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<b>16. Abstract</b> <p>Many state, county, and local agencies have deteriorating short to medium span bridges. These bridges are commonly single span or multiple span continuous structures, and are composed of rolled or welded longitudinal steel stringers used as part of a continuous slab-on-girder bridge. Most of these bridges continue to serve as an integral part of the transportation system yet need some level of strengthening due to increases in live load demand or loss of section due to deterioration. The bridges are usually not critical enough to warrant replacement so a structurally efficient but cost-effective means of strengthening must be employed. In the past, the use of bolted steel cover plates was a common retrofit option for strengthening such bridges.</p> <p>This report documents the design, testing and implementation of a bridge strengthened by installing carbon fiber reinforced polymer (CFRP) plates to the bottom flange of girders in the positive moment region. The bridge is a three span continuous span steel structure located in Pottawattamie County with supplemental steel cover plates in the negative moment regions. Based on calculations completed by the bridge owner, it was determined that the positive moment regions of the interior girders in both the end and center spans of the selected bridge were overstressed. Design computations indicated that the overstressed girders could be adequately strengthened by the addition of CFRP plates bonded to the bottom flange of the girders.</p> <p>Based upon load tests and evaluation of the strengthened bridge response (immediately after strengthening and one year later), it was found that the overstressed bridge girders were adequately strengthened by the addition of CFRP plates. All CFRP plates installed on both exterior and interior girders were visually inspected approximately one and two years after their installation to observe any degradation or other changes that may have an influence on the integrity of the strengthening system. In general, no obvious debonding was observed and the condition of the CFRP plates seemed intact. Overall, the use of the CFRP plates appears to be a viable strengthening alternative for steel girder bridges.</p>			
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# EVALUATION OF A STEEL GIRDER BRIDGE STRENGTHENED USING FRP PLATES

CTRE Project

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## **EXECUTIVE SUMMARY**

### **Background**

Many state, county, and local agencies are faced with an ever-deteriorating bridge infrastructure, a large percentage of which are relatively short to medium span bridges. In many cases, these older bridges are composed of rolled or welded longitudinal steel girders with a composite or non-composite concrete deck. A large of these bridges need some level of strengthening due to increases in live loads or loss of capacity due to deterioration. The bridges are usually not critical enough to warrant replacement so structurally efficient, cost-effective means of strengthening needs to be employed. In the past, the use of bolted or welded steel cover plates or steel angles was a common retrofit option for strengthening such bridges. However, the time and labor involved to attach such retrofit system can be prohibitive.

Previous research at the Iowa State University Bridge Engineering Center established the effectiveness of carbon fiber reinforced polymer (CFRP) plates for improving the strength of steel composite beams. As a result of the successful laboratory investigation, the next step was taken to implement this strengthening method on an existing steel girder bridge. Among the various strengthening materials, the use of CFRP materials is very appealing in that they are highly resistant to corrosion, have low weight, and have a high tensile strength. This report documents the design, construction, and evaluation of a CFRP strengthening system installed on an existing, structurally deficient steel girder bridge.

### **Strengthening system**

Based on calculations completed by the bridge owner, it was determined that the positive moment regions of the interior girders in both the end and center spans of the selected bridge were overstressed. The design of the CFRP plate strengthening system was based on Iowa legal loads utilizing the Load Factor Design (LFD) approach. To design the CFRP plate strengthening system, a design model was developed that satisfied both strain compatibility and force equilibrium. Design computations indicated that the overstressed girders could be adequately strengthened by the addition of CFRP plates bonded to the bottom flange of the girders.

The CFRP plates used in this project are composed of a pultruded carbon fiber reinforced polymer material consisting of continuous unidirectional carbon fiber in an epoxy matrix designed for flexural strengthening. The CFRP plates have a high tensile strength (300 ksi), a moderate modulus of elasticity (20,000 ksi) with an ultimate strain of 1.5%, and a high resistance to corrosion. Although only the interior girders were found to be in need of strengthening, several modifications to the overall design were made to investigate the ease of installation of multiple CFRP layers and to compare the behavior of different strengthening schemes. The exterior girders were strengthened to investigate durability on exterior girders. To this end, the CFRP strengthening system consisted of one to three layers of CFRP plates bonded to the bottom flanges of two interior and exterior girders in the positive moment region in all three spans.

The CFRP plate strengthening system consisted of properly preparing the surface of the girders and CFRP and installing the CFRP plates. For this installation, temporary scaffolding was constructed underneath the bridge in both end spans to provide easy access to the girders. Access to the center span was provided through the use of a “snooper” truck. In general, the handling and installation of the CFRP plates was relatively labor intensive and required some training. A

crew of at least three people was needed to install the system.

### **Experimental program**

The selected bridge was instrumented to measure flexural strains at critical locations and was tested three times; before installation of the CFRP plates, shortly following the installation, and after approximately one year in service. The initial test was conducted to establish a benchmark response of the bridge, while the follow-up tests were completed to determine changes in performance resulting from the addition of the strengthening system. Standard loaded 3-axle dump trucks, driven at a crawl speed, were used to load the bridge with data collected continuously as the truck crossed the bridge.

In general, all strains exhibited an elastic response (i.e., strains from all transducers returned to zero after each truck crossed the bridge). The measured response confirms the presence of significant rotational end restraint at the abutment as one would expect since the bearings at both abutment are encased in concrete. Also, with minor variation, composite action was found to be present at all sections of the exterior girders (i.e., the neutral axis location on the exterior girder was found to be close to the top flange in the positive moment region).

The measured strains indicated good lateral symmetry that corresponded to the symmetrical configuration of the bridge and the truck paths used. Some of the minor differences in transverse symmetry may be attributed to either difference in local stiffness, difference in rotational end restraint at the abutment, or possible experimental error that might have occurred during the testing (e.g., differences in truck wheel line distribution and/or truck lateral positioning).

By comparing strains in each test, it was determined that strains measured in the follow-up tests were consistent with those measured during the initial test. This consistency in strains indicates that the strengthening system did not significantly alter the behavior of the bridge. Thus, it is evident from the consistency of the strain data that the installation of the CFRP plates had little impact on changing the stiffness of the bridge. It also indicates that the live load distribution characteristics were virtually the same before and after the installation of the strengthening system.

Before the “after one year of service” test was conducted, additional transducers were installed on the bottom flange of the strengthened beams (on the CFRP plates) in the center span to investigate the bond performance; a simple tool was also developed based on strain compatibility relationships to predict the extreme fiber strain on the CFRP plate from strains measured on the steel girders. These simple analytical predictions were compared with the measured CFRP strains. From the comparison, it was found that the difference between the analytical and experimental strain was on the average only 3.8%. Considering the sensitivity of the neutral axis location that could change significantly with a small change in top flange strain, this percentage error is considered to be insignificant; thus, it appears that there was good bond between the girders and CFRP plates.

All CFRP plates installed on both exterior and interior girders were visually inspected approximately one and two years after their installation to observe any degradation or other changes that may have an influence on the integrity of the strengthening system. In general, no obvious debonding was observed and the condition of the CFRP plates seemed intact. However,

some corrosion was observed on the top of the bottom flange of Girder 6 in all three spans. It was initially expected that the coating of paint applied after the installation of CFRP plates would be good enough for corrosion protection. After the visual inspections, however, it was speculated that the applied paint was too thin or had some defects and that rain or snow might have penetrated the protective coating. Therefore, it is recommended that thicker or a proper coatings of paint or other water-proof material be applied to prevent the weather-induced corrosion and, thereby, protect the integrity of the strengthening system.

### **Summary, Conclusions & Recommendations**

The primary objective of this project was to investigate and evaluate the effectiveness of CFRP plates to strengthen an existing, structurally deficient steel girder bridge. The bridge strengthened with CFRP plates was a 150 ft x 30 ft three-span continuous I-beam bridge in Pottawattamie County, Iowa, on State Highway IA 92. The research program consisted of several tasks with the primary emphasis on the design process, installation of the strengthening system, and field testing. The design of the CFRP plates was based on Iowa legal loads utilizing the LFD approach. Before the CFRP plates were installed, a diagnostic load test was conducted on the bridge to establish a baseline static behavior of the unstrengthened bridge. The strengthening system consisted of preparing the bonding surfaces and bonding the CFRP plates to the overstressed tension regions of the girders with a high strength epoxy adhesive. The installation of the strengthening system was followed by additional diagnostic load tests (one performed shortly after the installation and another one approximately one year later) to assess the short-term effectiveness of the strengthening scheme.

Although relatively time-consuming to install, it is believed that with more experience, installation crews could develop more efficient techniques for installing CFRP plates. Based upon the design process, analysis, and field investigations, the following conclusions and observations were made:

- CFRP plates are a relatively easy strengthening system to design.
- Special care must be taken to ensure that galvanic induced corrosion does not occur from direct contact of the CFRP plates and the steel girders. A structural primer is one possible way to provide a boundary between the two materials.
- From the analytical predictions and experimental testing, it was found that installing CFRP plates had minimal impact on the behavior of the subject bridge while at the same time contributing to the strength of the system. In terms of strength, adding additional CFRP layers has a diminishing impact.
- Efficient installation of CFRP plates requires the development of standardized construction practices and tools.
- The bond performance between the beam and CFRP plates was good and, therefore, it appears that no significant debonding has occurred during the two years of service.

Overall, the use of the CFRP plates appears to be a viable strengthening alternative for steel girder bridges. However, the use of CFRP plates bonded to the steel girders is relatively a new technology that has not been broadly accepted by bridge owners. Therefore, it is recommended that more studies be done on this strengthening system to determine behavior characteristics such as its fire resistance, its long-term creep characteristics, the degradation of bond forces with time, etc. In addition, more extensive studies to fully understand its failure mechanisms and the development of design standards are needed for the future use. It is further recommended that

any signs of delamination or bond failure on the strengthened girders in the current project be closely observed and, if detected, appropriate actions be taken since these types of failure would lead to a significant reduction in the added strength.



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# **1. INTRODUCTION**

## **1.1. Background**

Recent studies have indicated that approximately three trillion dollars are needed for the rehabilitation/retrofitting/strengthening of the nation's infrastructure. To this end, nearly one third of the nation's bridges have become structurally deficient or functionally obsolete. Basically, bridges need to carry loads greater than what they were originally designed for and at the same time have deteriorated at a rate faster than they could be repaired. Although replacement would be the best solution, a large number of these bridges are not critical enough to warrant replacement. In addition, the number of bridges that can realistically be replaced is limited by financial considerations. Common methods for strengthening existing bridges, such as using bolted or welded steel cover plates or angles, are sometimes prohibitive due to the time and labor involved in their installation. Therefore, structurally efficient but cost effective means of strengthening are needed.

In the last decade, the use of fiber-reinforced polymers (FRP) has emerged as a promising technology in structural engineering [1]. High performance materials such as carbon fiber reinforced polymers (CFRP) have the potential of becoming a cost effective strengthening solution. Among the various strengthening materials, the use of CFRP materials is very appealing in that they are highly resistant to corrosion, have a low weight, and have a high tensile strength. With this knowledge, a project, funded through the Federal Highway Administration's Innovative Bridge Research and Construction (IBRC) Program, was initiated to investigate the effectiveness of using CFRP composite materials to strengthen an existing steel girder bridge by installing CFRP plates to the overstressed regions. The bridge selected for strengthening is a three-span continuous I-girder bridge in Pottawattamie County, Iowa, on state highway IA 92 over Walnut Creek. This report documents the design, construction, and evaluation of a CFRP strengthening system for that bridge.

## **1.2. Literature Review**

Although the use of CFRP plates in concrete (reinforced and prestressed) bridge repair is becoming more common practice in the U.S., its use on steel bridges is still limited. In the following paragraphs, brief summaries of the research involving the use of CFRP plates or laminates on steel girders, are presented.

Previous research by Al-Saidi [2] at Iowa State University has determined the effectiveness of CFRP plates for improving the strength of composite steel girders. The study investigated the behavior of steel-concrete composite girders strengthened with CFRP plates attached to their tension side. To predict the behavior and deformation of a strengthened/repaired composite section, an analytical procedure was developed assuming nonlinear constitutive relationships for both the concrete slab and steel girder, and a linear relationship for the CFRP plates. Given the computationally intensive nature of the required analysis, a computer program using FROTRAN 77 code was developed. In the experimental part of the study, a total of ten girders were tested under four-point bending. Six of the girders were tested in an undamaged condition: two girders were control girders and four were strengthened (without damage) with CFRP plates. The

remaining four girders were artificially damaged by reducing the bottom flange area and then repaired (one control girder and three repaired). CFRP plates of different widths and modulus of elasticity (29,000 ksi and 22,000 ksi) were used in the strengthening schemes. The test results indicated a significant increase in the strength of the strengthened/repared girders. It was found that the girder strengthened with CFRP plate with a modulus of elasticity of 22,000 ksi had a more ductile behavior than the girder strengthened with CFRP plate with a modulus of elasticity of 29,000 ksi.

A study was conducted by Mertz and Gillespie [3] to evaluate the feasibility of using advanced composite materials in rehabilitating deteriorated members of steel highway bridges. The study focused on the case of a single span girder where the bottom flange was subjected to tensile stresses. The retrofit was installed on a corroded steel member utilizing adhesive bonding. In the initial phase of this project, modeling, fabricating, and testing of two repair schemes were performed. After service-load testing on the repaired members, the authors found that the composite plates increased the section stiffness moderately. Several rehabilitation schemes were then developed and tested on a variety of field geometries. Analysis and test results determined that an increase in flexural stiffness of 20 to 30% was attainable. It was also found that accelerated bonding through induction heating was a viable field installation technique. Additional service-load testing of fabricated scale girders and large-scale testing of both intact and corroded steel girders were completed in the later phase of the project. The results indicated that composite materials improved the strength and fatigue life of steel components.

The report by Tavakkolizadeh et al [4] presents their results on the behavior of steel-concrete composite girders retrofitted with CFRP sheets (a width of 3 in. and thickness of 0.05 in per sheet) under static loading. Three large-scale composite girders consisting of a W14 x 38 A36 steel girder and a 3 in. thick by 36 in. concrete slab were tested. The three specimens had either 1, 3, or 5 layers of constant thickness CFRP. The test results showed that the strengthening system significantly increased the ultimate load-carrying capacities of the girders. It was also found that the efficiency of the CFRP was reduced when more layers were used. For the one-layer system, stress in the CFRP laminates was 75% of its ultimate strength while it decreased to 42% of the ultimate strength for the five-layer system.

Testing of six 20 ft long steel composite girders (W8 x 24 steel girder composite with 4 in. thick, 28 in. wide reinforced concrete slab) was conducted by Sen et al. [5]. Two different strength of steel girders were used; three girders had a yield strength of 45 ksi while the other three girders had a yield strength of 54 ksi. Before being strengthened with 0.08 in. x 0.2 in. x 12 ft unidirectional CFRP plates, the girders were initially loaded in four-point static bending to yield the tension flanges. An increase of approximately 11% to 50% in their ultimate strengths occurred depending on the yield strength of the specimen and the mode of failure.

A study by Miller [6] addressed the issues of force transfer, development length, and long-term durability of the CFRP/steel bond. First, force transfer and development were studied through the use of small-scale specimen testing and analytical solutions. From this test, it was found that 98% or more of the force transfer between the steel and CFRP plates occurred within 4 in. It was also determined that the force transfer length and force were sensitive to changes in bond-line thickness and CFRP material properties. No signs of significant creep or debonding in the CFRP/steel bond under sustained loads were observed in seven months of monitoring. The

developed rehabilitation scheme was installed on an existing steel bridge girder. Initial and follow-up load tests indicated a reduction in tension flange strains of 11%.

### **1.3. Objectives**

The primary objective of the project summarized herein was to investigate and evaluate the effectiveness of CFRP plates in strengthening an existing, structurally deficient steel girder bridge. This objective was met by:

- Documenting the design, construction, and performance of the CFRP plate that was used.
- Identifying changes in structural behavior due to the addition of the CFRP plates and with time through field inspections and load tests.

### **1.4. Scope**

The research program consisted of several tasks with the primary emphasis on the design process, installation of the strengthening system, and field testing. The design of the CFRP plates was based on Iowa legal loads utilizing the Load Factor Design (LFD) approach. Before the CFRP plates were installed, a diagnostic load test was conducted on the bridge to establish a baseline static behavior of the unstrengthened system. The strengthening system consisted of preparing the bonding surface and bonding the CFRP plates to the overstressed tension regions with a high strength epoxy adhesive. The installation of the strengthening system was followed by additional diagnostic load tests (one performed shortly after the installation and another one approximately one year after the installation) to assess the short-term effectiveness of the strengthening scheme.

A detailed description of the subject bridge and the strengthening system employed is presented in Chapter 2. The field tests are described in Chapter 3, and the corresponding test results are summarized in Chapter 4. General conclusions and recommendations are presented in Chapter 5 and Chapter 6, respectively.





## **2. BRIDGE AND STRENGTHENING SYSTEM DESCRIPTION**

This chapter describes the physical characteristics of the bridge strengthened as part of this investigation. Also, a description of the strengthening system including its design and installation is given.

### **2.1. Bridge Description**

The bridge (Number 7838.5S092) selected for strengthening is a 150 ft x 30 ft three-span continuous I-girder bridge shown in Fig. 1. The bridge is located in southwest Iowa in Pottawattamie County carrying State Highway IA 92 over Walnut Creek. The bridge has two 45 ft - 6 in. end spans and a 59-ft center span. The original non-composite bridge was built in 1938 and was widened on both sides in 1967 by the addition of exterior girders that were constructed composite with the deck. The original bridge deck is a nominal 6.5-in. thick cast-in-place, reinforced concrete slab. The current average deck thickness is approximately 11 in. with 4 - 1/8 in. of crown from the edge of the roadway to the center of the roadway. The bridge deck is supported by two W27x84 exterior girders and four interior girders (two W27x91s and two W27x98s). Girder spacing, diaphragm location, etc., are as shown in Fig. 2. Note that full depth 8-in. thick concrete diaphragms were installed between the W27x91 interior and W27x84 exterior girders when the bridge was widened in 1967. The bridge has no skew. Both abutments are integral concrete supported on treated wood friction piling, and the original piers are open-two-column concrete supported on untreated wood friction wood piling. The piers were also widened on both sides with the addition of a concrete pile at each end in 1967 (see Fig. 1a).

The roadway width is 30 ft allowing for two traffic lanes and two shoulders. The bridge has 12 in. thick curbs integral with the deck and concrete guardrails connected to the curbs.

There are a few random vertical and horizontal hairline cracks in both abutments and the east abutment footing has a delaminated area at the top of a construction joint. Several hairline, narrow transverse cracks and a few small delaminated areas exist in the top and bottom surfaces of the deck. Both curbs show moderate hairline cracks. The top of the deck has Portland Cement (PC) patches at several locations, and a few small spalls and delaminated areas along the construction joints.

### **2.2. Strengthening System**

The CFRP plate strengthening system consisted of preparing the surface of the girders and CFRP and installation of the CFRP plates. The strengthening system installed consisted of one to three layers of CFRP plates bonded to the bottom flanges in the positive moment region. More specific details are provided in the following sections.

The strengthening system basically consisted of three primary components: CFRP plates, primer, and structural epoxy. Each of these components is described in the following paragraphs.



(a) Side view



(b) End view

**Figure 1. Overall bridge photographs.**

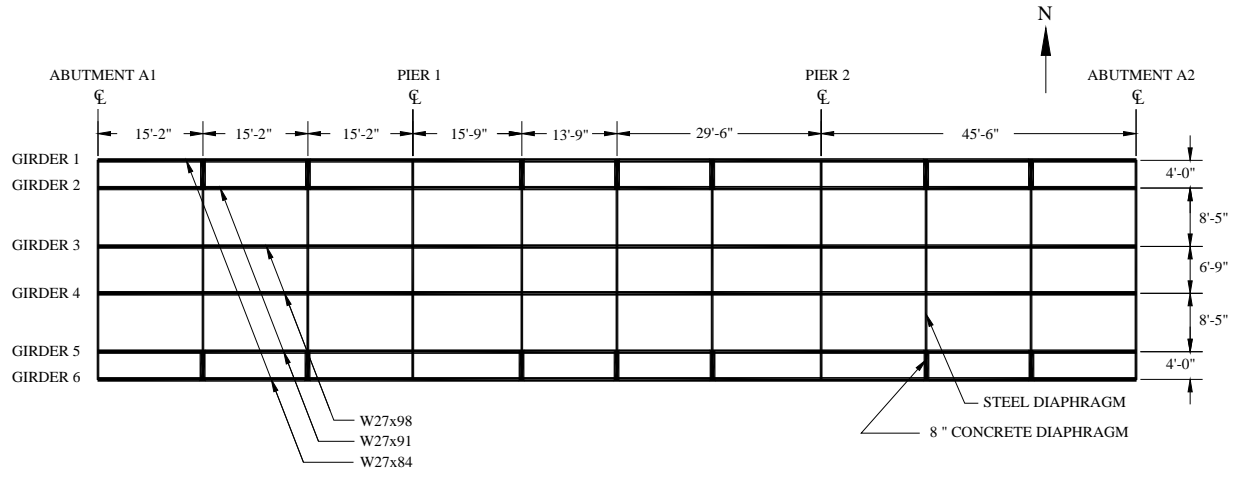


(c) Bottom view



(d) Typical abutment

**Figure 1. Overall bridge photographs - continued.**



**Figure 2. Bridge framing plan.**



**Figure 3. CFRP plates being unrolled.**

The specific CFRP material selected for this installation is a pultruded carbon fiber reinforced polymer consisting of continuous unidirectional carbon fiber in an epoxy matrix. The anisotropic CFRP plates were fabricated using a proprietary pultruding process in which the carbon fibers were pretensioned and aligned during impregnation and curing. The plates had a design tensile strength of 300 ksi (ASTM D3039) and a modulus of elasticity of 20,000 ksi (ASTM D3039). All plate bonding surfaces had a “sanded” condition to promote adhesion between the bonding surface of the steel girders and plates. Each plate was 4 in. wide and 0.04 in. thick (see Fig. 3). The CFRP plates had a tensile linear elastic behavior up to failure; mechanical properties of the CFRP plates are summarized in Table 1.

**Table 1. Material properties of CFRP plates.**

<b>Ultimate tensile stress, (ksi)</b>	<b>Modulus of elasticity, (ksi)</b>	<b>Ultimate strain</b>	<b>Fiber Content</b>
300	20,000	1.5 %	68 % by volume

A two-part, FRS 1000 Primer was used to provide a base surface between the steel girders and CFRP plates. The primary purpose of the primer was to prevent corrosion induced by galvanic reaction between the girder surface and the carbon fiber.

A two-part system ECS 104 Structural Epoxy was used for bonding the CFRP plates to the steel girders. The pot life of 3 ounces of the epoxy was listed as 3 hours at 77°F. The manufacturer’s listed properties of the epoxy adhesive are presented in Table 2.

**Table 2. Properties of ECS 104 Structural Epoxy.**

<b>Tensile strength, ksi</b>	<b>Adhesion, ksi</b>	<b>Flexural strength, ksi</b>	<b>Flexural modulus, ksi</b>
8.80	>0.29	14.50	310

### 2.2.1. Design

The design of the CFRP plate strengthening system was completed for the Iowa legal loads utilizing the LFD approach [7]. Based on an analysis completed by the bridge owner, it was found that the positive moment region of Girder 3 and Girder 4 in both the end and center spans was overstressed by 0.55 ksi and 1.57 ksi, respectively. In this section, the design process used to predict the increase in moment capacity due to the addition of CFRP plates is presented. In addition, the CFRP plate strengthening scheme that was employed is described.

#### 2.2.1.1. DESIGN MODEL

To design the CFRP plate strengthening system, a design model was developed that satisfied both strain compatibility and force equilibrium. To accomplish this, several assumptions were made:

- The girder was symmetric and initially straight.
- The steel was elastic-perfectly plastic.

- There was a “perfect bond” between the CFRP plate and the steel girder.
- The CFRP had a linear elastic behavior up to failure.

A summary of the step-by-step design procedure follows:

1. Divide the cross-section into numerous individual elements to idealize the strain distribution with a linear function (see Fig. 4).
2. Assume the CFRP plate reaches its ultimate strain ( $\epsilon_{CFRP} = 0.015$ ). Thus, the extreme tension fiber strain ( $\epsilon_R$ ) can be assumed to be  $\epsilon_R = 0.015$ .
3. Assume a neutral axis location,  $c$ .
4. Calculate strain in each element and generate a strain profile (see Fig. 4) based on the strain compatibility relationship.
5. From the strain profile generated in Step 4 and stress-strain relationship for each material shown in Fig. 5 (constitutive relationship), determine the stress distribution in each element.

For steel:

$$f_{Si} = E_S * \epsilon_{Si} \quad (\text{if } \epsilon_{Si} < \epsilon_y)$$

or

$$f_{Si} = f_y \quad (\text{if } \epsilon_{Si} \geq \epsilon_y)$$

where  $f_{Si}$ ,  $E_S$ ,  $\epsilon_{Si}$ , and  $\epsilon_y$  are stress in  $i^{\text{th}}$  steel element, elastic modulus of steel, strain in  $i^{\text{th}}$  steel element, and yield strain of the steel member, respectively.

For CFRP:

$$f_{CFRP} = E_{CFRP} * \epsilon_{CFRP}$$

where  $f_{CFRP}$  and  $E_{CFRP}$  are stress and elastic modulus of CFRP plate, respectively.

6. From the stress distribution, calculate the compressive force,  $F_C$ , and the tensile force,  $F_T$ , for each element with respect to the centroid of each element.
7. Check the equilibrium of horizontal force,  $\sum F_x = 0$ :
  - if  $F_{\text{Compression}} \neq F_{\text{Tension}}$ , assume a new neutral axis location (Step 3) until equilibrium is satisfied.
  - if  $F_{\text{Compression}} = F_{\text{Tension}}$ , move on to the next step.
8. Calculate the total moment (moment capacity,  $M_n$ ) by summing the moments of the internal forces about a convenient axis, which can be expressed as below:

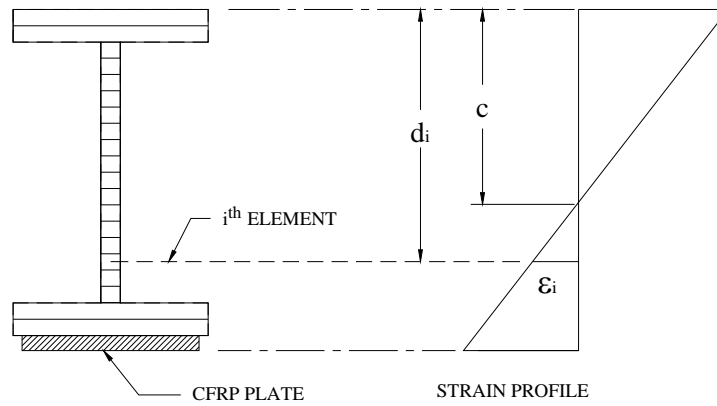
$$M_n = \sum(f_{Si} * A_{Si} * d_{Si}) + \sum(f_{CFRP} * A_{CFRP} * d_{CFRP})$$

where  $A$  ( $A_{Si}$  or  $A_{CFRP}$ ) and  $d$  ( $d_{Si}$  or  $d_{CFRP}$ ) are the cross sectional area of the corresponding element, and distance of that element from the top surface of the top flange.

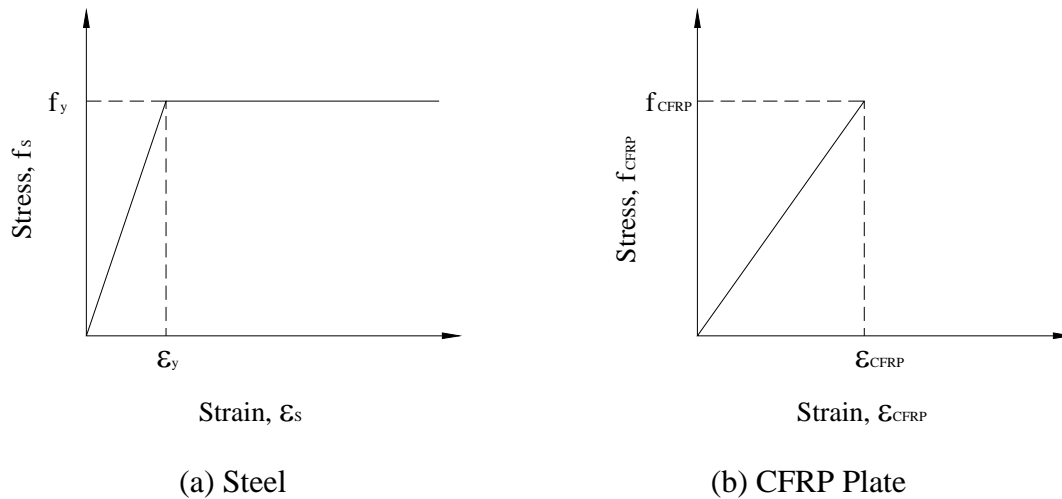
The change in moment capacity with increasing numbers of CFRP plate bonded to the bottom of

the bottom flange is shown in Table 3 and Fig. 6. From this analysis, it was found that the efficiency of using the CFRP plates decreased as the number of CFRP plates used increased. The increase in moment capacity was the greatest (12.3%) with only one layer of CFRP plate utilized while it only increased 3.2% from the three-layer system to the four-layer system.

Increase in stiffness due to the addition of CFRP plates to the bottom of the bottom flange was also investigated as shown in Fig. 6. Based on this, it was determined that an increase in stiffness of close to 1.2% per layer was obtained for Girders 3 & 4. The theoretical change in stiffness presented in this section is later compared with field test results in Chapter 4.



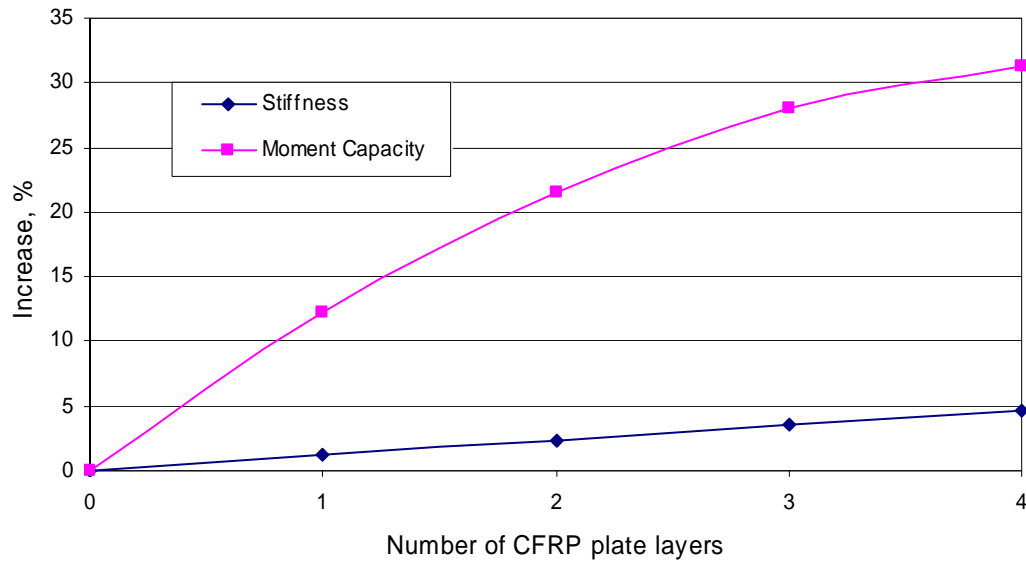
**Figure 4. Subdivision of girder cross-section and strain profile.**



**Figure 5. Constitutive relation.**

**Table 3. Increase in moment capacity in Girders 3 and 4 due to the addition of CFRP plates.**

$A_{CFRP}$ , (in <sup>2</sup> )	Increase in moment capacity, (%)	Increase in moment capacity per layer, (%)
No CFRP plate	-	-
One layer (0.32)	12.3	12.3
Two layers (0.64)	21.6	9.3
Three layers (0.96)	28.1	6.5
Four layers (1.28)	31.3	3.2



**Figure 6. Increase in moment capacity and stiffness of Girders 3 & 4 with number of layers of CFRP.**

#### 2.2.1.2. STRENGTHENING SCHEME

Following the procedures outlined previously, it was found that the overstressed girders could be adequately strengthened by the addition of CFRP plates bonded to their bottom flanges. Girders 3 and 4 required one layer of CFRP in the end spans, and two layers in the center span; each layer of CFRP plate measured 8 in. x 0.04 in.

Although only Girders 3 and 4 were found to be in need of strengthening, the following modifications were made with the final layout summarized in Figs. 7 and 8:

- Girders 1 and 6 were strengthened to investigate CFRP plate durability on exterior girders.



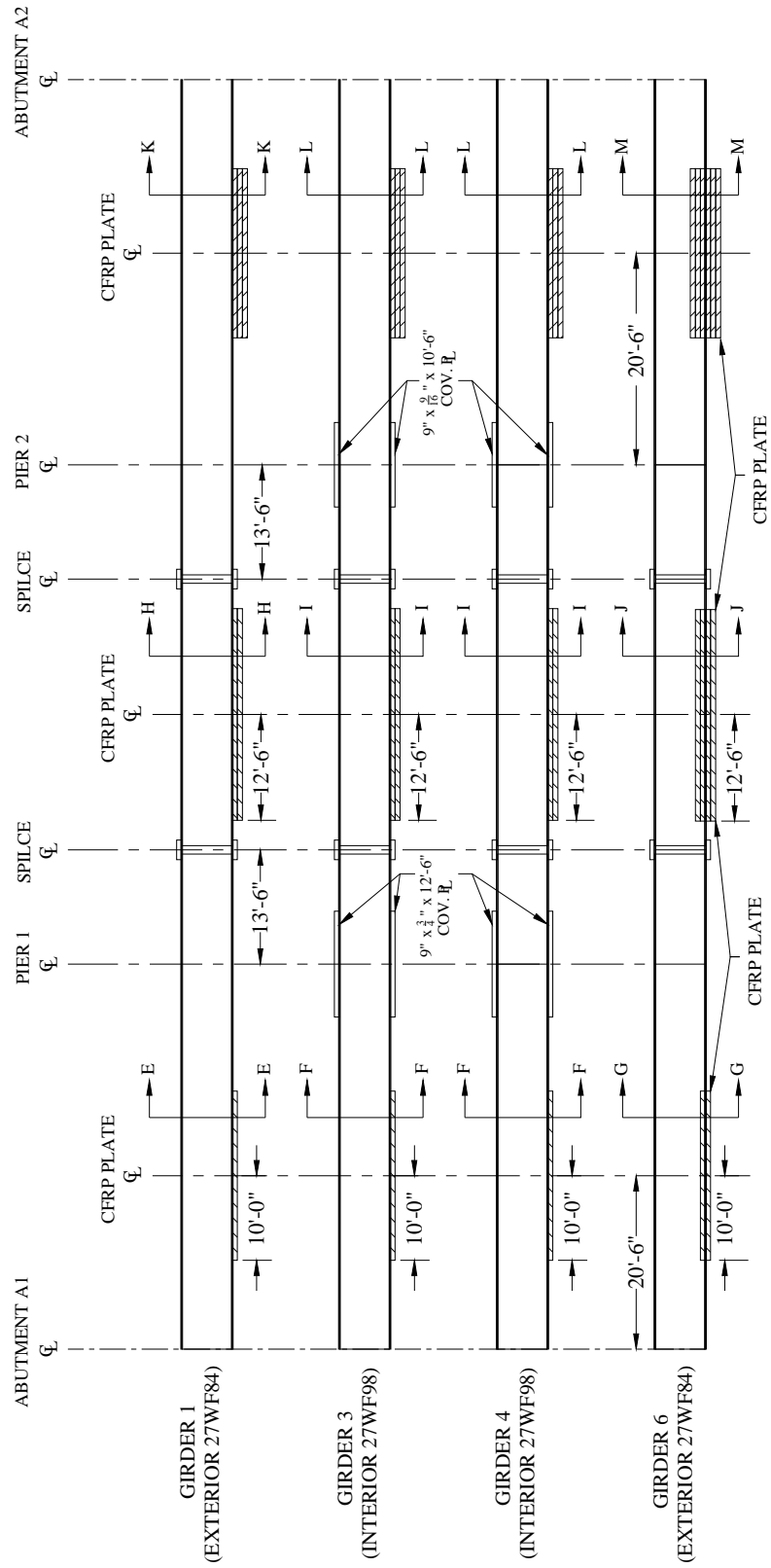
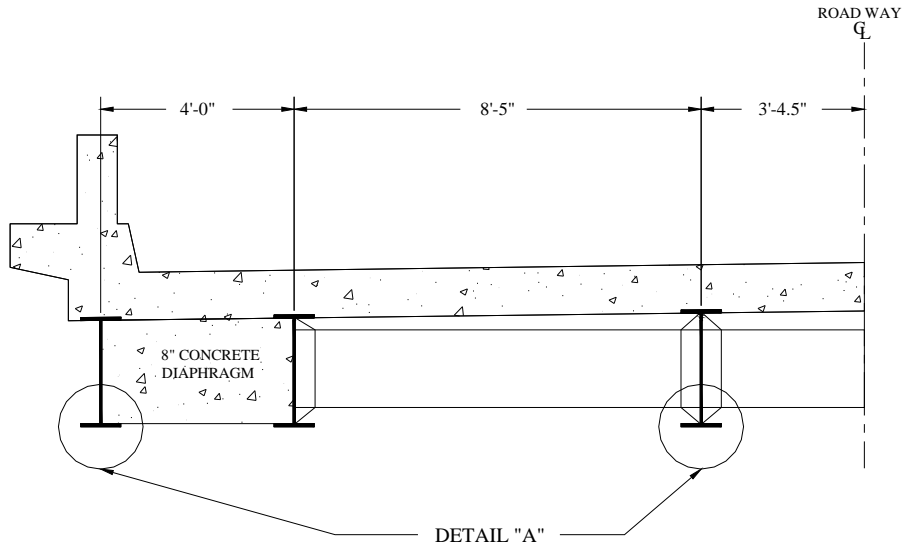
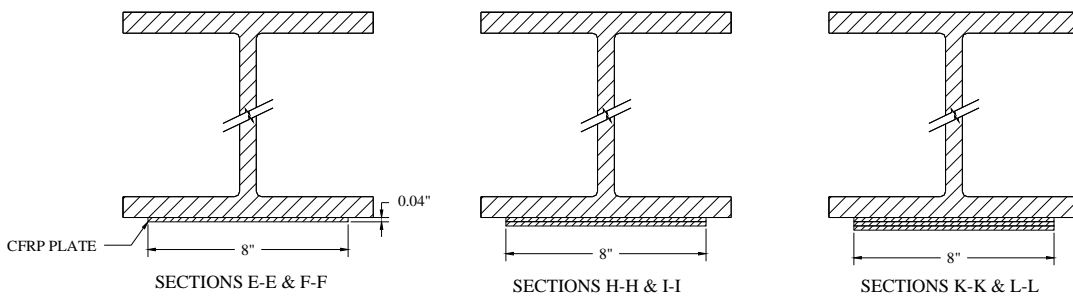


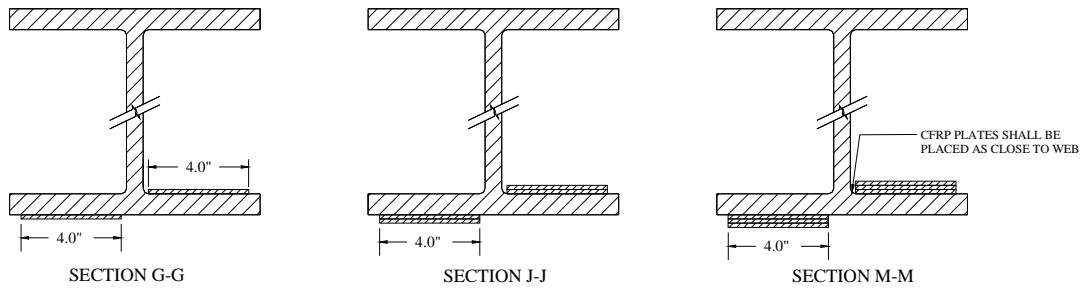
Figure 7. Final layout of CFRP plates (side view).



(a) Typical half cross-section



(b) Detail "A" for Girders 1, 3, and 4



(c) Detail "A" for Girder 6

**Figure 8. Girder cross-sections with CFRP plates.**

- Girders 1, 3, 4, and 6 in the east end span had three layers installed to evaluate performance and constructability of multi layers.
- Girder 6, in all three spans, had one-half of the CFRP plate installed on the bottom of the bottom flange and one-half on the top of the bottom flange to investigate durability under direct environmental exposure.

The intent of the modifications to the overall design was to:

- Investigate the ease of installation of multiple CFRP layers.
- Evaluate the durability of the installation.
- Compare the behavior of different schemes used in different conditions.

### *2.2.2. Installation Process*

As illustrated in Fig. 9, temporary scaffolding was constructed underneath the bridge in both end spans to provide easy access to the girders. Access to the center span was accomplished using a snooper truck. Proper preparation of the bonding surfaces of the plates and girders is critical to achieving a good bond between the girder and plate. The steel girder surfaces to which the CFRP plates were to be bonded were roughened to a coarse sandpaper texture by sandblasting to remove any paint and unsound material. As can be seen in Fig. 10, the work area was covered with a layer of plastic sheeting during sandblasting to contain the paint debris. The resulting sandblasted girder surface can be seen in Fig. 11. After sandblasting, the girder surface was cleaned (see Fig. 12) with a clean rag wetted with acetone. The bonding surfaces of the CFRP plates were also cleaned (see Fig. 13) with acetone to remove all dirt and debris. After the bonding surfaces were dry, the prepared girder surface was treated with a thin layer of FRS Primer (see Fig. 14) to prevent corrosion induced by a galvanic reaction – which occurs when two dissimilar materials are coupled with a corrosive electrolyte [8] – between the girder and carbon fibers. Once the primer had set, a specially designed pneumatic caulking nozzle was used to mix the two-components of the ECS 104 structural epoxy (see Fig. 15). The structural epoxy was applied to both the pre-cut (see Fig. 16) CFRP plates (see Fig. 17a) and the girder surface (see Fig. 17b) using a 1/8 in. v-notched trowel. The plate was then carefully placed on the girder surface (see Fig. 18), starting at one end, taking special care to ensure a “straight” application. A smooth, hand roller was then used to apply pressure to the plate to evenly distribute the epoxy and to remove any trapped air (see Fig. 19). A smooth trowel was then used to remove any excess epoxy (see Fig. 20) from the edges of the CFRP plates. After all plates were installed, all surfaces were painted. Photographs of the completed installation are presented in Fig. 21.

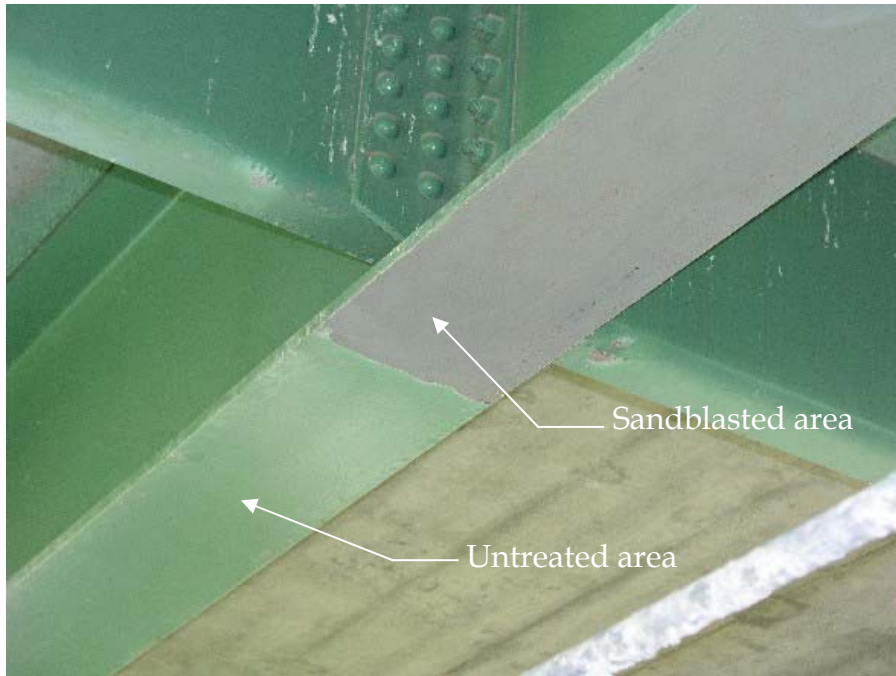
The handling and installation of the CFRP plates was relatively labor intensive and required some training. At least a three-man crew was needed to install the system (one day per layer: three days for three layers). It is recommended that a periodic visual inspection be made at the bond line on each strengthened girder to investigate any bond failure or slip at the bonding surface; if any signs of delamination or bond failure are detected, appropriate actions should be taken since these types of failure would lead to a significant reduction in strength.



**Figure 9. Temporary work scaffolding.**



**Figure 10. Confinement of work area during sandblasting.**



**Figure 11. Sandblasted girder surface.**



**Figure 12. Cleaning the girder surface after sandblasting.**



**Figure 13. Cleaning the CFRP plate bonding surface.**



**Figure 14. Applying FRS 1000 Primer to bottom girder flange.**



**Figure 15. Preparation of ECS 104 structural epoxy (mixing with a compressed air gun).**



**Figure 16. Cutting the CFRP plates.**



(a) Application of structural epoxy to CFRP plate.



(b) Application of structural epoxy to bottom girder flange

**Figure 17. Epoxy application.**





**Figure 18. Placement of CFRP plate on top of Girder 6 bottom flange.**



**Figure 19. Removal of excess air by applying pressure with a roller.**



**Figure 20. Removal of excess epoxy.**



(a) Girder 1 in the west end span

**Figure 21. Photographs of the completed installation.**



(b) Girder 3 in the west end span



(c) Girder 4 in the east end span

**Figure 21. Photographs of the completed installation - continued.**



(d) Top of bottom flange of Girder 6 in the west end span



(e) Bottom of bottom flange of Girder 6 in the east end span

**Figure 21. Photographs of the completed installation - continued.**

### **3. EXPERIMENTAL PROGRAM**

The subject bridge was instrumented to measure flexural strain at critical locations and tested before installation of the CFRP plates, shortly following the installation, and after the strengthening system had been in service approximately one year. The initial test was conducted to establish a benchmark response of the bridge, while the follow-up tests were completed to determine changes in its performance resulting from the addition of the strengthening system. Standard 3-axle dump trucks, driven at a crawl speed, were used to load the bridge with data collected continuously as the truck crossed the bridge. For clarity, the load tests that took place before and after the installation will be described separately in the following sections.

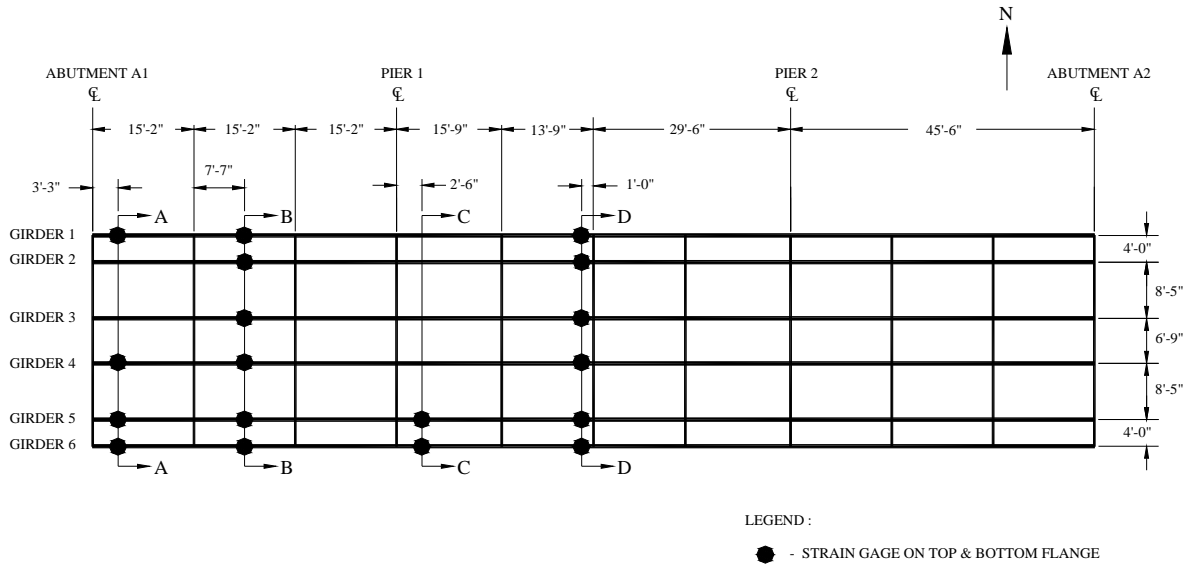
#### **3.1. Initial Test (September 4, 2002)**

A diagnostic initial load test was conducted prior to the installation of the CFRP plate strengthening system to establish a baseline static behavior of the unstrengthened bridge. The location of the instrumentation was selected to effectively give an overall understanding of global behavior of the bridge when subjected to live load. As can be seen in Fig. 22, a total of thirty-six strain gages were installed on the bridge with twenty-four gages in the positive bending moment regions and twelve gages in the negative bending moment regions. Due to the structural symmetry of the bridge, only one half of the bridge was instrumented.

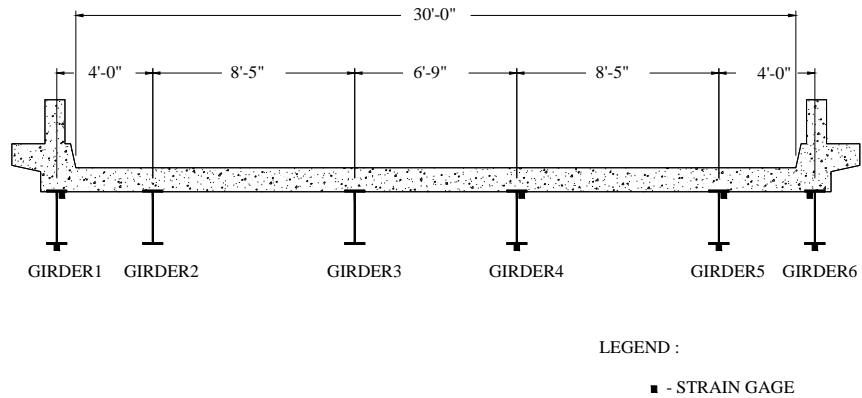
Three different truck paths were used to examine the performance of the bridge. These truck paths were chosen so that the maximum strains were generated in select girders. For convenience, each load path is referred to as Y1, Y2 and Y3 as shown in Fig. 23. All tests were conducted from west to east. For path Y1 heading east in the south lane, the passenger side wheels were placed 4 ft - 1.5 in. north of Girder 6. Path Y2 placed the passenger side wheel 3 ft south of the bridge centerline. For path Y3, heading east in the north lane, the driver's side wheel was placed 4 ft - 1.5 in. south of Girder 1. The Iowa DOT provided the load truck shown in Fig. 24. Truck 1 had a total weight of 52.76 kips with 13.78 kips and 38.98 kips on the front and rear axles, respectively. This truck was 16 ft - 10.5 in. long between the center of the front axle and the center of the rear axles, 6 ft - 11.5 in. wide between the centers of the front tires, and 5 ft - 10.5 in. wide between the rear tires.

#### **3.2. Shortly After Installation (August 19, 2003)**

The second load test was conducted approximately two months after the installation of the strengthening system to assess any immediate change in performance resulting from the installation of the CFRP plates. The procedures used for the second load test were the same (e.g., same load paths and sequences, etc.) as those used in the initial test except for the weight of the truck used and the number of strain gages used. In this test, four additional gages were installed on the CFRP plates of the strengthened girders (Girders 1, 3, 4, and 6) in the center span (Section D-D as shown in Fig. 25) to investigate the bond performance of each strengthened girder at that location. The load truck used for the second load test, Truck 2, was again provided by the Iowa DOT and had a total weight of 53.06 kips with 15.60 kips and 37.46 kips on the front and rear axles, respectively.

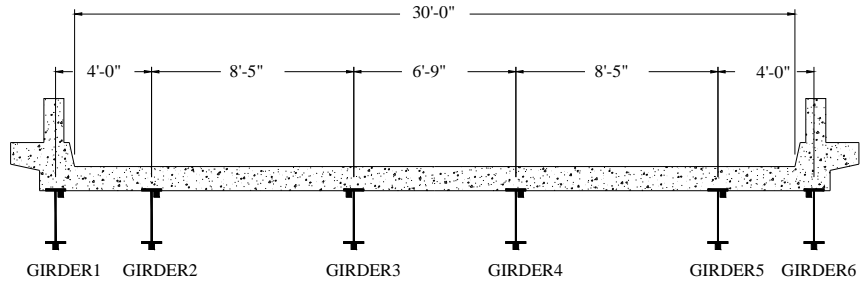


(a) Plan view



(b) Section A-A

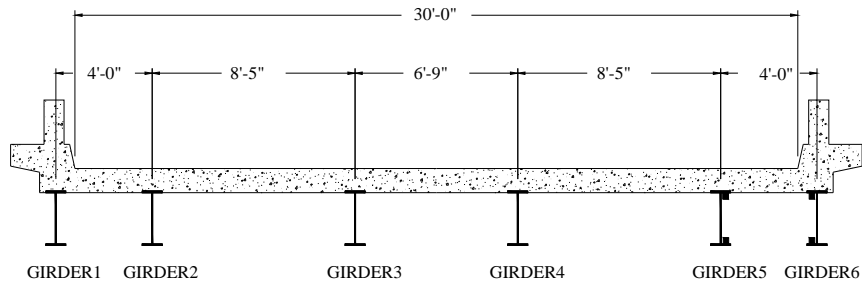
**Figure 22. Location of strain gages.**



LEGEND :

■ - STRAIN GAGE

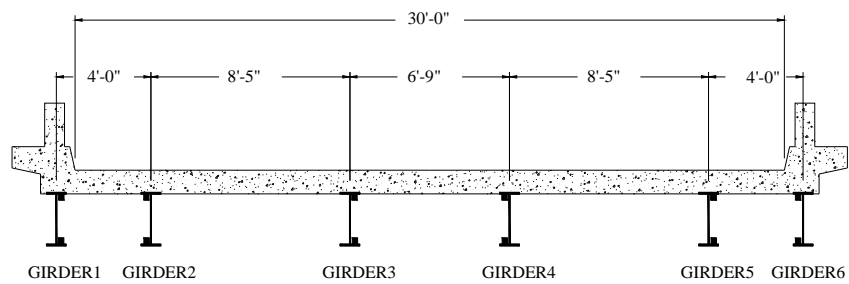
(c) Section B-B



LEGEND :

■ - STRAIN GAGE

(d) Section C-C

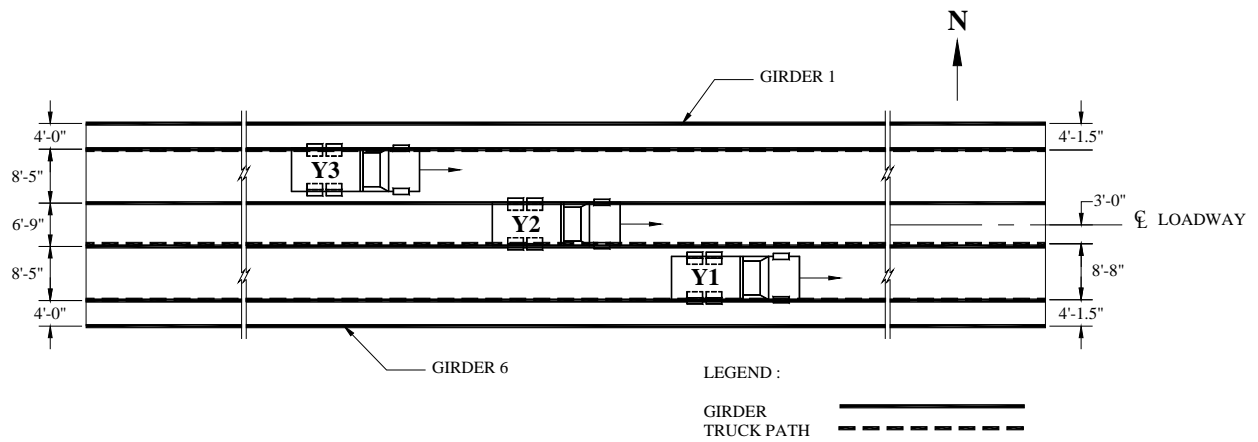


LEGEND :

■ - STRAIN GAGE

(e) Section D-D

**Figure 22. Location of strain gages – continued.**

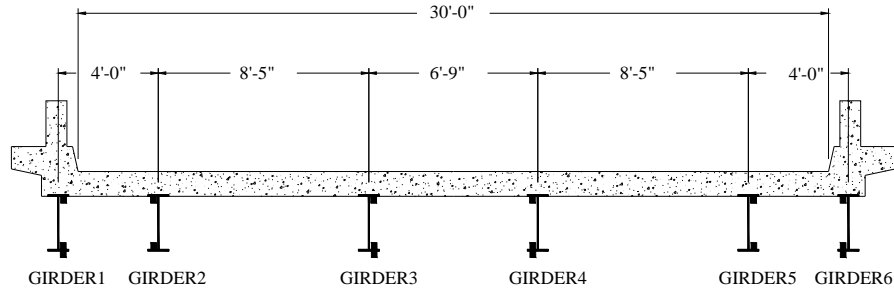


**Figure 23. Truck paths used during load test.**



**Figure 24. Photograph of typical load test truck.**





LEGEND :

■ - STRAIN GAGE

(a) Cross-section: Section D-D



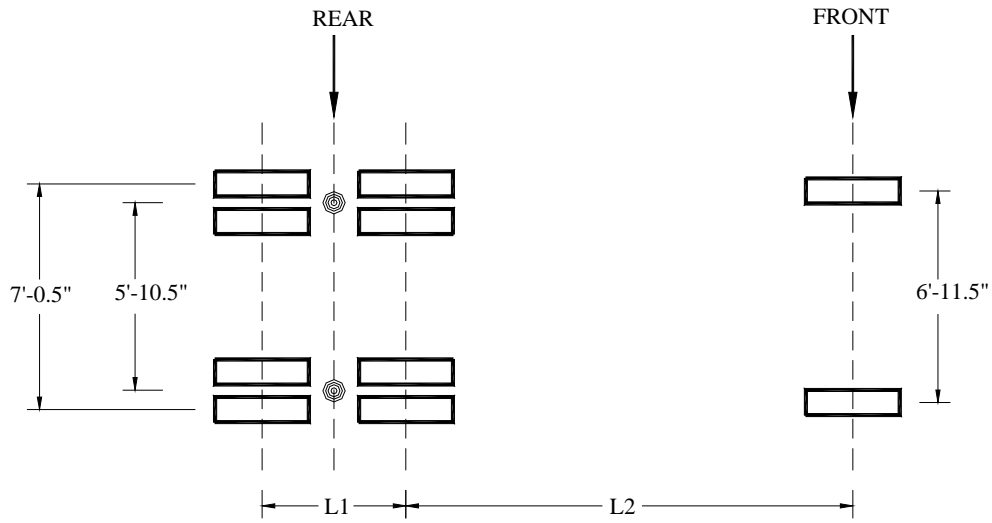
(b) Photograph of a strain gage installation on CFRP plate

**Figure 25. Location of strain gage in “shortly after installation” test.**

### 3.3. After One Year of Service (May 11, 2004)

On May 11, 2004, another follow-up load test was conducted to investigate any change in the behavior of the bridge after approximately one year of service. The bridge was instrumented in the same manner as used in the ‘shortly after installation’ test and the same truck paths were used. The total weight of the load truck used in this test, Truck 3, was 55.48 kips with 14.44 kips on the front axle and 41.04 kips on the rear axle weight. Dimensions and weights of the test truck used in each test are illustrated in Fig. 26 and summarized in Table 4. After the testing was completed, the CFRP plates installed on both exterior and interior girders were visually inspected to assess the durability and condition of the CFRP plates.

Based on the strains obtained in the follow-up tests, the change in stiffness was computed and these results were compared with those from the preliminary analysis which were reported in Chapter 2. Corresponding experimental results will be discussed subsequently.



**Figure 26. Dimension and weight of testing trucks.**

**Table 4. Summary of dimensions and weights of test trucks.**

Truck No.	L1	L2	Weight (lb)		
			Rear	Front	Total
1	4'-7"	14'-5"	38,980	13,780	52,800
2	4'-7"	14'-5"	37,460	15,600	53,060
3	4'-6"	14'-6"	41,040	14,440	55,480

## 4. RESULTS

Results from the previously described field testing and theoretical analyses are presented in this chapter. Where applicable, the field test results are compared with the theoretical results.

For simplification, in the following discussion, the data will be referenced to the four cross-sections (A through D) shown in Figs. 22 and 25a. The following convention was adopted to describe the data in the subsequent figures. The notation starts with a letter “Y” followed by a number (1, 2, or 3), such as “Y1”, to indicate the truck paths that were illustrated in Fig. 23. This, in turn, is followed by another letter and number such as “A1” to indicate a section location (i.e., “A”) and a girder number (i.e., “1”). Finally, the notation is completed with a letter such as “T” or “B” to indicate the location on the girder. For example, “Y2D3 (TB)” represents a data set collected from “Path Y2 - Section D - Girder 3 - top of bottom flange”. Some other notations used are described as follows:

- “Initial Test”: data set taken during “initial test” (Sept. 4, 2002).
- “t = 0 year” : data set taken during “shortly after installation test” (Aug. 19, 2003).
- “t = 1 year” : data set taken during “one year of service test” (May 11, 2004).
- “(T)” : strain on bottom of top flange.
- “(TB)” : strain on top of bottom flange.
- “(BB)” : strain on bottom of bottom flange.
- “(CFRP)” : strain on CFRP plate (installed on bottom of bottom flange).

For ease of interpretation, the data have been normalized based upon the total weight of the truck used in the initial test (i.e., subsequent test data were multiplied by ratio of initial test truck weight to individual test truck weight).

### 4.1. Initial Test

As previously mentioned, an initial diagnostic load test was conducted prior to the installation of the strengthening system to establish a baseline static behavior of the unstrengthened bridge. The goal of this test was to understand the general, global behavior of the bridge and to determine baseline live load strains.

In general, all strains exhibited an elastic response (i.e., strains from all gages returned to zero after each truck crossed the bridge). Flexural tensile and compressive strains at Section A (near Abutment A1) shown in Figs. 27a and 28a) confirmed the presence of rotational end restraint at the abutment. This was as one would expect since the bearings at both abutments are encased in concrete (i.e., integral abutment, see Fig. 1d). Also, with minor variation, composite action was found to be present at all sections of the exterior girder (i.e., the neutral axis location on the exterior girder was found to be close to the top flange in the positive moment region). As previously described in section 2.1, the bridge was originally a non-composite structure until the bridge was widened in 1967 with the addition of exterior girders that were made composite with the deck. The interior girders generally showed non-composite behavior. However, some composite behavior was observed at several locations.

The measured strains indicated good laterally symmetrical behavior that corresponded to the symmetrical configuration of the bridge and the truck paths utilized. For example, the strains due to the truck paths Y1 on Girder 6 and Y3 on Girder 1, respectively, exhibited a similar lateral symmetrical behavior as shown in Fig. 27. A similar pattern was observed for the truck path Y2 on Girder 3 and Girder 4 as shown in Fig. 28. Some of the minor differences in transverse symmetry may be attributed to either differences in local stiffness, differences in rotational end restraint at the abutment, or possible experimental error that might have occurred during the testing (i.e., differences in truck wheel line distribution and/or truck lateral positioning).

## **4.2. Influence of Strengthening System on Live Load Response**

### *4.2.1. General Behavior*

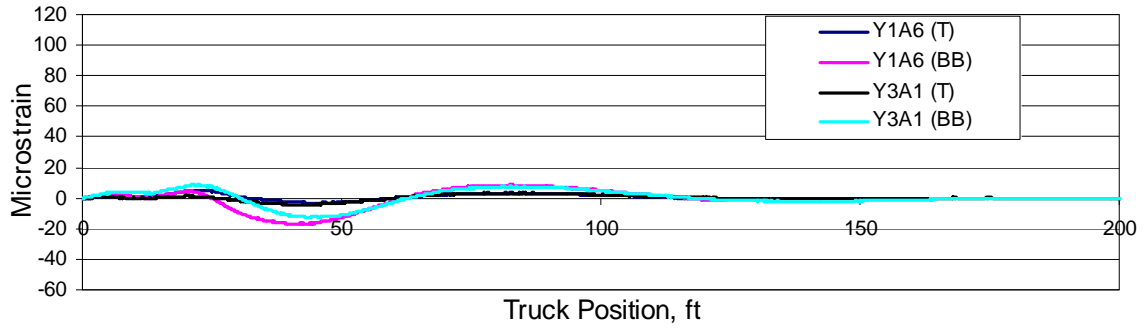
The follow-up load tests were conducted approximately two months and one year after the installation of the strengthening system, respectively, to investigate, by comparing strains, any change in performance resulting from the installation of the CFRP plates and time. Typical strains in the bridge collected during each test are presented in Figs. 29 through 31.

From the preliminary analysis conducted in Section 2.2.1, it was determined that an approximate stiffness increase of 1.2% per layer was obtained for the middle two girders (Girders 3 & 4). By comparing strains in each test, it was observed that the follow-up tests did produce fairly consistent strains with those measured during the initial test. This consistency in strain indicated that the strengthening system did not significantly alter the behavior of the bridge as predicted. Although it is not possible to precisely account for all the sources of strain, it was evident from the consistency of the strain data that the installation of the CFRP plates had minimal impact on changing the stiffness of the bridge. It also indicated that the live load distribution characteristics were virtually the same before and after the installation of the strengthening system.

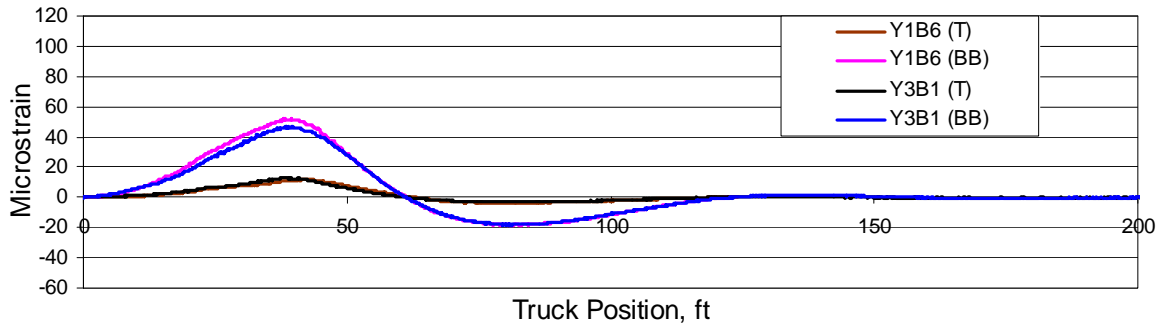
### *4.2.2. Bond Performance*

When a bonding technique is used for strengthening purpose, it is critical to have adequate bonding performance to transfer internal forces to the strengthening material. As was previously mentioned, additional gages were installed on the bottom flange of the strengthened girders (on the CFRP plates) in the center span (Section D-D) to investigate the bonding performance on each strengthened girder.

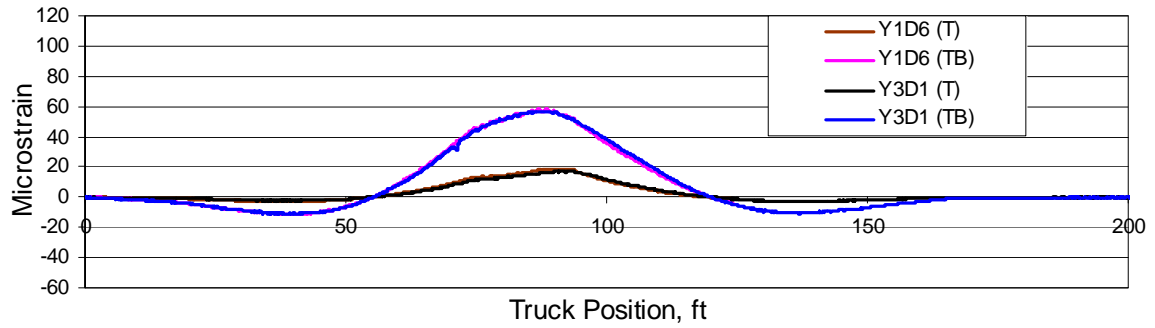
A simple analytical tool was developed based on strain compatibility relationship (see Equation 4.1 and Fig. 32) to predict the extreme fiber strain in the CFRP plate ( $\epsilon_{EXT}$ ) from strains measured on the steel girders. These simple analytical predictions were compared with the strains measured during the post-installation load tests ( $t = 0$  year and  $t = 1$  year).



(a) Section A-A

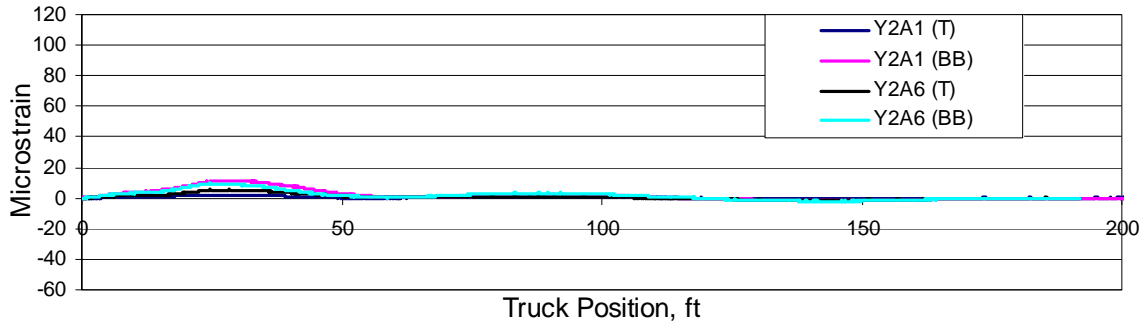


(b) Section B-B

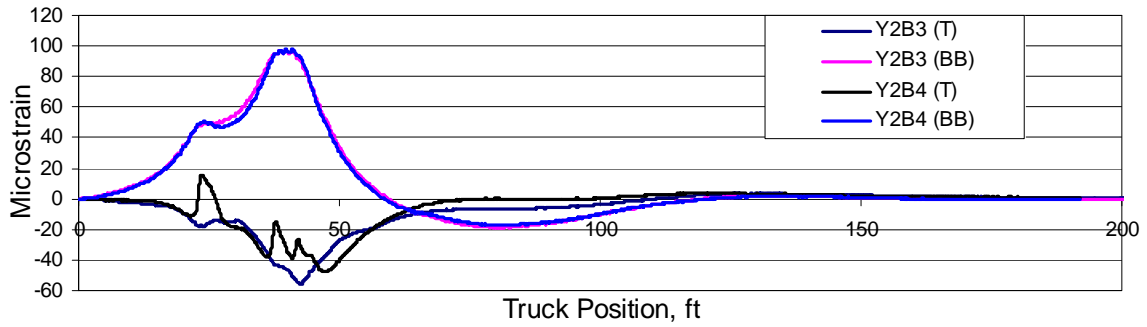


(c) Section D-D

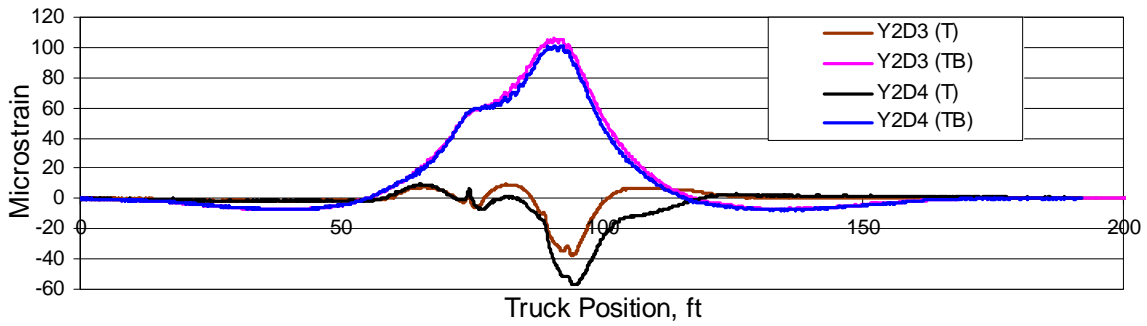
**Figure 27. Before strengthening: strains in Girder 6 (Path Y1) and Girder 1 (Path Y3).**



(a) Section A-A

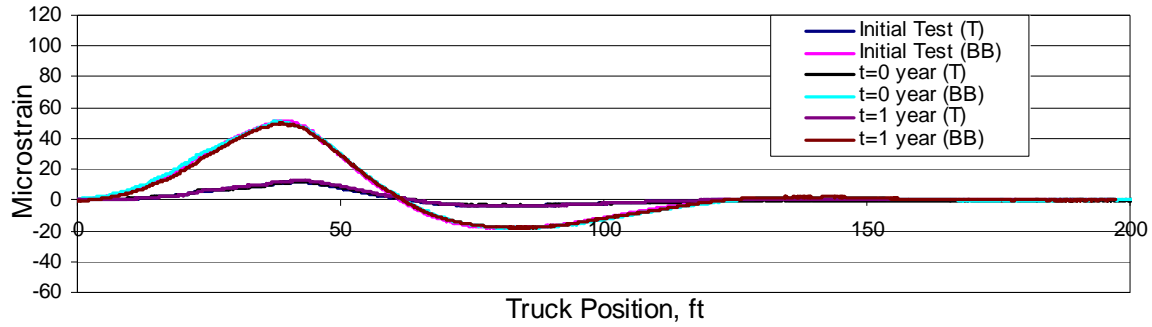


(b) Section B-B

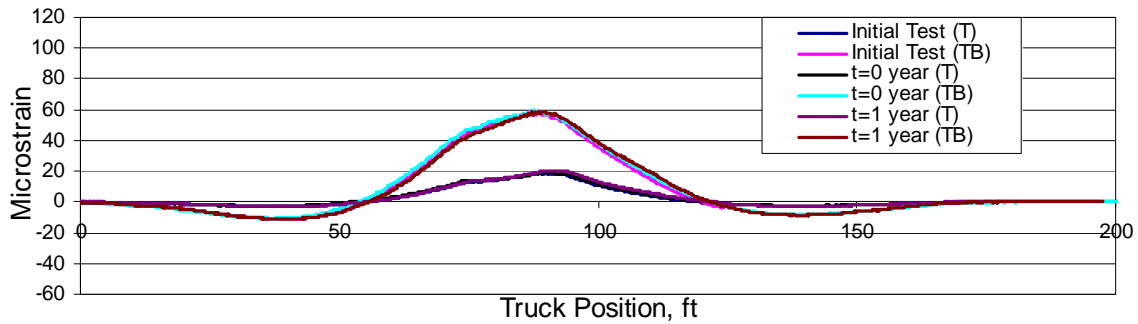


(c) Section D-D

**Figure 28. Before strengthening: strains in the bridge (Path Y2).**

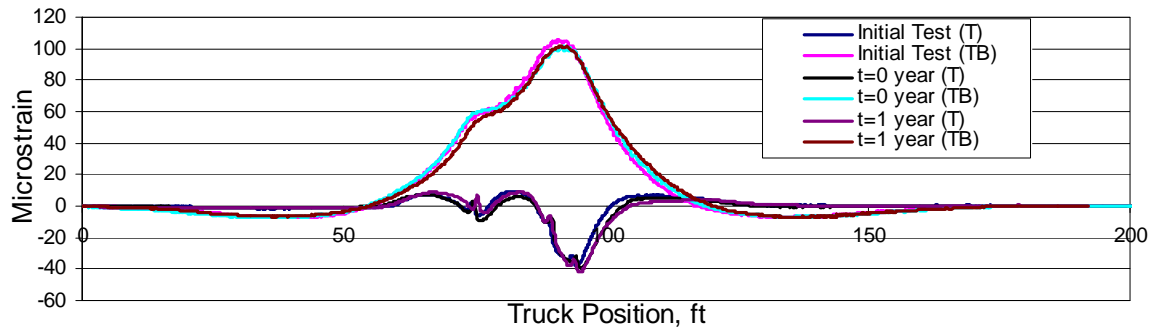


(a) Section B-B

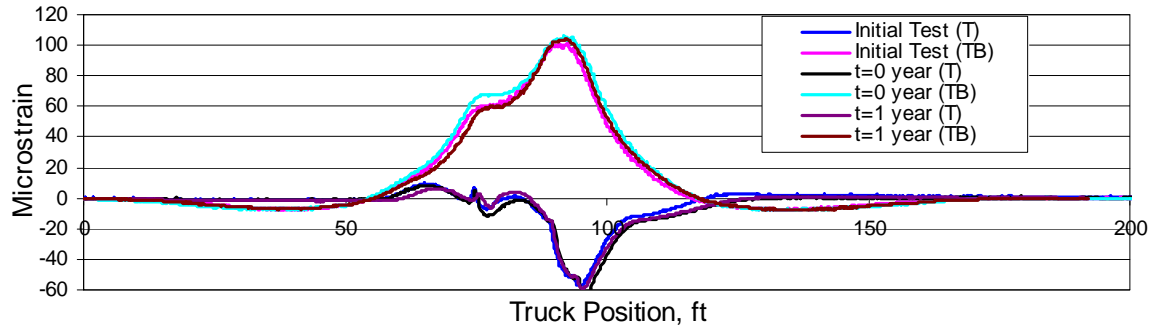


(b) Section D-D

**Figure 29. Before and after strengthening: strains in Girder 6 (Path Y1).**

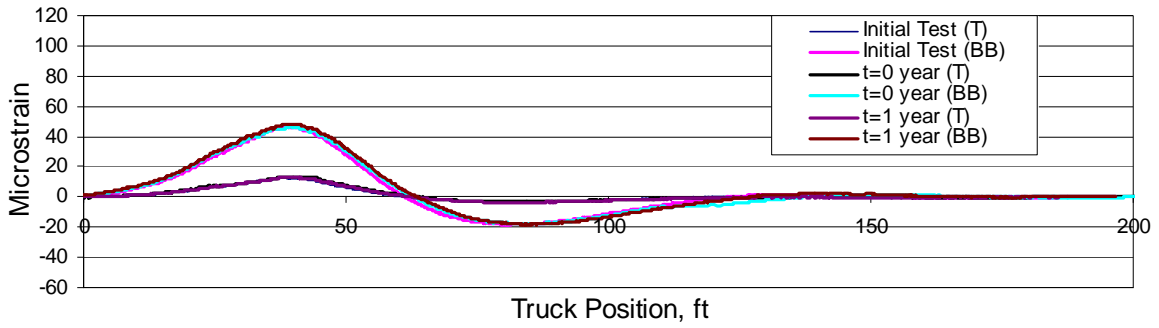


(a) Section D-D, Girder 3

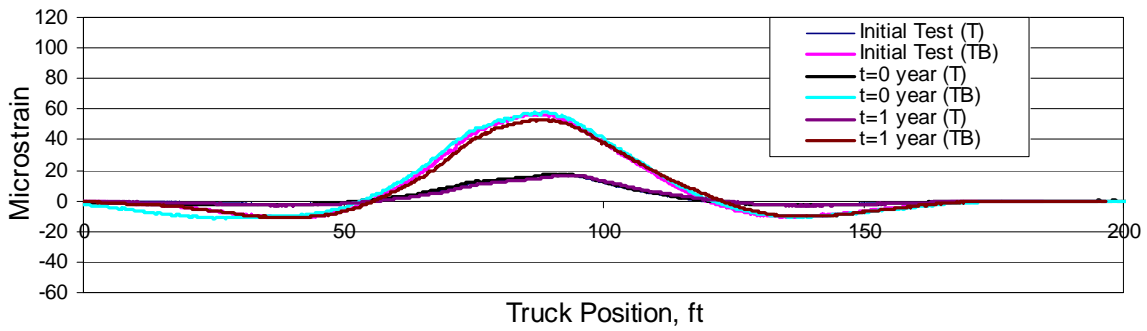


(b) Section D-D, Girder 4

**Figure 30. Before and after strengthening: strains in Girders 3 and 4 (Path Y2).**



(a) Section B-B



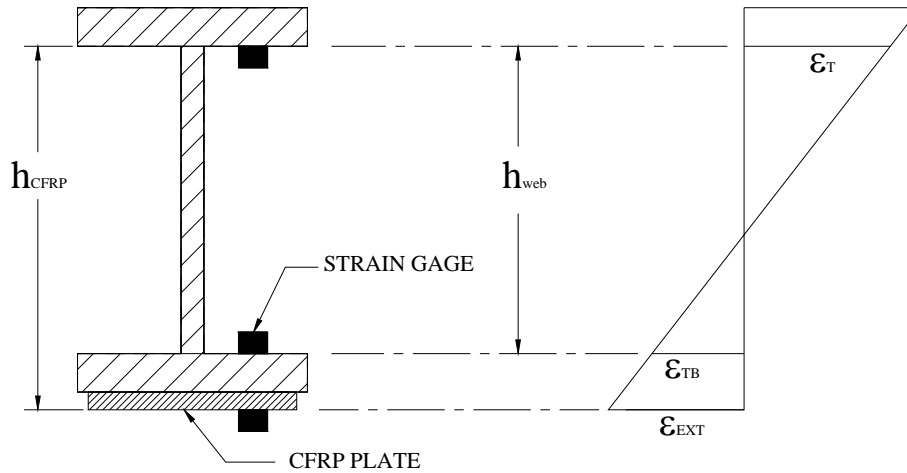
(b) Section D-D

**Figure 31. Before and after strengthening: strains in Girder 1 (Path Y3).**



$$\epsilon_{EXT} = \frac{(\epsilon_T + \epsilon_{TB}) * h_{CFRP}}{h_{web}} - \epsilon_T \quad (4.1)$$

where  $\epsilon_{EXT}$ ,  $\epsilon_T$  and  $h_{CFRP}$ ,  $h_{WEB}$  are strain at the corresponding location and distance from the bottom of the top flange to the corresponding location shown in Fig. 32, respectively.

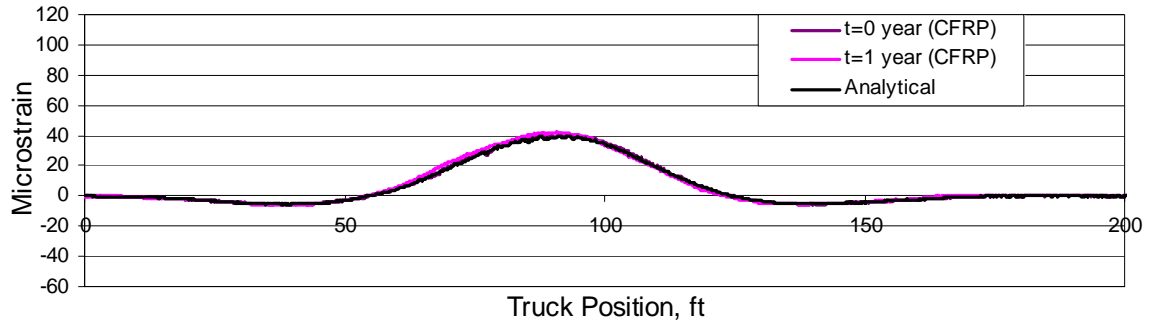


**Figure 32. Girder cross-section with gage location and strain profile.**

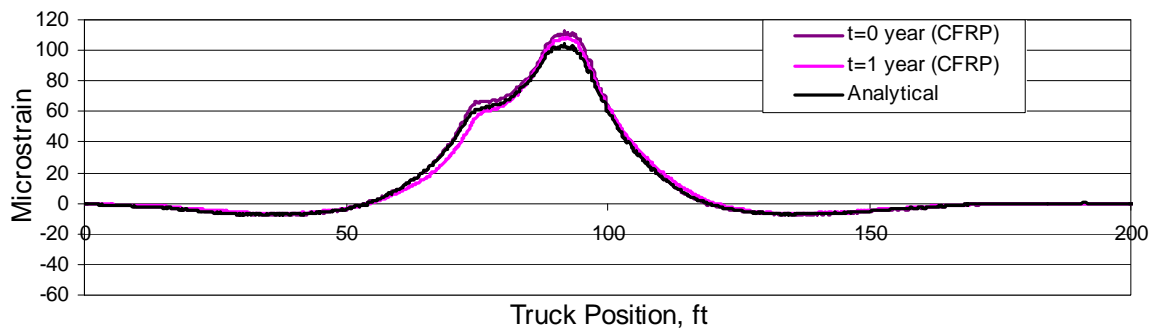
From the comparison, it was found that the difference between the analytical and experimental strain on the average was only 3.8%. Typical similarities between the analytical and experimental strains are shown in Figs. 33 and 34. Considering the sensitivity of the neutral axis location that could change significantly with a small change in top flange strain, this percentage error was considered to be insignificant; thus, it appeared that there was good bond between the girder and CFRP plates.

#### 4.2.3. Change in Stiffness

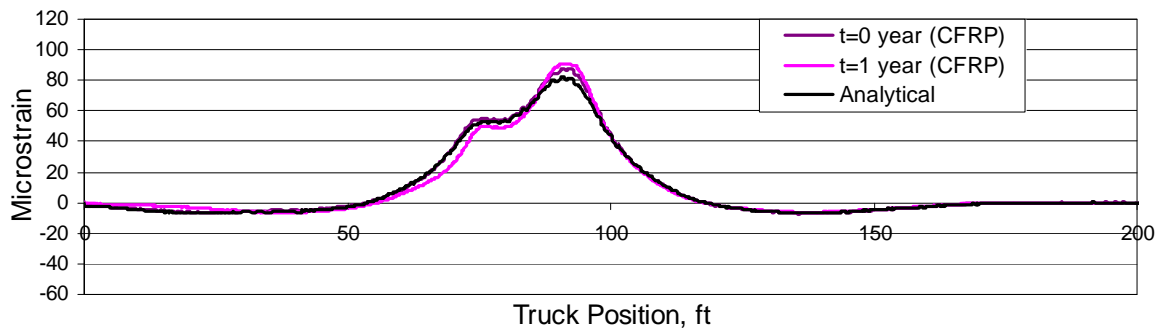
From the preliminary analysis presented in Chapter 2, stiffness increase of approximately 1.2% per layer in Girders 3 and 4 (2.34% for two layers of CFRP plates) was predicted. Given these values, an attempt was made to compute the change in stiffness factor (EI) based on the strain data collected from each test.



(a) Path Y1

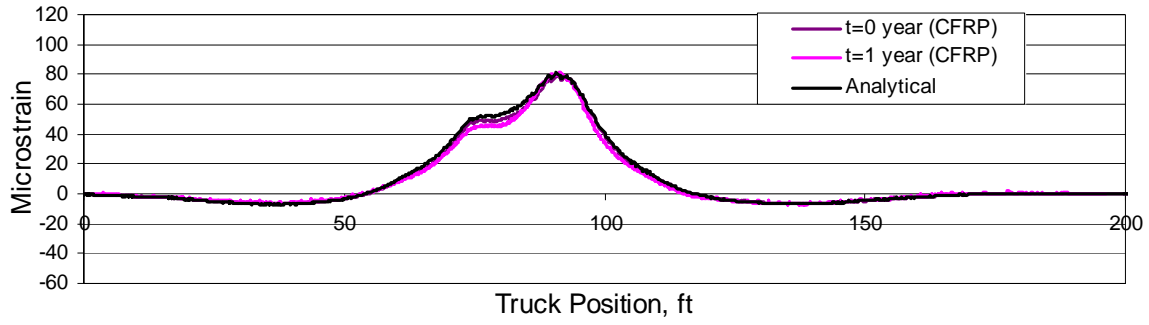


(b) Path Y2

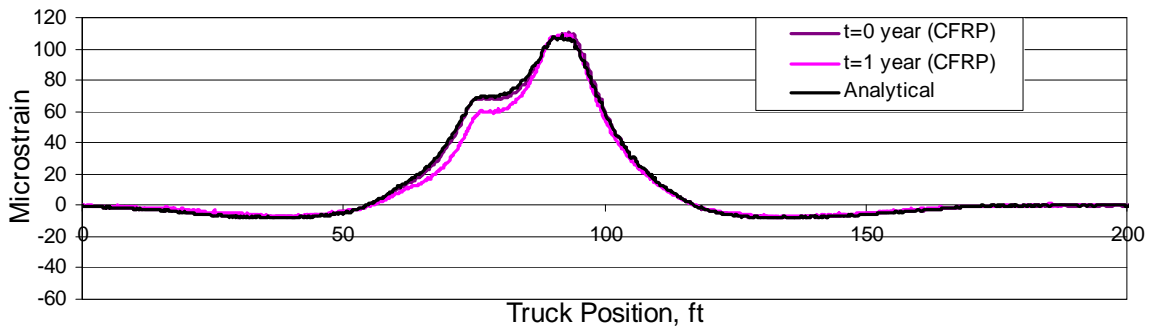


(c) Path Y3

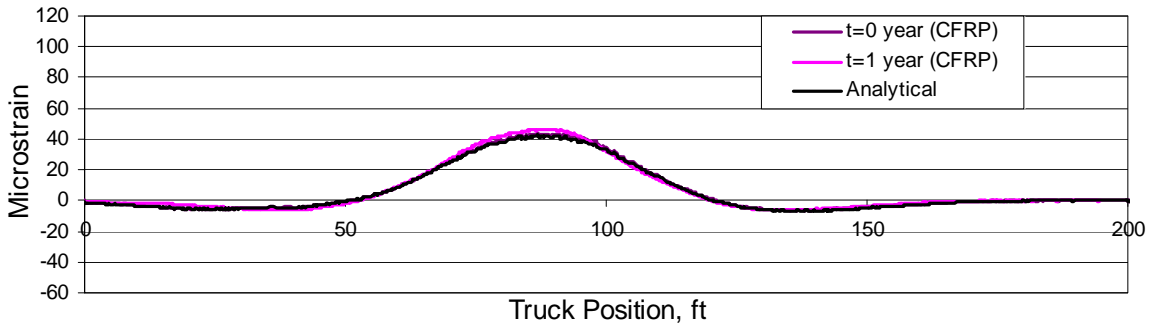
**Figure 33. After strengthening: strain in Girder 3 at Section D-D.**



(a) Path Y1



(b) Path Y2



(c) Path Y3

**Figure 34. After strengthening: strain in Girder 4 at Section D-D.**

To accomplish this, a simple equation (Equation 4.2) was developed through principles of basic mechanics. For bridges, the load can generally be directly related to the bending resistance that can be related to strain through the basic mechanics shown in the equation. The terms used in Equation 4.2 are terms that can be obtained from experimental data and measurement of the bridge elements. Note that the equation assumes plane sections remain plane and the modulus of elasticity ( $E$ ) in the exterior and interior girder was assumed to be constant. Using this equation, the change in stiffness before and after the installation of CFRP plates was determined to be 0.47%, 0.81% on Girder 3 and Girder 4, respectively. From the comparison with the preliminary analysis shown in Fig. 35, it was found that the change in stiffness based on the measured strains was slightly lower than that determined in the preliminary analyses.

$$(EI)_i = \frac{M_i y_i}{\varepsilon_i} \quad (4.2)$$

where

$$M_i = M * (DF)_i$$

$M$  = maximum moment at Section D induced by load truck (wheel line load).

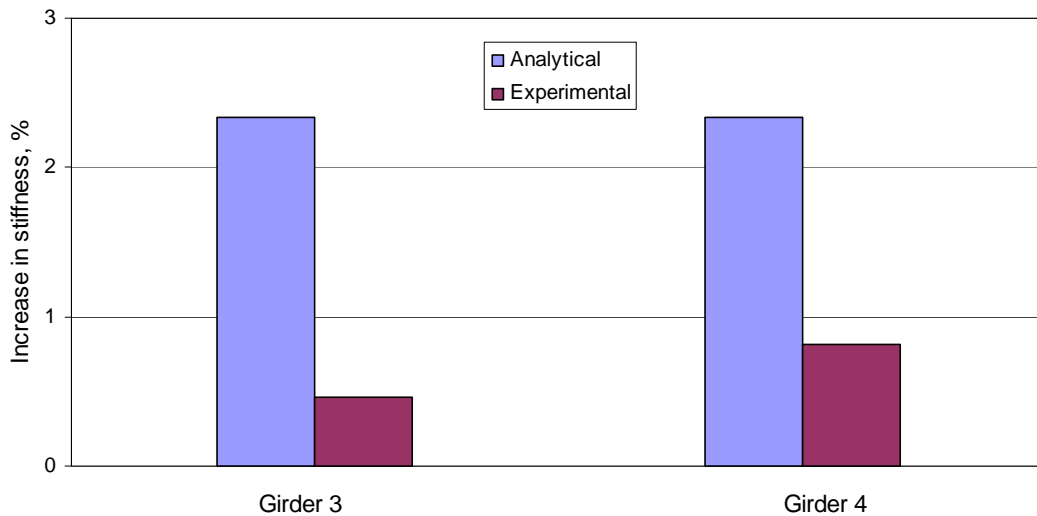
$$(DF)_i = \text{distribution factor} = \frac{\varepsilon_i S_i E_i}{\sum (E_i \varepsilon_i S_i)} = \frac{\varepsilon_i S_i}{\sum (\varepsilon_i S_i)}$$

$\varepsilon_i$  = bottom flange strain of  $i^{\text{th}}$  girder induced by load truck.

$S_i$  = section modulus of  $i^{\text{th}}$  girder.

$E_i$  = modulus of elasticity of  $i^{\text{th}}$  girder.

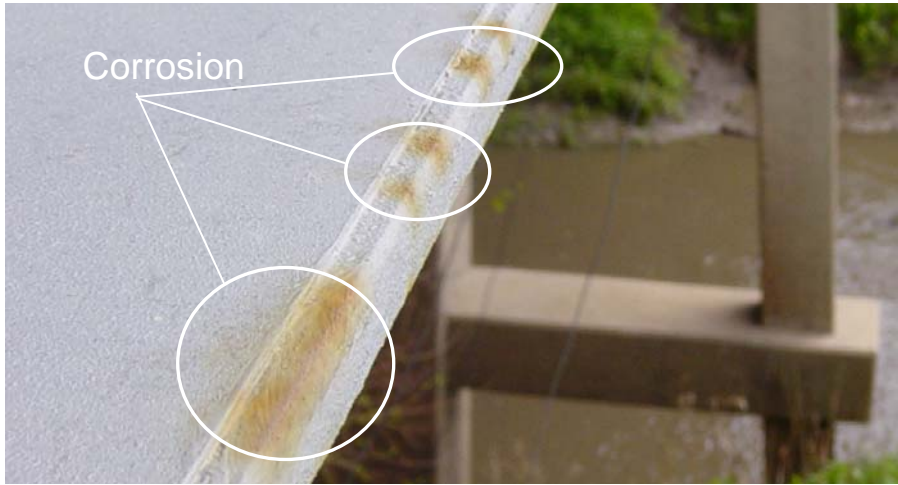
$y_i$  = distance from composite neutral axis (NA) to strain gage on  $i^{\text{th}}$  girder.



**Figure 35. Comparison of change in stiffness at Section D.**

## **4.2. Visual Inspection of CFRP Plates**

All CFRP plates installed on both the exterior and interior girders were visually inspected approximately one and two years after installation (May 2004 and 2005, respectively) to observe any degradation or other changes that may have an influence on the integrity of the strengthening system. In general, no obvious debonding was observed and the condition of the CFRP plates seemed intact. However, some corrosion in the steel girders was observed, as shown in Fig. 36, on the top of the bottom flange of Girder 6 in all three spans. It was initially expected that the coating of paint applied after the installation of CFRP plates would be adequate for corrosion protection. After the visual inspections, however, it is speculated that the applied paint was too thin or had defects and that rain or snow might have penetrated the protective coating. Therefore, it is recommended that thicker or more coatings of paint or other water-proof material be applied to prevent weather-induced corrosion and, thereby, protect the integrity of the strengthening system.



(a) West end span (May 2004)



(b) Center span (May 2004)



(c) East end span (May 2005)

**Figure 36. Corrosion on the top of the bottom flange of Girder 6.**

## **5. SUMMARY AND CONCLUSIONS**

### **5.1. Summary**

The primary objective of this project was to investigate and evaluate the effectiveness of CFRP plates to strengthen an existing, structurally deficient steel girder bridge. The bridge strengthened with CFRP plates was a 150 ft x 30 ft three-span continuous I-girder bridge in Pottawattamie County, Iowa, on State Highway IA 92. The research program consisted of several tasks, with the main emphasis on the design and installation of the strengthening system and associated field-testing. Before the CFRP plates were installed, a diagnostic load test was conducted on the bridge to establish a baseline static behavior of the unstrengthened system. The strengthening system consisted of preparation of the bonding surface and installation of the CFRP plates. The design of the CFRP plates, based on the Iowa legal loads, utilized the LFD approach. The implemented design consisted of one to three layers of CFRP plates bonded to the overstressed tension regions of the girders with a high strength epoxy adhesive. The CFRP plate strengthening system consisted of three primary components: CFRP plates, primer, and a structural epoxy. The installation of the strengthening system was followed by additional load tests performed shortly (two months) after the installation and approximately one year after being in service to assess the short-term effectiveness of the strengthening scheme.

### **5.2. Conclusions**

Although relatively time-consuming to install, it is believed that with more experience, construction crews could develop efficient techniques and tools for installing CFRP plates. Based upon the design process, analyses, and field investigations, the following conclusions and observations were made:

- CFRP plates are a relatively easy strengthening system to design.
- Special care must be taken to ensure that galvanic induced corrosion does not occur from direct contact of the CFRP plates and steel girders. A structural primer is one possible way to provide a boundary between the two materials.
- From the analytical predictions and experimental testing, it was found that installing CFRP plates had minimal impact on the behavior of the subject bridge, while at the same time contributing notably to the strength of the system. The analytical study indicated that, with the relatively small change in stiffness in the strengthened member, a considerable increase (approximately 10% increase per layer up to three layers) in the moment capacity on the strengthened girders was attainable and, thereby improved the overall live load carrying capacity of the bridge. In terms of strength, adding additional CFRP layers had a diminishing impact.

- The handling and installation of the CFRP plates was relatively labor intensive and required more training and time when compared to a post-tensioning strengthening system. A crew of at least three people was needed to install the CFRP plates (a day per layer: three days for three layers). Efficient installation of CFRP plates requires the development of standardized construction practices.
- The bond performance between the girder and CFRP plates was good and, therefore, it appeared that no significant debonding occurred after one year of service.
- Paint coating applied after the installation may not have been sufficient enough to protect the strengthening system from direct environmental exposure (e.g., rain, snow, etc.); minimal to moderate corrosion had occurred on the top of the bottom flange on the top of the bottom flange on Girder 6 and the CFRP plates. Therefore, it is recommended that thicker or more coatings of paint or other water-proof coating be applied to prevent the weather-induced corrosion and, thereby, protect the integrity of the strengthening system.
- Although an economic analysis was not conducted for this project, it is the authors' opinion that the use of the CFRP plates is a viable, economical strengthening alternative for steel girder bridges.



## **6. RECOMMENDATION**

The use of external bonded CFRP plates appears to be a feasible, practical alternative to traditional methods for strengthening an existing bridge. However, the use of CFRP plates bonded to the steel girder bridge is a relatively new technology that has not been broadly accepted by specifications or code authorities. Therefore, it is recommended that other steel bridges be fortified with this strengthening system to collect additional behavioral data and more studies be completed on the characteristic of this strengthening system such as its fire resistance, long-term creep characteristics, and degradation of bond with time. In addition, more extensive studies to fully understand its failure mechanisms and the development of design standards are needed so that the strengthening system will be more widely used. It is further recommended, for this project, that any signs of delamination or bond failure at a bond line on the strengthened girders be closely observed and, if detected, appropriate actions be taken since these types of failure would lead to a significant reduction in strength.



## **7. ACKNOWLEDGMENTS**

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