Feasibility Study Of Evaluating Durability Of Cfrp-strengthened Beams Using In-situ Load Test

Carlos Turizo-Rico
University of Central Florida

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FEASIBILITY STUDY OF EVALUATING DURABILITY OF CFRP-STRENGTHENED BEAMS USING IN-SITU LOAD TEST

by

CARLOS A. TURIZO
B.S. Universidad Nacional de Colombia, 2003

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

Fall Term
2006
ABSTRACT

In Florida a number of highway bridges were retrofitted on their reinforced concrete (RC) girders with carbon-fiber reinforced polymers (CFRP) during the 1990’s. Their conditions, after being in service for approximately 10 years, are of significant interest to the State’s highway authority, as well as researchers in the region.

This paper will evaluate if a load test on one of such bridges, which was retrofitted with CFRP at the girders in the splash-zone and thus was subjected to severe environmental conditions, is a feasible technique to evaluate the actual condition of the CFRP. A 3-dimensional Finite Element Model (FEM) was utilized to assess the load-deflection behavior of the bridge. An analytical study was used to evaluate the effective moment of inertia of the strengthened beams modeled on the FEM.

The results indicate that the deflection change due to the amount of CFRP sheets assumed to be effective on the beam is insignificant. The paper also shows that it would not be feasible to estimate changes in the properties in the CFRP based only on deflection and strain measurements.
To Andrea
ACKNOWLEDGMENTS

I would like to express my appreciation to my advisor, Dr. Lei Zhao, for his direction and encouragement during my study.

I would like to thank my family for their unconditional support.
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### LIST OF ABBREVIATIONS

\[ E_c = \] concrete modulus of elasticity, GPa

\[ E_s = \] steel modulus of elasticity, GPa

\[ E_{FRP} = \] CFRP modulus of elasticity, GPa

\[ \varepsilon_y = \] steel yield strain, mm/mm

\[ \varepsilon_{FRP} = \] CFRP strain, mm/mm

\[ f'c = \] concrete strength, MPa

\[ f_{FRP} = \] CFRP tensile strength, MPa

\[ f_y = \] concrete strength, MPa

\[ G = \] concrete shear modulus of elasticity

\[ I_{ACI} = \] effective moment of inertia calculated with the ACI Equation, mm\(^4\)

\[ I_e = \] effective moment of inertia, mm\(^4\)

\[ I_{ey} = \] effective moment of inertia of the beam at first yielding, mm\(^4\)

\[ I_{cr} = \] cracked transformed moment of inertia, mm\(^4\)

\[ I_g = \] gross moment of inertia, mm\(^4\)

\[ M = \] service moment of the beam, kN.m

\[ M_{cr} = \] cracking moment of beam, kN.m

\[ M_{\text{max}} = \] maximum applied moment, kN.m

\[ M_y = \] first yield (of steel reinforcement) moment, kN.m

\[ t_{FRP} = \] CFRP thickness, mm

\[ v = \] Poisson’s ratio
CHAPTER ONE: INTRODUCTION

In many countries, most of the bridge infrastructure was constructed in the mid 20th century and is approaching the end of its expected lifespan. To keep the ageing bridges functional and safe, there is a need for fast, efficient, and durable strengthening and rehabilitation methods. Bonding carbon-fiber-reinforced polymer (CFRP) composite sheets or laminates on reinforced concrete (RC) structures have been used to provide flexural and shear strengthening to beams and better confinement to concrete in columns (Gheorghiu et al, 2004).

Chloride-induced steel reinforcement corrosion is one of the major deterioration problems for reinforced concrete structures. The volume of the rust product is about 5 times larger than that of the original iron (Masoud et al, 2001). The volume expansion at the surface of the rebar causes longitudinal cracking in the concrete cover, which could spall off if severe corrosion progresses. Structural capacity could be reduced due to loss of the bond at the steel-concrete interface and loss of the reinforcing cross sectional area.

The short-term behavior of RC beams strengthened with CFRP laminates has been widely investigated and well documented. Experimental and analytical investigations demonstrated that CFRP can show significant increase in flexural strength. The increased flexural capacity can be as high as three times the beam’s original strength, depending on factors such as reinforcing steel ratio, concrete compressive strength, CFRP amount and properties, and the level of damage in the beam (Masoud et al, 2001). The American Concrete Institution (ACI) Committee 440 has developed design guidelines for external strengthening of concrete structures using CFRP.
In some states in the US, this strengthening technique has been used on bridge girders to restore or strengthen reinforced concrete beams that had suffered corrosion induced damages. In Florida, this practice began as early as 1994. After more than a decade in service, the durability of these strengthened beams needs to be evaluated, especially at the concrete-to-CFRP interface. Many techniques have been proposed including visual inspection, thermography, in-situ sample pull-off, and load testing.

Load testing is a standard and routine approach that the Florida Department of Transportation (DOT) uses to evaluate the conditions of typical bridges, in which response parameters such as deflections, strains, and accelerations are measured under known truck loads. Changes in response recorded over time are correlated to changes in projected strengths of a bridge.

Section analysis of RC beams with CFRP strengthening indicated that, while small amounts of CFRP can significantly increase a RC beam’s flexural strength, its impact on the beam’s stiffness under service load is limited. This paper presents the results of a parametric study using a finite element analysis, in which the deflection and strain of a CFRP-strengthened section of a bridge were correlated with the thickness of the CFRP strengthening sheets, which could be used to simulate the residual effectiveness of the CFRP sheets.

Field load tests on RC bridges strengthened with CFRP have been performed to study the effect on the beam’s stiffness. The stiffness of the FRP systems (Shahrooz and Boy, 2004) was small in comparison to the stiffness of the bridge deck and accordingly, the measured deflections did not change noticeably after retrofitting. A study presented by Klaiber and Widf (2003) showed how
the CFRP reduced the beam deflections from 3 mm to 2.54 mm (18 % decrease). In another study, Catbas et al (2006) tested and evaluated the performance of RC girders before and after retrofitting. The maximum deflection before and after retrofitting was 11.6 mm and 10 mm (16% average decrease) respectively.

This study is not intended to be a generalize tool to predict the effectiveness of a load test to evaluate the durability of the CFRP, however this approach can be extrapolated to study beams with other design parameters.

The results of the parametric study indicated that the deflection and strain changes are minimal when the number of layers of a commonly used fabric increased from one to five. In an in-situ load test, other factors, which include temperature, ambient vibration, etc., could also cause deflection changes up to the same order of magnitude (less than 1mm). Therefore it is concluded that it is not feasible to correlate the deflection of a bridge to the durability of the CFRP sheets. Efforts should be placed more on other evaluation and inspection techniques.
CHAPTER TWO: BRIDGE DESCRIPTION

The bridge under investigation has six 10.97-m-long spans. The original bridge was built between 1945 and 1946, and expanded in 1968 for more lanes. The cross section of the bridge is shown in Figure 1.

![Figure 1: Bridge Cross Section](image)

The concrete beams are assumed to be simply supported at the concrete bents (see Figure 2). These bents are supported by square prestressed concrete piles with dimensions varying from widths of 508 mm to 610 mm.

![Figure 2: Bridge Elevation](image)
In 1994 three deteriorated beams in the splash zone, located at the northwest side of the bridge, were wrapped with CFRP, see Figure 3 and 4. In 1999, the CFRP was removed and wrapped again due to concerns of debonding of the CFRP from the concrete substrate.

![Figure 3: Beams Strengthened with CFRP](image)

An inspection conducted in 2006 indicated that the CFRP sheets on the strengthened beams appear to be well attached to the concrete. A coin tap test revealed only one isolated debond on the side of the beam within an region of 76 mm x 76 mm, which is within the ACI prescribed allowable range.
Visual inspection also revealed signs of concrete spalling and rebar corrosion in several unstrengthened locations near the strengthened beams. After a visual inspection on the bridge, some unstrengthened beams with signs of corrosion were found. Considerable deterioration on these beams, which included concrete spalling and steel corrosion, was observed at the bottom of the beams or the “splash zone”. The splash zone is located approximately 1.2 m above the water level.
A Finite Element Model (FEM) is developed on a finite element analysis software platform, ETABS 9.0, to study the feasibility of the load test approach to evaluate the durability of the CFRP. The model is summarized as follows:

The T beams are modeled using frame elements (see Figure 5) which use a general, three-dimensional, beam-column formulation that includes the effects of biaxial bending, torsion, axial deformation, and biaxial shear deformations. CFRP contribution to the shear deformations is not considered due to the small area ratio between the concrete and the CFRP area. Elastic behavior is assumed on the model. Based on the assumed conditions of the CFRP on the beams, a maximum of four layers and a minimum of zero layers are considered to still be effective.

Figure 5: Beams Section Geometry
The concrete bridge deck is modeled using shell elements (see Figure 6) with a thickness of 184 mm. Loads applied to the shell elements are located at the elements’ nodes. The shell element is a four-node formulation that combines separate membrane and plate-bending behavior. The membrane behavior uses an isoparametric formulation that includes translational in-plane stiffness components and a rotational stiffness component in the direction normal to the plane of the element. The plate-bending behavior includes two-way, out-of-plane, plate rotational stiffness components and a translational stiffness component in the direction normal to the plane of the element.

![a) Undeformed shape, b) Deformed shape](image)

Figure 6: 3D Finite Element Model

The shell elements are connected to the adjacent shell element at each node. The shell and the frame elements are located on the same plane. Frame elements are connected at the intersections with the shell elements along their length as shown on Figure 7. In order to not take into account
the stiffness of the slab twice, a stiffness factor modifier (m11) equal to 0.1 for bending is used on the shell elements, see Figure 8. The m11 modifiers are essentially equivalent to modification factors on the thickness of the shell elements in the traffic direction.

Figure 7: Shell connectivity

Figure 8: Stiffness Modifier

Only two spans of the bridge are modeled in the FEM (only the concrete barrier is continuous along the expansion joint). Concrete and steel properties are estimated based on the available information from the original design plans of the bridge. CFRP properties were provided by the
material manufacturer. Each ply of CFRP is $t_{FRP} = 0.381$ mm thick. Table 1 summarizes the material properties.

Table 1: Assumed Material Properties

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_c'$</td>
<td>27.6 MPa</td>
</tr>
<tr>
<td>$f_y$</td>
<td>414 MPa</td>
</tr>
<tr>
<td>$f_{FRP}$</td>
<td>895 Mpa</td>
</tr>
<tr>
<td>$E_c$</td>
<td>24.9 GPa</td>
</tr>
<tr>
<td>$E_s$</td>
<td>200 GPa</td>
</tr>
<tr>
<td>$E_{FRP}$</td>
<td>65.4 GPa</td>
</tr>
<tr>
<td>$G$</td>
<td>10.4 GPa</td>
</tr>
<tr>
<td>$v$</td>
<td>0.2</td>
</tr>
</tbody>
</table>

The concrete barrier on the side of the sidewalk shown on Figure 9-a is continuous throughout the beam’s expansion joints at each support. The barrier is created on the FEM using frame elements connected at the slab. The concrete barrier is not a structural element that can be accounted for the bridge strength, but it would have an influence on the bridge stiffness. The shell elements used to model the concrete deck are discontinuous at the expansion joint. At each intermediate bent, the beams are separated by a 25.4mm expansion joint. The expansion joint also separates the road material and the concrete deck as shown on Figure 9-b.
A typical FDOT load test configuration, in which the rear axles of the truck are loaded with concrete blocks, is used in the model as shown in Figure 13. The locations and the values of loads for each axle are shown on Figure 10. Each rear axle is supported by four tires. P4 and P5 are divided by the number of tires and applied as point loads to the bridge deck as shown in Figure 11 and 12. The heaviest load case available in the FDOT loading manual is used in the analysis. The location of the point loads was selected in order to obtain the maximum deflection on the beams, with this load configuration the maximum moment will occur at midspan. The location to achieve the maximum absolute moment can be found using influence lines.
Figure 10: Loading Configuration

Figure 11: Finite Element Model and point loads
Figure 12: Truck Section

Figure 13: Truck Picture
CHAPTER FOUR: BEAM STIFFNESS

The moments of inertia of the reinforced concrete beams wrapped with CFRP are calculated using the simplified methods developed by El-Mihilmy (El-Mihilmy et al, 2000) and Charkas (Charkas et al, 2002).

Based on the concrete beams geometry and the applied loads, the maximum moment $M_{\text{max}}$ under the prescribed truck loading and the self weight of the structure causes the beam section to crack but not yield, i.e., $M_{\text{cr}} < M_{\text{max}} < M_{\text{y}}$

El-Mihilmy’s Equation was developed based on an experimental study performed on reinforced concrete beams strengthened with fiber-reinforced polymer (FRP) plates. The average thickness of the plates used was 3.2 mm.

Geometrical properties of the T section were evaluated using the section equilibrium approach. With the section properties evaluated, Equation 1 developed by El-Mihilmy was applied to calculate the effective moment of inertia:

$$I_e = I_{cr} \left[ 1 + \left( 1 - \frac{M_{\text{max}}}{M_{\text{y}}} \right)^3 \right]$$  \hspace{1cm} (1)
Charkas Equations were developed based on a parametric study performed on a large number of beams. All possible parameters were considered, including section dimensions and ratios of CFRP varying from zero to its maximum, covering both CFRP rupture and concrete crushing failure modes. This parameter variation generated 250 load-deflection solutions used in a statistical correlation.

With the section properties evaluated, Equations 2 and 3 developed by Charkas were also applied to calculate the effective moment of inertia. Results from the two models are then compared.

\[
I_e = \frac{M_{\text{max}} \cdot I_g \cdot I_{ey} \cdot (M_y - M_{cr})}{M_{\text{max}} \cdot (M_y \cdot I_g - M_{cr} \cdot I_{ey}) + M_{cr} \cdot M_y \cdot (I_{ey} - I_g)} \quad (2)
\]

Where,

\[
\frac{I_{ey}}{I_g} = 0.7323 \cdot \left( \frac{I_{cr}}{I_g} \right) + 0.111 \quad (3)
\]

In the bridge under investigation, Beam-1 and Beam-2 were wrapped with CFRP along the entire span. Beam-3, which was retrofitted with CFRP in only half of the span, is modeled with an average value of the effective moment of inertia of a CFRP-strengthened and unstrengthened beam (as shown in Equation 4). Both the El-Mihilmy and Charkas Equations are utilized.

\[
I_{e,\text{avg}} = \frac{I_e + I_{ACI}}{2} \quad (4)
\]
$I_{ACI}$ is calculated using the ACI Equation. This value is the effective moment of inertia for unstrengthened beams calculated from Equation 5.

$$I_{ACI} = I_{cr} + (I_g - I_{cr}) \left( \frac{M_{cr}}{M_{max}} \right)^3$$ (5)

In the FEM, the moment demand is first calculated using the gross moment of inertia of all three beams. The resulting moment demand is then used to calculate the effective moment of inertia of the beam sections under the truck load. The deflection and strain in the beams are then calculated by the FEM using the effective moments of inertia. The moment demand when the effective moment of inertia is used is compared to the one used for the first iteration. The iteration continues until the difference between the maximum moment on the beam is less than 2%
CHAPTER FIVE: FINITE ELEMENT MODEL VERIFICATION

The results from the FEM shown in Figure 6 are verified using the simplified model shown in Figure 14.

![Figure 14: FEM for validation.](image)

The properties of the frame elements are based on the dimensions shown on Figure 5. The effective moment of inertia is calculated using Charkas and ACI equations. The concrete deck is modeled using shell elements with a thickness of 184mm. The beams are simple supported at the ends and the shell elements are only connected to the beams.

The theoretical deflection for a simple supported beam with a point load at midspan is calculated using Equation 6 (shear deformations are not considered).
\[ \Delta_{\text{max}} = \frac{P \cdot L^3}{48 \cdot E \cdot I_e} \] (6)

Table 2 summarizes the deflections found from the FEM shown on Figure 14 and the calculated deflection using Equation 6. The deflection for zero layers is calculated using I_e from Charkas and ACI Equation. The shell elements are modeled with stiffness modifiers m11=0.1 and m22=1

Table 2: Deflection Comparison.

<table>
<thead>
<tr>
<th>Number of CFRP layers</th>
<th>0 I_e from ACI</th>
<th>1 I_e from Charkas</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max deflection from the FEM [mm]</td>
<td>1.97</td>
<td>1.95</td>
<td>1.91</td>
<td>1.90</td>
<td>1.88</td>
</tr>
<tr>
<td>Max Theoretical Deflection [mm]</td>
<td>1.93</td>
<td>1.91</td>
<td>1.86</td>
<td>1.85</td>
<td>1.83</td>
</tr>
</tbody>
</table>

The increase on the stiffness when 4 layers of CFRP are present on the beam using the FEM and the Equation 6 is 6% for both cases. The average difference on the deflection when a layer of CFRP is added is approximately 1%.

Different values for the stiffness modifier used on the shell elements were investigated. The bending stiffness modifiers m11 and m22 are used on the shell elements as shown on Figure 8. Table 3 shows the maximum deflection assuming four layers of CFRP are present on the beam.
Table 3: Effect of the Bending Stiffness Modifiers.

<table>
<thead>
<tr>
<th>m11</th>
<th>m22</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>0.5</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>0.05</td>
<td>1.86</td>
<td>1.86</td>
<td>1.87</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td><strong>1.86</strong></td>
<td>1.86</td>
<td>1.86</td>
<td></td>
</tr>
<tr>
<td>0.3</td>
<td>1.85</td>
<td>1.85</td>
<td>1.85</td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td>1.84</td>
<td>1.84</td>
<td>1.84</td>
<td></td>
</tr>
<tr>
<td>0.9</td>
<td>1.83</td>
<td>1.83</td>
<td>1.83</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.82</td>
<td>1.82</td>
<td>1.82</td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER SIX: RESULTS

The effective moment of inertia calculated using El-Mihilmy Equation for one layer of CFRP is smaller than $I_{ACI}$, which is calculated using the ACI Equation for the unstrengthened beam. $I_{e}^{a}$ is 13% and 4% less for Beam-1 and Beam-3 respectively when compared with $I_{ACI}$. For Beam-2, this value is approximately the same, 0.99 (see $I_{e}^{a}/I_{ACI}$ on Table 4).

The value of $I_{e}/I_{ACI}$, which is the ratio between the strengthened moment of inertia and the baseline, is expected to be higher than unity due to the contribution of the CFRP to the beam stiffness. Deflection calculations are based on the effective moment of inertia calculated using Charkas Equation. Due to the geometry and amount of CFRP on the beams, this equation seems to better fit the actual conditions on the bridge.

For all beams and amount of CFRP layers, the equation developed by Charkas gives a higher effective moment of inertia than the one calculated using the ACI Equation for the unstrengthened beam, with a maximum value of 15% for Beam-2 when four layers are used and a minimum of 6% for Beam-1 when one layer is used (see $I_{e}^{b}/I_{ACI}$ on Table 4).

In all cases, the deflection on the beams increased as the number of CFRP layers decreased. The deflections on Beam-1 and Beam-2 increased by 5% while the deflection on Beam-3 increased by only 2%. These percentages are based on the calculated deflections which highlight the difference between the use of four CFRP layers and unstrengthened beams.
Since the location of Beam-2 is under the loading points, it is subjected to the highest bending moment. Figs. 15-a and 15-b show that Beam-2 has a lower effective moment of inertia $I_e$ and a higher predicted deflection compared to Beam-1 and Beam-3.

As shown in Figure 15-a, the deflection in Beam-2 decreases linearly from 4.6 to 4.4 mm when the number of CFRP sheets increases from zero to four. When an additional layer of CFRP is added on the beam, the deflection reduces by an average of approximately 1%.
Table 4: Normalized Moment of Inertia

<table>
<thead>
<tr>
<th>Beam-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{max} = 512 \text{ kN-m}, I_{ACI} = 4.98 \times 10^{10} [\text{mm}^4]$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>No of Layers</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_e^a/I_{ACI}$</td>
<td>0.85</td>
<td>0.87</td>
<td>0.89</td>
<td>0.91</td>
<td>0.93</td>
</tr>
<tr>
<td>$I_e^b/I_{ACI}$</td>
<td>1.05</td>
<td>1.06</td>
<td>1.08</td>
<td>1.09</td>
<td>1.11</td>
</tr>
<tr>
<td>$\Delta_{max} [\text{mm}]$</td>
<td>2.29</td>
<td>2.26</td>
<td>2.24</td>
<td>2.22</td>
<td>2.19</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beam-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{max} = 605 \text{ kN-m}, I_{ACI} = 4.15 \times 10^{10} [\text{mm}^4]$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>No of Layers</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_e^a/I_{ACI}$</td>
<td>0.97</td>
<td>0.99</td>
<td>1.02</td>
<td>1.04</td>
<td>1.06</td>
</tr>
<tr>
<td>$I_e^b/I_{ACI}$</td>
<td>1.09</td>
<td>1.10</td>
<td>1.12</td>
<td>1.14</td>
<td>1.15</td>
</tr>
<tr>
<td>$\Delta_{max} [\text{mm}]$</td>
<td>4.60</td>
<td>4.55</td>
<td>4.50</td>
<td>4.45</td>
<td>4.40</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beam-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{max} = 496 \text{ kN-m}, I_{ACI} = 5.19 \times 10^{10} [\text{mm}^4]$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>No of Layers</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_{e,av}^a/I_{ACI}$</td>
<td>0.95</td>
<td>0.96</td>
<td>0.97</td>
<td>0.98</td>
<td>0.99</td>
</tr>
<tr>
<td>$I_{e,av}^b/I_{ACI}$</td>
<td>1.06</td>
<td>1.07</td>
<td>1.08</td>
<td>1.09</td>
<td>1.09</td>
</tr>
<tr>
<td>$\Delta_{max} [\text{mm}]$</td>
<td>2.46</td>
<td>2.44</td>
<td>2.43</td>
<td>2.42</td>
<td>2.41</td>
</tr>
</tbody>
</table>

Note: $I_e^a$ from El-Mihilmy Equation

$I_e^b$ from Charkas Equation
The bending moment distribution of Beam-2, which is the most heavily loaded when both the dead load and truck loads are considered, is shown in Figure 16. The maximum bending moment is close to 580 KN-m at midspan.

Figure 16: Moment Diagram on Beam-2.
The strain distribution along the bottom of the Beam-2, which is shown in Figure 17, changed by 5% when the number of CFRP layers changed from one to four. The highest strain value is approximately 400 με.

Figure 17: Strain Distribution on CFRP.
CHAPTER SEVEN: CONCLUSIONS

This study investigates the change in the deflection when different layers of CFRP are assumed to still be effective on the girders of a reinforced concrete bridge. A Finite Element Model of the bridge was created. Only two spans of the bridge were modeled. The effective moments of inertia of the concrete girders were evaluated using two different equations. These values are calculated assuming different number of layers of CFRP. The effective moments of inertia were used on the FEM. Under all the assumed conditions such as reinforcement ratio, bridge geometry, bridge modeling and loading values the FEA results show that the deflection difference due to the change in number of CFRP sheets is 1% (average) for each layer. It should be noted that the percentage change due to CFRP is affected by other parameters such as span to depth ratio, beam spacing, load magnitude and reinforcement ratio.

Due to the variation in field conditions, this amount of deflection change is not likely to provide conclusive information on the durability of the CFRP using load test alone on the bridge studied in this thesis. Load test could be considered in conjunction with other nondestructive test methods to evaluate the actual condition of the CFRP.
1 LAYER

Material Properties [units Kips - in]

Concrete Strength \( fc = 4000 \text{ psi} \)
Steel Strength \( fy = 60000 \text{ psi} \)
FRP Strength \( frp = 129800 \text{ psi} \)
Concrete Rupture \( fr = 7.5 \sqrt{fc} \)
Concrete Modulus \( Ec = 57000 \cdot fc^{0.5} \)
Steel Modulus \( Es = 29000000 \)
FRP Modulus \( Erp = 9492300 \text{ psi} \)

Geometry

Flange Width \( b = 77 \)
Web Width \( bw = 17 \)
Flange Thickness \( hf = 7.25 \)
Section Height \( h = 38.25 \)
Section Depth \( d = 32.75 \)
Steel Area \( As = 11.32 \)
FRP thickness \( tf = 0.015 \cdot 1 \)
FRP height \( f = 15 \)

Geometry

\[ n = \frac{Es}{Ec} \quad n = 8.044 \quad \text{"Yc=Sum(Aty)/A"} \]

\[ m = \frac{Erp}{Ec} \quad m = 2.633 \]

\[ Yc := \frac{(b \cdot hf) \left( h - \frac{hf}{2} \right) + bw \cdot \frac{(h - hf)^2}{2} + n \cdot As \cdot (h - d) + 2 \cdot m \cdot f^2 \cdot tf + bw \cdot tf \cdot \frac{m}{2}}{b \cdot hf + bw \cdot (h - hf) + 2 \cdot m \cdot f \cdot tf + m \cdot bw \cdot tf + n \cdot As} \]

\[ Yc = 23.772 \]
Moments of inertia

\[ ly_c = \left( bw - \frac{h^3}{12} \right) + (bw-h) \left( Y_c - \frac{h}{2} \right)^2 + (b-bw) \frac{hf^3}{12} + (h-bw)hf \left( h - Y_c - \frac{hf}{2} \right)^2 \]

\[ lys = n \cdot As \left( Y_c - (h-d) \right)^2 \]

\[ lyf = m \cdot bw \cdot tf \cdot Y_c^2 + m \cdot tf \left( Y_c - \frac{f}{2} \right)^2 \]

\[ lg = ly_c + lys + lyf \]

\[ lg = 1.774 \times 10^5 \]

\[ M_c := \frac{int}{12000Y_c} \]

\[ M_c = 294.987 \text{ Kip-ft} \]

Cracked section

\[ X_{cr} = A + B + C \]

\[ A = \frac{b - (h-bw)}{2} \]

\[ B = hf \cdot (b-bw) + n \cdot As + 2 \cdot m \cdot tf + m \cdot bw \cdot tf \]

\[ C = \left[ \frac{-(b-bw)}{2} \cdot hf^2 - n \cdot As \cdot d - 2 \cdot m \cdot tf \left( h - \frac{f}{2} \right) - m \cdot bw \cdot tf \cdot h \right] \]

\[ A = 8.5 \]

\[ B = 527.919 \]

\[ C = -4.621 \times 10^3 \]

\[ v := (C^T B^T A)^T \]

\[ r := \text{polyroots(v)} \]

Roots of the quadratic

\[ r = \begin{pmatrix} -69.8387 \\ 7.779 \end{pmatrix} \]

\[ X_{cr1} := r_{1,0} \]

\[ X_{cr1} = 7.779 \]

\[ \frac{X_{cr1}}{d} = 0.238 \]

\[ X_{cr2} = r_{0,0} \]

\[ X_{cr1} = 7.011 \times 10^4 \text{ in}^4 \]

\[ |a_c| = 7.011 \times 10^4 \text{ in}^4 \]
DESCRIPTION
SikaWrap Hex 230C is a unidirectional carbon fiber fabric. Material is field-rolled using Sikadur Hex 330 or Sikadur Hex 300/305 epoxy to form a carbon fiber reinforced polymer (CFRP) used to strengthen structural elements.

WHERE TO USE
A Loading increases
Increasing the live loads in warehouses, increased traffic volumes on bridges, installation of heavy machinery in industrial buildings.
Vibrating structures.
Changes of building utilization.
A Seismic strengthening
Column wrapping.
 Masonry walls.
A Damage to structural parts
Aging of construction materials. Vehicle impact.
Fire.
A Change in structural system
Removal of walls or columns.
Removal of slab sections for openings.
A Design or construction defects
Insufficient reinforcements.
Insufficient structural depth.

ADVANTAGES
A Approved by IBCO ER-5588.
A Light weight fabric ideal for confined spaces.
A Can be applied in dry or wet lay-up process.
A Used for shear, confinement or flexural strengthening.
A Flexible, can be wrapped around complex shapes.
A High strength.
A Light weight.
A Non-corrosive.
A Alkali resistant.
A Low aesthetic impact.

PACKAGING
Rolls: 12 in. x 150 ft.
24 in. x 150 ft.

HOW TO USE
SURFACE PREPARATION
Surface must be clean and sound. It may be dry or damp, but free of standing water and frost. Remove dust, lint, grease, curing compounds, imbedded materials, and other bond inhibiting materials from the surface. Existing concrete surfaces must be filled with an appropriate repair mortar.
The adhesive strength of the concrete must be verified after surface preparation by random pull-off testing (ACI 503) at the discretion of the engineer. Minimum tensile strength, 200 psi with concrete substrate failure.

TYPICAL DATA FOR SIKAWRAP HEX 230C

<table>
<thead>
<tr>
<th>Property</th>
<th>US Units</th>
<th>SI Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear Elastic Modulus</td>
<td>33.4 x 10^6 psi</td>
<td>230,000 N/mm²</td>
</tr>
<tr>
<td>% Elongation</td>
<td>1.5%</td>
<td>1.5%</td>
</tr>
<tr>
<td>Density</td>
<td>0.065 lb/in³</td>
<td>1.1 g/cc</td>
</tr>
</tbody>
</table>

Cured Laminate Properties with Sikadur Hex 330 Epoxy
Properties after standard cure (70°F - 5 days)

<table>
<thead>
<tr>
<th>Property</th>
<th>Average Value</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td>130,500</td>
<td>185,000</td>
</tr>
<tr>
<td>Compressive Modulus</td>
<td>9,740,000</td>
<td>13,900,000</td>
</tr>
<tr>
<td>20° Tensile Strength</td>
<td>5,660</td>
<td>8,060</td>
</tr>
<tr>
<td>90° Tensile Modulus</td>
<td>455</td>
<td>650</td>
</tr>
<tr>
<td>% Elongation</td>
<td>0.4%</td>
<td>0.6%</td>
</tr>
</tbody>
</table>

1 Average value of test series
2 Average value minus 2 standard deviations
Preparation Work:
Concrete: Blast clean, shielded or use other approved mechanical means to provide an open, roughened texture.
In certain applications and at the engineer's discretion, the intimate contact between the substrate and the fabric may be determined to be non-critical. In these cases, although cleaning of the substrate using low pressure sand or water blasting is sufficient.

Mixing
Consult Sikadur 330 or Sikadur Hex 300/306 technical data sheet for information on epoxy resin.

Application
SikaWrap Hex 230C can be applied using wet or dry lay-up methods.

Dry Lay-Up: Apply the mixed Sikadur 330 epoxy resin directly onto the substrate at a ratio of 4:0.5 (240 ml/gal) depending on the surface profile. Carefully place the fabric into the resin with gloved hands and smooth out any irregularities or air pockets using a plastic terminating roller. Allow the resin to cure between the layers of the fabric. If more than one layer of fabric is required, apply additional Sikadur 330 to the exposed surface at a rate of 100 ml/gal (10 ml).

Wet Lay-Up: Seal the prepared concrete surface using Sikadur Hex 300 or Sikadur Hex 306. Material may be applied by spray, brush or roller. SikaWrap Hex 230C can be impregnated using either the Sikadur Hex 300 or Sikadur Hex 306 epoxy. For best results, the impregnation process should be accomplished using an automated saturation device. Once saturated, apply fabric to the sealed concrete surface and smooth any irregularities or air pockets using a plastic terminating roller. If required, apply additional layers of fabric white epoxy on previous layer is still tacky. For overhead or vertical applications, prime concrete with Sikadur 330 to improve tack. Saturate fabric with Sikadur 300 or 306. Coat the exposed surface of final fabric layer using Sikagard 6700 or Sikagard 62.

Installation of SikaWrap Products should be performed only by specially trained approved contractors.

Cutting SikaWrap
Fabric can be cut to appropriate lengths using a commercial quality heavy duty scissors. Since dull or worn cutting implements can damage, weaken or fray the fiber mesh, slow cutting should be avoided. Consult MSDS for proper handling procedures.

Limitations
A. Design calculations must be made and certified by an independent licensed professional engineer.
B. System is a vapor barrier. Concrete should not be encased in areas of freeze/thaw.

CAUTION
SikaWrap fabric is non-reactive. However, caution must be used when handling since a fine "carbon dust" may be present on the surface. Gloves must therefore be worn to protect against skin irritation. Caution must also be used when cutting SikaWrap fabric to protect against airborne carbon dust generated by the cutting procedure. Use of an appropriate, properly fitted NOSH/MSHA approved respirator is recommended.

KEEP CONTAINER TIGHTLY CLOSED
NOT FOR INTERNAL CONSUMPTION
CONSULT MATERIAL SAFETY DATA SHEET FOR MORE INFORMATION

Sika warrants its products to be free from manufacturing defects and to meet Sika's current published properties when applied in accordance with Sika directions and tested in accordance with ASTM and Sika Standards. User determines suitability of product for use and assumes all risks. Buyer's sole remedy shall be limited to the purchase price or replacement of product and excludes labor or the cost of labor. Any claim for breach of this warranty must be brought within one year of the date of purchase.

NO OTHER WARRANTIES EXPRESSED OR IMPLIED INCLUDING ANY WARRANTY OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE SHALL APPLY. SIKA SHALL NOT BE LIABLE FOR ANY CONSEQUENTIAL OR SPECIAL DAMAGES OF ANY KIND, RESULTING FROM ANY CLAIM OF BREACH OF WARRANTY, BREACH OF CONTRACT, NEGLIGENCE OR ANY LEGAL THEORY. SIKA ASSUMES NO LIABILITY FOR USE OF THIS PRODUCT IN A MANNER TO INFRINGE ON ANOTHER'S PATENT.


LIST OF REFERENCES


[5]. ACI Committee 440, “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures (ACI 440.2R-02)”. American Concrete Institute, Farmington Hills, Michigan. 2002


[8]. ACI Committee 318, “Building Code Requirement for Structural Concrete (ACI 318-02)”. American Concrete Institute, Farmington Hills, Michigan. 2002

