

Strengthening Steel Girder Bridges with CFRP Plates

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ABSTRACT

A large percentage of short and medium span, steel bridges are deteriorating due to age and environment effects. Although these bridges are still in service, many need strengthening due to increases in legal live load and/or loss of section. This paper presents the results of two investigations – a laboratory study as well as a field study – in which carbon fiber reinforced polymer (CFRP) plates were used to strengthen composite steel stringers.

In the laboratory investigation, small-scale steel-concrete composite beams were tested; there were control beams (no damage or CFRP applied), damaged beams (a percentage of the bottom flange removed), and damaged beams with CFRP applied to the bottom flanges and/or webs. In all cases the strength of all damaged/repared beams was fully restored to their original undamaged state. This paper presents both details on the strengthening system and on the behavior of undamaged, damaged, and repaired scale-model specimens.

Based upon the laboratory results, a second project was undertaken in which an existing steel-girder bridge was strengthened using CFRP plates. This bridge is a 150 ft x 30 ft three-span continuous rolled I-beam bridge in southwestern Iowa. The original non-composite four beam bridge was widened in 1965 by adding two additional composite beams. A recent rating of this bridge determined that several of the original beams were understrength in the positive moment regions, thus CFRP was bonded to the positive moment regions of the bottom flanges of the two original interior beams and the “new” exterior beams. At some locations on the exterior beams, the plates were installed on the top surface of the bottom flange to investigate the performance and in-service durability under detrimental environmental conditions. This bridge has been load tested three times: before and after installation of the CFRP plates, and approximately one year later to determine the effectiveness of the strengthening system. Results are presented to illustrate this effectiveness.

STRENGTHENING STEEL GIRDER BRIDGES WITH CFRP PLATES

INTRODUCTION

Although glass-based composite materials have been used in applications involving concrete and masonry structures, their low-tensile modulus characteristic has made them ineffective for strengthening steel members. In recent years, however, composite materials with carbon fiber reinforced polymers (CFRP) have become more available and the use of CFRP materials has been gaining potential of becoming part of cost effective steel bridge solutions. This is primarily due to the fact that they are highly resistant to corrosion, have a low weight, and have a high tensile strength [1]. In the past, the use of bolted steel cover plates or angles was a common option for strengthening deteriorated/obsolete bridges. However, the time and labor involved to attach such a retrofit can sometimes be prohibited. It is also the authors' expectation that the overall savings resulting from the long-term performance of the CFRP materials, the labor cost associated with the installation, and the high strength likely offset the initial material costs which are higher than structural steel. To study the applicability of using CFRP on steel bridges, a project was initiated by Iowa State University to investigate the effectiveness of strengthening steel girders. Following that successful laboratory investigation, a second project, funded through the Federal Highway Administration's (FHWA) Innovative Bridge Research and Construction (IBRC) Program, was initiated by the Iowa Department of Transportation to further investigate the effectiveness of using CFRP composite materials by strengthening an existing steel girder bridge.

LABORATORY INVESTIGATION

The objective of the laboratory investigation was to determine the potential viability of using CFRP plates to strengthen steel girder bridges. This was principally accomplished by studying the performance of scale model specimens under controlled loading conditions. In addition, the behavior of the various specimens was analytically predicted to test the applicability of various prediction tools.

Analytical study

An analytical study was performed to predict the behavior of the strengthened/repared composite member and deformations. Nonlinear constitutive relations were assumed for both the concrete slab and the steel beam, and a linear relation was assumed for the CFRP. The numerical model developed, which was based on satisfying internal equilibrium and strain compatibility, was used to evaluate the response of the section and the member in the elastic, inelastic, and ultimate states. After the model was verified with experimental results, a parametric study was conducted to investigate the most important geometrical variables as well as the material variables.

Description of specimens

A total of 10 scale model steel-concrete composite beams were tested in the laboratory investigation; the six that are presented herein are described in Table 1. In all cases the steel beams were W8x15 grade A572 structural steel. A composite concrete slab 32 in. wide by 3 in. thick was used in all specimens. Shear studs (1/2 in. in diameter by 2 in. long on 2 in. centers) provided the shear connection between the slab and the steel beams.

The experimental program consisted of testing two undamaged (control) steel composite beams and four steel composite beams that were damaged by removing part of their bottom flange (i.e. a percentage of the bottom flange area) to simulate corrosion and then repaired by adding CFRP to restore their original strength.

The following notation is used herein to describe the specimens. Undamaged beams are designated with "U" while the designations for damaged beams start with the letter "D". In the case of the damaged control beam "D" was used alone to indicate that no repair was installed. In the case of the repaired beams, the letter "D" is followed by two numbers that indicate the percentage of damage to the bottom flange. For example, "50" means that 50% of the original bottom flange area was removed ("damaged"). This number is followed by a letter-number combination such as "R1" to reflect the repair scheme used. This combination is followed by a letter and two numbers such as "E22" to reflect the modulus of elasticity (MOE) in ksi of the CFRP plate being used. For example, "D50R1E29" describes a damaged beam with 50% of the bottom flange area removed that has been repaired using scheme 1 with a CFRP plate that has a MOE of 29,000 ksi. One of the repair schemes used (Repair 1) is shown in Fig. 1a. The second repair scheme employed (Repair 2) differs from Repair 1 in that CFRP plate is also bonded to the bottom flange.

Materials

The composite beams were cast using normal weight concrete for the deck with an average compressive strength that ranged from 4,500 psi to 5,000 psi as shown in Table 1. The material properties of the steel beams, provided by the supplier and determined according to ASTM specifications were yield and tensile strengths of 52.7 ksi and 72.0 ksi, respectively. Strengthening and repair of the beams was accomplished by using 0.055 in. thick, and either 1.97 in., or 3.94 in. wide CFRP plates installed along the entire length of the beams (from support to support). Two grades of uni-directional pultruded plates with a 70 percent fiber volume fraction and were used in this investigation. CFRP has a stress-strain relationship that is linear elastic up to failure. The only difference between the two grades is the modulus of elasticity; one grade has a modulus of elasticity of 22,000 ksi while the other has a modulus of elasticity of 29,000 ksi. Bonding of the CFRP plates to the steel beams was achieved with a 100% solids, high modulus, high strength, and moisture insensitive epoxy system designed for bonding carbon fiber laminates to most building materials. The tensile strength and the ultimate elongation of the epoxy were 10 ksi and 2 percent, respectively.

CFRP Installation

The first step in the installation of the CFRP system was preparation of the beam surface to receive the plates. According to the supplier of the CFRP, the steel should be free from paint, oil, scales, rust, dust or any other contaminants that might weaken the bond between the steel and the CFRP. To insure a sound bond, the tension flanges and the lower part of the web of the beams were sand blasted to a bare metal surface and the surfaces of the steel beam and the CFRP laminates were cleaned with acetone just prior to the application of the epoxy. The epoxy between the steel (beam webs and flanges) was permitted to cure at room temperature for a minimum of seven days prior to testing.

Test set up

All beams were tested in a four point bending configuration under static loads. Each test beam was 11 ft long with a 10 ft clear span (i.e., 6 in. of overhang at each support). The load was applied using a manually operated hydraulic jack at load points which were located 48 in. from the ends of the beam. The load was measured using a 50 kip capacity load cell at each load point. All tests were load controlled and data were recorded at a load increment of 500 lbs. It was necessary to use a small load increment in order to capture the behavior in the nonlinear stage. All loads reported here are point loads (i.e., the load being applied by each hydraulic jack).

Instrumentation

To observe the behavior of the beams, extensive instrumentation was installed on the beams (See Fig. 1b and 1c). Strains, loads, deflections, and slip were measured at the desired locations. Strains in the concrete slab as well as the steel beam were measured using electrical-resistance strain gages. Vertical deflections of the beams, the primary measurement metric used herein, were measured using string potentiometers positioned at the midspan and the quarter span points. The slip between the steel beam and the concrete slab was measured using Direct Current Differential Transducers (DCDT's) positioned at the two ends of the beams. Strains along the CFRP plates were also measured at a variety of locations including at the cut off points.

Results

Undamaged Beam Behavior

In the case of the undamaged control specimens (Beams U_1 and U_2), the behavior was typical of a composite steel beam. Since the response of these two beams were almost identical, only the results of one of them (Beam U_2) will be discussed in the subsequent sections and will be referred to simply as Beam U. The behavior of Beam U was linear until the steel beam began to yield at which time the beam deflected with only a slight increase in the applied load. Failure occurred when the concrete crushed after a significant period of yielding. Several longitudinal and diagonal cracks were observed on the top of the concrete slab directly above the shear studs; these cracks formed when the applied load was very close to the failure load (approximately 30,000 lb). These cracks didn't appear to alter the behavior as was indicated by the good agreement between the experimental and the theoretical behavior.

Effect of repair schemes

Discussion of the test results as well as the analytical results for the damaged beams is presented in this section. This discussion will focus on Beams D50, D50R1E29, D50R2E29, and D75R1E29. The purpose of testing these specimens was to investigate the effectiveness of two different FRP repair methods on various levels of damage.

Midspan deflections of Beams U and D50 are shown in Fig. 2a. These beams behaved as expected as is illustrated by the very good correlation between the measured and the analytical data (for details on the analytical model, see [2]). As may be seen in this figure, large ductility was observed prior to failure as indicated by the large midspan deflections. It is interesting to note that a more ductile behavior was observed in Beam D50 than in Beam U. At the failure load, Beam D50 (approximately 22,000 lb) deflected approximately 2.4 in., while Beam U deflected approximately 2.0 in.; this is because Beam D50 had less bottom flange area (i.e., more under reinforced). Failure of Beam D50 was due to crushing of the top surface concrete, similar to that of Beam U discussed previously.

Analytically predicted midspan deflections of Beam U, Beam D50, and Beam D75 are compared in Fig. 2b. As may be seen in this figure, both the stiffness and strength are significantly reduced as a result of the damage induced to the bottom flange.

Shown in Fig. 3a is a plot of the deflections of Beam D50R1E29 measured at the midspan; for comparison the analytically predicted deflection at midspan is also shown. This beam was damaged in a manner similar to Beam D50 and the repair consisted of attaching the 4 in. wide CFRP plates to both sides of the web. As seen in this figure, a very close agreement between the analytical and the measured midspan deflections existed until the applied load reached 33,000 lbs. Unfortunately, the beam did not reach the theoretical maximum load due to a

premature slip failure at the concrete-steel interface. The slip was initiated by the formation of longitudinal cracks on the top surface of the concrete slab along the lines of the shear studs. These cracks were formed as a result of high transverse tensile stresses induced by the shear studs. The fact that these specimens were scaled specimens required the use of thin concrete slabs; as a result, it was not possible to put double layers of reinforcement in the transverse direction. It appears that the internal transverse reinforcement was not sufficient to resist the tensile stresses in the transverse direction caused by the shear connectors. To overcome this problem in the remaining test specimens, external reinforcement was used to supplement the transverse reinforcement in the concrete slab. To accomplish this 4 in., wide uni-directional CFRP plates were bonded to the top of the concrete slab. The plates were cut in 20 in. lengths and placed on 14 in. centers, similar to the distribution of the transverse steel reinforcement. It should be mentioned that the addition of the CFRP plates to the top of the concrete slab will have negligible effect on the strength of the beam since the orientation of the plates is transverse to the length of the specimens.

Despite the fact that Beam D50R1E29 did not attain its ultimate predicted load (due to the reasons described above), reasonable results were achieved. This can be seen from the comparison of the responses of Beams D50, D50R1E29, and U illustrated in Fig. 3b, where it is obvious that both stiffness and strength were increased in the repaired beam (Beam D50R1E29). As a measure of elastic stiffness, deflections of the three beams were compared at the load that caused yielding in Beam D50 (15,000 lbs). The measured deflections of Beam D50, Beam D50R1E29, and Beam U at an applied load of 15,000 lbs were 0.396 in., 0.315 in., and 0.240 in., respectively. This indicates that approximately 50 percent of the lost stiffness in Beam D50 was restored in Beam D50R1E29. In the inelastic range, significant stiffness was gained as observed in Fig. 3b. In addition, the ultimate strength of the original beam (Beam U) was fully restored; in fact the ultimate load of Beam D50R1E29 reached 33,000 lbs, exceeding the failure load (31,000 lbs) of Beam U.

Beam D50R2E29 was repaired using 2 in. wide CFRP plates applied to both sides of the web and to the bottom flange. The total area of CFRP plates used in this beam was 75 percent of the area of CFRP used in Beam D50R1E29; however, theoretically, both schemes should result in an equivalent strengthening effect since it is more efficient to attach the CFRP plates to the bottom flange (i.e., a larger moment arm). However, in certain field situations it may not be possible to attach the CFRP directly to the bottom flange (e.g., due to corrosion or riveted joints). In this case, the web may be used alone to strengthen the beam as the case in Beam D50R1E29, or a combination of the bottom flange and the web as is the case with Beam D50R2E29. The responses of Beam U, Beam D50, and Beam D50R2E29 are compared in Fig. 4a. Similar comments to those given for Beam D50R2E29 can be said about the elastic response of Beam D50R1E29. Interestingly, the ultimate applied load for Beam D50R2E29 was 38,000 lbs, which is 52 percent higher than that of Beam D50 and 23 percent higher than that of Beam U. It can also be observed that the stiffness was significantly increased in the inelastic range.

More damage was induced in Beam D75R1E29 than in the previously damaged beams. In this beam, 75 percent of the bottom flange was removed which represents severe damage. More than 32 percent of the beam strength was lost due to this level of damage. The beam was repaired in a similar way to that of Beam D50R1E29 (i.e., the CFRP plates were attached to the sides of the steel beam web since most of the bottom flange was removed). Even though the measured ultimate load (33,000 lb) was less than the predicted ultimate load, the repaired beam was capable of resisting loads that were higher than the ultimate load of the undamaged beam (Beam U) as illustrated in Fig. 4b. As may be observed in Fig. 4b, no apparent increase in the stiffness was achieved in the elastic range. However, the observed experimental deflections of the previous beams, close to the yield load, were always larger than the analytically determined deflections.

Summary of Laboratory Investigation

Two grades of CFRP (a modulus of elasticity of 29,000 ksi and 22,000 ksi) were used in this investigation and only minor difference was found in the behavior of two types of CFRP. In summary, it can be stated that the repair methods investigated were very effective. The stiffness and the strength in all repaired beams were considerably increased. Only Beam D50R1E29 failed prematurely due to the slip failure at the concrete-steel interface as a result of the formation of longitudinal cracks. The predicted and measured gains in strength were calculated with respect to the undamaged case and ranged between 6 and 24 percent. However, when the measured ultimate loads were compared to the damaged control beams (Beams D50 and D75), the range was much higher. For example, in the case of D50 specimens, the gain in strength was found to be 32 and 52 percent for Beams D50R1E29 and D50R2E29, respectively.

Three modes of failure were observed which depended on the repair scheme used. In the case of the control specimens (i.e. Beams U and D50), the failure consisted of crushing of the concrete following yielding of the steel beams. The second mode of failure is a combination of slip at the concrete-steel interface followed by rupture of the CFRP plate. This mode of failure was observed in the repaired beam using Repair 2 (i.e. Beam D50R2E29). The third failure mode was characterized by a combination of slip failure at the steel-concrete interface followed by crushing of the concrete slab. This was observed in Beam D75R1E29 that had 75% of the bottom flange removed. There was no debonding or delamination of the CFRP plates in any of the specimens tested.

FIELD INVESTIGATION

Following the successful completion of the laboratory investigation, a field investigation was initiated through the FHWA IBRC program. This allowed the lessons learned in the laboratory to be directly applied to a bridge with known overstressed regions. The objective of this work was to: investigate field construction practices for bonding FRP to steel beams, determine the long-term performance of FRP, and to evaluate the feasibility of using CFRP plate in future steel girder strengthening projects.

Bridge Description

The bridge (Number 7838.5S092) selected for strengthening is the 150 ft x 30 ft three-span continuous I-beam bridge shown in Fig. 5. The bridge is located in southwest Iowa in Pottawattamie County on 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 beams and four interior beams (two W27x91s and two W27x98s). As can be seen in Fig. 5, full depth 8-in. thick concrete diaphragms were cast between the W27x91 interior and W27x84 exterior beams 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. The roadway width is 30 ft allowing for two traffic lanes and a shoulder on each side. The bridge has curbs integral with the deck with concrete guardrails connected to the curbs.

Both abutments have a few random vertical and horizontal hairline cracks with a small delaminated area at the top of one of the abutments. The top and bottom of the deck exhibit several hairline and narrow transverse cracks, and a few small delaminated areas. Both curbs have moderate hairline cracks through their length. The top of the deck has Portland Cement patches at several locations, and a few small spalls and delaminated areas along the construction joints.

Strengthening System Design

The design of the CFRP plate strengthening system was completed to support Iowa legal loads utilizing the Load Factor Design (LFD) approach [3]. Based on the analysis completed by the bridge owner, it was found that the positive moment region of the two center beams, in both the end and center spans, were overstressed by 0.55 ksi and 1.57 ksi, respectively. This section describes the design process used to predict an increase in moment capacity due to the addition of the CFRP plates. In addition, the strengthening scheme that was finally designed for the CFRP plate strengthening system is described.

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

- The beam is symmetric and initially straight.
- The steel is elastic-perfectly plastic.
- There is a “perfect bond” between the CFRP plate and steel beam.
- The CFRP has 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.
2. Assume the CFRP plate reaches its ultimate strain ($\epsilon_{CFRP} = 0.015$). Thus, the extreme tension fiber strain (ϵ_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 based on the strain compatibility relationship.
5. From the strain profile generated in Step 4 and stress-strain relationship for each material (i.e., constitutive relationship), generate the stress distribution in each section.
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 $F_{Compression} \neq F_{Tension}$, assume a new neutral axis location (Step 3) until the solution converges and equilibrium is satisfied.
 - when $F_{Compression} = F_{Tension}$, move on to the next step.
8. Calculate the total moment (moment capacity) by summing the moments of the internal forces about a convenient axis.

The change in moment capacity with varying numbers of CFRP plate bonded to the bottom of the bottom flange of the subject bridge is shown in Fig. 6. From this graph, it is clear that the efficiency of the CFRP plates decreases as the amount of CFRP plates used increases. For example, the increase in moment capacity was the greatest (12.3%) with only one layer of CFRP plate utilized and only increased an additional 3.2% when adding a fourth layer to a three-layer system. Additional details on the model used to predict the strength change can be found in [4].

The stiffness increase due to the addition of CFRP plates to the bottom of the bottom flange was also investigated as shown in Fig. 6. Based on the analysis, it was generally found that an approximate 1.2% stiffness increase per layer was obtained for the middle two beams (those that were originally understrength). This theoretical change in stiffness is qualitatively compared with field test results later.

Following the procedures outlined previously, it was found that the overstressed beams could be adequately strengthened by the addition of CFRP plates bonded to the bottom flange of the beams. The required amount of CFRP plate needed for strengthening the middle two beams was determined to be one layer 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 the middle two beams were found to be in need of strengthening, several modifications were made to the overall design to:

- Investigate the ease of installation of multiple CFRP layers.
- Evaluate the durability of the installation with load tests and visual inspections.
- Compare the behavior of different strengthening schemes.

To this end, the following modifications were made with the final layout summarized in Fig. 7:

- The exterior beams were strengthened to investigate CFRP plate durability on exterior girders.
- All of the beams in the east end span had three layers (as opposed to the one layer required for strength) installed to evaluate performance and construction of multi layers.
- The south exterior beam in all three span 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 (on the exterior side) to investigate durability under direct environmental exposure.

Installation of Strengthening System

Temporary scaffolding was constructed underneath the bridge in both end spans to provide easy access to the beams for installing the strengthening system. Access to the center span was provided through the use of a “snooper” truck. In the first step of the installation process the steel beam surfaces to which the CFRP plates were to be bonded were roughened to a coarse sandpaper texture by sandblasting to remove any lead free paint and unsound material. After sandblasting, the beam surface was cleaned with acetone. The bonding surfaces of the CFRP plates were also cleaned at that time with acetone. After the bonding surfaces were dry, the prepared beam surface was treated with a thin layer of FRS Primer to prevent corrosion induced by a galvanic reaction between the beam surface and carbon fibers. Once the primer had set, ECS 104 structural epoxy was applied to both the pre-cut CFRP plates and the beam surface using a 1/8 in. v-notched trowel. The plate was then carefully placed on the beam surface, 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. A smooth trowel was then used to remove any excess epoxy from the edges of the CFRP plates. After all plates were installed and the epoxy had cured, all surfaces were painted. Photographs of the completed installation are presented in Fig. 8.

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). It should, however, be pointed out that subsequent installations should be completed

much quicker as the construction crews were “learning” the installation process on the subject bridge.

Field Test Setup

An initial diagnostic 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 monitor the overall global response of the bridge to live load. As can be seen in Fig. 9a, a total of thirty-six strain gages were installed on the bridge with twenty-four gages on the positive bending moment regions and twelve gages on the negative bending moment regions.

Three different truck paths were used to examine the performance of the bridge. These truck paths were chosen so that maximum strains could be generated in select girders. For convenience, load paths are referred to as Y1, Y2 or Y3 as shown in Fig. 9b. All tests were conducted from west to east. The Iowa DOT provided the load truck which 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 - 8 in. wide between the centers of the front tires, and 6 ft wide between the rear tires.

The follow-up load tests were conducted approximately two months and one year after the installation of the strengthening system to assess changes in behavior due to the addition of the CFRP plates or time. The procedures used in the follow-up load tests were the same as those used in the initial test except for the truck weight and the number of strain transducers. In these tests, four additional transducers were installed on the CFRP plates as shown in Fig. 9c in the center span to determine the bond performance.

Results

Results from the field testing and theoretical analysis are presented in this section. Where applicable, the field test results are compared with theoretical analysis results.

In general, all strains exhibited an elastic response (i.e., strains from all gages 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 with concrete. Also, with minor variation, composite action was found to be present at all sections of the exterior beam (i.e., the neutral axis location on the exterior beam was found to be close to the top flange in the positive moment region).

As shown in Fig. 10a, the measured strains indicate good lateral symmetry that corresponds 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).

From the analysis discussed earlier, it was determined that an approximate stiffness increase of 1.2% per layer was obtained for the middle two beams. By comparing strains in each test it was observed that the follow-up test did produce fairly consistent strains with those measured during the initial test. This consistency in strain indicates 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 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 are virtually the same before and after the installation of the strengthening system. A typical comparison between the pre- and post-strengthening response is shown in Fig. 10b.

When a bonding technique is used for strengthening purpose, it is critical to have adequate bonding performance to transfer forces to the strengthening material. As was previously mentioned, additional gages were installed on the bottom flange of the strengthened beams (on the CFRP plates) in the center span to investigate the bond performance. To investigate the bond performance, a simple tool was developed based on strain compatibility relationship 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 strains measured during the post-installation load test.

From the comparison, it was found that the difference between the analytical and experimental strain was only 3.8% on average. The similarity between the analytical and experimental strain is shown in Fig. 10c. Considering the sensitivity of the neutral axis location that could change significantly with a small change in strain, this percentage error is considered to be relatively small; thus, it appears that, at least initially, that there was good bond between the beam and CFRP plates.

Summary of Field Investigation

Being relatively easy to design and showing good initial performance, it would appear that CFRP plates are a viable strengthening alternative for steel girder bridges. 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. Although relatively time-consuming to install, it is believed that with greater experience, construction crews could develop efficient techniques for installing CFRP plates.

CONCLUDING REMARKS

The following concluding remarks were developed from both the laboratory and field investigations:

- 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.
- CFRP strengthening systems are capable of returning the strength to even severely damaged steel girders. Only about 50% of the stiffness can typically be recovered.
- CFRP strengthening systems have negligible impact on changing bridge performance characteristics to service loads.
- In terms of strength, adding additional CFRP layers has a diminishing impact.
- Efficient installation of CFRP plates requires the development of standardized construction practices.
- Failure of the damaged/repaired specimens was ductile; however, the ductility was slightly less than that in the damaged (control) specimens. No sign of delamination or bond failure at the bond line was noted in all the beams tested. All failures were due to either crushing of concrete or rupture of the CFRP plate. Bond between steel and CFRP was not a problem for the system investigated.
- The general purpose of bonding the CFRP plates to a steel girder was to add additional strength to the member. With the strengthening system provided, the member would have increased moment capacity and a slightly lower neutral axis, which leads to reduction of the ultimate curvature and ductility of the section. It should be noted that the strengthening system used in this investigation is of value only at the ultimate limit state;

- since a small amount of material (i.e., CFRP plate) has been added to the beams, there are very minimal changes in behavior or stress at service level loads.
- Some of the areas needing additional work are:
 - A methodology for selecting the best adhesive for steel applications in both short and long term application is needed.
 - The effect of moisture, temperature, and sustained and fatigue loads need to be studied both in the laboratory and in the field (i.e., natural weathering).

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TABLE 1. Description of laboratory test specimens.

Specimen Type	Number of specimens	f'_c (ksi)	E_{pl} (ksi)	CFRP applied to web	CFRP applied to bottom flange	Remarks
U	2	5.4	29	None	None	Control beam
D50	1	5.4	29	None	None	Control beam
D50R1E29	1	5.4	29	4 in. wide	None	Repair1
D50R2E29	1	4.5	29	2 in. wide	2 in. wide	Repair2
D75R1E29	1	4.5	29	4 in. wide	No	Repair1

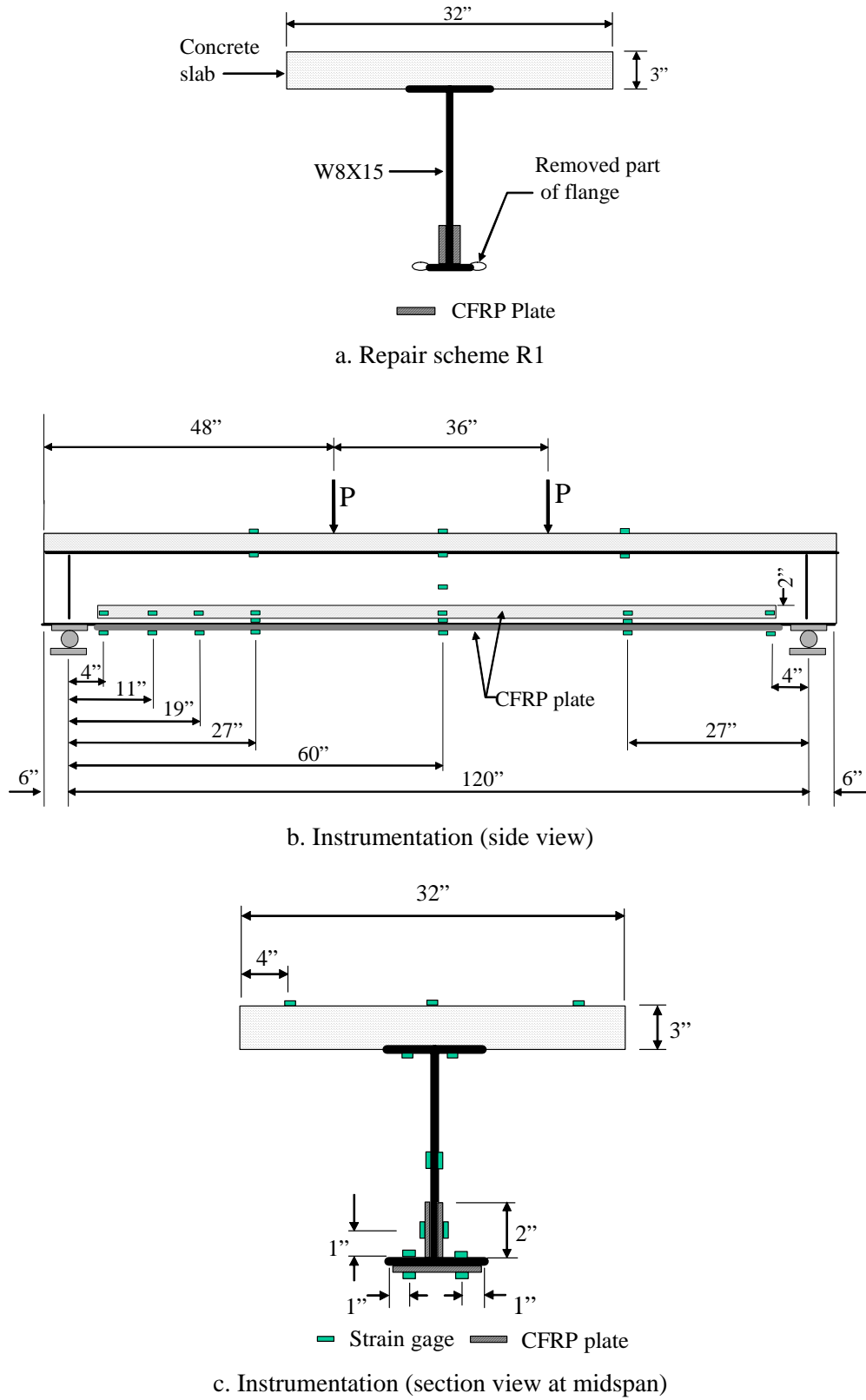
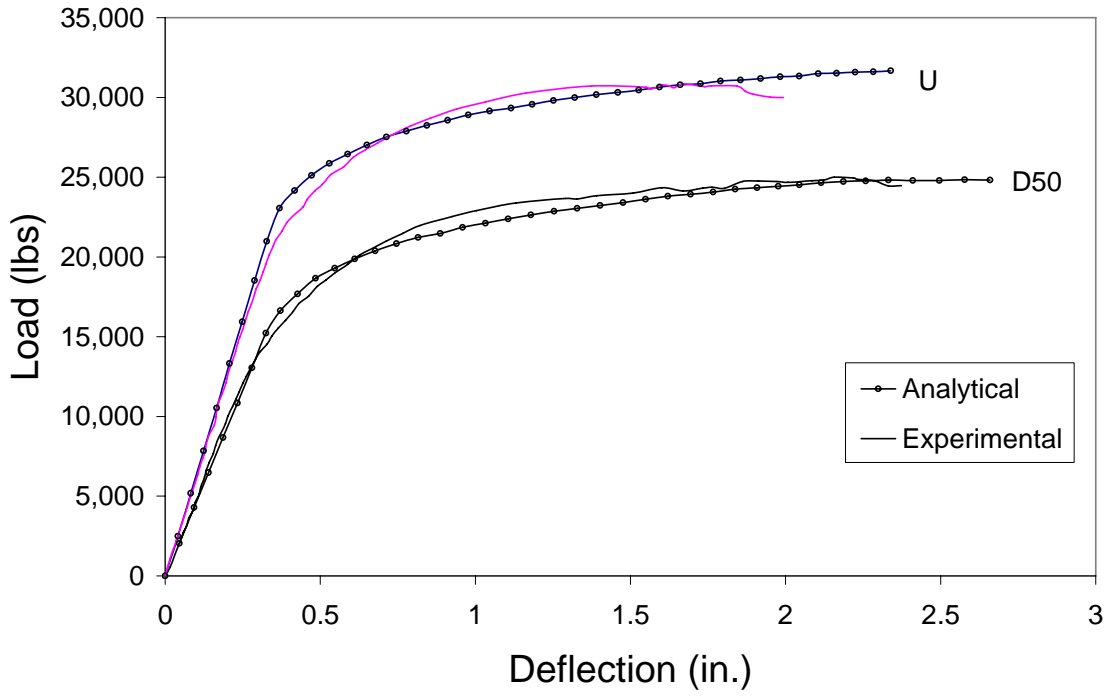
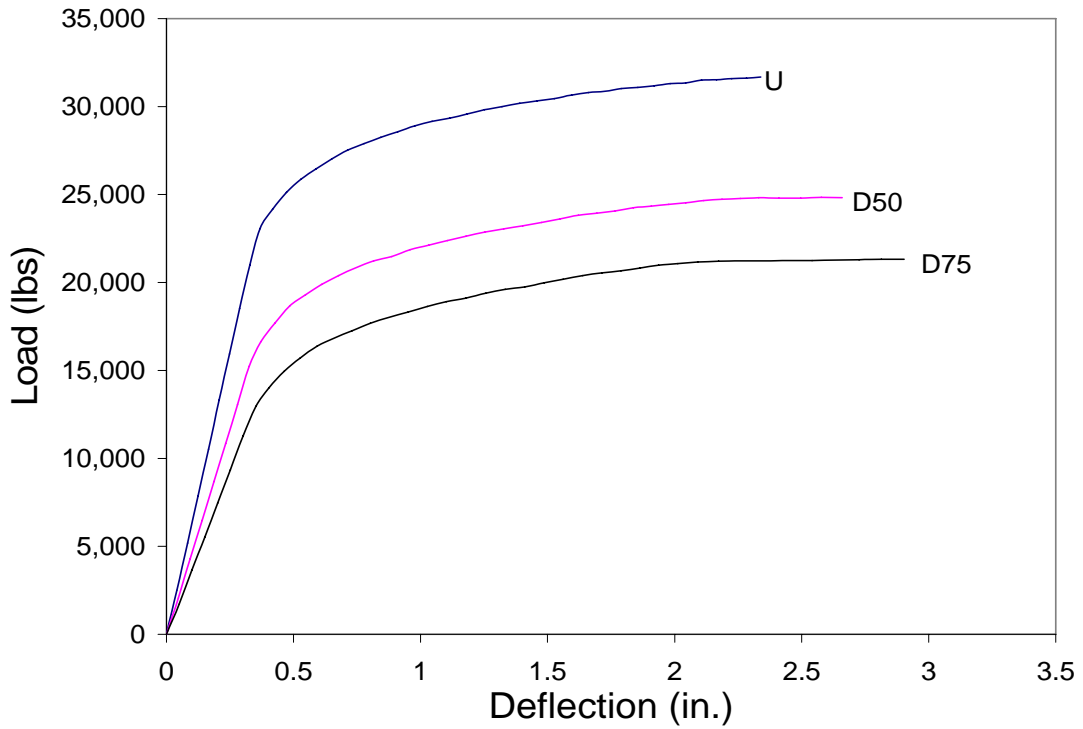


FIGURE 1. Repair scheme and instrumentation

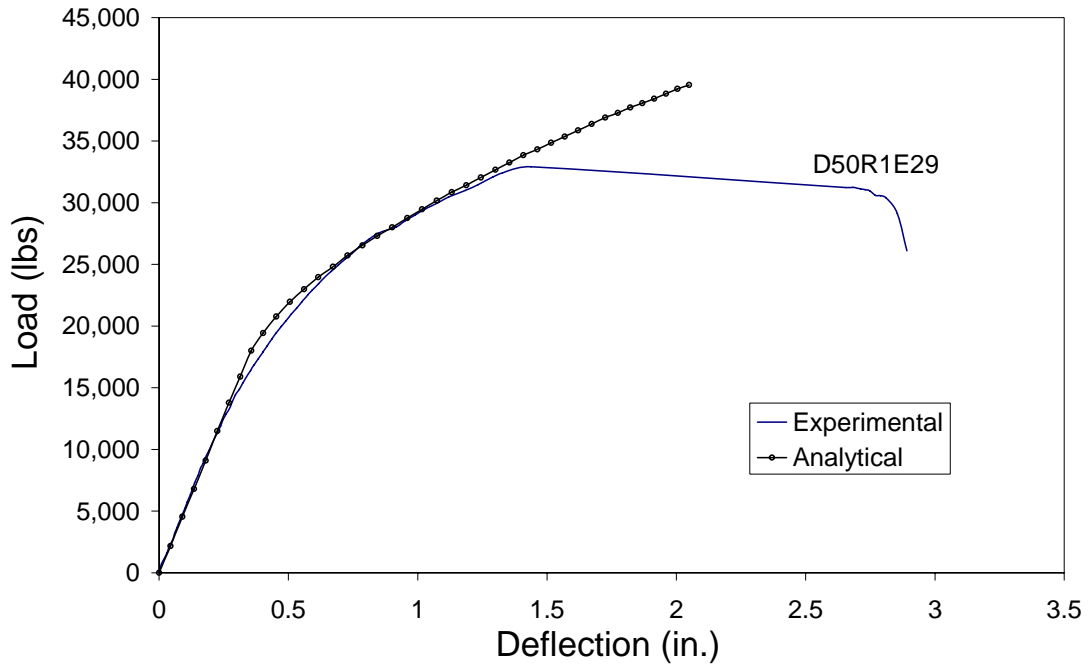


a. Comparison of analytical and experimental midspan deflections of Beam U and Beam D50

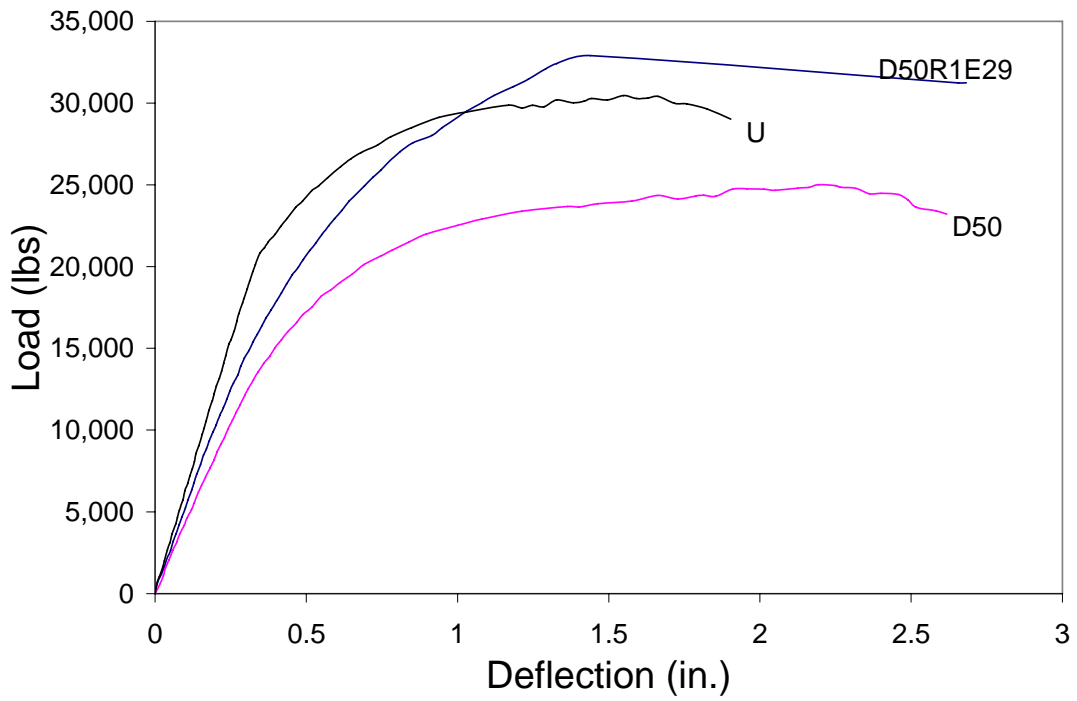


b. Analytical midspan deflections of Beam U, Beam D50, and Beam D75

FIGURE 2 Behavior of damaged and undamaged laboratory specimens.

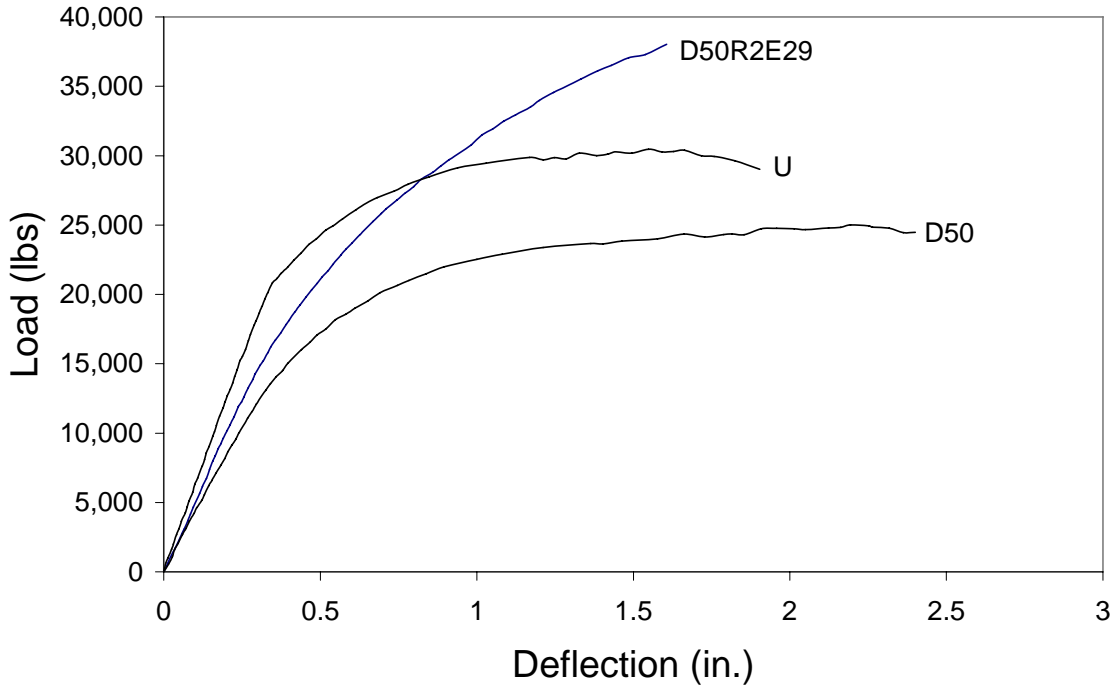


b. Deflections of Beam D50R1E29

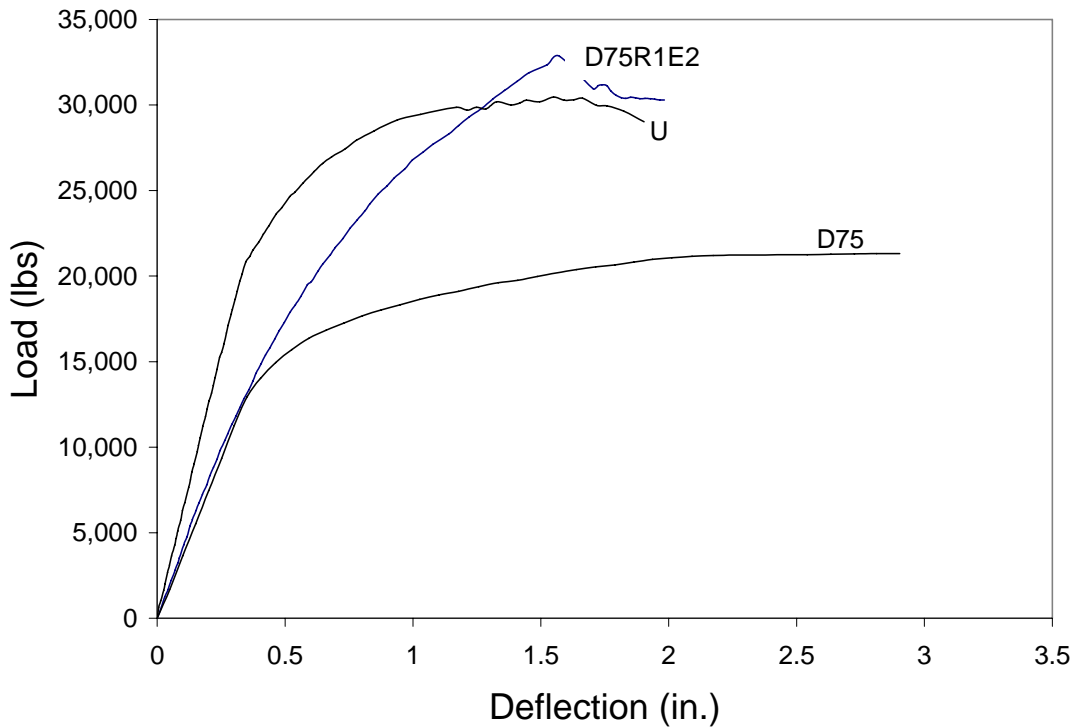


b. Comparison of measured experimental midspan deflections in Beam U, Beam D50, and Beam D50R1E29

FIGURE 3 Behavior of Beam D50R1E29



a. Comparison of measured experimental midspan deflections in Beam U, Beam D50, and Beam D50R2E29



b. Comparison of measured experimental midspan deflections in Beam U, Beam 75, and Beam D75R1E29.

FIGURE 4 Behavior of specimens repaired with two different repair schemes.



a. Side view – field bridge



b. Bottom view – field bridge

FIGURE 5 Photographs of Bridge 7838.5S092.

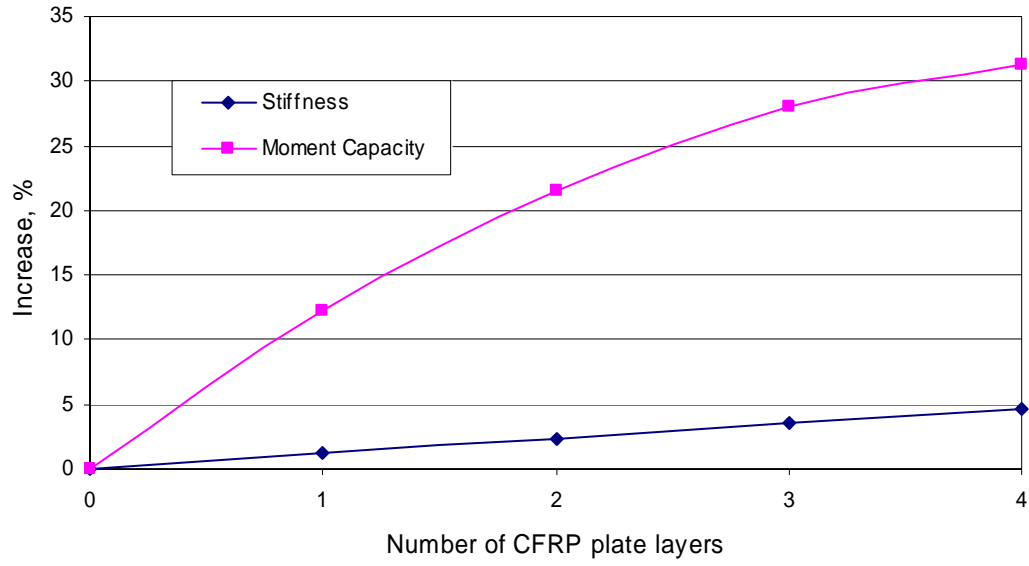


FIGURE 6 Theoretical change in stiffness and strength with increasing layers of CFRP.

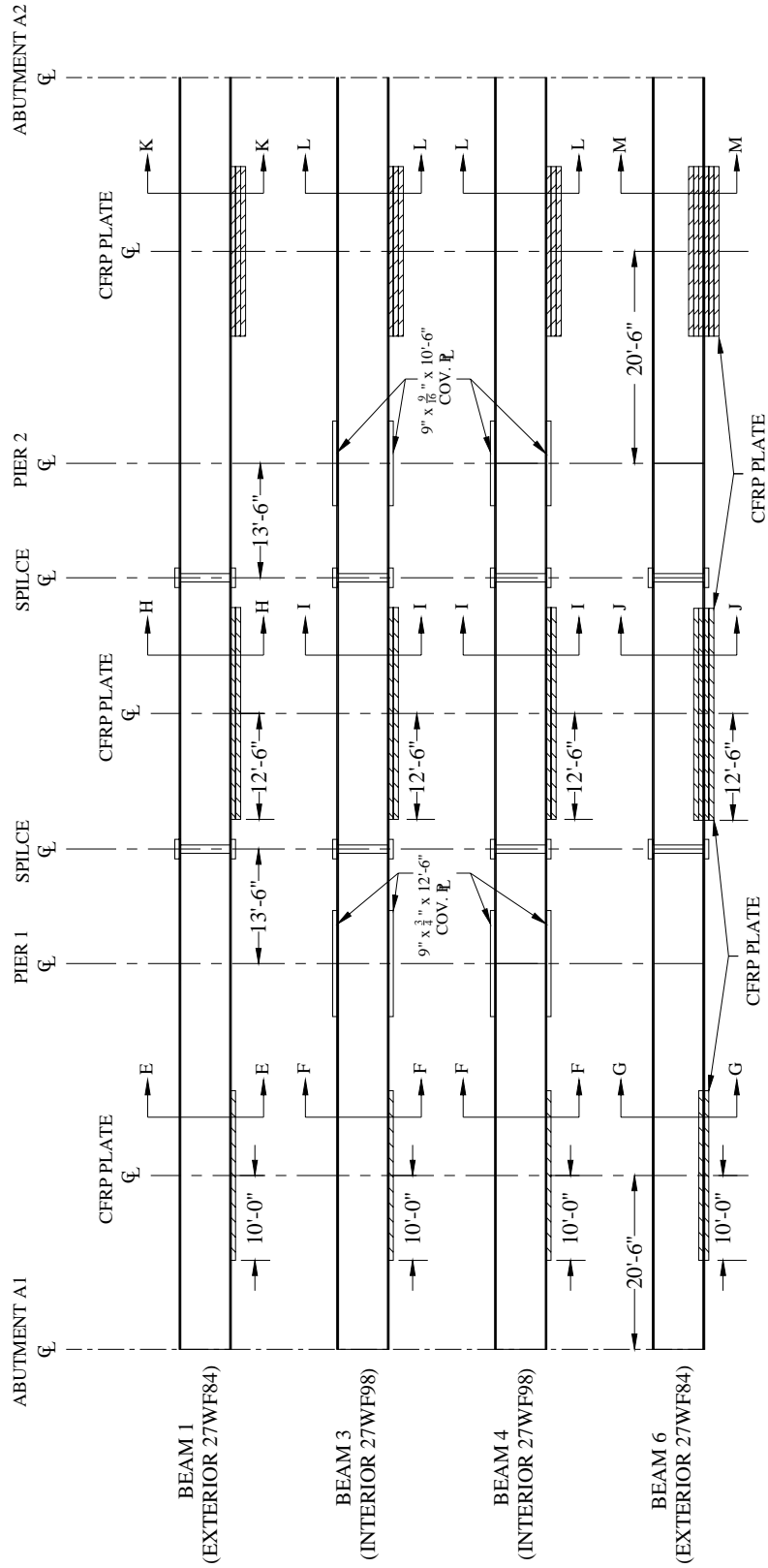


FIGURE 7 CFRP strengthening system layout.

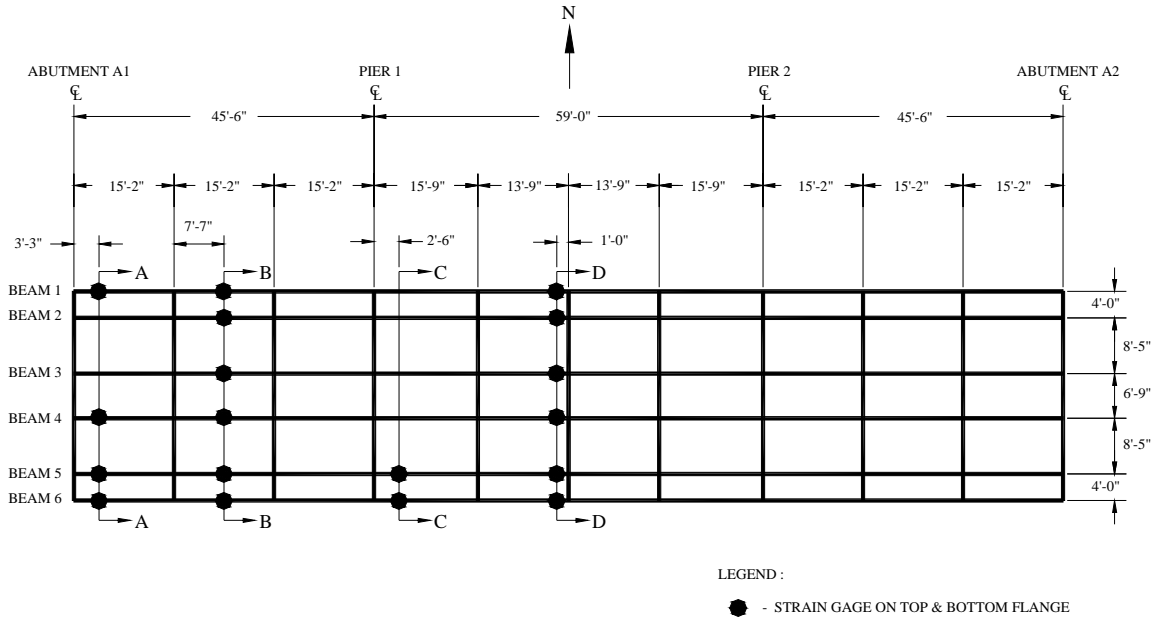


a. Beam 4 in the east end span

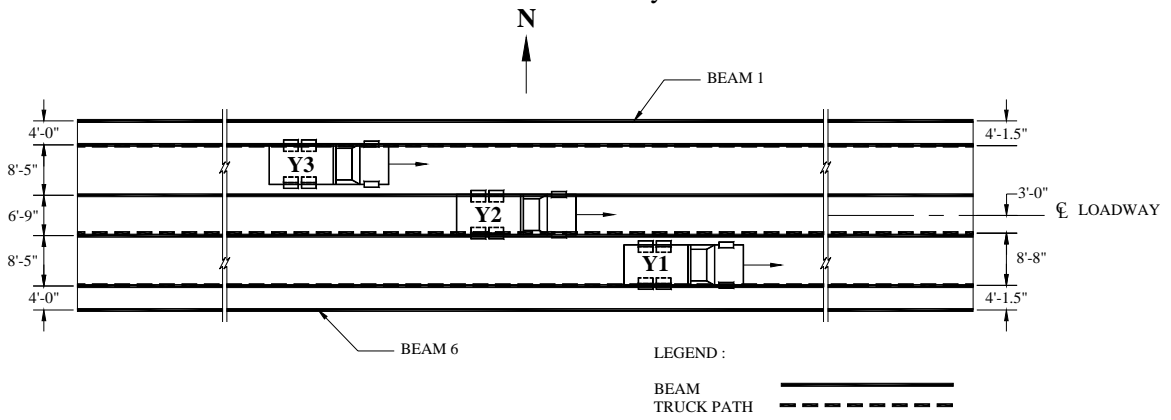


b. Top of bottom flange of Beam 6 in the west end span

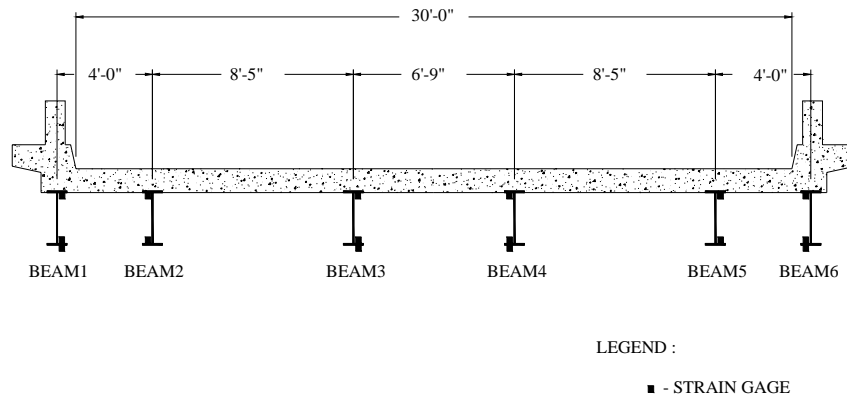
FIGURE 8 Photographs of installed CFRP strengthening system.



a. Instrumentation layout

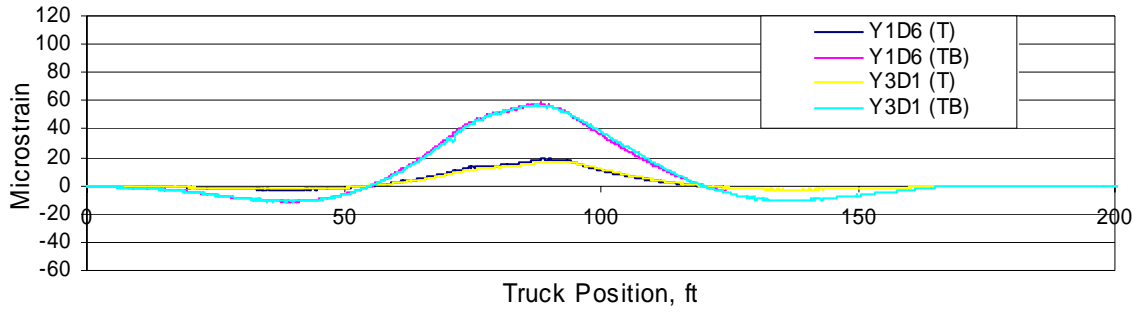


b. Truck positions

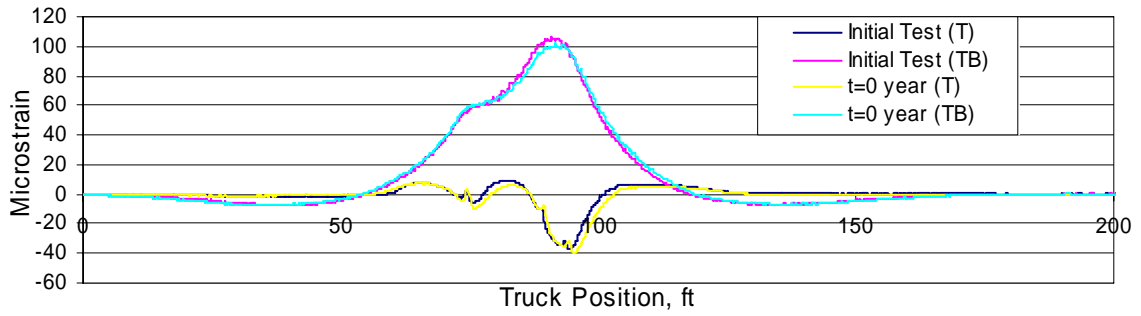


c. Cross-section (Section D)

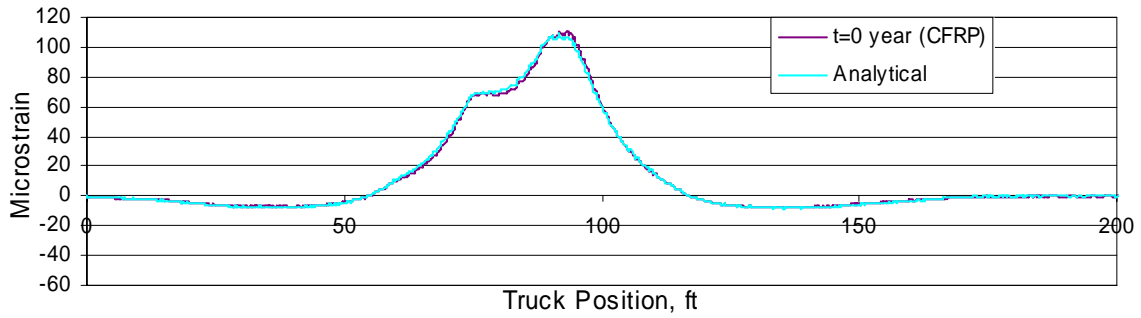
FIGURE 9 Instrumentation layout and truck path information.



a. Lateral symmetry: strain in Beam 6 (Path Y1) and Beam 1 (Path Y3)



b. Similarity in response before and after strengthening: strain in Beams 3



c. Strain in Beam 4 CFRP plate

FIGURE 10 Behavior of field bridge.