

Evaluation of Steel Girder Bridges Strengthened with FRP

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ABSTRACT

Many state, county, and local agencies are faced with an ever-deteriorating bridge infrastructure composed in large part of relatively short to medium span bridges. In many cases, these older structures 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 loads or loss of capacity due to deterioration. The bridges are usually not critical enough to warrant replacement so a structurally efficient, but cost-effective, means of strengthening needs to be employed. In the past, the use of bolted steel cover plates was a common retrofit option for strengthening such bridges. However, the time and labor involved to attach such a retrofit can be prohibitive.

This paper documents two projects funded through the Federal Highway Administration's Innovative Bridge Research and Construction (IBRC) program. The IBRC program was developed to assist bridge owners in applying emerging technologies in bridge engineering. In these projects, the Iowa Department of Transportation employed techniques for strengthening steel girder bridges using carbon fiber reinforced polymers (CFRP). The primary objective of these projects was to investigate the effectiveness of CFRP composite materials to strengthen existing, structurally deficient steel girder bridges. Two bridges were strengthened using CFRP composite materials in an effort to improve the live load carrying capacity of the bridges. In one case, a bridge was strengthened using CFRP post-tensioning bars. In the other case, a bridge was strengthened by installing CFRP plates to the bottom flange of girders. The research program consisted of several tasks with the main emphasis on the installation of the strengthening system and associated field testing and performance evaluation of the strengthening system.

INTRODUCTION

Based on data from the Federal Highway Administration (FHWA), approximately 30 percent of the nation's 600,000 bridges are in need of repair or replacement due to structural deficiencies or functional obsolescence. As these bridges continue to deteriorate, the problems have become further compounded by increases in legal load limits, limited budgets, and substantial cost associated with proper maintenance, rehabilitation, or replacement. These bridges, although in need of repair or strengthening, are usually not critical enough to warrant replacement so structurally efficient, but cost-effective, means of strengthening are needed. In the past, the use of bolted steel cover plates was a common option for strengthening such bridges. However, the time and labor involved to attach such a retrofit can sometimes be prohibitive.

Among various strengthening methods, the use of carbon fiber reinforced polymers (CFRP) composite materials is very appealing in that they are highly resistant to corrosion, have a low weight, and have a high tensile strength. In the last decade, the use of fiber-reinforced polymers (FRP) has emerged as a promising technology in structural engineering [1]. With this nationwide recognition, two projects, funded through the Federal Highway Administration's Innovative Bridge Research and Construction (IBRC) program, were initiated to investigate the effectiveness of using CFRP composite materials to strengthen existing steel girder bridges. Two bridges were strengthened using CFRP composite materials in an effort to improve the live load carrying capacity of the bridges. In one case, a bridge (Bridge 1) was strengthened using CFRP bars that had been post-tensioned in the positive moment regions. In the other case, a bridge (Bridge 2) was strengthened by installing CFRP plates to the bottom flange of girders in the positive moment region. The research program consisted of several tasks with the main emphasis on the installation of the strengthening system and associated field testing and performance evaluation.

STRENGTHENING WITH CFRP POST-TENSIONING BARS

Description of Bridge 1

Bridge 1 (number 3903.OS 141) which was strengthened with post-tensioning (P-T) CFRP bars is a 210 ft x 26 ft, three-span continuous, rolled shape steel girder bridge constructed in 1956 (shown in Fig. 1). It is located in southwest-central Iowa approximately 1.6 miles west of Bayard, Iowa carrying State Highway IA 141 over Willow creek. The bridge consists of two 64 ft end spans and a 82-ft center span with four beams spaced at 8'-3" on center. The bridge beams are spliced at locations 20 ft from the two piers in all spans (i.e., four splices per beam line). The bridge deck is a nominal 7-in. thick cast-in-place, reinforced concrete slab that was overlaid with dense low-slump portland cement concrete (PCC) in 1987. The current average deck thickness is approximately 10 in., including 3 in. of wearing surface with 2 1/2 in. of crown. The bridge deck is supported by two WF 30x116 exterior and two WF 33 1/4x141 interior I-beams.

Strengthening System

The P-T based strengthening system utilized was developed based on the strengthening recommendations of Klaiber et al [2] and material performance data provided by the manufacturer. The design of the CFRP P-T strengthening system was completed for the HS-20 load [3] utilizing the Allowable Stress Design (ASD) approach. Based on analysis completed by the bridge owner, it was found that the positive moment region of the exterior beams in both the end and center spans were overstressed.

The CFRP bars used in this project are 3/8 in. in diameter and have a high tensile strength (300 ksi), a moderate modulus of elasticity (20,000 ksi), low creep properties, and a high resistance to corrosion. For connection to the other P-T system components, the CFRP bars are embedded into steel tube anchors that have threaded ends. The CFRP bars, via the steel tubes, were connected to the steel beams with 5 in. x 5 in. x 3/4 in. stiffened angles 7 in. in length. Each of these stiffened angle assemblies, which are connected to the web of the steel beams with two 1 in. diameter A325 high-strength bolts, connects four CFRP bars to the web of the beam (two on each side) near the top surface of the bottom flange. The installation of the P-T system was completed in the positive moment region of the exterior girders in all three spans. The location of the P-T system and anchorage assembly details are shown in Fig. 2. A total of 12 kips was applied to each bar (four bars, 48 kips, per location). A list and brief description of the principal installation steps follow:

1. The location and layout of the anchorage assemblies was determined based upon the original design and field measurements.
2. 1 1/16 in. diameter holes were drilled through the web of each exterior beam for attaching the stiffened angle anchorage assemblies.
3. The surface of the web that was to be in contact with each stiffened angle anchorage assembly was cleaned without paint removal. All foreign materials, such as burrs and metal shavings due to drilling of holes were removed to ensure a satisfactory surface contact that is required for a bolted friction connection.
4. The anchorage assemblies were then bolted to the webs of the beams with two 1 in. diameter A325 high-strength bolts torqued in accordance with the manufacturer's recommendations.
5. Interference between the CFRP bars and the diaphragms was corrected by removal of a portion of the diaphragm/stiffener assembly.
6. The CFRP P-T bars were placed in position between anchorage assemblies on both sides of the web. Care was taken during the erection not to damage any of the bars by scratching or excessive sagging.
7. A nominal force of 12 kips was applied to all bars (four bars, 48 kips, per location) with a hollow-core hydraulic jack in a symmetrical manner following the sequence of steps listed below, where each event defines a specific step in the P-T process:
 - a. A nominal force of 6 kips was applied to the bottom and then to the top bar on the south side of the south exterior girder (Beam 4) in the west end span (Events 1-4).
 - b. A nominal force of 6 kips was applied to the bottom and then the top bar on the north side of the south exterior girder (Beam 4) in the west end span (Events 5-8).
 - c. Steps 1 and 2 were repeated in reverse order to increase the nominal force of 6 kips in each bar to the intended force of 12 kips (Events 9-16), thus completed the P-T at one location.
 - d. The jacking equipment was then moved to the north exterior girder (Beam 1) and Steps a through c were repeated (Events 17-32) for the four bars at that location.
 - e. Steps a through d were then repeated in the center span (Events 33-64) and in the east end span (Events 65-96) to complete the P-T.

The photographs of the installation, application of P-T force and completed installation are shown in Fig. 3.

In general, the handling and installation of the CFRP P-T system was relatively simple and not labor intensive requiring a three-man crew in just over one day to install. It is recommended, however, that a visual inspection be made at each bar grip after the force is applied to the system to make sure that no slippage has occurred between the bars and the grips.

Field Investigation and Analysis

Load Testing

The bridge was tested four times: before, shortly following the installation of the strengthening system, and after one and two years of service. The location of the instrumentation was selected so that the live load response of the bridge could be determined to provide an overall understanding of the global behavior. A total of thirty-six strain gages were installed on the bridge with thirty-two gages on the top and bottom flanges of the beams and four gages on the guardrails. Due to the structural symmetry of the bridge, only one-half of the bridge was instrumented.

After installation of the instrumentation, a loaded 3-axle dump truck was driven, at crawl speed, across the bridge with strain data collected continuously as the truck crossed the bridge. The initial test was conducted to establish a benchmark response of the bridge, while the follow-up tests were completed to assess changes resulting from the addition of the P-T system and time. Data were collected for four different load paths with two test runs for each path. For Y1 path, heading east, the driver side wheel was placed 3 ft north of the centerline of the bridge. Y3 path placed the passenger side wheel on the same path as Y1 but heading west. For path Y2, heading east in the north lane, and path Y4, heading west in the south lane, the driver side wheels were placed 2 ft from the north and south edge of the roadway, respectively. The location of the strain gages and the load paths used during the load testing are shown in Fig. 4.

Each subsequent load test produced fairly consistent strain readings with those established during the initial test (see Fig. 5 for typical examples). This consistency in strain is informative in that it indicates that the P-T system did not significantly alter the behavior of the bridge over the two years of service, as would be expected. 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 P-T system had negligible impact on changing the stiffness of the bridge. The data also indicate that the live load distribution characteristics are virtually the same before and after the installation of the P-T system.

Analytical Modeling

In order to better understand how the bridge behaves during the application of the P-T strengthening system, the same application sequence was followed with a concentric axial force and an applied couple applied to an analytical grillage model. The resulting internal moments of the P-T strengthening system are shown in Fig. 6a. Note that for the exterior beams, the largest moment was generated at the anchorage locations rather than at midspan region (i.e., maximum positive moment region). A different pattern, however, was observed on the interior beams; the effect of the P-T strengthening system is the highest near the maximum positive moment location. It was also observed analytically that larger moments were generated in the end spans than the center span. From the grillage analysis, the strain induced by the concentric axial force was found to be negligible (i.e., in the range of approximately 0 to 3 microstrain); therefore, a reduction of bottom flange stress due to this axial force component was not included in the analysis discussed below.

To illustrate how the P-T strengthening system improves the live load carrying capacity of the subject bridge, an individual beam analysis was performed for both an exterior and interior girder. The moments induced by the P-T force, dead load, and live load at each location of the bridge were obtained first and then combined to illustrate the overall impact of the P-T strengthening system. For this analysis, the live load moment was determined utilizing an HS-20 truck placed so that the maximum positive moment could be generated.

The P-T force generates moments opposite in sign to those induced by the dead load at both the maximum positive moment region (midspan) and maximum negative moment region (pier). Three point loads were used to represent the HS-20 truck [3]. The codified [3] lateral distribution factors for the exterior and the interior girders were applied to the total live load induced moments. The experimental lateral distribution factors obtained during the application of the P-T force were used in computing the P-T induced moment [4]. In order to investigate an overall effect of the P-T strengthening system on the bridge, the P-T induced moment, dead load moment and live load moment were added together as is illustrated in Fig. 6b. Note that the moment diagrams presented are not to scale.

As expected, the exterior girder experienced a larger reduction in total moment than the interior girder in both the end and the center spans. In general, the end spans showed larger moment reductions than the center span. The P-T strengthening system reduced the total moment by 5.3% on the exterior beam and 4.6% on the interior beam in the end span. Similarly, 3.3% and 2.7% of the total moment was reduced on the exterior and the interior beam in the center span, respectively [4].

Change in Post-tensioning Force over Time

The P-T force was removed from the bridge after two years of service so that any losses in P-T force could be determined. In comparison with the forces that were originally applied [4], it was found that the largest loss of 3.7 kips occurred on Beam 1 in the center span while the average loss was 2.6 kips (5.4% loss). Although some loss of P-T force occurred in most bars, it should be noted that these time-associated losses were accounted for in the design phase.

STRENGTHENING WITH CFRP PLATES

Description of Bridge 2

Bridge 2 (number 7838.5S092) strengthened with CFRP plates is a 150 ft x 30 ft three-span continuous I-beam bridge in Pottawattamie County, Iowa, on State Highway 92 (shown in Fig. 7). Originally the bridge was a non-

composite structure. In 1967, the bridge was widened with the addition of exterior girders that were made composite with the deck. The bridge has a total length of 150 ft consisting of two end spans of 45.5 ft and a center span of 59 ft and is supported by six beams: two W27x84 exterior I-beams installed in 1967, two W27 x91, and two W27x98 interior I-beams. The roadway width is 30 ft allowing two traffic lanes and a shoulder on each side.

Strengthening system

Previous research by Al-Saidi [5] at Iowa State University Bridge Engineering Center established the effectiveness of CFRP plates for improving the strength of steel composite beams. With this foundation study available, the next step was taken to implement this strengthening method on an existing steel girder bridge. The material used for strengthening is a pultruded carbon fiber reinforced polymer consisting of continuous unidirectional carbon fiber in an epoxy matrix, specially designed for flexural strengthening. The CFRP plates were selected due to their outstanding mechanical characteristics and relative ease of application. The CFRP plates used in this project have a high tensile strength (300 ksi), a moderate modulus of elasticity (20,000 ksi) with the ultimate strain of 1.5%, and a high resistance to corrosion. The strengthening system consists of preparation of the bonding surface and installing the CFRP plates to the beam surface with a high strength epoxy adhesive for the transfer of force to the high strength CFRP plates.

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. The following paragraphs describe 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, 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 cross sectional area of the corresponding element, and distance from the corresponding element to the top surface of the top flange.

The change in moment capacity with varying numbers of CFRP plate bonded to the bottom of the bottom flange of the subject bridge was investigated. From this analysis, it was found 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 [6].

The stiffness increase due to the addition of CFRP plates to the bottom of the bottom flange was also investigated. 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.
- Compare the behavior of different strengthening schemes.

To this end, the following modifications were made with the final layout summarized in Fig. 8:

- 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 Southernmost 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 CFRP Plates

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. 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. 9.

In general, 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 Investigation and Evaluation

Load Testing

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. 10a, 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 selected girders. For convenience, load paths are referred to as Y1, Y2 or Y3 as shown in Fig. 10b. All tests were conducted from west to east.

Follow-up load tests were conducted approximately two months and one year after the installation of the strengthening system, respectively, to assess any change in performance resulting from the installation of the CFRP plates and time. The procedures used for the follow-up load tests were the same (e.g., same load paths and sequences, etc.) as what was used in the initial test except for the weight of the truck used and the number of strain gages used. In these tests, four additional gages were installed on the bottom flange (on the CFRP plates as shown in Fig. 10c) of the strengthened beams in the center span to investigate the bond performance.

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 [6]. 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. 11a, 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 tests 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. 11b.

Bond Performance

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 [6]. These simple analytical predictions were compared with the strains measured during the post-installation load tests.

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. 11c. 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 there was good bond between the beam and CFRP plates.

CONCLUSION

Bridge 1

Based upon the ease of installation of the strengthening system and the monitoring and testing conducted in the field, following concluding remarks were developed:

- The installation of the P-T system required no special equipment or training other than access equipment, an acetylene torch to remove a portion of several diaphragms, and a hydraulic jack. A three-man crew was able to install the system in just over one day.
- The addition of the P-T strengthening system had a negligible impact on the stiffness of the bridge. This resulted in the live load distribution characteristics remaining virtually the same before and after the installation of the P-T strengthening system.
- The P-T system generates strain opposite to those produced by gravity loads and thereby improves the overall live load carrying capacity of the bridge. Based upon an analysis performed utilizing the HS-20 truck, it was found that the P-T strengthening system reduced the dead and live load induced moments by approximately 3 to 5%, thus allowing the bridge to carry additional live load.
- The use of the P-T CFRP bars to strengthen the structurally deficient bridge described in this project was found to be a viable, effective, and practical solution. In cases with greater overstresses, larger bars with appropriate capacity will be required.

Bridge 2

Although relatively time-consuming to install, it is believed that with more experience, construction crews could develop efficient techniques for installing CFRP plates. Based upon the design process 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 notably 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.
- The use of the CFRP plates appears to be a viable strengthening alternative for steel girder bridges.

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LIST OF FIGURES

FIGURE 1 Photographs and framing plan of Bridge 1.

FIGURE 2 Location of the P-T system and anchorage assembly details on Bridge 1.

FIGURE 3 Photographs of installation, P-T force application and completed installation on Bridge 1.

FIGURE 4 Instrumentation and load paths used for testing on Bridge 1.

FIGURE 5 Bottom flange strain data in center span, Beam1 of Bridge 1.

FIGURE 6 Theoretical P-T induced internal moments and Effect of P-T on maximum moments in Bridge 1.

FIGURE 7 Photographs of Bridge 2.

FIGURE 8 CFRP strengthening system layout on Bridge 2 (side view).

FIGURE 9 Photographs of installed CFRP strengthening system on Bridge 2.

FIGURE 10 Instrumentation layout and truck paths used for testing on Bridge 2.

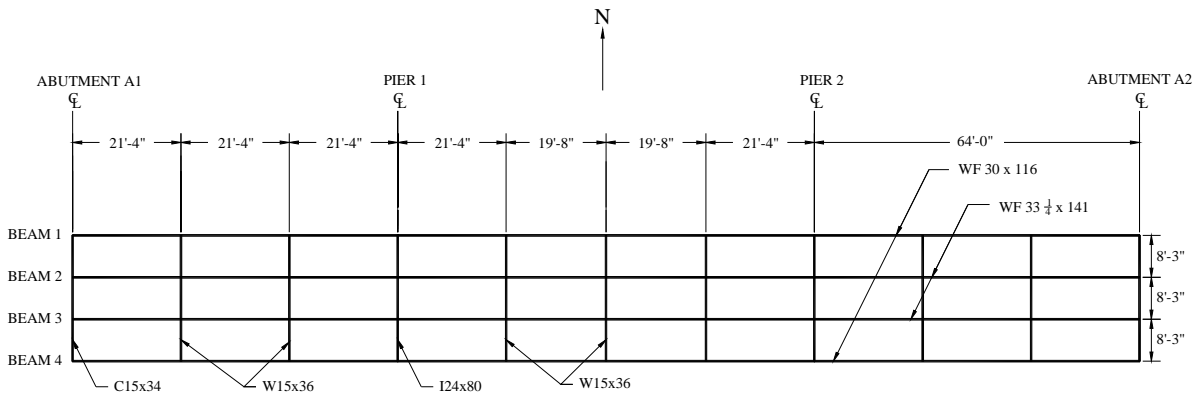
FIGURE 11 Behavior of Bridge 2.



(a) Side view



(b) Bottom view



(c) Framing plan

FIGURE 1 Photographs and framing plan of Bridge 1.

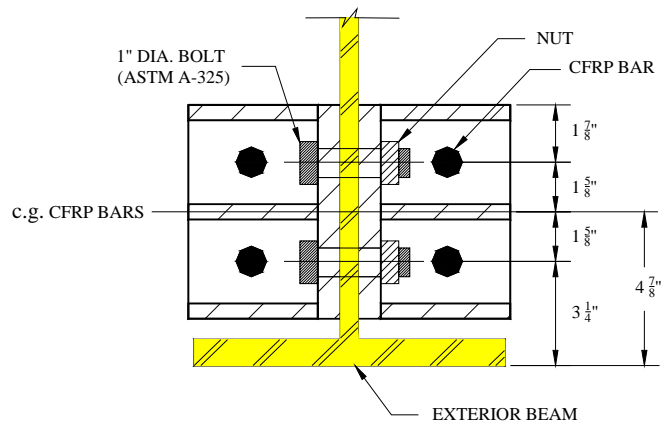
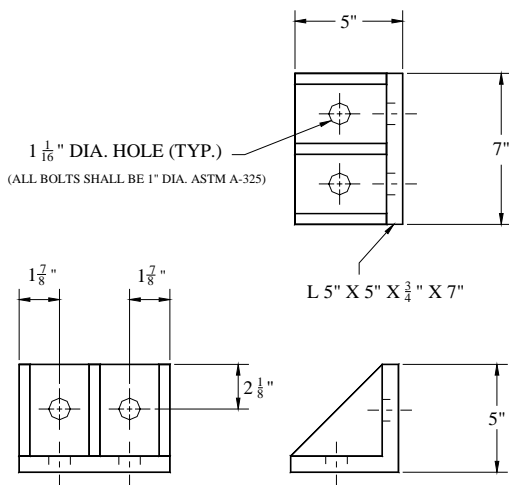
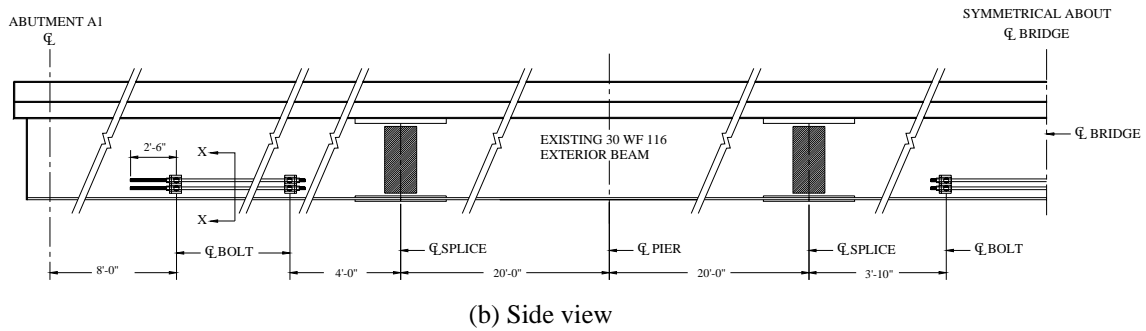
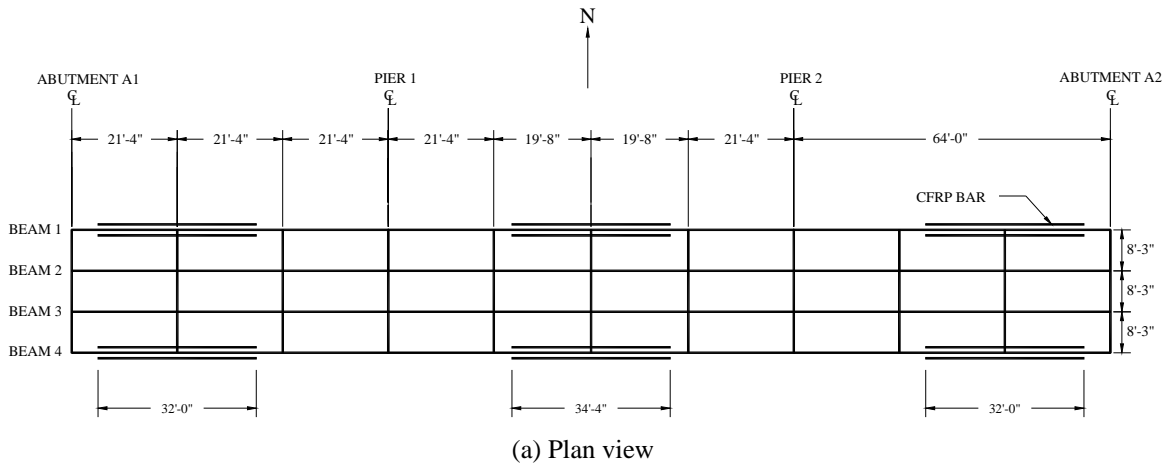


FIGURE 2 Location of the P-T system and anchorage assembly details on Bridge 1.



(a) Installation of anchorage assembly

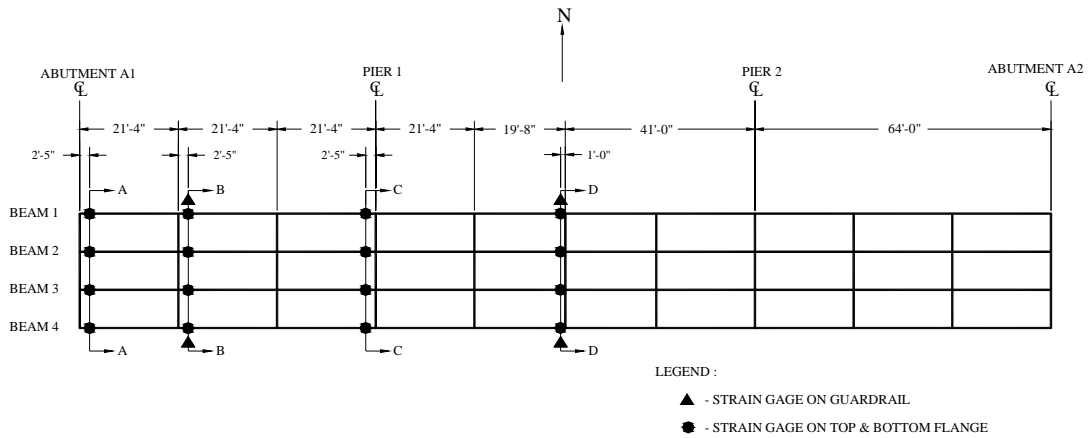


(b) Application of P-T force in the center span

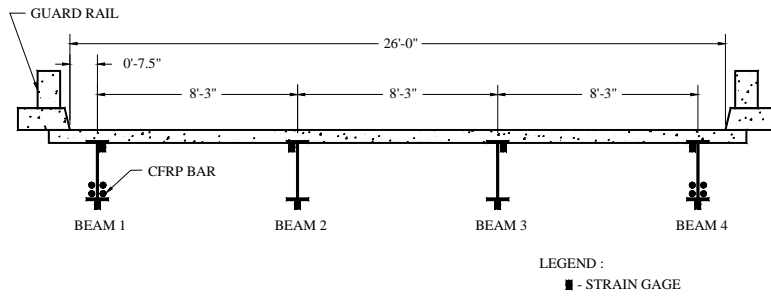


(c) Completed installation of exterior CFRP bars in west end span

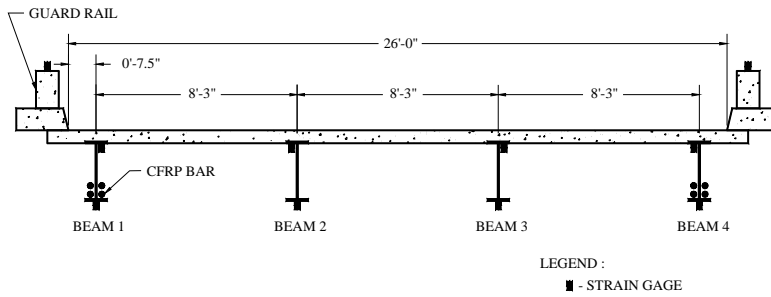
FIGURE 3 Photographs of installation, P-T force application and completed installation on Bridge 1.



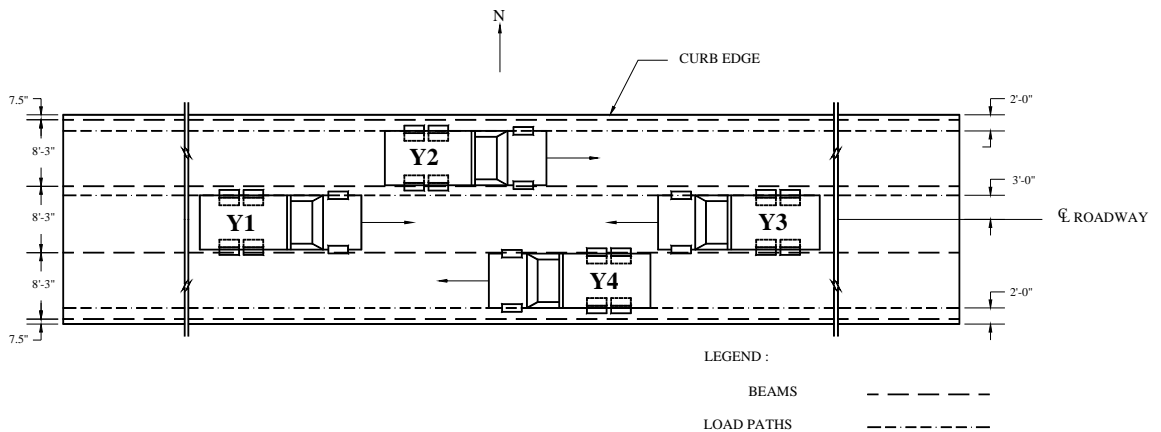
(a) Strain gage locations



(b) Sections A and C

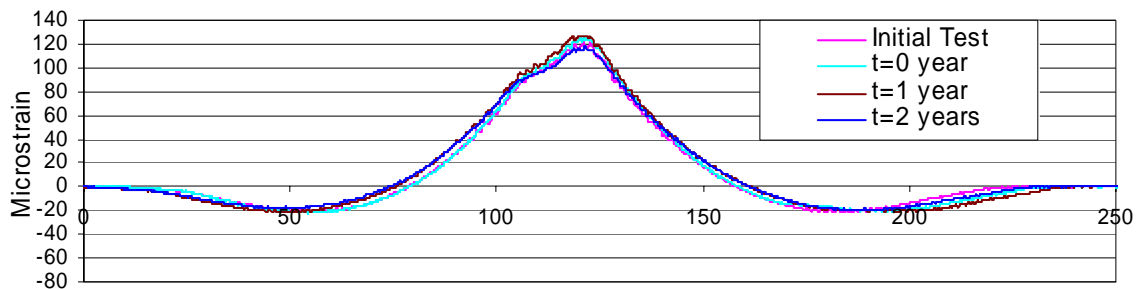


(c) Sections B and D

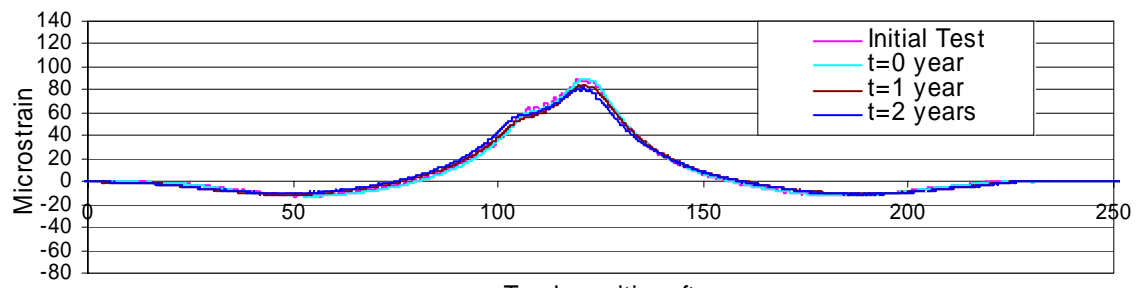


(b) Load paths used

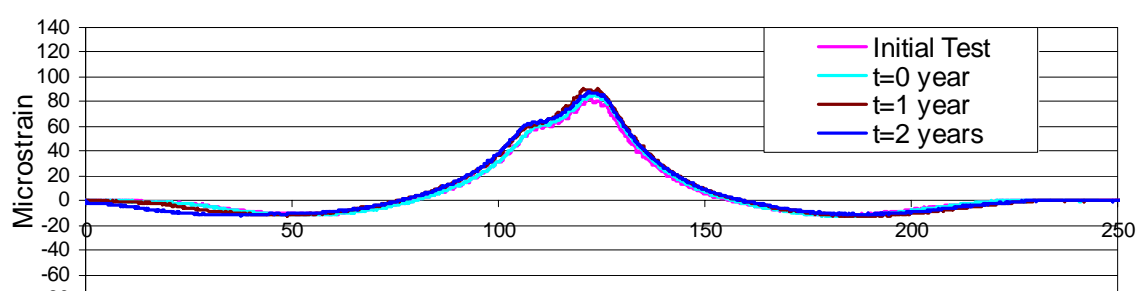
FIGURE 4 Instrumentation and load paths used for testing on Bridge 1.



(a) Beam 1 (Path Y2)

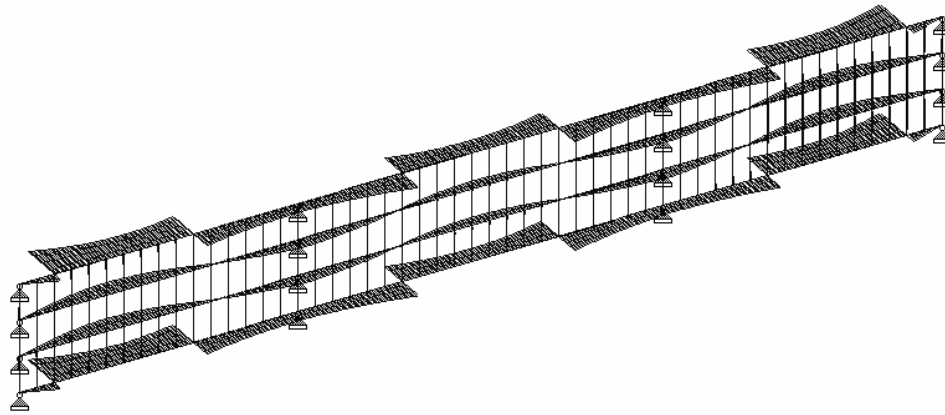


(b) Beam 2 (Path Y1)

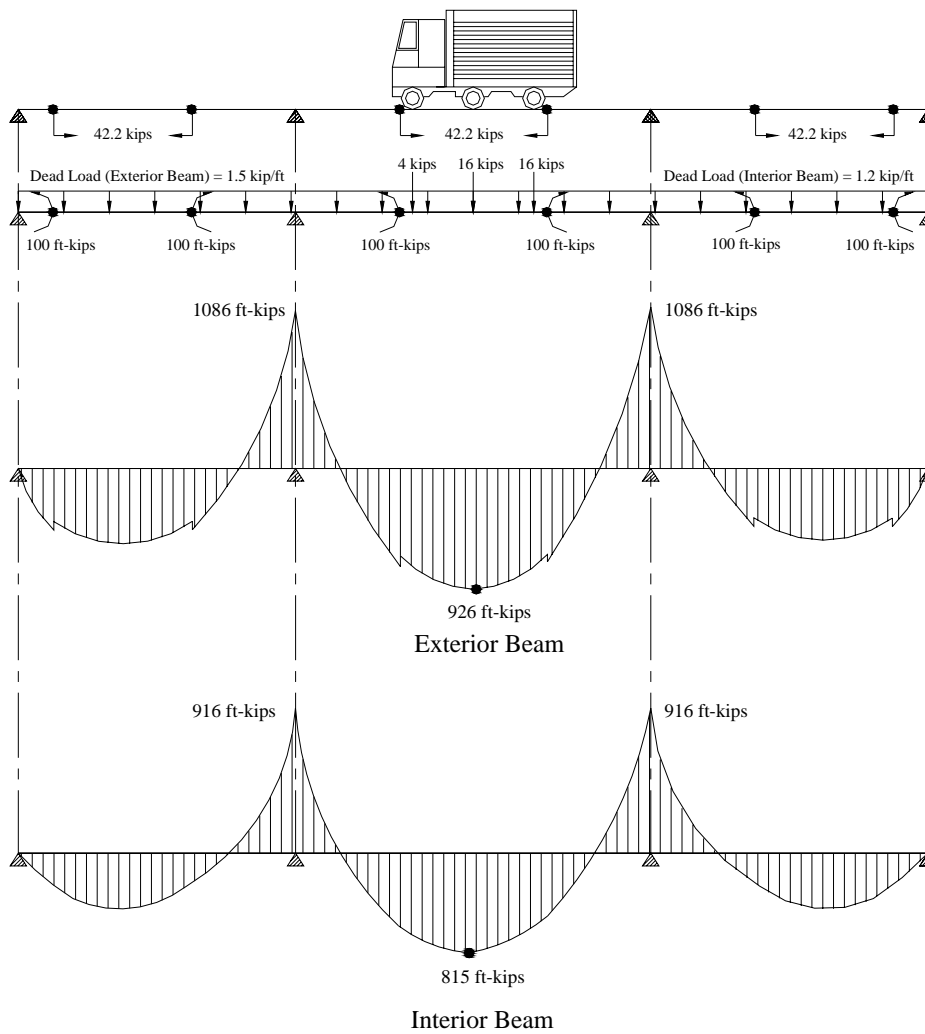


(c) Beam 3 (Path Y3)

FIGURE 5 Bottom flange strain data in center span, Beam1 of Bridge 1.



(a) Theoretical P-T induced internal moments.



(b) Effect of P-T on maximum moments in the center span.

FIGURE 6 Theoretical P-T induced internal moments and Effect of P-T on maximum moments in Bridge 1.



(a) Side view



(b) Bottom view

FIGURE 7 Photographs of Bridge 2.

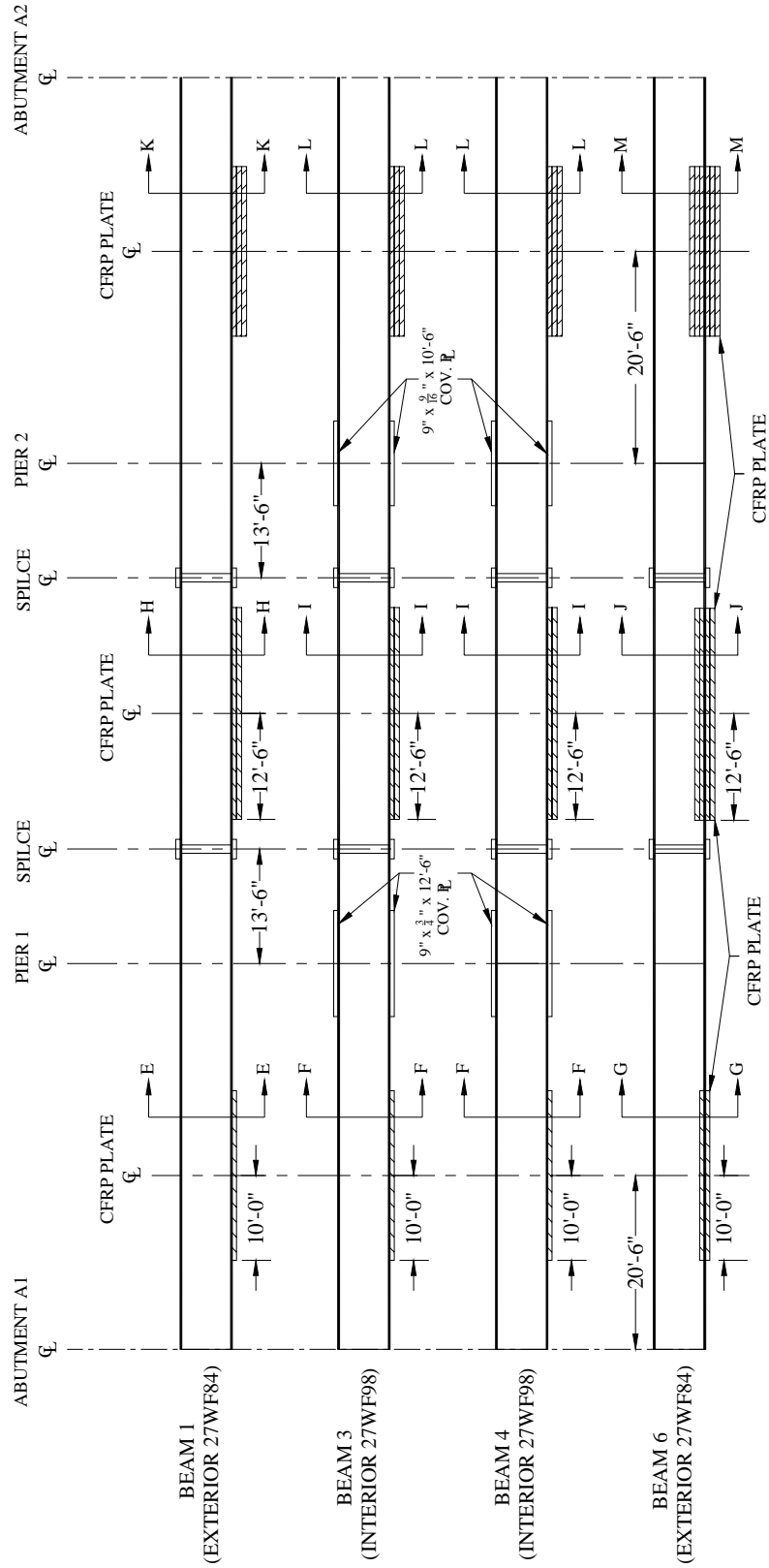
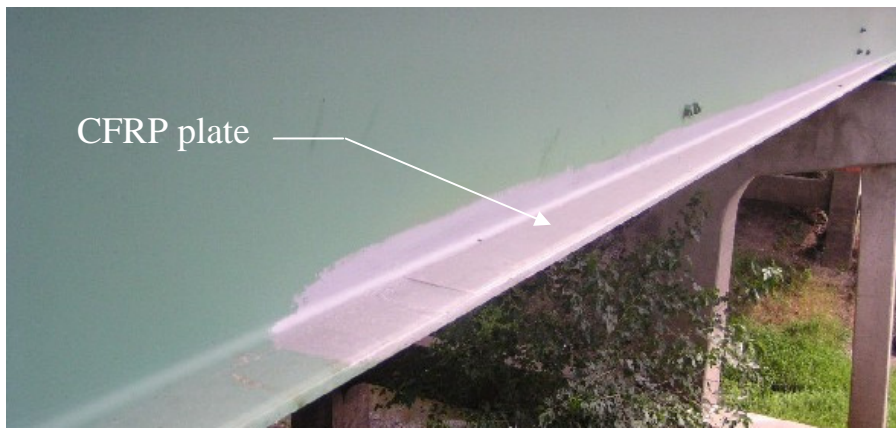


FIGURE 8 CFRP strengthening system layout on Bridge 2 (side view).



(a) Beam 4 in the east end span

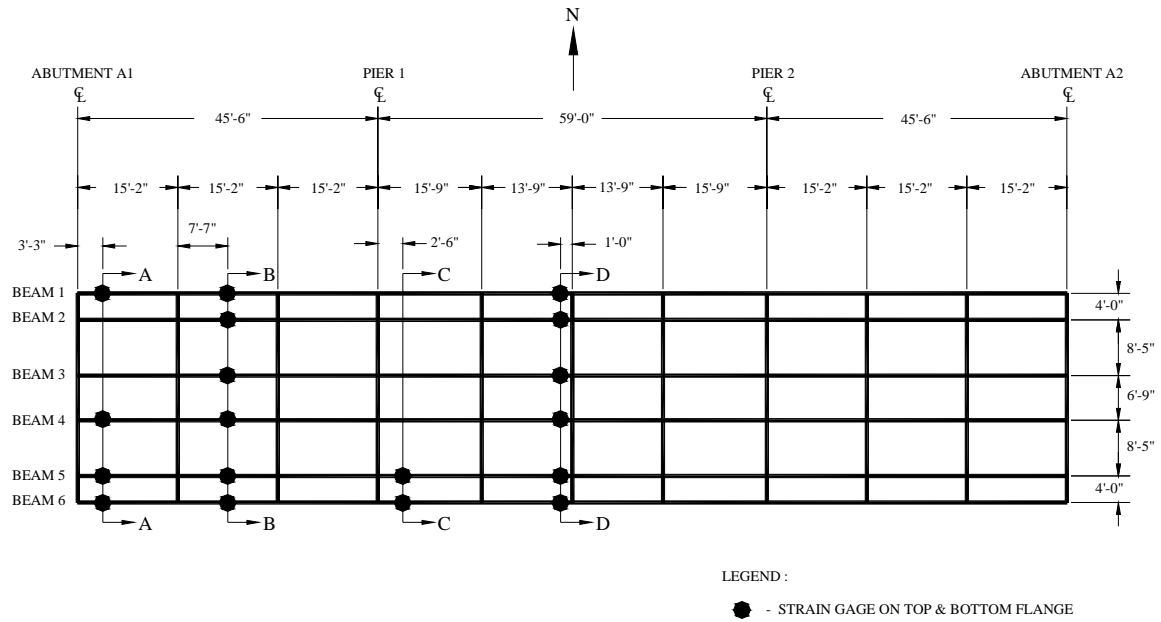


(b) Top of bottom flange of Beam 6 in the west end span

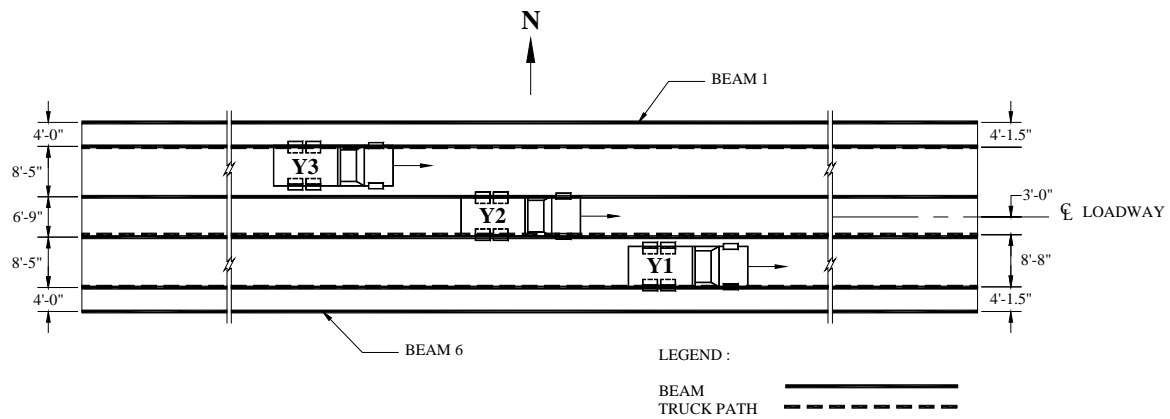


(c) Bottom of bottom flange of Beam 6 in the east end span

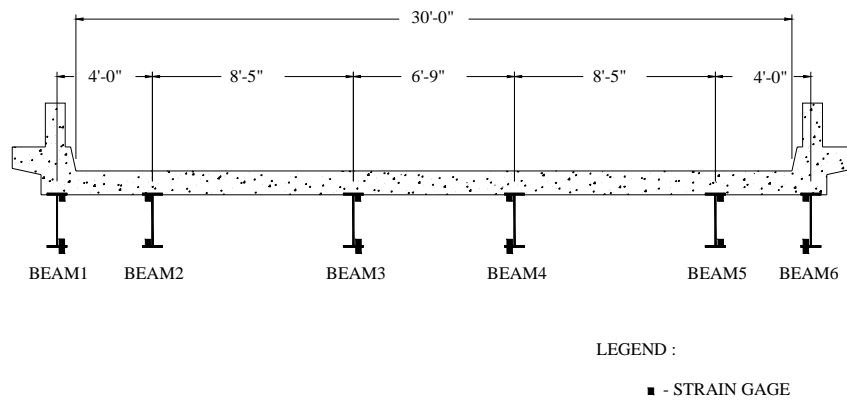
FIGURE 9 Photographs of installed CFRP strengthening system on Bridge 2.



(a) Instrumentation layout

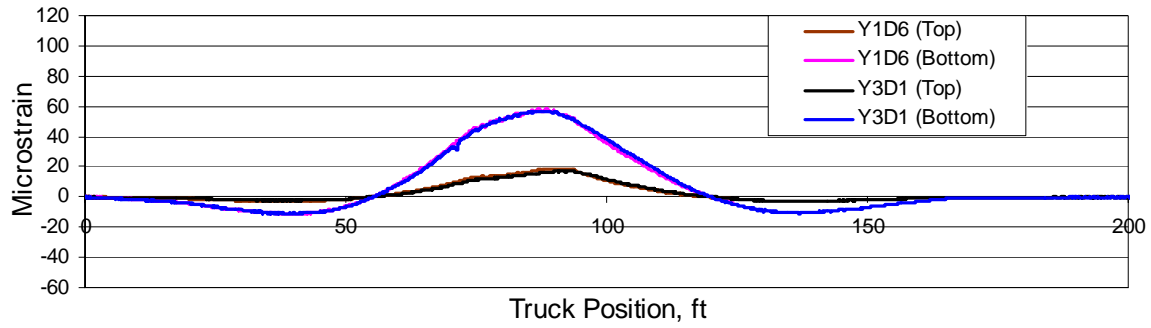


(b) Truck paths

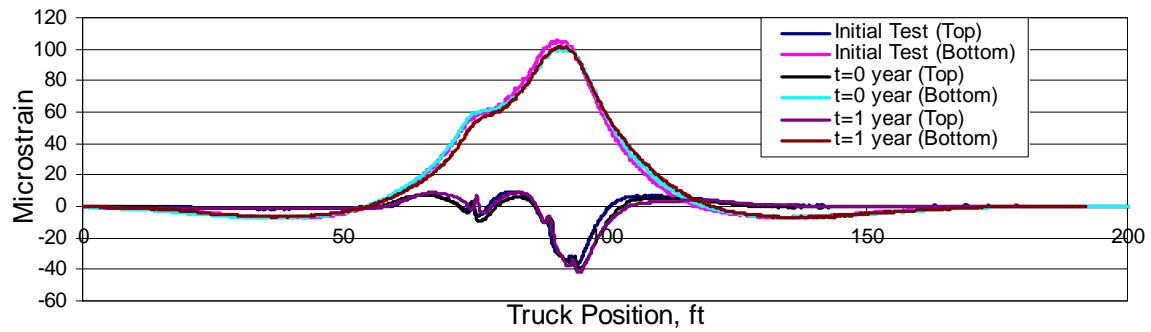


(c) Cross-section (Section D)

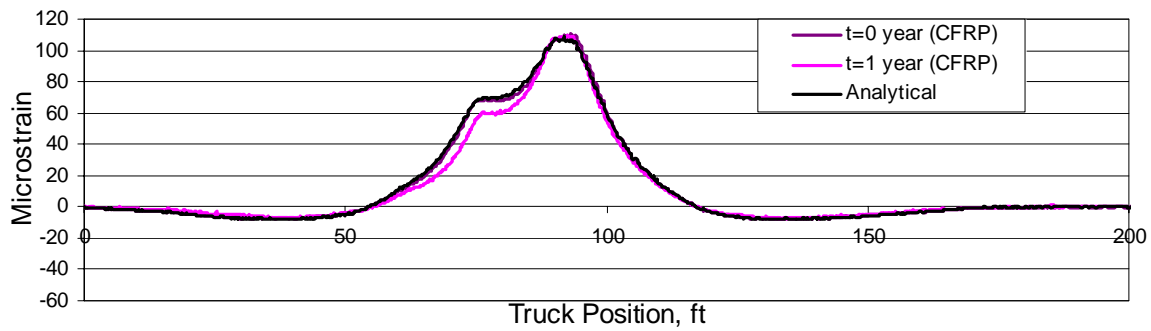
FIGURE 10 Instrumentation layout and truck paths used for testing on Bridge 2.



(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 (Path Y2).



(c) Strain in Beam 4 CFRP plate

FIGURE 11 Behavior of Bridge 2.