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FLEXURAL MECHANICAL DURABILITY OF CONCRETE BEAMS STRENGTHENED BY EXTERNALLY BONDED CARBON FIBER REINFORCED POLYMER SHEETS

by

MICHAEL ADAM OLKA
B.S. University of Central Florida, 2005

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in the Department of Civil and Environmental and Construction Engineering in the College of Engineering and Computer Science at the University of Central Florida Orlando, Florida

Spring Term 2009
ABSTRACT

About 77,600 bridges throughout the United States in the Federal Highway Association (FHWA) bridge database are listed as structurally deficient. This has created a need to either replace or strengthen bridges quickly and efficiently. Due to high costs for total replacement of deficient bridges, strengthening of existing bridges is a more economical alternative. A technique that has been developing over the past two decades is the strengthening of bridges using carbon fiber reinforced polymer (CFRP) sheets. The CFRP sheets are attached to the bottom of the bridge girders using structural adhesives so that the CFRP becomes an integral part of the bridge and carries a portion of the flexural loading. The CFRP sheets allow for an increase in the capacity of the bridge with minimal increase in the weight of the structure due to CFRP having a low density. Because the CFRP is expected to be an integral component and carry some of the long-term loading it is important to understand the long-term durability of the composite section.

This thesis is part of a larger project, in which the long-term durability of the CFRP composite on concrete beams is investigated experimentally. The CFRP strengthened beams are exposed to fatigue testing and thermal-humidity cycling followed by failure testing. The testing scheme for this experiment allows for the investigation of the individual effects of fatigue and thermal-humidity loading as well as to explore the effects from combined fatigue and thermal-humidity loading. The investigation of the combined effects is a unique aspect of this experiment that has not been performed in prior studies. Results indicate that a polyurethane-based adhesive could provide a more durable bond for the CFRP-concrete interface than possible with epoxy-based adhesives.
To my grandfather, David Franklin Tippit for all the lessons taught and the stories told throughout my life (RIP).
ACKNOWLEDGMENTS

- Thanks to Dr. Lei Zhao for all of the years of guidance, lessons, patience, and assistance. His help was invaluable in the pursuit of my education at UCF.

- Thanks to the Florida Department of Transportation for the funding and execution of the research that this thesis is based upon.

- Thanks to Dr. Kevin Mackie for becoming my advisor on such short notice.

- Thanks to the committee members for their time and input on refining this thesis.

- A special thanks to all of those who helped me at the UCF structures lab listed below in no particular order.
  
  - Patrick O’Connor, Chris O’Riordan-Adjah, Javier Perez, Kevin Francoforte, Jason Burkett, Melih Susoy, Jignesh Vyas, Ricardo Zaurin, Zach Haber, & Jun Xia

- Last but no least I would to thank my family and friends for their support and encouragement.
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<th>Description</th>
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<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>CFRP</td>
<td>Carbon Fiber Reinforced Polymers</td>
</tr>
<tr>
<td>DAQ</td>
<td>Data Acquisition</td>
</tr>
<tr>
<td>ENR</td>
<td>Engineering News Record</td>
</tr>
<tr>
<td>FHWA</td>
<td>Federal Highway Administration</td>
</tr>
<tr>
<td>FDOT</td>
<td>Florida Department of Transportation</td>
</tr>
<tr>
<td>FRP</td>
<td>Fiber Reinforced Polymers</td>
</tr>
<tr>
<td>GFRP</td>
<td>Glass Fiber Reinforced Polymers</td>
</tr>
<tr>
<td>I-95</td>
<td>Interstate 95</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Displacement Transducer</td>
</tr>
<tr>
<td>M-Φ</td>
<td>Moment-Curvature</td>
</tr>
<tr>
<td>NOAA</td>
<td>National Oceanic and Atmospheric Administration</td>
</tr>
<tr>
<td>RC</td>
<td>Reinforce Concrete</td>
</tr>
<tr>
<td>SR 192</td>
<td>State Route 192</td>
</tr>
<tr>
<td>SRC</td>
<td>Structural Research Center</td>
</tr>
<tr>
<td>UCF</td>
<td>University of Central Florida</td>
</tr>
<tr>
<td>US</td>
<td>United States</td>
</tr>
<tr>
<td>UTM</td>
<td>Universal Testing Machine</td>
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</table>
1. INTRODUCTION

In 2006 the Federal Highway Administration (FHWA) reported to Congress that 26.6% (FHWA, 2006) of the 594,101 bridges in the FHWA inventory are classified as functionally obsolete or structurally deficient. Of these 50.9% of them are classified as functionally obsolete meaning the bridges are structurally sound but can not meet the current traffic demands. However the remaining bridges are classified as structurally deficient which means the bridges are not capable of sustaining an increase in traffic loading. These deficient bridges are in need of either replacement or rehabilitation and strengthening.

Replacement of all the deficient bridges in the United States (US) is estimated at a cost of $140 billion in a press release from the American Association of State Highway and Transportation Officials (AASHTO) in July 2008 (AASHTO, 2008). Due to the rising cost of construction materials (ENR 7/25/07) the AASHTO estimate could increase over time. A less costly solution to replacement is rehabilitation and strengthening of these bridges. The strengthening and rehabilitation of existing bridges maintains or improves the capacity of the bridges without replacement of the bridge. This would provide a lower cost and rapidly deployed solution to fix this infrastructure problem.

This project investigates the durability of a method for rehabilitating and strengthening bridges that has been studied in the State of Florida in prior experiments. Since the early 1990’s the Florida Department of Transportation (FDOT) has been conducting studies on a method for rehabilitation and strengthening of bridges that utilizes Carbon Fiber Reinforced Polymers (CFRP). These studies have included the strengthening of two bridges; the Interstate 95 (I-95) bridge over Blue Heron Boulevard
in West Palm Beach and the State Route 192 (SR 192) Eastern Relief Bridge over the Indian River in Melbourne. Figure 1-1, Figure 1-2, and Figure 1-3 show the repairs to a concrete beam of the SR 192 Eastern Relief Bridge.

Figure 1-1: Concrete Removal

Figure 1-2: Coating New Reinforcement
Figure 1-3: Casting New Concrete

Figure 1-4 and Figure 1-5 show the pre-cast girders with CFRP on them from I-95 over Blue Heron Boulevard after they had been removed from the bridge. The girders had sustained severe damage during service and the whole bridge was replaced. Figure 1-6 shows the extent of damage to an exterior girder of the bridge as well as the steel plate girder that had to be installed in order to keep the bridge open to traffic.

Figure 1-4: CFRP Damaged on Bottom Flange
To further the FDOT investigation of utilizing CFRP to strengthen bridges the specimens in this experiment were subjected to fatigue testing as well as thermal-humidity cycling to simulate long-term field conditions to determine the durability of CFRP.
2. BACKGROUND

Due to the need to rehabilitate and strengthen bridges various methods are being researched and developed using different materials to determine the best possible combination for long-term rehabilitation and strengthening. One method of rehabilitation and strengthening bridges is to attach steel plates to the girders. This method utilizes mechanical fasteners or structural epoxies to affix the steel plates to the bottom of the girders to increase the tensile capacity. The use of steel plates for strengthening has been performed with successful results; however the installation of the plates presents logistical difficulties given the density of steel and that if a structural epoxy is used pressure must be maintained on the plates until the epoxy cures (Karbhari and Engineer 1996). Due to these shortcomings a lighter material that is rapidly installed would be beneficial for the strengthening and rehabilitation of the deficient bridges. A material that has been receiving more attention as a replacement for steel plates is Fiber Reinforced Polymers (FRP).

The use of FRP to strengthen and rehabilitate structures has been studied for the past two decades, with the focus on two types of FRP; Glass Fiber Reinforced Polymers (GFRP) and Carbon Fiber Reinforced Polymers (CFRP) (Bakis et al. 2002). The GFRP has demonstrated the ability to strengthen beams in flexure; however they are susceptible to acids and alkalis (Banthia et al. 1998) that weaken the GFRP systems and reduce their ability to strengthen the beams. It was demonstrated in one experiment that the environmental factors such as temperature and humidity reduce the overall performance for GFRP systems by as much as 45% (Karbhari and Engineer 1996). Therefore GFRP systems, while capable for short-term strengthening and rehabilitation are not the ideal
candidate as a long-term solution due to their susceptibility to chemical and environmental affects. On the other hand, CFRP are chemically inert and thus not as susceptible to acids, alkalis, ozone, or organic solvents (Banthia et al. 1998). In addition, the greatest reduction in capacity was 30% for the CFRP system due to humidity and temperature factors, which was less than that for the GFRP systems (Karbhari and Engineer 1996) thus making them a better alternative for rehabilitation and strengthening of bridges.

Due to CRFP being less susceptible to environmental conditions than GFRP, research has focused on the performance of CFRP. Research into the performance of CFRP-strengthened beams has shown that the ultimate capacity can be increased by 35% above that for an un-strengthened beam (O’Riordan-Adjah 2004). The ability of CFRP systems to increase the capacity for Reinforced Concrete (RC) beams was well documented through extensive experimentation; however, the increase in the ultimate capacity comes with a decrease in the ductility of the beam. After investigation of the strengthening ability of CFRP, previous research studies turned to the durability of the system. These studies investigated how different factors effected the time-dependant durability of the epoxy interface between the CFRP and concrete.

One of the factors studied was how the humidity of the surrounding environment would impact the CFRP system. The humidity experiments typically consisted of conditioning the beams in an environment of 100% humidity, which meant soaking the CFRP strengthened beams in large tanks of water for an extended period of time. The results from these tests revealed that the ultimate capacity for the strengthened systems decreased by as much as 30% from the unconditioned strengthened beams (Karbhari and
Engineer 1996). It should be noted though that in a similar experiment (Grace and Singh 2005) the conditioned CFRP strengthened beams exposed for 10,000 hours experienced only a nine percent decrease in performance. The difference between these two outcomes could be due to differences in specimen dimensions. The smaller specimens experienced the larger drop in the ultimate capacities. The smaller specimens would allow for faster saturation of the specimens. Therefore further investigation into the impact of humidity on CFRP strengthened RC beams is warranted.

Another factor investigated was the effect of saltwater on the CFRP systems. These experiments were similar to the humidity tests in that the strengthened beams were immersed in large tanks of saline solution for extended periods of time. The results from one experiment experienced a reduction of 25% in the ultimate capacity for the unconditioned beams (Karbhari and Engineer 1996). The specimens for this study were 330mm (13”) long, 50.8mm (2”) wide, and 25.4mm (1”) deep. However another experiment experienced only a five percent reduction in the ultimate capacity regardless of whether the duration was for 1,000 hours or as large as 10,000 hours of continuous exposure (Grace and Singh 2005). These specimens were 2.74m (9’) long, 152mm (6”) wide, and 254mm (10”) deep. The difference in the magnitude of impact due to saltwater could partially be due to the difference in the size of the specimens. The size difference between the experiments could impact the results due to a smaller specimen becoming fully saturated in less time than a larger specimen. However, for this study the effects of saltwater are not investigated because in typical bridge construction the beams are not exposed to constant immersion in a saline environment.
Another important environmental factor on the durability of CFRP systems is how temperature effects the overall performance. One method of testing the temperature effects on a CFRP system was to expose the specimen to temperature extremes. The experiments that dealt with temperature extremes typically exposed the CFRP system to these temperatures for a given duration. For instance in one study small shear-lap test CFRP specimens were exposed to elevated temperatures until the resin system maintained a temperature of 50°C and then a load was applied to the specimens (Gamage et al. 2006). The results from this experiment demonstrated that at resin temperatures below 45°C the failure mechanism between the CFRP and the concrete substrate tended to be due to bond failure and concrete rupture. However, after the resin temperature exceeded 45°C the failure mechanism changes to the CFRP sheet peeling off of the concrete specimen, this could be due to the temperature exceeding the glass transition temperature for the resin. Another temperature experiment was performed, but the test utilized concrete beams strengthened with CFRP and there were three specimen groups with exposure times of 1,000, 3,000, or 10,000 hours at 60°C(Grace and Singh 2005). The specimens in the experiment were allowed to return to room temperature prior to the four-point ultimate tests. The ultimate capacity tests for these beams actually resulted in a higher load capacity than the unconditioned beams. A similar test was conducted but the specimens were instead exposed to a temperature of -15.5°C for 1440 hours(Karbhari and Engineer 1996). This experiment resulted in a 15% reduction in the ultimate capacity for the specimens. The results from these experiments demonstrate that temperature extremes will affect the CFRP system’s durability and will be investigated in this experiment, but are not covered in this thesis.
A second method of testing the effects of temperature on CFRP systems was through cyclic thermal loading, which consisted of repeatedly cycling the temperature in the testing chambers between two temperatures. A common method for cyclic thermal loading was to expose the specimens to freeze-thaw cycles in which the temperature would be lowered to at least -15°C and then raised back up to at least 4°C. The cycling of the temperature above and below the freezing point of water reduced the ultimate capacity of the specimens between six and thirteen percent for 350 to 700 cycles respectively (Grace and Singh 2005). These results indicate that cyclic thermal loading impacts the durability of the CFRP systems thus warranting further investigation and will be covered in subsequent papers.

Another factor effecting the time-dependent durability of the epoxy interface of the CFRP strengthened beams is fatigue loading. Resistance to degradation from fatigue loading in a material used for rehabilitation and strengthening of bridges is of great concern. Therefore there have been numerous experiments conducted to determine the impact of fatigue on CFRP. An experiment concluded that beams exposed to two million cycles at 15%, 25%, or 40% of the ultimate load experienced a maximum reduction of only 2.7% in their ultimate capacities (Grace and Singh 2005). Another fatigue experiment performed determined that for fatigue loads ranging from 15% to 35% percent of the yielding load the “fatigue response of the CFRP-strengthened beams is not influenced by the number of cycles.” (Gheorghiu 2006). The fatigue response for CFRP-strengthened is dependent on the composition of the matrix (Harries and Aidoo 2006), therefore the fatigue durability for CFRP will be further investigated.
The experiment performed for this paper was conducted to investigate the long-term durability of CFRP strengthening for existing bridges. The primary focus for studying the durability of the CFRP system is to determine the reaction of CFRP to conditions found in the state of Florida. Due to the localized focus of this experiment, factors such as freeze-thaw cycling and freezing temperatures were not investigated.
3. MATERIALS

- CONCRETE

The concrete beams for the experiment were cast at the FDOT Structural Research Center (SRC) in Tallahassee, FL. The beams were poured in eight separate batches with two beams per batch. After each beam was poured a number was inscribed on the beams to aid in the identification of each beam. Compression cylinders measuring 152.4mm (6”) in diameter by 457.2mm (18”) in height were created according to American Society for Testing and Materials (ASTM) Standard C39 for each batch. See Table 3-1 for details about the concrete mix design.

<table>
<thead>
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<tr>
<td>Type: Class II Deck</td>
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<tr>
<td>Target Slump: 76.2mm 3in</td>
</tr>
<tr>
<td>Minimum f’c: 31Mpa 4,500psi</td>
</tr>
<tr>
<td>Max Water to Cement Ratio: 0.44</td>
</tr>
<tr>
<td>Minimum Cementitious Mat.: 362.5kg/m³ 611lb/yd³</td>
</tr>
<tr>
<td>Air Content Range: 1% - 6%</td>
</tr>
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</table>

Pours of consecutive batches were spaced a minimum of seven days apart, this allowed all beams to cure undisturbed in the molds. See Error! Reference source not found. for the pour dates, batch numbers, and the compressive strength results for the seven and twenty-eight day tests of the cylinders. After seven days of curing, the beams were removed from the molds and were then stored outdoors covered with a tarp in the storage yard at the SRC. As noted in Table 3-2, two of the beams had been cast with under-strength concrete and were therefore excluded from the experiment. The two under-strength beams and one of the full strength beams were damaged during handling.
of the specimens. The damage to the full strength beam was some tensile cracking in the compressive face of the beam but this determined not to pose a problem, therefore the beam was still utilized for the experiment.

Table 3-2: Compressive Strength, Fiber System, and Beam Numbers

<table>
<thead>
<tr>
<th>Batch Number</th>
<th>Date</th>
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<th>f'c (psi)</th>
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<td>3205</td>
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<td></td>
<td>2</td>
<td>35.5</td>
<td>5120</td>
</tr>
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<td>2</td>
<td>5/18/2005</td>
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<td>33.3</td>
<td>4836</td>
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<td></td>
<td>4</td>
<td>51.0</td>
<td>7401</td>
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<td>16</td>
<td>51.7</td>
<td>7502</td>
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1Understrength  2Damaged  3Excluded  4Pull-off

- STEEL

The steel reinforcement for the concrete beams was standard grade 60 deformed rebar. Samples of the reinforcement were taken and tested in tension to establish the yield strength. The tensile tests were performed using a Universal Testing Machine (UTM) at the University of Central Florida (UCF) Structures Lab. For the tensile tests an extensometer was mounted on each reinforcement sample to measure the strain during
testing. See Figure 3-1 for the stress-strain plots for each of the steel reinforcement samples tested. As shown in Figure 3-1 the steel reinforcement exhibited the three distinct regions of steel behavior; linearly elastic, yielding, and strain hardening.

Figure 3-1: Stress-Strain Plot for Steel Rebar

- CARBON FIBER

The CFRP systems used to strengthen the concrete beams consisted of three fabric-resin systems. The first two CFRP systems utilized the same unidirectional fabric that had the carbon fibers oriented in one direction with a polyester thread in the other direction to maintain alignment of the carbon fibers until the fabric is installed. The difference between these first two systems was that one of them utilized a generic structural epoxy system that had been used in other CFRP experiments; it was referred to as the GE system. The second system utilized a structural epoxy that had been utilized by the FDOT in prior CFRP research experiments; this system is referred to as the EP system. The third system was a commercially available system with a unidirectional pre-
impregnated fabric, which meant the fabric was already impregnated with the structural epoxy prior to installation. This system utilized a polyurethane base for the structural adhesive; it is referred to as the PU system. The PU system was chosen for two reasons; the system had been used in two structural rehabilitation projects in which the system performed well to environmental exposure and to determine how a different base for the structural adhesive would perform under the same conditions as the epoxy-based adhesives. Table 3-2 displays the fabric system that was used on each beam. See Table 3-3 for the properties of the three CFRP epoxy-resin systems that were used during this experiment.

Table 3-3: Fabric-Resin System Properties

<table>
<thead>
<tr>
<th>System</th>
<th>Fabric Weight g/m²</th>
<th>oz/yd²</th>
<th>Thickness mm</th>
<th>in</th>
<th>No. of Filaments</th>
<th>Adhesive System</th>
</tr>
</thead>
<tbody>
<tr>
<td>GE</td>
<td>447.556</td>
<td>13.2</td>
<td>0.508</td>
<td>0.020</td>
<td>12000</td>
<td>Epoxy</td>
</tr>
<tr>
<td>EP</td>
<td>447.556</td>
<td>13.2</td>
<td>0.508</td>
<td>0.020</td>
<td>12000</td>
<td>Epoxy</td>
</tr>
<tr>
<td>PU</td>
<td>440.775</td>
<td>13.0</td>
<td>0.454</td>
<td>0.018</td>
<td>12000</td>
<td>Polyurethane</td>
</tr>
</tbody>
</table>

Tensile test coupons were created for each of the two CFRP fabrics. The specimens were 25.4mm (1”) wide by 304.8mm (12”) long with a thickness equal to two layers of CFRP fabric. The tensile test coupons were created by laying the CFRP layers onto plastic sheeting as if the CFRP was being installed on the concrete beams. Once the resin had cured the CFRP was removed from the plastic sheet and cut into the tensile test coupons. The tensile coupons were tested until rupture using an UTM at the UCF Structures Lab. During testing an extensometer was used to obtain real-time strain measurements, whereas the stress was calculated using the load and averaged cross-section for each specimen. The extensometer was removed from the specimens prior to rupture in order to avoid potential damage to the extensometer. See Figure 3-2 for the
stress-strain plots for the CFRP fabric utilized by both the EP and GE systems. See Figure 3-3 for the Stress-Strain plots for CFRP fabric utilized by the PU system. Using the stress-strain plots from the tensile tests of the CFRP fabrics the average modulus of elasticity was obtained for each sample. See Table 3-4 for the modulus of elasticity values of the EP and GE CFRP samples. See Table 3-5 for the modulus of elasticity values of the PU tensile specimens.

![Stress-Strain Plot for EP/GE Systems](image)

**Figure 3-2: Stress-Strain Plot for EP/GE Systems**
Figure 3-3: Stress-Strain Plot for PU System

Table 3-4: Modulus for EP & GE Systems

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Modulus of Elasticity GPa</th>
<th>Modulus of Elasticity ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>EP/GE1</td>
<td>51.33</td>
<td>7445</td>
</tr>
<tr>
<td>EP/GE2</td>
<td>54.38</td>
<td>7887</td>
</tr>
<tr>
<td>EP/GE3</td>
<td>54.96</td>
<td>7971</td>
</tr>
<tr>
<td>EP/GE4</td>
<td>64.72</td>
<td>9388</td>
</tr>
<tr>
<td>EP/GE5</td>
<td>63.88</td>
<td>9264</td>
</tr>
<tr>
<td>Average</td>
<td>57.85</td>
<td>8391</td>
</tr>
</tbody>
</table>
The modulus of elasticity was averaged for the fabric-resin systems and then utilized in the analytical models of the CFRP strengthened beams. The averaged modulus of elasticity was used in the analytical models to more closely represent the actual experimental beam specimens. The analytical models of the strengthened beams are discussed in the next section.
4. ANALYTICAL MODELS

Analytical models were developed in MathCAD for the concrete beams in order to predict the behavior and capacity of the beams. The models were created using the Modified Hognestad Method. This model was used to develop moment-curvature relations for prediction of beam response to a given loading condition, which for this experiment was four-point loading. A model for the un-strengthened control concrete beam was created first to establish an initial behavior for the beams.

The first step in creating the control model for the un-strengthened beam was to create a model for the behavior of the steel reinforcement based upon the rebar tensile tests that were conducted. See Figure 4-1 for the stress-strain plot of the rebar behavior; model used in the MathCAD program, as well as the plots from the rebar tensile tests. The curve for the rebar model was created using a linear piecewise function to approximate the stress-strain curve obtained experimentally in the tensile tests. This rebar model was used to define the behavior of the reinforcement as a function of stress and strain. The experimentally-obtained yielding stress for the reinforcement was utilized in the analytical models. The piecewise function approximating the stress-strain curve also allowed for the analytical models to take into account the strain hardening behavior of the steel reinforcement.
The second step in creating the control model for the un-strengthened beams was to create a model for the behavior of the concrete. The compressive behavior for the concrete was modeled using the equations from the Modified Hognestad Method; the tensile behavior was modeled using the Hordijk model; which had been used in prior analytical models (Perez 2005). The equations from the Modified Hognestad Method model the compressive behavior of concrete with a non-linear relation between the stress and strain until 90% of the 28 day compressive stress, then the stress and strain are related by a linearly decreasing line until a strain of 0.0038. Since the equations from the Modified Hognestad Method only model the compressive behavior the Hordijk model was utilized to obtain a more complete concrete behavior model. The Hordijk model utilizes a linear relation between the stress and strain until a percentage of the compressive stress is reached. The model then utilizes a non-linear function to model the behavior until the strain reaches ultimate tensile strain capacity. The tensile stress
capacity was determined using the previously calibrated tensile models (Perez 2005) The stress-strain model behavior is shown in Figure 4-2. It should be noted that the figure shown for the concrete model was created by using a finite number of points and linear segments, whereas the actual behavior was modeled using a piecewise function.

![Concrete Behavior Graph](image)

**Figure 4-2: Plain Concrete Stress-Strain Model**

Once the reinforcement and concrete behaviors had been entered into the MathCAD model of the control beam; the moment-curvature (M-Φ) behavior for the reinforced concrete beam model was created using the Modified Hognestad method. After the M-Φ model for the un-strengthened beam was created, the load-deflection curve was then generated by integration the M-Φ curve. Half of the beam was divided into 15 discrete strips and the moment in each strip due to a given load was calculated and the curvature for that strip was then determined by linearly interpolation. This interpolated deflection was then multiplied by two so that the load deflection curve for the whole beam was generated. After creation of the control model the MathCAD program was
modified to take into account the CFRP attached to the bottom of the strengthened beams. This modification allowed for the behavior for the CFRP strengthened beams to be predicted. See Figure 4-3 for the moment-curvature curves for the control, EP, GE, and PU beams.

![Graph of analytical moment-curvature models for different systems.](image)

**Figure 4-3: Analytical Moment-Curvature Models**

For all of the models the ultimate capacity of the beams was limited to a compressive strain of 0.003 and a tensile strain of 0.0001 for the concrete. See APPENDIX: MATHCAD PROGRAM FLOWCHART for a flowchart of how the analytical beam models were created using the Modified Hognestad Method.

In an effort to more accurately predict the capacity of the beams, the 28 day compressive strength from the test cylinders was entered into the corresponding MathCAD models. For all of the MathCAD models, the same reinforcement behavior curve used in the control was utilized as well as the same concrete behavior curve. See Figure 4-4 for the load deflection curves for each fabric-resin system.
In the analytical models the variance in the predicted behavior of the CFRP strengthened beams varied was due in part to the thickness of the fabric layers for the systems. The difference in fabric thickness is evident by the behavior of the thinner PU system when compared to the thicker EP and GE systems. Another factor that might have influenced the predicted system behavior was the actual concrete compressive strength. The impact due to the concrete compressive strength is evident by the small differences between the models for the EP and GE systems. The analytical models were utilized to predict the ultimate capacity for each system in the experimental testing program. The ultimate capacity for each beam was used to determine the loading steps for the ultimate tests experimental protocol. The testing program, the testing setup, and the test specimens for the experiment are covered in the next section.

Figure 4-4: Analytical Load-Displacement Models
5. TEST SETUP

• TESTING PROGRAM

The testing program for this study was composed of three distinct phases the ultimate testing (U), fatigue testing (F), and thermal conditioning (T). See Table 5-1 for the testing matrix that displays the different phases to which each specimen was exposed. As shown in the table one specimen from each of the fabric-resin systems was exposed to all three of the testing phases. The specimens that were exposed to all three of the testing phases would allow for the combined effects of fatigue and thermal conditioning to be studied.

Table 5-1: Experimental Testing Matrix

<table>
<thead>
<tr>
<th>Beam ID No.</th>
<th>Specimen ID</th>
<th>Fatigue (F)</th>
<th>Thermal (T)</th>
<th>Failure (U)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PU</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>PU-F</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>PU-T</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>PU-F-T</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>5</td>
<td>EP</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>6</td>
<td>EP-F</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>EP-T</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>EP-F-T</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>9</td>
<td>Pull Off Test</td>
<td>X</td>
<td></td>
<td>N.A.</td>
</tr>
<tr>
<td>10</td>
<td>GE</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>11</td>
<td>GE-F</td>
<td>X</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>12</td>
<td>GE-F-T</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>13¹,²,³</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14¹,²,³</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15²</td>
<td>GE-T</td>
<td>X</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>16</td>
<td>Control</td>
<td></td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

¹ Under Strength ² Damaged ³ Excluded
The ultimate testing phase consisted of loading the specimen until failure. Failure was defined as a drop of at least 10% to 15% from the maximum load resisted by the specimen. The loading for the ultimate phase used displacement control to maintain control of the actuator movement during testing. The testing was done at a rate of 2.54mm/min (0.1in/min) with a pause at specific loads to visually inspect the specimen and mark all visible cracking on the beam. Once the inspection and marking of the specimen was complete the loading would resume. The specific loads that corresponded to the pauses during the loading of the specimens were 50% of the theoretical yielding, 75% of the theoretical yielding, at theoretical yielding, and 90% of the theoretical ultimate load. It should be noted that for all of the CFRP strengthened specimens once the theoretical yielding load was obtained the visual inspection of the beam was done from afar, with no marking of the cracks. The reason for not marking the cracks was that it was deemed an unacceptable risk for anyone to get close to the loaded CFRP-strengthened specimens. Thus all cracks that formed after the theoretical yield were marked once testing was completed.

The first set of beams tested to failure was the un-strengthened beam and one specimen from each of the three fabric-resin systems, this allowed for establishment of the control and initial strengthened behaviors for the beams. The second set of beams to be subjected to loading until failure consisted of specimens that had previously experienced two million cycles of the fatigue loading. The third set of beams tested until failure consisted of specimens that had been exposed to the thermal-humidity cycling. The final batch of beams to undergo ultimate testing consisted of the specimens that were exposed to both fatigue loading as well as the thermal-humidity cycling phases. The
ultimate tests from the batch that was exposed to thermal-humidity cycling as well as the batch that underwent fatigue loading and thermal-humidity cycling are not included within this thesis.

The fatigue phase of the experiment consisted of subjecting the appropriate specimens to two million cycles of cyclic loading at a frequency of two hertz. The load for the fatigue testing ranged from 2.2kN (0.5kip) up to 85.4kN (19.2kip) which induced a calculated change in stress of 158.6MPa (23ksi) in the reinforcement. During the fatigue loading the beam displacement and CFRP strain were measured. The strain gauges were calibrated to compensate for the temperature changes that occurred in the lab during the two million cycles. The fatigue loading in the beams were set at a level that would not cause yielding of the steel reinforcement in the specimens and was within the stress range limits defined in AASHTO 5.5.3.2, which was calculated to be 161.3MPa (23.4ksi). This stress range would allow for an unlimited fatigue life for the steel reinforcement, which ensured that the reinforcement would not fail during the fatigue cycling. During the fatigue tests the data was recorded on cycles 1, 1000, 20,000, 100,000 and every 100,000 there after until cycle 2 million. The data that was recorded was the maximum and minimum of load, displacement, beam temperature, and CFRP strains.

The thermal conditioning phase of the experiment consisted of exposing the selected beams to heating and cooling cycles inside of an environmental testing chamber. Inside of the environmental chamber the beams were also exposed to cycling levels of humidity. The purpose of the temperature and humidity cycles was to simulate long-term exposure of the beams to climate conditions similar to that of Florida, therefore the high and low temperatures were typical for Florida as well as the range for the humidity. The
heating and cooling for the environmental chamber was performed by an air conditioning unit with an onboard heater that had been sized to deliver the desired temperatures within the allotted time. The moisture for the humidity was supplied by a humidifier that corresponded to the heating cycle for the chamber. This provided hot moist air on the heating cycle and cold dry air for the cooling cycle that is similar to the climate of Florida. See Figure 5-1 for a diagram of the environmental testing chamber. Figure 5-2 is a picture of the completed environmental testing chamber in the SRC lab. It should be noted that all of the beams received some environmental conditioning due to the fact that the beams were stored outdoors of the FDOT SRC as shown in Figure 5-3. The beams also received some thermal conditioning when inside of the SRC due to the fact that the facility does not have a climate control system for the testing area. See Figure 5-4 for the average monthly air temperatures for the duration the specimens were stored. See Figure 5-5 for the monthly precipitation for during the storage duration. The temperature and precipitation data was taken from the National Oceanic and Atmospheric Administration (NOAA) archives for the Tallahassee Regional Airport. The effects due to the environmental conditioning of the beams are outside of the scope of this paper, but were mentioned here since this paper is a portion of a larger experiment, which investigates the climate dependant behavior of CFRP durability. Another paper will investigate the environmental effects on the durability of the CFRP strengthened beams.
Figure 5-1: Environmental Conditioning Chamber (Plan View)

Figure 5-2: Environmental Testing Chamber

Figure 5-3: Specimens Stored Outside of the SRC
Figure 5-4: Monthly Temperatures in Tallahassee

Figure 5-5: Monthly Precipitation Totals
• TESTING CONFIGURATION

For this experiment all of the concrete beam specimens were loaded using a four-point bending configuration. This configuration was used for both the ultimate and fatigue tests that were performed. The use of the four-point configuration created a 76.2mm (3’) long section of the beam that was exposed to pure bending without shear. See Figure 5-6 and Figure 5-7 for the dimensions of the testing setup. Figure 5-8 shows the un-strengthened RC beam prior to ultimate capacity testing.

![Experimental Test Setup](image)

**Figure 5-6: Experimental Test Setup**

![Close-up View of Supports](image)

**Figure 5-7: Close-up View of Supports**
As shown in Figure 5-7 both of the supports for the beams had hinges built into the supports that allowed for the supports to rotate freely as the beams deflected during loading. The loads for the experiment were applied using a hydraulic ram and a spreader beam. The spreader beam was used so that the two point loads would be spaced at the specified 914.4mm (36”) and would be equal in magnitude. The testing configuration created a clear span of 4.572m (15’) between the supports. Given the large-scale of the specimens utilized for this experiment, the behavior of the concrete and CFRP interface should be very similar to the behavior of strengthened beams in the field. The large-scale specimens would exhibit thermal properties and resistance to moisture penetration close to that of actual beams in the field. This allows for the results from this experiment to more closely represent actual behavior of strengthened beams exposed to the climate of Florida.

The instrumentation of the specimens was done using two main groups of sensors one for measuring deflection of the specimen and the other for measuring strains. The first group of sensors was displacement gauges that were either linearly variable displacement transducers (LVDT) or laser displacement gauges. The location of the
displacement gauges are shown in Figure 5-6, Figure 5-9, and Figure 5-11 as D1, D2, D3, and D4. The displacement gauges at the ends of the specimen were placed on opposite sides of the beam as this allowed for monitoring of any rotation about the longitudinal axis of the beam. As shown in Figure 5-11 the displacement gauge in the center was placed slightly offset from the centerline so as not to interfere with the strain gauge at the midpoint. The second group of gauges was foil strain gauges that were attached to the tensile face of the beam directly onto the CFRP fabric. The strain gauges were coated with a protective coating to keep out moisture and to shield the gauges from damage. The locations for the stain gauges are shown in Figure 5-9, Figure 5-10 and Figure 5-11 as SG1, SG2, SG3, SG4, and SG5. The strain gauges were placed symmetrically about the center of the specimens. The strain gauges on all of the beams were the same type and were installed on all of the specimens. During both the ultimate and fatigue tests the strain gauges were monitored and recorded electronically.

![Baseline Concrete Beam](image1)

**Figure 5-9: Bottom View of Beams with Instrumentation**
Testing Specimens

The dimensions of the specimens were 304.8mm (12”) wide by 457.2mm (18”) high and 4876.8mm (16’) long. The layout for the reinforcement is shown in Figure 5-12, as shown in the figure the stirrups were not continued throughout the beam. The stirrups were not placed in the middle portion of the beam between the symmetric point loads to reduce the amount of material required for each specimen and there would be minimal shear demand between the two loads.
The cross-section of the specimens is shown in Figure 5-13, as shown in the figure the specimens were doubly reinforced beams with stirrups. The top reinforcement within the beams was to aid in the fabrication of the rebar cages and to resist the bending moment due to self-weight when the beams were flipped over for installation of the CFRP fabric to the bottom of the beams. For the dimensions of the stirrups see Table 5-2. See Figure 5-14 for the bar diagram and types.

Figure 5-12: Reinforcement Layout

Figure 5-13: Cross-Section of the Beams
Table 5-2: Rebar Designations and Dimensions

<table>
<thead>
<tr>
<th>Mark</th>
<th>Bar Size</th>
<th>Type</th>
<th>A or G</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>22B01</td>
<td>22</td>
<td>#7</td>
<td>1</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>7B01</td>
<td>7</td>
<td></td>
<td></td>
<td>4699</td>
<td></td>
</tr>
<tr>
<td>10B02</td>
<td>10</td>
<td>#3</td>
<td>1</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>3B02</td>
<td>3</td>
<td></td>
<td></td>
<td>4699</td>
<td></td>
</tr>
<tr>
<td>10B03</td>
<td>10</td>
<td>#3</td>
<td>4</td>
<td>101.6</td>
<td>358.8</td>
</tr>
<tr>
<td>3B03</td>
<td>3</td>
<td></td>
<td></td>
<td>14-1/8&quot;</td>
<td>228.6</td>
</tr>
</tbody>
</table>

Figure 5-14: Reinforcement Bending Diagram

Two layers of CFRP sheets were bonded to the bottom of the CFRP strengthened beams starting at 101.6mm (4") from the centerline of the support at one end and terminating at 101.6mm (4") from the centerline of the supports at the other end. The CFRP sheets were the full width of the beams 304.8mm (12"). Figure 5-15 illustrates the layout of the CFRP on the bottom of the specimens.

Figure 5-15: CFRP Layout
To install both the EP and GE systems the components for the resin systems were mixed according to their perspective ratios and then applied to the bottom of the beam. The first layer fabric was then hand lain onto the resin and a metal roller was used to press the fabric into the resin. Once the first layer had been completed pressed into the resin a second coating of the resin was applied. After the second coating of resin was applied the second layer of fabric was installed. Once the second layer had been completely pressed a final coating of resin was then applied. It should be noted that for the GE epoxy-based resin system there was several difficulties due to a sudden hardening of the resin prior to finishing the installation. In one case the fabric was not fully saturated prior to the hardening of the resin. When the GE system hardened prior to completing installation, more of the resin was mixed and the fabric was re-saturated. The PU system however came with the fabric completely pre-impregnated with the resin. Installation of the PU system consisted of applying a primer to the beam then laying the first layer of pre-impregnated fiber. The layer is lightly misted with water to activate the resin and then the fabric was rolled with a metal roller to work out the air pockets that form due to the resin curing. After rolling the air pockets out of the first layer the second layer of pre-impregnated fabric was laid on top of the first. Then the second layer was misted with water and the air pockets were rolled out. See Figure 5-16 and Figure 5-17 for photos from the lay-up of the PU system.
After all of the CFRP sheets for all three systems had been installed on the beams, all of the specimens received a FDOT Class 5 finish which is a standard finish applied to all exposed surfaces of bridges in Florida. The Class 5 finish helps to protect the beams from moisture and provides a uniform finish. After being coated the specimens were stored outdoors at the SRC. Once the SRC facility was configured to perform testing, the first phase of the experiment was performed. The first phase of testing consisted of the flexure tests for the control and baseline specimens. After the baseline tests had been completed the second phase commenced. The second phase entailed fatigue testing for all
specimens that were to undergo fatigue loading. Upon completion of the fatigue loading, specimens that were to be exposed to the thermal-humidity cycling were placed in the environmental chamber to undergo conditioning while the remaining fatigued beams were tested until ultimate failure. After the specified duration of conditioning all of the remaining beams were tested until failure. The results for all of the specimens that were conditioned in the environmental chamber are not contained within this thesis.
6. RESULTS

The first phase of the experiment was the flexure testing of the specimens until failure to determine ultimate capacity. The first specimen tested was the control beam followed by one specimen from each of the three fabric-resin systems. The load-displacement curves from these baseline tests are displayed in Figure 6-1. Noticeable results due to the CFRP strengthening are the increase in the yielding load and an increase in the ultimate load capacity of the specimens; however the increase in the ultimate load capacities are accompanied by a decrease in the overall ductility of the beam. However the deflection of the strengthened beams at the yielding load was higher than the deflection for the un-strengthened control beam.
During the baseline ultimate tests the surface strains for the CFRP sheets were measured. Strain profile plots were created for each of the fabric-resin systems at 50%, 60%, 70%, 90% and 97% of the maximum load as well as at the maximum load. These strain profiles are shown in Figure 6-2, Figure 6-3, and Figure 6-4. As shown in Figure 6-2 for the PU fabric-resin system the strain gauge at the mid-span of the specimen exceeded the limits set in the data acquisition (DAQ) system, thus there is a gap in the strain plots. To ensure that a complete strain profile could be obtained for the other CFRP specimens the strain limits in the DAQ system were increased.

![PU Strain Profile](image)

Figure 6-2: PU Strain Profile
Figure 6-3: EP Strain Profile

Figure 6-4: GE Strain Profile
Prior to initiation of the ultimate tests all visible cracking on the specimens were marked. During the ultimate tests for the baseline and control specimens the visible cracks were marked at specific loads; which were 50% of yielding, 75% of yielding, and at yielding. The cracks that occurred after yielding were not marked until after the specimens failed and were unloaded. When marking the cracks on the specimens a line was drawn across the trace of the crack to signify where the crack ended at each load step. Upon completion of the ultimate tests the specimen was measured and photographed so that diagrams could be created which display the approximate cracking patterns for the specimens. Figure 6-5 displays an example of the cracks having been marked and the specimen being measured.

![Figure 6-5: Example of Cracking Marking](image)

The following figures are drawn to the same orientation, with the North end of the beam to the left, thus giving the perspective of looking through the specimens; also the reinforcement is shown for clarity and scale. Figure 6-6 and Figure 6-7 display the cracking patterns on the baseline specimen. Figure 6-8 and Figure 6-9 display the cracking patterns for the PU specimen. Figure 6-10 and Figure 6-11 display the cracking patterns for the EP specimen. Figure 6-12 and Figure 6-13 display the cracking patterns
for the GE specimen. It should be noted that the two large cracks that extend almost through the full height of the GE specimen on both sides are due to the mishap during handling of the specimen. Also for the GE specimen there was evidence of the resin not fully saturating the fibers near the North end of the beam. Figure 6-14 displays the unsaturated CFRP fibers after the CFRP sheet de-bonded from the beam. It should be noted that for all of the CFRP-strengthened baseline specimens the failure mode was de-bonding of the laminate from the concrete. For the control specimen the failure mode was concrete crushing at the top of the specimen after extensive cracking of the tensile face.

![Figure 6-6: Control Eastern Face Crack Pattern](image1)

![Figure 6-7: Control Western Face Crack Pattern](image2)

![Figure 6-8: PU Eastern Face Crack Pattern](image3)

![Figure 6-9: PU Western Face Crack Pattern](image4)
The second phase of the testing program was the fatigue loading of two specimens from each of the three fabric-resin systems for a total of six beams. It should be noted that during the first fatigue loading tests, the PU-F specimen, the laser sensors,
which were used for measuring deflection, experienced a large amount of drift in the readings; therefore all other tests were performed with the LVDT displacement gauges to ensure accurate readings. The mid-span deflection amplitude versus the cycle number is displayed in Figure 6-15. For the PU-F specimen, which experienced the sensor drift there was no data recorded prior to cycle number 10,000 and for the PU-F-T system there is no data between cycles 110,000 and 190,000 due to an error with the DAQ system. Also evident in Figure 6-15 is the detrimental effect the laser displacement sensor drift had on obtaining useful data for the PU-F specimen. Therefore it will be expunged from all other plots of fatigue testing data. In order to better visualize how the specimens reacted to the fatigue loading the compliance, which is the inverse of stiffness, was plotted versus cycle in Figure 6-16. Figure 6-16 allows for the stiffness of the beam to be observed during the fatigue loading. The compliance plot for the specimens was constructed using the deflection value at the minimum loading for the cycle. The numerical values on the plot are of no importance due to the fact that each specimen had a different begin value for the displacement gauges. The importance of the figure is that the increasing trend is due to the specimens not rebounding up to the same level after each cycle. This indicates that the beam was softening as the cycles progressed.
In addition to obtaining displacement information for the specimen during fatigue loading the strains were also recorded. For the PU-F specimen the strain data was not recorded by the DAQ system. However the strain data for all the other fatigue specimens
was collected. Figure 6-17 is a plot of the mean strains measured by the strain gauges of the PU-F-T specimen during the fatigue cycling. It should be noted that due to errors with the DAQ system the data between cycles 110,000 and 190,000 was not recorded. Also shown in Figure 6-17 is the calculated strain based on the measured deflections. As shown in Figure 6-17 there may have been some softening of the resin-concrete interface since the calculated strains were higher than those measured. Also collected during the fatigue cycling was the average beam surface temperature; Figure 6-18 displays the data for the PU-F-T specimen. For all of the other specimens there was no error with the DAQ system and the complete mean strain values are displayed in Figure 6-19, Figure 6-21, Figure 6-23, and Figure 6-25. Upon inspection of these figures it becomes evident that for the EP-F, EP-F-T, and GE-F-T specimens there were similar trends between the calculated strains based upon deflections and those measured. It is possible that the spike in the calculated strain for the EP-F-T specimen shown in Figure 6-21 could be due to the displacement gauge being disturbed. For the GE-F specimen the calculated strains were higher than those measured, this could be due to poor bonding of the CFRP to the RC beam. The average surface temperature for each specimen directly follows the mean strain plot for that specimen, which are Figure 6-20, Figure 6-22, Figure 6-24, and Figure 6-26.
Figure 6-17: PU-F-T Mean Strains due to Fatigue

Figure 6-18: PU-F-T Average Beam Temperature
Figure 6-19: EP-F Mean Strains due to Fatigue

Figure 6-20: EP-F Average Beam Temperature
Figure 6-21: EP-F-T Mean Strains due to Fatigue

Figure 6-22: EP-F-T Average Beam Temperature
Figure 6-23: GE-F Mean Strains due to Fatigue

Figure 6-24: GE-F Average Beam Temperature
Upon completion of the fatigue testing, the specimens that would undergo thermal-humidity conditioning were placed inside the environmental testing chamber. The other three specimens which had been subjected to the fatigue loading were visually

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inspected for any cracking and the cracks were marked. The specimens were also checked for any delamination or voids that might have arisen using a screwdriver handle to tap the CFRP while listening for any hollow sounds. The location of the delaminations and voids were marked, measured, and are plotted in Figure 6-27 and Figure 6-28. When observing the void diagrams in Figure 6-27 and Figure 6-28 a portion of the voids could be due to workmanship when the CFRP was bonded to the concrete. It should be noted that the PU-F specimen did not appear to have any voids in the CFRP-concrete interface; however the EP-F and GE-F specimens both had de-bonding of the CFRP sheets from the concrete. A possible reason the polyurethane CFRP system experienced less de-bonding due to the fatigue cycling is that the polyurethane adhesive remains more elastic once cured than the epoxy-based adhesives.

![Figure 6-27: EP-F Approximate Voids](image1)

![Figure 6-28: GE-F Approximate Voids](image2)

After the three specimens had been inspected and all information was recorded the specimens were tested until failure. The failure testing for the three fatigue specimens occurred in the same manner as the baseline ultimate tests. The load versus displacement...
curves for the three specimens are displayed in Figure 6-29 with the control beam results for reference.

![Figure 6-29: Load Displacement for Fatigued Specimens](image)

The yield loads and the maximum loads with the corresponding deflections for the ultimate tests are listed in Table 6-1.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Beam No.</th>
<th>Yielding Load</th>
<th>Yielding Deflection</th>
<th>Ultimate Load</th>
<th>Ultimate Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>16</td>
<td>190.8 kN</td>
<td>19.1 mm 0.75 in</td>
<td>232.3 kN</td>
<td>66.8 mm 2.63 in</td>
</tr>
<tr>
<td>PU</td>
<td>1</td>
<td>275.8 kN</td>
<td>26.2 mm 1.03 in</td>
<td>330.6 kN</td>
<td>46.2 mm 1.82 in</td>
</tr>
<tr>
<td>PU-F</td>
<td>2</td>
<td>270.5 kN</td>
<td>22.9 mm 0.90 in</td>
<td>315.9 kN</td>
<td>39.5 mm 1.56 in</td>
</tr>
<tr>
<td>EP</td>
<td>5</td>
<td>275.8 kN</td>
<td>21.3 mm 0.84 in</td>
<td>312.5 kN</td>
<td>30.3 mm 1.19 in</td>
</tr>
<tr>
<td>EP-F</td>
<td>6</td>
<td>275.8 kN</td>
<td>19.6 mm 0.77 in</td>
<td>282.4 kN</td>
<td>21.7 mm 0.86 in</td>
</tr>
<tr>
<td>GE</td>
<td>10</td>
<td>270.9 kN</td>
<td>22.6 mm 0.89 in</td>
<td>317.8 kN</td>
<td>40.9 mm 1.61 in</td>
</tr>
<tr>
<td>GE-F</td>
<td>11</td>
<td>267.8 kN</td>
<td>20.3 mm 0.80 in</td>
<td>268.2 kN</td>
<td>20.6 mm 0.81 in</td>
</tr>
</tbody>
</table>

For determination of the effect fatigue had on the specimens each type of fabric-resin system is plotted with the baseline and fatigue results as well as the control and
analytical model for references. Figure 6-30 displays the PU system and upon inspection it is apparent that the fatigue loading seems to have had a minor impact on the overall strength of the system. The minor impact is evident by the similar slope of the post yielding portion of the load versus displacement curves. Another piece of evidence that the PU system withstood the fatigue loading with minimal damage is that the ultimate capacity for the fatigued beam is within five percent of the baseline load. Figure 6-31 displays the EP system and upon inspection this system was weakened by the fatigue loading. The weakening of the EP system is evident by the failure of the specimen within ten percent of the yielding load, which indicates that as the steel began to yield and the beam deflect, the CFRP-concrete interface could not sustain the increased load and failed. Figure 6-32 displays the GE system and upon inspection this system was susceptible to the fatigue loading which is evident by failure of the system just as the steel reinforcement began to yield. The increase in stiffness for the PU-F, EP-F, and GE-F specimens could be in part, due to the increase in the compressive strength of the concrete given that over a year passed between the baseline tests and the fatigued tests.
Figure 6-30: PU System Ultimate Tests

Figure 6-31: EP System Ultimate Tests
The strain profiles for the fatigue specimens are displayed in Figure 6-33, Figure 6-34, and Figure 6-35.
Upon inspection of the strain profiles for the EP-F and GE-F specimens the existence of voids in the CFRP-concrete interface are evident by the low strain values. Given the low strains within the EP-F and GE-F specimens the CFRP was not bonded to
the concrete and thus was not resisting the loading. This caused the EP-F and the GE-F specimens to fail at loads lower than the baseline counterparts.

Prior to initiation of the ultimate testing for the fatigued specimens the cracks due to the fatigue cycling were marked on the specimens. As with the baseline tests the ultimate tests for the fatigued specimens the loading was conducted with the similar steps of 50% of yield, 75% of yield, and at yield. The visible cracks on the specimens were marked at the end of each of the loading steps as well as after failure of beams. The crack diagrams are shown below and maintain the same orientation as the diagrams for the baseline tests. Figure 6-36 and Figure 6-35 display the visible crack patterns for the PU-F specimen. The PU-F specimen did not have any detectable voids after completion of the fatigue cycling. Figure 6-38 and Figure 6-39 display the visible crack patterns for the EP-F specimen. The EP-F specimen did incur de-bonding of the CFRP at the mid-span due to the fatigue cycling, as indicated by the presence of the void. Figure 6-40 and Figure 6-41 display the visible crack patterns for the GE-F specimen. The GE-F specimen did not appear to have any large areas of unsaturated fibers, which indicates good saturation of the fibers with the resin.

Figure 6-36: PU-F Eastern Face Crack Pattern

Figure 6-37: PU-F Western Face Crack Pattern
Upon completion of the baseline ultimate tests and the fatigued ultimate tests the following observations were made; use of pre-impregnated fibers should be utilized by either all of the specimens or none, inspection of specimens should be conducted before and after every step of a testing sequence, and the use of thermal imaging would have aided in determination of voids in the CFRP-concrete interface. The consistent use of pre-impregnated fibers would provide ensure a fair comparison between resin systems. A through inspection of the specimens would have assisted in determining when voids and cracks occurred. The use of thermal imaging would have allowed for a more complete detection of voids, thus allowing for better correlation between visible cracks and void locations.
All of the results for all of the specimens that underwent the temperature and humidity cycling were not within the scope of this project and are covered elsewhere.
7. CONCLUSION

Upon completion of this project the performance of the polyurethane-based PU specimens unexpectedly exceeded the performance for the epoxy-based adhesives. The PU system with the pre-impregnated fabric allowed for the experimental baseline ultimate capacity to fall within 10% of the theoretical ultimate capacity. The ultimate capacity of the fatigued specimen was only 11% below the theoretical ultimate capacity. The epoxy-based systems did not obtain an ultimate capacity any higher than 80% of the theoretical ultimate. The difference between the ultimate capacities of the systems could partially be due the fact that the PU system was pre-impregnated, which allows for better and more uniform saturation of the fiber with the resin. Therefore further studies that compare different bases for the structural adhesives would want to ensure that all systems are either pre-impregnated or all hand laid, as this would allow for better comparison between the different bases.

Another aspect of this project in which the PU system excelled over the epoxy bases was durability of the resin-concrete interface during fatigue loading. The stiffer epoxy-based systems exhibited large voids that were created by the degradation of the CFRP-concrete interface during fatigue cycling. However the more elastic PU system allowed for the interface to seemingly remain intact even after two million cycles. There was some amount of damage to the CFRP-concrete interface as demonstrated by the ultimate capacity of the PU-F specimen being lower than then baseline PU specimen; however the PU-F was within five percent of PU ultimate capacity. The durability of the epoxy-based system was not as high as that of the polyurethane system as demonstrated by the failure of the fatigue specimens EP-F and GE-F shortly after yielding of the steel
reinforcement and the voids that were detected. A portion of the difference in the durability of the epoxy and the polyurethane systems could be attributed to the pre-impregnation of the fabric for the PU system. However the EP-F and GE-F specimens were barely able to obtain an ultimate capacity with 90% of the corresponding baseline capacities.

Overall the CFRP systems regardless of the base for the structural adhesive experienced an increase in the ultimate capacity of the specimens. The ultimate capacities of the fatigued specimens still exceed that of the un-strengthened control beam. This increase in the ultimate capacities for the specimens came at the cost of reduced ductility of the specimens as well as a non-ductile failure mode. Therefore, future experiments will need to focus on obtaining better ductility as well as a ductile failure mode.

Also, the environmental effects on the different CFRP adhesives is of interest especially given that polyurethane-based systems are prone to moisture uptake when exposed to humidity. The continuing experimentation of this project is of interest to establish the environmental durability of the PU system. The environmental durability of the EP and GE systems will be of interest to compare with the PU system.
Choose number of discrete strips for beam

Choose curvature

Assume depth to neutral axis

Calculate strain in each concrete strip

Calculate stress in each strip

Calculate force in each strip

Sum forces the forces in the strips

Calculate strain in reinforcement

Calculate stress in reinforcement

Calculate forces in reinforcement

Sum forces in reinforcement

Yes

Is there CFRP?

Calculate strain in CFRP

Calculate stress in CFRP

Calculate forces in CFRP

Sum forces in CFRP

Sum Tensile and Compressive Forces

Yes

Do the forces equal 0?

No

Adjust depth value

No

Moment-curvature relation determined. Repeat with for different curvature
REFERENCES


