IDEA

Innovations Deserving Exploratory Analysis Programs

Highway Program

The Rehabilitation of Steel Bridge Girders Using Advanced Composite Materials

Final Report for NCHRP-IDEA Project 51

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EXECUTIVE SUMMARY

The rehabilitation of steel bridges using advanced composite materials offers many advantages to bridge owners who are looking for feasible and cost-effective solutions for an increasing number of deficient bridges. A previous IDEA (Type 1) effort investigated the use of advanced composite materials to strengthen and stiffen steel bridges (NCHRP-93-ID011). This project resulted in the development of a selection process for adhesives that demonstrate durability under a variety of anticipated field conditions and the demonstration of repair schemes on steel girders taken out of service in Pennsylvania due to excessive corrosion. The rehabilitated beams were tested in the laboratory under service-load conditions. The tests showed that significant increases in strength and stiffness were realized with the composite repair.

The Phase 2 effort described in this report focused on applying advanced composites to steel-bridge retrofitting in the field. The major issues investigated were fatigue resistance and environmental durability. An existing steel bridge girder on Delaware Bridge 1-704 was rehabilitated using carbon-fiber-reinforced polymer (CFRP) plates. It was shown that a small crew could quickly and easily perform the rehabilitation without the need for special tools, experience, or training. Load tests performed prior to and after the rehabilitation indicate a reduction in tension flange strains of 11%. Monitoring will continue for an indefinite period of time to enable the durability of the CFRP/steel bond to be assessed. The research demonstrated that this rehabilitation approach is a feasible and potentially cost-effective repair solution for deteriorated steel bridges.

IDEA PRODUCT

The product developed under this project is a method for rehabilitating structurally deficient steel bridge girders using advanced composite materials. Several laboratory studies conducted at the University of Delaware had previously shown that CFRP plates can be used to effectively strengthen steel bridge girders. This project was aimed at demonstrating the acceptability of this concept for deployment into mainstream bridge retrofit practice through laboratory and field experimentation. The project resulted in demonstration of this technology on a bridge on Interstate 95 in Newark, Delaware (Bridge 1-704).

CONCEPT AND INNOVATION

The Federal Highway Administration (FHWA) had a total of 587,550 bridges in the National Bridge Inventory (NBI) as of September of 2000 (*1*). Of these nearly 600,000 bridges, roughly 15% are classified as structurally deficient. Of the structurally deficient bridges, 56% have steel superstructures. Because of the substantial cost associated with replacing all of these deficient bridges, owners are searching for novel yet viable and cost-effective rehabilitation techniques. The application of advanced composite materials for bridge rehabilitation represents one such innovative solution.

Previous research at the University of Delaware established the effectiveness of bonding CFRP plates to the tension flange of steel bridge girders to increase strength and stiffness. Researchers have also verified the durability of the CFRP/steel bond under varying environmental conditions (*2*) and examined the durability of the CFRP/steel bond subjected to cyclic and sustained loads as well as force transfer issues applicable to CFRP plate joints (*3*). With this groundwork set, the next step was to apply this rehabilitation system to an existing bridge. This report presents the full-scale rehabilitation of an existing steel bridge girder.

Such an effort offers two significant opportunities. First is the opportunity to demonstrate the rehabilitation technique under actual field conditions. The application procedure is presented, highlighting the relative ease and speed with which a minimal crew can complete the procedure, thereby minimizing the traffic disruption associated with bridge repair. A field demonstration of the rehabilitation technique also provides an opportunity to address typical construction issues that arise during the application process and the steps used to deal with them.

Second, full-scale field rehabilitation provides the opportunity to monitor the in-service durability of the retrofit when exposed to a combination of detrimental conditions. The issues of fatigue durability, environmental durability, and force transfer are now imposed concurrently. Long-term monitoring of this system provides important information on the effectiveness and longevity of this rehabilitation system.

INVESTIGATION

INTRODUCTION

The project was performed in two stages. The first stage of involved selecting a steel bridge in collaboration with DelDOT and developing site-specific design, installation, and monitoring requirements for retrofitting in-service girders, followed by performance of laboratory-scale preparatory tests representing site-specific conditions. The next step was establishment of the adequacy of the installation process and the test procedures for in-service testing of the system in the actual steel bridge selected. Finally, the preliminary results were reviewed by a regional expert panel, and, based on this review and the recommendations made, a detailed field testing plan was developed for evaluating in-service performance.

The second stage was field installation and monitoring of the performance and durability of the composite retrofit system under actual highway traffic and environmental conditions, followed by review of the results and preparation of a guidance report for applying the system to steel bridge members. In addition, a long-term monitoring program was put into place in collaboration with the Delaware Department of Transportation (DelDOT).

CANDIDATE BRIDGE DOWNSELECTION

Bridges considered for rehabilitation were chosen from the DelDOT bridge inventory. Several criteria were used in narrowing the inventory to potential candidates. The first selection criterion was the type of steel bridge. Slab-on-girder bridges were selected for consideration in this demonstration. Bridges with high average daily truck traffic (ADTT) would be beneficial with respect to assessing fatigue durability of the CFRP/steel bond. Similarly, exposure to adverse environmental factors such as de-icing agents or moisture from underlying water sources would be useful in verifying the durability of the bond over an extended exposure period. Also considered in the selection of a demonstration bridge was the ease and safety with which the span could be reached for the application and inspection of the CFRP plates.

DEMONSTRATION BRIDGE

The bridge selected for rehabilitation using CFRP plates was Bridge 1-704, which carries southbound I-95 traffic over Christina Creek just outside Newark, Delaware (Figure 1).

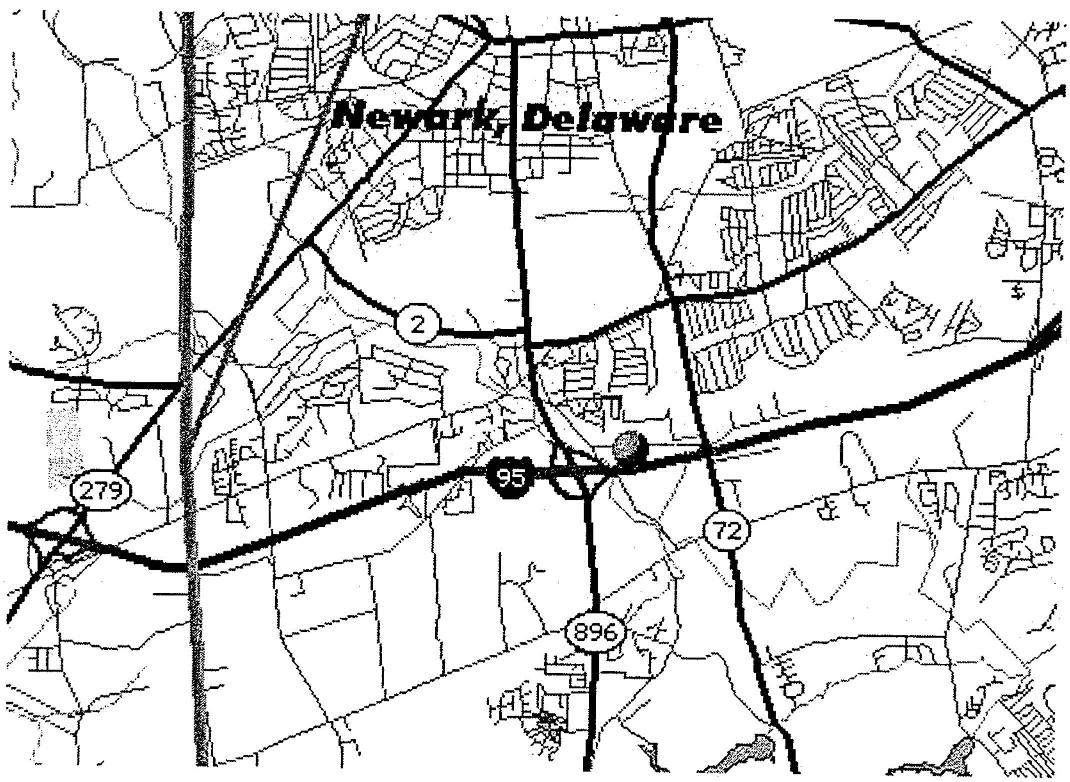


FIGURE 1 Location of CFRP demonstration bridge (1-704).

This specific bridge was selected because it satisfied the criteria discussed in the previous section. Carrying I-95 traffic, the bridge has a reasonably high ADTT (estimated by DelDOT to be approximately 5,920). The bridge also spans Christina Creek, resulting in increased moisture in the area of the bond. The lack of traffic under the bridge provides a relatively safe work environment during the rehabilitation and inspection processes. In addition, the girders of the northern approach span are easily accessible, eliminating the need for extensive scaffolding during both of these operations.

The bridge comprises three simple spans with a total length of 115 ft. and a skew angle of 13 degrees (Figure 2). The main span is 62.5 ft. long with two approach spans measuring 24.5 ft. The original construction on the northern approach span consisted of four W24x84 steel girders spaced at 7 ft. 11 in. Two W36x150 fascia girders at the same spacing were also used. Two separate road widenings added three W36x150 girders to each side of the bridge.

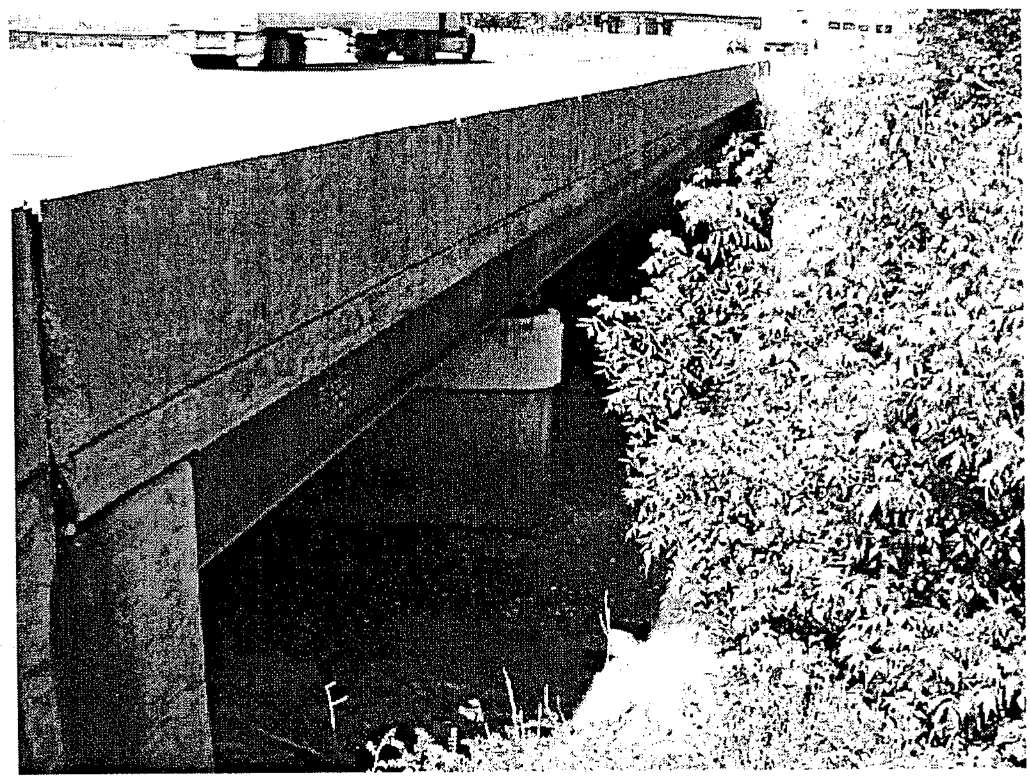


Figure 2 Demonstration bridge (1-704).

GIRDER SELECTION FOR REHABILITATION

One bridge girder was chosen to undergo rehabilitation using the CFRP plates. The use of a single test girder provides an adequate demonstration of the rehabilitation technique as well as sufficient data on long-term durability. In-service peak strains were recorded for four girders under the primary driving lanes to determine which girder is subjected to the largest stress range.

The monitoring system consisted of a strain transducer, data-acquisition system, and power supply. The strain transducer was attached to the bottom flange of the girder being tested. Since the entire system runs off battery power, strain data could be taken continuously without supervision.

Peak strains recorded for several girders for approximately 12–14 hours indicated that girder G5 is subjected to the largest live-load stress range. This is due to the position of girder G5 between the first two driving lanes, which contains the majority of truck traffic. Subsequent 14-day strain-time histories were monitored on girder G5 to determine hourly and daily variations in live-load strains. A trigger threshold of 75 $\mu\varepsilon$ was used for this particular test; therefore, peak strains below 75 $\mu\varepsilon$ are not shown. The extended test indicates typical stress ranges for the heaviest trucks between 2 and 3 ksi, with roughly 75% of the stresses between 2.25 and 2.5 ksi (only stresses above 2.25 ksi were recorded) and an average of 2.4 ksi. If one considered all stresses (those above and below 2.25 ksi), the average stress due to trucks would be less than the 2.4 ksi.

FORCE TRANSFER LENGTH FOR CFRP PLATES BONDED TO STEEL

Background

The use of a single joint or series of joints is advantageous in the practical application of this rehabilitation technique. Short lengths of the CFRP plates allow for easier transportation to the bridge site; but perhaps more importantly, only a limited number of workers are needed to install the shorter plate sections. When considering a joint, as proposed here, force transfer between the steel flange and CFRP plates becomes a primary focus. This work addresses this issue by examining the force development along the CFRP plates through small-scale specimens and an analytical model. The transfer distance becomes particularly important when using staggered joints to connect plate sections. The principle is similar to development lengths considered for reinforcement splices in concrete members. When a single CFRP plate is severed, the load carried in the plate must shed back through the steel substrate and into the adjacent plates. A specific distance is required for the plates to fully develop this additional force and then transfer it back to the steel flange when that strip is severed.

Variations in the amount of adhesive applied during the rehabilitation, clamping pressure, and bonding surface affect the amount and location of adhesive squeezed between plates. Force transferred through the adhesive interface between consecutive or adjacent plates is not considered here.

Test Overview

The purpose of this test program is to analyze the force transfer length for a steel specimen doubly reinforced with CFRP plates. Six 36-in. specimens were placed under tensile loading to determine the rate of force transfer between the steel substrate material and CFRP reinforcement. Eighteen foil strain gages were placed along the length of each specimen to capture the longitudinal strain distribution in the CFRP plates. The test results were then compared with a one-dimensional linear-elastic solution for verification.

Test Specimens

The test specimens consisted of one CFRP plate (.21" x 1.44" x 18") bonded to either side of an A36 steel bar (1/2" x 1.5" x 36"). A glass fabric layer was placed between the steel substrate and the CFRP plates to replicate the full-scale rehabilitation technique. In addition, the ends of the CFRP plates were beveled at a 45-degree angle to reduce adhesive shear and peel stresses. Figure 3 is a schematic of the test specimen, and Figure 4 shows the location of the strain gages.

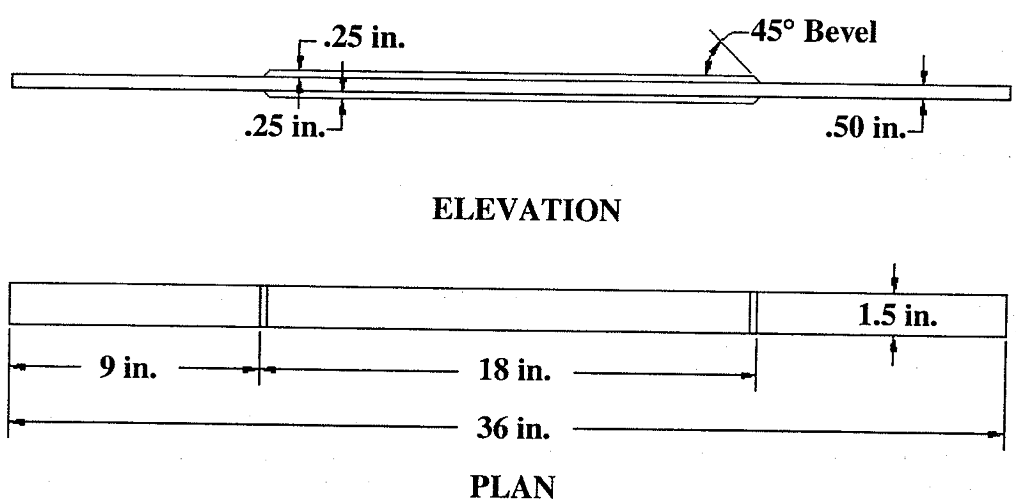


FIGURE 3 Test specimen schematic.

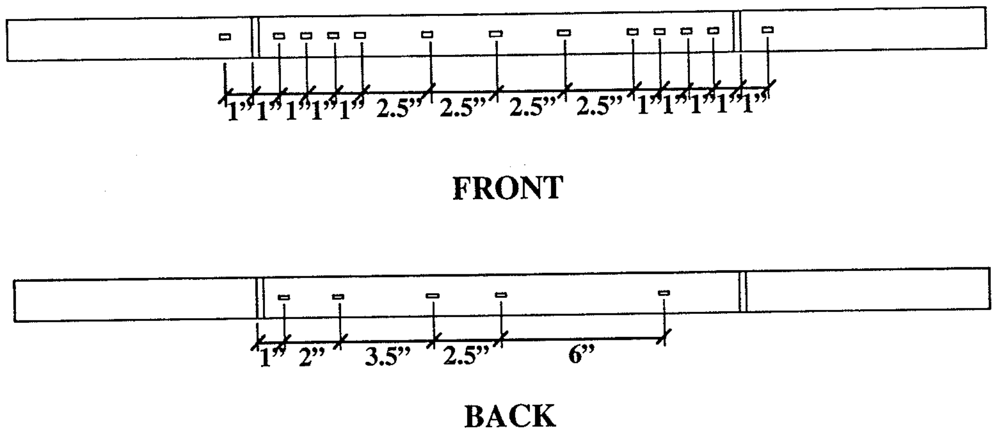


FIGURE 4 Strain gage locations.

Test specimens were divided into two sets. Set one consisted of four specimens using Ciba-Geigy AV8113/HV8113 epoxy. The steel surface was pretreated with Dow Corning Z-6040 Silane adhesion promoter. Measurements indicated that the average adhesive bondline thickness was 0.014 in. Set two comprised three test specimens bonded using ITW Plexus MA555 structural adhesive. Similarly, the steel surface was pretreated with ITW PC120 primer/conditioner. Measurements for the Plexus specimens indicated an average bondline thickness of 0.012 in.

Instrumentation

Eighteen foil strain gages were attached to each test specimen. Eleven strain gages were placed along the length of one side of the CFRP plate to capture the longitudinal strain development along the plate. Five strain gages were placed on the backside of the specimen to act as "backup" gages and for comparison with the primary gage readings. Through these gage readings, specimen bending and primary gage accuracy could be monitored. Two additional strain gages were attached to the steel bar before the initiation of the CFRP plates and were used to determine the actual load applied to the steel bar during testing.

Test Procedure

Each test specimen was placed in an Instron Model 1332 testing machine, where a series of increasing tensile loads was applied and the accompanying strain data was recorded. The loading steps were as follows: zero to 3 kips, zero to 6 kips and zero to 9 kips. A constant strain rate of 3,000 lbs./min. was used throughout the test program. A maximum tensile load of 9 kips was chosen to coincide with the stress range used in the fatigue study for similar specimens. The test specimen was then loaded until the steel bar yielded, at approximately 31 kips. Strain, applied load, and crosshead displacement were recorded at one-second intervals using an Mg5000 data-acquisition system.

Test Results and Discussion

Longitudinal strains captured during static testing, corresponding to 9,000 lbs., were compared to the analytical solution previously discussed (Figures 5 and 6). Tables 1 and 2 provide the values for the equation parameters used in this analysis. The figures represent a single reinforcement plate or one side of the test specimen. The strain data obtained from the opposite side of the specimen verified that both CFRP plates responded similarly during testing. In the case of both adhesives tested, the analytical solution accurately predicted the normal strain distribution occurring along the CFRP plates. Deviations around the centerline of the specimen indicate some degree of bending in the test piece. This is most likely attributed to grip misalignment in the testing machine.

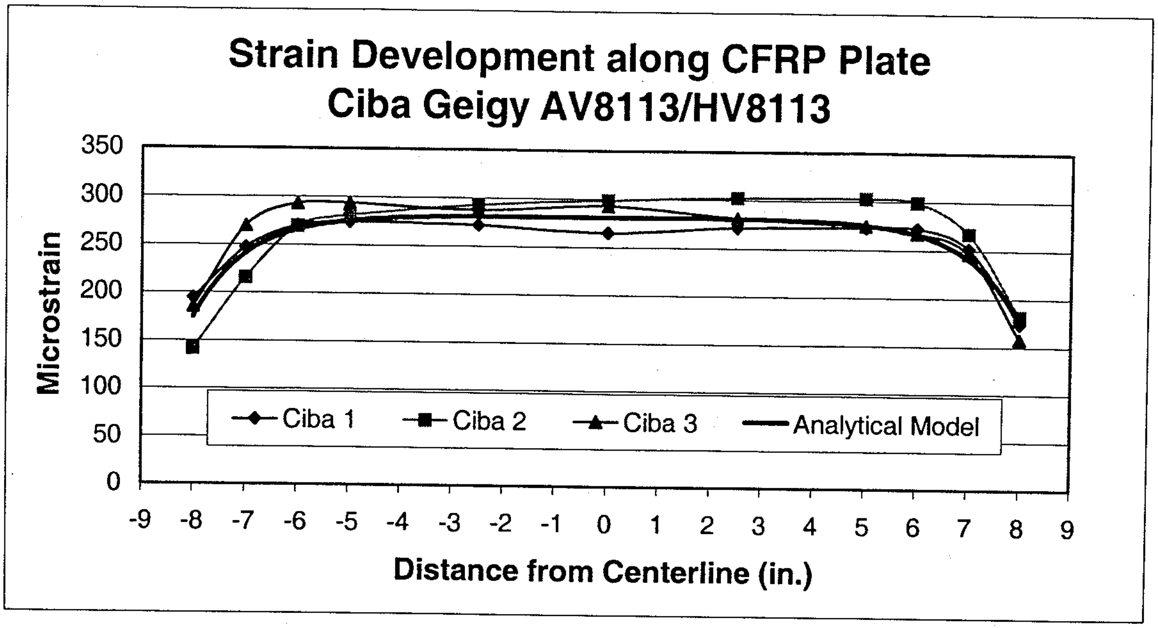


FIGURE 5 Strain development along CFRP plate (Ciba-Geigy AV8113/HV8113).

TABLE 1 Equation Parameters for Figure 5 Analysis

Material	Equation Parameter	Value
Steel	E_s	29,000 ksi.
	t_s	0.5 in.
	G_s	11,154 ksi.
CFRP	E_p	16,300 ksi.
	t_p	0.21 in.
	G_p	650,000 psi.
Adhesive	E_a	15,600 psi.
	t_a	0.014 in.
	G_a	43,000 psi
	P	6,000 lbs./in. of reinforcement

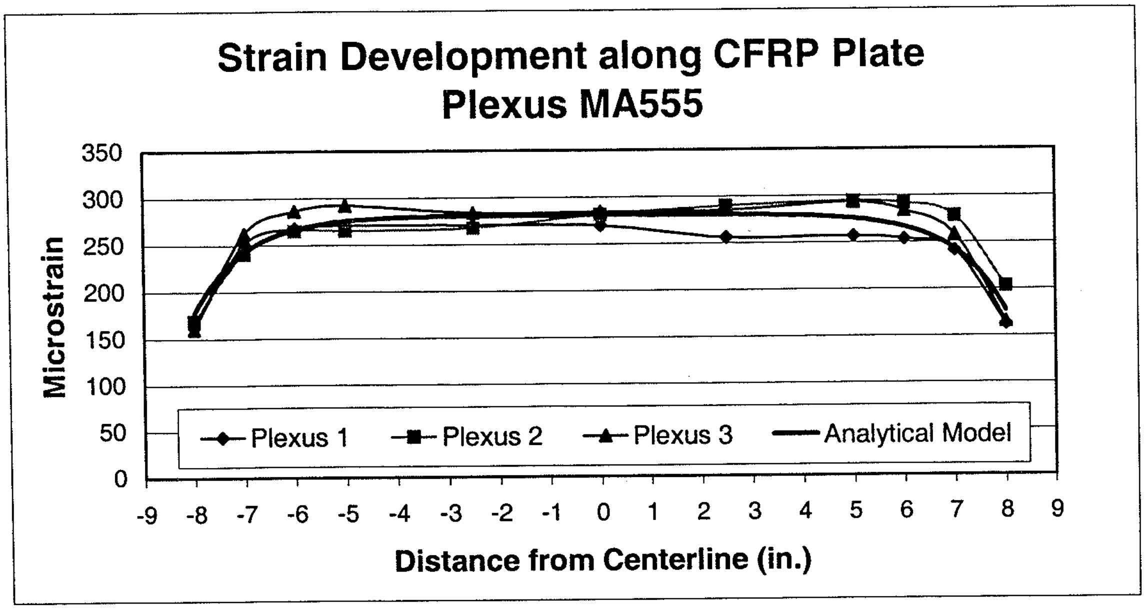


FIGURE 6 Strain development along CFRP plate (Plexus MA555).

TABLE 2 Equation Parameters for Figure 6 Analysis

Material	Equation Parameter	Value
Steel	E_s	29,000 ksi.
	t_s	0.5 in.
	G_s	11,154 ksi.
CFRP	E_p	16,300 ksi.
	t_p	0.21 in.
	G_p	650,000 psi.
Adhesive	E_a	15,600 psi.
	t_a	0.012 in.
	G_a	43,000 psi
	P	6,000 lbs./in. of reinforcement

Through the recorded strain data, the distribution of force along the CFRP plates is obtained (Figures 7 and 8). Once again, for both adhesives, the analytical model closely predicted the maximum force developed in the CFRP plates as well as the rate at which the force is transferred. Approximately 98% and 99% of the total force transfer occurs within the first 3 and 4 in. of reinforcement, respectively.

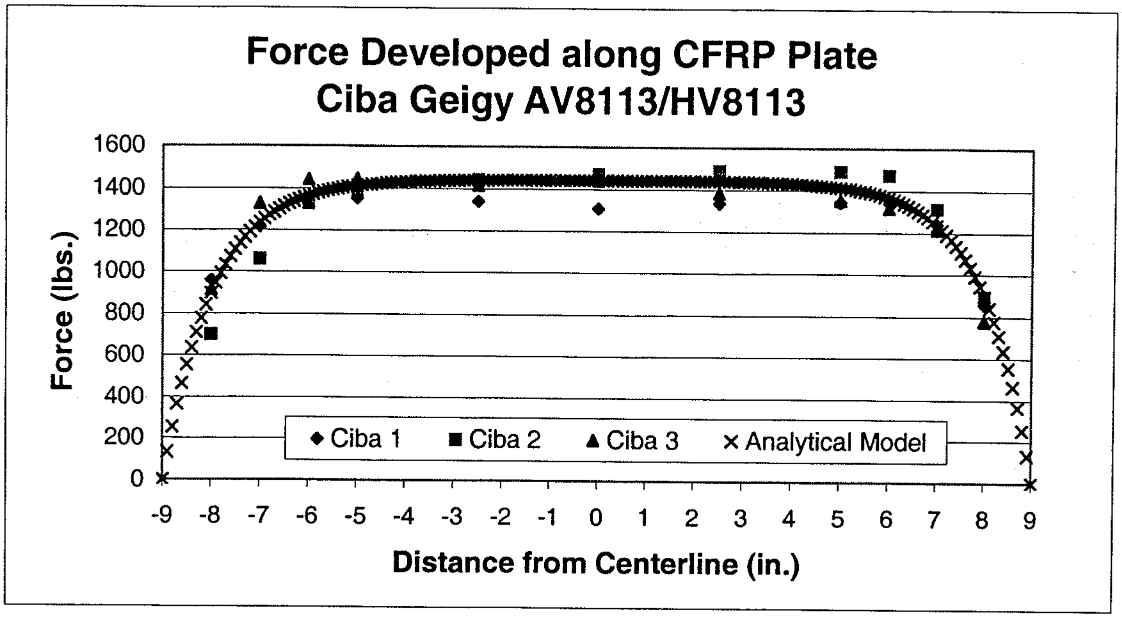


FIGURE 7 Force distribution along CFRP Plate (Ciba-Geigy AV8113/HV8113).

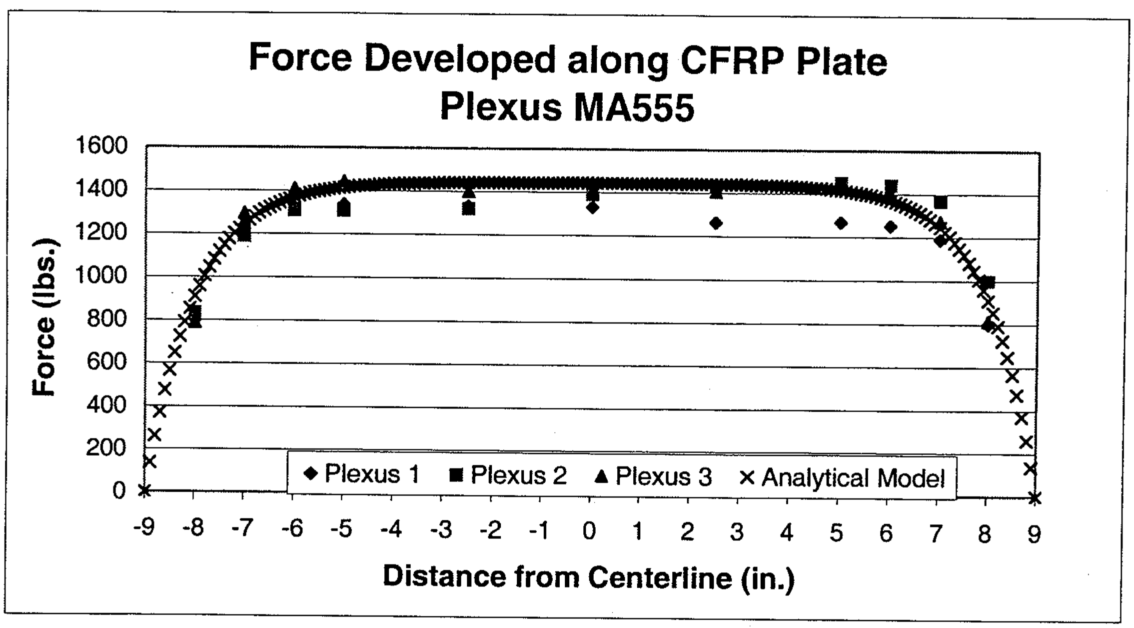


FIGURE 8 Force distribution along CFRP plate (Plexus MA555).

When tested to yield the steel bar, observable debonding occurred in only one test specimen. The edge of the CFRP plate most likely debonded as a result of high shear and peel stresses. This is the expected location for debonding to occur because shear stresses are at their maximum at that location. The debonding occurred on a single Ciba specimen, while no visible debonding occurred in any of the Plexus specimens even though the Plexus adhesive has a lower shear strength. This may be due to the much larger elongation to failure (140%–160%) inherent in the Plexus MA555 adhesive. Lap-shear tests indicated that the Plexus adhesive exhibits cohesive bond failures (*4*). In other words, the adhesive shears considerably when overstressed instead of debonding from the steel substrate.

Additional Issues

As the force in the steel increases toward yield, for specific specimen parameters (i.e., steel substrate and CFRP plate thickness, material properties, etc.), only a certain amount of load will be carried by the CFRP plates. For the specimens tested at 9,000 lbs., the maximum force transferred was 1,443 lbs. Given that each 1.44-in.-wide CFRP plate can carry approximately 41,000 lbs., the material is not fully utilized. While one could utilize more of the plate capacity with a thinner plate, the% utilization would still be far from 100%. The more effective way to obtain a higher%age utilization is to increase the Young's modulus of the plate.

Bond performance at elevated loads is also of concern. Decreases in the force transfer rate at high tensile loads are possible and may be due to excessively high adhesive shear stresses. When the shear stress in the bond reaches the maximum shear strength of the adhesive, plastic behavior results. This non-linear behavior may produce force transfer rates different from those observed at lower loads. To observe the degree to which this occurs, each test piece was loaded until the steel substrate yielded. This loading produced adhesive shear stresses in excess of 2,500 psi., even when considering the beveled edges of the CFRP plates. This shear stress exceeds the shear strength of both adhesives.

Figures 9 and 10 show the force transferred at 25, 50, 75, and 100% of the steel yield load. One can see that the development length for the six test specimens remained approximately the same as the applied force was increased to the yield point of the steel bar.

In Figures 9 and 10, the experimental results are compared to the analytical solution. The analytical model used to analyze the adhesive shear stress and CFRP normal strain distributions was a one-dimensional linear-elastic solution proposed by Albat and Romilly (*5*). The solution includes a correction for shear-lag in the adherends. This is particularly important when considering composite materials as reinforcements due to the generally low shear moduli associated with these materials. It is clear that the analytical model results compare very well with the test results.

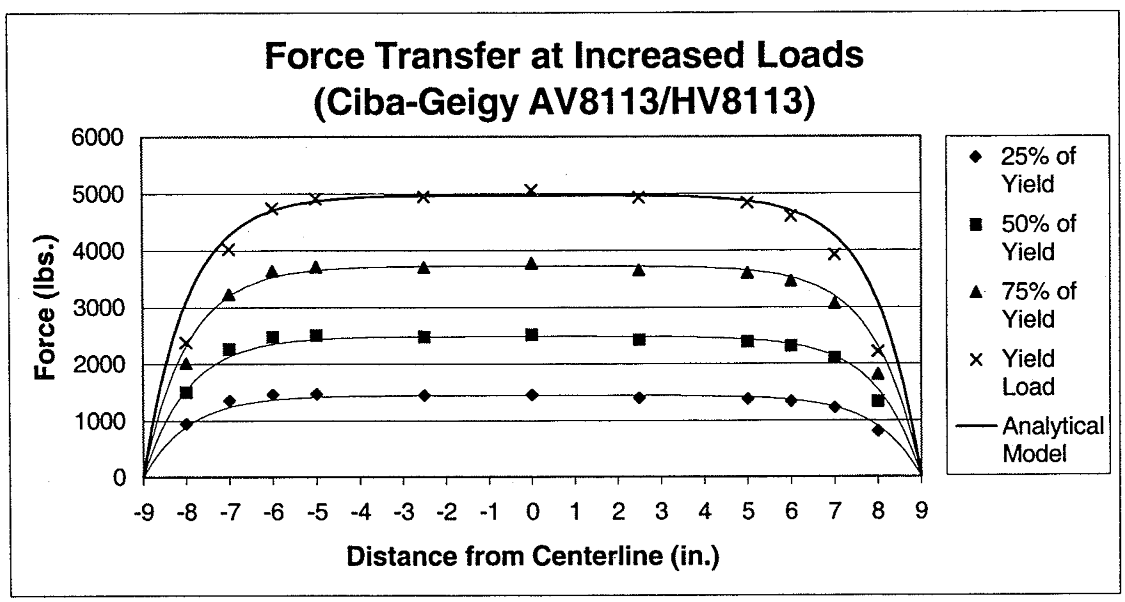


FIGURE 9 Force transfer at high tension loads (Ciba-Geigy AV8113/HV8113).

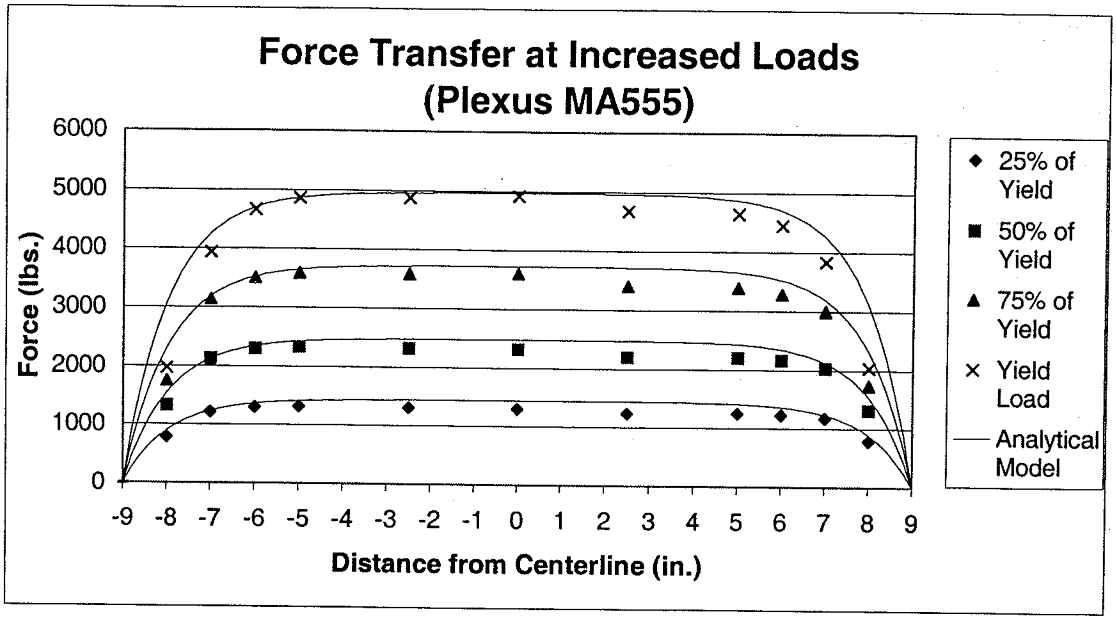


FIGURE 10 Force transfer at high tension loads (Plexus MA555).

Application of Test Data to the 704 Bridge Rehabilitation

After the accuracy of the analytical model was verified through experimental data, the solution was applied to actual field rehabilitation conditions. Complete details of the rehabilitated bridge and CFRP plate application are discussed later in this report. The retrofit contains three staggered joints and makes use of both the Ciba AV8113/HV8113 and Plexus MA555 structural adhesives. Since the rehabilitated girder is a W24X84 (Grade 36) section, a ¾-in flange or steel substrate is used in the analytical model. An in-service monitoring system captured actual strains in the tension flange of the pre-rehabilitated bridge girder resulting from daily truck traffic. Recorded strain-time histories indicated that the girder experiences maximum strains of approximately 131 $\mu\varepsilon$. Therefore, the outer fibers of the tension flange will be subjected to maximum live load stresses of approximately 3.8 ksi.

A bondline of 1/64 in. (0.015625 in.) is also assumed in the analytical model. The test specimens discussed here had a bondline thickness of approximately 0.013 in. Although laboratory conditions generally allow for a thinner bondline, the in-field retrofit in this case will have an adhesive thickness near that of the test specimens. The girder flange has little corrosion and after sand blasting will provide for a uniform bonding surface. The unobstructed upper surface of the tension flange allows for a uniform clamping force, similar to that used in the test specimens, to be applied along the length of the CFRP plates, thus ensuring a thin bondline.

The analytical solution was calculated using the specific parameters of the 704 bridge-girder rehabilitation. While the analytical solution does not exactly reflect the 704-bridge retrofit (i.e., CFRP plates are bonded to only one side of the tension flange), it does provide a reasonable approximation of the force transfer length. The analytical model indicates that a distance of 4 in. is required to develop 97% of the transferred force. While a stagger length of four in. may be acceptable, it is recommended to provide an additional 4 in. to avoid overlapping areas of force interactions on each CFRP strip. Therefore, a minimum distance of 8 in. will be provided to fully develop the force and then transfer the force back to the girder flange prior to the plate termination point. For this field rehabilitation, a stagger distance of 12 in. was selected to provide four CFRP plates sections of equal length. Alternate strips are staggered to reduce the overall joint length. Scarf joints were chosen to join consecutive CFRP plates. Although plate discontinuities are assumed in the force transfer analysis, the bonded end-plate to end-plate interface provides a direct load path from one plate to the next. The amount of force transferred across this interface was not examined, but it is reasonable to assume that a considerable portion of the plate force is transferred across the joint.

Conclusions: Force Transfer Length for CFRP Plates Bonded to Steel

The proposed analytical solution has been shown to accurately predict the strain and force distribution along a CFRP plate bonded to a steel bar for two structural adhesives. For the specific test specimens used, both theoretical and experimental data indicated that, regardless of load levels, approximately 99% of the total load transferred from the steel substrate to the CFRP plate occurs within the first four in. It was also noted that the amount of load transferred to the CFRP plates depends on the material properties and thicknesses. The efficient use of the composite material depends on the proper selection of these variables.

Since the analytical model showed that the distance over which force is transferred between the steel flange and the CFRP plate is especially sensitive to changes in bondline thickness, it is important to ensure a thin bondline during the application process. This may not always be possible in the case of extensive girder corrosion. If an uneven surface is the case, this issue must be accounted for when determining the force transfer distance.

As demonstrated with the test specimens, excessively high adhesive shear stresses cause non-linear behavior and affect the resulting force transfer rate within the first 4 in. Within this region, the results begin to differ from the analytical model. Even though the nature of force transfer changes, the development length remains fairly constant. For field applications using this rehabilitation technique, the maximum bond shear stress should be determined and compared with the shear strength of the adhesive to assess whether non-linear behavior may occur.

Through this test program, the analytical solution was verified as an effective tool in determining force transfer associated with this rehabilitation system. Therefore, the analytical solution was adjusted to reflect the rehabilitation conditions of an existing steel bridge girder. The analysis indicated that a joint stagger of 8 in. is recommended to provide adequate force transfer. A final stagger length of 12 in. was chosen for convenience in this rehabilitation. It is also noted that the scarf joint used for plate-to-plate connections provides additional direct force transfer across the bonded interface, although this load path is not considered in the analysis.

FATIGUE TESTING

The fatigue durability of the CFRP/steel bond was evaluated through two test programs. First, a series of small-scale, double reinforcement specimens was subjected to cyclic loads at a stress range corresponding to the fatigue threshold for common fatigue-sensitive details. A similar test program was then applied and validated using two full-scale rehabilitated bridge girders. The bridge girders were fatigued while the CFRP plates were monitored and inspected for debonding occurring during the test. The full-scale test offers the opportunity to examine several parameters that are representative of an actual rehabilitation installation. First, the bond is subjected to shear stresses representative of in-service live load demands. Second, the girders that were used had a high degree of corrosion. The severely pitted flange surface introduces the opportunity for voids to exist if the adhesive fails to fully spread during application. These existing delaminations may spread when subjected to cyclic loads. Surface irregularities also cause the bondline thickness to vary from location to location, resulting in possible areas of weaker fatigue resistance. These tests enable the CFRP/steel bond to be assessed for its effectiveness and durability as either a short-term repair or a long-term solution.

Small-Scale Fatigue Tests

Test Overview

The purpose of this test program was to demonstrate the fatigue durability of the CFRP/steel bond. Seven double reinforcement specimens were fatigued at a stress range of 12 ksi for 2.55 million cycles. The test specimens were divided into two sets to enable examination of the durability of two structural adhesives.

The stress range and number of loading cycles chosen represents the lower bound for the infinite life of an AASHTO category C' detail (*6*). In other words, details that experience a maximum stress range of 12 ksi or below are considered not to develop fatigue problems. At stress ranges above 12 ksi, the fatigue life of the detail is finite. The category C' detail was chosen for this test program because of the inclusion of these types of details in common bridge designs. Details with lower fatigue resistance are generally avoided when possible. AASHTO category C' details include fillet-welded connections with welds normal to the direction of stress such as the toe of transverse stiffener-to-flange and transverse stiffener-to-web welds (*6*). The test program will not yield an S-N curve for the adhesive, nor will it determine whether the CFRP/steel bond has infinite life. The purpose of testing the CFRP/steel bond at the fatigue threshold for the category C' details is to determine whether the retrofit or the fatigue- sensitive details will be the limiting factor in terms of fatigue life. Since 5% of category C' details are expected to fail at roughly 2.55 million cycles for a stress range of 12 ksi or just above, we aim to show that the bond is at least as durable by testing at these parameters and not observing any failures. If this is shown to be true, one would expect that the bridge will experience fatigue problems prior to the CFRP plates debonding. It is important to note that the rehabilitated bridge would have accumulated some fatigue damage prior to CFRP plate installation. Further, 12 ksi is a relatively high stress range. For most applications, the bond will not be repeatedly subjected to such high adhesive shear stresses associated with this range. These factors must be taken into account when assessing the durability of the bond.

Test Specimens

The test specimens used in this program were identical to the specimens used in the force transfer study. Four specimens using Ciba-Geigy AV8113/HV8113 epoxy comprised the first test set. The second set of three test specimens used Plexus MA555 adhesive.

Instrumentation

Six resistance foil strain gages were attached to the CFRP plates on each test specimen. Three strain gages were attached to each side of the specimen to capture normal strains in the CFRP plates. Figure 11 shows the location of the strain gages on a typical test specimen. Strain data results obtained during static tests throughout fatigue testing were compared to monitor the CFRP/steel bond. Progressive changes in strains during testing could indicate that the CFRP plates were detaching from the steel substrate.

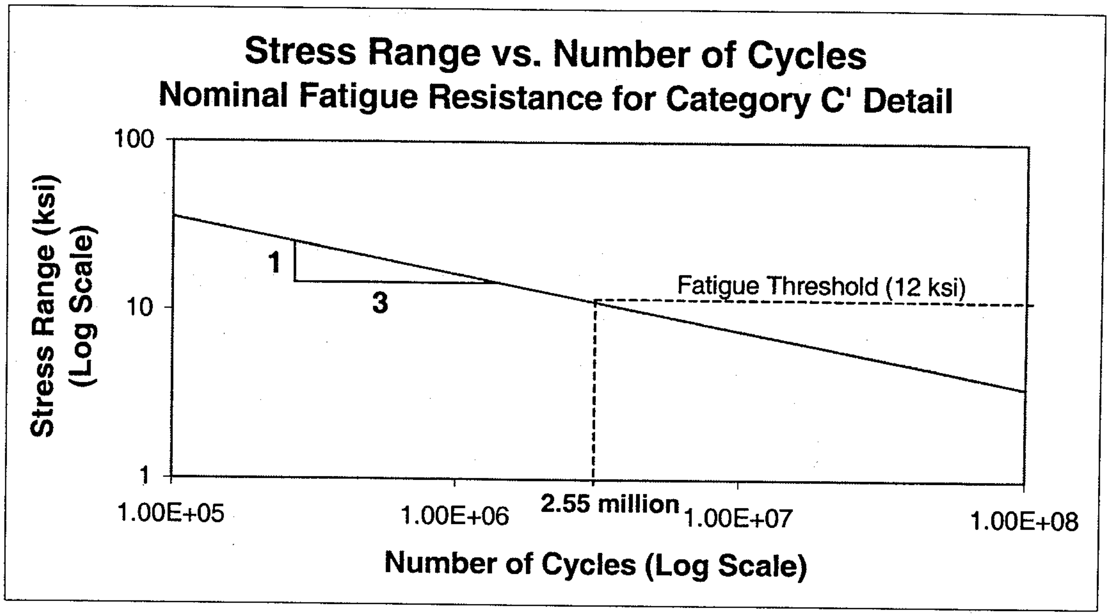


FIGURE 11 Nominal fatigue resistance for AASHTO Fatigue Category C'.

Test Setup

Each test specimen was placed in a 50-kip Instron Model 1332 testing machine. Strain readings, taken during intermittent static tests, were recorded using an MG5000 data acquisition system. Load measurement and crosshead displacements were recorded directly from the Instron test machine.

Test Procedure

Each specimen was subjected to a fatigue stress range of 12 ksi corresponding to a tension load range from 500 lbs. to 9500 lbs. The specimens were fatigued at a frequency of 22 Hz for 2.55 million cycles. Initial and intermittent static tests were performed. Static tests consisted of loading the specimen in tension from zero to 9 kips at a load rate of 3000 lbs./min. Strain and load data were recorded at 1 Hz. In addition, visual inspections of the bond were conducted when fatigue testing was stopped for static testing. Specimens were also tapped lightly with a hammer to detect debonding.

Test Results/Discussion

Strain data recorded during intermittent static tests were compared to pre-fatigue strain data to detect debonding of the CFRP plates. Debonding is expected to occur where the adhesive shear stresses are greatest, i.e., at the ends of the CFRP plates. Strain gages near the ends of the plates would capture a significant decrease in strain as debonding occurs. Force would then be transferred further down the length of the specimen or under large enough loads the bond would continue to progressively detach or "unzip" during the fatigue testing. The beveled edges of the CFRP plates aided in reducing adhesive shear and peel stresses at the ends of the plate. Comparison of pre- and post-fatigue data indicates no sign of debonding of the CFRP plates in either adhesive test group. Figures 12 and 13 show representative strain data for the Ciba and Plexus test specimens.

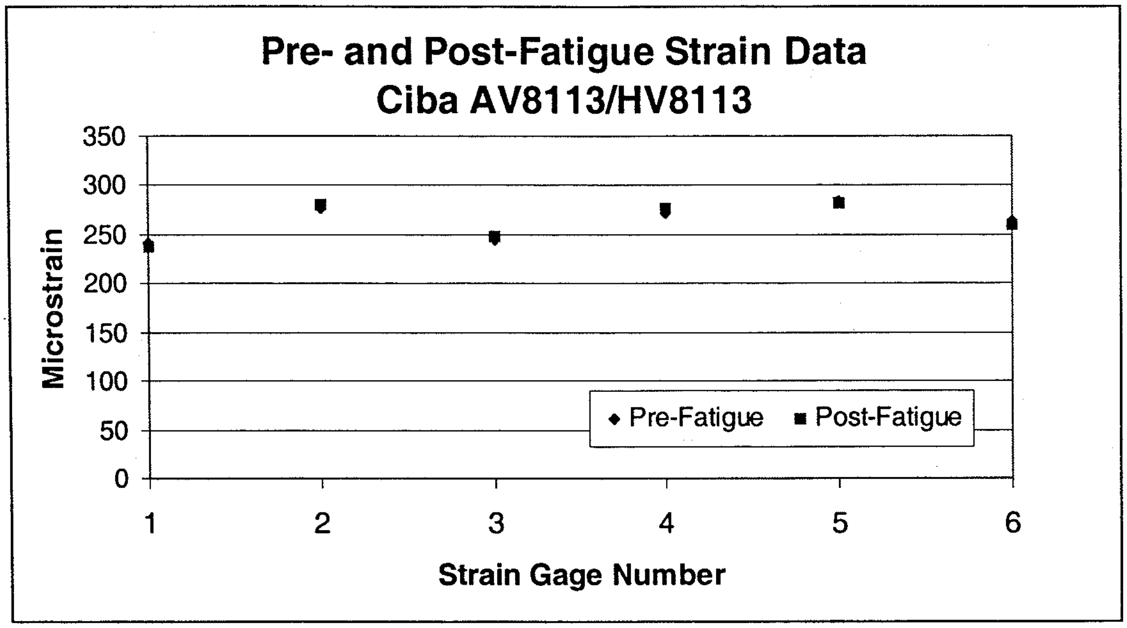


FIGURE 12 Pre- and post-fatigue strain data (Ciba-Geigy AV8113/HV8113).

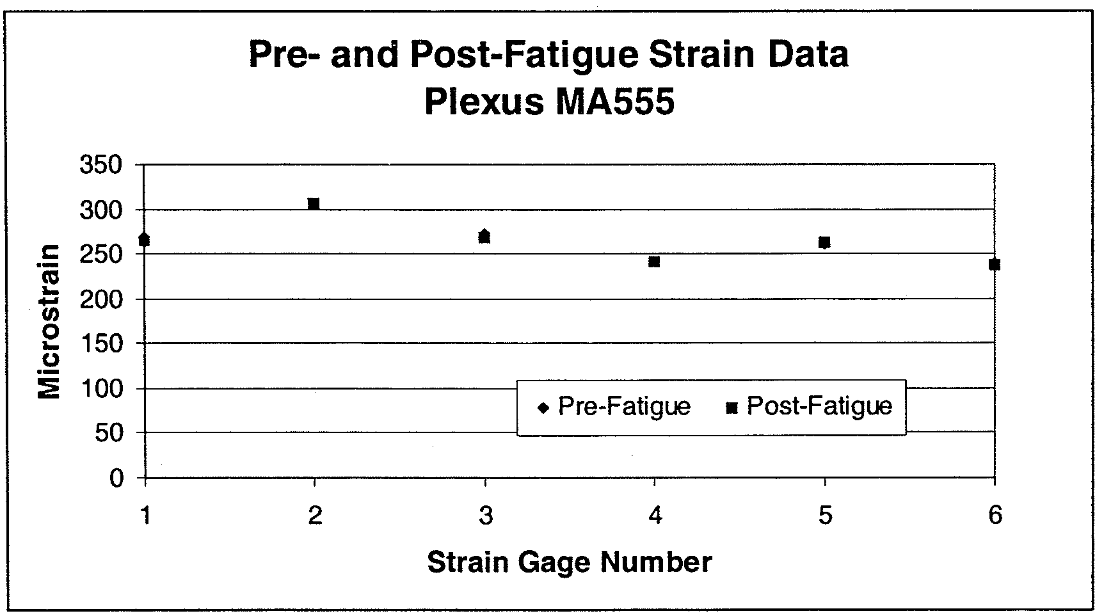


FIGURE 13 Pre- and post-fatigue strain data (Plexus MA555).

Visual inspections along the edges of the CFRP plates showed no sign of bond failure or initiation of debonding. Similarly, tapping the CFRP plates with a hammer indicated no areas of plate detachment. Table 3 provides a summary of the test specimens including maximum% variation in strain data. Decreases in the recorded strain data are relatively small. It must be noted that the first Ciba specimen was released from the test machine grips and reattached during a static test. The adjustment may have altered the initial boundary conditions and have contributed to the 8.4% variation in strain data. Other factors such as background noise and differences in the applied load during gage zeroing are also attributed to the observed variation in recorded strains.

TABLE 3 Summary of Small-Scale Fatigue Specimen Results*

Specimen	Adhesive	Max. % Strain Decrease	Inspection
F1	Ciba-Geigy AV8113/HV8113	8.4%	No visible or audible signs of debonding.
F2	Ciba-Geigy AV8113/HV8113	0.8%	No visible or audible signs of debonding.
F3	Ciba-Geigy AV8113/HV8113	1.2%	No visible or audible signs of debonding.
F4	Ciba-Geigy AV8113/HV8113	3.3%	No visible or audible signs of debonding.
PF1	Plexus MA555	1.6%	No visible or audible signs of debonding.
PF2	Plexus MA555	6.9%	No visible or audible signs of debonding.
PF3	Plexus MA555	1.5%	No visible or audible signs of debonding.

** 2.55 million cycles @ 12 ksi*

Conclusions: Small-Scale Fatigue

After seven specimens were fatigued at a stress range of 12 ksi for 2.55 million cycles, recorded strain data, as well as visual and tapping inspections, indicated that the CFRP plates remained fully bonded to the steel substrate for both the Ciba AV8113/HV8113 and Plexus MA555 structural adhesives. The test parameters represent the lower bound of the fatigue threshold for an AASHTO category C' detail commonly included in the current bridge designs. These results suggest that, in practical application of this rehabilitation system, the CFRP/steel bond is likely to perform better than category C' details and will not be the limiting detail in terms of fatigue durability. If one assumes that the CFRP/steel detail has the same 3-to-1 S-N curve when plotted on a log-log plot, the test results indicate that the CFRP/steel detail has a curve outside of that of a C' detail. As a result, a bridge subjected to a maximum stress range greater than 12 ksi will be limited by fatigue problems in the category C' details before the CFRP plates debond. Although the fatigue resistance or S-N curve for the CFRP/steel bond was not determined, the bond needs to be only as strong as the limiting fatigue detail in the existing bridge.

Also note that throughout the test program and this discussion, the assumption of a virgin bridge superstructure was made. In other words, both the adhesive bond and the fatigue-sensitive details accumulate the same number of fatigue cycles. This is the worst case possible when the fatigue durability of the CFRP/steel bond is considered. In reality, again assuming the maximum stress range is above 12 ksi, the CFRP plates will be applied as a retrofit to a bridge that has likely received a significant number of fatigue cycles prior to rehabilitation. Therefore, fewer fatigue cycles will be needed to initiate fatigue cracks in the mentioned bridge details, and the probability of the CFRP plates debonding prior to this occurring is further reduced. Similarly, the adhesive shear stress developed during this test program should be considered. The maximum shear stress, which occurred at the ends of the CFRP plates, reached approximately 1000 psi. This stress is approximately 40% and 67% of the ultimate shear strength of the Ciba AV8113/HV8113 epoxy and Plexus MA555 structural adhesive, respectively. Even under the excessive shear stress, no indications of debonding were found after 2.55 million cycles. Most practical rehabilitation applications will not subject the CFRP/steel bond to anywhere near such a high degree stress. The average adhesive shear stress due to normal traffic is expected to be approximately 157 psi, with a maximum adhesive shear stress of approximately 300 psi.

Large-Scale Fatigue Test

Test Overview

This program consisted of fatigue testing two, 20-ft bridge girders. Both girders were rehabilitated by bonding CFRP plates to the top and bottom of the tension flanges. The test was designed to monitor the bond between the CFRP and the steel girder when subjected to cyclic loads. Both girders were tested in three-point bending at a load that subjected the CFRP/steel bond to an adhesive shear stress of 179 psi for 10 million cycles. Static testing and inspections were performed during the fatigue testing to examine the CFRP plate for debonding. The test program was conducted in the structural laboratory at the University of Delaware.

Test Setup

The two bridge girders used for this test program were placed side-by-side in a load frame on 4-in.-diameter roller supports with a center-to-center spacing of 20 ft. 3 in. (Figure 14).

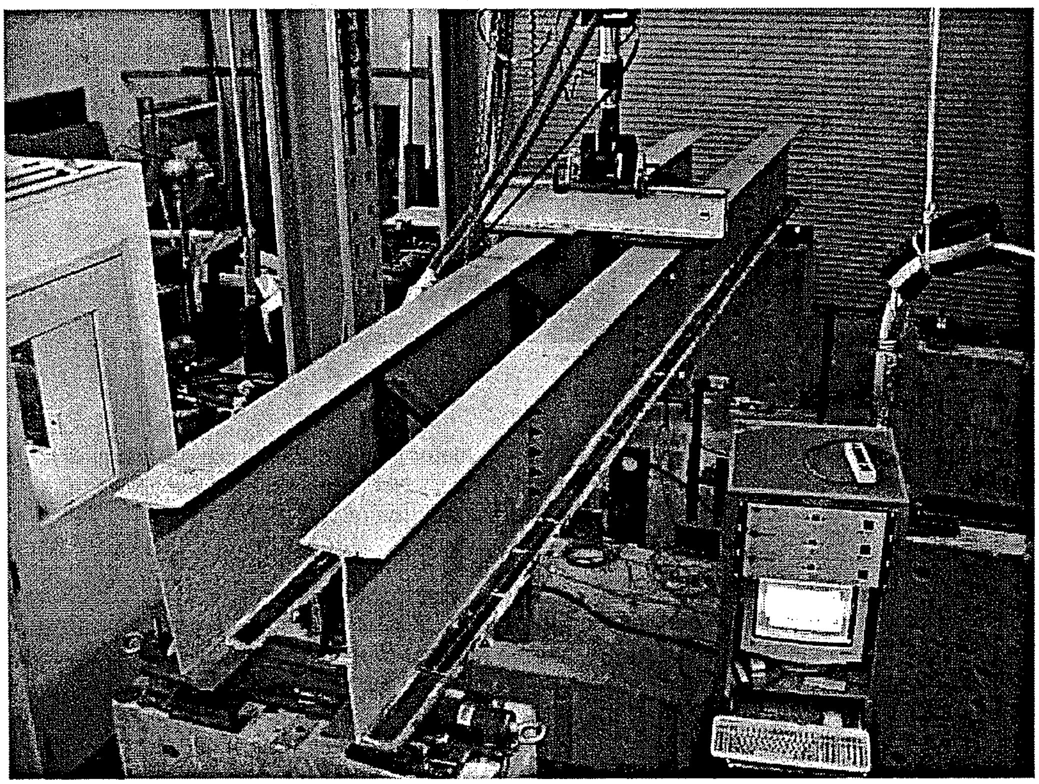


FIGURE 14 Full-scale fatigue test setup.

During previous static testing, the girders exhibited lateral torsional buckling behavior under relatively low loads. Therefore, the girders were connected using cross bracing at four locations (Figure 15). These locations coincided with the position of the original cross bracing. Not only could the girders brace each other during testing, but both girders could be tested simultaneously.

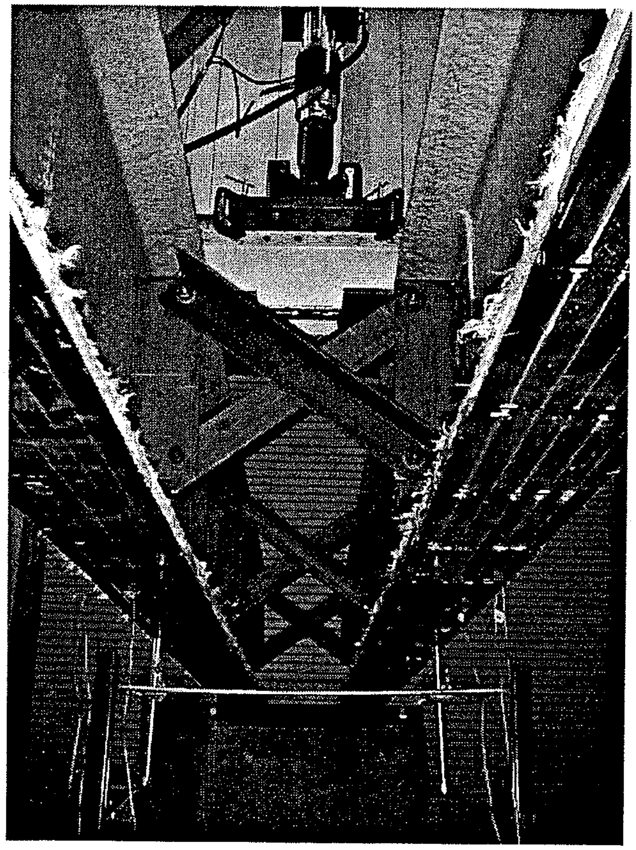


FIGURE 15 Test girder cross bracing.

The load was applied at midspan using a 55-kip hydraulic actuator positioned between the test girders. The load was transferred to each girder through an eight-in.-deep spreader beam. Neoprene load patches (9.25 in. x 12 in.) placed below the spreader beam provided the required AASHTO load distribution of 125 psi (*6*).

Instrumentation

Six resistance foil strain gages were used during static load tests. Three strain gages were attached to each girder at midspan. Two gages measured the strains in the CFRP plates attached to the top and underside of the tension flange of the girders. Two gages were also attached directly to the steel girders on the underside of the compression flange.

Four direct-current differential transformers (DCDT) were also used during static loading. One DCDT was placed under each girder at midspan to measure vertical deflections. An additional DCDT was placed at each end between the girders to measure vertical support movement.

The applied hydraulic actuator load was measured using an MTS system. The load transferred to each girder during static tests was measured using Geokon 100-kip load cells placed beneath the spreader beam. This was done to ensure that both beams were loaded equally.

Test Procedure

An initial static test was performed on the girders prior to fatigue testing. Static tests consisted of loading the girders to 40,000 lbs. at a rate of 6500 lbs./min. and then unloading at the same rate. Applied load, midspan strains, and deflections were recorded at 1 Hz.

Fatigue testing was performed using a calculated load of 27,600 lbs. (13,800 lbs. per beam). The load would subject the tension flange of both girders to a stress range of 5 ksi and produce an adhesive shear stress of 179 psi. This stress range is more representative of what the CFRP/steel bond may be exposed to in the field. Monitoring of a test girder on Bridge 1-704 indicated that the tension flange experiences a typical stress range of approximately 2 ksi. The analytical solution mentioned earlier predicted a corresponding adhesive shear stress of 157 psi. This would suggest that the fatigue performance of the CFRP/steel bond in this test would be comparable to that expected on the rehabilitated bridge girder.

To maintain constant contact between the hydraulic actuator and the test piece, the applied load was cycled between 5,000 lbs. and 32,600 lbs. at a frequency of 6.0 Hz. The fatigue test was interrupted at 500,000-cycle increments to perform a static load test, tighten the load frame and cross-bracing bolts, and inspect the CFRP plates for debonding. Testing continued until 10 million cycles were accumulated. A final static test was then performed.

CFRP plate inspection consisted of two steps. First, the CFRP plates were visually inspected. Plate boundaries, primarily the ends of the plates, were examined for possible debonding induced by shear and peel stresses. Second, a tapping test was used to locate areas where the CFRP plates may have debonded. The CFRP plates were lightly struck using a small hammer to produce an audible tone. Frequency changes or a "hollow" sound can be used to detect areas of bond delamination much in the same way that concrete bridge decks are tested for deterioration by dragging chains across the deck.

Test Results/Discussion

Comparisons of strain and deflection data, recorded during static testing, were used to monitor bond durability. Stiffness changes, evident through strain changes and increased deflection, would indicate a significant detachment of the CFRP plates. Virtually no change in deflection or strain data was observed during the fatigue test (Figure 16).

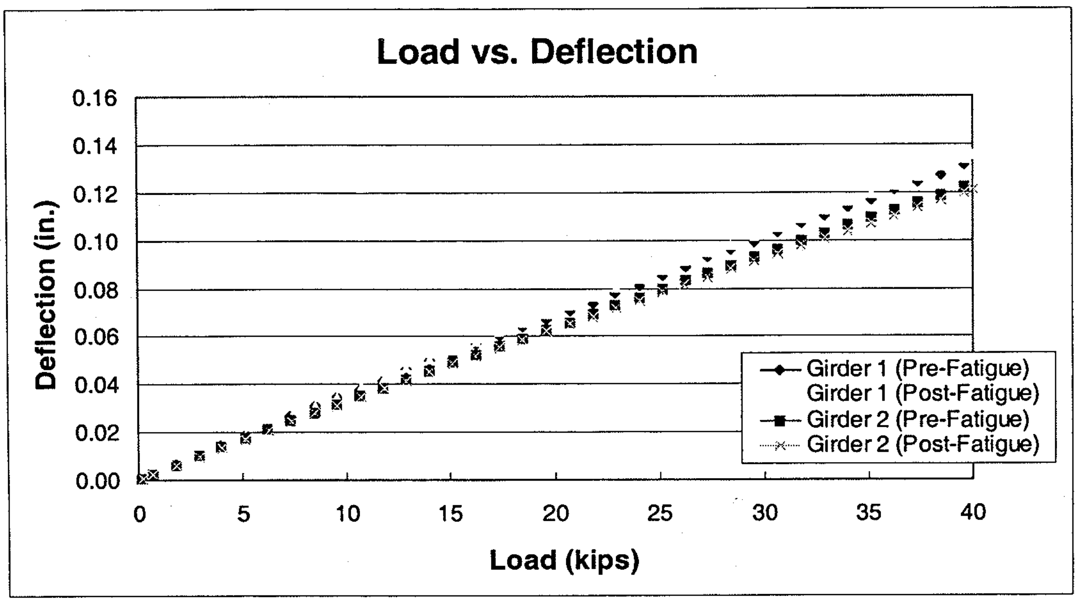


FIGURE 16 Pre- and post-fatigue load vs. deflection data.

CFRP plate inspections were used to detect localized areas of debonding not apparent from an observable change in girder stiffness. Visual inspections of the CFRP plates indicated no sign of CFRP plate debonding. Initial tapping inspections yielded no signs of pre-existing adhesive voids, confirming that the adhesive spread evenly and thoroughly during the CFRP plate installation. This is an important observation because the flange surface was severely pitted and irregular from corrosion. Subsequent inspections also yielded no sign of stress-induced delamination growth occurring during the fatigue test.

Conclusions: Large-Scale Fatigue

The large-scale test program focused on the fatigue durability of the CFRP/steel bond on two rehabilitated bridge girders. The test program provided the opportunity to monitor the CFRP/steel bond under conditions representative of actual in-service demands. Static testing revealed no stiffness changes after 10 million fatigue cycles that would be indicative of CFRP plate detachment. As with common fatigue-sensitive details, the fatigue resistance of the CFRP/steel bond is affected by the fatiguing stress range. For this test program, the adhesive shear stress developed under the fatigue load was approximately 17.9% (179 psi) of that developed in the small-scale fatigue tests. This less severe stress range contributes to the fatigue resistance of the bond.

The flange surface of the bridge girder is also worth discussing. As described before, the bonding surface was severely pitted from corrosion. This irregular surface not only creates a varied bondline thickness but also allows for localized adhesive voids. Both situations have the potential to affect the durability of the bond when subjected to cyclic loads. For the range of stresses exerted on the adhesive during this test program, no noticeable changes in the bond were detected. Inspections revealed that there were no preexisting adhesive voids and that no areas of debonding developed during fatigue testing.

These results would suggest that the relatively low shear stress range, indicative of practical in-service applications, would not produce a condition detrimental to the integrity of the CFRP/steel bond for at least a finite number of cycles (10 million in this research). In addition, the large surface area of attachment aids in redistributing stresses around adhesive voids and areas of increased bondline thickness. Once again, it should be noted that this rehabilitation system would most likely be applied to a bridge near the end of its effective life. The intention is then to use the CFRP retrofit for a short period of time until the bridge is replaced. When considering this period of exposure to cyclic loads, the number of fatiguing cycles is considerably less than if a new bridge were constructed using this system.

Application of Fatigue Test Results to the 704 Bridge Girder Rehabilitation

Results from both the small-scale and large-scale fatigue tests can be used to evaluate the durability of the 704-bridge retrofit in terms of fatigue life. As previously stated, an average adhesive shear stress of 157 psi is expected. The large-scale testing program simulated this condition by testing two rehabilitated girders at an adhesive shear stress range of 179 psi for 10 million cycles. The test results indicate that the CFRP/steel bond of the field rehabilitation will not show signs of debonding prior to 10 million fatigue cycles. With an estimated ADTT of 5920, this translates to a period of approximately 4.6 years.

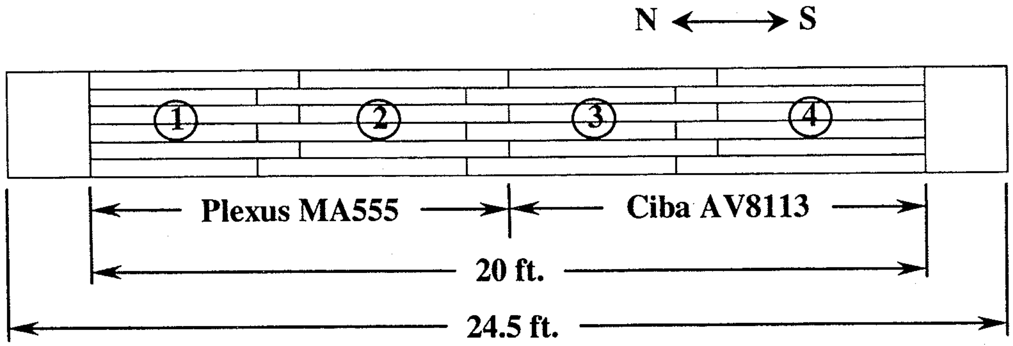
The small-scale fatigue test results may also aid in evaluating the fatigue life of the 704-bridge retrofit. As discussed before, the fatigue specimens were subjected to an adhesive shear stress range over 6 times (1000 psi) that expected on the field retrofit. If one assumes that the lower bound for the fatigue resistance of the CFRP/steel bond follows a 3:1 slope in log-log space through this point, then the fatigue life of the retrofit may be estimated. It is not clear whether this is a valid assumption, since the small-scale fatigue program did not set out to develop an S-N curve for the adhesive. It does, however, provide a starting place for estimating the nominal fatigue resistance. Using this method, the fatigue life of the 704-bridge CFRP/steel bond is approximately 305 years. Although infinite fatigue life was not determined in the test programs, these results indicate that the 704-bridge retrofit should not experience debonding due to fatigue over the life of the bridge.

It should be noted that in the case of both testing programs, no fatigue-induced failures were encountered. Therefore, the estimates of fatigue life were based on the point at which testing was halted. Since the S-N curve for the CFRP/steel bond is not known, it is reasonable to suspect that the actual fatigue life is higher than predicted here.

FULL-SCALE GIRDER REHABILITATION

Rehabilitation Scheme

The full-scale rehabilitation program consists of bonding CFRP plates to the underside of the tension flange of girder G5. The girder is located in the northern approach span of the 1-704 bridge carrying I-95S over Christina Creek outside Newark, Delaware. The retrofit consisted of six 20-ft. CFRP plates placed side-by-side to cover the entire 9-in. wide flange. The CFRP plates are installed in four sections using three staggered joints (Figure 17). Each plate section is 5 ft. length. This allows for easier transportation, adhesive application, and installation. The use of staggered joints also provides a greater opportunity for moisture to reach the underlying bondline. Long-term observation of the joints will provide valuable insight into the effectiveness and durability of joints in this application.



NOTE: Drawing not to scale.

FIGURE 17 CFRP retrofit scheme.

Two structural adhesives, Ciba-Geigy AV8113/HV8113 and Plexus MA555, were used in this rehabilitation project to enable examination of the in-field application and long-term durability of both. Each adhesive is applied to half of the test girder.

Rehabilitation Preparation

Prior to the field rehabilitation, several steps were taken to prepare the retrofit materials. The CFRP plates were cut to the appropriate lengths and preassembled into four plate sections (Figure 18). The individual 1.5-in wide plates were connected using 1-in. x 2-in. wood laths hot-glued to the underside of the CFRP plates at 12-in. increments. Previously rehabilitated girders using clamping points at 12-in. increments showed sufficient adhesive distribution after the clamping force was applied. The CFRP plates could then be manipulated as full-width plate sections.

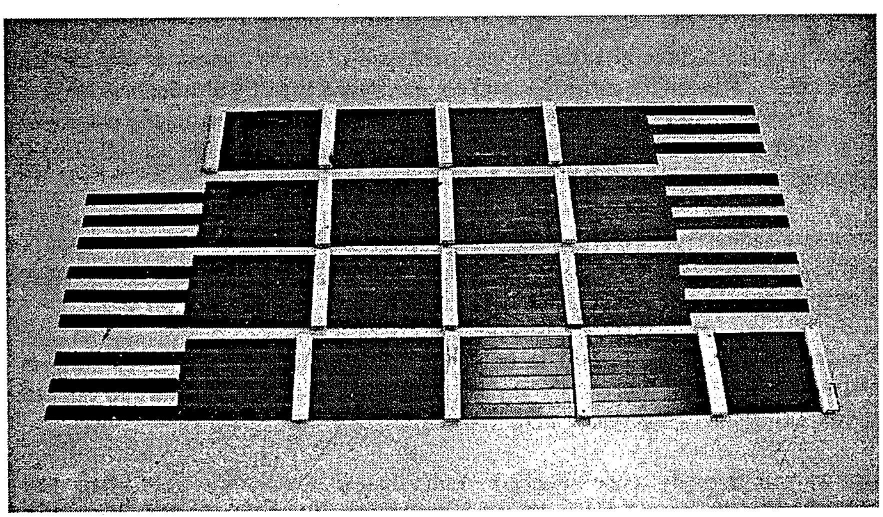


FIGURE 18 Full-width CFRP plate sections.

A 12-in stagger of every other CFRP plate was used for the joint configuration in this application. Consecutive CFRP plates were beveled at a 45-degree angle to form a scarf joint instead of a typical butt joint. This detail provides for increased force transfer through the bonded plate-to-plate interface. Beveled edges are also used at the very ends of the plates to minimize peel stresses.

After the CFRP plate sections were preassembled, the bonding surface of the plates was sanded or "roughed-up" using abrasive scrub pads. The purpose of this procedure is to remove the glossy surface of the plates produced during the pultrusion process. A rough surface aids adhesive bonding.

The glass-fiber fabric used for galvanic insulation was also cut to length. The fabric layer was intentionally oversized to allow for movement during the clamping process. Excess glass fabric could be trimmed after the adhesives cured and the clamps were removed.

The test girder was also prepared for rehabilitation. The surface of the girder was sandblasted to remove all primer, paint, and rust on the underside of the tension flange. Paint removal was performed on the same day as the rehabilitation to prevent the development of rust on the bonding surface.

During the rehabilitation, traffic was closed on the lanes immediately above girder G5 to avoid subjecting the adhesive bond to significant shear stresses until after it was cured. Strain monitoring prior to rehabilitation indicated that the diverted traffic would expose the tension flange of the test girder to maximum strains of 10 $\mu\varepsilon$. The associated shear stresses were not expected to be detrimental to the curing of the adhesive.

Girder Rehabilitation

A temporary platform was constructed underneath girder G5 that provided access to the tension flange during the entire rehabilitation procedure. Since the rehabilitation procedure was done at night, temporary lighting was positioned along the girder and work areas.

The girder was then divided into two sections, one half for the Ciba AV8113 epoxy (southern end) and the other for the Plexus MA555 adhesive (northern end). The steel surface was pretreated accordingly using Silane (Ciba AV8113) and PC120 primer/conditioner (Plexus MA555) adhesion promoters. The pretreatments were applied using foam brushes. The adhesion promoters were left to dry for approximately 15 min. During this time, the CFRP plates were thoroughly cleaned using acetone to remove graphite dust and any other substances from the bonding surface. Once the CFRP plates were cleaned and the steel surface was dry, the four plate sections were placed on the work platform for application of the adhesives.

The next step in the girder rehabilitation was to bond the CFRP plates to the girder. The two components for the Ciba AV8113 epoxy were measured and mixed on the work platform using plastic paddles.

The two-part Plexus MA555 structural adhesive was packaged in single cartridges and applied using a compressed-air gun with a mixing nozzle. The single cartridges and air gun saved time by eliminating the need to measure and mix adhesive components. However, the Plexus cartridges needed to be changed frequently during the application process. Both adhesives were applied to the girder flange surface using plastic paddles. Care was taken to cover the entire surface with a thin layer (approximately 1/16-in).

Similarly, the structural adhesives were applied thinly to the CFRP plates in a layer approximately 1/16-in thick. Again, care was taken to cover the entire surface of the plates with adhesive. The beveled plate edges along the section joints were also coated to ensure a bonded plate-to-plate interface.

After the structural adhesives were applied to both the CFRP plates and the flange surface, the glass fabric was placed on the girder flange. The fabric was evenly pressed into the adhesive to eliminate air pockets. This procedure was important to prevent voids after the CFRP plates were in place.

The CFRP plates were now ready to be attached. Individual plate sections were lifted into position (Figure 19), starting with section number 3. This section was placed first because of the change of adhesives at the centerline. It was important that the specific adhesion promoters applied to the steel surface interacted with the correct adhesive. While the plate sections were held in place, the clamps were attached to the girder flange and wood blocks were hot-glued to the CFRP plates. The same procedure was used for each of the other plate sections, ensuring that the joints were tight and flush before clamping. After all the plate sections were attached, the clamps were firmly tightened to achieve a thin, consistent bondline and allow the adhesives to spread and fill any dry areas (Figure 20). Adhesive squeezed out between consecutive CFRP plates in each of the section joints was noted. This indicates a bonded plate-to-plate interface and load path directly across the joint. Further, adhesive squeezed out along the ends of the retrofit indicates the plates are sufficiently bonded at these locations and prevents penetration of moisture to the bondline, which may initiate plate debonding.

FIGURE 19 Maneuvering CFRP plates into position.

FIGURE 20 CFRP plates fully clamped to girder.

The excess adhesive was cleaned from the CFRP plates after clamping. The clamps were left in place for approximately 8 hours while the adhesives cured. The Plexus adhesive reaches 75% of its full strength in 2 hours at room temperature. The Ciba adhesive reaches handling strength in 4 hours at room temperature. Considering that the temperature during the rehabilitation procedure was approximately 70°F, eight hours of curing time provided enough time for both adhesives to reach sufficient strength for the clamps to be removed and traffic allowed back over the rehabilitated girder. A longer period of time might have been better but due to traffic coordination issues was not possible.

After the eight hours of curing time had expired, the wood blocks used for clamping and assembling the CFRP plate sections were easily removed using a hammer. Excess glass fabric was trimmed using a knife. A final inspection of the CFRP plates was then made. The CFRP plate joints were flush and tight. Plate boundaries were fully bonded and the adhesive had hardened.

LOAD TESTING OF REHABILITATED GIRDER G5

The purpose of bonding the CFRP plates to the tension flange of steel girders is to add additional stiffness and strength. The ability to accurately determine these increases allows an engineer to design the retrofit for each particular application. To demonstrate the effect of the retrofit, diagnostic load tests were performed before and after the application of the plates. To evaluate the accuracy of the analysis method, results from the load tests can be used. One can determine the stiffness increase resulting from the CFRP retrofit directly from the load test and then compare the results to predicted values.

As mentioned, two load tests were performed on Bridge 1-704, one before and one after the rehabilitation (Figure 21). The tests monitored strains in girder G5 as a three-axle dump truck was driven along lane 1 with the driver's side wheels on the marked lane line. Strains were recorded on the inside of the tension flange at the midspan of girder G5. In the initial load test, a dump truck filled with sand and weighing 57.26 kips caused a maximum strain of 69.39 $\mu\varepsilon$. After the CFRP plates were installed and the adhesive was fully cured, the second load test was performed. This test was identical to the first except that in this case the truck (having slightly more sand) weighed 64.24 kips. Strains were recorded at the same location on girder G5, as described before. Normalizing the results for truck weight, the maximum strain for the second load test was 61.5 $\mu\varepsilon$.

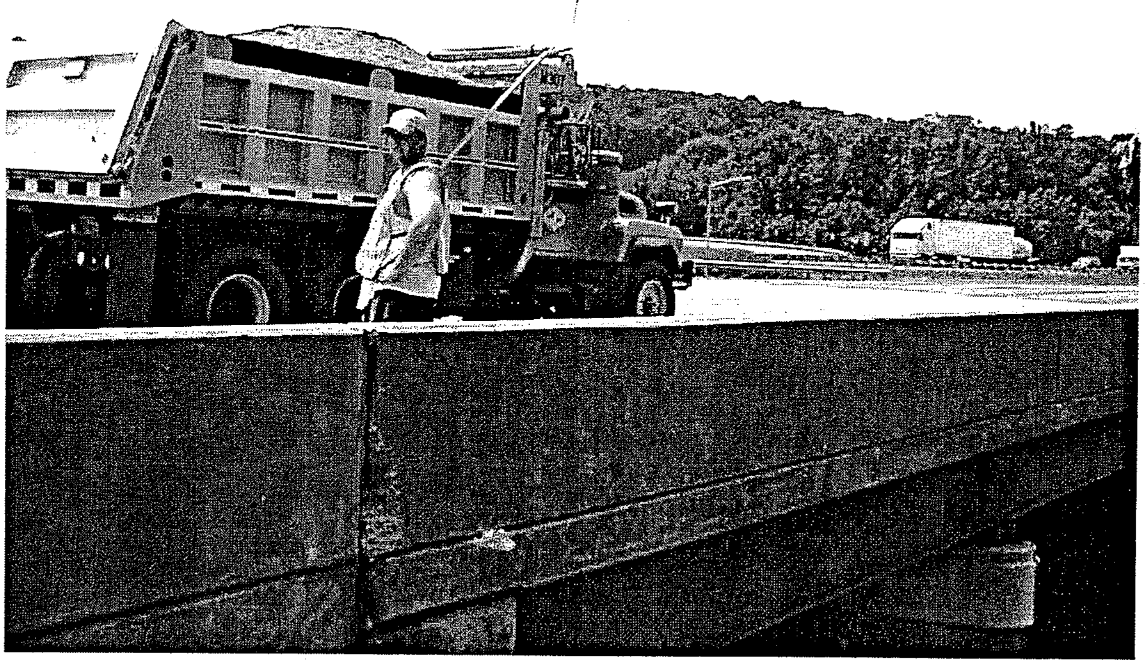


Figure 21 Diagnostic load test of Bridge 1-704.

A comparison of the load test data indicates that the CFRP plates reduce tension flange strains by 11%. Using the method of transformed section, a strain decrease of 10% was calculated (10% difference). It can be seen that the relatively simple analysis method was able to sufficiently predict the decrease in strain provided by the CFRP retrofit. More complex methods of analysis, including laminate theory or finite element models, may be used but are not needed.

CONCLUSIONS

The rehabilitation of an existing steel bridge girder was successfully demonstrated. In general, the steps associated with the process, including preparation, were not difficult to execute and did not require special tools, experience, or training. While transporting equipment to and setting up the work site added to the overall project time, the actual time rehabilitating the bridge girder, from applying the adhesion promoters to finishing cleaning the girders of excess adhesive, lasted approximately one hour and 15 minutes. Seven workers aided in the rehabilitation process.

No significant problems arose during the rehabilitation process. With the CFRP plates cut and pre-assembled, the process progressed rapidly. The only concern associated with the process itself was working quickly enough to get the CFRP plates clamped before the adhesives started to set up. As mentioned earlier, a compressed-air gun was used to mix and apply the Plexus adhesive, while the Ciba epoxy was mixed by hand. Although mixing time was saved, the individual cartridges required frequent changing. While that operation is not difficult, the application of the adhesive is continually interrupted. Another advantage to using the Plexus cartridges is the ability to mix the adhesive as the work progresses, instead of all at one time. This insures fresh adhesive along the entire length of the retrofit. When coupled with the use of plate sections, this also allows a smaller crew to perform the CFRP plate installation. It should be noted that for future bridge rehabilitation procedures, both the Ciba or Plexus adhesives can be applied using a compressed-air system similar to that used here, but drawing from large quantities of the adhesive.

The future application of glass fiber laminae to the CFRP plates to eliminate the need for a glass fabric layer would be beneficial to the rehabilitation process. The step of cutting and applying the glass fabric would be removed, thus saving time. More importantly, the glass fabric must be applied smoothly to avoid creating voids in the bondline. Excluding this step would provide a greater assurance of a consistent bondline, especially when working in the field.

The steel girder rehabilitation was shown to require a relatively short amount of time to complete and was not difficult. A small crew can quickly and effectively apply the CFRP plates to a steel girder bridge. It is important to note that the reduced time and effort is translated into considerable project savings for the bridge owner. The use of jointed plate sections is both applicable and feasible for long span applications. Plate sections are easily transported and allow for smaller construction crews to complete the rehabilitation.

The CFRP retrofit will be monitored for an indefinite period of time to assess the durability of the CFRP/steel bond. Monitoring will consist of regular inspections used to identify areas of debonding along the CFRP plates. Visual inspections of the plate boundaries, including the joints, will be conducted to check for debonding along the plate edges and problems in the joint interfaces. The CFRP plates will also be lightly tapped using a hammer to detect debonded areas not apparent from the visual inspections. Debonded regions sound hollow when tapped. Should debonded areas be discovered, the growth of those areas will be monitored and recorded. The long-term monitoring will provide valuable information on the durability of the two structural adhesives considered in this testing program.

While prior research has indicated that both adhesives are acceptable for this application, the Plexus MA555 is advantageous because of the relatively short cure time and low sensitivity to thick bondlines. A shorter cure time means the bridge may be opened to normal traffic sooner, thereby reducing the inconvenience to the public. The low sensitivity to thick bondlines allows the CFRP plates to be placed over irregular surfaces, such as severely corroded girders, without sacrificing strength. The Plexus MA555 adhesive was not tested in the initial adhesive selection program. Although no problems with the use of this adhesive have been identified, further environmental testing may be warranted.

Load testing was used to verify the predicted decrease in strains (9.9%) caused by the CFRP retrofit. A decrease in tension flange strains of 11.4% was experimentally determined. While the long-term durability of the retrofit is the primary focus of this research, it is worth noting the relative accuracy of the proposed analysis method (transformed sections) for determining stiffness increases due to the CFRP plates.

As mentioned before, the primary focus of this project is the long-term durability of the CFRP retrofit when exposed to actual field conditions including fatigue and the environment. The fatigue test programs used to predict the fatigue life of the CFRP/steel bond on the 704 bridge indicated there should be no signs of fatigue-induced debonding over the life of the bridge. In fact, the data indicated a fatigue life far exceeding the life of the bridge (305 years). What is not clear at this point is whether the durability of the CFRP/steel bond will be affected by the interaction of moisture, temperature, de-icing agents, and fatigue imposed concurrently. Long-term monitoring of the CFRP retrofit should provide valuable insight into the durability of the CFRP/steel bond under these conditions.

PLANS FOR IMPLEMENTATION

This research investigated the use of advanced composite materials for the rehabilitation of steel bridge girders. While the project successfully demonstrated the short-term performance and use of this rehabilitation approach, data on long-term durability are not yet available. The research team will continue working with DelDOT to monitor the in-service behavior of the repair to establish the long-term durability of the procedure. Once the durability of the technique has been established, this technology will be applicable for long-term retrofit of steel bridge girders.

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