Full-Scale Testing of 40 Year Old Prestressed AASHTO Girders That Have Been Retrofitted in Shear by Externally Applied Carbon Fiber Reinforced Polymer Wraps

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FULL-SCALE TESTING OF 40 YEAR OLD PRESTRESSED AASHTO GIRDER
THAT HAVE BEEN RETROFITTED IN SHEAR BY EXTERNALLY APPLIED
CARBON FIBER REINFORCED POLYMER WRAPS

by

David A. Petty

A thesis proposal submitted in partial fulfillment
of the requirements for the degree
of
MASTER OF SCIENCE
in
Civil and Environmental Engineering

Approved:

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UTAH STATE UNIVERSITY
Logan, Utah

2010
ABSTRACT

The Utah Department of Transportation (UDOT) is interested in the application of rehabilitation techniques to strengthen their AASTHO prestressed bridge girders for shear. Utah’s bridges are exposed to deterioration from rain, snow, and the introduction of salt for ice removable. This requires innovative rehabilitation techniques to address the deteriorations of their highway bridges, especially the ends of bridge girders where water and salt are more common due to construction joints. Carbon Fiber Reinforced Polymers (CFRP) are becoming more prevalent as a tool in highway bridge rehabilitation.

This research investigates the application of various CFRP systems that can be used as shear reinforcement for prestressed concrete girders. The experimental program involved full-scale destructive testing of six 40-year-old, AASHTO prestressed I-girders that were salvaged from the 45th South/I-215 bridge in Salt Lake City, Utah. The testing involved retrofitting five of the girders with various configurations of CFRP fabric. Based on the initial tests, the most effective configuration was then applied to another set of I-shaped concrete girders for verifications. After the experimental testing, two analytical models developed for predicting the additional shear contribution of the CFRP reinforcement were compared with the measured results from the experimental program. After testing and comparisons, a CFRP reinforcement configuration and theoretical model was selected as a reliable and effective method for application of external shear reinforcement of AASHTO prestressed I-shaped girders.
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David Petty
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CHAPTER 1
INTRODUCTION

1.1 Context

The Utah Department of Transportation (UDOT) is interested in the application of rehabilitation techniques to strengthen their AASTHO prestressed bridge girders for shear. Utah’s bridges are exposed to deterioration from rain, snow, and the introduction of salt for ice removable. This requires innovative rehabilitation techniques to address the deteriorations of their highway bridges, especially the ends of bridge girders where water and salt are more common due to construction joints. Carbon Fiber Reinforced Polymers (CFRP) are becoming more prevalent as a tool in highway bridge rehabilitation. The focus of this research is to investigate how CFRP fabrics can be used to strengthen AASTHO prestressed I-girders for shear.

This research investigates the application of various CFRP systems that can be used as shear reinforcement for prestressed concrete girders. The experimental program involved full-scale destructive testing of six, forty-year-old, AASHTO prestressed I-girders that were salvaged from the 45th South/I-215 bridge in Salt Lake City, Utah. The testing involved retrofitting five of the girders with various configurations of CFRP fabric. Based on the initial tests, the most effective configuration was then applied to another set of I-shaped concrete girders for verifications. After the experimental testing, two analytical models developed for predicting the additional shear contribution of the CFRP reinforcement was compared with the measured results from the experimental program. After testing and comparisons, a CFRP reinforcement configuration and
theoretical model was selected as a reliable and effective method for application of external shear reinforcement of AASHTO prestressed I-shaped girders.

1.2 CFRP Reinforcement Design

The research program consisted of the testing of a total of five different CFRP configurations. The CFRP fabric selected for this research was the MBrace® CF 160 system that was generously provided by The Chemical Company (BASF). This product was selected based on its simplicity in application and proven superior performance. A specific performance issue was acknowledged when using external CFRP fabrics for I-shaped girders in comparison to typical rectangular cross sections used in previous research. When loaded in shear, a large normal force begins to develop in the CFRP fabric on the web to flange corner which would lead to a premature delamination resulting in a small increase in capacity. Therefore, four of the five CFRP configurations had anchorage systems integrated into them.

Four of the five CFRP configurations included a U-wrap used as a stirrup anchored by one of two proposed anchorage systems. The remaining CFRP configuration did not include an anchorage system and was used as a baseline comparison to those with anchorage systems. The U-wraps were applied as either vertical or diagonal stirrups that were overlapped on the bottom of the girder. The anchorage system was applied as either a horizontal strip of CFRP fabric placed along the web and over the CFRP stirrups or a CFRP laminate that was imbedded into the girder by means of a cut at the web to flange intersection.
1.3 Theoretical Models of Shear Contribution of CFRP

This research also presents a comparison of two analytical design procedures to calculate the contribution of the CFRP reinforcement for shear for AASHTO prestressed girders. The design, philosophy is a natural extension to current procedures used to calculate the nominal shear capacity of a girder:

\[ V_n = V_c + V_s + V_f \]

where \( V_c \) is the shear contribution from the concrete, \( V_s \) is the shear contribution from the embedded steel stirrups, and \( V_f \) is the shear contribution from the CFRP reinforcement.

The first method evaluated in this research is found in ACI 440.2R-8 entitled Guide for the “Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures”. The second method to evaluate \( V_f \) was a methodology presented in a research paper entitled “FRP for Shear Strengthening of AASHTO Bridge Girders” by Hutchinson, Donald, and Rizkalla (1999). Each of these methodologies is used to calculate the additional contribution of the CFRP reinforcement to the nominal shear capacity of the girder. The focus of this research is to investigate the effectiveness of the two methods for predicting the shear contribution of the CFRP reinforcement.

There are two predictive methods used to calculate \( V_c \) and \( V_s \) in this research. The first method is the current AASHTO LRFD Bridge Design Specifications (AASHTO, 2009). This is the preferred method for most state DOTs and for the Federal Highway Administration when designing bridges. The second predictive method is from
Chapter 11 of the American Concrete Institute’s (ACI) concrete building code ACI-318-08 (ACI Committee 318 2008a). The intent of this research is not to focus on in-depth evaluation of the methods for predicting $V_c$ and $V_s$ but to provide a comparison of the measured and predicted results.

1.4 Organization of Thesis

The organization of the thesis is as follows:

1. Chapter 2 presents a summary of past research done on the shear contribution of CFRP fabrics on concrete girders.
2. Chapter 3 presents the full-scale experimental program for the AASHTO prestressed girders. This chapter outlines the various configurations of the CFRP systems. A comparison of results between girders is also presented.
3. Chapter 4 describes the analytical models that were evaluated for predicting the shear contribution of the different CFRP reinforcement configurations. This chapter also presents the calculated results of the methodologies and a comparison with the measured results presented in Chapter 3.
4. Chapter 5 provides a summary of the research findings and conclusions based on these results. Also provided are recommendations for application of the CFRP system to AASHTO prestressed I-shaped girders.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

All over the United States those responsible for the maintenance of our highway bridges are looking for better methods to rehabilitate them. The use of carbon fiber reinforced polymers (CFRP) for the rehabilitation of reinforced concrete members has been a rapidly growing rehabilitation option over the last few years. CFRP has been found to be useful due to its high strength, light weight, corrosion resistance, non-metallic properties, and its ease in application. The purpose of this literature review is to summarize the application of CFRP in the case of shear reinforcement for in service highway bridge girders.

There has been a large amount of research and testing on the use of CFRP for flexural strengthening but little on its application for shear strengthening. A selection of papers on the subject of CFRP for shear reinforcement is summarized in the following sections. These papers focus on the design and effectiveness of CFRP reinforcement for shear. Some of the important parameters that are needed for accurate design are the fiber thickness, fiber orientation, strip spacing, the fiber wrapping, and anchorage. In the papers, the authors present equations that can be used to calculate the additional shear capacity. Also provided are testing results that compare and verify the test and analytical results.
This paper presents testing and research on the use of Carbon Fiber Reinforced Polymers for shear strengthening of reinforced concrete members. The paper addresses parameters of CFRP such as fatigue, anchorage, and the strain field in shear spans. There are several methods in designing of reinforcement with CFRP. These CFRP reinforcement design varies with respect to the orientation of the fibers, strip orientation, and strip thickness. The aim of the study was to address the various methods and compare them to provide insight into their application.

To provide data for the experimental study of CFRP for shear reinforcement the authors tested 23 rectangular beams. Each beam had a different configuration of the CFRP with respect to their angle of orientation, spacing, and fiber thickness. Each beam was loaded with a two point scheme to failure. Strains, stress, and shear strength were measured. Particular attention was paid to the failure mode of the reinforcement, whether it be anchorage or fiber rupture.

The study was able to provide insight into the use of CFRP in shear reinforcement. The authors found that the orientation of the fibers was a critical parameter. To maximize their performance, they must be aligned perpendicular to the shear cracks. Another aspect that was found to be of importance was the anchorage of the CFRP. Full wrapping was ideal, but in the field it may not be plausible. The author recommended further study in the field of anchorage. Measurement of the strain at specific points was found to be insufficient due to non-uniformity. Therefore, the authors suggest using strain measuring methods that cover the beam as a whole.
2.3 Zhang and Hsu (2005)

In this paper the authors present four objectives for their research of CFRP as shear reinforcement for concrete beams. The first is to increase the test database of shear strengthening using externally bonded composites. The second, to investigate the shear behavior and modes of failure of RC beams with shear reinforcement deficiencies with CFRP laminates. Thirdly, to study the effect of various CFRP types and shear reinforcement configurations on the shear behavior of the beam; and finally, to propose design methodologies that are based on experiments and analytical results.

Experimental data was obtained by testing 11 beams in shear with four CFRP configurations. Vertical strips, strips at a 45-degree angle from the longitudinal axis, a longitudinal strip along the middle, and a CFRP fabric placed along the whole side walls of the beam. The reinforced beams test results were compared to the test results with a control beam. Two design equations were used for calculating the shear contribution of the CFRP reinforcement. The design approaches were based on the traditional truss analogy.

Comparison of the test results led to the conclusion that CFRP provides an increase in shear capacity. CFRP strips were found to be very effective compared to CFRP fabrics. The diagonal side strips with angles of 45 and 135 degrees were found to provide the greatest increase in shear strength. The proposed design equations provided acceptable predictions for the reinforced beams shear strength.
2.4 Deniaud and Cheng (2004)

In this article the authors present their findings on shear design methods for concrete beams strengthened with Fiber Reinforced Polymer sheets. The two methods presented combine both the strip method and the shear friction approach. The methods describe the interaction between the concrete, the stirrups, and the FRP sheets. The equations were used and compared to 35 experimental test results.

The Strip Method is described in detail in the paper. An interface shear strength curve is needed for the use of the strip method and is explained in detail. One aspect of the method that was found was, as the width of the FRP sheets become smaller, the bond strength increases. The Shear Friction Method is also explained in detail. The continuous and discrete equations were used to support of the proposed method. Examples were also provided to demonstrate the usefulness of the two methods.

Various conclusions were obtained concerning the two methods. The first was that the design formulations can conservatively predict the experimental results. In addition, the strip method can be used and adapted in various anchorage configurations. Finally, despite the simplicity of the method, it well describes the interaction between the concrete, the stirrups, and the FRP sheets. Overall, the paper presents viable information to the formulation of design equations for FRP reinforcement.

2.5 Adhikary and Mutsuyoshi (2004)

The authors of this research tested and analyzed the effectiveness of using Carbon Fiber Sheets (CFS) as shear reinforcement of RC beams. CFS can be oriented in many
ways with respect to fiber orientation, CFS thickness, and sheet depth. The authors address various methods of design in the experimental program to evaluate the contribution of CFS reinforcement.

Eight beams were used with different CFS reinforcement configurations. Different configurations were varied with respect to vertical and horizontal fiber reinforcement, U wrap or just side beam wraps only, thickness, and height of reinforcement on the side of the beam. The beams were loaded and failed in shear as expected. During the test strains, vertical deflection, and applied load was monitored and recorded. There were two prediction models evaluated that were developed to calculate the contribution of the CFS in shear.

The CFS was found to provide up to 109% increase in the shear capacity for the RC beams. This was based on the results of the configuration consisting of vertical U-wrapped beams. The researchers compared the two equations to the test data and found that both provided satisfactory results in predicting the added shear strength from the CFS. In conclusion, the authors provided sufficient analysis of the equations and test data to provide a confirmation on the usefulness of CFS in shear reinforcement.

2.6 Diagana, Gedalia, and Dlemas (2002)

In this research the authors studied the shear behavior of RC beams reinforced with CFRP. CFRP have been shown to be an effective option for the retrofitting of concrete beams for flexure and shear. The paper focuses on the reinforcement of shear because it is important to insure flexural failure of beams instead of shear. A total of ten
beams were tested in this research. Two were used as control specimens, while the other eight were reinforced with CFRP’s in various configurations. An equation was also used to calculate the increased shear strength of beams retrofitted with CFRP.

The two control beams were constructed with longitudinal steel for flexure and with steel stirrups for shear. An important part of this study was that the beams are already provided with steel stirrups, which is typically of in-service beam conditions. Four beams were given U-shaped CFF strips at 90- and 45-degree orientations at different spacing. The other four were given full-wrap CFF strips at 90- and 45-degree orientations at different spacing’s. The beams were then loaded with a single point load to failure. An equation used in many design codes was used to calculate the increase shear capacity and was then compared with the test results.

Each configuration was found to have its pros and cons. However, all configurations were found to increase the shear capacity. Vertical full-wraps were found to produce the largest increase in shear strength but in field operations full-wraps are not always plausible. Diagonal U strips were found to provide the next highest increase in shear strength, which is a more plausible method in the field. An important aspect of failure of U strips is that they fail due to debonding which is addressed in the predictive equation. The equation used in design was found to provide accuracy up to 14% for most of the beams. The authors concluded that the equation is acceptable for CFRP design.

2.7 Hutchinson, Donald, and Rizkalla (1999)

This research paper presents the results of scale-model testing of AASHTO girders that had been strengthened in shear by applying external carbon fiber reinforced
polymer (CFRP). The authors tested ten different configurations of CFRP wraps. The AASHTO I-girders present special needs when anchoring the CFRP wraps to the web to flange connection, which the authors addressed in the paper.

The experimental program consisted of seven scale-model pretensioned concrete girders. The girders were divided into two types consisting of two different internal stirrup configurations that are typically found in practice. The CFRP wraps configurations consisted of vertical wraps, diagonal wraps, and full wraps. The CFRP wraps were anchored by either clamping or a horizontal strip along the web. The beams were then loaded to their ultimate shear capacity. The capacity of the CFRP was then analytically calculated and compared to the actual found capacity.

The authors found that externally bonded CFRP wraps increased the shear capacity. The configurations that yielded the highest capacities were the diagonal and horizontal wraps anchored by a horizontal strip. They yielded a 36% and 35% increase, respectively.

2.8 Khalifa et al. (1998)

The authors of this research paper present their findings on the contribution of externally bonded FRP to shear capacity of RC beams. They reviewed research on shear reinforcement and testing of RC beams. The aim of the paper was to use the previous research to propose simple design algorithms for computing the contribution of FRP to shear strength of RC members.

The experimental results from 48 test specimens were used to validate the proposed design algorithms. The 48 specimens were collected from eight different
research studies previously published. Two different design approaches were used; one based on Effective FRP Stress, and one based on Bond Mechanism. The paper presents all aspects of the approaches that need to be defined for design. These aspects include fiber orientation, fiber thickness, spacing, bond lengths, etc. Examples were provided for the use of each method. This provided the reader with a thorough explanation of each design approach.

Each design approach was found to be consistent with the ACI 318 protocol and were able to be easily applied for FRP reinforcement on RC beams. The first approach is based on effective FRP stress. This method was found to be valid for CFRP continuous sheets or strips with any orientation angle. The key aspect of that method is that the failure is controlled by sheet rupture. The second approach based on bond mechanism was also found to be valid for CFRP continuous sheets or strips. The key to this method is the effective width of the FRP sheet at delamination. This is because the method is controlled by the sheet delaminating. Both methods were found to conservatively underestimate the actual shear strength of the beams. The authors concluded that the design approaches can be used in calculating the contribution of CFRP’s as shear reinforcement of RC beams.
CHAPTER 3
EXPERIMENTAL PROGRAM

3.1 Introduction

The experimental program consisted of performing sixteen destructive shear tests on eight forty-year old highway bridge girders that were removed from service from the 44th South and I-215 Highway Bridge in Salt Lake City, Utah (see Figure 3.1 for Girder Removal). Two different types of AASHTO girders were evaluated in testing. Four of the tests were used as controls to verify the existing shear strength of the aged girders. The twelve remaining tests were performed on girders that had been retrofitted with various configurations of a Carbon Fiber Reinforced Polymer (CFRP) fabric system. The girders were then tested to failure and the data was analyzed to determine the increase in shear capacity of the CFRP system and then compared to design equations found in the ACI manual.

Figure 3.1. 44th South I-215 highway bridge removal of girders for testing.
3.2 Test Setup

The test setup consisted of a simply supported beam loaded with a single point load (see Figure 3.2). Each test had a single point load placed at a distance of D (depth of beam) plus one foot from the center of the barring plate. The supports were located on the center of the barring plates on each end. Since each girder was tested twice (once on each end), to provide for comparative results, the second test of each girder required shortening of the span to ensure the support was out of the previously failed end. The testing was performed at USU’s Systems Materials and Structural Health (S.M.A.S.H) laboratory. The facility has a strong floor which allowed for full-scale testing of structural members with large loads.

Figure 3.2. Reaction frame and point loading of simply supported girder.
3.2.1 Reaction frame and supports

A reaction frame was required for the testing to provide a reaction for the single point loading of the girders. A part of this research a reaction frame was designed and fabricated. The reaction frame was designed using the LRFD specification. The frame was required to withstand a point load of 1000 kips with a 1.6 live load factor acting anywhere along the beam. The frame had to have the flexibility of varying heights and had to be bolted and anchored to the strong floor. The design led to choosing a W36 X 395 I-section for the beam and two W14 X 283 I-sections for the columns. Beam and column plates were also designed for the connections of the members (see Figures 3.2 and 3.3).

Figure 3.3. 3-D Drawing of reaction frame.
The supports for the simply supported beam were fabricated using steel I-beam sections stiffened along the webs. To allow for rotations that would occur during the test, two inch steel cylinders were welded to the top flange of the I-beam sections (see Figure 3.4).

3.2.2 Instrumentation

Each test had three measurements made from instrumentation. Two hydraulic jacks, a 250 ton and a 600 ton, were used to apply the point load to the girders. To measure the applied force, a pressure gauge and load cell were used to provide accurate measurements (see Figure 3.5). With the measurement of the beam span (support to support) and the shear span (load to nearest support) the shear resistance was measured. An LVDT was used to measure the deflection for each test (see Figure 3.5). The location of the measured deflection was right next to the applied load.
A total of ninety five electrical resistance strain gauges of various lengths were used to measure changes in the strains. To provide for comparative analysis of the control girder and the carbon fiber reinforced girders, locations for the strain gauges on the control girders were selected to match the locations on the carbon fiber reinforced girders. The configurations of the strain gauges for the control tests 1A and 1B are shown in Figures 3.6 and 3.7, respectively. Each test for the reinforced girders had similar configurations depending on the location of the CFRP reinforcement. A data acquisition system provided by Vishay was used to monitor and record the load, deflection, and strains during the testing of the girders.
Figure 3.6. Strain gauge configurations for 1A.

Figure 3.7. Strain gauge configurations for 1B.

3.3 Beam Description

Two types of AASHTO Prestressed Concrete I-girders were tested for this project. The first set of girders consisted of six AASHTO Type II prestressed concrete girders. The lengths of the beams were 23 feet and 7 inches and weighed approximately 15,000 lbs. The cross-sectional dimensions are found on Figure 3.8. The stirrups for the shear reinforcement were spaced at 23 inches on center. For detailed drawings and plans of the first set of AASHTO Type II girders, see Figure A.1 in Appendix A. The second set of prestressed girders were taken from the 10000 West and 1400 North Bridge in Salt
Lake City, UT. The cross-sectional dimensions were the same as the first set of six girders (see Figure 3.8). The stirrups for shear reinforcement were spaced at 12 inches on center. Each girder had a span length of 34.5 feet. In all, fourteen 7/16-inch diameter strands were used to impose a prestressing force on these girders. Two rows of four strands were placed 6 inches from the bottom of the girder. In addition, three rows of two strands remained. All strands were spaced at 2 inches on center.

3.4 CFRP Design

Over the years CFRP fabrics have been found to be useful in providing external reinforcement of structural members. While most of the research has focused on the testing of new members, little amounts of testing and research have been done on retrofitting aged, full-scale girders, with CFRP fabrics. The CFRP fabric system chosen

Figure 3.8. Cross-section of prestressed girders.
for this testing project was the MBrace® CF 160 system that was generously provided by The Chemical Company (BASF). This product was chosen because of its simplicity in application and proven superior performance.

There were five different CFRP configurations tested for this research. Each configuration was tested twice by applying it on six 40-year-old AASHTO girders. After the testing of the first set of 6 girders (12 tests), the most efficient configuration was selected and then tested two more times on the second set of differently reinforced AASHTO girders to allow for further comparison. A specific issue had to be addressed when using external CFRP fabrics for I-shaped girders. When loaded in shear a large normal force begins to develop on the web to flange connection (see Figure 3.9). During loading this would cause the CFRP strips to delaminate prematurely resulting in a small increase in capacity. This delamination of the carbon fiber was one of the main criteria used in developing the CFRP application schemes.

![Figure 3.9. Location of web to flange connection.](image)
3.4.1 Configurations

The first girder tested was used as the control. There was no external reinforcement put on either of the ends. Figure 3.10 is a drawing of the beams configuration while Figure 3.11 is an actual picture of one of the ends of the control girder before testing.

Figure 3.10. Girder 1: No external reinforcement.

Figure 3.11. Girder 1: No external reinforcement.
The second girder tested was the first one that was reinforced with CFRP. Each side consisted of three vertical U-shaped strips that were 20 inches wide placed right next to each other (see Figures 3.12 and 3.13). The strips were anchored with an embedded CFRP laminate along the web to flange connection (see Figure A.2 in Appendix A for detail). This configuration was provided by engineers from The Chemical Company (BASF). It was selected because it addressed the anchorage of the CFRP to the web to flange connection.

Figure 3.12. Drawing of Girder 2 CFRP design.

Figure 3.13. Girder 2 CFRP design
The third girder had CFRP strips oriented at a different angle. Each side consisted of six diagonal (45°) strips 10 inches wide spaced at 4.5 inches and two horizontal strips with a height of 15 inches and 70 inches in length applied over the diagonal strips along the web (see Figures 3.14 and 3.15). Since the diagonal strips could not be continuous, the strips were overlapped on the bottom flange to simulate continuity. This configuration was selected from previous research entitled “FRP for Shear Strengthening of AASHTO Bridge Girders” by Hutchinson, Donald, and Rizkalla (1999). The authors found this configuration to be one of the most effective in increasing shear capacity of AASHTO prestressed girders.

Figure 3.14. Drawing of Girder 3 CFRP design.

Figure 3.15. Girder 3 CFRP design.
The fourth reinforced girder was similar to the third except that the web was not reinforced with a horizontal strip of CFRP. Specifically each side consisted of six diagonal (45°) strips 10 inches wide spaced at 4.5 inches (see Figures 3.16 and 3.17). Since the diagonal strips could not be continuous, the strips were overlapped on the bottom flange to simulate continuity. This configuration was selected for comparison with Girder 3 results. The configuration did not have an anchorage system which provided for comparative results with the anchored configurations. This allowed us to see how the horizontal anchorage system was performing and adding to the shear capacity.

Figure 3.16. Drawing of Girder 4 CFRP design.

Figure 3.17. Girder 4 CFRP design.
The fifth girder was instrumented with individual vertical strips of CFRP as well as the horizontal middle strip. Each side consisted of four vertical U-shaped strips with a width of 10 inches spaced at 4.5 inches and two horizontal strips with a height of 15 inches and 63 inches in length were applied over the vertical strips for anchorage along the web (see Figures 3.18 and 3.19). This configuration was selected from previous research entitled “FRP for Shear Strengthening of AASHTO Bridge Girders” by Hutchinson, Donald, and Rizkalla (1999). The authors found this configuration to be one of the most effective in increasing shear capacity of AASHTO prestressed girders.

Figure 3.18. Drawing of Girder 5 CFRP design.

Figure 3.19. Girder 4 CFRP design.
The last reinforced girder of the group of six had a combination of reinforcing schemes. Each reinforced side consisted of six diagonal (45°) strips 10 inches wide spaced at 4.5 inches (see Figures 3.20 and 3.21). The strips were anchored using an embedded CFRP laminate along the web to flange connection (see Figure A.2 in Appendix A for detail). Since the diagonal strips could not be continuous, the strips were overlapped on the bottom flange to simulate continuity. This configuration was selected to see how the embedded anchorage system would perform with diagonal strips.

Figure 3.20. Drawing of Girder 6 CFRP design.

Figure 3.21. Girder 6 CFRP design
In addition to the previously mentioned six girders, two additional girders were tested. These last two girders had similar prestressing strand configurations but had smaller stirrup spacing. By testing these two girders it was believed that the CFRP reinforcement on other girders could be evaluated. Figures 3.22 and 3.23 show a drawing and picture of the seventh girder tested, the control girder from the second set of two girders.

Figure 3.22. Girder 7 with no external reinforcement.

Figure 3.23. Girder 7 with no external reinforcement.
The eighth girder tested was the same girder as Girder 7 but was reinforced with the same configuration as girder 5. Each side consisted of: Four vertical U-shaped strips with a width of 10 inches spaced at 4.5 inches and two horizontal strips with a height of 15 inches and 63 inches length were applied over the vertical strips for anchorage along the web (see Figures 3.24 and 3.25). This configuration was selected because it yielded a high increase in shear capacity and its ease in application.

Figure 3.24. Drawing of Girder 8 CFRP design.

Figure 3.25. Girder 8 CFRP design.
3.4.2 CFRP application

The Chemical Company (BASF) provided detail instruction on how to apply the CF 160 System to the prestressed concrete girders. The installation required preparing the concrete surface for the application of the MBrace® materials. The concrete preparation required crack repair, sand blasting to at least an ICRI CSP 3 profile, and removal of all dust, laitenance, and bond inhibiting compounds. After the surface preparation was completed the MBrace materials were applied in the following order; the MBrace® Primer, the MBrace® Putty, the MBrace® Saturant, then the MBrace® CF 160 fabric (see Appendix A Figures A.3-A.11 for material detail). For testing, the MBrace® Topcoat was not applied for the last step because the topcoat is for a cosmetic appeal. In practice, the MBrace® Topcoat would be applied as the last step (for detailed instruction on the application see Figures A.12-A.17 in Appendix A). After application of all products the girders were given a seven day curing period.

Figure 3.26. Application of CFRP MBrace® system.
3.5 Testing Analysis

Each load test consisted of placing a hydraulic jack at a distance D (depth of the girder) plus one foot from the end support. The girder was then monotonically loaded until complete failure was achieved. Before each test the support and loading locations were measured and used to calculate the ultimate shear capacity. During each test the load, strain, and deflection (next to applied load) were monitored and recorded. Figure 3.27 is typical graph of the different types of measured data. Graphs of the measured data can be found in Appendix B.

3.5.1 Test 1A

For this test, the hydraulic jack applied load a distance of 48 inches from the end support with a beam span of 268.25 inches. The beam failed in shear at an applied load of 183 kips (see Figure 3.28 for failure crack orientation) with a typical shear crack roughly at

![Figure 3.27. Typical graph of measured data.](image)
a 45-degree angle, which yielded an ultimate shear force of 150.25 kips. The strain gauge configurations are found on Figure 3.6. The data recorded for load, strain, and deflection are found in Appendix B Figures B.1 to B.14.

3.5.2 Test 1B

For the other end of the beam, the hydraulic jack applied load a distance of 48 inches from the end support with a beam span of 208.25 inches. This span length was shorter because the support was moved in on the failed end. The beam failed in shear at an applied load of 229.84 kips. The cracks were roughly 45-degrees from the load to support with multiple roughly vertical cracks under the applied load (see Figure 3.29 for failure crack orientation). The test yielded an ultimate shear force of 176.86 kips. The strain gauge configurations are found on Figure 3.7. The data recorded for load, strain, and deflection are found in Appendix B Figures B.15 to B.26.

Figure 3.28. Failure of Test 1A.
3.5.3 Test 2A

For this test, the hydraulic jack applied load at a distance of 48 inches from the end support with a beam span of 208 inches. The beam failed in shear at an applied load of 333.36 kips. The CFRP system failed due the CFRP laminate anchorage failing, which led to a large normal force at the flange to web connection. After the anchorage failure, the concrete surface attached to the CFRP fabrics was ripped off causing the reinforcement to fail (see Figure 3.31 for detail). Under the CFRP reinforcement there were multiple cracks that were roughly 45-degrees from the load to support (see Figure 3.30 for failure crack orientation). The failure cracks were pushed closer to the support and towards the top of half of the girder. The test yielded an ultimate shear force of 255.68 kips which is an increase of 92.12 kips or a 36.03% increase in shear capacity compared to the average control capacity of 163.56 kips. The data recorded for load, strain, and deflection are found in Appendix B figures B.27 to B.33. The strain gauge
orientation can be found on Figure 3.30 and compared to the strain gauges on control test 1B.

Figure 3.30. Failure of Test 2A and strain gauge orientation.

Figure 3.31. Anchorage failure and concrete surface failure.
3.5.4 Test 2B

For the second test on this beam, the hydraulic jack applied load a distance of 48 inches from the end support with a beam span of 268 inches. The beam failed in shear at an applied load of 198.2 kips. The CFRP system failed due the CFRP laminate anchorage failing, which led to a large normal force at the flange to web connection. After the anchorage failure the concrete surface attached to the CFRP fabrics was ripped off causing the reinforcement to fail (see Figures 3.32 and 3.33 for detail). Under the CFRP reinforcement there were multiple cracks that were roughly 45-degrees from the load to support. The failure cracks were pushed farther from the support and towards the bottom half of the girder (see Figure 3.33 for failure crack orientation). The test yielded an ultimate shear force of 162.70 kips which is a decrease of 0.86 kips or a 0.53% decrease in shear capacity compared to the average control capacity of 163.56 kips. The two tests of this CFRP configuration yield large differences in increased shear capacity. The inconsistent results are assumed to be from the cuts made into the girder for the anchorage system. Further inconsistencies were found in tests 6A and 6B which had the same anchorage system. The data recorded for load, strain, and deflection are found in Appendix B Figures B.34 to B.40. The strain gauge orientation can be found on Figure 3.32 and compared to the strain gauges on control test 1B.

3.5.5 Test 3A

For this beam, the hydraulic jack applied the load at a distance of 48 inches from the end support with a beam span of 210 inches. The beam failed in shear at an applied
load of 255.4 kips. The CFRP system failed due to the horizontal strip of CFRP fabric ripping the top layer of concrete off, which led to a large normal force at the flange to web connection. After the anchorage failure the CFRP diagonal strips ripped off the top layer of concrete leading to delamination causing the reinforcement to fail (See Figures 3.34 and 3.35 for detail). Under the CFRP reinforcement there were multiple cracks that
were roughly 45-degrees from the load to support. The failure cracks were similar to the controls crack orientation (see Figure 3.35 for failure crack orientation). The test yielded an ultimate shear force of 197.02 kips which is an increase of 33.46 kips or a 16.98% increase in shear capacity compared to the average control capacity of 163.56 kips. The data recorded for load, strain, and deflection are found in Appendix B Figures B.41 to B.49. The strain gauge orientation can be found on Figure 3.34 and compared to the strain gauges on control test 1A.

Figure 3.34. Anchorage failure and strain gauge orientation of Test 3A.

Figure 3.35. Failure of Test 3A.
3.5.6 Test 3B

For the second test on this beam, the hydraulic jack applied the load at a distance of 48 inches from the end support with a beam span of 269 inches. The beam failed in shear at an applied load of 255 kips. The CFRP system failed due the horizontal strip of CFRP fabric ripping the top layer of concrete off, which led to a large normal force at the flange to web connection. After the anchorage failure the CFRP diagonal strips ripped off the top layer of concrete leading to delamination causing the reinforcement to fail (see Figures 3.36 and 3.37 for detail). Under the CFRP reinforcement, there were multiple cracks that were roughly 45-degrees from the load to support. The failure cracks were similar to test 3A’s crack orientation but were pushed up towards the top half of the girder (see Figure 3.37 for failure crack orientation). The test yielded an ultimate shear force of 209.5 kips which is an increase of 45.94 kips or a 21.93% increase in shear capacity compared to the average control capacity of 163.56 kips. The data was not recorded for load, strain, and deflection but the maximum applied load was recorded.

Figure 3.36. Anchorage failure and strain gauge orientation of Test 3B.
3.5.7 Test 4A

For this girder, the hydraulic jack applied the load at a distance of 48 inches from the end support with a beam span of 215.5 inches. The beam failed in shear at an applied load of 231.08 kips. The CFRP system failed due to the large normal force generated at the web to flange connection. Since there was no anchorage system the CFRP fabric began to prematurely delaminate. There were still small amounts of the CFRP fabric ripping the concrete off the girder instead of delamination but the failure was primarily delamination (see Figures 3.38 and 3.39 for detail). Under the CFRP reinforcement there were multiple cracks that were roughly 45-degrees from the load to support. The failure cracks were similar to the controls crack orientation (see Figure 3.39 for failure crack orientation). The test yielded an ultimate shear force of 179.61 kips which is an increase of 16.05 kips or a 8.94% increase in shear capacity compared the average control.
capacity of 163.56 kips. The data recorded for load, strain, and deflection are found in Appendix B Figures B.50 to B.58. The strain gauge orientation can be found on Figure 3.38 and compared to the strain gauges on control Test 1A.

Figure 3.38. CFRP delamination and strain gauge orientation of test 4A.

Figure 3.39. Failure of Test 4A.
3.5.8 Test 4B

For the second test of this girder, the hydraulic jack applied a load at a distance of 48 inches from the end support with a beam span of 268.5 inches. The beam failed in shear at an applied load of 212.85 kips. The CFRP system failed due to the large normal force generated at the web to flange connection. Since there was no anchorage system the CFRP fabric began to prematurely delaminate. There were still small amounts of the CFRP fabric ripping the concrete off the girder instead of delamination but the failure was primarily delamination (see Figures 3.40 and 3.41 for detail). Under the CFRP reinforcement there were multiple cracks that were roughly 45-degrees from the load to support. The failure cracks were similar to test 4A’s cracks orientation (see Figure 3.41 for the failure crack orientation). The test yielded an ultimate shear force of 174.8 kips which is an increase of 11.24 kips or a 6.43% increase in shear capacity compared to the average control capacity of 163.56 kips. The data recorded for load, strain, and deflection are found in Appendix B Figures B.59 to B.67. The strain gauge orientation can be found on Figure 3.40 and compared to the strain gauges on control Test 1A.

Figure 3.40. CFRP delamination and strain gauge orientation of Test 4B.
3.5.9 Test 5A

For the fifth girder, the hydraulic jack applied the load at a distance of 48 inches from the end support with a beam span of 180.5 inches. The beam failed in shear at an applied load of 306.6 kips. The CFRP system failed due the horizontal strip of CFRP fabric ripping the top layer of concrete off, which led to a large normal force at the flange to web connection. After the anchorage failure, the CFRP vertical strips ripped off the top layer of concrete leading to delamination causing the reinforcement to fail (See Figures 3.42 and 3.43 for detail). Under the CFRP reinforcement there were multiple cracks that were roughly 45-degrees from the load to support. The failure cracks were similar to the controls crack orientation except for there were no vertical cracks under the applied load (see Figure 3.42 for the failure crack orientation). The test yielded an ultimate shear force of 225.07 kips which is an increase of 61.51kips or a 27.33% increase in shear capacity compared to the average control capacity of 163.56 kips. The data recorded for load,
strain, and deflection are found in Appendix B Figures B.68 to B.78. The strain gauge orientation can be found on Figure 3.42 and compared to the strain gauges on control test 1B.

Figure 3.42. Anchorage failure and strain gauge orientation of Test 5A.

Figure 3.43. Failure of Test 5A.
3.5.10 Test 5B

For the second test on this girder, the hydraulic jack applied the load at a distance of 48 inches from the end support with a beam span of 270 inches. The beam failed in bending at an applied load of 273.58 kips. The CFRP system did not fail in shear. Right under the applied load some concrete under the CFRP system broke off but the girders reinforced shear capacity was greater than the girders moment capacity. This led to the concrete in the top flange crushing and having more of a bending failure than a shear failure (see Figure 3.45 for detail). Some shear cracks did form in the girder (see Figure 3.44 for failure crack orientation). Since the girder failed in bending the ultimate shear capacity was not obtained but the test yielded a shear force of at least 224.94 kips which is an increase of 61.37 kips or a 27.29% increase in shear capacity compared to the average control capacity of 163.56 kips. The data recorded for load and strain are found in Appendix B Figures B.79 to B.89. The strain gauge orientation can be found on Figure 3.44 and compared to the strain gauges on control test 1B.

Figure 3.44. Shear cracks and strain gauge orientation of Test 5B.
3.5.11 Test 6A

For this test, the hydraulic jack applied the load at a distance of 42 inches from the end support with a beam span of 210 inches. The beam failed in moment at an applied load of 310.07 kips. The CFRP system did not fail in shear. The girders reinforced shear capacity was greater than the girders moment capacity. This led to the concrete in the top flange crushing and failing in bending (see Figures 3.46 and 3.47 for detail). Since the girder failed in bending we were unable to find the ultimate shear capacity but the test yielded a shear force of at least 248.06 kips which is an increase of 84.5 kips or a 34.06% increase in shear capacity compared to the average control capacity of 163.56 kips. The data recorded for load, strain, and deflection are found in Appendix B Figures B.90 to B.97. The strain gauge orientation can be found on Figure 3.46 and compared to the strain gauges on control test 1A.
3.5.12 Test 6B

For the second test on this beam, the hydraulic jack applied the load at a distance of 42 inches from the end support with a beam span of 268 inches. The beam failed in shear at an applied load of 180 kips. The concrete in the beam failed in shear before allowing the load to be transferred to the CFRP system. This led to some of the CFRP fabrics to delaminate without the anchorage system failing (see Figures 3.48 and 3.49 for detail). The premature failure in the girders with this type of anchorage system is assumed to be caused by the grooves cut into the girder. Under the CFRP reinforcement
there were multiple cracks that were roughly 45-degrees from the load to support. The failure cracks were similar to the control (see Figure 3.49 for failure crack orientation).

The test yielded an ultimate shear force of 151.79 kips which is a decrease of 11.77 kips or a 7.75% decrease in shear capacity compared to the average control capacity of 163.56 kips. The data recorded for load, strain, and deflection are found in Appendix B Figures B.98 to B.106. The strain gauge orientation can be found on Figure 3.48 and compared to the strain gauges on control Test 1A.

Figure 3.48. Concrete shear failure and strain gauge orientation of Test 6B.

Figure 3.49. Failure of Test 6B.
3.5.13 Test 7A

For this girder, the hydraulic jack applied the load at a distance of 51.5 inches from the end support with a beam span of 199.5 inches. The beam failed in shear at an applied load of 355.01 kips (see Figure 3.50 for failure crack orientation) with a typical shear crack roughly at 45-degrees, which yielded an ultimate shear force of 263.36 kips. The data recorded for load and deflection are found in Appendix B Figures B.107 and B.108.

3.5.14 Test 7B

For the second test of this girder, the hydraulic jack applied the load at a distance of 48 inches from the end support with a beam span of 163 inches. The beam failed in shear at an applied load of 367.99 kips (see Figure 3.51 for failure crack orientation) with a typical shear crack roughly at 45-degrees, which yielded an ultimate shear force of 259.63 kips. The data recorded for load and deflection are found in Appendix B Figures B.109 and B.110.

Figure 3.50. Failure of Test 7A.
3.5.15 Test 8A

For the eighth girder, the hydraulic jack applied the load at a distance of 48.5 inches from the end support with a beam span of 150.5 inches. The CFRP system failed due the horizontal strip of CFRP fabric ripping the top layer of concrete off, which led to a large normal force at the flange to web connection. After the anchorage failure, the CFRP vertical strips ripped off the top layer of concrete leading to delamination causing the reinforcement to fail (see Figures 3.52 for detail). Under the CFRP reinforcement there were multiple cracks that were roughly 45-degrees from the load to support. The failure cracks were similar to the controls crack orientation except for there were no vertical cracks under the applied load (see Figure 3.53 for the failure crack orientation). The test yielded an ultimate shear force of 311.96 kips which is an increase of 50.44 kips compared to the average control capacity of 280.44 kips. The data recorded for load and deflection are found in Appendix B Figures B.111 and B.112. There were no strains measured for this test.
3.5.16 Test 8B

For the second test of the eighth girder, the hydraulic jack applied the load at a distance of 49 inches from the end support with a beam span of 196 inches. The beam
failed in shear at an applied load of 410.63 kips. The CFRP system failed due the horizontal strip of CFRP fabric ripping the top layer of concrete off, which led to a large normal force at the flange to web connection. After the anchorage failure, the CFRP vertical strips ripped off the top layer of concrete leading to delamination causing the reinforcement to fail (see Figures 3.54 for detail). Under the CFRP reinforcement there were multiple cracks that were roughly 45-degrees from the load to support. The failure cracks were similar to the controls crack orientation except for there were no vertical cracks under the applied load (see Figure 3.55 for the failure crack orientation). The test yielded an ultimate shear force of 307.97 kips which is an increase of 46.48 kips compared to the average control capacity of 261.5 kips. The data recorded for load and deflection are found in Appendix B Figures B.113 and B.114. There were no strains measured for this test.

Figure 3.54. Anchorage failure of Test 8B.
3.6 Results

The results of the experimental tests were analyzed by comparing the measured shear capacities, measured strains, and measured deflections. By analyzing these measurements we are able to quantify the contribution of the MBrace® CF160 CFRP reinforcement. The measured shear capacities provided the magnitude of increased shear capacity. The measured strains verify that the CFRP reinforcement was partially resisting the applied shear. The measured deflections demonstrate how the stiffness of the girder was being affected by the CFRP reinforcement. Each of these results are described in detail in sections 3.5.1 and 3.5.2.
3.6.1 Comparison of measured shear capacity

During testing the externally applied load was measured using a load cell and pressure gauge. The beam distance from support to support was called the beam span and the distance from the applied external load to the nearest support (shear span). With these measurements, ultimate shear force using elemental beam theory was calculated. Table 3.1 shows the measured results recorded for each test. The unreinforced baseline shear force was obtained from the two control tests. The 163.56 kip shear force was obtained based on an average of the two tests which were 150.25 kips and 176.86 kips. After each experimental test with the CFRP reinforcement the total shear force was obtained and by subtracting the baseline shear force the magnitude of shear that was contributed by the CFRP reinforcement was obtained.

The experimental program was successful in providing evidence that CFRP reinforcement on I-shaped prestressed AASHTO girders does provide additional shear strength. Not all CFRP reinforcement configurations were as successful as others. The configurations on Girders 2 and 6 had very inconstant results of roughly -8% to 34% changes in shear capacity, which was assumed to be due to the cuts in the girders needed for the anchorage system. The CFRP reinforcement configurations on girders 3 and 5 were found to be the most effective, ranging from an increase of 17% to 33%.

The configuration on Girder 4 was similar to Girders 3 and 5 except the configuration did not have the horizontal anchorage system. This configuration was roughly 8-20% less effective than the other two. That decrease in capacity is evidence
Table 3.1. Comparative results of experimental program

<table>
<thead>
<tr>
<th>Test</th>
<th>Shear Span (inches)</th>
<th>Beam Span (inches)</th>
<th>Applied Load (kips)</th>
<th>Existing Shear Force (kips)</th>
<th>CFRP Shear Force (kips)</th>
<th>Total Shear Force (kips)</th>
<th>Percent Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>48</td>
<td>268.25</td>
<td>183.00</td>
<td>150.25</td>
<td>0.00</td>
<td>150.25</td>
<td>0.00%</td>
</tr>
<tr>
<td>1B</td>
<td>48</td>
<td>208.25</td>
<td>229.84</td>
<td>176.86</td>
<td>0.00</td>
<td>176.86</td>
<td>0.00%</td>
</tr>
<tr>
<td>2A</td>
<td>48</td>
<td>206</td>
<td>333.36</td>
<td>163.56</td>
<td>92.12</td>
<td>255.68</td>
<td>36.03%</td>
</tr>
<tr>
<td>2B</td>
<td>48</td>
<td>268</td>
<td>198.20</td>
<td>163.56</td>
<td>-0.86</td>
<td>162.70</td>
<td>-0.53%</td>
</tr>
<tr>
<td>3A</td>
<td>48</td>
<td>210</td>
<td>255.40</td>
<td>163.56</td>
<td>33.46</td>
<td>197.02</td>
<td>16.98%</td>
</tr>
<tr>
<td>3B</td>
<td>48</td>
<td>269</td>
<td>255.00</td>
<td>163.56</td>
<td>45.94</td>
<td>209.50</td>
<td>21.93%</td>
</tr>
<tr>
<td>4A</td>
<td>48</td>
<td>215.5</td>
<td>231.08</td>
<td>163.56</td>
<td>16.05</td>
<td>179.61</td>
<td>8.94%</td>
</tr>
<tr>
<td>4B</td>
<td>48</td>
<td>268.5</td>
<td>212.85</td>
<td>163.56</td>
<td>11.24</td>
<td>174.80</td>
<td>6.43%</td>
</tr>
<tr>
<td>5A</td>
<td>48</td>
<td>180.5</td>
<td>306.60</td>
<td>163.56</td>
<td>61.51</td>
<td>225.07</td>
<td>27.33%</td>
</tr>
<tr>
<td>5B</td>
<td>48</td>
<td>270</td>
<td>273.58</td>
<td>163.56</td>
<td>61.38</td>
<td>224.94</td>
<td>27.29%</td>
</tr>
<tr>
<td>6A</td>
<td>42</td>
<td>210</td>
<td>310.07</td>
<td>163.56</td>
<td>84.50</td>
<td>248.06</td>
<td>34.06%</td>
</tr>
<tr>
<td>6B</td>
<td>42</td>
<td>268</td>
<td>180.00</td>
<td>163.56</td>
<td>-11.77</td>
<td>151.79</td>
<td>-7.75%</td>
</tr>
<tr>
<td>7A</td>
<td>51.5</td>
<td>199.5</td>
<td>355.00</td>
<td>263.36</td>
<td>0.00</td>
<td>263.36</td>
<td>0.00%</td>
</tr>
<tr>
<td>7B</td>
<td>48</td>
<td>163</td>
<td>368.00</td>
<td>259.63</td>
<td>0.00</td>
<td>259.63</td>
<td>0.00%</td>
</tr>
<tr>
<td>8A</td>
<td>48.5</td>
<td>150.5</td>
<td>460.30</td>
<td>261.50</td>
<td>50.47</td>
<td>311.96</td>
<td>16.18%</td>
</tr>
<tr>
<td>8B</td>
<td>49</td>
<td>196</td>
<td>410.63</td>
<td>261.50</td>
<td>46.48</td>
<td>307.97</td>
<td>15.09%</td>
</tr>
</tbody>
</table>

that the horizontal anchorage system was effective in anchoring the CFRP sheets and giving an overall increase in shear capacity.

From the first set of six girders we were able to conclude that the fourth CFRP reinforcement configuration (vertical strips with a horizontal anchorage strip) was the most effective in increasing the shear capacity. That configuration was then tested on the second set of two girders. On Girder 8 the same CFRP reinforcement was found to increase the shear capacity of the girder. The increase of shear capacity of was an average of 30 kips. This increase was less than that found on Girder 5 which had the same configurations; this can be due to the larger existing shear strength in the girder.
3.6.2 Comparison of measured strains

During testing strain gauges were placed on the CFRP system parallel to the direction of fibers. Measuring the strain along the fibers would allow for conclusive evidence that while the girder was loaded, the shear was being transferred to the CFRP reinforcement. Figure 3.56 shows a graph of Load vs Strain for a strain gauge on the control and a strain gauge on a reinforced girder. It can be seen on the graph that as the externally applied load increases the control girders concrete begins to yield at that spot and the strain begins to increase. It can be seen also that the strain of the CFRP reinforcement begins to increase around the same external load at the same point. This is seen as evidence that the shear resistance of the girder is being transferred to the CFRP shear reinforcement. Figure 3.57 is another example of Load vs Strain comparisons of a non reinforced girder and a reinforced girder. These two graphs of Load vs Strain are evidence that the CFRP shear reinforcement is resisting the shear force applied to the girder.

![Load vs Strain Graph](image)

Figure 3.56. Load vs strain of strain gauges 3 on control Test 1A and CFRP reinforced Test 4A.
Figure 3.57. Load vs strain of strain gauges 3 on control Test 1A and CFRP reinforced Test 3A.

There is another observation made from the measurement of strain that is vital to understanding the how the CFRP reinforcement is reacting. Gauges 4 and 8 are horizontal located on the horizontal strip used for anchorage, while gauges 3 and 9 are vertical located on one of the vertical strips (see Figure 3.42 for exact of locations). It can be seen that in Figure 3.58 that the max strain measured for gauges 3, 4, 8, and 9 are .001, .006, .009, and .001, respectively. Plotting the max measured strains on Figure 3.59 shows us that the max stress (ksi) in the fibers are 33, 198, 297, and 33 for their respective locations. This shows that the max stress of 297 ksi is well below the rupture stress of 550 ksi. It can also be noted that there were large stresses in the horizontal strip which provides evidence that the anchorage system was successful in increasing the capacity.
Figure 3.58. Load vs strain of strain gauges 3, 4, 8, and 9 on Test 5A.

Figure 3.59. Stress vs strain graph of the CFRP fabric (CF 160).
3.6.3 Comparison of measured deflections

During each test the vertical deflection was measure at the applied load. Taking this measurement and plotting it as the load increased allowed for observations on the changes in deflection of the girder due to the CFRP reinforcement throughout reading. In Figure 3.60 it can be seen that as the load is increased the deflection is linear until yielding began, at which point the deflection began to increase more rapidly with less applied load. It can be seen that from Figure 3.60 that the girder with CFRP reinforcement was stronger and was able to produce a larger deflection than the girder without reinforcement. We can conclude that the CFRP reinforcement does provide the system with increased deflections as the girder and CFRP reinforcement act compositely.

Another observation found in Figure 3.60 is that during the loading stage where the concrete is remaining linear, the stiffness of the girder remains the same with either no reinforcement or if there is reinforcement.

![Graph showing Load vs Deflection](image)

Figure 3.60. Load vs deflection of control and Girder 5 configuration.
CHAPTER 4
ANALYSIS OF MEASURED AND PREDICTED RESULTS

4.1 Introduction

In this chapter, a comparison of two analytical methods that calculate the contribution of the CFRP reinforcement for shear in AASHTO prestressed girders is presented. The general design equation used to calculate the nominal shear capacity of a girder is:

\[ V_n = V_c + V_s + V_f \]  \hspace{1cm} (4.1)

where \( V_c \) is the shear contribution from the concrete, \( V_s \) is the shear contribution from the steel stirrups, and \( V_f \) is the shear contribution from the CFRP reinforcement.

There are two methods for calculating the carbon fiber contribution for shear, \( V_f \) that will be evaluated in this research. The first method evaluated to calculate \( V_f \) is described in ACI 440.2R-8 entitled Guide for the “Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures” (ACI, 2008b). The second method to evaluate \( V_f \) is a method presented in a research paper by Hutchinson, Donald, and Rizkalla (1999). Each of these methods are used to calculate the additional contribution of the CFRP reinforcement \( V_f \) to the overall nominal shear capacity \( V_n \) of the eight tested bridge girders. The nominal shear capacity from the two different methods will then be compared to the ultimate shear capacity found in the experimental program in Chapter 3.

There are also two predictive methods used to calculate \( V_c \) and \( V_s \) in this research. The first is the current AASHTO LRFD Bridge Design Specifications
(AASHTO, 2009). This is the preferred method for most state DOTs and for the Federal Highway Administration when designing bridges. Chapter 5 of this code, which describes the shear and torsion behavior of concrete beams, was the main section utilized in this research. The second predictive method is described in the American Concrete Institute’s (ACI) concrete building code ACI-318-08 (ACI, 2008b). This design code is for structural concrete both in buildings and otherwise. Chapter 11 of the ACI code was the main portion that was relevant for this research. This chapter describes the shear strength design codes as they apply to prestressed concrete girders. The intent of this research is not to provide details regarding the derivation of these methods but to provide a comparison between the predicted measured values. Further details regarding the calculation of $V_c$ and $V_s$ by the various methods can be found in, “Existing Shear Capacity of Full Scale, Forty-Year-Old Prestressed-Concrete Beams” by Osborn (2010).

### 4.2 AASHTO Analytical Methods for $V_c$ and $V_s$

The AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) provides two different methods for the determination of the design shear capacity of a prestressed concrete girder. These methodologies take in account the components of shear from the concrete $V_c$, the shear resistance provided by the transverse reinforcement $V_s$, and the vertical component of the prestressing force $V_p$. The nominal shear capacity is calculated as the lesser of AASHTO Equations 5.8.3.3-1, and 5.8.3.3-2 which are provided here as Equations 4.2 and 4.3, respectively.

\[ V_n = V_c + V_s + V_p \]  \hspace{1cm} (4.2)
\[ V_n = 0.25 f'_c b_v d_v + V_p \] (4.3)

\[ V_s = A_v f_y d_v (\cot \theta + \cot \alpha) \sin \alpha \] (4.4)

According to the code, there are two different methods of calculating \( V_c \). The first method, titled the general procedures, come from a modified compression field theory and assumes that the concrete shear stresses are uniformly distributed over a width \( b_v \) and a depth \( d_v \). It is also assumed that the directions of the principal compressive stresses (\( \theta \)) remain constant over \( d_v \), and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. AASHTO Equation 5.8.3.3-3 provides the relationship for \( V_c \) and is shown here as Equation 4.5.

\[ V_c = 0.0316 \beta \psi_g f_o f'_c b_v d_v \] (4.5)

The second method, titled the simplified method, for calculating \( V_c \) is very similar to the ACI method presented in the next section. In this method the values of \( V_c \) and \( V_s \) are calculated differently based on the method the shear cracks develop, namely flexure-shear cracking and web-shear cracking. If flexure-shear cracks develop, the value \( V_{ci} \) (Equation 4.6), is used and if web-shear cracks develop \( V_{cw} \) (Equation 4.7) is used for the value of \( V_c \). \( V_c \) is defined to be the lesser of \( V_{ci} \) and \( V_{cw} \). In the AASHTO specifications the requirements are given in Article 5.8.3.4.3 and listed herein as follows. The component of shear resistance provided by the transverse steel, \( V_s \) (Equation 4.4), shall be computed via Equation 4.3 with \( \cot \theta = 1.0 \) where \( V_{ci} < V_{cw} \), and \( \cot \theta = 1.0 + 3 \frac{f_p}{f'_c} \leq 1.8 \) where \( V_{ci} > V_{cw} \).

\[ V_{ci} = 0.02 \sqrt{f'_c} b_v d_v + V_d + \frac{V_t M_{cre}}{M_{max}} \geq 0.06 \sqrt{f'_c} b_v d_v \] (4.6)
\[ V_{cw} = (0.06\sqrt{f'_c} + 0.30f_{pc})b_v d_v + V_p \]  

(4.7)

4.2.1 Results for AASHTO analytical methods for \( V_c \) and \( V_s \)

The AASHTO specifications procedures for calculating shear capacities are both based on bending theory, and St. Venant’s principle. This means it is assumed that the shear stresses are distributed evenly through the depth of the beam as long as the load is applied at a distance larger than the depth of the beam, or outside the d-region. The eight beams that were tested for this research were tested at the boundary of the d-region where the shear stresses have been found not to be evenly distributed through the depth of the beam. Having the load applied in the d-region causes the stresses in the girder to be concentrated in some regions, and almost non-existent in other areas. The design codes examined in this research are intended to give a value of shear outside of this region and be conservative for values within this region. The codes allow this because the values computed using the design equations are conservative. A comparison of the measured and calculated shear capacities are presented in Table 4.1 showing the predicted values as calculated using the equations presented above.

<table>
<thead>
<tr>
<th>Method</th>
<th>Girders 1-6 Vn (kips)</th>
<th>Percent of Measured</th>
<th>Girders 7-8 Vn (kips)</th>
<th>Percent of Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO General</td>
<td>47.79</td>
<td>29.22%</td>
<td>37.66</td>
<td>14.4%</td>
</tr>
<tr>
<td>AASHTO Simplified</td>
<td>82.27</td>
<td>50.30%</td>
<td>100.28</td>
<td>38.35%</td>
</tr>
<tr>
<td>Measured Value</td>
<td>163.56</td>
<td>100%</td>
<td>261.5</td>
<td>100%</td>
</tr>
</tbody>
</table>
4.3 ACI Analytical Methods for $V_c$ and $V_s$

The ACI code (2008a) presents two different methods for computing the shear strength of prestressed concrete members. The first is the approximate method which estimates the contribution of shear strength using a simplified expression. This method can only be used in prestressed members if the effective prestress force is equal to or greater than 40% of the tensile strength of the flexural reinforcement. The nominal shear capacity of a prestressed girder according to ACI 11-9 is given here as Equation 4.8.

$$V_c = \left(0.6\lambda\sqrt{f'_c} + \frac{700V_u d_p}{M_u}\right)b_w d$$  \hspace{1cm} (4.8)

This value must not be less than Equation 4.16,

$$V_c = 2\lambda\sqrt{f'_c} b_w d$$ \hspace{1cm} (4.9)

or greater than Equation 4.10.

$$V_c = 5\lambda\sqrt{f'_c} b_w d$$ \hspace{1cm} (4.10)

where:

$\lambda$ = unit weight of concrete modification factor (1 for normal weight concrete)

$V_u$ = the maximum design shear at the section being considered (kips)

$M_u$ = the design moment at the same section occurring simultaneously with $V_u$ (kip-in)

$d_p$ = the distance from the extreme compression fiber to the centroid of the prestressing strands (in)

$d$ = the distance from the extreme compression fiber to the centroid of the tension reinforcement (in)

$f'_c$ = the compressive stress of the concrete (psi)
The contribution of shear from the web shear reinforcing steel \((V_s)\) must be added to the contribution of the concrete \((V_c)\). ACI equation 11-15 should be used to calculate the shear contribution from the stirrups and is given as Equation 4.11.

\[
V_s = \frac{A_s f_y d}{s}
\]  

(4.11)

where

\(V_s\) = the shear resistance provided by the transverse shear steel (kips)

\(A_s\) = the area of transverse steel \((\text{in}^2)\)

\(f_y\) = the yield strength of the transverse (ksi)

The second method described in the ACI code (2008a) is the detailed method in which \(V_c\) is taken as the smaller of \(V_{ci}\) of \(V_{cw}\). This method may be used for any beam, and must be used when the effective prestress force is less than 40% of the tensile strength of the flexural reinforcement. The term \(V_{ci}\) is used to describe the shear strength of a member when the diagonal shear cracks form due to a combination of shear and moment. \(V_{cw}\) is used to define the nominal shear strength of a member when the diagonal cracks form due to excessive principal tensile stress in the concrete. \(V_{ci}\) can be approximated with ACI Equation 11.3.3.1 as follows in Equation 4.12.

\[
V_{ci} = 0.6\lambda \sqrt{f'_c h_w d_p} + V_d + \frac{V_i M_{cre}}{M_{max}} \geq 1.7\lambda \sqrt{f'_c h_w d_p}
\]  

(4.12)

where

\(V_d\) = the shear at the section in question due to service dead load (lbs)

\(V_i\) = the shear that occurs simultaneously with \(M_{max}\) (lbs)

\(M_{cr}\) = the cracking moment (lb-in)
$$M_{cr} = \left(\frac{1}{y_c}\right) \left(6\lambda\sqrt{f'_c} + f_{pc} - f_d\right) \text{(lb-in)}$$

(4.21)

$I$ = the moment of inertia of the section that resists the externally applied load (in$^4$)

$Y_t$ = the distance from the centroidal axis of the gross section (neglecting the reinforcing) to the extreme tension fiber (in)

$f_{pc}$ = the compressive stress in the concrete due to prestress after all losses at the extreme fiber of the section where the applied loads cause tension (psi)

$f_d$ = the stress due to unfactored dead load at the extreme fiber where the applied loads cause tension (psi)

The equation for $V_{cw}$ gives the shear capacity of a concrete beam in units of pounds as derived from a rather simplified principle tension theory and is given by Equation 4.13 which comes from ACI Equation 11-22.

$$V_{cw} = \left(3.5\lambda\sqrt{f'_c} + 0.3f_{pc}\right)b_w d_p + V_p \geq 1.5\lambda\sqrt{f'_c}b_w d$$

(4.13)

The equation for calculating $V_s$ for the detailed ACI method is the same as the simplified ACI method (see Equation 4.11).

### 4.3.1 Results for ACI analytical methods for $V_c$ and $V_s$

The ACI procedures for calculating shear capacities are both based on bending theory, and St. Venant’s principle. This means that plane sections are assumed to remain plane at a distance larger than the depth of the beam, or outside the d-region. The eight beams that were tested for this research were tested at the boundary of the d-region where the shear stresses were not likely to be evenly distributed through the depth of the beam. Having the load applied that close to the support causes the stresses in the girder to be
concentrated in some regions, and almost non-existent in other areas. The design codes examined in this research did not take into account the effects of shear stresses in the d-region through the depth of the beam. The codes allow this because the values computed using the design equations are conservative. The results are presented in Table 4.2 showing the predicted values as calculated using the equations presented above. The calculated values are also compared to the measured values as a percentage of the measured values.

4.4 ACI Analytical Method for $V_f$

The first analytical method that is evaluated for this research is the recommendations found in the ACI 440.2R-08 manual entitled, “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures.” This research covers how the ACI code compares to actual experimental results found in CFRP reinforcement for I-shaped prestressed girders. The reader should note that the ACI code does not have specific design equations for I-shaped sections but in this research an evaluation was performed how the standard rectangular section equations in the code ACI code apply to other shapes. The ACI equation for the

<table>
<thead>
<tr>
<th>Method</th>
<th>Method</th>
<th>Girders 1-6 $V_n$ (kips)</th>
<th>Percent of Measured</th>
<th>Girders 7-8 $V_n$ (kips)</th>
<th>Percent of Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI Simplified</td>
<td></td>
<td>101.74</td>
<td>62.20%</td>
<td>131.09</td>
<td>50.13%</td>
</tr>
<tr>
<td>ACI Detailed</td>
<td></td>
<td>90.98</td>
<td>55.62%</td>
<td>136.75</td>
<td>52.29%</td>
</tr>
<tr>
<td>Measured Value</td>
<td></td>
<td>163.56</td>
<td></td>
<td>261.5</td>
<td></td>
</tr>
</tbody>
</table>
contribution of CFRP systems for shear is expressed in Equation 4.14 as:

\[ V_f = \psi_f \frac{A_{f\ell} f_{c\ell} (\sin\alpha + \cos\alpha) d_f}{S_f} \]  

(4.14)

where

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \psi_f )</td>
<td>Reduction factor for U-wraps</td>
<td>0.85</td>
</tr>
<tr>
<td>( A_{f\ell} = 2nt_f w_f )</td>
<td>Area of CFRP shear reinforcement, in²</td>
<td></td>
</tr>
<tr>
<td>( n )</td>
<td>number of plies</td>
<td></td>
</tr>
<tr>
<td>( t_f )</td>
<td>thickness of CFRP sheet, in.</td>
<td></td>
</tr>
<tr>
<td>( w_f )</td>
<td>width of CFRP strip, in.</td>
<td></td>
</tr>
<tr>
<td>( f_{c\ell} = \varepsilon_c E_f )</td>
<td>Tensile Stress</td>
<td></td>
</tr>
<tr>
<td>( E_f )</td>
<td>Tensile Modulus</td>
<td></td>
</tr>
<tr>
<td>( \varepsilon_c = K_v \varepsilon_{fu} \leq 0.004 )</td>
<td>Max. effective strain</td>
<td></td>
</tr>
<tr>
<td>( \varepsilon_{fu} )</td>
<td>design rupture strain, in/in</td>
<td></td>
</tr>
<tr>
<td>( K_v = \frac{k_1 k_2 L_e}{468 \varepsilon_{fu}} \leq 0.75 )</td>
<td>Bond reduction Coefficient, in.-lb units</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.1. Dimensional variables for CFRP shear reinforcement design found in ACI 440.2R-08.
The equation presented above is presented in more detail in the ACI 440.2R-08. Equation 4.14 was developed based on U-wraps, calculated using standard units, while the code found in the ACI is used for both full wraps and U-wraps in both metric and standard units.

The results for the ACI method in Table 4.3 are the calculated results for the shear contribution of CFRP reinforcement $V_f$, using the ACI equation explained in Section 4.2. The ACI code is used for the design of girders with rectangular sections and the research presented is how the equations can apply to I-shaped girders. In Table 4.3 are the results of the ACI code for I-shaped girders.

Table 4.3. Calculated results for ACI analytical methods for $V_f$

<table>
<thead>
<tr>
<th>Method</th>
<th>Girder 2</th>
<th>Girder 3</th>
<th>Girder 4</th>
<th>Girder 5</th>
<th>Girder 6</th>
<th>Girder 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>$ACI \ V_f$ (kips)</td>
<td>84.00</td>
<td>63.72</td>
<td>63.72</td>
<td>57.93</td>
<td>63.72</td>
<td>55.82</td>
</tr>
<tr>
<td>$d_f$ (in)</td>
<td>28.80</td>
<td>28.80</td>
<td>28.80</td>
<td>28.80</td>
<td>28.80</td>
<td>27.80</td>
</tr>
<tr>
<td>$w_f$ (in)</td>
<td>20</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>$S_f$ (in)</td>
<td>20.00</td>
<td>18.64</td>
<td>18.64</td>
<td>14.50</td>
<td>18.64</td>
<td>14.50</td>
</tr>
<tr>
<td>$\alpha$ (deg)</td>
<td>90.00</td>
<td>45.00</td>
<td>45.00</td>
<td>90.00</td>
<td>45.00</td>
<td>90.00</td>
</tr>
<tr>
<td>$f'c$ (psi)</td>
<td>7000</td>
<td>7000</td>
<td>7000</td>
<td>7000</td>
<td>7000</td>
<td>7000</td>
</tr>
<tr>
<td>$L_e$ (in)</td>
<td>1.35</td>
<td>1.35</td>
<td>1.35</td>
<td>1.35</td>
<td>1.35</td>
<td>1.35</td>
</tr>
<tr>
<td>$k_1$</td>
<td>1.45</td>
<td>1.45</td>
<td>1.45</td>
<td>1.45</td>
<td>1.45</td>
<td>1.45</td>
</tr>
<tr>
<td>$k_2$</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
</tr>
<tr>
<td>$K_v$</td>
<td>0.24</td>
<td>0.24</td>
<td>0.24</td>
<td>0.24</td>
<td>0.24</td>
<td>0.24</td>
</tr>
<tr>
<td>$e_{fe}$</td>
<td>0.004</td>
<td>0.004</td>
<td>0.004</td>
<td>0.004</td>
<td>0.004</td>
<td>0.004</td>
</tr>
<tr>
<td>$n$</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>$A_{fv}$ (in(^2))</td>
<td>0.52</td>
<td>0.26</td>
<td>0.26</td>
<td>0.26</td>
<td>0.26</td>
<td>0.26</td>
</tr>
<tr>
<td>$t_f$ (in)</td>
<td>0.013</td>
<td>0.013</td>
<td>0.013</td>
<td>0.013</td>
<td>0.013</td>
<td>0.013</td>
</tr>
<tr>
<td>$f_{fe}$ (ksi)</td>
<td>131.98</td>
<td>131.98</td>
<td>131.98</td>
<td>131.98</td>
<td>131.98</td>
<td>131.98</td>
</tr>
</tbody>
</table>
4.5 Analytical Method in Hutchinson, Donald, and Rizkalla (1999) for $V_f$

In a research paper entitled “FRP for Shear Strengthening of AASHTO Bridge Girders” by Hutchinson, Donald, and Rizkalla (1999), the authors present an alternative rational method to the ACI procedure to calculate the additional contribution of a CFRP system for shear reinforcement. This contribution due to the CFRP reinforcement is calculated using the following expression:

\[
V_{f\text{max}} = \frac{\varepsilon_{f\text{ave}}E_f2n_frw_fd_f}{s_f} (\cot\theta+\cot\alpha_f)\sin\alpha_f
\]

where

- $E_f = \text{Tensile Modulus}$
- $n_f = \text{Number of layers}$
- $t_f = \text{Thickness of sheet}$
- $d_f = \text{Length from top of flange to the centroid of longitudinal steel in bottom flange}$
- $d = \text{Length from centroid of longitudinal steel to top of beam (or top of deck if deck exists)}$
- $w_f = \text{Width of sheet}$
- $s_f = \text{Length of spacing from edge of one strip to the same edge of next strip}$
- $\varepsilon_{f\text{ave}} = \text{Average CFRP strain for I-shaped sections}$
- $\psi_{g,o0} = 0.004$ (diagonal strips 45-degrees)
- $\psi_{g,o0} = 0.0028$ (Vertical strips)

\[
\varepsilon_{f\text{ave}} = \varepsilon_{f\text{max}} \left[ \left( \frac{d}{2} \right) + 0.5 \left( d_f - \frac{d}{2} \right) \right] / d_f
\]

where:

- $\varepsilon_{f\text{max}} = 0.004$ (diagonal strips 45-degrees)
- $\varepsilon_{f\text{max}} = 0.0028$ (Vertical strips)
Figure 4.2. Dimensional variables for CFRP shear reinforcement design found in Hutchinson, Donald, and Rizkalla (1999).

Table 4.4 lists the calculated results for the shear contribution of the CFRP reinforcement for I-shaped girders using the method presented in Hutchinson, Donald, and Rizkalla (1999) that is explained in Section 4.4.

Table 4.4. Results for method in Hutchinson, Donald, and Rizkalla (1999) for $V_f$

| Analytical method in Hutchinson, Donald, and Rizkalla (1999) |
|------------------|------------------|------------------|------------------|------------------|------------------|------------------|
|                  | Girder 2 | Girder 3 | Girder 4 | Girder 5 | Girder 6 | Girder 8 |
| $V_f$ (kips)     | 90.50    | 51.75    | 51.75    | 62.41    | 51.75    | 62.41    |
| $d$ (in)         | 39.00    | 39.00    | 39.00    | 39.00    | 39.00    | 39.00    |
| $d_f$ (in)       | 24.00    | 24.00    | 24.00    | 24.00    | 24.00    | 24.00    |
| $w_f$ (in)       | 20.00    | 10.00    | 10.00    | 10.00    | 10.00    | 10.00    |
| $S_f$ (in)       | 20.00    | 18.64    | 18.64    | 14.50    | 18.64    | 18.64    |
| $n_f$ (in)       | 0.013    | 0.013    | 0.013    | 0.013    | 0.013    | 0.013    |
| $n$ (ply)        | 1        | 1        | 1        | 1        | 1        | 1        |
| $L_f$ (in)       | 37.73    | 37.73    | 37.73    | 37.73    | 37.73    | 37.73    |
| $e_{f_{max}}$    | 0.003    | 0.003    | 0.003    | 0.003    | 0.003    | 0.003    |
| $e_{f_{ave}}$    | 0.0025   | 0.0025   | 0.0025   | 0.0025   | 0.0025   | 0.0025   |
| $\theta$ (deg)  | 30       | 30       | 30       | 30       | 30       | 30       |
| $\alpha$ (deg)  | 90       | 45       | 45       | 90       | 45       | 90       |
4.6 Comparison of Analytical Models for $V_f$ and measured $V_f$

The calculated magnitudes of $V_f$ for both methods were found to compare well with the measured values of shear but the effectiveness of their predictions were dependent on the CFRP reinforcement configuration. The analytical and measured values of additional shear capacity are compared in Table 4.5. This section will evaluate each CFRP reinforcement configuration and how it compared to both analytical predictive models when compared to the shear capacity of the control.

**Girder 1.** There was no CFRP reinforcement applied to this girder. The average measured shear capacity of this girder was used for comparison of Girder 2 through 6 results. The average shear capacity of Girder 7 was 163.37 kips.

**Girder 2.** The CFRP configuration for this girder was found to be effective in providing additional shear capacity but was also found to be extremely sensitive to the application process which led to even decreases in shear capacity. This sensitivity has been attributed from the anchorage system which involved cutting one inch grooves into the girder (see Chapter 3 for detail). For Test 2A the CFRP reinforcement provided an additional shear force of 92.11 kips. The ACI model predicted the CFRP reinforcement would yield 84 kips while the Hutchinson, Donald, and Rizkalla (1999) model predicted a 90.5 kip increase. For this test both models were close and conservative but the Hutchinson, Donald, and Rizkalla (1999) model predicted 98% of the actual shear capacity increase while the ACI predicted 92.2%. It must be noted that for Test 2B there was a decrease in shear capacity for one of the tests. This decrease is believed to be due to the sensitivity of the configuration and the required cuts for the anchorage system.
**Girder 3.** The CFRP reinforcement configuration on this girder was found to increase the shear capacity and was found to be more consistent in comparison to Girder 2. This consistency is believed to be due to the horizontal strip of CFRP placed over the diagonal stirrups. The average increase of shear capacity from the two tests on Girder 3 was 39.7 kips. The ACI model predicted the CFRP reinforcement would yield 63.72 kips while the Hutchinson, Donald, and Rizkalla (1999) model predicted a 51.75 kip increase. For this test both models overestimated the increased shear capacity. The difference is believed to be due the shape of the girder causing early debonding. The actual increased shear capacity was only 62.3% of the ACI analytical prediction and 76.7% of the Hutchinson, Donald, and Rizkalla (1999) analytical prediction.

**Girder 4.** The CFRP reinforcement configuration for this girder was similar to Girder 3 but with the absence of the horizontal anchorage strip. It was expected that this capacity would be less than Girder 3. The decrease in the additional shear capacity for this girder was evidence that the horizontal strip for anchorage was successful in increasing the capacity of the diagonal and vertical strips. The average increase of shear capacity from the two tests on Girder 4 was 13.64 kips. The ACI model predicted the CFRP reinforcement would yield 63.72 kips while the Hutchinson, Donald, and Rizkalla (1999) model predicted a 51.75 kip increase. The analytical methods predicted the same increase in shear capacity as Girder 3 even though Girder 4 did not have the anchorage system. For this test both models overestimated the increased shear capacity. The actual increased shear capacity was only 21.4% of the ACI analytical prediction and 26.35% of the Hutchinson, Donald, and Rizkalla (1999) analytical prediction.
**Girder 5.** The CFRP reinforcement configuration of Girder 5 provided a large increase in shear capacity for both tests. Due to its consistency and ease in application this girder was selected as the most effective reinforcement configuration and the same configuration was applied again on Girder 8. The analytical calculations were also found to be effective in predicting the increase in magnitude of shear capacity. The average increase of shear capacity from the two shear tests on Girder 5 was 61.43 kips. The ACI model predicted the CFRP reinforcement capacity at 57.93 kips while the Hutchinson, Donald, and Rizkalla (1999) model predicted a 62.41 kip capacity increase. For this test, both predictive methods underestimated the measured shear capacity of the CFRP reinforcement. Both models were close and conservative but the Hutchinson, Donald, and Rizkalla (1999) method predicted 94.3% of the actual shear capacity increase while the ACI predicted 101.6%. The Hutchinson, Donald, and Rizkalla (1999) predictive method was found to be only 5.7% over conservative for this girder, while the ACI method only overestimated by 1.6%.

**Girder 6.** The CFRP configuration for this girder was found to be effective in providing additional shear capacity but similar to Girder 2 was also found to be extremely sensitive to the application process which led to decrease in shear capacity for one test. This is the same issue that was found in the CFRP reinforcement configuration on Girder 2. This sensitivity has been attributed from the anchorage system which involved cutting one inch grooves into the girder at the interface of the web and bottom flange (see Chapter 3 for detail). For Test 6A, the CFRP reinforcement provided an additional shear force of 84.49 kips. The ACI model predicted the CFRP reinforcement would add an
additional 63.72 kips while the Hutchinson, Donald, and Rizkalla (1999) method predicted a 51.75 kip increase. For this test both models were close but very conservative. The Hutchinson, Donald, and Rizkalla (1999) model predicted an increase of 61.25% of the actual shear capacity while the ACI methodology predicted an increase of 75.42%.

Girder 7. There was no CFRP reinforcement applied to this girder. The average measured shear capacity of this girder was used for comparison of Girder 8 results. The average shear capacity of Girder 7 was 261.50 kips.

Girder 8. The CFRP reinforcement configuration for Girder 8 provided a large increase in shear capacity for both tests. Due to the consistency and ease in application on Girder 5 the same configuration was applied and tested on Girder 8. The analytical models were also found to be effective in predicting the increase in shear capacity of Girder 5. The average increase of shear capacity from the two tests on Girder 8 was 48.46 kips. The ACI methodology predicted the CFRP reinforcement would provide an additional capacity of 55.82 kips while the Hutchinson, Donald, and Rizkalla (1999) methodology predicted a 62.41 kip increase. For this test both models overestimated the increased shear capacity. The actual increased shear capacity was only 86.82% of the ACI analytical prediction and 77.65% of the Hutchinson, Donald, and Rizkalla (1999) analytical prediction.

The success of the predictive methodologies was dependent on the CFRP reinforcement configurations. The configurations of Girder 5 and 8 were found to be most consistent in matching the predictive methodologies and most consistent in increasing the shear capacity. For Girders 5 and 8 the average increase in shear capacity
due to the same configurations, was 54.96 kips. The ACI predicted an average increase for girders 5 and 8 of 56.88 kips while the Hutchinson, Donald, and Rizkalla (1999) model predicted 62.41 kips. The ACI method overestimated by 3.5% while the Hutchinson, Donald, and Rizkalla (1999) overestimated the average measured increases by 13.55%.

4.7 Comparison of Analytical Models for $V_n$ and Measured $V_n$

This section compares the nominal shear capacity, $V_n$, of the 8 tested girders. It compares the measured values and calculated for $V_c$, $V_s$, and $V_f$. Table 4.6 provides the results of this comparison. There are numerous options for comparing the calculated $V_n$. There were four different methods for calculating $V_c$, two methods for calculating $V_s$, and two methods for calculating $V_f$. Section 4.6 provides an example of comparing the ACI simplified method for $V_c$ and $V_s$ and the ACI method $V_f$. This will provide insight

Table 4.5. Results of actual $V_f$ and analytical methods for $V_f$

<table>
<thead>
<tr>
<th>Method (kips)</th>
<th>Test 1A</th>
<th>Test 1B</th>
<th>Test 2A</th>
<th>Test 2B</th>
<th>Test 3A</th>
<th>Test 3B</th>
<th>Test 4A</th>
<th>Test 4B</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_n$</td>
<td>150.28</td>
<td>176.86</td>
<td>255.68</td>
<td>162.70</td>
<td>197.02</td>
<td>209.50</td>
<td>179.61</td>
<td>174.80</td>
</tr>
<tr>
<td>$V_f$ (actual)</td>
<td>0</td>
<td>0</td>
<td>92.11</td>
<td>-0.87</td>
<td>33.45</td>
<td>45.93</td>
<td>16.04</td>
<td>11.23</td>
</tr>
<tr>
<td>$V_f$ (ACI)</td>
<td>0</td>
<td>0</td>
<td>84.00</td>
<td>84.00</td>
<td>63.72</td>
<td>63.72</td>
<td>63.72</td>
<td>63.72</td>
</tr>
<tr>
<td>$V_f$ (hut. et al.)</td>
<td>0</td>
<td>0</td>
<td>90.50</td>
<td>90.50</td>
<td>51.75</td>
<td>51.75</td>
<td>51.75</td>
<td>51.75</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Method (kips)</th>
<th>Test 5A</th>
<th>Test 5B</th>
<th>Test 6A</th>
<th>Test 6B</th>
<th>Test 7A</th>
<th>Test 7B</th>
<th>Test 8A</th>
<th>Test 8B</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_n$</td>
<td>225.07</td>
<td>224.94</td>
<td>248.06</td>
<td>151.79</td>
<td>263.36</td>
<td>259.63</td>
<td>311.93</td>
<td>307.97</td>
</tr>
<tr>
<td>$V_f$ (actual)</td>
<td>61.52</td>
<td>61.37</td>
<td>84.49</td>
<td>-11.78</td>
<td>0</td>
<td>0</td>
<td>50.44</td>
<td>46.48</td>
</tr>
<tr>
<td>$V_f$ (ACI)</td>
<td>57.93</td>
<td>57.93</td>
<td>63.72</td>
<td>63.72</td>
<td>0</td>
<td>0</td>
<td>55.82</td>
<td>55.82</td>
</tr>
<tr>
<td>$V_f$ (hut. et al.)</td>
<td>62.41</td>
<td>62.41</td>
<td>51.75</td>
<td>51.75</td>
<td>0</td>
<td>0</td>
<td>62.41</td>
<td>62.41</td>
</tr>
</tbody>
</table>
into how the calculated shear capacity, $V_n$, would compare to the actual shear capacity found in testing when taking into account the various components of shear, $V_c$, $V_s$, and $V_f$.

**Girder 1.** Testing resulted in an actual nominal average measured shear capacity of 163.57 kips for Girder 1. Using the ACI simplified method for calculating the nominal shear capacity gave a capacity of 101.75 kips. This is a very conservative capacity and left 62.83 kips of capacity not accounted for. This girder did not have CFRP reinforcement and was used as the control specimen that all other girders could be compared to and provide insight into differences in measured and predicted values.

**Girder 2.** Testing resulted in an actual nominal measured shear capacity of 255.68 kips for Girder 2. Using the ACI methods for calculating the nominal shear capacity ($V_c + V_s$) gave a combined capacity of 185.74 kips. This conservative approach left 69.94 kips of capacity not accounted for. When comparing the controls actual capacity to the reinforced capacity it can be seen that the increase in capacity from $V_f$ is 92.11 kips. It was calculated from ACI that $V_f$ would be 84 kips. This implies that 8.11 kips of the unaccounted shear capacity of 69.94 kips can be attributed to the ACI method for calculating $V_f$. This results in a residual value of 61.83 kips attributed to the ACI simplified method for $V_c$ and $V_s$. This shows that the ACI simplified method for calculating $V_c$ and $V_s$ is more conservative than the ACI method for calculating $V_f$. It should also be noted that on Test 2B the shear capacity was reduced due to the sensitivity of the anchorage system. Therefore, only Test 2A was used in the comparison.
Girder 3. Testing resulted in an actual nominal average measured shear capacity of 203.26 kips for Girder 3. Using the ACI methods for calculating the nominal shear capacity gave a combined capacity of 165.46 kips. This conservative approach left 37.80 kips of capacity not accounted for. When comparing with the control tests actual capacity to the reinforced capacity it can be seen that the increase in capacity from $V_f$ is 39.69 kips. It was calculated from ACI that $V_f$ would be 63.72 kips. This means that 24.03 (overestimated) kips of the unaccounted shear capacity of 39.69 kips can be attributed to the ACI method for calculating $V_f$. This leaves 13.77 kips attributed to the ACI simplified method for $V_c$ and $V_s$. This shows that the ACI method for calculating $V_f$ is more conservative that the ACI simplified method for calculating $V_c$ and $V_s$.

Girder 4. Testing resulted in an actual nominal average measured shear capacity of 177.21 kips for Girder 4. Using the ACI methods for calculating the nominal shear capacity gave a combined capacity of 165.46 kips. This approach left only 11.75 kips of capacity not accounted for. When comparing the controls actual capacity to the reinforced capacity it can be seen that the increase in capacity from $V_f$ is 13.64 kips. It was calculated from ACI that $V_f$ would be 63.72 kips. This means that the ACI method overestimated the contribution of $V_f$ by 50.09 kips which would account for all of the unaccounted shear capacity of 11.75 kips. This provides inconclusive evidence when comparing the contribution of $V_c$ and $V_s$ versus $V_f$.

Girder 5. Testing resulted in an actual nominal average measured shear capacity of 226.51 kips for Girder 5. Using the ACI methods for calculating the nominal shear capacity gave a combined capacity of 159.67 kips. This conservative approach left 66.84
kips of capacity not accounted for. When comparing the controls actual capacity to the reinforced capacity it can be seen that the increase in capacity from \( V_f \) is 62.94 kips. It was calculated from ACI that \( V_f \) would be 57.93 kips. This means that 5.01 kips of the unaccounted shear capacity of 66.84 kips can be attributed to the ACI method for calculating \( V_f \). This leaves 61.83 kips attributed to the ACI simplified method for \( V_c \) and \( V_s \). This shows that the ACI simplified method for calculating \( V_c \) and \( V_s \) is more conservative than the ACI method for calculating \( V_f \).

**Girder 6.** Testing resulted in an actual nominal measured shear capacity of 248.06 kips for Girder 6. Using the ACI methods for calculating the nominal shear capacity gave a combined capacity of 165.46 kips. This conservative approach left 82.60 kips of capacity not accounted for. When comparing the controls actual capacity to the reinforced capacity it can be seen that the increase in capacity from \( V_f \) is 84.49 kips. It was calculated from ACI that \( V_f \) would be 63.72 kips. This means that 22.77 kips of the unaccounted shear capacity of 84.49 kips can be attributed to the ACI method for calculating \( V_f \). This leaves 61.83 kips attributed to the ACI simplified method for \( V_c \) and \( V_s \). This shows that the ACI simplified method for calculating \( V_c \) and \( V_s \) is more conservative than the ACI method for calculating \( V_f \). It must be noted also that on Test 6B the shear capacity was reduced due to the sensitivity of the anchorage system.

**Girder 7.** Testing resulted in an actual nominal average measured shear capacity of 261.50 kips for Girder 7. Using the ACI simplified method for calculating the nominal shear capacity gave a capacity of 161.09 kips. This is a very conservative capacity and
left 130.41 kips of capacity not accounted for. This girder did not have CFRP reinforcement and was used as a control for Girder 8.

Girder 8. Testing resulted in an actual nominal average measured shear capacity of 309.95 kips for girder 8. Using the ACI methods for calculating the nominal shear capacity gave a combined capacity of 186.91 kips. This conservative approach left 123.04 kips of capacity not accounted for. When comparing the controls actual capacity to the reinforced capacity it can be seen that the increase in capacity from \( V_f \) is 48.46 kips. It was calculated from ACI that \( V_f \) would be 55.82 kips. This means that 7.36 kips (overestimated) kips of the unaccounted shear capacity of 123.04 kips can be attributed to the ACI method for calculating \( V_f \). This leaves 115.68 kips attributed to the ACI simplified method for \( V_c \) and \( V_s \). This shows that the ACI simplified method for calculating \( V_c \) and \( V_s \) is more conservative than the ACI method for calculating \( V_f \).

Overall it was seen that the calculated nominal shear capacity, \( V_n \), was conservative when compared to the tested actual nominal shear capacity explained in Chapter 3 for each of the tested girders. When evaluating the shear components of \( V_c \), \( V_s \), and \( V_f \), it was found that the methods for calculating \( V_c \) and \( V_s \) were more conservative than the methods for calculating \( V_f \). This assumption was found to be true for girders 1, 2, 5, 6, 7, and 8 (see above for detail).
Table 4.6. Comparisons of $V_n$ using ACI simplified for $V_c$ and $V_s$ and ACI for $V_f$

<table>
<thead>
<tr>
<th></th>
<th>Girder 1</th>
<th>Girder 2</th>
<th>Girder 3</th>
<th>Girder 4</th>
<th>Girder 5</th>
<th>Girder 6</th>
<th>Girder 7</th>
<th>Girder 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_n$ (actual)</td>
<td>163.57</td>
<td>255.68</td>
<td>203.26</td>
<td>177.21</td>
<td>226.51</td>
<td>248.06</td>
<td>261.50</td>
<td>309.95</td>
</tr>
<tr>
<td>$V_n$ (Calculated)</td>
<td>101.74</td>
<td>185.74</td>
<td>165.46</td>
<td>165.46</td>
<td>159.67</td>
<td>165.46</td>
<td>131.09</td>
<td>186.91</td>
</tr>
<tr>
<td>$V_c$ and $V_s$ Difference</td>
<td>61.83</td>
<td>69.94</td>
<td>37.80</td>
<td>11.75</td>
<td>66.84</td>
<td>82.60</td>
<td>130.41</td>
<td>123.04</td>
</tr>
<tr>
<td>$V_f$ Difference</td>
<td>0</td>
<td>8.11</td>
<td>(24.03)</td>
<td>(36.45)</td>
<td>5.01</td>
<td>20.77</td>
<td>0</td>
<td>(7.36)</td>
</tr>
<tr>
<td>$V_c$ and $V_s$ Difference</td>
<td>61.83</td>
<td>61.83</td>
<td>13.77</td>
<td>0</td>
<td>61.83</td>
<td>61.83</td>
<td>130.41</td>
<td>115.68</td>
</tr>
</tbody>
</table>
CHAPTER 5
SUMMARY AND CONCLUSION

5.1 Summary

Carbon Fiber Reinforced Polymers are being found to be effective in retrofitting highway bridges for many different applications. This Corrosive environment can affect many bridge components but especially the ends of girders where expansion joints are present. Utah’s bridge girders experience corrosive environments from rain, snow, and salt for roadway de-icing. The Utah Department of Transportation was interested in how bridges with deteriorated AASTHO prestressed girders could be reinforced with shear. The focus of this research was to investigate how a CFRP fabric system can be applied for shear reinforcement to the deteriorated ends of I-shaped prestressed concrete girders.

The objectives of this research were accomplished by the destructive testing of eight full-scale, AASTHO I-shaped prestressed girders that were retrofitted in shear with a CFRP fabric system. The CFRP product was provided by The Chemical Company (BASF). There are inherent difficulties in applying CFRP to typical precast sections. To provide insight on how CFRP behaves on I-shape cross-sections, five different configurations of the CFRP fabric were tested. Of the five different configurations, two anchorage systems were implemented to increase the shear capacity of the CFRP. Each girder was then tested to failure in shear to quantify the increased shear capacity. During the load test, deflections, and strains were measured to provide conclusive evidence of the influence of the CFRP on shear capacity.
Another aspect of the research was to investigate how two different theoretical models that predicted the increase in shear capacity from the CFRP system. The first method evaluated to calculate $V_f$ is found in ACI 440.2R-8 entitled Guide for the “Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures”. The second method to evaluated $V_f$ is a method presented in a research paper entitled “FRP for Shear Strengthening of AASHTO Bridge Girders” by Hutchinson, Donald, and Rizkalla (1999). The calculated results for the two methods were then compared to the actual increased shear capacity.

5.2 Conclusions

The experimental program consisting of the load testing of five different CFRP reinforcement configurations was found to increase the shear capacity of the AASHTO I-shaped prestressed girders. The magnitude of the increased shear capacity was found to be highly dependent on the CFRP reinforcement configuration and anchorage system. The theoretical models effectiveness in predicting the increased shear capacity was also highly dependent on the CFRP reinforcement configuration and anchorage system. The CFRP reinforcement was able to allow for larger deflections before failure. From the strain measurements it was concluded that the CFRP fabric was not overstressed and failed due to debonding.

5.2.1 Effects of CFRP configurations on increased capacity

The increased shear capacity was highly dependent on the configuration of the CFRP reinforcement. CFRP has a more difficult time resisting shear forces of I-shaped
girders due to the large normal forces developed in the web to flange corners. In order to help resist these normal forces, two different anchorage systems were applied to four different CFRP sheet configurations. One girder was reinforced without an anchorage system to provide comparative results with the girders with anchorage systems.

Girder 4 was reinforced with diagonal (45 degrees) CFRP fabric stirrups but without an anchorage system. The result when loaded to failure was an increase of only 13.64 kips in shear capacity. Girder 3 had the same diagonal CFRP fabric stirrups as Girder 4 but had a horizontal strip of CFRP fabric applied over the diagonal strips for anchorage, resulting in an increased shear capacity of 39.69 kips which is 26.05 kips larger than Girder 4 without the anchorage. This is conclusive evidence that the horizontal anchorage system greatly increases the capacity of the CFRP reinforcement.

Girder 2 was reinforced with diagonal (45 degrees) CFRP fabric stirrups but with the inserted CFRP laminate anchorage system at the web to flange corner. The result when loaded to failure was an increase of 92.11 kips for the first test and a decrease of 0.87 kips of shear capacity for the second test. Girder 6 had the same anchorage system but had vertical wraps of CFRP fabric for the whole shear span. The result for the first test increased the shear capacity by 84.49 kips and decreased the shear capacity by 11.78 kips for the second test. Both of these configurations had the potential to have high increases in shear capacity but were found to be very sensitive to the anchorage system cutes and unreliable. The imbedded anchorage system which involved cutting a 1 inch slit into the girder at the web to flange corner weakened the girder for two of the four tests and is concluded to be the cause of a very sensitive system.
Girders 5 and 8 had vertical CFRP fabric stirrups and a horizontal strip of CFRP fabric over the vertical stirrups. This configuration was found to be the most reliable and consistent in increasing the shear capacity. The four tests on Girders 5 and 8 produced an average increased shear capacity of 55.70 kips. The CFRP reinforcement configuration on Girders 5 and 8 were also the easiest to apply due to its simplicity in design.

Overall, the CFRP fabric reinforcement was found to be successful in increasing the shear capacity of AASHTO prestressed I-shaped girders. The configuration on Girders 5 and 8, which consisted of vertical stirrups and a horizontal strip placed over the vertical stirrups for anchorage, was found to produce the largest consistent increase in shear capacity consistently. This configuration was also the easiest to apply and can be credited for its consistency. Therefore, this CFRP reinforcement configuration was found to be the most effective in increasing the shear capacity of AASHTO prestressed I-shaped girders.

5.2.2 Observations from theoretical models

The theoretical models for predicting the total shear capacity $V_{n}$, were found to be very conservative and can mainly be contributed to the conservatism in calculating $V_{c}$ and $V_{s}$, which made it more challenging to compare the two $V_{f}$ theoretical models. We were able to find conclusive evidence when comparing the actual $V_{f}$ against the two predictive models for $V_{f}$.

The ACI method overestimated Girders 3, 4, and 8 by 37.7%, 78.6%, and 13.18% respectively and underestimated Girders 2, 5, and 6 by 7.9%, 7.96% and 24.42% respectively. The Hutchinson, Donald, and Rizkalla (1999) method overestimated Girders
3, 4, and 8 by 23.3%, 73.65%, and 22.35%, respectively, and underestimated Girders 2, 5, and 6 by 1.75%, 0.84% and 22.35%, respectively. Both methods for predicting the shear contribution of the CFRP fabric were found to be conservative and over conservative for the same reinforcement configurations.

The CFRP reinforcement configuration on Girders 5 and 8 were found to be most consistent and reliable in increasing the shear capacity. When comparing the average actual shear capacity increase of 55.70 kips, the ACI estimated 56.88 kips which is only a 2.11% overestimation, while the Hutchinson, Donald, and Rizkalla (1999) method estimated 62.41 which is a 12.05% overestimation. Therefore, the ACI method was found to be the most accurate in predicting the increased shear capacity of AASHTO prestressed I-shaped girders with this configuration and anchorage system.

5.2.3 Observations from deflections

During each girder test of the shear load vs deflection, curves were monitored and compared against the unreinforced shear load vs deflection curves. The comparisons concluded that the CFRP reinforcement acted compositely with the girder and allowed for increased deflections. This provided a failure that was less brittle when loaded and failed in shear.

5.2.4 Observations from strains

During the load testing, strains were measured at various locations on the CFRP fabric that provided evidence that this external reinforcement was resisting the applied shear load. It was also observed that the maximum strain observed yielded a stress of half the maximum allowable stress in the CFRP C160 fabric. This provides evidence the
system failed due to delamination and concrete surface rupture. The CF130 fabric which is half the thickness of the CF160 fabric, would have also been adequate in providing a similar increased shear capacity for the I-shaped girders since its failure mechanism was not fiber rupture.

5.3 Recommendations for Application of CFRP Shear Reinforcement

This research was funded by the UDOT with the goal of finding a solution for increasing the shear capacity of deteriorated AASHTO prestressed I-shaped girders. The experimental program consisted of testing various forms of application of a CFRP fabric system provided by The Chemical Company (BASF). The following summary was found to be the most effective application for increased shear capacity of I-shaped girders retrofitted with a CFRP fabric system and what analytical model would best fit that configuration.

The simplest configuration to apply was also found to be the most effective in increasing the shear capacity. The recommended configuration was on Girders 5 and 8 which consisted of four vertical strips 10 inches wide spaced at 4.5 inches and a horizontal strip 15 inches in height and 63 inches in length placed over the vertical strips along the web (see Section 3.3.1 for detail). This configuration is very simply to apply which leaves little room for error, making it more reliable. The configurations with angled stirrups are harder to apply and since they cannot be continuous they must be overlapped on the bottom of the girder. The anchorage requiring a cut in the girder (Girders 2 and 6) made the system very sensitive and more difficult to apply, making the
system response uncertain. The recommended configuration was also found to be consistent over its four individual tests.

Another observation was that during testing, the highest observed stress in the CFRP CF160 fabric was approximately 297 ksi, which is roughly half the max stress of 550 ksi. This shows that the CF130 fabric which is half the thickness of the CF160 fabric could have been used and produced the same capacity. This would have also made the theoretical models more conservative but producing the same actual increased capacity. The recommendation that is proposed then is that for smaller girders where the depth or bond lengths are smaller the CF130 fabric would be sufficient but for girders with larger depths or bond lengths, the CF160 would be more effective.

The ACI method for calculating the predicted shear capacity of the CFRP, $V_f$, found in ACI 440.2R-8 entitled Guide for the “Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures” was found to be the most accurate for predicting the recommended CFRP reinforcement configuration. It overestimated the actual increased shear capacity by only 2.11% and with reduction factors it would fall below the design code requirements.
REFERENCES

ACI. 2008a. Committee 318. Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary. Farmington Hills, MI: American Concrete Institute, 465.

ACI. 2008b. Committee 440. Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures (ACI 440.2R-08) and Commentary. Farmington Hills, MI: American Concrete Institute, 465.


Appendix A

CFRP Properties and Bridge Plans
Figure A.1. Girder specifications from bridge plans (UDHSD 1967).
Figure A.2. Embedment anchorage detail.
**MBRACE® PRIMER**

Unique moisture tolerant epoxy primer for the MBrace Composite Strengthening System

### DESCRIPTION

The MBrace Primer is a clear, low viscosity, 100% solids epoxy compound based on a unisil, adduct curing technology. The technology results in superior tolerance for surface moisture, curing at temperatures as low as 2°C and stability to cure in the presence of moisture.

MBrace Primer is tolerant to a wide variety of field conditions. When applied to concrete, the surface is upgraded to give high tensile bond strength to the system being used.

### TYPICAL PERFORMANCE DATA

- Adhesion strength on glass fibre, N/mm²: 2.62 (ASTM D4541-95(e1))
- Adhesion strength on carbon, N/mm²: 2.87 (ASTM D4541-95(e1))
- Tensile strength, MPa (ASTM D638-05): 35
- Tensile strain at yield, % (ASTM D638-05): 1.8
- Tensile modulus, MPa (ASTM D638-05): 2,097
- Flexural strength, MPa (ASTM D790-01): 55
- Flexural modulus, MPa (ASTM D790-01): 1,672
- Compressive strength, MPa (ASTM D695-95): 73
- Compressive modulus, MPa (ASTM D695-95): 2,320

### FEATURES AND BENEFITS

- Clear: Non-staining
- Adhesion: Improves adhesion of subsequent coatings to substrates

### APPLICATION

**Surface Preparation**

Proper substrate preparation is critical for optimum performance. The prepared surface should be structurally sound and free from contaminants such as oil, grease, curing membrane, previous coatings, dust, fungus, moss, etc.

Depending on the substrate condition and environmental requirements, use an effective method recommended by ICRI: Guideline No. 03732 for selecting and specifying concrete surface preparation for sealers, coatings and polymer overlays.

### PROPERTY

<table>
<thead>
<tr>
<th>Type</th>
<th>Part A</th>
<th>Part B</th>
<th>Mixed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Colour</td>
<td>Amber</td>
<td>Clear</td>
<td>Amber</td>
</tr>
<tr>
<td>VOC Content</td>
<td>3.22%</td>
<td>1.49%</td>
<td></td>
</tr>
<tr>
<td>Flash point</td>
<td>95°C</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mixing ratio</td>
<td>3 to 1</td>
<td>100 to 60</td>
<td></td>
</tr>
</tbody>
</table>

### WEIGHT/VOLUME

<table>
<thead>
<tr>
<th>Type</th>
<th>Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>Part A</td>
<td>1.139 g/L</td>
</tr>
<tr>
<td>Part B</td>
<td>1.095 g/L</td>
</tr>
<tr>
<td>Mixed</td>
<td>1.120 g/L</td>
</tr>
</tbody>
</table>

**Figure A.3.** Primer specifications page 1.
# MBRACE® PRIMER

## Primer

**Mixing:**
3 volumes of MBRace Primer Part A to 1 volume of MBRace Primer Part B. Blend with a mechanical mixer for at least 3 minutes or until it is homogeneous. Mixing time may be adjusted according to the temperature during application.

**Painting**:
Apply the MBRace Primer to the intended substrate using a brush or short nap roller. Spray application of MBRace Primer is not recommended.

## ESTIMATING DATA

- **Concrete**: 0.10 - 0.25 kg/m²
- Coverage may vary depending on the density, texture, and porosity of concrete.

## Packaging

MBRace Primer is supplied in 1 kg and 6 kg packs with the individual weights of the components as below:

<table>
<thead>
<tr>
<th></th>
<th>1 kg set</th>
<th>6 kg set</th>
</tr>
</thead>
<tbody>
<tr>
<td>Part A</td>
<td>0.625 kg</td>
<td>3.75 kg</td>
</tr>
<tr>
<td>Part B</td>
<td>0.374 kg</td>
<td>2.25 kg</td>
</tr>
</tbody>
</table>

## Shelf Life

MBRace Primer can be kept for 12 months from date of manufacture if stored in original unopened packaging in a dry enclosed place at temperatures at 21°C without exposing to direct sunlight.

## Precautions

MBRace Primer contains reactive resins and diuretics. Observe the following health and physical precautionary measures before using this product:

- Wear gloves, mask, eye protection, apron or overalls and appropriate work clothing while handling the product. Wash thoroughly after handling. Should skin contact occur, wash immediately with soap and water, or an effective hand cleaner. In case of accidental eye contact, wash with copious quantity of water and seek medical help immediately. If ingested, do not induce vomiting. Consult doctor immediately. Ventilation is required with special consideration for enclosed or confined areas.
- Air movement must be designed to ensure turnover at all locations in work and adjacent areas to avoid build up of heavy vapours.

For detailed Health, Safety and Environmental Recommendations, please refer to all instructions on the product Material Safety Data Sheet.

---

**Statement of Responsibility**

The technical information and application advice given in this BASF Construction Chemicals publication are based on the present state of our best scientific and practical knowledge. As the information herein is of a general nature, no assumption can be made as to a product's suitability for a particular use or application and no warranty as to its accuracy, reliability or completeness either expressed or implied is given other than those required by law. The user is responsible for checking the suitability of products for their intended use.

**Note**

Field service where provided does not constitute supervisory responsibility. Suggestions made by BASF Construction Chemicals either orally or in writing may be followed, modified or rejected by the owner, engineer or contractor since they, and not BASF Construction Chemicals, are responsible for carrying out procedures appropriate to a specific application.

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Figure A.4. Primer specifications page 2.
Figure A.5. Putty specifications.
MBRACE® SATURANT

Impregnation resin for the MBrace Composite Strengthening System

DESCRIPTION

MBRAE SATURANT is a 100% solid, low viscosity epoxy material that is used to impregnate MBrace Fibre Reinforcement Sheets. Cured with the fibre sheet.

MBRAE SATURANT resin produces a high performance composite system for use in external structural repair or upgrade applications.

FIELDS OF APPLICATION

Impregnation of MBrace Fibre Reinforcement sheets to produce a high performance MBrace composite system.

FEATURES AND BENEFITS

Coloured to ensure full coverage of fibre sheets
Performance improves strength of MBrace composite system

TYPICAL PERFORMANCE DATA

Flexural strength, MPa (ASTM D790:01) : 62
Flexural modulus, MPa (ASTM D790:01) : 2,123
Compressive strength, MPa (ASTM D696:96) : 80
Compressive modulus, MPa (ASTM D696:96) : 2,006
Tensile strength, MPa (ASTM D638:02) : 29
Tensile strain at yield, % (ASTM D638:02) : 1.91
Tensile modulus, MPa (ASTM D638:02) : 2,400

APPLICATION

Surface Preparation
Proper substrate preparation is critical for optimum performance. The prepared surface should be structurally sound and free from contaminants such as oil, grease, curing membrane, previous coatings, dust, fungus, mass, etc.

Depending on the substrate condition and environmental requirements, use an effective method recommended by ICR Guideline No. 03732 for selecting and specifying concrete surface preparation for sealers, coatings and polymer overlays.

Flash Point (Pensky-Martin, closed cup):

| Saturant Part A | 110°C |
| Saturant Part B | >100°C |

Mixing ratio:

<table>
<thead>
<tr>
<th>Part A</th>
<th>Part B</th>
</tr>
</thead>
<tbody>
<tr>
<td>By volume</td>
<td>62.5 : 37.5</td>
</tr>
<tr>
<td>By weight</td>
<td>52.5 : 37.5</td>
</tr>
</tbody>
</table>

Weight/Volume:

| Part A | Approx. 950 g/L |
| Part B | Approx. 950 g/L |
| Mixed | Approx. 950 g/L |

Maximum non sag thickness: 625 μm

Mixed viscosity:

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Viscosity</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 °C</td>
<td>Approx 4,000 cPs</td>
</tr>
<tr>
<td>25 °C</td>
<td>Approx 3,000 cPs</td>
</tr>
<tr>
<td>32 °C</td>
<td>Approx 2,000 cPs</td>
</tr>
</tbody>
</table>

Working Time, based on 3.8L sample:

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Pot-Life</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 °C</td>
<td>100 min</td>
</tr>
<tr>
<td>25 °C</td>
<td>24 min</td>
</tr>
<tr>
<td>32 °C</td>
<td>15 min</td>
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</tbody>
</table>

PROPERTIES

<table>
<thead>
<tr>
<th>Generic Type</th>
<th>100% solids amine-cured liquid epoxy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Colour</td>
<td>Part A: Blue</td>
</tr>
</tbody>
</table>

VOC Content:

BS55: Part B2: Approx. 1.0%
MBRACE® SATURANT

Mixing
Mix 62.5% of MBRACE SATURANT Part A with 37.5% of MBRACE SATURANT Part B. Blend with a mechanical mixer for at least 3 minutes until it is homogeneous. Mixing time may be adjusted according to the temperature during application.

Note: Mix sufficient material to be used as once the viscosity increases, gelation takes place and the product will lose its adhesion performance. Do not use high speed mixing as this entraps air.

Keep material cool and shaded from direct sunlight in warm weather. Work time can be extended by resealing material cool before and after mixing. Do not freeze or chill the material.

Pasting
Apply the mix MBRACE SATURANT to the MBrace Fibre Reinforcement Sheets until it is properly wet-out to ensure it is fully saturated. The appearance of the MBRACE SATURANT, when applied by roller should be translucent blue. The colour variation could be due to application technique, fibre orientation and thickness fluctuations.

ESTIMATING DATA
0.3-0.7 kg/m² per layer of fibre sheet depending on type of fibre.

PACKAGING
10kg set: 6.25 kg of Part A
2.75 kg of Part B

SHELF LIFE
MBRACE SATURANT can be kept for 24 months from date of manufacture if stored in original unopened packaging, in a dry enclosed place at temperatures at 25°C without exposing to direct sunlight.

PRECAUTIONS
MBRACE SATURANT contains reactive resins and diluents. Observe the following health and safety precautionary measures before using this product.

Wear gloves, mask, eye protection, barrier creams and appropriate work clothing while handling the product.

Wash thoroughly after handling. Should skin contact occur wash immediately with soap and water, or an effective hand cleaner. In case of accidental eye contact wash with copious quantity of water and seek medical help immediately. If ingested, do not induce vomiting. Consult doctor immediately. Ventilation is required with special consideration for enclosed or confined areas.

Air movement must be designed to ensure turnover of all locations in work and adjacent areas to avoid build-up of heavy vapours.

For detailed Health, Safety and Environmental recommendations, please consult or follow all instructions on the product Material Safety Data Sheet.

STATEMENT OF RESPONSIBILITY
The technical information and application advice given in this BASF Construction Chemicals publication are based on the present state of our best scientific and practical knowledge. As the information herein is of a general nature, no assumption can be made as to the product's suitability for a particular use or application and no warranty as to its accuracy, reliability or completeness either expressed or implied is given other than those required by law. The user is responsible for testing the suitability of products for their intended use.

NOTE: Field service where provided does not constitute supervisory responsibility. Suggestions made by BASF Construction Chemicals either orally or in writing may be followed, modified or rejected by the owner, engineer or contractor since they, and not BASF Construction Chemicals, are responsible for carrying out procedures appropriate to a specific application.

Figure A.7. Saturant specifications page 2.
MBRACE® CF 160

Unidirectional high strength carbon fiber fabric for the Mbrace® Composite Strengthening System

**Features**
- High strength to weight ratio
- Excellent resistance to creep and fatigue
- Extremely durable
- Easy installation
- Low aesthetic impact

**Benefits**
- Can add significant strength to a structure without adding significant dead load
- Withstands sustained and cyclic load conditions
- Extremely resistant to a wide range of environmental conditions
- Can be installed quickly, even in areas of limited access
- Easy to conceal, will not significantly change existing member dimensions, will form around complex surfaces

**Shear**
- Store in a cool, dry area (50 to 90°F [10 to 32°C]) away from direct sunlight, flame or other hazards

**Where to Use**
- Increase load bearing capacity of concrete beams, slabs, walls and columns
- Restore structural capacity to damaged or deteriorated concrete structures
- Increase the strength of concrete pipes, alas, tanks, chimneys and bridges
- Substitute reinforcing steel mistakenly omitted in the construction of concrete and masonry structures
- Improve the seismic ductility of concrete columns
- Improve the seismic response of concrete beam-column connections, shear walls and re-fill walls
- Improve the blast resistance of concrete and masonry structures
- Strengthening of some steel and timber structures

**LOCATION**
- Vertical
- Horizontal
- Exterior
- Interior

**SUBSTRATE**
- Concrete
- Masonry
- Timber
- Steel

**PRODUCT DATA**

Mbrace® CF 160 is a dry fabric constructed of very high strength, aerospace grade carbon fibers. These fabrics are applied onto the surface of existing structural members in buildings, bridges, and other structures using the Mbrace® family of performance polymers. The result is an externally bonded FFP (fiber reinforced polymer) reinforcement system that is engineered to increase the strength and structural performance of these members. Once installed, the Mbrace® System delivers externally bonded reinforcement with outstanding long-term physical and mechanical properties.

Mbrace® CF 160 is twice the thickness of Mbrace® CF 130. Two layers of Mbrace® CF 130 can be replaced with one layer of Mbrace® CF 160.

**Yield**
- 200 MPa per in²

**Packaging**
- Available in rolls: 20 lb (900 mm) wide

**SIZES**
- Roll Width (in) 20 (900 mm)
- Roll Length (in) 60 (1500 mm)
- Roll (m) 0.5 (50 m)
- Roll (m²) 25 (50 m)
- Roll (m³) 0.5 (50 m³)

**Color**
- Black

**Maintenance of Concrete**

Figure A.8. CF 160 fabric specification page 1.
<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fiber Material</td>
<td>High Strength Carbon</td>
</tr>
<tr>
<td>Fiber Tensile Strength</td>
<td>7200 ksi (500 MPa)</td>
</tr>
<tr>
<td>Density Weight</td>
<td>0.134 oz/ft² (4.03 g/m²)</td>
</tr>
<tr>
<td>Fabric Width</td>
<td>20 in (500 mm)</td>
</tr>
<tr>
<td>Nominal Thickness, $t^*$</td>
<td>0.003 in (0.076 mm)</td>
</tr>
<tr>
<td>Functional Properties</td>
<td></td>
</tr>
<tr>
<td>$e$</td>
<td>0.21 - 109 rad/° (0.39 - 10⁻⁹ rad/°)</td>
</tr>
<tr>
<td>Tensile Properties</td>
<td></td>
</tr>
<tr>
<td>Ultimate Tensile Strength</td>
<td>35000 psi (240 MPa)</td>
</tr>
<tr>
<td>Tensile Modulus, $E_{11}$</td>
<td>237 GPa</td>
</tr>
<tr>
<td>Ultimate Tensile Strength per Unit Width, $f_{11}$</td>
<td>7.14 kips/in² (51.25 kN/m²)</td>
</tr>
<tr>
<td>Tensile Modulus per Unit Width, $E_{11}$</td>
<td>1450 kips/in² (10.082 MN/m²)</td>
</tr>
<tr>
<td>Ultimate Young's Modulus, $E_{11}$</td>
<td>1.47%</td>
</tr>
</tbody>
</table>

Figure A.9. CF 160 Fabric Specifications Page 2.
PRODUCT DATA

MBRACE® TOPCOAT ATX
Protective acrylic topcoat for the MBrace® Composite Strengthening System

Description
MBRACE® Topcoat ATX is a protective coating for use with the MBrace® Composite Strengthening System. This "cementitious"-gray topcoat conceals the MBrace® system and protects the system from ultraviolet radiation and mild abrasion. A unique component, high alkali technology, MBrace® Topcoat ATX can be installed quickly, safely, and economically.

Yield
Coverage Rate (approximately):
One Coat: 80 to 100 ft²/gal
(1.94 to 3.43 m²/L)

Packaging
Available in 5 gallon (18.9 L) units.

Color
Concrete gray

Features
- Low VOC's
- High build finish
- Suitable for low temperature application
- Abasion resistant
- Color and texture mimics concrete substrates

Benefits
- Environmentally friendly
- Adequate coverage with one coat
- Can be applied at temperatures down to 50°F (10°C)
- Extends application window in cooler climates
- Minimizes maintenance and recoating
- Conceals the MBrace® system on concrete substrates

Shell Life
One year properly stored in unopened containers

Storage
Store in a cool, dry place (50 to 90°F [10 to 32°C]) away from direct sunlight, flammable, or other hazards

Where to Use
APPLICATION
- Coating the MBrace® system in most service conditions

LOCATION
- Vertical
- Horizontal
- Exterior
- Interior

SUBSTRATE
- Concrete
- Masonry
- Steel

How to Apply
Surface Preparation
1. MBrace® Topcoat ATX should be applied as the final component of the MBrace® system.

2. It should be applied to the outermost layer of MBrace® Saturation or Saturation TTC only after the saturation has cured but not more than 48 hours after the application of the saturation.

3. The surface of the saturation should be clean and dry.

Application
Time to Membrane
47°F (8°C): 6 to 8 hours

MBrace® Topcoat ATX can be applied by brush or roller for small areas or by spray application for large areas.
- Brush: Use stiff fiber or nylon short fillet brush.
- Spray: Consult with manufacturer on spray application equipment.

Clean Up
Use warm soap water to clean brushes, rollers, and other tools.

Maintenance
Periodically inspect the applied material and repair localized areas as needed. Consult a BASF representative for additional information. Visit us on the web for the most current product information and news: www.buildingsystems.basf.com.

Figure A.10. Topcoat specifications page 1.
For Best Performance
- Only apply MiraSeal® Topcoat AT when the ambient temperature is between 65 and 100°F (18 and 38°C).
- The MiraSeal® Topcoat AT should be applied within 48 hours of applying the undercoat. Failure to do so may affect bond strength.
- The system is designed for application under controlled conditions. In the event of misapplication, the paint system may not perform as expected.
- MiraSeal® Topcoat AT is a water-based acrylic latex paint specifically formulated for use with the MiraSeal® Basecoat System. It is recommended for use in new construction and renovation applications.

Health and Safety
- MiraSeal® Topcoat AT: HAZARDOUS MATERIALS, CAUTION.
- Risk: Sensitization, skin and eye irritant. May cause skin and eye irritation. Do not inhale. Do not store in a tightly closed container. Do not use in the presence of asbestos or other hazardous materials.
- First Aid: In case of contact with skin, wash thoroughly with soap and water. In case of eye contact, flush for at least 15 minutes. Seek immediate medical attention. Do not inhale. Do not use in the presence of asbestos or other hazardous materials. This product contains a material which is a known or suspected cause of cancer, birth defects or other reproductive harm.

VOC Content: 220 g/L or 1.5 lb/gal less water and exempt solvents.

Product Material Safety Data Sheets (MSDS) are available and should be obtained and reviewed by all personnel before handling or storage. These products are for professional and industrial use only and are not intended for home use. The information in this document is intended to provide general guidance and is not a substitute for professional advice. It is the responsibility of the user to determine the suitability of the product for a particular application. BASF cannot be held responsible for any errors or omissions in this document.
MBrace®
Fabric Systems

The following installation procedure is should be fully understood prior to beginning any work. To ensure proper installation and performance of the system, the following actions must be completed by the installer.

1) Carefully read and understand installation procedure. Contact our Technical Service Department at (800) 243-6739 for product assistance.

2) Inspect all shipments and materials for missing or damaged components. Contact Customer Service at (800) 443-9517 with your BBS order number and invoice for prompt assistance.

3) Inspect substrate or adjacent construction for acceptance before beginning work. Report unacceptable construction to the project manager for scheduled repair work.

4) Review BASF Building Systems working drawings for project specific detailed information if available.

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Figure A.12. Application instructions page 1.
Installation Procedure

1. Repair to deteriorated concrete per ICRI guidelines prior to installation of the MBrace® system.

2. Utilizing a grinder with a masonry disc, contractor must round any corners that the MBrace® fabric will wrap around to a minimum radius of .50” (13mm).

3. Prepare surface to a minimum profile of ICRI CSP 3. Remove all dust, dirt, laitance and bond inhibiting compounds.

4. Carefully layout the area of the structure that is to be reinforced with the MBrace® System according to BASF working drawings for the project or project plans. Measure the MBrace® Fabric and cut into appropriately sized strips using heavy duty scissors or utility knife.

Note: When measuring for fabric length, consider the number and length of lap splices.
5 After the concrete surface has been prepared, contractor shall inspect existing cracks and epoxy inject all cracks greater than .010" wide before installation of MBrace®.

6 Inspect all surface conditions where MBrace® will be installed. Review environmental conditions and second inspection results on a daily field report.

Notes: Do not proceed with application of MBrace® if any of the following conditions apply:
1. Temperatures are above 120°F or below 40°F
2. Prepared surface is saturated with water
3. Potential water leakage

7 After the surface has been prepared, the 2 part MBrace® Primer can be mixed.

Mixing Notes:
1. The mix ratio is: 3 units of Part A to 1 unit of Part B (By Volume)
2. Use a low speed drill with an appropriate mixing paddle to mix the combined components for 5-min

Working time for the mixed Primer is approximately 20-min at 77°F (25°C).

8 Using a brush or paint roller, apply primer to properly prepared substrate. Typical coverage rates are approximately 150 s.f. - 200 s.f./gallon.

9 After the Primer has been applied to the structure, the 2 part MBrace® Putty can be mixed.

Mixing Notes:
1. Premix Part A using a low speed drill with an appropriate mixing paddle for 3 minutes
2. The mix ratio is: 3 units of Part A to 1 unit of Part B (By Volume)
3. Use a low speed drill with an appropriate mixing paddle to mix the combined components for 3-min
4. Optional - Add silica powder (Cab-O-Sil, Si-11 powder or similar) until desired consistency is achieved.

Silica powder adds body to the putty for warmer temperatures.

Working time for the mixed Putty is approximately 40-min at 77°F (25°C).
10. Apply MBrace® Putty to the primed substrate utilizing a trowel. Use a light trowel technique, only filling low areas and voids in the substrate. MBrace® putty can be applied immediately following application of the MBrace® Primer or up to 24 hrs after application of the MBrace® Primer.

11. After the Putty has been applied to the structure, the 2 part MBrace® Saturant can be mixed.
   **Mixing Notes:**
   1. Premix Part A using a low speed drill with an appropriate mixing paddle for 3 minutes
   2. The mix ratio is: 3 units of Part A to 1 unit of Part B (by volume)
   3. Use a low speed drill with an appropriate mixing paddle to mix the combined components for 3-min

   Working time for the mixed Saturant is approximately 45-min at 77°F (25°C).

12. After MBrace® Saturant is mixed, apply Saturant to substrate with a 3/8" Nap roller in areas where MBrace® Fabric will be applied. Coverage rates for MBrace® Saturant is approximately 35 to 55 Sq Ft of fabric depending on type of fabric being used. Saturant should be applied to a Wet film thickness of 18-20 mils.
   **Note:** Saturant can be applied immediately after application of the MBrace® Putty or up to 24 hrs after application of the Putty.

13. Once first coat of MBrace® Saturant is applied to substrate, apply dry Fabric to saturated substrate. Fabric must be applied while saturant is still wet. At splice locations, lap MBrace Fabric in the direction of fiber strands. Press dry fabric onto substrate by squeezing and or hand in direction of Fibers only.
14 Using a Laminating (Rib) Roller tool, roll fiber/epoxy composite in direction of the fiber. Start rolling from the middle and work air bubbles toward outside edge. Roll until visible signs of saturant bleeding through the fabric are seen.

15 Mix MBrace® saturant as per Step 11 and apply a second coat of Saturant to the composite fibers in same manner as Step 12. This is done after all air bubbles have been removed from the fabric. Second layer of Saturant should be applied to a Wet film thickness of 18-23 mils.

16 If additional layers of fabric are required, repeat Steps 11 thru 15.

Topcoat Preparation and Information:
Before any Top coat material is applied, the saturant must be cured to a tack-free state. If the Saturant has cured for more than 24 hrs, the surface should be roughened with 100-grit sandpaper to ensure proper adhesion of top coat material to completed MBrace system.

Urethane Topcoats: Mix 4 Parts A to 1 Part B (By Volume) for 5 minutes. Pot Life of mixed components is Approximately 3 hours at 77°F (25°C).

ATX - UV Protection: Acrylic one part top coat material. The coverage rate of ATX Top coat is approximately 80 to 100 Sq. Ft / Gallon (One Coat). Installation temperatures should be between 35°F (2°C) and 100°F (38°C). ATX Top coat should be applied within one week of outermost layer of saturant.

FRL - Fire Retardent Top coat: Two coats should be applied to achieve adequate flammability protection. Installation temperatures should be between 60°F (16°C) and 120°F (50°C). FRL Top coat shall be applied within one week of outermost layer of saturant.
Figure A.17. Application instructions page 6.
Appendix B

Load, Strain, and Deflection Charts
Figure B.1. Test 1A Load vs Time.

Figure B.2. Test 1A Strain Gauge 1.
Figure B.3. Test 1A Strain Gauge 2.

Figure B.4. Test 1A Strain Gauge 3.
Figure B.5. Test 1A Strain Gauge 4.

Figure B.6. Test 1A Strain Gauge 5.
Figure B.7. Test 1A Strain Gauge 6.

Figure B.8. Test 1A Strain Gauge 7.
Figure B.9. Test 1A Strain Gauge 8.

Figure B.10. Test 1A Strain Gauge 9.
Figure B.11. Test 1A Strain Gauge 10.

Figure B.12. Test 1A Strain Gauge 11.
Figure B.13. Test 1A Strain Gauge 12.

Figure B.14. Test 1A Strain Gauge 13.
Figure B.15. Test 1B Load vs Time.

Figure B.16. Test 1B Load vs Deflection.
Figure B.17. Test 1B Strain Gauge 1.

Figure B.18. Test 1B Strain Gauge 2.
Figure B.19. Test 1B Strain Gauge 3.

Figure B.20. Test 1B Strain Gauge 4.
Figure B.21. Test 1B Strain Gauge 7.

Figure B.22. Test 1B Strain Gauge 8
Figure B.23. Test 1B Strain Gauge 9.

Figure B.24. Test 1B Strain Gauge 10.
Figure B.25. Test 1B Strain Gauge 11

Figure B.26. Test 1B Strain Gauge 12.
Figure B.27. Test 2A Load vs Time.

Figure B.28. Test 2A Load vs Deflection.
Figure B.29. Test 2A Strain Gauge 1.

Figure B.30. Test 2A Strain Gauge 2.
Figure B.31. Test 2A Strain Gauge 3.

Figure B.32. Test 2A Strain Gauge 4.
Figure B.33.  Test 2A Strain Gauge 5.

Figure B.34.  Test 2B Load vs Time.
Figure B.35. Test 2B Load vs Deflection.

Figure B.36. Test 2B Strain Gauge 1.
Figure B.37. Test 2B Strain Gauge 2.

Figure B.38. Test 2B Strain Gauge 3.
Figure B.39. Test 2B Strain Gauge 4.

Figure B.40. Test 2B Strain Gauge 5.
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Figure B.42. Test 3A Load vs Deflection.
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Figure B.44. Test 3A Strain Gauge 2.
Figure B.45. Test 3A Strain Gauge 3.

Figure B.46. Test 3A Strain Gauge 4.
Figure B.47. Test 3A Strain Gauge 5.

Figure B.48. Test 3A Strain Gauge 6.
Figure B.49. Test 3A Strain Gauge 7.

Figure B.50. Test 4A Load vs Time.
Figure B.51. Test 4A Load vs Deflection.

Figure B.52. Test 4A Strain Gauge 1.
Figure B.53. Test 4A Strain Gauge 2.

Figure B.54. Test 4A Strain Gauge 3.
Figure B.55. Test 4A Strain Gauge 4.

Figure B.56. Test 4A Strain Gauge 5.
Figure B.57. Test 4A Strain Gauge 6.

Figure B.58. Test 4A Strain Gauge 7.
Figure B.59. Test 4B Load vs Time.

Figure B.60. Test 4B Load vs Deflection.
Figure B.61. Test 4B Strain Gauge 1.

Figure B.62. Test 4B Strain Gauge 2.
Figure B.63. Test 4B Strain Gauge 3.

Figure B.64. Test 4B Strain Gauge 4.
Figure B.65. Test 4B Strain Gauge 5.

Figure B.66. Test 4B Strain Gauge 6.
Figure B.67. Test 4B Strain Gauge 7.

Figure B.68. Test 5A Load vs Time.
Figure B.69. Test 5A Load vs Deflection.

Figure B.70. Test 5A Strain Gauge 1.
Figure B.71. Test 5A Strain Gauge 2.

Figure B.72. Test 5A Strain Gauge 3.
Figure B.73. Test 5A Strain Gauge 4.

Figure B.74. Test 5A Strain Gauge 5.
Figure B.75. Test 5A Strain Gauge 6.

Figure B.76. Test 5A Strain Gauge 7.
Figure B.77. Test 5A Strain Gauge 8.

Figure B.78. Test 5A Strain Gauge 9.
Figure B.79. Test 5B Load vs Time.

Figure B.80. Test 5B Load vs Deflection.
Figure B.81. Test 5B Strain Gauge 1.

Figure B.82. Test 5B Strain Gauge 2.
Figure B.83. Test 5B Strain Gauge 3.

Figure B.84. Test 5B Strain Gauge 4.
Figure B.85. Test 5B Strain Gauge 5.

Figure B.86. Test 5B Strain Gauge 6.
Figure B.87. Test 5B Strain Gauge 7.

Figure B.88. Test 5B Strain Gauge 8.
Figure B.89. Test 5B Strain Gauge 9.

Figure B.90. Test 6A Load vs Time.
Figure B.91. Test 6A Strain Gauge 1.

Figure B.92. Test 6A Strain Gauge 2.
Figure B.93. Test 6A Strain Gauge 3.

Figure B.94. Test 6A Strain Gauge 4.
Figure B.95. Test 6A Strain Gauge 5.

Figure B.96. Test 6A Strain Gauge 6.
Figure B.97. Test 6A Strain Gauge 7.

Figure B.98. Test 6B Load vs Time.
Figure B.99. Test 6B Load vs Deflection.

Figure B.100. Test 6B Strain Gauge 1.
Figure B.101. Test 6B Strain Gauge 2.

Figure B.102. Test 6B Strain Gauge 3.
Figure B.103. Test 6B Strain Gauge 4.

Figure B.104. Test 6B Strain Gauge 5.
Figure B.105. Test 6B Strain Gauge 6.

Figure B.106. Test 6B Strain Gauge 7.
Figure B.107. Test 7A Load vs Time.

Figure B.108. Test 7A Load vs Deflection.
Figure B.109. Test 7B Load vs Time.

Figure B.110. Test 7B Load vs Deflection.
Figure B.111. Test 8A Load vs Time.

Figure B.112. Test 8A Deflection vs Load.
Figure B.113. Test 8B Load vs Time.

Figure B.114. Test 8B Load vs Deflection.