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Strengthening of Route 378 Bridge Over Wynantskill Creek In New York Using FRP Laminates

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ABSTRACT

This report describes application of Fiber-Reinforced Polymer (FRP) composite laminates to strengthen an aging reinforced-concrete T-beam bridge in South Troy, Rensselaer County, New York. Leakage at the end joints of this single-span structure led to substantial moisture and salt infiltration in the bridge superstructure. Presence of efflorescence was observed, and freeze-thaw cracking and concrete delamination at some locations on the beams were noted. Concerns about integrity of the steel reinforcing and overall safety of the bridge were raised. These concerns were heightened by the absence of any documents pertaining to the bridge design, such as rebar size, steel type, concrete strength, and design loads. Thus, a decision was made to strengthen the bridge using bonded FRP-laminates. Load tests were conducted before and after the laminates were installed to evaluate effectiveness of the strengthening system, and investigate its influence on structural behavior of the bridge. Results from these tests and those obtained using classical analysis are compared in the report. Based on these results, it was concluded that under service loads, the laminate system slightly reduced main steel rebar stresses and moderately improved transverse live load distribution to the bridge beams. Use of an FRP laminate system in this project demonstrated cost-effectiveness of such systems in strengthening applications, with the benefit of minimal to no interruption to traffic.

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I. INTRODUCTION

Transportation agencies are faced with a continuous challenge to keep bridges under their jurisdiction in a good operating condition despite limited resources. Bridge structures are deteriorating at an alarming rate and costs for repair and replacements are continuously rising [1]. Even when resources are available, extended time is often required for performing needed remedies, causing disruption to traffic, and inconvenience to the traveling public. Faced with these challenges, these agencies may find a solution in application of Fiber-Reinforced Polymer (FRP) composite materials for bridge rehabilitation. Although these materials have recently been introduced for civil-engineering applications, the coming years may see a significant increase in their use because of their desirable properties and easy installation. Bridge strengthening using these materials is generally less costly than replacement and is preferable to posting for lower loads. Use of FRP materials significantly shortens downtime for rehabilitation, which reduces inconvenience to the traveling public and economic loss to areas served [2]. Composite materials are also beneficial in improving bridge rating either directly through strengthening of deteriorated components or indirectly through replacing existing concrete decks with much lighter FRP decks [3].

A. LITERATURE REVIEW

Epoxy resins were occasionally used as early as the 1960s to bond steel plates to the tension zones of flexural concrete members of buildings and bridges [4]. However, because steel corrosion can lead to loss of bond and consequent member failure, focus has shifted to alternative materials. Fiber-reinforced polymer (FRP) sheets made from glass (GFRP), aramid (AFRP), and carbon (CFRP) fibers embedded in polymeric resins are now being substituted for steel plates. Besides their noncorrosive properties, composite materials also have a higher strength to weight ratio.

The study by Ritchie et al. [5] on behavior of concrete beams strengthened by bonding FRP- (glass, carbon, and aramid) plates to the tension zone showed that FRP reinforcement increased beam stiffness by 17 to 79 percent and beam ultimate-strength by 40 to 97 percent. O'Conner et al. [6] used bonded FRP laminates to strengthen a cap-beam and concluded viability of the technique for bridge rehabilitation and its cost-effectiveness for that type of application. Mayo et al. [7] applied bonded FRP-laminates to strengthen and lift load restriction from a simple span, reinforced concrete slab bridge in Missouri. Behavior of reinforced-concrete beams strengthened with bonded CFRP-plates was investigated by Spadea et al. [8] Their study emphasized the important consideration of end-anchorage stresses in the design and indicated a possible increase of about 70 percent in load capacity when external anchorages are used. Saadatmanesh and Ehsani [9] investigated ultimate load capacity and deflection of reinforced concrete beam specimens retrofitted with GFRP plates. Failure of FRP laminate-bonded concrete beams was investigated by Meier and Winistorfer [10], who indicated peeloff of CFRP laminates due to the development of shear cracks as the failure mode of the laminate system. Other reported types of failures include reinforcement yielding, concrete crushing, laminate rupture, and delamination at the reinforcement surface [4]. Sharif et al. studied strengthening of pre-loaded reinforced-concrete beams using GFRP plates [11]. Hag-Elsafi et al. discussed application of FRP materials in retrofitting reinforced-concrete bridge members [2], and Hag-Elsafi and Alampalli investigated similar applications for prestressed concrete bridge members [12].

Anchorage stresses and bond were studied by various researchers, including Mukhopadhyaya and Swamy [13], Neubaurer and Rostasy [14], Ueda et al. [15], Brosens and Van Gemert [16], and Rabinovitch and Frostig [17]. Some of these efforts resulted in development of equations to estimate anchorage length, and all emphasized the importance of proper anchorage and consideration of laminate bond in design. Fatigue strength of concrete beams (reinforced and non-reinforced) strengthened by externally bonded CFRP laminates was studied by Muszynski and Sierakowski [18]. Shear behavior of concrete members strengthened using composite laminates was studied by Khalifa et al. [19], Lees et al. [20], Kachlakev et al. [21], and Hutchinson and Rizkalla [22].

B. PRESENT STUDY

The application discussed in this report is one of several FRP demonstration projects the New York State Department of Transportation has recently completed. Bonded FRP laminates were used in this project to contain cracking, and increase flexural and shear capacities of a reinforced concrete T-beam bridge structure. Total cost of the bridge rehabilitation was estimated at $300,000 in contrast to $1.2 million that would have been required for complete structural replacement. The bridge was instrumented and load tested twice, before and after installation of the FRP laminate system. This report describes these tests and their results, and investigates effectiveness of the strengthening system and its influence on the bridge structural behavior.

II. BACKGROUND

In this chapter, the bridge structure is described, study objectives are stated, and service load stresses in the concrete and main steel rebars are estimated for the American Association of State Highway and Transportation Officials (AASHTO) M18 and MS18 loadings [23].

A. BRIDGE STRUCTURE

The bridge carries State Route 378 over the Wyantskill Creek in the City of South Troy, Rensselaer County, New York. This simple span, reinforced concrete, T-beam structure was built with an integral deck in 1932. The bridge is 12.19-m long, about 36.58-m wide, and is supported by a total of 26 beams spaced at 1.37-m center to center. A plan view of the bridge and a transverse section across the deck are shown in Figure 1. The bridge has been open to traffic without weight-limit restrictions and has an average daily-traffic volume of approximately 30,000 vehicles. It has 5 traffic lanes, and is a vital route linking the City of South Troy with areas west of the Hudson River. During routine inspection, excessive moisture and salt infiltration was observed in the bridge superstructure. Many of the beams had large areas covered with efflorescence, freeze-thaw cracking, and a few beams showed signs of concrete delamination. Concerns about section loss of the reinforcing steel to corrosion and overall safety of the structure was heightened by the absence of any documentation containing complete information needed for reliable evaluation or load rating of the bridge structure. The New York State Department of Transportation (NYSDOT) elected to rehabilitate the structure as opposed to replacement or load posting. An FRP-laminate strengthening system was selected based on its application being the least intrusive with traffic and the most practical. Rehabilitation work, including erection of a full-size platform underneath the bridge, surface preparation, and installation of the laminates was conducted between August and November of 1999.

B. OBJECTIVES

The objectives of this study were to evaluate effectiveness of the FRP strengthening system used in this project and investigate its influence on the bridge structural behavior, using results from load tests conducted before and after installation of the system.

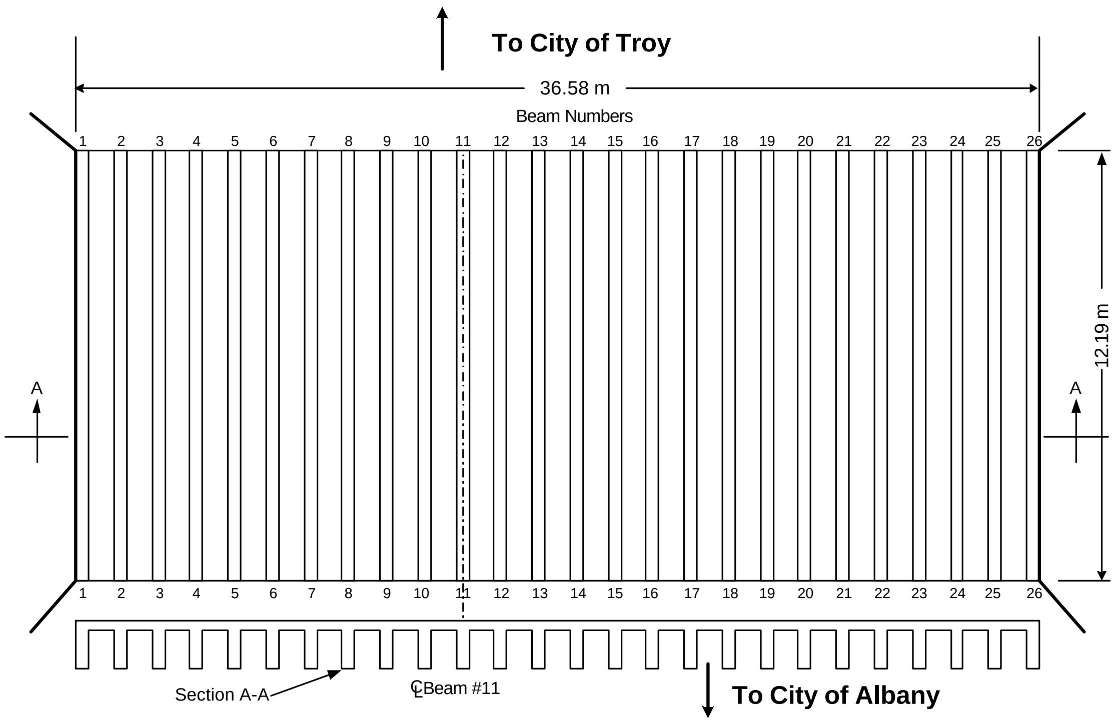


Figure 1. Bridge plan and transverse section.

C. ESTIMATING SERVICE LOAD STRESSES

A typical interior-beam is shown in Figure 2. The main reinforcement is 8, 32 x 32 mm^2, steel rebars. The rebar size was measured when they were exposed to mount strain gages on them during the testing program. The number of main rebars and their layout, and details of the web and deck reinforcement were shown in a shop drawing, which may not represent as built details. Strength of the concrete and reinforcing steel were not available during the design and rehabilitation stages.

Table 1. Summary of service load stresses.

Design Load	M18 (H-20)	MS18 (HS-20)	Allowable Stress (MPa)
Steel-Rebar Stress (MPa)	85.63	97.15	113.76
Concrete Stress (MPa)	4.21	4.76	8.27

Conservatively assuming steel yield strength F_y to be 206.84 MPa and concrete compressive strength f_c' to be 20.68 MPa, and ignoring contribution of compression reinforcing steel, maximum service-load stresses were estimated for the AASHTO M-18 and MS-18 live loadings and are shown in Table 1 [23]. Calculated main steel-rebar and concrete stresses are also compared to their respective allowables in Table 1 [23, 24]. Note that nominal steel-rebar areas were used in the analysis, ignoring any degradation due to corrosion. Although estimated service-load stresses were relatively low based on this analysis, a decision was made to strengthen the bridge for flexure and shear, and to contain the cracking using FRP laminates.

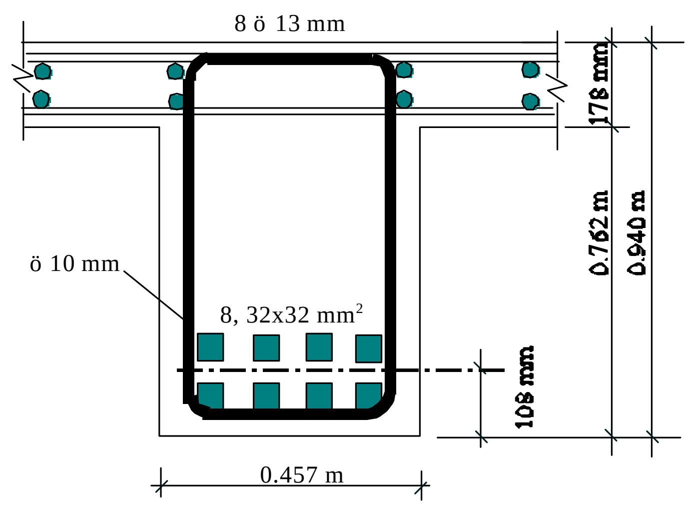


Figure 2. Typical interior-beam section.

III. FRP-LAMINATE SYSTEM DESIGN AND INSTALLATION

Design of the strengthening system, and surface preparation and laminate installation are discussed in the following sections. Design of the laminates was prepared by a third party under a contract with the laminates' manufacturer. The design was part of the warranty agreement, pertaining to the overall performance of the system, between the Department and the manufacturer. Surface preparation and laminate installation were performed according to procedures recommended by the manufacturer.

A. FRP-LAMINATE DESIGN

Flexural and shear design of the system, as designed by the laminate manufacturer, was based on an assumed 15 percent loss, due to corrosion, of the steel reinforcement area (see Appendix A). According to this approach, the required area of laminates A_l is calculated based on the following equation:

$$A_1 = \frac{0.15 A_s F_y}{F_1} \tag{1}$$

where A_s and F_y are, respectively, area and yield stress of steel-rebars, and F_l is the design stress of the laminate material. It is important to note that this method does not account for strain compatibility, and was used here only for its simplicity. For more precise analysis, accounting for strain compatibility is recommended. A more accurate approach to size the laminates, under the same premise of compensating for steel rebar area lost to corrosion, would require the laminate area A_1 be based on:

$$A_1 = \frac{0.15\,(d - \frac{b_1 c}{2})A_s F_y}{(h - \frac{b_1 c}{2})\,F_1} \tag{2}$$

where d is the beam effective depth, $\hat{a}_1c$ is the depth of the Whitney equivalent rectangular stress block, and h is the total beam depth [25]. The webs were strengthened for shear using U-jackets to contain further propagation of the delamination and freeze-thaw cracking and provide additional anchorage for the main laminates. The design of these jackets was also based on a similar percentage-loss of the shear reinforcing-stirrups. However, this approach is also not precise, and the method for estimating laminates contribution to shear strength described in [25] would be more appropriate. The laminate system assumed in the design (Figure 3) was Replark System®, consisting of Replark 30® unidirectional carbon fibers and three types of Epotherm materials, primer, putty, and resin, all manufactured exclusively by Mitsubishi Chemical Corporation of Japan [26]. Properties of the Replark 30® laminates are summarized in Table 2. The ultimate strength of 3400 MPa, corresponds to a guaranteed ultimate strain of 1.5 percent. Design strength is specified as **b** the ultimate strength, and is based on tests of the Replark 30® system applied to a concrete surface (see Appendix A). Unlike most materials, the stress-strain curve for FRP-laminates generally exhibits elastic (linear) behavior until failure is reached. Laminates located at the bottom of the webs and those between beams, indicated as full span length, are oriented parallel to the beams (see Figure 3). Those at the flange soffits, spanning between the beams, are oriented at a right angle to the beams. The U-jacket laminates, applied on the bottom and sides of the beams, are oriented parallel to the legs of the U-jackets.

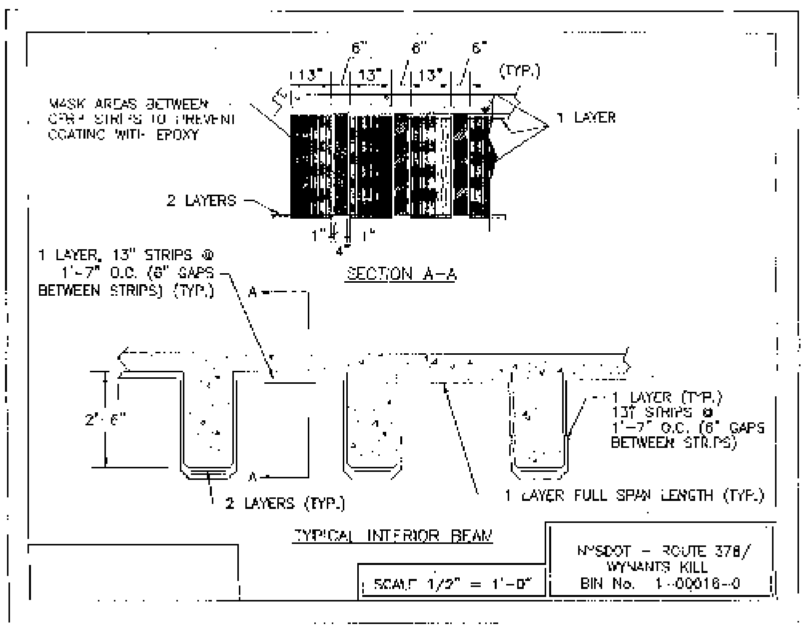


Figure 3. Proposed strengthening FRP-laminate system.

Table 2. FRP-laminate material properties.

Property	Value
Modulus of Elasticity (E_l) (MPa)	2.3×10^5
Ultimate Strain (%)	1.5
Maximum Strain (%)	1.8
Ultimate Strength (MPa)	3400
Design Strength (MPa)	2267

B. INSTALLATION OF FRP-LAMINATE SYSTEM

Surface preparation and laminate installation were performed according to the procedures recommended by the laminate manufacturer [26]. Installation was performed by a contractor under the general directions of a manufacturer's representative. A total of 956 m^2 of Replark 30® laminates was estimated for the project, based on the design details in Appendix A. As recommended by the manufacturer, application of the system closely followed the procedure outlined in the flowchart of Appendix B. This procedure is briefly described next.

Repairing and smoothing a concrete surface is very important for effective development of bond between a laminate and the concrete surface. As such, areas of the beams with visible cracking were first repaired (by removing loose concrete and replacing it with new patching concrete, and filling the cracks with a cement based grout material) and those with uneven surfaces ground to a smooth finish. Sharp edges around the beam corners were then rounded, and the bridge underneath was sand-blasted and pressure washed with water to remove any loose surface materials that could lead to debonding of the laminates. After the surface was dry, laminate locations on the beams and flange soffits were clearly marked. A 15 mm gap was provided between U-jackets laminates to allow an avenue for moisture to escape.

A primer was applied followed by a putty at the locations where the FRP laminates were to be installed (Figure 4A). The primer is expected to penetrate the concrete surface, increase its strength, and improve laminate bonding to the surface [26]. After primer application, gaps and pinholes greater than 1 mm can be seen on the concrete surface. The putty application smoothed the surface by filling the gaps and pinholes.

An epoxy resin was applied to the surface, followed by placement of the laminates. The resin functions as an adhesive to bond the carbon sheets to the concrete surface. It impregnates the fibers and, upon curing, positively bonds the laminate to the concrete surface. Roller pressure was applied to impregnate the laminate as per manufacturer's specifications [26] and heaters were used to control curing temperatures. Depending on type of resin and ambient temperature, complete curing and full load transfer occurs in 5 to 14 days according to the literature provided by the manufacturer. The putty, primer, and resin contain 2-part systems consisting of a main agent and hardener. Properties of the primer, putty, and resin are given in Section 1.2 of Reference [26].

Finally, the FRP laminates were painted, with TAMMS Duralkote 240 paint, for protection from ultraviolet light and aesthetic reasons (Figure 4B). Based on the installation temperatures and resin system applied, the manufacturer recommended a 7-day minimum cure time. The after load test was conducted 10 days after the laminate installation was completed.

Figure 4A. Primer and putty applied at marked locations.

Figure 4B. Installed FRP-laminate system in place (painted to match concrete color).

IV. INSTRUMENTATION AND LOADING

The main objective of the testing program was to evaluate effectiveness of the strengthening system and investigate its influence on structural behavior of the bridge. Two lanes centered directly above Beam 11 were loaded.   Instrumenting 9 beams was judged to be adequate to reflect transverse live-load distribution. These are labeled Beams 7 to 15 in Figure 1. For flexural evaluation, flexural steel and laminate strains were acquired at the midspan of these beams to provide information on live-load distribution. Three other locations on the center beam were also instrumented: near the support to investigate the effect of the strengthening system on shear, and at quarter and mid-spans to assess laminate bond to concrete and laminate stresses. Locations of the instrumentation used to measure these strains on the steel rebars, concrete, and laminates are shown in Figures 5, 6, and 7, respectively. Based on this plan, steel-reinforcement and laminate stresses, as well as effective flange width and position of the neutral axis on the center beam can be determined. Additionally, concrete shear stresses at one end of the center beam can also be determined.

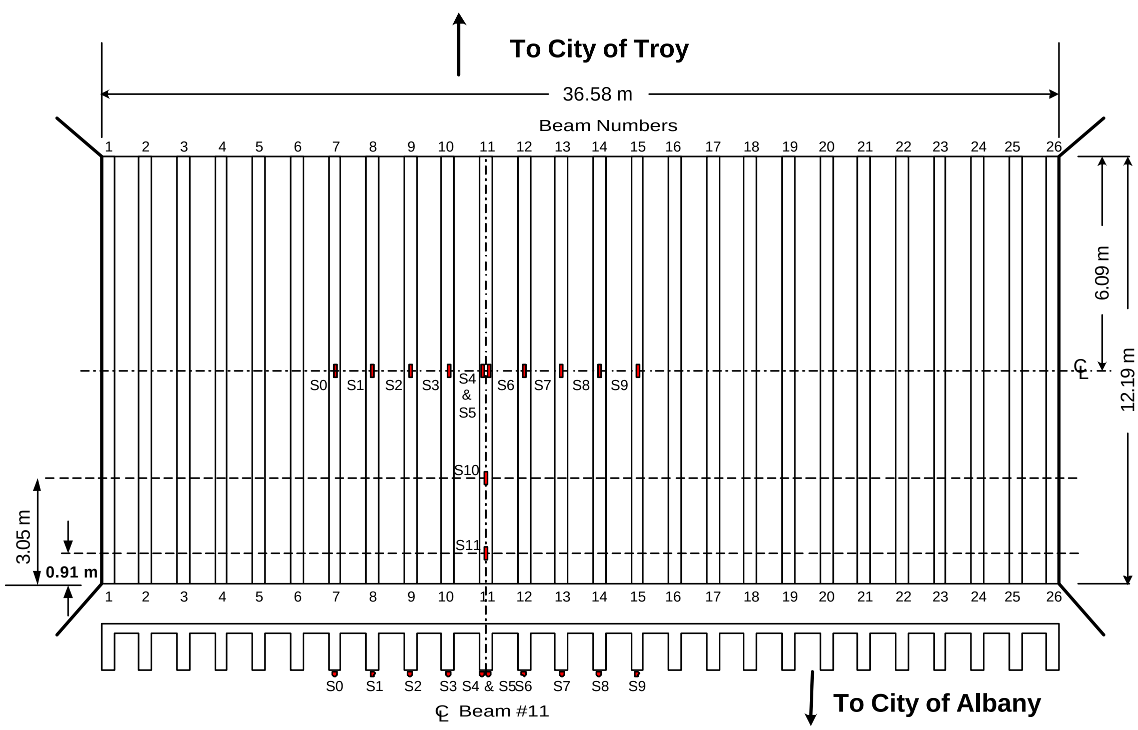


Figure 5. Locations of strain gages mounted on steel rebars.

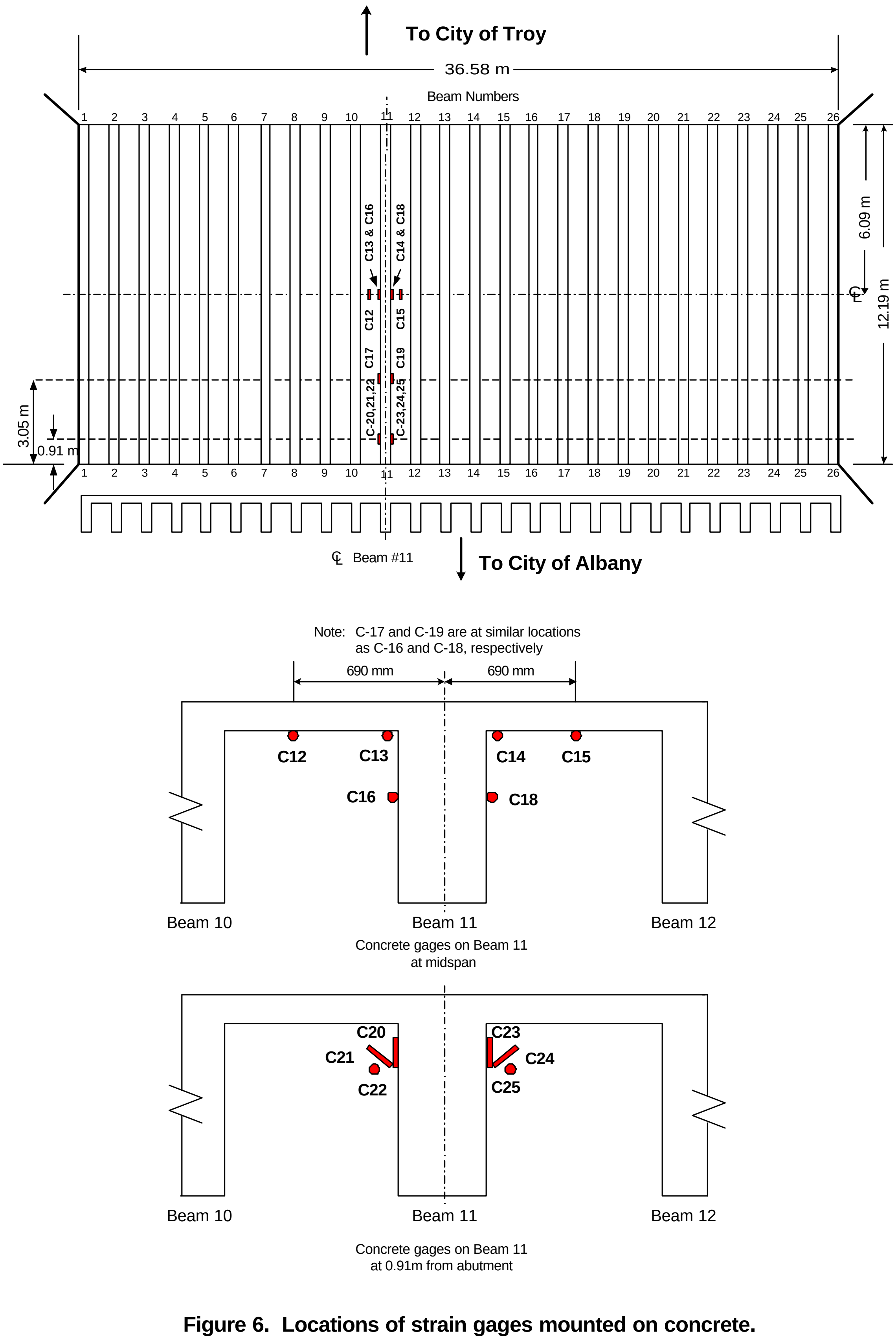


Figure 6. Locations of strain gages mounted on concrete.

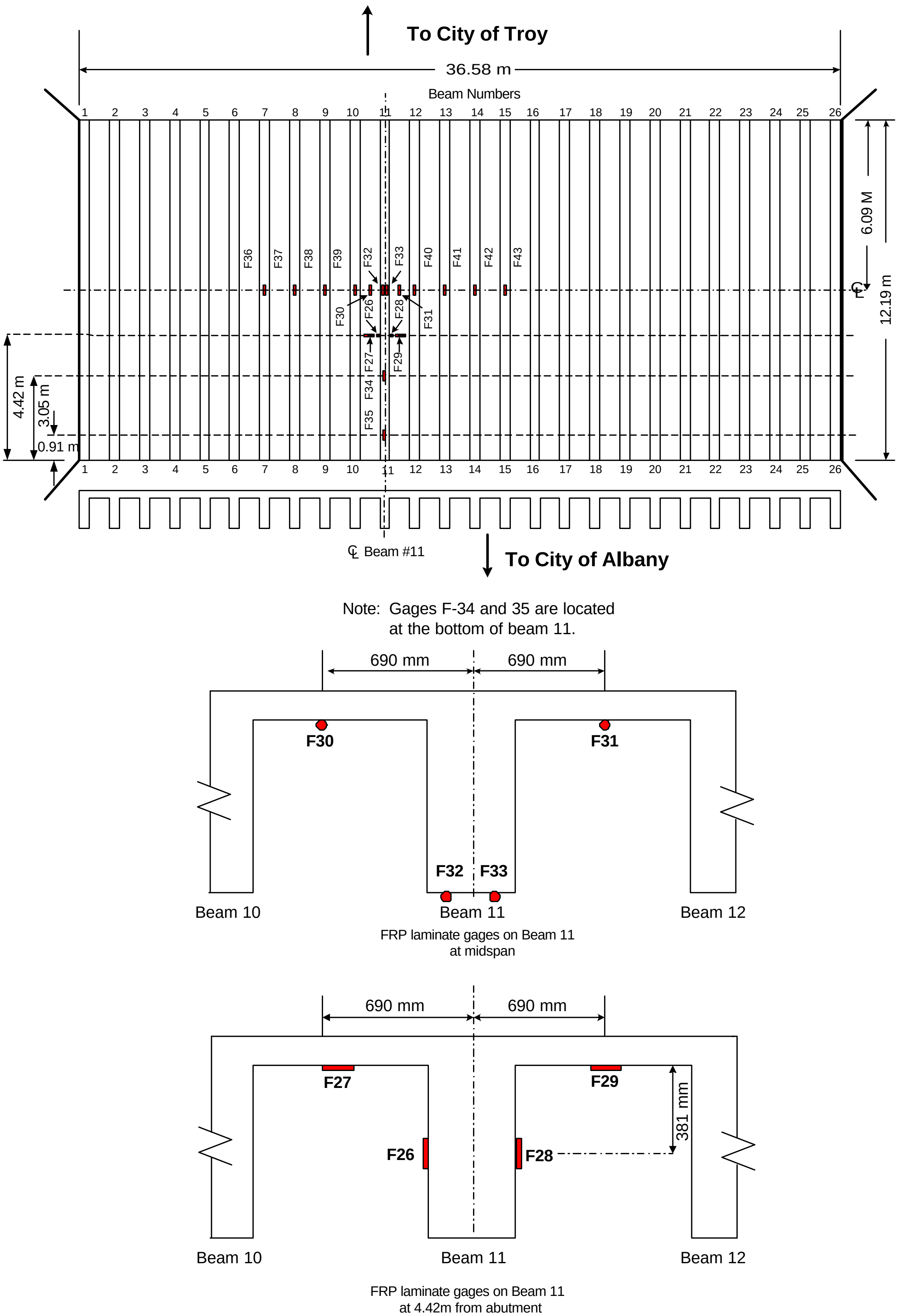


Figure 7. Locations of strain gages mounted on FRP laminates.

A. INSTRUMENTATION AND DATA ACQUISITION

Two types of conventional strain gages were used for measuring strains. All the gages were manufactured by the Measurements Group, Inc.® of Raleigh, North Carolina. General purpose 350Ù, (Type EA-06-250AE-350), self temperature-compensating, constantan foil strain gages, were mounted directly on the reinforcing steel and FRP laminates. On concrete, 120Ù (Type EA-06-20CBW-120), constantan strain gages with large measuring grids were bonded using an epoxy resin. In all, 10 strain gages were mounted on steel rebars and 13 on concrete in the before installation test (shown in Figures 5 and 6). An additional 18 gages were bonded to laminates for the after installation test (shown in Figure 7).   All gages used were made watertight and protected from the environment for long-term monitoring purposes. System 4000, a general purpose data acquisition system, also manufactured by the Measurements Group®, was used for data collection.

B. LOAD-TEST TRUCKS

Four trucks, each of the typical configuration shown in Figure 8, were used in the before and after installation load tests. Average weight of each of these trucks was approximately 196 kN.   By assigning a unique letter, A through D, to each of the four trucks (Figure 9), the testing was sequenced as follows: Truck A, Trucks A+C, Trucks A+B+C, Trucks A+B+C+D, Trucks B+C+D, Trucks B+D, Truck D. Based on prior analysis, the 4.42 m truck position was determined to result in safe stress-levels, assuming a simply-supported condition (Figure 9). Since the actual strength of the structure was not known, 3 truck positions (3.66, 4.11, 4.42 m from each abutment) were marked to gradually increase applied moment on the bridge. Strains were continuously monitored during the tests to determine if it was safe to advance the trucks to the next critical position. On this basis, a new sequence was added to the after installation test in which trucks were parked back-to-back at 4.42 m positions, to maximize load effects on the bridge. The total moment on the bridge due to this configuration was about 2.75 times that due to MS-18 loading [24].

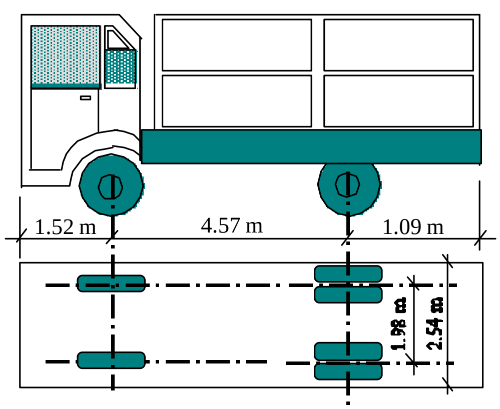


Figure 8. Load-test truck configuration.

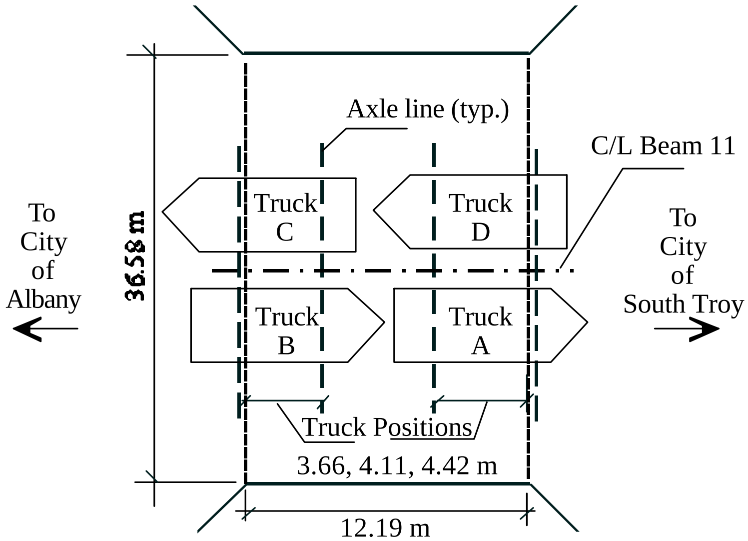


Figure 9. Truck positions for the before and after installation load tests.

V. LOAD-TEST AND CALCULATED RESULTS

Recorded data from the before and after load tests were processed, and checked for validity and consistency. In this section, results from these tests are presented and compared with those obtained using a computer program based on the working stress method [23]. The program may be used for design/analysis of reinforced-concrete rectangular/T-beams strengthened with any combination of top or bottom steel rebars and FRP laminates. For completeness, readings for some of the gages which malfunctioned during the testing were estimated using other readings, when possible, and ignored otherwise.

A. LINEAR BEHAVIOR AND DATA CONSISTENCY

Linear behavior of the bridge structure is investigated, for the before and after installation tests, using calculated moments and measured midspan strains for Beam 11 (see Figures 10 and 11). Data for all truck sequences was used in these figures, including those for Trucks A+B+C+D parked back-to-back at 4.42-m position in the after installation test. Relatively small scatter of the recorded data was observed about the best-fit lines. This not only confirms linear behavior of the structure, but also proves consistency of recorded data. From these figures it is also evident that the beam stiffness, as measured by the slope of the two best fit lines, did not exhibit significant change after installation of the laminates.

B. GENERAL FLEXURAL BEHAVIOR

Before and after installation strains in flexural steel at midspan of Beams 7 to 15 (Figure 5), for the various truck combinations at 3.66, 4.11, and 4.42-m positions on the bridge are shown in Figure 12. These results clearly confirm the adequacy of the test plan (limiting instrumentation to 9 beams) and show Beam 11 to be the most stressed beam, as planned.

Before and after installation strains for gages mounted on steel-rebars and concrete for all 4 trucks (Trucks A+B+C+D) at the 4.42-m position on the bridge are shown in Figure 13. Similar results for the after installation strains for gages mounted on the laminates are shown in Figure 14. Comparing the before and after readings for gages mounted on the steel rebars (Figure 13), it can be concluded that installation of the FRP laminates slightly reduced rebar stresses. Relatively higher rebar strains after installation of the FRP laminates may be attributed to random variations and minor changes in truck positions during the testing.

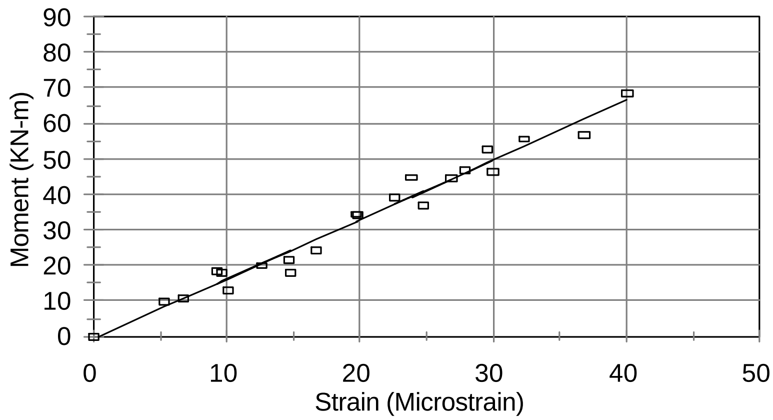


Figure 10. Moment versus strain at the midspan of Beam 11 for the before installation load test (each truck sequence at 3.66, 4.11, and 4.42-m positions on the bridge).

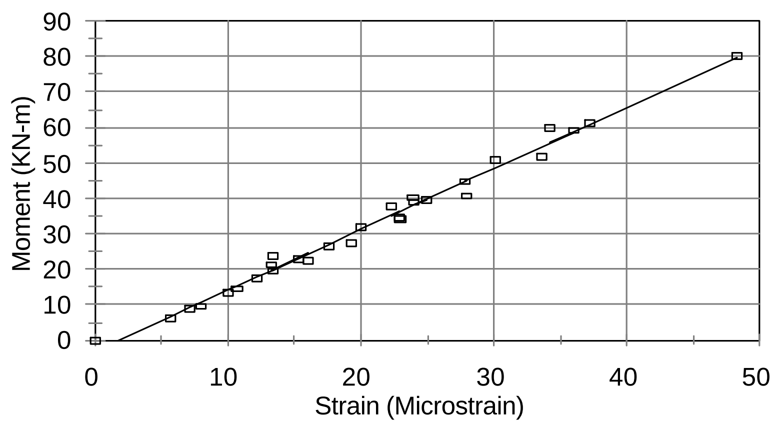


Figure 11. Moment versus strain at the midspan of Beam 11 for the after installation load test (each truck sequence, including back to back, at 3.66, 4.11, and 4.42-m positions on the bridge).

Gage S-11, located 0.914-m from the Albany side abutment at the bottom of Beam 11 (see Figure 13), consistently measured negative (compressive) strains indicating end fixity of the beam. This was later verified through back-calculation of moments based on measured strains, and was attributed to malfunctioning of the expansion bearings. Malfunctioning of expansion bearings is known to cause such fixity, which substantially reduces live-load moments. For example, Beam 11 moment with all four trucks positioned at 4.42 m from the abutments was reduced from 209.10 (based on simply supported conditions) to 75.93 kN-m. This may be compared to 79.72 kN-m calculated based on recorded strains. Comparing the FRP strains in Figure 14 with those recorded on the rebars (Figure 13), it can be concluded that laminate strains for some gages were lower than expected. Since the laminates were physically located below the main rebars in the beam section, strain compatibility would require laminate strains to be higher than rebar strains.

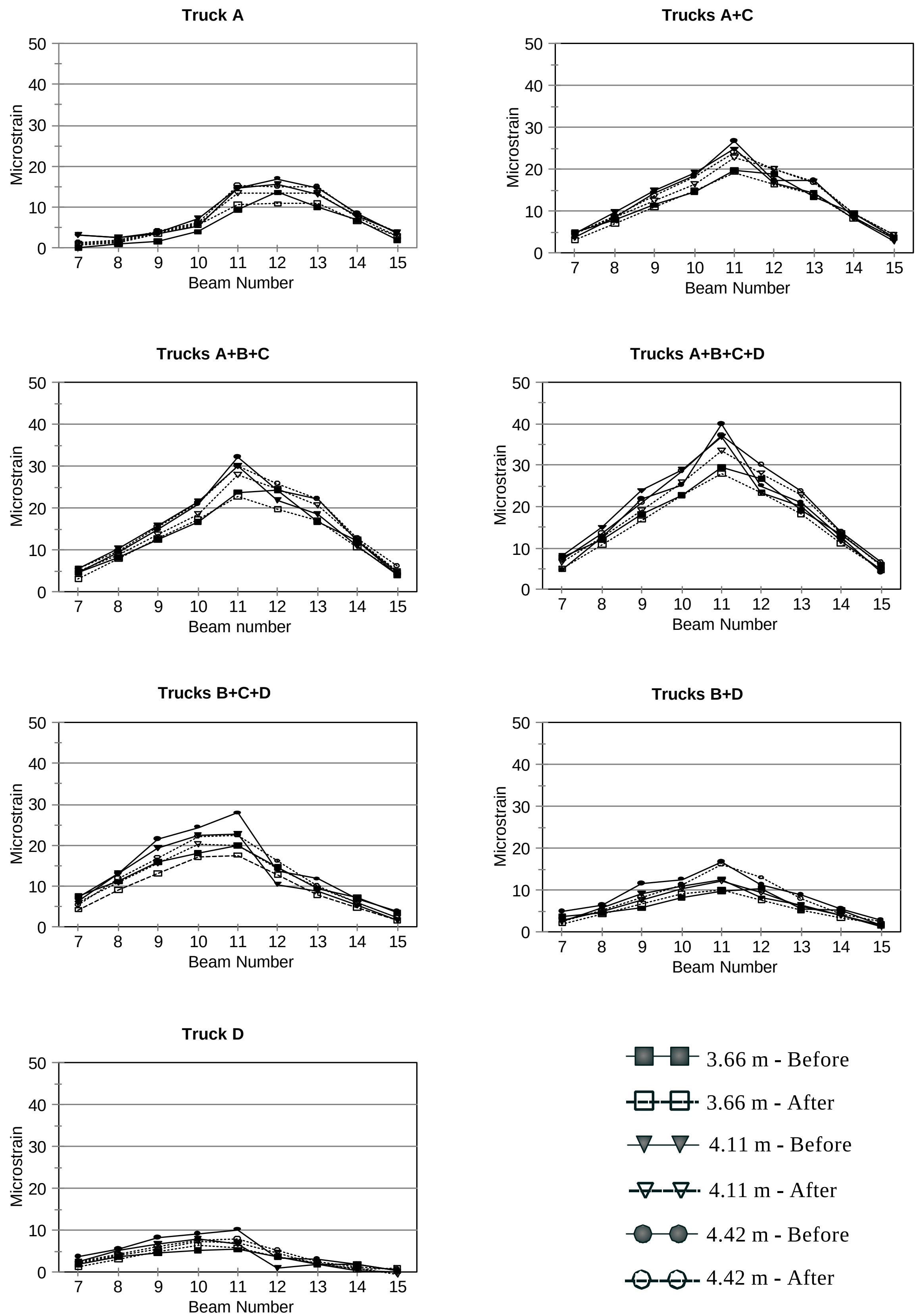


Figure 12. Maximum recorded strains in steel rebars in the before and after load tests.

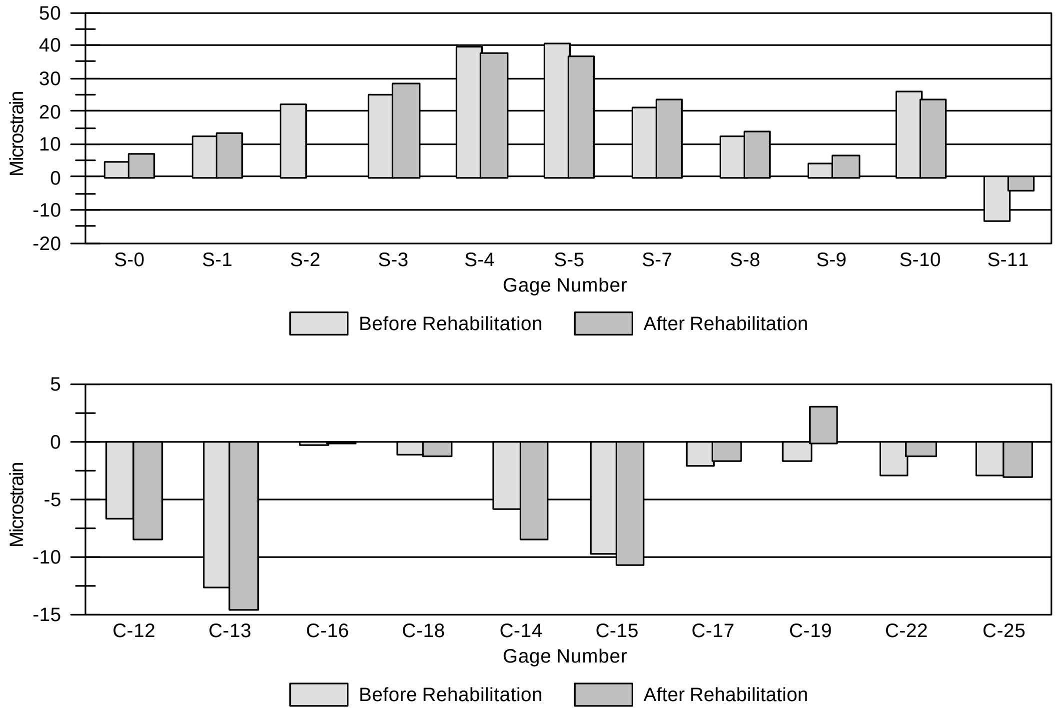


Figure 13. Recorded strains in the before and after tests for gages mounted on steel rebars and concrete (all 4 trucks parked at 4.42-m position).

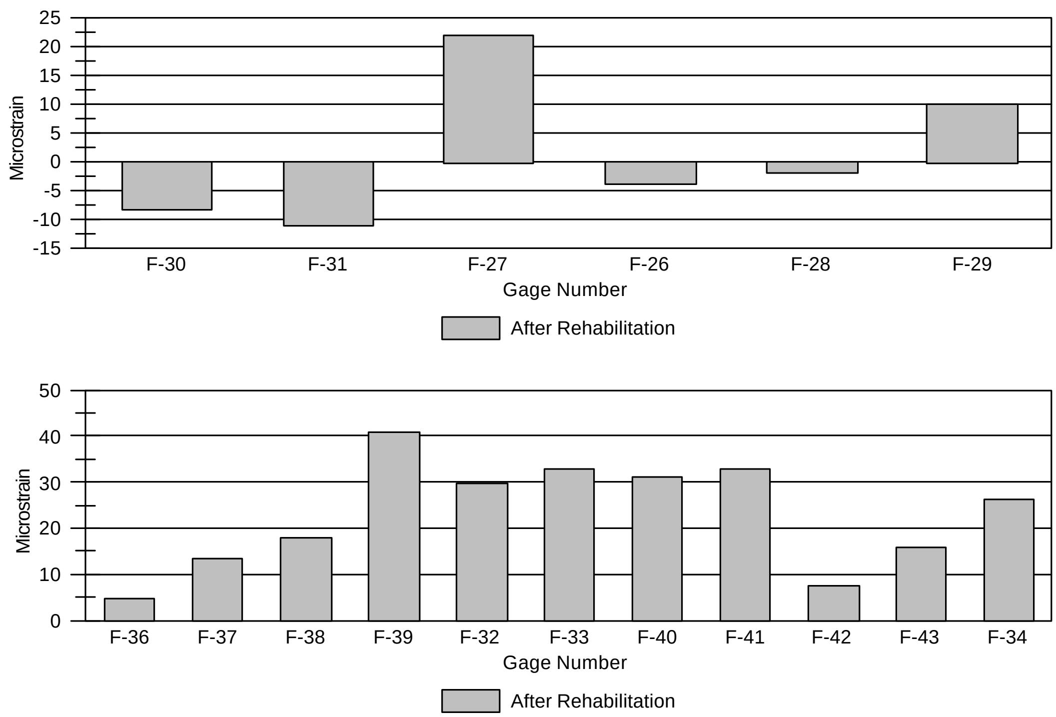


Figure 14. Recorded strains in the after test for gages mounted on FRP laminates (all 4 trucks parked at 4.42-m position on the bridge).

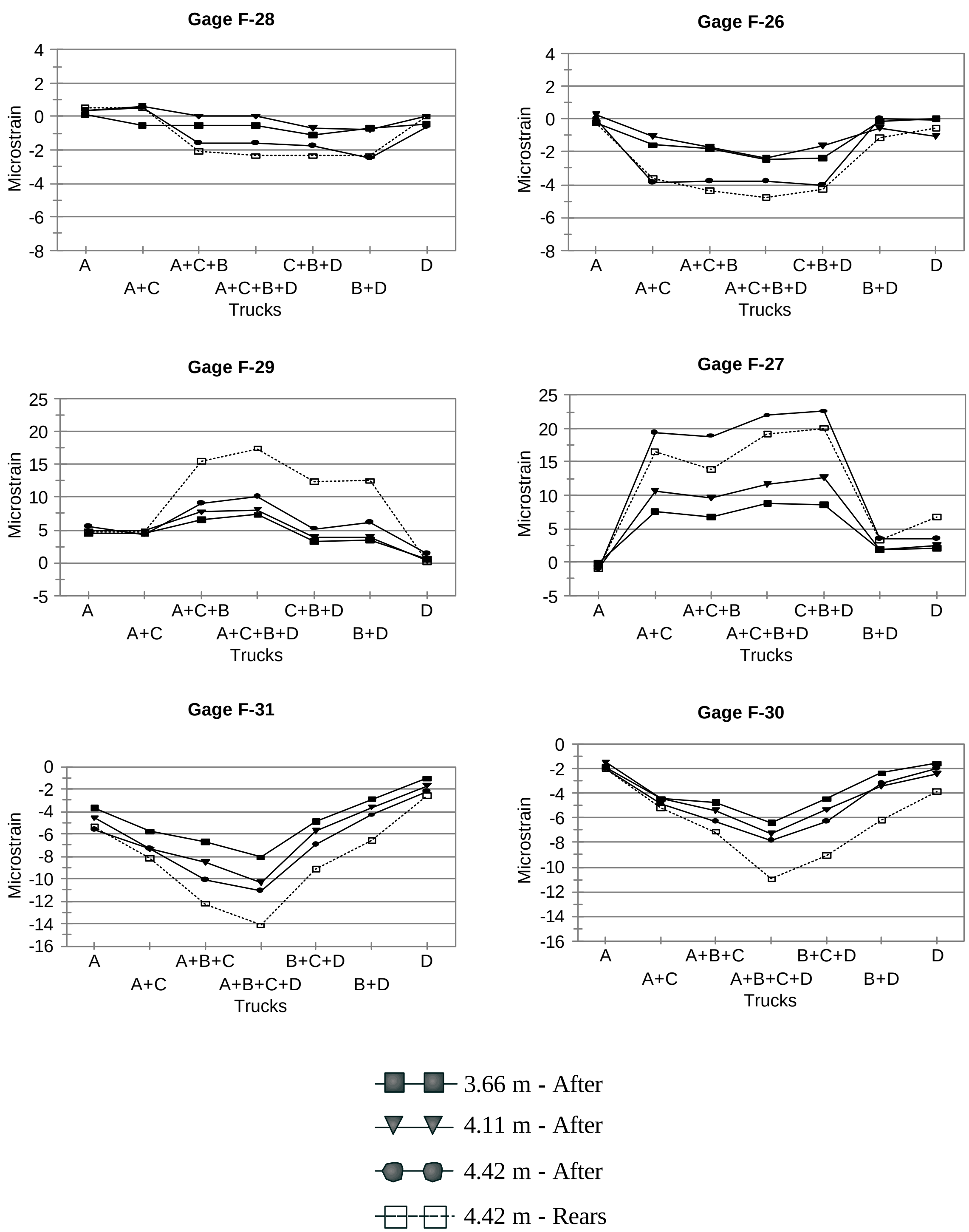


Figure 15. Recorded strains in the after test for gages mounted on FRP laminates (various truck combinations parked at 3.66, 4.11, and 4.42-m, and back-to-back at 4.42-m positions on the bridge).

Measured laminate strains (Figure 15), for the various truck combinations and positions, further confirm consistency of the data and effectiveness of the laminates in carrying load. From this figure, for any given truck position, Gages F-26 and F-28 (mounted on web sides at ¼ span) clearly recorded strains that are proportional to applied shear. The negative (compressive) strain readings for these gages may be attributed to localized effect of truck tire loads, being applied in close proximity to the gage locations, and beam fixity. Gages F-27 and F29 (mounted on transverse laminates between beams) measured flexural strains due to bending of the deck slab between Beams 10 and 11, and 11 and 12, respectively. Gages F-30 and F-31 (mounted on longitudinal laminates between beams at flange soffits) recorded flexural strains that are proportional to the moment in Beam 11.

C. TRANSVERSE LOAD DISTRIBUTION

The data in Figure 12 was used to determine live-load distribution factors for Beam 11. These factors are presented in Table 3 for the cases of Trucks A+B+C+D parked at 3.66, 4.11, and 4.42- m positions on the bridge. Comparing the distribution factors from the before and after installation tests, it can be concluded that live-load distribution improved by about 12 percent after the laminates were installed. This was mostly influenced by the laminates installed transversely beneath the deck between the beams (see Figure 3).

Table 3. Live-load moments and distribution factors for Beam 11.

Trucks Position from Abutments (m)	Before Rehabilitation		After Rehabilitation	
	Total Moment at Midspan (kN-m)	Live-Load Distribution Factor	Total Moment at Midspan (kN-m)	Live-Load Distribution Factor
3.66	220.5	0.230	204.7	0.198
4.11	278.3	0.214	259.1	0.199
4.42	321.1	0.236	299.0	0.204

D. FLEXURAL STRESSES AND COMPARISON WITH CLASSICAL ANALYSIS

Before and after installation rebar stresses at the midspan of Beam 11, for all 4 trucks placed at 3.66, 4.11, and 4.42-m positions on the bridge, are compared with those obtained analytically [23, 27] in Figure 16. The results indicate excellent agreement with the analytical results presented in Figure 16. A comparison between before and after installation stresses obtained from test data (see Figure 17) clearly shows that installation of the FRP laminates moderately reduced rebar stresses at the midspan of Beam 11.

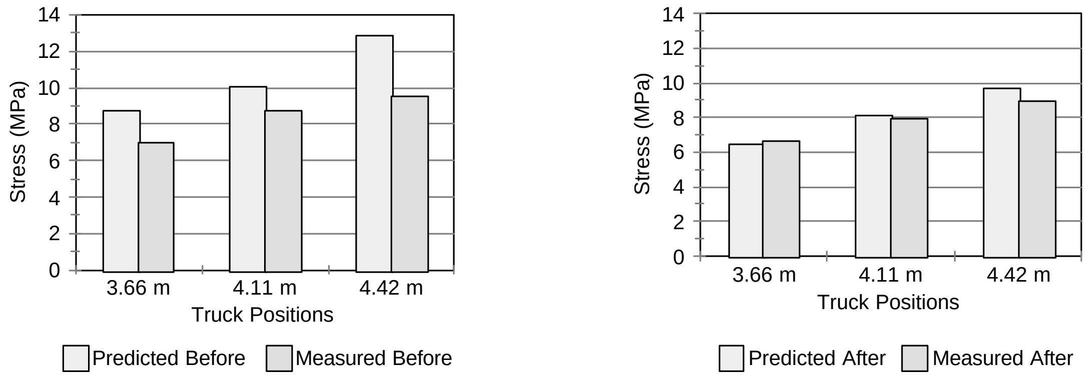


Figure 16. Comparison of steel-rebar stresses: Classical analysis versus those based on test results (all 4 trucks parked at 3.66, 4.11, and 4.42-m positions on the bridge).

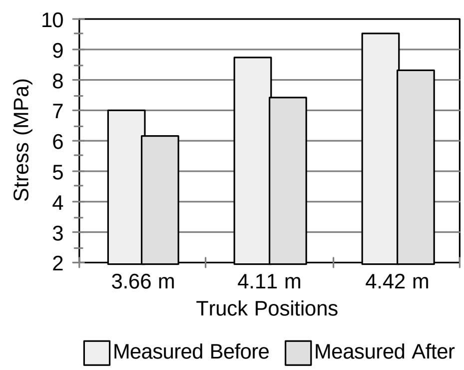


Figure 17. Comparison of steel-rebar stresses: Before versus after installation of laminates (all 4 trucks parked at 3.66, 4.11, 4.42-m positions on the bridge).

E. EFFECTIVE FLANGE WIDTH AND NEUTRAL AXIS LOCATION

Using strain data from gages mounted on Beam 11 flange soffit (Gages C-12 and C-13), effective flange-widths were estimated for all three loading positions (see Table 4). These results show that effective flange-width increased slightly for the 3.66-m position, and remained unchanged for the other two positions after the laminates were installed. Comparing the before and after installation strains recorded for these strain gages, it is clear that compressive strains in the concrete were higher after the laminates were installed. To investigate this further, neutral axis locations in Figure 18 were determined as shown in Table 5. These results indicate that, as expected, the neutral axis migrated downwards by about 33 mm (1.30 in.) after the laminates (mainly influenced by flexural laminates) were installed. A simple strain diagram showing a reduced strain at the bottom of the beam with a lower neutral axis location explains the increase in concrete strains where Gages C-12 and C-13 were mounted.

Table 4. Effective flange width investigation (Trucks A+B+C+D parked at 3.66, 4.11, and 4.42-m positions on the bridge).

Truck Position (m)	Measured Compressive Strain (µå)				Effective Flange Width (m)	
	Gage C-12		Gage C-13			
	Before	After	Before	After	Before	After
3.66	-4	-6	-9	-11	1.143	1.194
4.11	-6	-7	-11	-13	1.168	1.168
4.42	-7	-8	-12	-14	1.168	1.168

Table 5. Neutral axis investigation (Trucks A+B+C+D parked at 3.66, 4.11, and 4.42-m positions on the bridge).

Truck Position (m)	Before Installation			After Installation				
	Top Strain[a] (µå)	Bottom Strain[b] (µå)	Neutral Axis Location[c] (mm)	Top Strain[a] (µå)	Bottom Strain[b] (µå)	Neutral Axis Location[c] (mm)	Predicted Laminate Strain[d] (µå)	Measured Laminate Strain[e] (µå)
3.66	-9	30	163	-11	28	197	32	25
4.11	-11	37	160	-13	34	195	38	30
4.42	-12	40	161	-14	38	190	42	33

[a] Gage C-13 strains; [b] Average Gages S-4 and S-5 strains; [c] Measured below flange soffit; [d] Calculated based on Top and Bottom strains; [e] Gage F-33 strains.

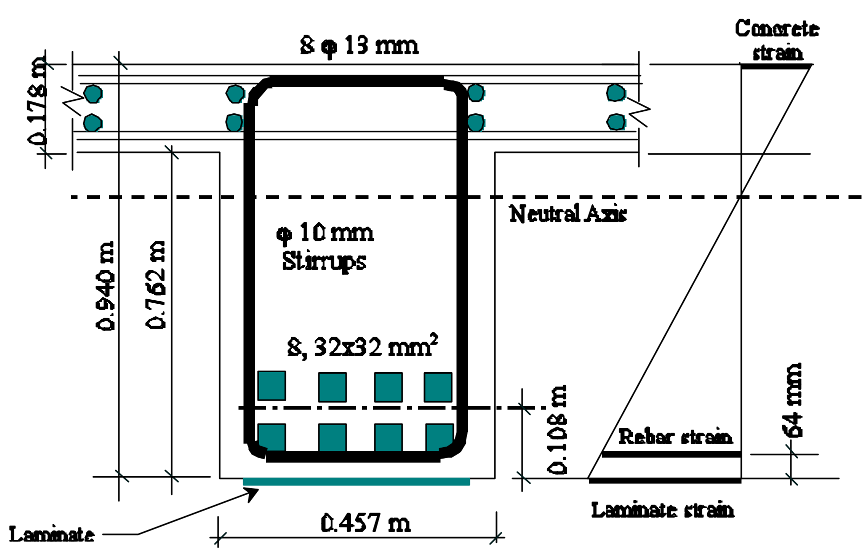


Figure 18. Neutral axis calculations.

F. FRP-LAMINATE BOND TO THE CONCRETE

Bond between the FRP laminates and the concrete was also investigated on basis of strain compatibility. Good bond is concluded when insignificant difference is observed between laminates and concrete strains at the same location. Such a comparison at two locations, above and below the neutral axis, is shown in Table 6 for Trucks C+B+D (for maximum shear at midspan) on the bridge. Bottom concrete strains for comparison with Gage F-32 strain were calculated using Gage S-4 readings and an average location of the neutral axis – calculated based on Table 5 data. From Table 6, better bond can be concluded for the laminates located above (under compression) than those located below (under tension) the neutral axis. The weaker bond may be attributed to the level of precision in strain measurements and/or a lack of full bond development between the laminates and concrete at the time the testing was conducted. Another load test is planned to further investigate this issue.

Table 6. Comparison between concrete and laminate strains at similar locations for bond investigation (Trucks B+C+D parked at 3.66, 4.11, and 4.42-m positions on the bridge).

Truck Position (m)	Strain (µå)					
	Gage C-12 Versus Gage F-30			Bottom Concrete Versus Gage F-32		
	Gage C-12	Gage F-30	Difference (%)	Bottom Concrete	Gage F-32	Difference (%)
3.66	-5	-4	20	20	14	30
4.11	-5	-5	0	24	17	29
4.42	-6	-6	0	27	19	30

G. SHEAR STRESSES AND COMPARISON WITH CLASSICAL ANALYSIS

Figure 19 shows measured shear stresses for the before and after laminate installation plotted against applied shear forces. Shear stresses and forces were calculated using data from strain gage rosettes and applied truck loads. Comparing the before and after installation linear fit lines in Figure 19, a slight increase in concrete shear stress is noted after the laminates were installed. However, presence of the U-jacketed laminates is expected to provide confinement of web concrete and shear-resisting interlock mechanism, hence improving its ultimate shear capacity [19, 20, 21]. Comparison of either of these lines with the analytical prediction line, which is based on classical analysis, in the figure indicates good agreement between the experimental and analytical results. Linear behavior is also noted under both approaches. A more detailed investigation of shear behavior was not possible because of the uncertainty of steel stirrup size and placement, and the quality of the cracked web concrete.

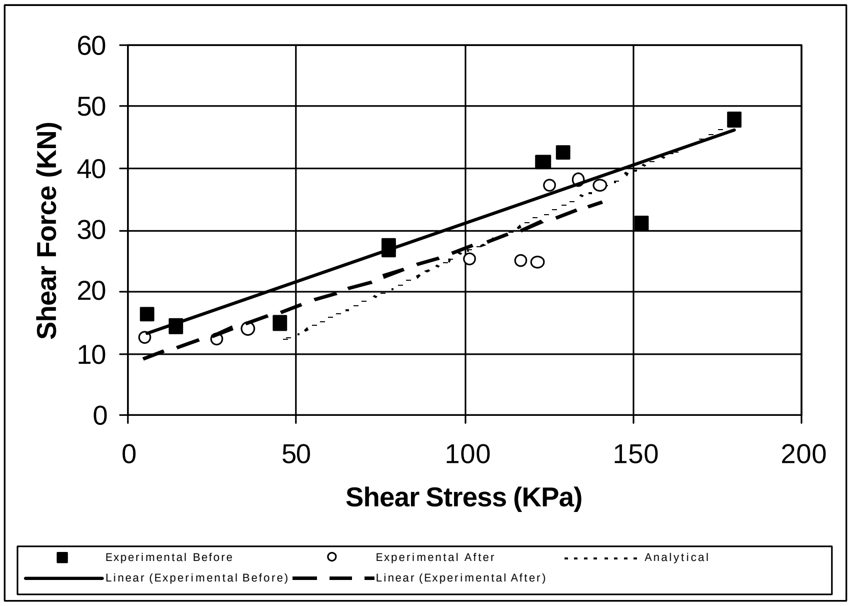


Figure 19. Shear force versus shear stress: Classical analysis versus those based on test results.

VI. CONCLUSIONS

Bonded FRP laminates were used in the application discussed in this report to contain freeze-thaw cracking and improve flexural and shear strength of a reinforced-concrete T-beam bridge structure. Load tests were conducted before and after installation of the laminates to evaluate effectiveness of the strengthening system and investigate its influence on structural behavior of the bridge. Test results were analyzed and compared with those obtained using classical analysis. The main conclusions based on these results are summarized below.

1. Under service live load, after the laminates were installed, main rebar stresses were moderately reduced, concrete stresses (flexural and shear) moderately increased, and transverse live-load distribution to the beams slightly improved. Although the laminates participated in load carrying, compatibility of strains was not satisfied at some locations, attributed to the level of precision in strain measurements and/or a lack of full bond development at the time of the testing.

2. Unintended fixity of the beam ends was discovered, which substantially reduced anticipated live load moments.

3. As expected, after the laminates were installed, the neutral axis migrated downwards, but effective flange width remained almost unchanged for all truck positions.

The benefits of the FRP-laminate system used in this project may not be fully realized within the loading range used in the testing program. However, various studies have concluded significant increase in ultimate capacities of concrete members strengthened using these laminates. The maximum load applied during the testing program, about 2.75 MS-18 loading, was not sufficient to induce nonlinear behavior.

Using bonded FRP laminates in this project provided an opportunity for NYSDOT to demonstrate their use and investigate their feasibility as a cost-effective bridge rehabilitation technique. The project caused minimal traffic interruptions which should encourage similar applications in highly-populated metropolitan areas. Total cost of the rehabilitation is estimated at $300,000, which may be compared to $1.2 million required for replacement of the structure.

ACKNOWLEDGMENTS

Instrumentation and data collection for this project was conducted by George Schongar and Harry Greenberg of the Transportation Research and Development Bureau. Ryan Lund, also with the Bureau, and Frank Owens, formerly with the Bureau, assisted in this project. NYSDOT Region 1 staff members assisted with the testing and traffic control. This project was partially funded by the Federal Highway Administration (FHWA).

REFERENCES

1. *"The Status of the Nation's Highway Bridges: Highway Bridge Replacement and Rehabilitation Program and National Bridge Inventory,"* Thirteenth Report to the United States Congress, Federal Highway Administration, U.S. Department of Transportation, May 1997.

2. Hag-Elsafi, O., Alampalli, S., Kunin, J., and Lund, R. "*Application of FRP Materials in Bridge Retrofit*," 7th International Conference on Composites Engineering, Denver, CO, 2000.

3. Alampalli, S., O'Connor, J., and Yannotti, A. "*Design, Fabrication, Construction, and Testing of an FRP Superstructure*," Special Report 134, Transportation Research & Development Bureau, New York State Department of Transportation, FHWA/NY/SR-00/134.

4. Reddy, D. V., Grevois, G. B., and Carlson L. A. "*Laminate Bonding for Concrete Repair and Retrofit*," Materials for the New Millennium, Proceedings of the Materials Engineering Conference, Washington, D.C., Vol. 2, 1996, pp 1579-91.

5. Ritchie, P. A., Thomas, D. A., Lu, L. W., and Connely, G. M. "*External Reinforcement of Concrete Beams Using Fiber Reinforced Plastics*," ACI Structural Journal, Vol 88, No. 4, July-August 1991, pp 490-500.

6. O'Connor, J., Hoyos, H., Yannotti, A., Alampalli, S., and Luu, K. "*Reinforced Concrete Cap-Beam Strengthening Using FRP Composites*," Proceedings of the Fourth International Symposium, Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, Baltimore, 1999.

7. Mayo, R., Nanni, A., Gold, W., and Barker, M. "*Strengthening of Bridge G270 with Externally Bonded Carbon Fiber Reinforced Polymer Reinforcement*," Proceedings of the Fourth International Symposium, Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, Baltimore, 1999, pp 429-440.

8. Spadea, G., Bencardino, F., and Swamy, R. N. "*Structural Behavior of Composite RC Beams with Externally Bonded CFRP*," Journal of Composites for Construction, Vol. 2 No 3, August 1998, pp 132-137.

9. Saadatmanesh, H., and Ehsani, M. R. "*RC Beams Strengthened with GFRP Plates, I: Experimental Study*," Journal of the Structural Division, ASCE, Vol. 117, No. 11, 1991, pp 3417-3433.

10. Meier, U. and Winistorfer, A. "*Retrofitting of Structures through External Bonding of CFRP Sheets*," Non-metallic (FRP) Reinforcement for Concrete Structures, L Taerwe, Ed., Rilem, E&FN Spon., London, 1995.

11. Sharif, A., Al-Sulaimani, G. J., Basunbul, I. A., and Ghaleb, B. N. "*Strengthening of initially Loaded Reinforced Concrete Beams Using FRP Plates,*" ACI Structural Journal, March-April 1994 pp 160-168

12. Hag-Elsafi, O. and Alampalli, S. "*Strengthening Prestressed-Concrete Beams Using FRP Laminates*," Structural Materials Technology, An NDT Conference, Edited by Sreenivas Alampalli, Atlantic City, NJ, 2000.

13. Mukhopadhyaya, P., and Swamy, R. N. "*Critical Review of Plate Anchorage Stresses in Premature Debonding Failures of Plate Bonded Reinforced Concrete Beams*," Proceedings of the Fourth International Symposium, Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, Baltimore, 1999, pp 359-368.

14. Neubauer, U., and Rostasy, F. S. "*Bond Failure of Concrete Fiber Reinforced Polymer Plates at Inclined Cracks- Experiments and Fracture Mechanics Model*," Proceedings of the Fourth International Symposium, Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, Baltimore, 1999, pp 369-381.

15. Ueda, T., Sato, Y., and Asano, Y. "*Experimental Study of Bond Strength of Continuous Carbon Fiber Sheet*," Proceedings of the Fourth International Symposium, Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, Baltimore, 1999, pp 407-416.

16. Brosens, K., and Van Gemert, D. "*Anchorage Design for Externally Bonded Carbon Fiber Reinforced Polymer Laminates*," Proceedings of the Fourth International Symposium, Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, Baltimore, 1999, pp 635-648.

17. Rabinovitch, O. and Frostig, Y. "*Edge Effect in Retrofitting of Concrete Beams Using Fiber Reinforced Polymer Stripes -- Closed-Form, High Order Theory Approach*," Proceedings of the Fourth International Symposium, Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, Baltimore, 1999, pp 179-191.

18. Muszynski, L. C., and Sierakowski, R. L. "*Fatigue Strength of Externally Reinforced Concrete Beams*," Materials for the New Millennium, Proceedings of the Materials Engineering Conference, Washington, D.C., 1996, Vol. 2, pp 648-656.

19. Khalifa, A., Tumialan, G., Nanni, A., and Belarbi, A. "*Shear Strengthening of Continuous Reinforced Concrete Beams Using Externally Bonded Carbon Fiber Reinforced Polymer Sheets*," Fourth International Symposium-Fiber Reinforce Polymer Reinforcement for Reinforced Concrete *Structures*, American Concrete Institute, MI, 1999, pp 995-1003.

20. Lees, J.M., Winistoerfer, A., and Meier, U. "*Nonlaminated Pin-Loaded Carbon Fiber Reinforced Plastic Straps for Shear Enhancement of Concrete*," *Fourth International Symposium-Fiber Reinforce Polymer Reinforcement for Reinforced Concrete Structures*, American Concrete Institute, MI, 1999, pp 985-993.

21. Kachlakev, D.I., and Barnes, W.A. "*Flexural and Shear Performance of Concrete Beams Strengthened with Fiber Reinforced Polymer Laminates*," Fourth International Symposium-Fiber Reinforce Polymer Reinforcement for Reinforced Concrete Structures, American Concrete Institute, MI, 1999, pp 959-967.

22. Hutchinson, R.L., and Rizkalla, S.H. "*Shear Strengthening of AASHTO Bridge Girders Using Carbon Fiber Reinforced Polymer Sheets*," Fourth International Symposium-Fiber Reinforce Polymer Reinforcement for Reinforced Concrete Structures, American Concrete Institute, MI, 1999, pp 945-958.

23. "*Standard Specifications for Highway Bridges,*" 1996 (16th edition), American Association of State Highway and Transportation Officials, Washington, D.C.

24. "*Manual for Condition Evaluation of Bridges*," 1994, American Association of State Highway and Transportation Officials, Washington, D.C.

25. "*Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures*," Revised January 2000. Report by ACI Committee 440, American Concrete Institute (ACI), Detroit, MI.

26. "*Replark System Technical Manual*," November 1999, Mitsubishi Chemical Corporation, Japan, Distributed by Sumitomo Corporation of America, USA.

27. "*Building Code Requirements for Reinforced Concrete and Commentary ACI318M/318RM-95*," 1995, American Concrete Institute (ACI), Detroit, MI.

28. "*Design Proposal and Specifications for CRFP Repair and Retrofit of Everett Road/I-90 & Route 378/Wyantskill Bridge*," July 1999, Mitsubishi Chemical Corporation with Ammann & Whitney Consulting Engineers.

APPENDIX A

DESIGN CALCULATIONS AND QUANTITY ESTIMATES [28]

By: A. Martecchini
Date: 7/17/99

Project: NYSDOT -Route 378 / Wynants Kill
BIN No. 1-00016-0

Design of Underdeck CFRP Wrap (Flexure)

Deck Reinforcement is # 5 @ 11 in. spacing

Total Reinforcement = A_s = 0.31 in^2 x (12 in. / 11 in.) = 0.34 in^2 / ft.

Assume 15% of section loss which will be replaced with REPLARK 30

$A_{s(lost)}$ = 0.34 in^2 / ft. x 0.15 = 0.051 in^2 / ft.

$A_{Rep} \times f_{Rep} = A_s \times f_s$ Where

A_s	=	Area steel lost
f_s	=	f_y = 40 ksi (assume ASTM A615 rebar Gr. 40)
A_{Rep}	=	Area REPLARK 30 sheet
f_{Rep}	=	0.67 x 493 ksi = 330 ksi

$$A_{Rep} = \frac{40 \text{ ksi} \times 0.051 \text{ in}^2/\text{ft.}}{330 \text{ ksi}} = 0.0062 \text{ in}^2/\text{ft.}$$

For standard REPLARK 30 t_{Rep} = 0.0066 in.

Area REPLARK 30 / sheet = 0.0066 in. x 13 in. = 0.086 in^2

Sheets spaced with 6 in. gaps to allow drainage. Center-to-center spacing = 1.58 ft.

Area REPLARK 30 / foot = 0.086 in^2 / 1.58 ft. = 0.054 in^2 / ft. $\gg$ 0.0062 in^2/ ft.

$\therefore$ **Use 1 layer REPLARK 30 (13 in. wide sheets) transverse to beams**

Add 1 layer of REPLARK 30 parallel to beams at midspan of deck slab to act as distribution for concentrated loads.

By: A. Martecchini **Project**: NYSDOT -Route 378 / Wynants Kill
Date: 7/17/99 BIN No. 1-00016-0

Design of Beam CFRP Wrap (Shear)

Shear stirrups are # 4 bars @ 8 in. minimum spacing

Total Reinforcement = $A_s = 0.20\ in^2 \times (12\ in. / 8\ in.) = 0.30\ in^2 / ft.$

Assume 15% of section loss which will be replaced with REPLARK 30

$A_{s(lost)} = 0.30\ in^2 / ft. \times 0.15 = 0.045\ in^2 / ft.$

This value is less than the $A_{s(lost)}$ for underdeck

∴ **Use 1 layer of REPLARK 30 (13 in. wide sheets) spaced with 6 in. gaps.**

Design of T-Beam CFRP Wrap on Bottom Flange (Flexure)

Actual bar sizes not available from original plans

Assume 8 - # 8 bars $A_s = 0.79\ in^2$ / bar

$A_{s(total)} = 8 \times 0.79\ in^2 = 6.32\ in^2$

Assume 15% of section loss which will be replaced with REPLARK 30

$A_{s(lost)} = 6.32\ in^2 \times 0.15 = 0.95\ in^2$

$A_{Rep} \times f_{Rep} = A_s \times f_s$ Where

A_s = Area steel lost
f_s = f_y = 40 ksi (assume ASTM A615 rebar Gr. 40)
A_{Rep} = Area REPLARK 30 sheet
f_{Rep} = 0.67 x 493 ksi = 330 ksi

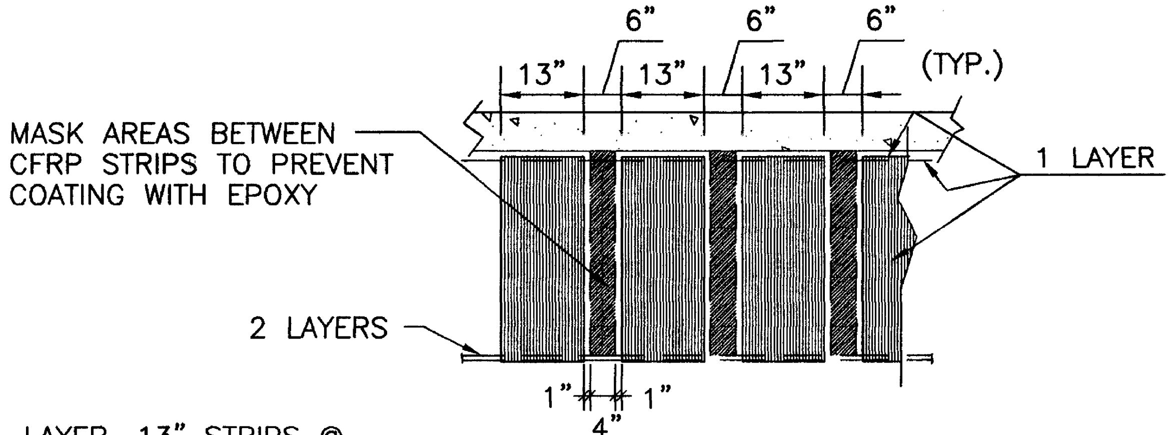
$$A_{Rep} = \frac{40\ ksi \times 0.95\ in^2}{330\ ksi} = 0.115\ in^2$$

Width REPLARK 30 required = A_{Rep} / t_{Rep} Where $t_{Rep} = 0.0066$ in.

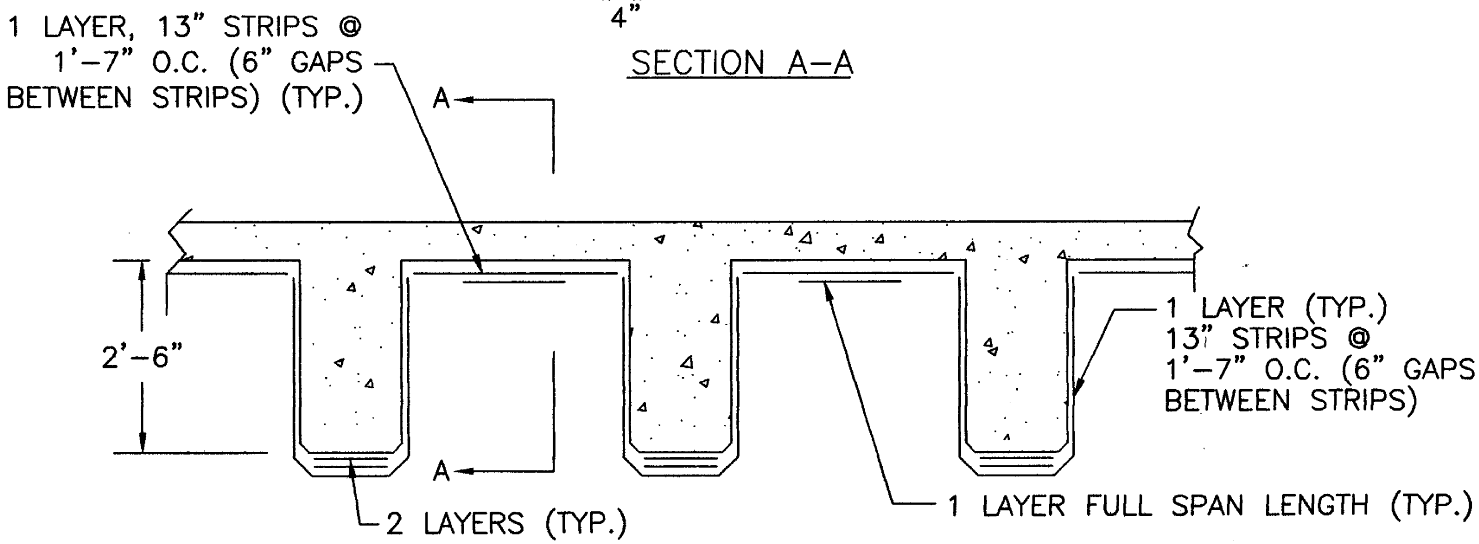
Width REPLARK 30 required = $0.115\ in^2 / 0.0066$ in. = 17.4 in.

Number of Sheets required = 17.4 in. / 13 in. per sheet = 1.3 sheets

∴ Use 2 layer REPLARK 30



SECTION A-A



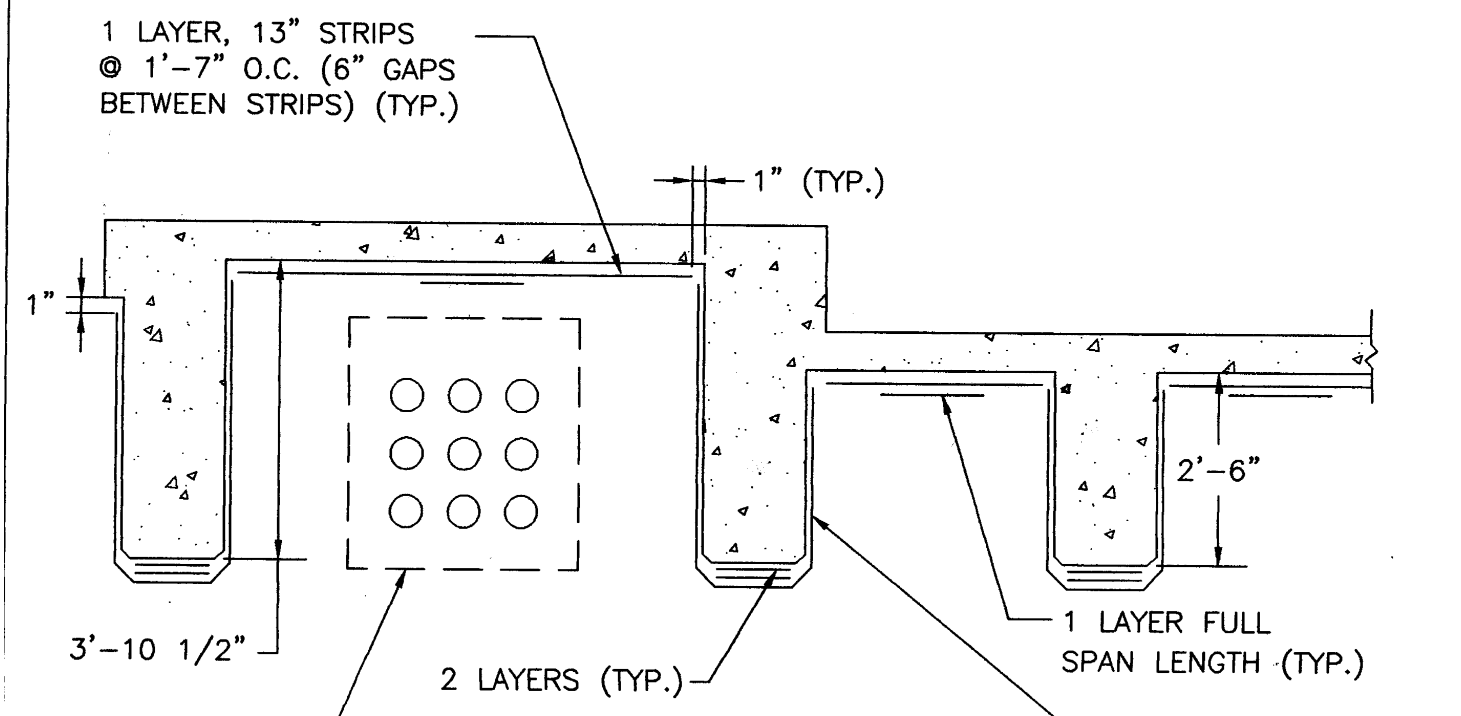
TYPICAL INTERIOR BEAM

AMMANN & WHITNEY

Ammann & Whitney Consulting Engineers
225 Friend Street
Boston, MA 02114

SCALE 1/2" = 1'-0"

NYSDOT – ROUTE 378/
WYNANTS KILL
BIN No. 1-00016-0



TYPICAL EXTERIOR BEAM

AMMANN & WHITNEY

Ammann & Whitney Consulting Engineers
225 Friend Street
Boston, MA 02114

SCALE 1/2" = 1'-0"

NYSDOT – ROUTE 378/
WYNANTS KILL
BIN No. 1-00016-0

Ammann & Whitney (MA) P.C. - Boston, MA

By: A. Martecchini . **Project**: NYSDOT -Route 378 / Wynants Kill
Date: 7/17/99 BIN No. 1-00016-0

QUANTITY ESTIMATE OF REPLARK 30

Bottom of Tee Beams

Length of Beam is 38' - 6" Use 4 strips 10.13' long which includes 3 - 8" overlaps

L_{total} = 25 beams x 2 layer x (4 x 10.13') = 2,026 ft.

Beam Shear Strips

For 2' - 6" Typical Interior Beams (21 beams of this type):

L / strip = $(2 \times 2.5'_{sides}) + (1 \times 1.5'_{bottom})$ = 6.5'

Beam Length is 38.5' Strips are 13" wide and spaced at 6" between strips

No. Strips / beam = $\dfrac{38.5'}{(13" + 6") / 12"/'}$ = 24 strips

Use one additional strip at each end to make 26 strips

L_{total} = 21 beams x 1 layer x 26 strips x 6.5' = 3,549 ft.

Second Interior Beams (2 beams of this type):

L / strip = $3.875'_{side} + 2.5'_{side} + 1.5'_{bottom}$ = 7.875'

L_{total} = 2 beams x 1 layer x 26 strips x 7.875' = 410 ft.

Fasica Beams (2 beams of this type):

L / strip = $3.875'_{side} + 3.375'_{side} + 1.5'_{bottom}$ = 8.75'

L_{total} = 2 beams x 1 layer x 26 strips x 8.75' = 455 ft.

Ammann & Whitney (MA) P.C. - Boston, MA

By:	A. Martecchini	**Project**:	NYSDOT -Route 378 / Wynants Kill
Date:	7/17/99		BIN No. 1-00016-0

Underdeck Strips

Transverse:
Length between beams = (2 x 6.25') + (2 x 3.25') + (20 x 3.0') = 79'
Total No. Strips = 26 (Line up with shear strips)
L_{total} = 26 strips x 1 layer x 79' = 2,054 ft.

Longitudinal:
Length = 38.5' (Same as beam) Use 4 strips 10.13' long
L_{total} = 24 bays x 1 layer x (4 x 10.13') = 973 ft.

Summary for Bridge

Bottom of Tee Beams:		2,026 ft.
Beam Shear Strips:	3,549' + 410' + 455' =	4,414 ft.
Underdeck Strips:	2,054' + 973' =	3,027 ft.
	TOTAL	9,467 ft.
	SAY	**9,500 ft.**

APPENDIX B

MATERIAL PROPERTIES AND APPLICATION PROCESS [26]

1 MATERIAL PROPERTIES OF REPLARK™ SYSTEM

The REPLARK™ SYSTEM consists of the carbon fiber sheet and three types of EPOTHERM materials, primer, putty and resin, all manufactured exclusively by Mitsubishi Chemical Corporation, as shown in the picture below.

1.1 CARBON FIBER SHEET

The table below summarizes the properties of the carbon fiber sheet.

TABLE 1.1 CARBON FIBER SHEET PROPERTIES

PRODUCTS		REPLARK 20	REPLARK 30	REPLARK MM [1]	REPLARK HM
Product Number		MRK-M2-20	MRK-M2-30	MRK-M4-30	MRK-M6-30
Fiber Areal Weight	*g/m²*	200	300	300	300
	lb/ft²	0.041	0.061	0.061	0.061
Thickness	*mm*	0.111	0.167	0.165	0.143
	in	0.0044	0.0066	0.0065	0.0056
Tensile Strength	*N/mm²*	3,400	3,400	2,900	1,900
	kgf/cm²	35,000	35,000	30,000	20,000
	psi	493×10^3	493×10^3	421×10^3	276×10^3
Tensile Modulus	*N/mm²*	2.3×10^5	2.3×10^5	3.9×10^5	6.4×10^5
	kgf/cm²	2.35×10^6	2.35×10^6	4.0×10^6	6.5×10^6
	psi	33.4×10^6	33.4×10^6	56.6×10^6	92.8×10^6
Color of Glass Fiber Mesh [2]		White	Black	Brown	Green
Standard Width and Length	*Width* [3]	25 cm (10 in), 33 cm (13 in), 50cm (20 in)			
	Length (Roll) [4]	100 m (328 ft)			

REPLARK™ SYSTEM — MITSUBISHI CHEMICAL CORPORATION

(1) Replark MM is available only in Japan. The most common type used in the US and other parts of the world is Replark 30.
(2) Glass Fiber Mesh is attached to the back of carbon fibers to form sheets of carbon fiber. It does not have any structural function.
(3) Standard size available is 33 cm (13 in). Other sizes are special order items.
(4) Length of 100 m (328 ft) in factory packed rolls is a standard length. Smaller quantities are not usually supplied.

1.2 PRIMER, PUTTY & RESIN

Currently there are two types of PRIMER, one type of PUTTY and two types of RESIN available in the North American market. The table below summarizes the properties of such materials.

TABLE 1.2 PRIMER, PUTTY & RESIN PROPERTIES

PRODUCTS			PRIMER (1)		PUTTY	RESIN	
Product Number			PS301	PS401	L525	L700W	L700S-LS
Good for			Cool Season	Warm Season	All Year	Cool Season	Warm Season
Usable Temperature (Recommended)			41°F - 77°F (5°C - 25°C)	68°F - 95°F (20°C - 35°C)	41°F - 95°F (5°C - 35°C)	41°F - 59°F (5°C - 15°C)	59°F - 95°F (15°C - 35°C)
Solvents			Non-solvent				
Base Resin			Epoxy-based				
Appearance (Two-part system)		Main Agent	Pale Yellow Liquid		White Putty	Green and Thixotropic Liquid	
		Hardener	Brown Liquid		Black Putty	Brown Liquid	
Mix Proportion (by weight)		Main Agent	2		2	2	
		Hardener	1		1	1	
Specific Gravity 77°F (25°C)		Main Agent	1.11	1.11	1.49	1.13	1.13
		Hardener	1.02	0.97	1.44	1.05	0.99
Tensile Strength 73°F (23°C)		N/mm2				over 29.4	
		kgf/cm2				over 300	
		psi				over 4200	
Flexural Strength 73°F (23°C)		N/mm2				over 39.2	
		kgf/cm2				over 400	
		psi				over 5500	
Tensile Shear Strength 73°F (23°C)		N/mm2				over 9.8	
		kgf/cm2				over 100	
		psi				over 1400	
Adhesive Strength 73°F (23°C)	steel	N/mm2	over 4.9			over 4.9	
	steel	kgf/cm2	over 50			over 50	
	steel	psi	over 700			over 70	
	concrete	N/mm2	over 1.5		over 1.5	over 1.5	
	concrete	kgf/cm2	over 15		over 15	over 15	
	concrete	psi	over 200		over 200	over 200	
Compressive Strength 73°F (23°C)		N/mm2			over 49		
		kgf/cm2			over 500		
		psi			over 7000		
Viscosity (mPa·S)		86°F (30°C)		200			2,400
		73°F (23°C)	500	500		3,500	2,700
		50°F (10°C)	1700			7,800	
Standard Usable Quantity		kg/m²	0.25		0.50(3)	0.60 - 0.80 (Undercoat 0.40 - 0.50, Overcoat 0.20 - 0.30)	
		lb/ft²	0.05		0.10(3)	0.12 - 0.16 (Undercoat 0.08 - 0.10, Overcoat 0.04 - 0.06)	
Pot Life(2) (minutes)		86°F (30°C)	25	140	30		85
		73°F (23°C)	40	240	50	20	120
		50°F (10°C)	95		100	70	
		41°F (5°C)				100	
Tack-free Time (hours)		86°F (30°C)		4.0	3.0		6.0
		73°F (23°C)	3.5	7.0	3.5	3.5	7.0
		50°F (10°C)	6.5		5.0	6.0	
Curing Time (days)		86°F (30°C)					5
		73°F (23°C)				7	7
		50°F (10°C)				14	
Standard Kit		kg/set	12.0		15.0	12.0	
		lb/set	26.5		33.0	26.5	

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(1) Primer type XPS 511C for extremely cold temperature is available on special order basis.
(2) Pot life is determined by heat generation rising method.
(3) Quantity of putty will depend on the surface condition of the structure.

2.2 APPLICATION PROCESS FLOW OF REPLARK SYSTEM

TABLE 2.2 APPLICATION PROCESS FLOW CHART

Step	Notes
SURFACE PREPARATION	▸ Remove all deteriorated concrete layers. ▸ Rust proof corroded steel reinforcing bars. ▸ Inject resin into cracks. ▸ Mortar repair of defective parts.
▼ PRIMER MIX	▸ Mix correct amount of primer (main agent & hardener). ▸ Consider pot-life of mix and application time factor when determining amount of mix.
▼ **PRIMER APPLICATION(1)**	
▼ PRIMER TACK-FREE TIME	▸ Tack-free time varies depending on temperature. Primer becomes tack-free in 3.5 to 7 hours or overnight.
▼ PUTTY MIX	▸ Mix correct amount of putty (main agent & hardener). ▸ Consider pot-life of mix and application time factor when determining amount of mix.
▼ **PUTTY APPLICATION(1)**	▸ Putty is required when cracks and holes are noticed after application of primer.
▼ PUTTY TACK-FREE TIME	▸ Tack-free time varies depending on temperature. Putty becomes tack-free in 3 -5 hours.
▼ CARBON FIBER SHEETS CUTTING	▸ Carbon fiber sheets can be cut at the job site or pre-cut at anytime. The cut sheets must be handled with care and in accordance with manufacturer's recommendation.
▼ RESIN MIX	▸ Mix correct amount of resin (main agent & hardener). ▸ Consider pot-life of mix and application time factor when determining amount of mix.
▼ **RESIN UNDERCOAT APPLICATION(1)**	
▼ **CARBON FIBER SHEET APPLICATION(1)**	▸ Carbon fiber sheets should be applied immediately after application of undercoat resin.
▼ RESIN IMPREGNATION TIME	▸ Allow time for resin impregnation after applying carbon fiber sheet and before applying overcoat resin. Time required is approximately 30 minutes.
▼ **RESIN OVERCOAT APPLICATION(1)**	▸ For multiple-layer carbon fiber sheets application, the overcoat resin also serves as the undercoat resin for the next sheet of carbon fiber; therefore, the quantity applied is about two times that of a single or final layer of carbon fiber sheet.
▼ RESINCURING TIME	▸ Curing time varies depending on temperature and type of resin applied. Refer to the material properties data for curing time. In general the application is ready for finish coat the next day.
▼ FINISH COAT	▸ Type of finish and paint varies and need to be compatible with epoxy surface. Finish coat is recommended to protect the final layer of resin and to prevent discoloration.

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(1) The five steps in black boxes indicate the main process of the Replark System application procedure. This is reduced to four main steps when putty is not used.