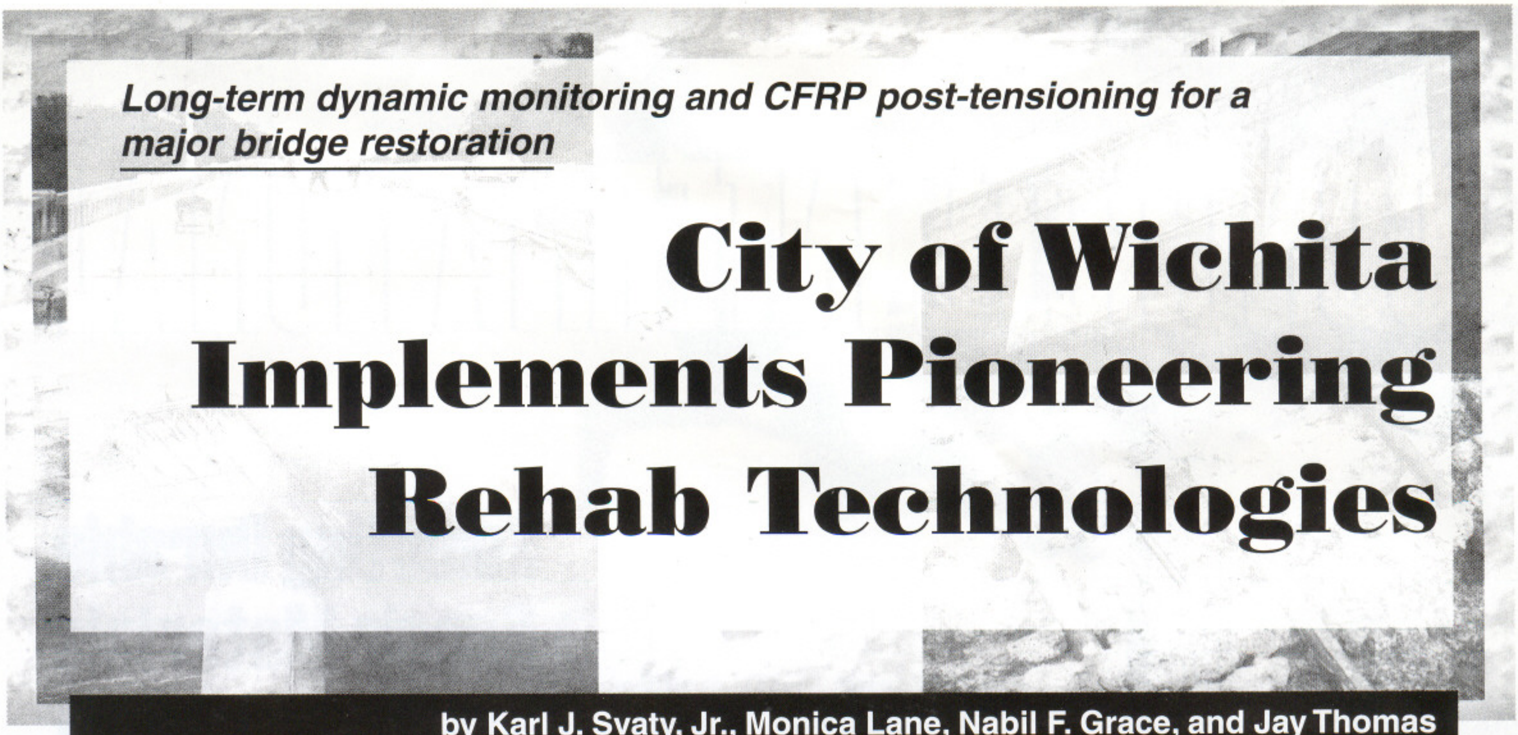

Long-term dynamic monitoring and CFRP post-tensioning for a major bridge restoration

City of Wichita Implements Pioneering Rehab Technologies

by Karl J. Svaty, Jr., Monica Lane, Nabil F. Grace, and Jay Thomas

Planning for transportation needs involves many questions concerning aging bridges and their ability to carry ever-increasing traffic loads. For example, should a particular structure be closed to the detriment of traffic flow and nearby business activities, or is the bridge still in good structural condition? Could aggressive rehabilitation methods prolong the life of the structure, plus keep it open for use during that rehabilitation? The South Broadway Railroad Overpass in Wichita, Kan., is one case where the rehabilitation strategy worked extremely well.

Project description

The structure, built in 1937, was one of the first continuous concrete slab bridges built in the U.S. The 37-span structure is 784 ft (239 m) long, 44 ft (13 m) wide, and carries four lanes of traffic. As of 1998, the structure had a sufficiency rating of four. The Wichita City Council launched several studies to consider whether to replace the structure with a grade crossing, to replace the structure with a new one, or to rehabilitate the existing structure. The recent increase in railway traffic eliminated the grade crossing option. A complete bridge replacement would add \$5 to \$6 million to the project and increase the construction time by at least a year and a half. It would also require the purchase of a very expensive right of way. City officials favored rehabilitation because it would save \$4 to \$5 million over replacement. Moreover, the bridge would remain open, allowing traffic to continue flowing, access for emergency vehicles, and continuity of business activity. Before the final decision was made, however, the engineers performed a number of tests on the structure to determine if rehabilitation was a viable option.

Preliminary testing

Core and hydrodemolition tests

Core samples were taken from the bridge deck for extensive testing. Initially, easily broken samples with a number of horizontal fracture surfaces indicated that the superstructure and substructure were potentially in poor condition. It was later discerned that these fractures were actually cold joints from the original casting 63 years ago. Furthermore, a hydrodemolition test on a trial area indicated that the concrete was still in excellent condition. Thus, the bridge owner and engineers were further encouraged to consider the repair and strengthening approach.

Electronic monitoring

To further verify the structural condition of the bridge, a continuous dynamic electronic monitoring system was attached to the bridge. The system consisted of an onboard computer and five load transducers that were placed on what were considered to be the five worst spans. The system sampled and saved data from the dynamic action of the structure at a rate of 200 Hz every 10 s. Every 15 min for 18 months, the on-board computer was remotely downloaded via radio transmission to the central offices of the engineers. The data suggested that the bridge was in good structural condition and a prime candidate for rehabilitation, based on the findings that follow.

The structure demonstrated an excellent distribution of loading, including responses obtained when vehicles exceeded the posted 10-ton loading restriction. The load transducers were initially calibrated using known vehicle weights, and the maximum loads were continuously recorded. Figure 1 shows the recorded response of a 35-ton concrete transit mix truck that crossed the bridge. The time function of the strain shows the distribution of the load over the span as the truck crossed and its return to normal after the truck passed.

The vehicle loading response was significantly less than that of the thermal response of the structure. This

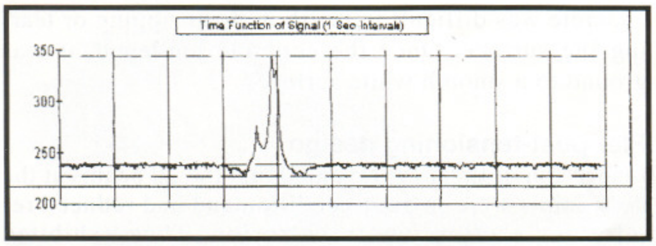

Fig. 1 — Recorded signal from heavily loaded 3-axle concrete mixer. Maximum load = 346 microstrain; minimum load = 229 microstrain; difference = 117 microstrain.

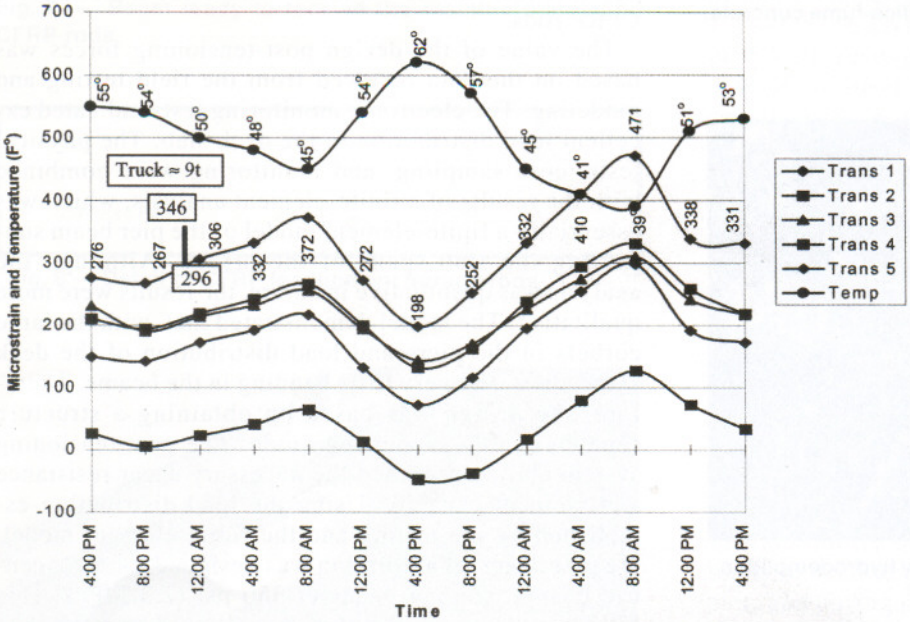

Fig. 2 — Microstrain variations due to thermal response over 48 h — Dec. 9 to 11, 1997.

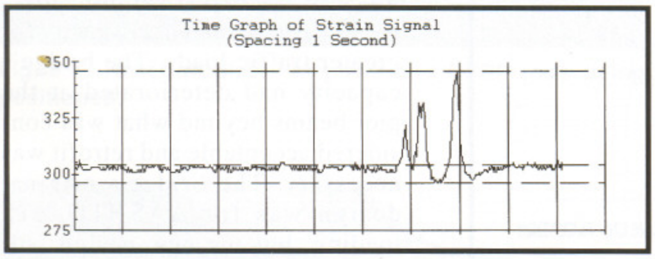

Fig. 3 — Microstrain on Transducer 5. 5-axle vehicle, 9 tons. Maximum = 346 microstrain; minimum = 296 microstrain; difference = 50 microstrain.

indicated that the structural components were still in excellent load-carrying condition. Figure 2 shows the thermal response of five spans over a 48-h period. On Transducer Number 5 (Trans 5), the microstrain ranged from 198 to 471 when the temperature at the on-board computer changed from 62 to 39 F (17 to 4 C). The upward direction of the curve indicated that the structure soffit was moving into tension as the temperature cooled. As the temperature warmed, the curve turned downward, indicating that the bridge soffit was going into compression. The wide range of movement was attributed not only to the condition of the structure, but also to the black asphalt deck, which was highly responsive to direct sunlight warming and evening cooling.

In Fig. 2, a 9-ton truck is represented by a vertical line, showing the maximum and minimum microstrains recorded in the signal graph in Fig. 3. The difference in the vehicle's microstrain is 50. The maximum microstrain difference in the transducer response due to thermal reaction is 273. The overloaded truck in Fig. 1 produced a microstrain of 117. Therefore, even vehicles exceeding the legal load limit were not affecting the structure as much as the thermal response. The engineers postulated that by removing the asphalt deck and replacing it with silica-fume concrete (Fig. 4), the movement of the structure due to thermal response would be greatly reduced.

The evidence from the monitoring demonstrated that a significant load-carrying capacity remained in the sound material to warrant rehabilitation.

Rehabilitation

The rehabilitation design contained three main strategies:

- The deck would be prepared by hydrodemolition and receive a silica-fume concrete replacement.
- Of the 36 pier beams, the 10 pier beams under the deck joints were in poor condition due to chloride exposure, freezing-and-thawing deterioration, and water erosion. The deteriorated concrete on the 10 beams, their columns, and soffit areas on the underside of the deck would be chipped off, sandblasted, and then encased in shotcrete.
- The 10 pier beams would be externally post-tensioned using carbon fiber-reinforced polymer (CFRP) leadline rods to restore their structural capacity.

The rehabilitation took place from March to December, 1998.

Deck rehabilitation

Removal of the unsound deck concrete was carried out by hydrodemolition because this method would not harm the sound concrete. It also helped to avoid concussion loads to the concrete deck associated with demolition by pneumatic tools. The quality of the old concrete ranged from excellent to extremely poor, most likely due to suspect quality control and a lack of available admixtures in 1937. During the demolition, some areas lost only 1/2 in. (13 mm) of concrete thickness while other areas lost up to 5 in. (130 mm). Existing reinforcement was found to be in good to excellent condition. In many places, the original wire ties were still intact (Fig. 5).

The concrete for the new deck wearing surface contained silica fume. The Kansas Department of Transportation

Fig. 4 — Asphalt deck replaced by white silica-fume concrete. Note the bridge is still open to traffic.

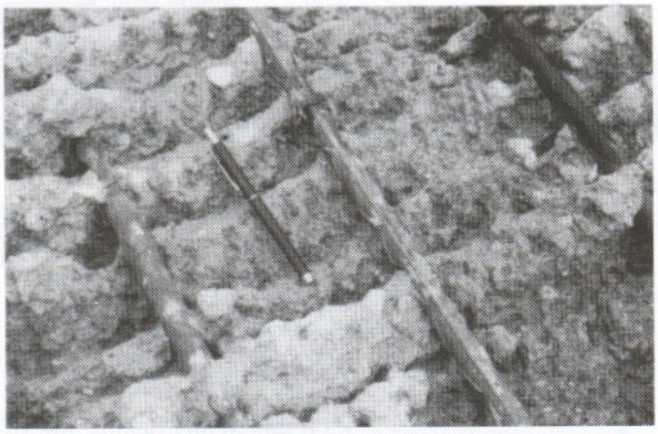

Fig. 5 — Existing reinforcement exposed by hydrodemolition.

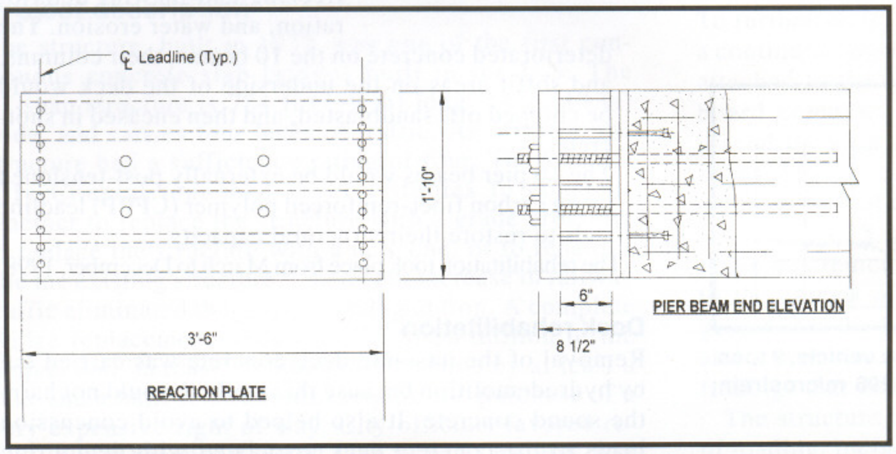

Fig. 6 — Elevation details of reaction plates and bearing plates.

now uses this type of concrete almost exclusively. Silica-fume concrete is very impermeable and much stronger than conventional concrete, which justifies the increased expense of the material. One disadvantage, however, is that it is more susceptible to heat shrinkage. Another is that the concrete can be difficult to work with. On this project, a superplasticizer had to be added to the mixture to provide a workability time of about 45 min. The concrete was difficult to finish without ripping or tearing the surface. Once the concrete hardened, it was ground to a smooth white surface.

Pier post-tensioning design

Inspection of the piers revealed that the 10 beams at the deck joints were in poor condition and had reduced reinforcing section due to corrosion. They exhibited spalling and corrosion from years of continual water and chloride exposure. The 26 intermediate pier beams were in relatively good condition. The locations of deck joints created the potential for future deterioration, so post-tensioning systems with conventional high-strength strands or bars would be susceptible to stress corrosion. For this reason, the engineers chose the noncorrosive CFRP rods.

The value of the design post-tensioning forces was based on the data received from the field testing and modeling. The electronic monitoring tests indicated excellent load distribution in the deck slab. The physical tests, core sampling, and monitoring were combined with the results of a finite-element analysis, which was essentially a finite-element model of the pier beam supporting the slab spans of the bridge. Although the analysis was quantitative in nature, the results were more qualitative. The model demonstrated that, with the large corbels in the piers and load distribution of the deck slabs, there was very little bending in the beams. Therefore, the design was based on obtaining a structure capable of this supporting mode. The post-tensioning system chosen provided the necessary shear resistance and moment capacity. Using the load distribution established by the testing and the finite-element model, the pier beam rehabilitation was designed for a concentric post-tensioning of about 350 psi (2.3 MPa). This level of force would be more than adequate to re-establish the structural integrity of the pier beam.

The purpose of the rehabilitation was not only to recover lost capacity, but also to increase capacity for greater traffic loads. The bridge's capacity had deteriorated at the pier beams beyond what was considered acceptable and retrofit was necessary. The bridge's original design was for AASHTO H15 loading, but the new design will give the bridge the capacity to resist AASHTO HS20 loading. The strength gained in the pier beams was verified with the use of the finite-element model, which indicated the necessary strength would be provided through the post-tensioning and shotcrete cover.

CFRP leadline post-tensioning rods

This was the first known bridge in the U.S. to undergo a rehabilitation using CFRP leadline rods for pier post-tensioning. These rods are stronger than equally sized steel rods and offer the advantages of corrosion resistance,

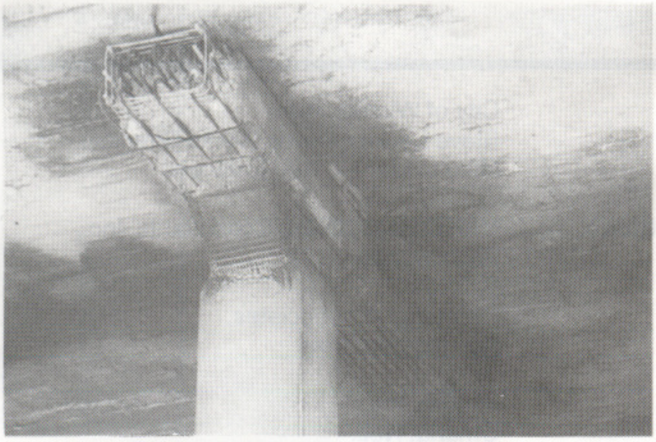

Fig. 7 — Beam ready to receive the reaction plates and CFRP rods.

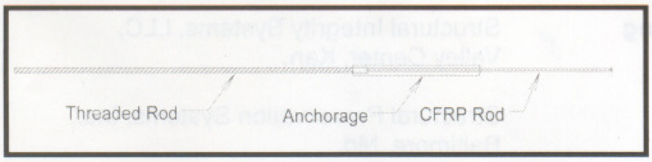

Fig. 8 — Detail of CFRP leadline anchorage rods.

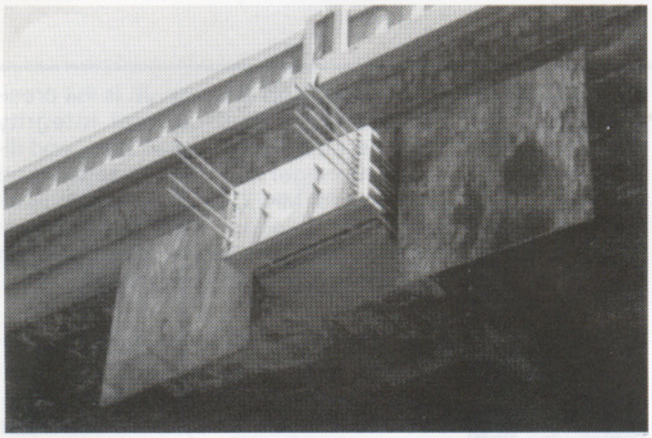

Fig. 9 — Reaction plates in place and CFRP rods being positioned.

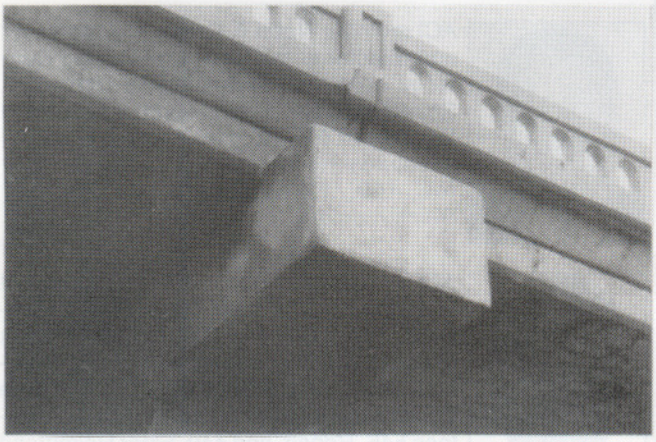

Fig. 10 — Completed pier after encasing all elements in shotcrete.

good fatigue and damping response, low relaxation losses, and low unit weight.

Each pier beam had 20 such rods, 10 on each side, affixed to it. Reaction and bearing plates (Fig. 6) held the rods in place and compressed the pier beams to their design capacity. The reaction plates were of galvanized steel and weighed 800 lb (360 kg) each. The bearing plates were of galvanized steel and had bearing bars welded to them. Each end of the pier beam was chipped off using light chipping hammers to obtain adequate bearing on the existing concrete for the bearing plates, which were then cast in place (Fig. 7).

A performance engineering specification was developed that required the anchors of the rod assembly to have an ultimate capacity of two times the service tension of 12,500 lb (56 kN) and would fail in the CFRP rod at ultimate load. Testing was done at the University of Missouri at Rolla to meet these criteria, as well as to develop a constructible, economic anchorage. An anchorage system of galvanized threaded pipe filled with grout was tested and utilized (Fig. 8). Each rod was 0.31 in. (8 mm) in diameter and was 65% fiber by volume in a 35% epoxy-resin matrix. The galvanized threaded anchor ends were added after the pier beam length was measured in the field. The complete system of CFRP rods with two anchorages was then fabricated on site. This was done by inserting the rod into the center of the anchorage, which was then filled with a grout and allowed to set up for 24 h.

Placing the CFRP leadline rods required extreme care so that they did not twist at any time during the installation or tensioning procedure (Fig. 9). Twisting could cause loosening in the anchorage and torsion in the rod. The possibility of a small angle of twist in the rod was likely to occur during assembly of the anchor head, coupler, and during post-tensioning. This twist would have significant adverse effects on the breaking force of the post-tensioning rods.

After the CFRP rods were placed, they were each tensioned to 12,500 lb (56 kN). During the construction, a rod failure occurred below the specified tension due to twisting, and corrective field procedures eliminated this problem by utilizing the jack only for tensioning. This would ensure that only a linear force was applied. Seven days after the tensioning, the rods were examined and tested. The tests showed an average loss of 125 lb (556 N), which was not considered significant. The loss probably had some creep involved, but the majority of the loss was elastic shortening.

Although the work on the 10 pier beams occurred in different stages throughout the project, the time required to accomplish the post-tensioning of a single pier beam was estimated by the supervising foreman to be 7 days with a crew of five. The foreman stated that it was his experience that the CFRP rods are not difficult to work with, but because they are brittle and break if bent sharply, they require care when tensioning and shipping.

Application of shotcrete

The 10 pier beams and their columns were sandblasted before the post-tensioning and all the deteriorated concrete chipped out. After the post-tensioning, partial depth repair areas were shotcreted to build up the required cross section. Shotcrete was chosen because it did not require the use of forms and had a high strength. The pier beams and their columns were completely encased in a 4 in. (100 mm) layer of shotcrete to strengthen and protect them from further deterioration. The samples of old concrete were tested to 3500 psi (24 MPa), and after the shotcrete application the samples tested to 6000 psi (41 MPa).

Even though the CFRP rods are inert to most common contaminants, they were still encased in shotcrete (Fig. 10) to prevent debris and other objects from being directed at them. Testing was conducted to determine whether the shotcrete would adversely affect the post-tensioning properties of the CFRP leadline rods; the results suggested it did not. The only apparent change was the dull to shiny finish of the rods when the test was completed. The reaction plates were also encased to protect them from weather, salts, and other deicing substances.

Credits

Owner	City of Wichita, Kan. Steve Lackey, Public Works Director Mike Lindebak, City Engineer
Engineering consultants	MKEC Engineering Consultants, Inc., Wichita, Kan.
General contractor	Cramer and Associates, Des Moines, Iowa
Hydrodemolition	Meylan, Inc., Omaha, Neb.
Shotcrete Application	Bob Burks and Associates, Wichita, Kan.
Electronic monitoring	Structural Integrity Systems, LLC, Valley Center, Kan.
Post-tensioning	Structural Preservation Systems, Inc., Baltimore, Md.
CFRP rod manufacturer	Mitsubishi Chemical Corp.

Conclusion

The South Broadway Overpass was one of the first bridges in the United States to use long-term electronic monitoring to evaluate structural stability and aid in rehabilitation design. It was the first known bridge to undergo the use CFRP leadline rods to externally post-tension pier beams. The bridge was open to traffic in at least one direction during the entire course of the construction. Traffic flow, public safety vehicles, and businesses were able to function with a minimum of inconvenience. One of the most important benefits was that taxpayers saved millions of dollars for a bridge that will continue to serve the community for decades.

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Selected for reader interest by the editors.

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ACI member **Nabil F. Grace** is a professor and director of the Structural Testing Center at Lawrence Technological University, Southfield, Mich. His research interest is in the use of FRP in prestressed concrete structures. He is a member of ACI Committee 440, Fiber Reinforced Polymer Reinforcement.

ACI member **Jay Thomas** is vice president of sales for Structural Preservation Systems, Inc., Baltimore, Md. He has 17 years of construction experience involving structural repair and strengthening of concrete structures. He is a member of ACI Committees 546, Repair, and 440, Fiber Reinforced Polymer Reinforcement.