this debonding adhesive failure is shown in Figure 4-20. Test results are also reported for all tests in Table 4-11.

Table 4-11 One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the Weld-On SS620 adhesive tests

<table>
<thead>
<tr>
<th>L_D (mm)</th>
<th>Strain at Failure (%)</th>
<th>Stiffness Increase (%)</th>
<th>Service Capacity (%)-L/360</th>
<th>Service Capacity (%)-60% Fy</th>
<th>Reserved Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>203</td>
<td>0.308</td>
<td>18.2</td>
<td>18.2</td>
<td>18.0</td>
<td>28.8</td>
<td>rupture</td>
</tr>
<tr>
<td>152</td>
<td>0.296</td>
<td>15.4</td>
<td>16.2</td>
<td>16.1</td>
<td>26.4</td>
<td>rupture</td>
</tr>
<tr>
<td>102</td>
<td>0.316</td>
<td>14.4</td>
<td>15.3</td>
<td>15.2</td>
<td>25.2</td>
<td>rupture</td>
</tr>
<tr>
<td>76</td>
<td>0.290</td>
<td>9.9</td>
<td>11.0</td>
<td>10.9</td>
<td>21.1</td>
<td>rupture</td>
</tr>
<tr>
<td>51</td>
<td>0.259</td>
<td>10.5</td>
<td>10.9</td>
<td>10.9</td>
<td>10.6</td>
<td>debonding</td>
</tr>
</tbody>
</table>

* Note that underlined values are the average result of two tests
The Weld-On SS620 adhesive performed slightly less than the SP Systems Spabond 345 adhesive in all categories listed in Table 4-11 except for the strain at failure for all five rupture failure tests and the 76 mm (3 inch) development length test results. It achieved high strain in the CFRP strip for every test for an average of 0.303 percent, which was above the average for all adhesives and closest to the 0.330 percent strain expected for the strip material alone. It also attained the shortest development length among the elected adhesives in the test program. Inter-laminar shear of the CFRP laminate strip was diminutive in this series of tests. Average stiffness determined before failure was 13.7 percent. The average strength increases were 14.5 percent, 35.4 percent and 24.6 percent respectively for each category listed in Table 4-11.

4.3.5 Test Results of Vantico Araldite 2015 Epoxy Adhesive

Vantico Araldite 2015 was another strong candidate in the adhesive selection. Test results signify that it was competitive in stiffness and strength with the other adhesives averaging stiffness increases of 14.8 percent while strengths, such as the reserved capacity, averaged 25.7 percent, which was more than 8.5 percent higher than the overall average. Like the other adhesives, the performance of the adhesive decreased with decreasing development length. All test results are given in Table 4-12.
One failure mode of the adhesive selection tests was rupture of the laminate strip followed by debonding. Typically rupture occurred at the mid-span of the beam, as was the case for all rupture failures of this epoxy adhesive. A photo of this failure at mid-span is shown in Figure 4-21. A development length of 152 mm (6 inches) was bonded to the tension flange of this beam specimen shown in the figure. There was no observance of any inter-laminar shear near the location of failure for these tests.

![Figure 4-21 Failure of 152 mm (6 inch) CFRP laminate strip using Vantico Araldite 2015](image)

<table>
<thead>
<tr>
<th>$L_D$ (mm)</th>
<th>Strain at Failure (%)</th>
<th>Stiffness Increase (%)</th>
<th>Service Capacity (%) - $L/360$</th>
<th>Service Capacity (%) - 60% $F_y$</th>
<th>Reserved Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>203</td>
<td>0.309</td>
<td>21.0</td>
<td>21.2</td>
<td>21.0</td>
<td>30.7</td>
<td>rupture</td>
</tr>
<tr>
<td>152</td>
<td>0.298</td>
<td>14.8</td>
<td>15.4</td>
<td>15.3</td>
<td>26.5</td>
<td>rupture</td>
</tr>
<tr>
<td>102</td>
<td>0.282</td>
<td>12.1</td>
<td>12.4</td>
<td>12.3</td>
<td>23.8</td>
<td>rupture</td>
</tr>
<tr>
<td>76</td>
<td>0.277</td>
<td>11.3</td>
<td>12.3</td>
<td>12.2</td>
<td>21.8</td>
<td>debonding</td>
</tr>
</tbody>
</table>
4.3.6 Test Results of Jeffco 121 Epoxy Adhesive

The performance of the Jeffco 121 amine-epoxy adhesive was based on a longer length of the CFRP laminate strips than the other adhesives described. Stiffness of 16.8 percent and almost 31 percent for the reserved capacity were reported for the two longest lengths of the strips, while decreases in the stiffness and strengths were noticed as the development length was shortened. Out of all the adhesives, the Jeffco 121 adhesive showed the most variance between validation testing, either by rupture of the material or by debonding from the steel surface. This can be observed in Figure 4-23 for the 127 mm and 102 mm development lengths tested. Test results for the series of beam specimens are given in Table 4-13.

Figure 4-22 shows the figure of one of four rupture failures that occurred in the Jeffco 121 series of tests. The light-colored line drawn across the CFRP strip is the mark for the mid-span of the beam, where strain was recorded. Rupture of the strip in this figure divided the foil type strain gauge into two sections.
4. EXPERIMENTAL RESULTS

Table 4-13 One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the Jeffco 121 adhesive tests

<table>
<thead>
<tr>
<th>L₀ (mm)</th>
<th>Strain at Failure (%)</th>
<th>Stiffness Increase (%)</th>
<th>Service Capacity (%) - L/360</th>
<th>Service Capacity (%)-60% Fy</th>
<th>Reserved Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>203</td>
<td>0.298</td>
<td>19.4</td>
<td>20.1</td>
<td>20.0</td>
<td>30.7</td>
<td>rupture</td>
</tr>
<tr>
<td>152</td>
<td>0.328</td>
<td>14.3</td>
<td>14.5</td>
<td>14.4</td>
<td>29.2</td>
<td>rupture</td>
</tr>
<tr>
<td>127</td>
<td>0.266</td>
<td>13.6</td>
<td>13.7</td>
<td>13.6</td>
<td>21.6</td>
<td>rupture</td>
</tr>
<tr>
<td>102</td>
<td>0.244</td>
<td>10.6</td>
<td>10.8</td>
<td>10.7</td>
<td>19.7</td>
<td>debonding</td>
</tr>
</tbody>
</table>

*Note that underlined values are the average result of two tests*

![Graph](image_url)

Figure 4-23 Varying validation test results for Jeffco 121 adhesive
4. EXPERIMENTAL RESULTS

4.3.7 Test Results of Fyfe Tyfo MB2 Epoxy Adhesive

The Fyfe Tyfo MB2 adhesive was an amine-epoxy that performed well for the starting 203 mm (8 inch) CFRP laminate strip development length but later failed in shear by debonding the laminate strip from the steel surface for the shorter development lengths. It achieved a strain highest among all adhesives by exceeding the expected failure strain of 0.330 percent for the high modulus laminate strip in the first test, and later achieved one of the lowest strain readings of the six adhesives in the last test. Stiffness of the adhesive was on average 16.1 percent, while service load strength increases were 16.5 percent and 16.4 percent respectively. An averaged reserved capacity of 26.8 percent was the highest of all the tests. Values of the strain at failure, stiffness, strengths, and failure modes for the Fyfe Tyfo MB2 adhesive tests are given in Table 4-14.

Table 4-14 One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the Fyfe Tyfo MB2 adhesive tests

<table>
<thead>
<tr>
<th>$L_0$ (mm)</th>
<th>Strain at Failure (%)</th>
<th>Stiffness Increase (%)</th>
<th>Service Capacity (%)-L/360</th>
<th>Service Capacity (%)-60% $F_y$</th>
<th>Reserved Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>203</td>
<td>0.347</td>
<td>18.0</td>
<td>18.9</td>
<td>18.8</td>
<td>33.5</td>
<td>rupture</td>
</tr>
<tr>
<td>152</td>
<td>0.306</td>
<td>14.9</td>
<td>15.1</td>
<td>15.0</td>
<td>28.8</td>
<td>debonding</td>
</tr>
<tr>
<td>102</td>
<td>0.210</td>
<td>15.2</td>
<td>15.6</td>
<td>15.6</td>
<td>18.1</td>
<td>debonding</td>
</tr>
</tbody>
</table>
Failure by debonding occurred for both the 152 mm and 102 mm development length tests. A photo of this debonding failure is shown in Figure 4-24. Though most of the adhesive debonded from the steel surface and stayed bonded to the CFRP laminate strip, part of the adhesive and the strip remained bonded to the steel surface at one end causing a combination of splitting of the laminate strip from the debonded end to mid-span and inter-laminar shear failure within the strip. Adhesive pockets were also seen on the steel surface near the mid-span of the beam. Inter-laminar shear failure within the strip was not clearly seen in the other two beam tests.

4.3.8 Test Results of Sika Sikadur 30 Epoxy Adhesive

The 203 mm (8 inch) CFRP development length beam test was the only test completed using the Sika Sikadur 30 epoxy adhesive. The reason was due to the sudden debonding failure of the CFRP laminate strip from the tension flange of the test specimen causing stiffness and strength capacity values to be lower than most all other adhesives tested for this development length. Results in Table 4-15 show that the laminate strip failed at the strain of 0.281 percent. Had the CFRP laminate strip not debonded from the steel beam, stiffness over the unstrengthened control beam and the reserved capacity of 22.1 percent could have been higher.
4. EXPERIMENTAL RESULTS

The debonding failure of the Sika Sikadur 30 adhesive, shown in Figure 4-25, left a smooth surface on the steel. Almost all the adhesive remained bonded to the CFRP strip. Only a small fraction of the strip remained on the steel surface.

Table 4-15 One ply CFRP laminate strip strain at failure, stiffness, strengths, and failure modes for the Sika Sikadur 30 adhesive tests

<table>
<thead>
<tr>
<th>L₀ (mm)</th>
<th>Strain at Failure (%)</th>
<th>Stiffness Increase (%)</th>
<th>Service Capacity (%)-L/360</th>
<th>Service Capacity (%)-60% Fy</th>
<th>Reserved Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>203</td>
<td>0.281</td>
<td>14.0</td>
<td>14.3</td>
<td>14.2</td>
<td>22.1</td>
<td>debonding</td>
</tr>
</tbody>
</table>

4.3.9 Costs

Though not a part of the evaluation criteria for determining the most suitable adhesive for bonding the high modulus CFRP laminate strip material, costs of the adhesives was of importance to the manufacturer for practical application of the strengthening system in the industrial market. For the high modulus CFRP strengthening system to be a viable application and competitive in the field, the costs of adhesives should also be affordable. Thus, Table 4-16 was prepared to compare the costs of the top adhesives so that the manufacturer could make his final decision after tests were completed.

Figure 4-25 Failure of 203 mm CFRP laminate strip using Sika Sikadur 30
4. EXPERIMENTAL RESULTS

Table 4-16 2004 Costs comparison for adhesive bonding CFRP laminate strips to steel

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>Manufacturer</th>
<th>Cost ($$) / liter</th>
<th>Costs ($$$) / gallon</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jeffco 121</td>
<td>Jeffco Products</td>
<td>8.11</td>
<td>30.71</td>
</tr>
<tr>
<td>SS620</td>
<td>IPS Weld-On</td>
<td>15.27</td>
<td>57.81</td>
</tr>
<tr>
<td>Spabond 345</td>
<td>SP Systems</td>
<td>25.42</td>
<td>96.23</td>
</tr>
<tr>
<td>Araldite 2015</td>
<td>Vantico</td>
<td>31.40</td>
<td>118.85</td>
</tr>
<tr>
<td>Tyfo MB2</td>
<td>Fyfe</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sikadur 30</td>
<td>Sika</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

- Adhesive not considered

4.3.10 Summary

To fully utilize the high modulus CFRP laminate strip for the laminate strip strengthening system, the bonding adhesive must withstand strains close to the expected rupture strain of 0.330 percent measured for the laminate CFRP strip. This expected strain value was attained from several tension tests completed by the manufacturer. The adhesive must also endure high shear and normal stresses to provide good bond between the steel and the CFRP. The failure most desired is rupture of the CFRP material. This failure ensures the adhesive effectively transferred tensile forces within the tension flange of the steel beam to the bonded CFRP laminate strip causing the strip to carry extra load until its limit state of rupture.

Five of the six adhesives tested in the adhesive selection provided the bond that was necessary to cause the CFRP laminate strip to fail by rupture. The effectiveness of each adhesive depended on the length of each strip. Though comparisons of stiffness and strengths were made to the unstrengthened control specimen, the strain at failure within the CFRP material was the most important verdict. Strains measured at mid-span on the edge of the laminate strips were measurements that could be directly compared to other tests, while calculations of the stiffness and strengths were based on analysis and estimations. Values of
the stiffness and strengths were most useful in providing information that could be used for
design purposes and modeling.

The adhesive that was determined as the most suitable adhesive for bonding the high
modulus CFRP laminate strips to steel was SP Systems Spabond 345. After this adhesive
followed Weld-On SS620, Vantico Araldite 2015, and Jeffco 121 as other adhesives that
were competitive in performance. The Fyfe Tyfo MB2 and the Sika Sikadur 30 adhesives
were not very effective.

4.4 Phase 2(a) Development Length of CFRP Sheets

4.4.1 Introduction

The second phase of the experimental program was
the evaluation of the development length using the
most suitable resins for bonding the high modulus
CFRP material to steel. Based on the symmetric four
point bending test configuration, the development
length was defined as the distance from the loading
point to the edge of the CFRP sheet. Two resins
were selected from tests conducted in the CFRP sheet
resin selection phase; these resins were Degussa
MBrace Saturant and Sika Sikadur 330. Thicknesses
of one and two plys of the high modulus CFRP sheet
were considered for this study. Development lengths
ranging from 51 mm to 102 mm (2-4 inches)
were tested, as shown in Figure 4-26. After

Figure 4-26 Three varying development
lengths for the Sika Sikadure 330 resin
completion of the tests, a development length was determined for both CFRP sheet thickness applications.

The test specimen used for these experiments were small scaled wide flanged beams, called Super Light Beams, (SLB) that were used for the adhesive selection phase. The beams provided comparable behavior to that of large scale steel members, and thus was an effective specimen to study the lengths of the CFRP sheet needed to develop the high modulus carbon fiber material. Properties and measurements of these specimens were described in Section 3.5.1 of the Experimental Program.

Results of the development length tests using the high modulus CFRP sheets are discussed according to the thickness application of one and two plies of the carbon fiber material. Ultimate load and moment capacity, as well as stress and strain at failure were noted from each test. Stiffness and several strength increases over the unstrengthened control beam were also determined. The performances of all tests were characterized according to the notable results mentioned, as well as the failure modes for all tests.

4.4.2 One Ply CFRP Sheet Tests

Minimum development length was defined as the shortest development length that resulted in full utilization of the CFRP material. Under utilization of the CFRP material was typically characterized by low CFRP failure strains and often debonding of the CFRP. This general behavior was learned from the adhesive selection phase beam specimens. A practical development length of 51 mm was selected as the starting point for determining the development length required to achieve full utilization of the CFRP material. For the beams strengthened with using one ply or one layer of the CFRP sheet, two tests were first conducted.
The first two beams tested using the 51 mm development length were found to have achieved full utilization of the carbon fiber material using both resins. As a result, two additional tests were conducted to verify the results. These second tests had results almost identical to their first. Averages of these four tests are shown in Table 4-17. Results from all tests showed that the averaged failure strain of 0.349 percent for the CFRP sheets approached the dry fiber rupture strain of 0.40 percent as published by the manufacturer and matched closely with results completed for the CFRP laminate strip tests in the adhesive selection phase. Observed failure by rupture of the CFRP sheets showed that the CFRP was fully utilized as well within such a short development length. From literature reviews smaller development length did not deem practical for field applications, thus no further tests were completed to determine if a shorter development length would also result in rupture of the CFRP sheets.

<table>
<thead>
<tr>
<th>Resin</th>
<th>No. of Tests</th>
<th>Shear Stress at Failure (MPa)</th>
<th>Strain at Failure (%)</th>
<th>Ultimate Load (kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degussa MBrace Saturant</td>
<td>2</td>
<td>26.7</td>
<td>0.357</td>
<td>81.6</td>
<td>rupture</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td>2</td>
<td>11.4</td>
<td>0.341</td>
<td>84.9</td>
<td>rupture</td>
</tr>
</tbody>
</table>

* Note that underlined values are the average result of two tests

With successful test results from both saturating resins, the most noted difference was the bond shear stresses that were determined from strain measurements taken along the length of the beam. Further details of how these stresses were calculated are described in Section 4.6. The Degussa MBrace Saturant resin achieved stresses more than double that measured for the Sika Sikadur 330 resin, indicating that possibly better adhesion and saturation of the fibers was achieved. The Sika Sikadur 330 resin did have slightly higher loads before failure than the Degussa MBrace Saturant resin, though all beams failed at similar ultimate strains within the CRFP sheet. The Sika Sikadur 330 tests had more variability than the Degussa
4. EXPERIMENTAL RESULTS

MBrace Saturant tests, though this difference was less than 4 percent for load and 8 percent for strain.

The 51 mm development length also achieved high strength and stiffness in comparison to the unstrengthened control specimen. A plot of the applied load versus mid-span displacement for these tests is shown over the plot of the unstrengthened control specimen in Figure 4-27. The unstrengthened control specimen is shown as a thin line and is marked with an arrow. Performance of the strengthened beams is shown with thicker lines above the control line with failure of the CFRP sheet at the location of sudden load drop. Comparison of these strength increases at several locations along the load displacement curves was determined at selected design criteria points as described in Section 4.3.1. All three strength calculations, as well as the stiffness are given in Table 4-18 for this series of tests.

<table>
<thead>
<tr>
<th>Resin</th>
<th>No. of Tests</th>
<th>Stiffness (%)</th>
<th>Service Capacity (%)-L/360</th>
<th>Service Capacity (%)-60% Fy</th>
<th>Reserved Capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degussa MBrace Saturant</td>
<td>2</td>
<td>7.9</td>
<td>8.1</td>
<td>8.0</td>
<td>11.6</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td>2</td>
<td>10.6</td>
<td>11.0</td>
<td>11.0</td>
<td>15.9</td>
</tr>
</tbody>
</table>

*Note that underlined values are the average result of two tests*

The calculation of all strength values used in Table 4-18 were determined by comparing the applied load of the strengthened beams to the load of the unstrengthened control beam at the same mid-span deflection. Stiffness was calculated by comparing the elastic slopes of the applied load verses mid-span displacement curves for the strengthened beams to the elastic stiffness of the unstrengthened control beam. All values were determined for use as a guide to the behavior of the strengthened beams in comparison to its unstrengthened counterpart.
Figure 4-27 Applied load verses mid-span displacement for beams strengthened with one ply of CFRP sheet 51 mm in length using (MB) Degussa MBrace Saturant and (S) Sika Sikadur 330

Failure for all tests was by rupture of the carbon fibers followed by debonding from the tension flange surrounding the midspan region. Two photos shown in Figures 4-28 and 4-29 show this failure for both Degussa MBrace Saturant and Sika Sikadur 330 respectively. The photo in Figure 4-25 is taken after the beam was unloaded and placed onto a flat surface. Photo of the rupture of the CFRP sheet shown in Figure 4-29 was taken shortly after failure. All failures occurred at mid-span within the maximum moment region of the test specimens.
4.4.3 Two Ply CFRP Sheet Tests

Tests completed using two plies of CFRP sheets consisted of three different development lengths. The testing lengths were 102 mm, 76 mm, and 51 mm (4, 3, and 2 inches). Relationship of the thickness to the development length was assumed to be linear, thus the 102 mm development length was first tested since the development length for the one ply thickness was 51 mm. Test results for the 102 mm length showed that the CFRP sheets could fully be utilized when failure by rupture occurred at mid-span of the beams. Results for the 76 mm length and 51 mm length tests showed the sheets fail by debonding from the steel surface. The CFRP sheets were not fully utilized for development lengths less than 102 mm.

Strain at failure, ultimate load, and failure modes were recorded for all development length tests, as given in Table 4-19. Strength and stiffness increases in comparison to the
unstrengthened control beam were also determined. These values are shown in Table 4-20. Bond stresses and stiffness were higher than the one ply applications for this series of tests as anticipated, while similar strengths and ultimate load were achieved.

Table 4-19 Two ply CFRP sheet ultimate shear, strain, and load

<table>
<thead>
<tr>
<th>Resin</th>
<th>L-D (mm)</th>
<th>Shear Stress at Failure (MPa)</th>
<th>Strain at Failure (%)</th>
<th>Ultimate Load (kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degussa MBrace Saturant</td>
<td>51</td>
<td>-</td>
<td>0.258</td>
<td>76.8</td>
<td>debonding</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>21.8</td>
<td>0.204</td>
<td>65.3</td>
<td>debonding</td>
</tr>
<tr>
<td></td>
<td>102</td>
<td>33.2</td>
<td>0.324</td>
<td>87.2</td>
<td>rupture</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td>51</td>
<td>-</td>
<td>0.233</td>
<td>74.0</td>
<td>debonding</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>20.6</td>
<td>0.284</td>
<td>77.1</td>
<td>rupture</td>
</tr>
<tr>
<td></td>
<td>102</td>
<td>28.0</td>
<td>0.345</td>
<td>96.6</td>
<td>rupture</td>
</tr>
</tbody>
</table>

Table 4-20 Two ply CFRP sheet stiffness and strengths

<table>
<thead>
<tr>
<th>Resin</th>
<th>L-D (mm)</th>
<th>Stiffness (%)</th>
<th>Service Capacity (%)-L/360</th>
<th>Service Capacity (%)-60% Fy</th>
<th>Reserved Capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degussa MBrace Saturant</td>
<td>51</td>
<td>7.8</td>
<td>8.5</td>
<td>8.4</td>
<td>10.1</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>14.5</td>
<td>14.8</td>
<td>14.6</td>
<td>15.2</td>
</tr>
<tr>
<td></td>
<td>102</td>
<td>15.3</td>
<td>15.6</td>
<td>15.5</td>
<td>19.6</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td>51</td>
<td>7.9</td>
<td>8.7</td>
<td>8.7</td>
<td>12.8</td>
</tr>
<tr>
<td></td>
<td>76</td>
<td>11.8</td>
<td>12.6</td>
<td>12.5</td>
<td>13.4</td>
</tr>
<tr>
<td></td>
<td>102</td>
<td>13.1</td>
<td>14.0</td>
<td>14.0</td>
<td>16.1</td>
</tr>
</tbody>
</table>

The development length of 102 mm was the best performing of the lengths tested. Failure by rupture for both resins occurred only for this length, and the failure strain was higher than the other CFRP sheet development lengths. Stiffness and strength gains were also the largest from all tests at this development length.
The relationship between the thickness of the CFRP sheets and the development length appeared to follow the linear assumption prior to testing. The 0.19 mm effective thickness of the one layer of CFRP sheet had a development length of 51 mm (2 inches), while the development length of 102 mm (4 inches) for the two ply 0.38 mm effective thickness CFRP sheets was the best performing length tested from the beams strengthened with two ply CFRP sheet tests. This can be clearly seen from Figure 4-30, where the results of the tests completed using two plies of the CFRP sheets are plotted with their average CFRP rupture strain of 0.340 percent. A solid marker indicates tests in which the CFRP sheet ruptured; an unfilled marker indicates tests that resulted in debonding. The average rupture strain was determined from the tests that resulted in rupture of the CFRP sheets.

![Figure 4-30 Maximum CFRP strains for beams strengthened with two plies of CFRP sheets using different development lengths](image)

*Figure 4-30 Maximum CFRP strains for beams strengthened with two plies of CFRP sheets using different development lengths*
Failures of the sheets were a combination of rupture and debonding from the steel surface. These failures depended upon the resin and development lengths. Rupture of the carbon fiber sheet material was the cause of failure for the 102 mm development length for both the Degussa MBrace Saturant, shown in Figure 4-31, and also for Sika Sikadur 330.

For the beams strengthened with the 76 mm development lengths, only the Sika Sikadure 330 resin failed by rupture. Figure 4-32 shows how this rupture failure resulted in the splitting of the fiber along its length from midspan with only a small portion remaining bonded. Sudden debonding after rupture of the fiber was a common observance in all tests that failed by rupture.

Debonding failure both by the Sika Sikadur 330 resin and the Degussa MBrace Saturant resin occurred for the 51 mm (2 in) development lengths. The debonding failures of these tests did not split the CFRP sheet into an unbonded and...
bonded strip, but instead came off the steel surface as one piece from the end of the CFRP.

4.4.4 Summary

The development lengths for the one and two ply thicknesses of the high modulus CFRP sheets were 51 mm (2 inches) and 102 mm (4 inches) respectively. Both resins failed by rupture of the sheets at these lengths, thus utilizing the material effectively. A short number of tests were conducted to save time and costs, though enough information was gathered for a useful understanding of the behavior of the CFRP sheet material at these tested development lengths.

Additional information that was useful from these tests was the linear relationship between thickness of the CFRP material and development length for this application. Test results suggest that a linear relationship existed, though this relationship has not been tested for three or more ply thickness applications. Also noted was the resin performance of the two tested resins used for the two ply thickness application. Degussa MBrace Saturant resin appeared to achieve small improvements in stiffness and strength increases from the beams tested with two layers of the unidirectional CFRP fiber sheet over the beams strengthened with only one layer, while the Sika Sikadur 330 resin achieved higher ultimate loads in almost every test and experienced similar stiffness and strengths for both one and two layers CFRP sheet applications. Large scale member tests can use the test results from these findings as a guide for designing the CFRP sheet application and length needed to develop desired strength and stiffness.

Due to the limited number of shear stresses determined from these beam tests, no correlation could be determined between the maximum shear stress and the failure mode. This was likely due to the nature of the critical bond stresses, which are highly localized near the end of the CFRP strip (Garden et. al, 1998) and the fact that the calculated shear stresses were
limited to an average stress between two locations on the CFRP strip. For the case of the Degussa MBrace Saturant resin, the critical bond stress could be said to be no lower than 21.8 MPa, since this stress resulted in debonding. For the Sika Sikadur 330 resin, the critical bond stress may be at least 28.0 MPa. However, since the CFRP sheets ruptured at this stress, higher stresses could have been reached had this failure mode not occurred. Further details are discussed in Section 4.6.

**4.5 Phase 2(b) Development Length of CFRP Strips**

**4.5.1 Introduction**

Based on the results of the adhesive selection, validation testing was continued for the development of the CFRP laminate strip lengths. This testing included new and some repeated development length tests using the same beam specimens and CFRP laminate strips that were used in the adhesive selection. Three adhesives were selected for this continued testing; those adhesives were SP Systems Spabond 345, Weld-On SS620 and Jeffco 121. The SP Systems Spabond 345 adhesive and the Weld-On SS6720 adhesive were selected based upon consistent performance in the adhesive selection; while the Jeffco 121 adhesive was added because of its performance and low cost.

Like the development length tests conducted for the CFRP sheets, two plies, or two layers of the CFRP laminate strips were incorporated into the testing program to study the relationship of thickness of the CFRP material and development length. A proposed hypothesis of a linear relationship was tested and found true for the CFRP sheet thickness, thus this linear assumption was thought to also apply to the CFRP laminate strips. Therefore, testing of the development length was divided into one and two ply thickness of CFRP laminate strips.
4. EXPERIMENTAL RESULTS

4.5.2 One Ply CFRP Laminate Strip Tests

Because development length was incorporated into the adhesive selection phase, the number of tests needed for determining development length for one layer of CFRP strips was reduced to five tests. Test results from the adhesive selection provided enough information such that additional tests could complete the phase without retesting a new series of beam tests. In every test series for each adhesive in the adhesive selection, certain development lengths caused the laminate strip to fail by debonding from the steel surface. It was at these development lengths that further evaluation continued with additional testing. In this phase, the tests served the purpose of verifying those test results from the adhesive selection phase as well as determining the bond stresses within the adhesive. This was accomplished through the addition of extra strain gauge measurements taken along the bottom surface of the bonded CFRP strip.

Test results for both the verification development length tests along with the adhesive selection tests of four of the adhesives with the shortest development lengths are shown in Figure 4-33. In this figure, the failure strain of the CFRP strip is plotted against the development length for each adhesive. Solid markers indicate a failure by rupture of the CFRP strip, whereas unfilled markers indicate a failure of the CFRP strip by debonding. The average CFRP rupture strain of 0.300 percent is plotted as a solid horizontal line for comparison. From the graph, it can be seen that each adhesive achieved its highest strains within the CFRP material at a proposed development length.
4. EXPERIMENTAL RESULTS

Weld-On SS620 and SP Systems Spabond 345 adhesives were found to have the shortest development lengths of 76-102 mm. Both the Vantico Araldite 2015 adhesives and the Jeffco 121 adhesives had development lengths of 127-152 mm, while the Fyfe Tyfo MB and Sika Sikadur 30 adhesives, Not shown in Figure 4-32, had development lengths of 152 mm and more than 203 mm, respectively. Maximum strains within the CFRP strip for all tests were reached within the upper bound of their development lengths.

All five beams tested for the one ply application had additional strain gauges bonded to the CFRP strips that allowed determination of the adhesive shear stress. The test with the SP System Spabond 345 adhesive had the highest shear stresses of the tests with one ply of the CFRP strip. It is possible that some of the other adhesives could have developed higher shear stresses had the CFRP strips not ruptured first. With the exception of the Weld-On
SS620 adhesive tested at the 76 mm development length, these tests also showed that all adhesives achieve higher bond stresses, ultimate CFRP strains, and ultimate load capacities for the strengthened beams at the upper bound of their development lengths. Maximum adhesive shear stresses, CFRP strains at failure, and ultimate load capacities of the beams are given in Table 4-21.

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>L_D (mm)</th>
<th>Shear Stress (MPa)</th>
<th>Strain at Failure (%)</th>
<th>Ultimate Load (kN)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jeffco 121</td>
<td>102</td>
<td>15.0</td>
<td>0.244</td>
<td>90.7</td>
<td>debonding</td>
</tr>
<tr>
<td></td>
<td>127</td>
<td>21.0</td>
<td>0.266</td>
<td>92.5</td>
<td>rupture</td>
</tr>
<tr>
<td>SP Systems Spabond 345</td>
<td>76</td>
<td>-</td>
<td>0.243</td>
<td>80.3</td>
<td>debonding</td>
</tr>
<tr>
<td></td>
<td>102</td>
<td>36.2</td>
<td>0.311</td>
<td>94.8</td>
<td>rupture</td>
</tr>
<tr>
<td>Weld-On SS620</td>
<td>76</td>
<td>32.8</td>
<td>0.290</td>
<td>101.5</td>
<td>rupture</td>
</tr>
<tr>
<td></td>
<td>102</td>
<td>17.5</td>
<td>0.316</td>
<td>98.7</td>
<td>rupture</td>
</tr>
</tbody>
</table>

*Note that underlined values are the average result of two tests*

### 4.5.3 Two Ply CFRP Laminate Strip Tests

Two tests were conducted to determine the relationship between thickness of the CFRP laminate strip and development length. This relationship was assumed to be linear like that experienced for the CFRP sheets and therefore development lengths were designated to reflect this relationship for both beams. The decision of which adhesives to use in these tests was determined from the manufacturer. The two chosen adhesives were the Jeffco 121 adhesive and the SP Systems Spabond 345 adhesive. A development length of 254 mm (10 inches) was selected for the Jeffco 121 adhesive and a development length of 203 mm (8 inches) was selected for the SP Systems Spabond 345 adhesive. Although the development length for the one ply CFRP laminate strip bonded to steel with the Jeffco 121
adhesive was between 127-152 mm (5-6 inches), the spacing between the roller supports on the beam test setup limited the two ply laminate strip development length to 254 mm (10 inches). Results for both tests are given in Table 4-22 and shown in Figure 4-34.

### Table 4-22 Two ply development length test results

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>$L_0$ (mm)</th>
<th>Failure Strain (%)</th>
<th>Shear Stress (MPa)</th>
<th>Stiffness Increase (%)</th>
<th>Service Capacity (%)-L/360</th>
<th>Service Capacity (%)-60% $F_y$</th>
<th>Reserved Capacity (%)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP 345</td>
<td>203</td>
<td>0.309</td>
<td>60.9</td>
<td>30.6</td>
<td>31.9</td>
<td>31.6</td>
<td>48.2</td>
<td>rupture</td>
</tr>
<tr>
<td>J 121</td>
<td>254</td>
<td>0.202</td>
<td>49.1</td>
<td>32.1</td>
<td>32.2</td>
<td>32.1</td>
<td>41.1</td>
<td>debond</td>
</tr>
</tbody>
</table>

* $SP\ 345 = SP\ Systems\ Spabond\ 345$

* $J\ 121 = Jeffco\ 121$

![Figure 4-34 Load displacement response of both beams strengthened with two plys of the CFRP strips using different adhesives and development lengths](image-url)

**Figure 4-34** Load displacement response of both beams strengthened with two plys of the CFRP strips using different adhesives and development lengths
Significant stiffness and strength gains were achieved with the two ply application of the laminate strips. Stiffness increases in comparison to the unstrengthened control beam were over 30 percent, while strengths of 32 percent and up were achieved in service and at ultimate loads. Failure of the SP Systems Spabond 345 adhesive resulted in rupture of the laminate strip; this failure supports the linear relationship between CFRP laminate thickness and development length. Failure of the Jeffco 121 adhesive resulted in debonding of the strip from the steel; this failure does not fully support the assumed relationship between CFRP laminate thickness and development length. Assuming the relationship between thickness of the CFRP material and development length is true, the 127 mm (5 inch) development length of the one ply thickness would not make the best use of the CFRP laminate strip, as higher strains were achieved for the 152 mm (6 inch) development length causing rupture of the strip during the adhesive selection phase. As determined from the adhesive selection, the upper bound of all proposed development lengths for each adhesive achieved the highest strains in the bonded CFRP. Further testing by bonding longer sections of the double ply CFRP strip using Jeffco 121 adhesive could not be tested due to the existing lengths of the beams.

Full utilization of both CFRP strips by rupture for the beam strengthened with two plys using the SP Systems Spabond 345 adhesive occurred at the same location at midspan of the beam. A sudden debonding surround midspan immediately followed. This is shown in Figure 4-35 and explains the sudden drop in load shown in Figure 4-34. The debonding failure exhibited by the Jeffco 121 adhesive resulted in exceeding the shear strength of the adhesive. This is shown in Figure 4-36 by the clean breakage of the adhesive at the CFRP end and the visible tears within the excess sealant exposed along the length of the strip.
4. EXPERIMENTAL RESULTS

Figure 4-35 Rupture failure of the double ply CFRP strip using SP Systems Spabond 345 adhesive at 203 mm development length

Figure 4-36 Debonding failure of the double ply CFRP strip using Jeffco 121 adhesive at 254 mm development length
4. EXPERIMENTAL RESULTS

### 4.5.4 Summary

One ply and two ply thicknesses high modulus CFRP laminate strips were tested to determine the development length needed to fully maximize the use of the material. For the one ply thickness CFRP laminate strip, a development length closer to 102 mm (4 inches) was determined for the SP Systems Spabond 345 and Weld-On SS620 adhesives and 127-152 mm (5-6 inches) for the Vantico Araldite and Jeffco 121 adhesives. The Fyfe Tyfo MB adhesive achieved highest CFRP strains at the 152 mm development length, while the development length for the Sika Sikadur 30 adhesive was more than 203 mm. Maximum strains within the CFRP strip and typically adhesive shear stresses for all tests were reached within the upper bound of their development lengths. A prudent approach for field applications would be to use a development length of 152 mm (6 inches), since five out of the six adhesives tested achieved full utilization of the CFRP material at this development length.

The two ply thickness tests indicated a linear relationship between the thicknesses of the laminate strip to the development length, though a direct correlation was still undetermined. Significant strength and stiffness enhancement to the structural member was achieved with the two ply thickness application.

### 4.6 Bond Behavior

#### 4.6.1 Introduction

Several beam specimens were instrumented with additional electric resistance foil strain gauges that were bonded to the CFRP surface along one half of the CFRP length from the mid-span of the beam. These additional strain gauges were used to determine the strain profile of the CFRP material and the shear stress profile of the bonding adhesive during
loading of the beam. These strain gauge locations along the beam specimens are shown in Figure 3-16 in Chapter 3.

The purpose of generating the strain and shear stress profiles of the CFRP material and bonding adhesive was to understand the bond behavior of the CFRP material to steel. Measured strain profiles revealed that strains were typically highest at the mid-span of the beam specimens and reduced towards the ends of the CFRP material lengths. This behavior was expected since the highest applied moment was located at mid-span of each beam specimen. Also, the strain profiles revealed the location of the failure mechanism of the tested beam specimens. Most CFRP material failures were by rupture of the carbon fibers at its ultimate limiting strain for the CFRP applications; maximum strain occurred at mid-span for everyone of these failure types. Beams that experienced debonding failures of the CFRP material revealed a changeable location of maximum strain. Once failure occurred at a specific location along the beam, bonding stresses were shifted to other remaining bonded sections of the CFRP material, thus causing a change in the maximum strain achieved by the CFRP material with continued applied load. The changes in location of maximum measured strains are not directly pointed out in the strain profiles yet are easily noticed in the shift of peak strain within the plot.

Shear stresses of the tested adhesives and resins were determined from the strains measured at the strain gauge locations. The stresses were used to determine a maximum adhesive stress that could be used for design purposes. The shear stress profiles that were determined from the measured strain showed the trend as reported by Garden et. al. (1998). This trend was that shear stresses were maximum at the CFRP material ends and lowest at mid-span of the CFRP material lengths. These higher shear stresses at the ends of the CFRP material were initiated by peeling forces that were caused by the applied loading configuration placed on the beams. Maximum applied load and moment were located at mid-span of the beam specimens; this forced the CFRP material to carry much of the applied load of the beam at
the mid-span location. Forces within the CFRP material were induced by forces that were transferred from the steel beam to the CFRP material through the bonding adhesive layer. As load was continually applied during the tests, forces in the CFRP material applied peeling forces on the adhesive, which caused shear stresses to develop within the adhesive layer. These shear stresses were greatest at the ends of the CFRP material because of the transition from a bonded region of the beam to an unbonded section. Without additional area of bonded adhesive to help distribute the peeling forces, the shear strength capacity of the adhesive was limited at this adhesive layer transition. Once the forces required by the CFRP material to support the applied load reached the limiting ultimate shear strength capacity of the adhesive at the CFRP material ends, the CFRP debonded from the steel.

Both the strains profiles of the CFRP material and the shear stress profiles of the adhesives were plotted for all beams tested with the additional strain gauges. The plots are shown as data points connected with a line for typical 20, 40, 60, 80, and 100 percent of the ultimate load at failure of the beam. Several plots contained data shown at other percentages of the ultimate applied load for clarification; these extra lines are shown for clearer determination of the behavior of the bond stresses before failure.

Complete understanding of the bond stresses and failure mechanisms of the CFRP strengthening application require discussion from each beam tested with the additional strain gauges. Select beams were tested to represent typical bond behavior. Since different adhesives and resins were used, distinguishing results between the adhesives and CFRP applications was desired by the carbon fiber manufacturer. Thus, the strain and shear stress profiles for the CFRP unidirectional fiber sheet will be discussed first, followed by the CFRP laminate strip application.
4.6.2 Measured Strain and Shear Stress for CFRP Fiber Sheets

Two of the beams strengthened with one ply of the CFRP sheets and four of the beams strengthened two plys of the CFRP sheets were instrumented with strain gauges along the development length of the sheet on one side of the beam. Strain measurements were recorded at these discrete locations along the length of the bonded CFRP strip. The difference in tensile strain between two gauge locations must be balanced by the shear force acting between the CFRP plate and the steel substrate, as noted by Garden et al. (1998). The average shear stress could then determined between the two gauge locations as,

$$\tau_{av} = \frac{E_{frp} t_{frp}}{x_2 - x_1} (\varepsilon_2 - \varepsilon_1)$$

where $\varepsilon_2 - \varepsilon_1$ is the difference in strain between two adjacent gauges and $x_2 - x_1$ is the distance between the gauges. The longitudinal strain at the tip of the CFRP sheet was taken to be zero in order to calculate the shear stress between the end of the strip and the location of the first strain gauge. For the tests with the CFRP sheets, the modulus was taken as that of the fibers or 640 GPa, together with the effective thickness of the CFRP sheets of 0.19 mm. The maximum shear stress before rupture or debonding was determined for each of these beams. The results are listed in Table 4-23.
4. EXPERIMENTAL RESULTS

Table 4-23 Maximum shear stress (MPa) and failure mode for beams strengthened by wet lay-up of CFRP sheets using different development lengths

<table>
<thead>
<tr>
<th>Resin</th>
<th>No. of plys</th>
<th>Development Length</th>
<th>102 mm</th>
<th>76 mm</th>
<th>51 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degussa MBrace Saturant</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>26.7 rupture</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>33.2 rupture</td>
<td>21.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sika Sikadur 330</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>11.4 rupture</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>28.0 rupture</td>
<td>20.6</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Most of the strain profiles (plotted as percentages of ultimate load for the test beam) along the length of the CFRP sheets were typical as shown in Figure 4-37 showing strains increasing under applied load highest at the midspan of the beam and lowest at the edge of the CFRP material. If failure by rupture of the CFRP occurs, the strain profiles typically follow this anticipated profile until sudden failure occurs with an immediate drop in strain measurement at mid-span. Debonding failures have been observed by a shift in the strain profile once the adhesive no longer provides compaction of the CFRP material to the steel surface of the beam. An example of this can by seen in Figure 4-38. This can occur at any load where the bond peeling stresses of the CFRP material at the end of the sheets exceed the allowable shear strength of the adhesive before utilization of the CFRP by rupture. Additional load can still be carried by the beam as other sections of the CFRP remain composite with the steel until the adhesive stresses in those sections are exceeded.
Figure 4-37 Measured strain profile along one ply CFRP sheet with development length of 51 mm using Degussa MBraze Saturant resin

Figure 4-38 Measured strain profile along two ply CFRP sheet with development length of 76 mm using Degussa MBraze Saturant resin
Most of the shear stress profiles along the length of the CFRP sheets were similar to those established by Garden et. al. (1998), where the highest stresses are located at the tip end of the CFRP and lowest at the center of the beam. These profiles generally show the shear stresses exponentially high at the end of the bonded CFRP material, as shown in Figure 4-39 for the one ply CFRP sheet beam test using the Degussa MBrace Saturant resin at the 51 mm development length. Test specimens that achieve this shear stress profile have good compaction for almost the entire duration of the test until failure by rupture of the CFRP or debonding of the adhesive occurs. This also indicates that the resin had good saturation of the fibers within the CFRP sheets. Figure 4-40 is a plot of the average adhesive shear stress for this same test specimen showing an almost perfect linear stress transfer to the bonded CFRP for approximately 84 percent of the ultimate load before showing any nonlinear behavior approaching the rupture strain of the CFRP material.

![Shear stress profile for beam strengthened with one ply of CFRP sheets using Degussa MBrace Saturant resin and 51 mm development length at different load stages](image)

*Figure 4-39 Shear stress profile for beam strengthened with one ply of CFRP sheets using Degussa MBrace Saturant resin and 51 mm development length at different load stages*
Figure 4-40 Average shear stress at different distances from midspan to the end of the CFRP sheet for the test using Degussa MBraze Saturant resin and 51 mm development length

However, for the same one ply CFRP sheet beam strengthening using the Sika Sikadur 330 resin, shown in Figure 4-41, the location of the maximum shear stress can shift away from the end of the strengthening material as the load level increases. In this case, the maximum shear stress continued to shift towards the center of the beam as the adhesive slightly separated from the CFRP end. Load transfer to the closest bonded section is clearly observed as the shear stress taken from the next strain gauge measurement (shown as (c) in the Figure) begins to increase at a much faster rate under applied load. The failure by rupture of the CFRP sheet is noticed by the abrupt decrease of the stress values at ultimate load. The shear stresses immediately after rupture of the fiber were not shown in both the previous figures due to the observed sudden drop in strain measurements.
Shear stress and strain behavior for beams strengthened with the two ply CFRP sheet applications followed the same bond behaviors observed for the one ply CFRP sheet applications and at a greater level for both the 76 mm and the 102 mm development lengths tested. This was observed for the Degussa MBrace Saturant resin in Figures 4-42 and 4-43 on the following page. An almost linear relationship between the adhesive shear stress at the end of the CFRP sheets to the applied load can be seen in the shear profiles. The same debonding behavior before rupture of the CFRP was also observed using the Sika Sikadur 330 resin for both the 76 mm and the 102 mm development lengths. The difference between the one and two ply applications was that the maximum shear stress continued to shift towards the center of the beam at a much faster rate as debonding progressed from the end of the strip. This is shown clearly in the stress profile in Figure 4-44 for the beam strengthened with two plys of the CFRP sheets at a 102 mm development length. Increased shear stresses
in this profile were due not only to the increase in load being applied to the beam, but also to the reduction in the bonded area resulting from the progression of debonding.

**Figure 4-42** Shear stress profile for beam strengthened with two ply of CFRP sheets using Degussa MBrace Saturant resin and 102 mm development length at different load stages

**Figure 4-43** Average shear stress at different distances from midspan to the end of the two ply CFRP sheets for the test using MBrace Saturant resin and 102 mm development length
Maximum shear stresses for all beams tested with the Desgussa MBrace Saturant have shown higher than the maximum shear stresses for the Sika Sikadur 330 resin as given previously in Table 4-23. All of these maximum stresses occurred at the end of the CFRP strengthening material, as shown by the thick line in Figure 4-45 for the two ply 102 mm development length beam tests. High shear stresses, especially at the end edge of the CFRP sheet, match the profiles established from previous testing by Garden et. al. (1998) and appear to result in rupture of the CFRP sheet(s), with the exception of the two ply application at the development length of 76 mm for Degussa MBrace Saturant. High shear stresses away from the CFRP end indicates that the allowable adhesive shear stress is exceeded directly at the CFRP ends resulting in the transfer of the beam loads closer towards the center of the beam. For all beams using the Sika Sikadur 330 resin, this behavior was consistent before rupture of the fibers occurred. Due to the small number of tests completed using the multiple strain gauges for determining bond stresses, no direct correlation could be established relating failure modes with bond behavior for these two saturating resins.
4. EXPERIMENTAL RESULTS

Figure 4-45 Maximum shear stress profiles for two beams strengthened with two plys of CFRP sheets and 102 mm development length for different resins

4.6.3 Measured Strain and Shear Stress for CFRP Laminate Strips

Similar to the tests using the CFRP sheets, six beams using the CFRP strips had additional strain gauges bonded to the CFRP strips that allowed determination of the adhesive shear stress. The test with the SP Systems Spabond 345 adhesive had the highest shear stresses of the tests with one ply of CFRP strips. Although it is possible that some of the other adhesives from the adhesive selection phase could have developed higher shear stresses had the CFRP strips not ruptured first. This may not be the case for all adhesives, as shown from the low shear stress of the Weld-On SS620 adhesive at the same development length as the SP Systems Spabond 345 adhesive. This is given in Table 4-24. Likewise, shear stresses at the edge of the CFRP strip could have been higher had strain gauge measurements been located closer to the strip edge instead being at 25.4 mm from the tip. The configuration used for the strain gauges did not capture the shear stresses at the tip of the CFRP, therefore it was not adopted for other test specimens.
4. EXPERIMENTAL RESULTS

Table 4-24 Maximum shear stress (MPa) and failure mode for beams strengthened by adhesive bonding of CFRP strips using different development lengths

<table>
<thead>
<tr>
<th>Resin</th>
<th>No. of plys</th>
<th>Development Length</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>254 mm</td>
</tr>
<tr>
<td>Weld-On SS620</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>rupture</td>
</tr>
<tr>
<td>SP Spabond 345</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>rupture</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jeffco 121</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>49.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* this average shear stress was determined over the last 25.4 mm of the CFRP strip, unlike the remaining values that were determined over the last 6.4 mm

The shear stresses of the adhesives are more clearly observed within their prospective profiles for the one ply adhesives. Equilibrium of forces require that all of the longitudinal stress in the CFRP strip must be fully transmitted to the beam through the interface within the adhesive joint in order for the beam to fully utilize the strengthening material. Because the cross-sectional area of the CFRP strips was kept constant for each of the tests using one ply of strips, the total adhesive shear stress should be equal in each case. However, the shear stress profile varied for each of the adhesives. In Figure 4-46, the average shear stress at several locations along the length of the CFRP strip for the beam using Jeffco 121 adhesive and a 102 mm development length show that the shear stresses are highest at the end, as would be expected, noting that for this case the average shear stress was determined over the last 25 mm of the CFRP strip. For comparison, the shear stress profile for a beam with the same strengthening configuration, but using the SP Spabond 345 adhesive, is shown in Figure 4-47. Although the magnitude of the shear stress is higher at the end, it should be taken into account that the stresses quickly diminish away from the end of the strip right before rupture of the material. The Weld-On S620 adhesive has this same quick diminish, shown in Figure 4-48, even though the shear stress is not constantly increasing under the
applied load. This indicates that shear transfer that is expected within the adhesive does not provide direct composite action between the steel surface and the applied CFRP. Possibilities of this occurrence are described in Section 4.7.

![Figure 4-46 Average shear stress at different distances from midspan to the end of the CFRP strip for the test using Jeffco 121 adhesive and 102 mm development length](image-url)

*Figure 4-46 Average shear stress at different distances from midspan to the end of the CFRP strip for the test using Jeffco 121 adhesive and 102 mm development length*
4. EXPERIMENTAL RESULTS

Figure 4-47 Average shear stress at different distances from midspan to the end of the CFRP strip for the test using SP Spabond 345 adhesive and 102 mm development length.

Figure 4-48 Average shear stress at different distances from midspan to the end of the CFRP strip for the test using Weld-On SS620 adhesive and 102 mm development length.
Comparison of the shear stress profile for all three beams at the same applied load of 74 kN are shown in Figure 4-49. This load corresponds to the maximum induced shear stress in the beam using the SP Spabond 345 adhesive. The average shear stress between each pair of strain gauges is plotted at the midpoint between each two gauges. Although the force in the CFRP strip is approximately equal for all beams, the beams using the Spabond 345 adhesive and the Weld-On SS620 adhesive shows a more developed profile. For the Spabond 345 adhesive, the high shear stress would not have been captured had strain gauges not been placed very close together in this region. It is likely that since the last shear stress interval was only 6.4 mm, compared to 25.4 mm of that used for the Jeffco 121 adhesive, this test and the one used for the Weld-On SWS620 adhesive better captured the true strain profile of the adhesive.

Figure 4-49 Shear stress profile comparisons for three beams strengthened with one ply of CFRP strip and 102 mm development length for different resins
The nonlinear behavior of the average shear stress for the Weld-On SS620 adhesive at 102 mm development length was exasperated when a shorter development length was tested at 76 mm. This is shown in Figure 4-50 and in the stress profiles comparing these two development lengths in Figure 4-51 and their max stress profiles in Figure 4-52. The profile of the shear stresses within the adhesive show that stresses can occur at localized sections of the bonded material, causing significant stresses and shifting of the applied load on the beam to the CFRP material. These stresses appear to increase and shift as the development length becomes more critical. The selection of not using the Weld-On SS620 for further study was partially based upon this observed nonlinear behavior of the adhesive.

Figure 4-50 Average shear stress at different distances from midspan to the end of the CFRP strip for the test using Weld-On SS620 adhesive and 76 mm development length
4. EXPERIMENTAL RESULTS

Figure 4-51 Shear stress profiles for beams strengthened with one ply of CFRP strip using Weld-On SS620 adhesive and different development lengths

Figure 4-52 Maximum shear stress profile comparisons for beams strengthened with one ply of CFRP strip using Weld-On SS620 adhesive and different development lengths
Shear profiles for the double ply applications of the CFRP strips resulted in extremely high shear stresses at the ends of the material. This is shown in Figures 4-53 and 4-54. These profiles were anticipated since the stiffness of the two ply CFRP strips is much greater than for the one ply. Due to such high stresses at the ends, the other stresses determined away were significantly lower. For a given adhesive strength, debonding failures could occur due to these types of shear profiles. This may have been part of the cause why the Jeffco 121 adhesive failed by debonding, due to the material potentially not being fully utilized because of the limited development length placed by the test configuration. Rupture of the CFRP for the Spabond 345 adhesive occurred when the ultimate strains within the strengthening materials were reached before the adhesive strength was exceeded.

Figure 4-53 Average shear stress at different distances from midspan to the end of the two ply CFRP strip for the test using Jeffco 121 adhesive and 254 mm development length
In general for this study, strengthening applications using 2 plys of the CFRP sheets were found to provide similar increases of the ultimate load carrying capacity of the member as well as stiffness as one ply application of the CFRP strips at the same development length. Because of the similar end results, two stress profiles were considered for comparison to determine the differences in behavior between the two systems. Two strengthening systems that matched almost identically to each other upon review of the load displacement response were reviewed in Figure 4-55. The system using one ply of the CFRP strip used the Weld-On SS620 adhesive, while the two ply strengthening system used the saturant resin of Sika Sikadur 330. Upon review of their shear profiles, it can be seen that stresses are consistently highest near the ends of the bonded CFRP strip, increasing significantly under applied load. The shear profile of the two ply sheets at almost the same applied load revealed stresses changing from the fiber ends towards the center of the beam. This indicates that the allowable adhesive shear stress is exceeded directly at the CFRP ends resulting in the transfer
of the beam loads closer towards the center of the beam. This behavior was consistent before rupture of the fibers occurred for all beams using the Sika Sikadur 330 resin in this section.

![Shear stress profile comparison for beams strengthened with two ply of CFRP sheets using Sika Sikadur 330 resin and with one ply of CFRP strip using Weld-On SS620 adhesive at the same 102 mm development length](image)

**Figure 4-55** Shear stress profile comparison for beams strengthened with two ply of CFRP sheets using Sika Sikadur 330 resin and with one ply of CFRP strip using Weld-On SS620 adhesive at the same 102 mm development length

### 4.7 Shear Lag Effect

Shear lag effects of the adhesive bond layer were also investigated using the measured strain of the tested beams strengthened with the CFRP material. These diagrams were determined from strains measured at the mid-span cross section at three locations. These locations included the CFRP material, the top of the tension flange of the steel beam and the bottom of the compression flange of the steel beam. Plots of the strains at these locations were selected at specific loading percentages to determine the overall diagram.

Strain diagrams for unstrengthened beams typically exhibit strains that closely meet Bernoulli-Euler Beam Theory of plane sections remaining place. This design assumption is
used for strengthening steel members in regards to the compatibility of the adherends for complete transfer of applied stresses within the steel beam to the CFRP. For plane sections to remain plane, a perfect bond between the applied CFRP and the steel must exist.

The strain diagrams of the strengthened beams for this study typically followed this same linear profile for the steel beam, leaving only the measured strains at the CFRP layer resulting in small deformations from the expected linear interpolations. Two examples of the strain diagrams selected from the test series are shown in Figures 4-56 and 4-57. For both diagrams the strains measured at mid-span for the strengthened beam were plotted at 20, 40, 60, 80 and 100 percent of the applied ultimate load. The top compression steel plate strains were extrapolated linearly to complete the diagram; these strains are represented as dotted lines. The neutral axis remains constant until the ultimate load and failure of the CFRP material was reached.

For a majority of the strengthened beams in the test program, the strains measured at the bonded CFRP material were observed noticeably higher than if linearly extrapolated. A representative example is shown in Figure 4-56 for the beam strengthened with a development length of 152 mm using Vantico Araldite 2015 adhesive. The neutral axis remained at approximately 50 mm until the ultimate load was reached.

The strain profiles of other tests showed that the measured strains within the CFRP material were less than the linear interpolated values for the Bernoulli-Euler Beam Theory. A representative example is shown in Figure 4-57 for the beam strengthened with the Jeffco 121 adhesive at a 102 mm development length. Again, the neutral axis remained at approximately 50 mm until the ultimate load was reached.

From all of the beam tests, no relationships could be directly determined between the failure modes of the strengthened test beams to the small variances observed in these measured
CFRP strains. The variances appear to result only from the adhesive layer, where installation and thus bond of the CFRP material controls.

Figure 4-56 Strains at mid-span v. beam depth for specimen bonded with 152 mm development length using Vantico Araldite 2015
To consider the possibility of residual stresses within the steel as a possible cause of the slight strain differences between the CFRP and the steel, an additional control beam was tested with additional strain gauges at the mid-span cross section of the member. These strain gauges were located across the web and directly in line with each other at the bottom and tops of both the tension and compression flanges. The measurement of the strain at the top of the welded steel plate could then be compared with the linear interpolation assumed for the strengthened steel beams. Residual stresses were thought possible due to the small

Figure 4-57 Strains at mid-span v. beam depth for specimen bonded with 102 mm development length using Jeffco 121 adhesive
size and thickness of the member being hot-rolled into shape and as well as potential stresses developing from the stitch welding performed along the top of the compression flange.

The first of the remaining figures representing the test results of this unstrengthened control beam is the complete strain diagram of the entire member for the full duration of the load. A stop in the load was made where previous tests resulted in instability for the strengthened beams. This diagram is shown in Figure 4-58 below. From this diagram you can see how the strain in the top welded compression plate is definitely not linear along with the steel beam itself, putting in question the composite action of the welded plate to the steel beam.

![Figure 4-58 Strains at mid-span v. beam depth for unstrengthened beam until load at stop of test](image)
A closer look at the 0.2% yield region of this same diagram is presented in Figure 4-59. From this figure one can easily see the change in depth of the neutral axis of the steel as yielding of the tension flange occurs under applied load.

![Figure 4-59 Strain diagram for unstrengthened beam until load at stop of test: 0.2% Yield Region](image)

Additional strain gauges placed at the center and tip of the steel tension flange at mid-span helped to create a strain profile within the tension flange where the bond of the CFRP material is most important. This profile is shown in Figure 4-60. Strains recorded at certain load levels are listed in a table along with the figure. Strains across the tension flange were
measured until one of the strain gauges became unbonded under flexure of the beam at mid-span. Results show that strains indeed were not the same across the tension flange under the applied load.

**Strain Profile along Tension Flange: until 89.3 kN**

<table>
<thead>
<tr>
<th>Distance from end to midpoint (mm)</th>
<th>20%</th>
<th>40%</th>
<th>60%</th>
<th>80%</th>
<th>89.4 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>0.065</td>
<td>0.130</td>
<td>0.218</td>
<td>0.449</td>
<td>1.597</td>
</tr>
<tr>
<td>12</td>
<td>0.066</td>
<td>0.129</td>
<td>0.206</td>
<td>0.363</td>
<td>1.844</td>
</tr>
<tr>
<td>28.6</td>
<td>0.068</td>
<td>0.134</td>
<td>0.210</td>
<td>0.401</td>
<td>1.780</td>
</tr>
</tbody>
</table>

**Strain Profile on Tension Flange**

*Figure 4-60 Measured strain across the bottom half of the tension flange of the unstrengthened beam*
The differences in the measured strain at all three locations along the tension flange are very small. In fact, the strain gauge between the strain gauges measured at the center of the beam and the tip shows the only apparent difference among all three measurements. Since this particular strain gauge became unbonded near the end of the test, this indicates that the strain gauge between the other two gauges may not have been measuring strain properly. If this occurred, then there is little evidence from the strain measurements that there is a presence of residual stresses in the steel. Therefore, it is not likely that the steel beam affected the observed shear lag by the bonded CFRP material.

4.8 Nonlinear Strain Measurements

Nonlinear strain measurements were observed in the strain profile of several test beams. The most common was from the strain gauge measurements taken from the welded steel plate on top of the compression flange of each measured test beam. The other occurrence, given in Figure 4-58, was at the strain gauge placed on top of the tension flange of the control beam.

Both nonlinear strains are best observed from the strain profile of the control beam given in Figure 4-61. This profile included the compression flange as well as all of the measured strains along the tension flange. This plot was helpful to determine if the strains remained elastic until failure by yield of the steel. As the figure shows, both strains measured at the tops of the tension flange produced nonlinear behavior before yielding of the steel within the tension flange. Their profiles are highlighted by an arrow. In 2005, this behavior was verified by Dawood (2005) for the same test specimens. Strain gauges placed on tension flange resulted in nonlinear behavior right before yield of the steel. Potential causes for this behavior was accredited to one or a combination of possible lateral movement of the tension flange due to web side-sway buckling near the end of the load tests and/or local instability.
Figure 4-61 Applied Moment vs. Measured Strain at Mid-span of Control Beam

The nonlinear strain observed at the top of the steel plate welded to the compression flange of the steel beams is also highlighted with an arrow in Figure 4-61. This behavior indicates that the welded steel plate does not have complete composite action to the steel beam.

4.9 **WELDED STEEL PLATE EFFECT**

The steel plate at the top of the compression flange of the steel beam was designed to help raise the location of the neutral axis of the steel beam when strengthened with the CFRP material. This raising of the neutral axis higher within the beam helps ensure that the bonded CFRP material is utilized effectively. The neutral axis for the steel beam alone is measured at 50 mm due to symmetry of the cross section. If the steel beam alone was strengthened with only ply of the CFRP laminate strip only, the neutral axis depth would increase to approximately 67 mm. After the installation of the welded steel plate to the top of the beam, the neutral axis depth can be anticipated at 42 mm. This relationship is shown in Figure 4-62 on the following page.
From strain measurements, the neutral axis depths could be determined for all of the strengthened beams in the test program. Although an average depth of 42 mm was determined, the neutral axis depths of each test beam varied. This is clearly shown in Figures 4-63, 4-64, and 4-65 for replica tests on the test beams strengthened with one ply of the CFRP laminate strip at the 102 mm development length for the Weld-On SS620, SP Spabond 345, and Jeffco 121 adhesive, respectfully. For each figure, the neutral axis is compared with the unstrengthened control beam specimen. The sudden changes in the neutral axis depth occur at the rupture failure of the CFRP material.
4. EXPERIMENTAL RESULTS

Figure 4-63 Neutral axis depth of Weld-On SS620 test beams at 102 mm development length

Figure 4-64 Neutral axis depth of SP Spabond 345 test beams at 102 mm development length
In most cases, the measured neutral axis depth was close to the same measurement of the unstrengthened control beam with the welded steel plate alone. Therefore, the application of the CFRP material did not appear to have increased the neutral axis depth much at all. The neutral axis depth does however affect the measured strain at the extreme fiber of the bonded CFRP material, and thus affects the stresses and forces within the CFRP material to cause failure. Since the neutral axis depth varies for each beam test, the strain at failure of the CFRP may not always be the same at any given development length. The neutral axis depth does not however affect the failure mode of the CFRP material.

Based on these findings, the measured strains of the strengthened test beams could not be considered accurate, and thus stresses experienced within the bonded CFRP material. The test results may only be useful for relative comparison and as a indicator of how the selected adhesives would perform for larger scale field applications.
5. CONCLUSIONS

5.1 SATURANT RESIN SELECTION FOR THE WET LAY-UP OF CFRP DRY FIBER SHEETS

Ten different resins were evaluated as saturants for wet lay-up bonding of the unidirectional carbon fiber sheets to steel. The relative performance of these ten different saturating resins was compared through a series of double lap shear coupon tests. Resins that bonded the CFRP sheets to the steel the best were those whom achieved the highest average shear stress and failed the CFRP sheet by rupture. Two resins that consistently demonstrated the highest value among all resins when tested at room temperature and with the additional variables were Degussa MBrace Saturant and Sika Sikadur 330. An average shear stress of 12.3 MPa was achieved for these resins prior to rupture of the fibers.

The remaining resins did not appear to show any performance improvements within their shear strengths and failure modes with the presence of a wetting agent, additional bonding resin, or when heat curing methods were applied. Pull-out failures that were observed indicate that there may have been incomplete wetting of the fibers by the resin. Also, the poor performance of some of the resins was due to debonding of the CFRP from the steel surface. This was most evident for the three additional prepreg resins that were also tested with three bonding resins for application of the CFRP sheets by a prepreg process. The tests using the prepreg resins exhibited much lower bond strengths than for the best performing saturating resins, and were discontinued from further study.
5. CONCLUSIONS

5.2 ADHESIVE SELECTION FOR BONDING OF CFRP LAMINATE STRIPS

Six different adhesives were evaluated as bonding adhesives for adhering CFRP pultruded laminate strips to the steel surface. The relative performance of these different adhesives was compared through a series of small scaled steel beams loaded in flexure. Varying development lengths were used as a distinguishing method of their bond performance. The adhesive that achieved the highest ultimate strains within the CFRP material, resulting in failure by rupture within the shortest development length was considered the best adhesive for further study in large scale strengthening applications. Both the Weld-On SS620 adhesive and the SP Systems Spabond 345 adhesives were found to achieve highest strains by rupture of the CFRP strip at the shortest development lengths, while the remaining adhesives achieved their highest strains at other lengths. And although both adhesives averaged similar results for strength increases, the SP Systems Spabond 345 averaged stiffness increases better than its competitor and the overall average for all tests.

5.3 DEVELOPMENT LENGTH STUDY AND TWO PLY APPLICATIONS FOR BOTH CFRP MATERIALS

Performance of the 0.19 mm thick and 0.38 mm thick CFRP sheet applications tested for determining development length of the sheets showed an average reserved capacity in comparison to the unstrengthened control beam of almost 14 and 18 percent and stiffness increases of over 9 and 14 percent, respectively. Averages of the strain of the tested CFRP sheets were 0.349 percent for the one ply and 0.322 percent for the two ply sheets. These strains occurred at their prospective determined development lengths of 51 mm for the single ply wet lay-up applications and 102 mm for the application of the double ply wet lay-up process. Additional information that was useful from these tests was the linear relationship between thickness of the CFRP material and development length for this application. Test
results suggest that a linear relationship existed, though this relationship has not been tested for three or more ply thickness applications.

Like the development length tests conducted for the CFRP sheets, two plies, or two layers of the CFRP laminate strips were also incorporated into the testing program to study the relationship of thickness of the CFRP material and development length. From only two tests, results validated expected increases in stiffness and strengths of that more than double for the single ply averages. Stiffness increases of for the single ply of almost 14 percent and reserved capacity increases of 22 percent were determined. From the large test series conducted in the adhesion selection phase, development lengths for the single strip were determined to be a minimum of 102 mm, with a maximum achievement for most all of the adhesive at 152 mm.

5.4 BOND CHARACTERISTICS

Bond stresses can be determined from strain measurements taken very close to the edge of the bonded strengthening material, where stresses are considered the highest. Resistance by the adhesive against these load related stresses requires good surface preparation without the presence of any local defects. Peeling forces in the adhesive layer can cause stresses to shift closer to the center of the beam, resulting is a debonding failure mechanism. Failure by rupture of the CFRP material occurs when the ultimate strain of the material is reached, even though nonlinear adhesive properties exist.

5.5 SHEAR LAG EFFECT

Although, there were no direct correlations between failure modes of the strengthened beams and shear lag effects observed from the measured strains in the bonded CFRP, the shear lag effect appeared to be caused by the adhesive layer and not by the residual stresses in these
steel beam specimens. This could be possible if the thickness of the adhesive bond layer was not kept at a consistent thickness during application and curing.

5.6 **Nonlinear Strain Measurements**

Potential causes for the nonlinear behavior of the measured strains of the steel include possible lateral movement of the member itself due to web side-sway, which was a common failure mechanism for these beams when loaded past their plastic limit, or local instability of part of the test specimen.

5.7 **Welded Steel Plate Effect**

The welded steel plate was effective in raising the depth of the neutral axis of the strengthened beams very close to the measured neutral axis depth of the unstrengthened control beam. Due to the process of stitch welding the steel plate to the compression flange of the existing steel beam, the neutral axis depth of the beams strengthened with the CFRP material was not always the same. Variances in the neutral axis depth can affect the measured strains of the CFRP material and thus the stresses and forces experienced within the material to cause failure. Therefore, the strain at failure of the CFRP material may not always be the same at a given development length.

5.8 **Future Work**

Future research can be directed in several areas to develop more confidence in the use of high modulus CFRP and to gain a more detailed understanding of the bond behavior between the bonded strengthening system and the steel surface. This work is as follows:

- Determination of the durability of the bonded CFRP material must be evaluated. Deterioration of the bonded joint could occur due to galvanic corrosion or from
deterioration of the adhesive bond due to prolonged exposure to extreme temperatures and moisture. Both of these factors should be considered with different methods of improving the durability, such as silane promoters, should be investigated.

- Due to the differences in the coefficients of thermal expansion of steel and CFRP, temperature changes may induce significant stresses on the strengthened steel beam and or on the bond itself. The response of a strengthened beam to thermal loading should be investigated.

- Bond stresses have been shown both experimentally and analytically to be very localized at the ends of the CFRP material. Non-destructive test methods to evaluate the bond strength at these critical locations would be beneficial to confirm that the surface preparation conducted in the field allows sufficient bond stresses to develop.

- Although shear lag has been shown small for the tests conducted in the experimental program, very thick FRP materials or the use of a thicker adhesive layer could make these effects more noticeable. Although CFRP strips were used with up to a 4.0 mm thickness, several demonstration bridges have used CFRP strips that are up to 12 mm in thickness. It is unclear whether the findings of this research hold for thicker strips as well.
6 REFERENCES


6. REFERENCES


REFERENCES


REFERENCES


6. REFERENCES


6. REFERENCES


Figure 7-1 Test Results of Degussa MBrace Saturant Epoxy Resin

Figure 7-2 Test Results of Sika Sikadur 330 Epoxy Resin
7. APPENDIX

Figure 7-3 Test Results of 3M DP810 Acrylic Resin

Figure 7-4 Test Results of 3M DP460 Epoxy Resin
Figure 7-5 Test Results of SP Systems Ampreg 22 (fast hardener) Epoxy Resin

Figure 7-6 Test Results of SP Systems Ampreg 22 (slow hardener) Epoxy Resin
Figure 7-7 Test Results of Resinlab EP1246 Acrylic/Epoxy Resin

Figure 7-8 Test Results of Sika Sikadur 300 Epoxy Resin
Figure 7-9 Test Results of Jeffco 121 Epoxy Resin

Figure 7-10 Test Results of Reichhold Atprime 2 Urethane Resin
Figure 7-11 Test Results of Sika Sikadur 330 Epoxy Resin with Wetting Agent

Figure 7-12 Test Results of 3M DP460 Epoxy Resin with Wetting Agent
Figure 7-13 Test Results of 3M DP810 Acrylic Resin with Wetting Agent

Figure 7-14 Test Results of Jeffco 121 – Sika Sikadur 330 Hybrid Resin
Figure 7-15 Test Results of Degussa MBrace Primer – Saturant Hybrid Resin

Figure 7-16 Test Results of Sika Sikadur 330 – 300 Hybrid Resin
Figure 7-17 Test Results of Jeffco 121 – Sika Sikadur 300 Hybrid Resin

Figure 7-18 Test Results of Sika Sikadur 330 Heat Cured Epoxy Resin
Figure 7-19 Test Results of SP Systems Ampreg 22 Heat Cured Resin

Applied Load v Mid-span Displacement for 51 mm $L_D$

Figure 7-20 Load Displacement Response for 51 mm development length beam tests
Applied Load v Mid-span Displacement for 76 mm $L_D$

Figure 7-21 Load Displacement Response for 76 mm development length beam tests

Applied Load v Mid-span Displacement for 102 mm $L_D$

Figure 7-22 Load Displacement Response for 102 mm development length beam tests
Figure 7-23 Load Displacement Response for Weld-On SS620 adhesive replica beam tests

Figure 7-24 Load Displacement Response for SP Spabond 345 adhesive replica beam tests
Figure 7-25 Load Displacement Response for Jeffco 121 adhesive replica beam tests

Figure 7-26 Load Displacement Response for 127 mm development length beam tests
Figure 7-27 Load Displacement Response for 152 mm development length beam tests

Figure 7-28 Load Displacement Response for 203 mm development length beam tests
Neutral Axis Depth v Applied Load for 51 mm $L_D$

![Graph showing neutral axis depth versus applied load for 51 mm development length beam tests with different materials.]

*Figure 7-29 Neutral axis depth for 51 mm development length beam tests*

Neutral Axis Depth v Applied Load for 76 mm $L_D$

![Graph showing neutral axis depth versus applied load for 76 mm development length beam tests with different materials.]

*Figure 7-30 Neutral axis depth for 76 mm development length beam tests*
Figure 7-31 Neutral axis depth for 102 mm development length beam tests

Figure 7-32 Neutral axis depth for 127 mm development length beam tests
Neutral Axis Depth v Applied Load for 152 mm $L_D$

![Graph of Neutral Axis Depth vs Applied Load for 152 mm development length beam tests](image)

*Figure 7-33 Neutral axis depth for 152 mm development length beam tests*

Neutral Axis Depth v Applied Load for 203 mm $L_D$

![Graph of Neutral Axis Depth vs Applied Load for 203 mm development length beam tests](image)

*Figure 7-34 Neutral axis depth for 203 mm development length beam tests*