Louisiana Transportation Research Center

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I-10 Girder Repair Using Post-Tensioned Steel Rods and Carbon Fiber Composite Cables (CFCC)

by

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Having been partially submerged for several decades, the girders of the I-10 Littlewoods Bridge have become a safety concern due to structural deterioration. As a response, Louisiana Department of Transportation and Development (DOTD) initiated a rehabilitation project that utilized external post-tensioning using both high-strength steel rods and Carbon Fiber Composite Cables (CFCC) as the secondary support. Instrumentation including thermistors, strain gages, and load cells were installed to monitor the behaviors of the girders.

Based on the data collected within the monitoring period, the rehabilitation effort appears to be functioning as design by slowly transferring load to the external support system. There are several observable differences between the high strength steel rods and CFCCs. However, without a clear baseline, it is very difficult to determine the causes of the differences. Many factors, such as anchorage installation, structural conditions, and instrumentation types, could have contributed to the behavior differences. Both materials appear to be functioning as designed. CFCCs have the advantages of smaller thermal effect; thus, the interpretation of the structural performance is less impacted by the temperature effect. Steel rods have the cost advantage and prove to be as effective in supporting the structure at least in the short term. Due to the short observation period, the long-term behaviors of the steel rods and CFCCs should only be determined after longer monitoring period.</td></tr>
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Each research project will have an advisory committee appointed by the LTRC Director. The Project Review Committee is responsible for assisting the LTRC Administrator or Manager in the development of acceptable research problem statements, requests for proposals, review of research proposals, oversight of approved research projects, and implementation of findings.

LTRC appreciates the dedication of the following Project Review Committee Members in guiding this research study to fruition.

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ABSTRACT

Having been partially submerged for several decades, the girders of the I-10 Littlewoods Bridge have become a safety concern due to structural deterioration. As a response, Louisiana Department of Transportation and Development (DOTD) initiated a rehabilitation project that utilized external post-tensioning using both high-strength steel rods and Carbon Fiber Composite Cables (CFCC) as the secondary support. Instrumentation including thermistors, strain gages, and load cells were installed to monitor the behaviors of the girders.

Based on the data collected within the monitoring period, the rehabilitation effort appears to be functioning as design by slowly transferring load to the external support system. There are several observable differences between the high strength steel rods and CFCCs. However, without a clear baseline, it is very difficult to determine the causes of the differences. Many factors, such as anchorage installation, structural conditions, and instrumentation types, could have contributed to the behavior differences. Both materials appear to be functioning as designed. CFCCs have the advantages of smaller thermal effect; thus, the interpretation of the structural performance is less impacted by the temperature effect. Steel rods have the cost advantage and prove to be as effective in supporting the structure at least in the short term. Due to the short observation period, the long-term behaviors of the steel rods and CFCCs should only be determined after longer monitoring period.

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Arthur D'Andrea's continuous involvement of this project from project inception to completion and continuing monitoring made this project reality. Justin Guilbeau, DOTD's project engineer for the construction, has been a tremendous help in getting the contractor to maintain the often problematic instrumentation system running. The contributions of the project review committee members are also acknowledged.

IMPLEMENTATION STATEMENT

External reinforcement or post-tensioning has been used to either enhance the structure capacity or repair damaged structures. The effectiveness of the methods and impact of the environmental conditions, as demonstrated by this project, can be effectively and objectively evaluated using instrumentation. Both high-strength steel rod and CFCC are acceptable materials. The use of CFCCs requires careful planning as the anchors of the CFCCs have to be factory-installed. UV protection is also important for CFCC.

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INTRODUCTION

The I-10 Littlewoods Bridge is a twin structured 250-ft. four-span prestressed concrete girder bridge located just east of the city of New Orleans within the Bayou Sauvage National Wildlife Refuge area. Lake Pontchartrain is located about one mile to the west of the site. I-10 at the location runs in a southwest to northeast direction (see Figure 1).

Figure 1
Project location

The design bottom elevations of the girders for these bridges range from -0.2 ft. to +0.5 ft. Typical water levels of the Lake Pontchartrain at the Lakefront Airport, located at about 11 miles southwest of the site range, from -.25ft. to +2.5ft. with an average of about 0.8ft. Typical low water levels occur in March and from July to mid-September. The bottom of the girders, set at an elevation about 9 in. below the average water level, have been submerged in relatively stagnant water since completion of the bridge construction in 1967. As a result, some girders experienced severe corrosions in the prestressing strands and spalls of the concrete in the submerged portion of the girders. Several of the strands were observed to be broken, as shown in Figure 2. Figure 2 (a) shows the conditions of east bound bridge after dewatering. Figure 2 (b) shows a close look of the deterioration of the west bound bridge. The exposed and corroded pre-stressing strands are clearly in view. The discoloring shows the high water mark during girder submergence near the top flange, while the normal water level is near the bottom of the web where severe concrete spalls and steel corrosion can be clearly seen.

Figure 2 (a)
Distance view of the eastbound bridge

Figure 2 (b)
Close view of the westbound bridge

Concerning with the public safety, DOTD's Bridge Rating Gang performed a load rating of the bridge. The results showed that the bridge had the sufficient capacity at the time of rating. However, the continuing deterioration of the girders may compromise the bridge integrity and is therefore a major concern. Not being able to observe the conditions due to its submergence during regular bridge inspections, the DOTD Bridge Design Section had a concern that a potential sudden collapse would be a significant thread to the safety of the traveling public.

Considering the importance of the bridge and the current conditions, it is scheduled to be replaced in 2020. However, there is no guarantee that the bridge can open for another 5 years without any repair. Even if the replacement schedule can be moved up, the time it takes to generate a capital investment program, design, and construct a replacement bridge is likely to take several years. In the meantime, the continuing deterioration may endanger the travelling public. Knowing that the bridge is scheduled to be replaced within a 5- to 10-year time frame, the DOTD Bridge Design Section determined that the best course of action was to rehabilitate the bridge to ensure adequate services and safety of the structure before the bridge replacement. It was decided to use external reinforcements as a means of providing secondary support in the event that the prestressed girders fail. The advantages of using the external post-tension are that the work can be done relatively quickly and without traffic disruptions.

Traditional practice for external reinforcement at DOTD has been the use of high-strength steel tendons or high-strength steel rods applying tensions to increase the compression stress level in the concrete section. The increased compression stress can then provide some buffer to resist cracking from the loads. One example of such a project is the Alexander Creek project in West Feliciana Parish. The external tensioning construction is relatively fast and can be done without impacting traffic. Due to the harsh environment at this site and the continuing submersion of the girders, the design team elected to experiment with the use of carbon fiber composite cables (CFCC) to reinforce one of the spans (Span C, Westbound). Traditional high-strength steel rods were used for all other girders that require reinforcement. All steel rods were galvanized to counter the high corrosion potential environment.

The DOTD design team made several site visits prior to the rehabilitation design. It was very difficult, if not impossible, to determine the extent of the damage due to the submergence of the girders and the murky water. The initial design was therefore based on the observations of the conditions of the exterior girders. Because of the uncertainty of the concrete strength of the deteriorated girders, it was decided that the tension forces applied to the external reinforcement should not cause significant increase of stresses in the girders. Instead, they are to be used as the secondary support. Rather, only nominal amounts of tension force are applied to ensure the “taking out” of slacks in the external reinforcing elements. It is anticipated that, should the deterioration compromising the girder integrity, the excessive loads can be transferred to the external reinforcing elements and reduce the potential for sudden failure, thus affording the department the time to repair failed girders.

As discussed previously, two types of material were used: high-strength steel rods and carbon fiber composite cables. The high strength rods used for this project are DYWIDAG bars (ASTM A722 150 ksi threaded bars) manufactured by the Williams Form Engineering Corp. This type of bar is frequently used in the ground anchors, soil nails, and post tensioning projects. Similar to reinforcing carbon steels, the properties of the DYWIDAG bars are well understood. Advantages of using steels for post tensioning include:

- Well-known properties
- Vast experience for the intended purposes
- Cost
- Flexible – amenable to field changes for splicing and cutting
- Contractor's familiarity with the material
- UV resistance
- Similar thermal property as the existing concrete girders
- Availability

Potential disadvantages include:

- High corrosion potential when exposed and without special treatment
- Heavy – difficult to handle in tight spaces

A photograph of the DYWIDAG bars used for the post tensioning is shown in Figure 3(a).

Figure 3(a)
DYWIDAG bars

Figure 3(b)
Carbon fiber composite cable

Carbon fiber [Figure 3(b)] is a high modulus synthetic fiber made from an acrylic containing carbon, hydrogen, and nitrogen atoms that is heated in three successive stages to eliminate all but the carbon atoms. It is naturally black in color and is essentially unaffected by UV exposure. First used successfully in the manufacturing of the racing boats in America's Cup, carbon fiber laminates provide exceptionally low stretch for their weight. Recent manufacturing advances have led to improved fiber flexibility, which translates to a longer life in exchange for lower modulus numbers. The balance between low stretch and high flexibility means that carbon fiber can be extremely brittle and damage intolerant. Advantages of the carbon fiber cable are:

- High Strength to weight ratio – the ratio for carbon fiber is about 10 times of carbon steel
- Flexible
- Corrosion resistance
- Good fatigue resistance
- Good tensile strength
- Fire resistance/not flammable
- High thermal conductivity in some forms
- Low coefficient of thermal expansion
- Non poisonous
- Biologically inert
- Excellent EMI (Electromagnetic Interference) shielding property

The disadvantages of the carbon fiber composites are:

- High cost
- Difficult to splice as such the cables are cut to the exact design lengths
- High skill set and special equipment needed for the installation
- Anchorages have to be factory installed
- Supplier has no current US manufacturing plant and a long lead time is needed to order the material

The project site is located in a relatively high corrosive potential environment, which makes the use carbon fiber reinforcement viable. However, to balance the high corrosion resistance with the high cost, a lack of experience with the material, and the anticipated short design life (5 to 10 years), the DOTD design team felt that, while CFCC appears to be a better long-term solution from the corrosion resistance point of view, steel reinforcing bars with corrosion treatment should be adequate within the design life. It was then determined that CFCC would be used along with the steel reinforcing bars. A picture of CFCC is shown in Figure 3(b). The performance of these reinforcing elements would be monitor and evaluate to provide guidance for future projects in material selections when a high corrosion potential is a concern.

There are many reported cases of CFCC applications throughout the world. In 2000, Noisternig described the use of carbon fiber composite wires for stay cables and field stress tests of the cables for a cable-stay bridge in Europe *[1]*.   In the US, the Michigan Department of Transportation reportedly performed a laboratory study of unbonded CFCC for transvers post-tensioning of side-by-side box beam bridges in 2006 *[2]*. The test results showed that transverse post-tensioning significantly improved the load distribution among side-by-side box beams, and that increasing post-tensioning levels improved the overall behavior of the bridge model. The load test showed that loads were evenly distributed until failure, and none of the carbon fiber composite cables ruptured. Virginia DOT used CFCC as replacement of the prestressing strands for production piles *[3]*. Florida DOT through Florida State University also reported using CFCC in lieu of prestressing strands in 10 piles *[4]*. They concluded that the performance of piles prestressed with CFCC strands is comparable to those prestressed with steel. Using CFCC strands in prestressed concrete piles for bridge foundations, particularly in harsh environments, could potentially result in bridges that require less maintenance and have longer lifespans. In the “Use of Composite Materials in Civil Infrastructure in Japan”, several structures were listed in Japan that utilized CFCC as post-tensioning element *[5]*. The cited projects were new construction.

OBJECTIVE

The primary purpose of the repair is to rehabilitate the bridge to maintain a safe service level. The accompanying instrumentation for this study is thus to provide a system that can monitor the performance of the girders and the post-tensioning system to ensure the safe operation of the brides. A secondary objective of the instrumentation is to offer the opportunity in providing performance comparisons between the two external reinforcement materials: steel and CFCC.

Due to the corrosive environment of the bridge, there was a concern of continuing deterioration of the bridge girders and reinforcing steel bars. Specifically, the following potential modes of deterioration are of concern:

- Sudden failure of the girders due to breakage of the prestressing tendons or sudden loss of girder section from the deterioration of the concrete – it is anticipated that this type of failure can result in drastic increase and or decrease in the measured strains in the external post-tensioning elements.

- Gradual deterioration – with the deterioration, some of the bridge dead and live loads would be transferred to the external tension bars. Eventually, the excessive forces, if unabated, can lead to catastrophic bridge failure.

- Gradual but uneven deterioration, when the rate of deterioration varies from one side to the other, the girder(s) become unsymmetrical, resulting in the bending or warping in the weak axis. With this type damage progression, it is possible the girder(s) will cease to function due to the distortion.

- Sudden or gradual decreases in the post-tension forces indicating of anchorage failure of the post-tensioning members.

As for the comparison of steel reinforcing to the CFCCs, the interests are:

- To determine the long-term performance of the external reinforcing elements,
- To compare the behavior of steel and CFCC, and
- To determine future implementation potential of CFCC.

SCOPE

In order to achieve the objectives, the scope of this research was to continuously monitor the stresses in the external reinforcement for changes that can impact the performance of the bridge to ensure public safety. Specifically, the work included:

- Routinely monitor data to ensure the system is working order;
- Observe the behavior differences between steel rods and CFCCs, if any;
- Observe behavior changes of the repaired bridge; and
- Analyze the data to provide recommendations for the future implementation potential of CFCC for external post-tensioning.

While extensive data have been collected, it is not the intention of this project to provide detailed analytical or numerical studies of the bridge structures or the post-tension system. Rather, the scope was to provide practical recommendations on the safety of the bridge and potential future application of the CFCC based on the data collected.

METHODOLOGY

To ensure a successful repair of this bridge and to confirm all design assumptions were not violated, all external reinforcing elements have been instrumented with either vibrating wire strain gauges or vibrating wire load cells, depending upon the material used. Details of the site conditions, girder repair methods, instrumentation effort, and observations are provided.

Project Site

Site and Bridge Descriptions

The bridge site is located on the flood plain of the Mississippi River just south of Lake Pontchartrain between New Orleans and Slidell. A general vicinity map was presented in Figure 1. The interstate highway, I-10, crosses this site in a southwest to northeast direction crossing a swampy wildlife management area. The water body that the bridge crosses is open to Lake Pontchartrain to the northwest through series canals and bayous. Thus, the water level is mostly influenced by the seasonal fluctuating with the water level in Lake Pontchartrain. A water gate is located on the southeast side of the bridge protecting the water from migrating from Bayou Sauvage National Wildlife Refuge to Lake Pontchartrain.

The bridge is a four-span simple span structure comprising of six prestressed concrete girders in each span. The design bottom elevations of the girders range from -0.2 ft. to +0.5 ft. The average water level at the Lake Pontchartrain is about 0.8 ft. As a result, the lower portion of the prestress girders have been submerged [see Figure 2(a)] since completion of the bridge construction. Due to the submergence, some deterioration has been observed. As such, the deterioration became a concern for bridge safe service.

Repair Concept

As previously stated, the bridge repair consists of patching the deteriorated concrete and installing one external reinforcement on each side of the deteriorated girders. The girders selected for repair are shown in Table 1. All except the girders at Span C of the east bound bridge were repaired using high strength steel rods. CFCCs were used for the girders at Span C of the east bound bridge. The anchoring system for the steel rods are shown in Figures 4 and 5. Figure 4 shows the locations of the anchoring points. The anchor details are shown in Figure 5. Note that the anchoring points for the steel rods are located at 3 ft. 10 in. from girder ends. The anchoring system for the CFCCs is presented in Figure 6. The locations for the anchors for the CFCC's are located at the girder ends since only one span is reinforced using CFCC and there would not by any conflict. The initial tension forces are 20 kips and 30 kips for steel rods and CFCCs, respectively.

Table 1
Girders selected for repair

GIRDERS TO BE REPAIRED					
STRUCTURE 02364509021242 (WESTBOUND)			STRUCTURE 02364509021241 (EASTBOUND)		
SPAN	END PLATE	GIRDER	SPAN	END PLATE	GIRDER
A	8" x 3'-10" x 2	G1, G5, G6	A	8" x 3'-10" x 2	G6
B	8" x 2'-10" x 2	G1, G5, G6	B	8" x 2'-10" x 2	G5, G6
C	8" x 3'-10" x 2	G1, G3, G4, G5, G6	C	8" x 3'-10" x 2	G1, G2, G3, G4, G5, G6
D	8" x 2'-10" x 2	G1, G3	D	8" x 2'-10" x 2	G1, G2, G6

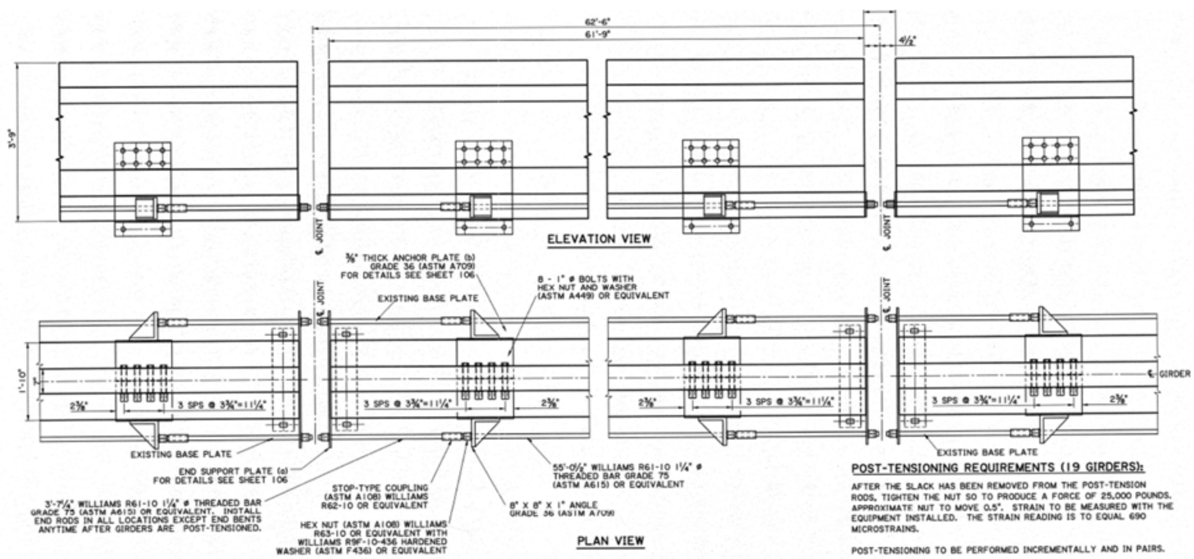

Figure 4
External steel reinforcement and anchor locations

The typical purpose of external post-tensioning is to prevent or to reduce the tensile stresses in the girder induced by dead and live loads to enhance the capacity. Therefore, the tension forces are normally designed to achieve the maximum benefit of increasing moment resistance without cracking the concrete while ensuring safety of the girders. In this case, however, the tensions of the existing tendons in the girders were assumed to remain unchanged from the constructed condition with less creep effect. Because of unknown conditions of the girder concrete, applying tensions to the external reinforcements could potentially exceed the compressive strength of the deteriorated concrete and thus cause the bridge to fail, if not carefully planned. Therefore, it was decided in the early stage of the design that only minimum tension would be applied to the external reinforcement to take out the slack of the tensioning system. The reserved tensile capacities of the external reinforcement were then used to provide the support of the bridge by transferring the support to the external reinforcements in case any girder failed unexpectedly.

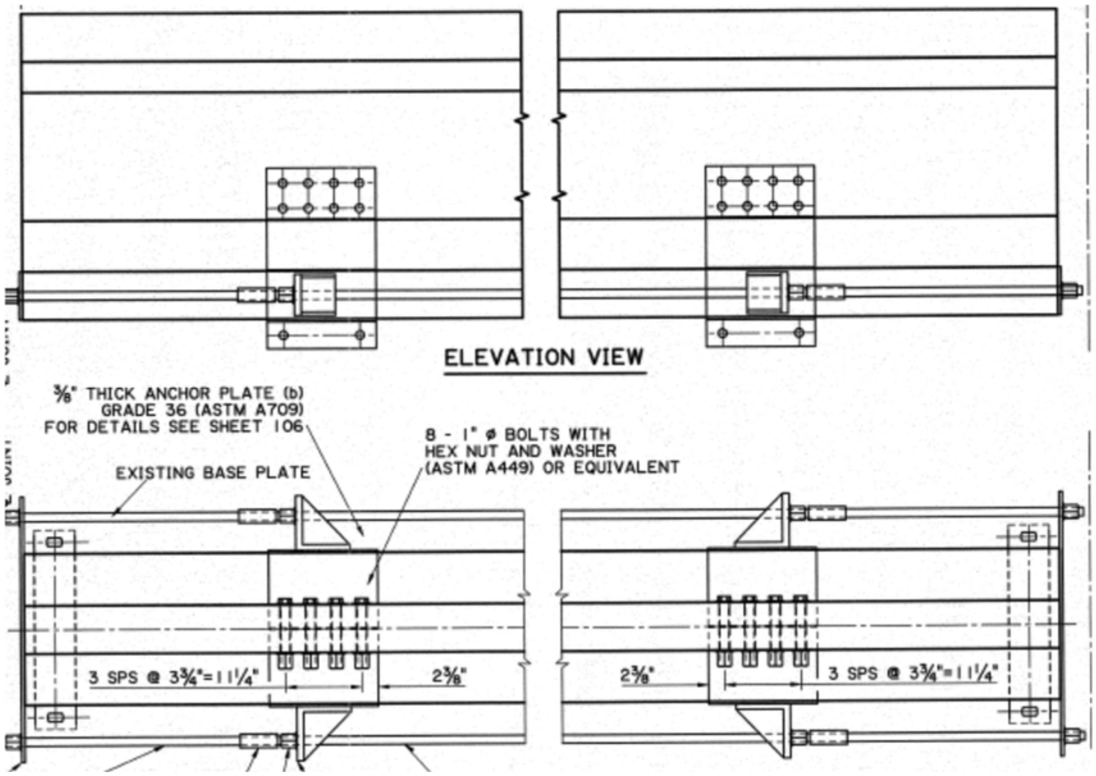

Figure 5
Anchor details for external steel rods

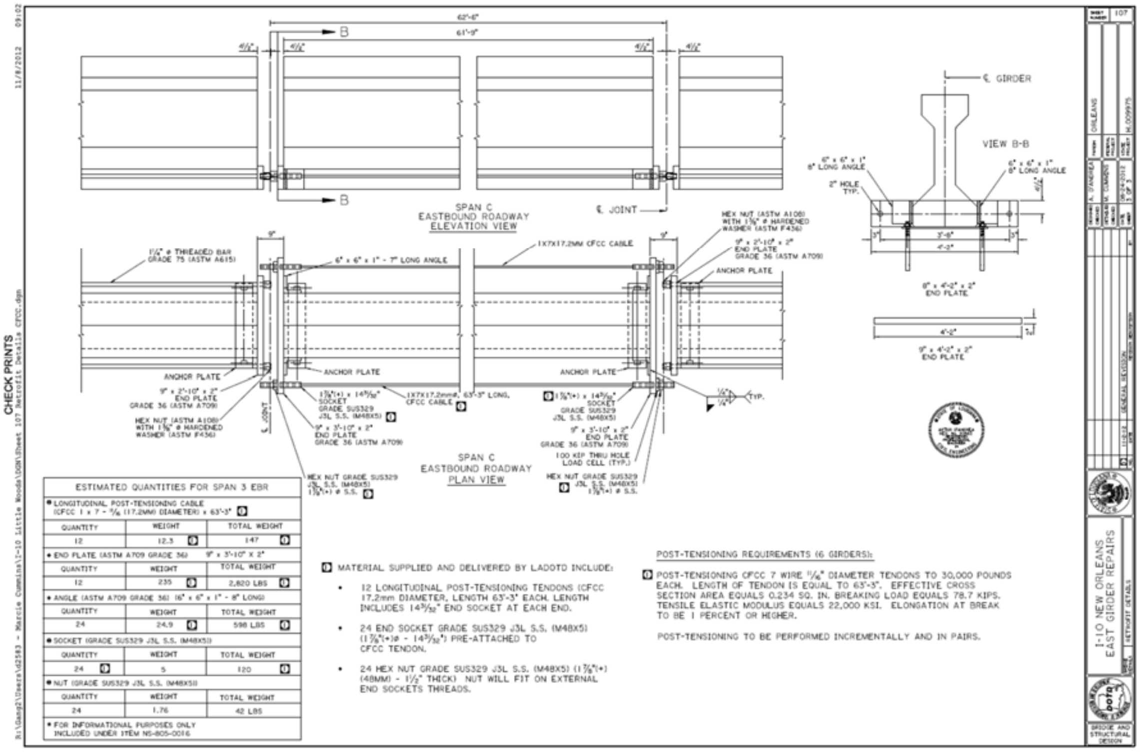

Figure 6
Anchor details for CFCCs

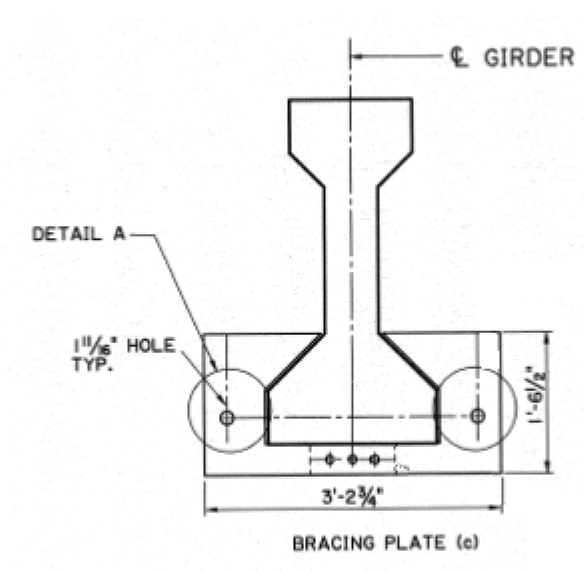

Figure 7
Anchor plate details for CFCCs

External Reinforcement Installation

After dewatering, all observable defects in the girders were patched. Concrete cores were made near girder ends for anchor installation. The concrete cores appear to be in relatively good shape with no obvious cracks except for the outer 2 in. Once the anchors were installed, the external reinforcements were put in place. Prior to tensioning, the strain gages, load cells, and cables were installed and protected. While the CFCCs have relatively high UV resistance, they are protected with a reflective cover encased in a clear PVC tube.

The contractor applied the tensions using hand pumps, one on each side of the girder tensioning. The applied forces were measured using a pressure dial gage accurate to about 10 psi level connected to the hydraulic line of hand pumps. As soon as the pressure gages indicated the desired tensions had been reached and the tension difference between the two sides of the girders were within 5% of each other, the contractor manually tightened a hex nut to lock in the force applied. The actual tensions measured a couple hours after installation are presented in Table 2. In that table, the tension measurements on 6/28/2014 (one day after the initial tensioning) are also presented. The variations between the initial and one-day tensions and the tension differences between the two sides of the girders are quite significant. As previously described the contractor used a pressure dial gauge to control the applied tension forces. Just before locking the applied forces, the dial readings have maximum differences of less than 5%. However, after a few minutes, the tensions started to drift. It is postulated that the cause of the tension drift was due to the stress redistribution of within the girder mass. Since the girder deterioration was not uniform, the stress redistributions within the girder mass reflected the direction and the magnitude of the tension drift.

Table 2
Measured tensions within 24 hours of installation

Sensor	Location	Tension 6/27 (kips)	Tension 6/28 (kips)	% Difference (North vs South side of Girder)	% Difference of intended applied force
SG1	Span A Westbound Girder 1	19.82	20.37	10%	1.3%
SG2		21.59	22.05		
SG3		17.85	18.05		
SG4		19.73	19.78		
SG5	Span A Westbound Girder 5	19.53	18.82	0%	0.3%
SG6		20.38	19.73		
SG7		19.40	18.97		
SG8		20.44	19.97		
SG9	Span A Westbound Girder 6	20.65	20.62	8%	-1.3%
SG10		21.40	21.29		
SG11		19.78	19.42		
SG12		19.18	18.76		
SG13	Span B Westbound Girder 1	18.24	17.06	6%	6.0%
SG14		20.56	19.55		
SG15		17.08	16.95		
SG16		19.37	19.52		
SG21	Span B Westbound Girder 6	21.51	21.56	16%	1.8%
SG22		20.65	20.59		
SG23		16.23	15.66		
SG24		20.18	19.52		
SG25	Span C Westbound Girder 1	19.52	18.74	-2%	5.4%
SG26		17.88	16.81		
SG27		18.54	18.10		
SG28		19.76	19.32		
SG29	Span C Westbound Girder 3	15.80	14.75	-18%	4.7%
SG30		18.54	17.26		
SG31		19.41	18.95		
SG32		22.47	22.07		
SG45	Span D Westbound Girder 1	15.80	19.53	-14%	22.4%
SG46		12.90	16.56		
SG47		15.42	18.22		
SG48		17.93	20.67		
SG49		14.44	17.76	-2%	28.5%

Sensor	Location	Tension 6/27 (kips)	Tension 6/28 (kips)	% Difference (North vs South side of Girder)	% Difference of intended applied force
SG50	Span D Westbound Girder 3	13.84	17.24		
SG51		12.30	15.63		
SG52		16.65	20.17		
SG53	Span A Eastbound Girder 6	9.73	9.57	-44%	21.6%
SG54		12.86	12.73		
SG55		17.55	16.01		
SG56		22.59	21.02		
SG65	Span D Eastbound Girder 1	19.83	18.61	22%	6.0%
SG66		21.53	19.80		
SG67		16.68	16.70		
SG68		17.16	17.26		
SG69	Span D Eastbound Girder 2	17.72	16.36	13%	11.1%
SG70		20.09	18.77		
SG71		17.40	15.78		
SG72		15.94	14.35		
SG89	Span A Westbound Girder 4	18.93	18.56	1%	-0.1%
SG90		21.37	20.94		
SG91		18.17	17.79		
SG92		21.65	21.36		
SG93	Span B Westbound Girder 2	20.41	19.59	14%	-0.9%
SG94		22.62	22.02		
SG95		18.32	17.56		
SG96		19.40	18.93		
SG97	Span B Westbound Girder 4	17.32	16.43	24%	13.5%
SG98		21.04	19.91		
SG99		19.72	18.88		
SG100		11.14	10.17		
SG105	Span D Westbound Girder 4	17.77	22.36	78%	34.4%
SG106		15.87	20.17		
SG107		12.41	14.70		
SG108		6.45	8.32		
LC1	Span C Eastbound Girder 5	26.08	26.50	3.7%	13.1%
LC2		27.08	29.45		9.7%
LC3	Span C Eastbound Girder 1	28.13	28.04	3.5%	6.2%
LC4		29.13	29.21		2.9%

Sensor	Location	Tension 6/27 (kips)	Tension 6/28 (kips)	% Difference (North vs South side of Girder)	% Difference of intended applied force
LC5	Span C Eastbound Girder 2	27.95	28.11	6.0%	6.8%
LC6		29.72	29.72		0.9%
LC7	Span C Eastbound Girder 3	27.32	27.38	2.8%	8.9%
LC8		28.12	28.10		6.3%
LC9	Span C Eastbound Girder 4	27.01	27.04	-5.1%	10.0%
LC10		25.71	25.95		14.3%

Thermal Properties of Steel, CFCC, and Concrete

In typical reinforced concrete or prestressed concrete designs, the thermal property difference between steel and concrete is not considered since typically steel is embedded inside the concrete which makes any temperature deviation unlikely. Furthermore, the two materials have very similar thermal expansion properties (Table 3). In the case of external reinforcement, the reinforcing is separated from the concrete. The differential temperature between the external elements and structure can cause additional stress that is not normally considered in the design. The differential thermal responses can induce additional stresses. In the case of CFCCs, the linear thermal expansion coefficient is only about 5% to 8% of the concrete. The difference further complicated the comparison between the behavior of the steel and CFCC. During winter when ambient temperature is lower than the girders, the CFCC can shrink more than the concrete girders imposing additional stress on the girder. On the other hand, if the ambient temperature is higher than the concrete, it is possible that one can observe a reduction of the cable tension.

Table 3
Thermal expansion coefficients (10^{-6}/ ^{o} C)

Steel	CFCC	Concrete
11-12	0.6	7.4-13

Bridge Instrumentation

The instrumentation layout is presented in Figures 8 and 9. Figure 8 shows the number of sensors on each of the spans. Span C of the eastbound bridge is the only span that was instrumented with load cells. All others were instrumented with two strain gages on each of the reinforcing bars.

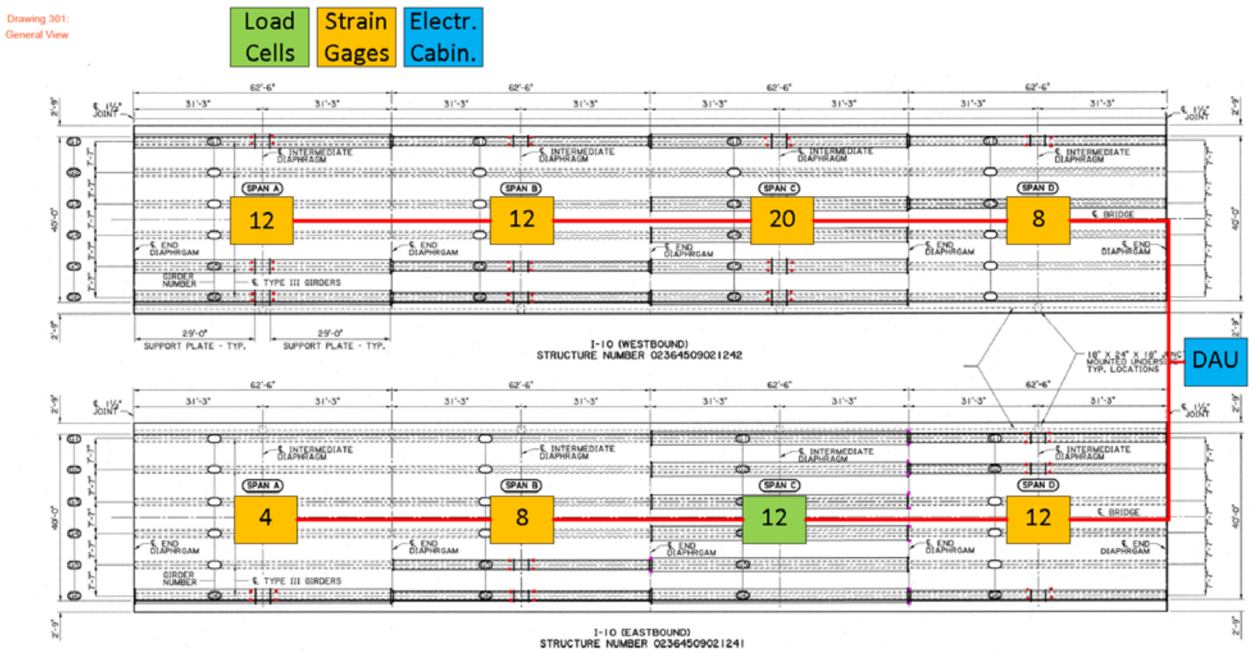

Figure 8
Sensor types and counts

Figure 9 shows the actual locations of the sensors and cables that connect these sensors to the data acquisition system. Note that two cabinets house the data acquisition system and multiplexers are located on the east side of the bridge as shown on the figure as "DAU."

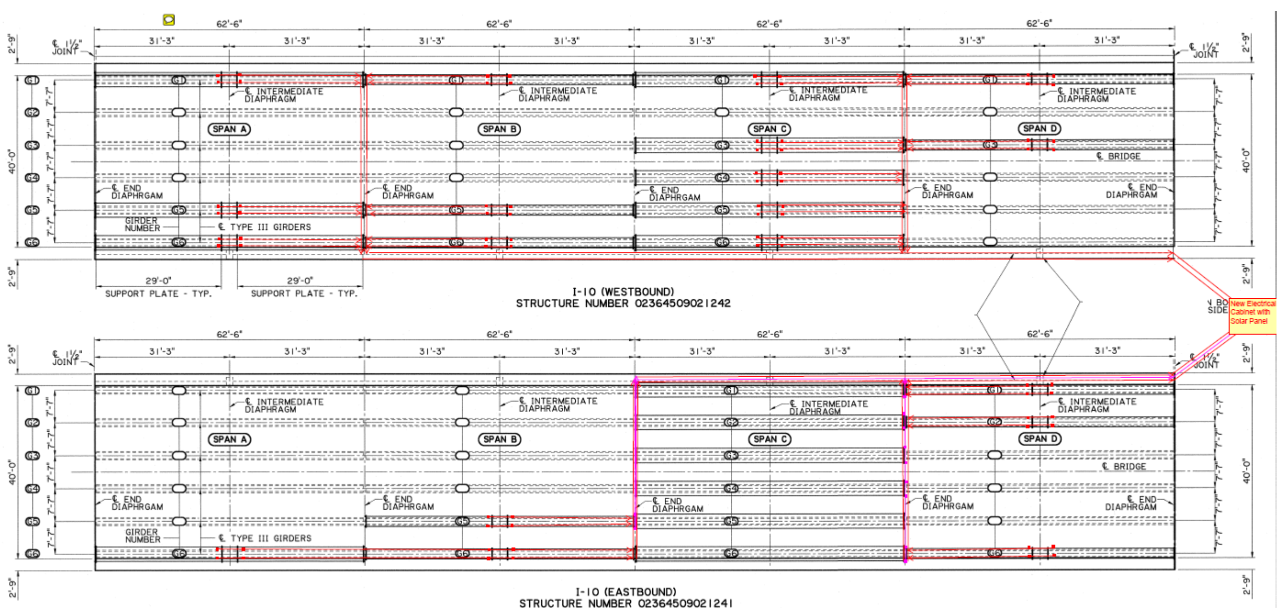

Figure 9
Sensor locations and cabling

All strain gages and load cells are vibrating wire sensors. The complete instrumentation consists of 76 spot weldable strain sensors (Geokon 4100) and 12 load cells (Geokon 4900). Two each of the spot weldable gages were installed on each of the steel rods. Only one load cell was installed on each of the CFCCs. The strain sensors were welded on the midsection of the reinforcing steel rods. The load cells were installed at east end of the anchor of the CFCCs. These sensors are connected to six multiplexers (Geokon 8032) and they in turn are connected to a micro datalogger (Geokon Micro 1000). A 20-W solar panel provides the power for the system and a 12-volt deep cycle marine battery is used for power storage to maintain functioning of the sensors at night time or during bad weather. The sensors were set to take readings at the 15-minute intervals. It should be noted that all sensors including load cells are vibrating wire gauges. These are not dynamic gauges and cannot not monitor dynamic responses of the bridge such as those caused by traffic live loads. Only static responses (long-term effects) can be recorded.

Monitoring System Reliability

Due to the contractor's underestimating of the battery storage needs, the system has been down six times since completion of construction to May 2016. It took the contractor about two weeks each time to restart the system after each outage. Thus, there are at least five gaps in the data. In addition, 17 of the 76 strain gauges and one load cell malfunctioned or were damaged during the monitoring period. One possible reason for the high rate of damages is that the majority of these sensors were submerged most of the time. Inadequate water proofing or attacks by aquatic creatures may have caused the damages. Note that there were two alligators known to reside in the water at the bridge site. The load cells were encased in stainless steel housings and are better protected, only one of the 12 load cells failed. The high failure rate makes certain interpretation of the results difficult. Only the general trends are discussed in this report.

DISCUSSION OF RESULTS

Instrumentation

The instrumentations at this site measure two types of information: temperature and strain. A standard alone thermistor is located inside of the data acquisition cabinet that measures the temperature inside the cabinet. In addition, 12 of the strain gages are temperature compensated. These gages also provide temperature measurements. The temperatures from these strain gages are assumed to represent the temperature of the respective external post-tensioning elements.

Since the primary objective of this project was to monitor the performance of the repaired bridge, strain readings were the most important indicator of any behavior changes. In conjunction with the temperature measurements, the strain data offered some very interesting insight into the behavior of both steel rod and CFCC reinforced girders. Results from the observations are discussed in the following sections.

It should be noted that due to the large number of temperature and strain gages, only representative data are presented in this report. The entire dataset for this report and all continuing monitoring data are preserved in the DOTD's Bridge Data server.

Temperature Observations

DOTD's experience in the bridge structure distress indicates that most observed structure distress could mostly be attributed to temperature effect. Temperature can affect the structures in many ways. One well understood phenomenon is the lock-in stress. Other effects not included in the design considerations such as biaxial bending can also contribute to structure behavior that causes concerns. The external reinforcement being placed outside of the girder and being exposed to the environment is likely exacerbate the problem. The temperature data obtained from this site can serve as the bases for the future design considerations.

As described previously, 13 temperature readings are available. The readings from the thermistor installed inside of the data acquisition cabinet are assumed to be closely represent the ambient temperatures. The 12 temperature measurements from the temperature compensated gages provides both temperature and strain readings. The temperature readings from these gages represent the actual temperature of the external reinforcing elements. All 12 temperature compensated gages are installed at Span D of the eastbound bridge. In addition to the temperature compensated gages, all 12 load cells also have thermistors. Their readings represent the temperatures of the CFCCs.

Figure 10 shows the ambient temperatures vs. time from March 1, 2015 to April 26, 2016. This period is selected based on the data consistency. Note there are four gaps in the data within this period. Also there were two longer outage gaps outside of this period. The maximum temperature measured is about 115°F, and the lowest temperature observed is about 36°F during the same period, a difference of about 79° F. The maximum daily temperature fluctuation in the summer month of 2015 was about 30° F, while the maximum daily temperature fluctuation in the winter was about 38° F. It is worth mentioning that these are the temperatures measured inside of the data acquisition cabinet, not the true ambient conditions. The greenhouse effect inside the cabinet may exaggerate the high temperatures, while the insulation of the cabinet may also result in the apparent higher temperatures readings than actual ambient conditions during winter. Regardless of the environmental effects, the daily temperature ranges appear to vary only slightly in general.

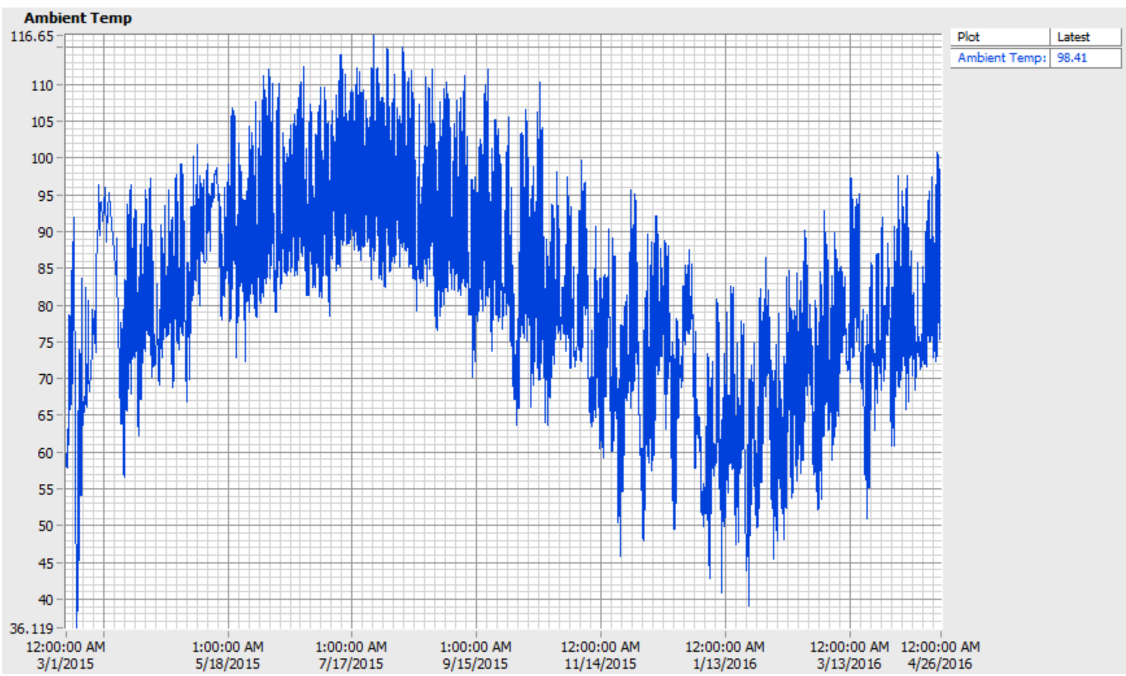

Figure 10
Temperature inside the data acquisition cabinet

The temperatures of the external reinforcing elements measured during the same period are presented in Figures 11 through 14. Note that all temperature readings are taken from Span D of the eastbound bridge. Figure 11 shows the temperature of the steel rods on the two sides of Girder 1. The red line represents the temperatures at the south side of the girder and the blue line represents temperatures at the north side. Since Girder 1 is the southernmost girder, the south side is exposed to ambient conditions or under a few inches of water, while the north side is either under constant shade or underwater. During the summer months, the south side of the girder experienced higher temperature fluctuations and the north side remains in a narrow band of fluctuation. The temperature differential between the two sides can reach as high as 35°F from June to October. It is suspected that the high temperature fluctuations were the results of the lower water level during the summer and the temperatures measured reflected the true ambient conditions. Detail discussions of the water level effect are presented later in this report. Based on the temperature difference between the two sides of the girder, it appears that the temperatures of unexposed reinforcement roughly near the daily lows in the summer. The night time temperatures in December, on the other hand, appeared to be near the daily highs. The temperature effect can have a significant impact on the girder behavior since the differential temperature between the two sides of the same girder can cause the girder to bend along the weak axis. Depending upon the magnitude of stresses resulting from the differential temperatures, it is possible that deterioration rate of the concrete can be significantly impacted.

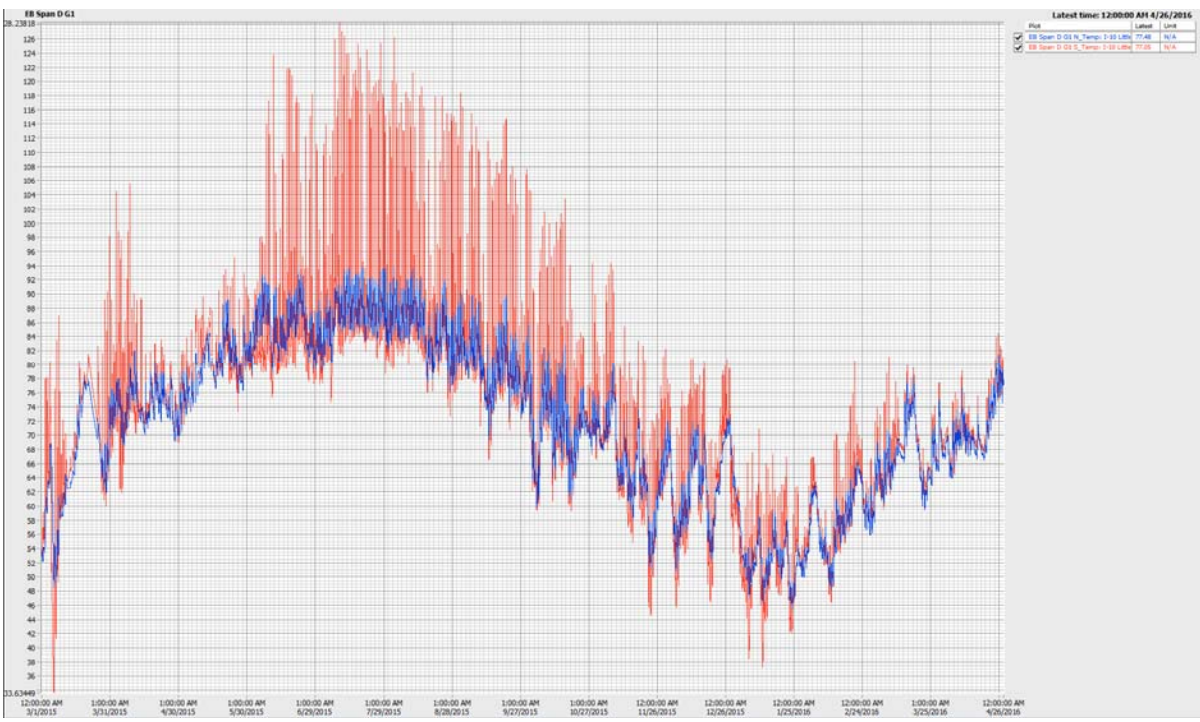

Figure 11
Temperature of the external steel rods for girder 1 (Span D) eastbound

As expected, there is no observable difference in temperatures between the two sides of internal girders as shown in Figure 12 since the girder is not exposed to the ambient conditions. The maximum observed temperature differential was only about 2° F. Typically, the large deviations occurred during the summer months.   During the winter months, the deviations were typically less than 1° F. The recorded temperatures for the northernmost girder (Girder 6) are shown in Figure 13. As expected, large temperature fluctuations were also observed during the summer months on the north side of the girder due to the exposure to the ambient conditions. As a consequence, the temperature differential between two sides of that girder was also greatest in the summer. While both the north and south girders show large temperature deviations in the summer month, it is interesting to note that there is a one- to two-month difference in timing. The large deviations in Girder 6 (north) were from August to November and between June and October in Girder 1 (south). The difference in timing is likely due to the elevation difference between the two girders. The southern girder being about 6 in. higher than the northern, which results in the earlier and longer exposure to the ambient conditions. As can be observed in Figure 13, one of the strain gages on the reinforcement of Girder 6 failed in January 2016. Only the data prior to that day are available for comparison. The maximum daily deviation was nearly 40° F, somewhat greater than observed in Girder 1. The daily temperature fluctuation of Girder 6 reached 52° F in mid-September.

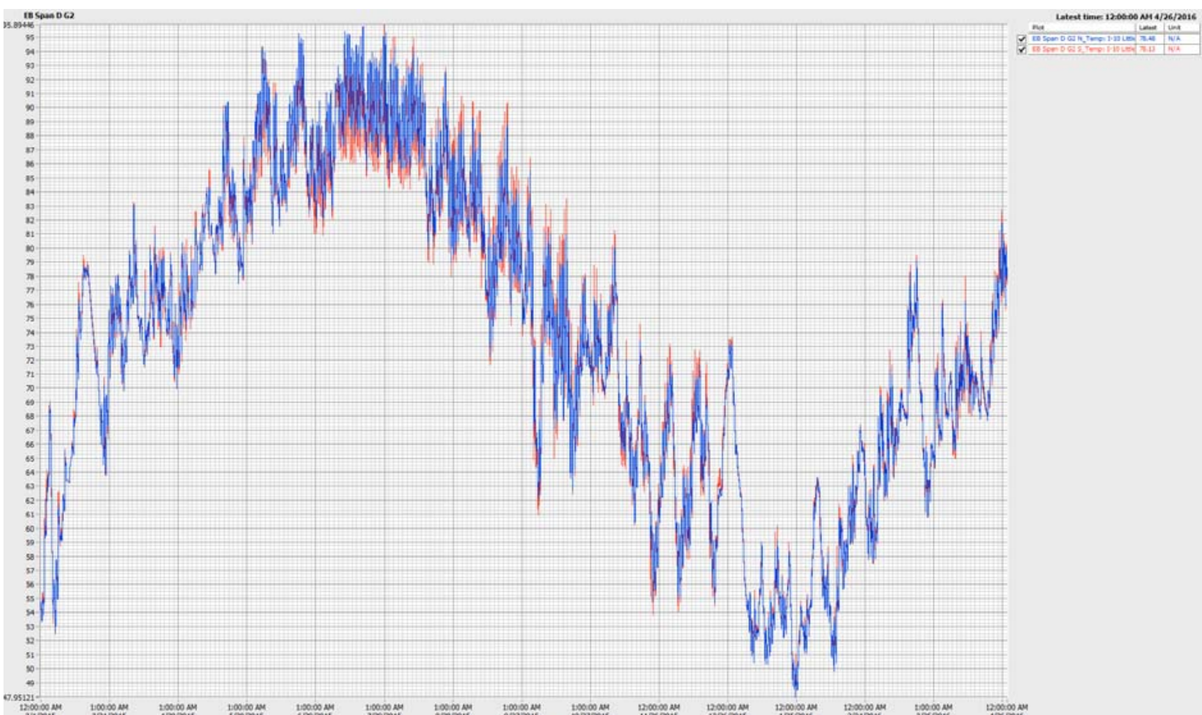

Figure 12
Temperature of the external steel rods for girder 2 (Span D) eastbound

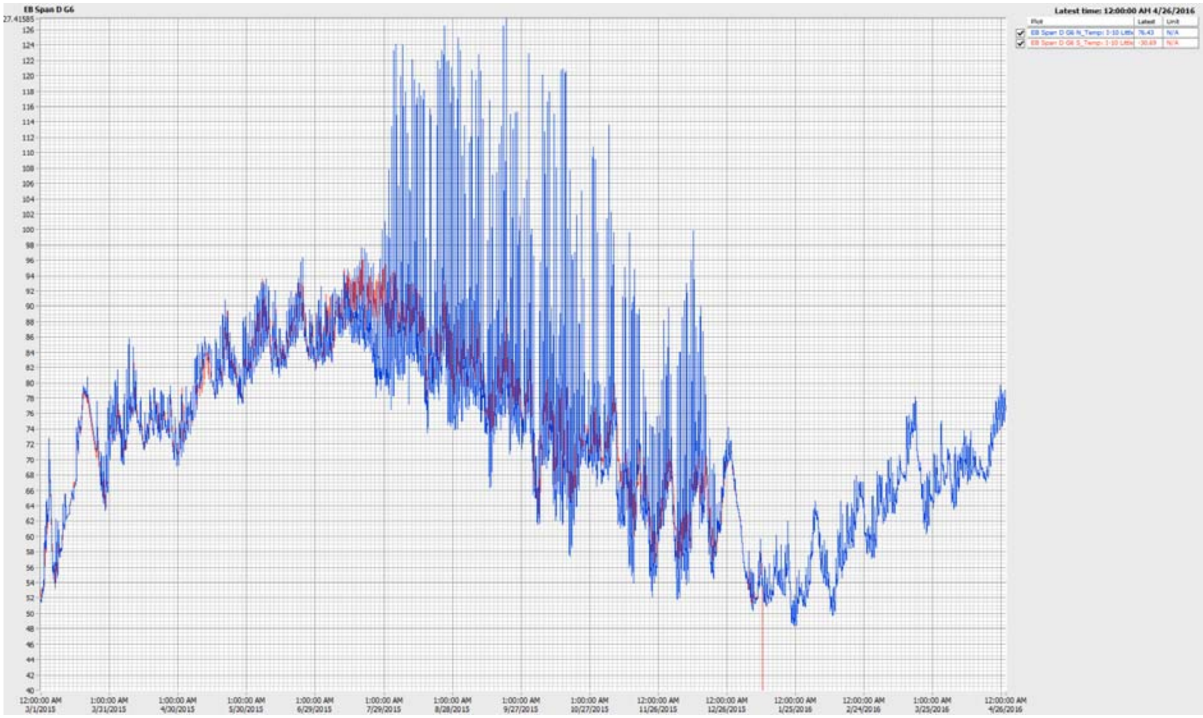

Figure 13
Temperature of the external steel rods for girder 6 (Span D) eastbound

Another interesting comparison is to look at the temperatures at the same side of the girders within the same span. Again, Span D of the eastbound bridge was used for the comparison since this is the span that is equipped with thermistors. The north side temperature readings of Girders 1, 2, and 6 are presented in Figure 14(a). It is very clear from the figure that the temperatures of Girders 1 and 2 were consistently similar. The northernmost girder (Girder 6), however, showed wide range of fluctuations as previously described. This clearly showed that the water level and ambient condition effect on the temperature measurements. It appears that the unexposed (within the shade of the deck) reinforcement experienced much smaller temperature fluctuations and that the temperatures of these reinforcing elements were very close to the water temperature as evident by the observed temperatures near the low end of daily temperatures.

The comparison of the south side temperatures for the girders is shown in Figure 14(b). A few observations can be made from the figure. Firstly, the south facing side of Girder 1 showed even larger temperature ranges (128^o F and 40^o F highest day time and lowest night time temperatures). Unlike the north facing girder, the high daily temperature fluctuations were not limited to summer months only. Part of March and April also show significant daily fluctuations and to a lessor extends in the winter months between November and January. The comparison view of the temperatures measured on the northern and southern sides of the Span D is presented in Figure 14(c) to contrast the differences. The red line represents the temperature on the north side of Eastbound Span D, while the blue line represents the south side of the same span. It is clear that the south side had subjected to greater temperature fluctuations. This observation coincides with the observed damages of the bridge at that span. Note that the repair was done on Girders 1, 2, and 6. Girders 1 and 2 are located on the south side of the bridge. It can be inferred that the bridge deterioration can attributed in a big part to the temperature fluctuations with some contribution from submergence. No temperature observations are available for the other spans. However, one can infer from the above discussions that similar behaviors are expected for the other spans.

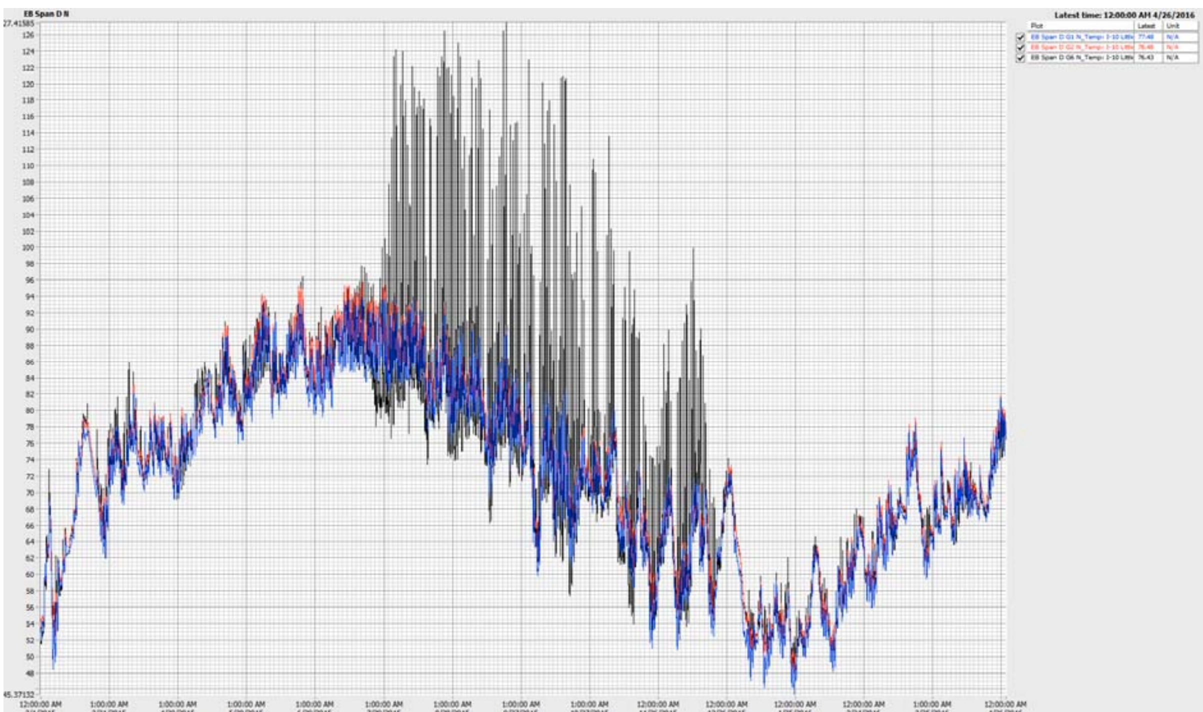

Figure 14(a)
Temperature of the external steel rods on the north side of the girders in Span D eastbound

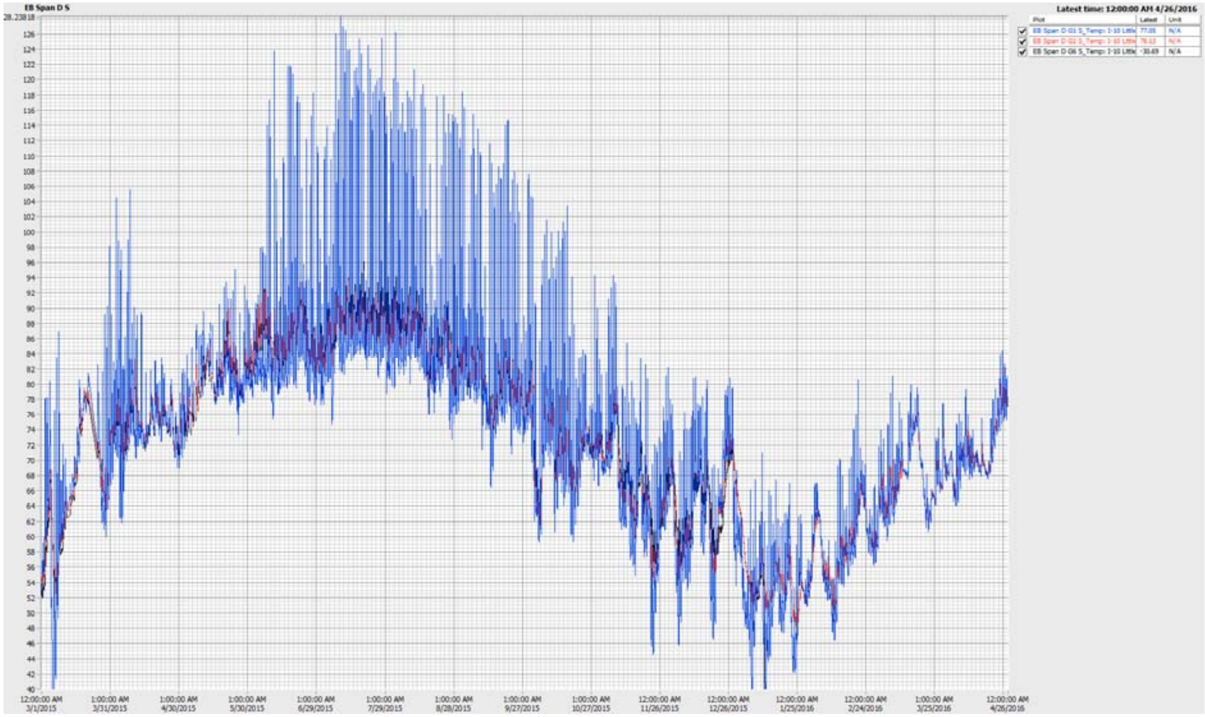

Figure 14(b)
Temperature of the external steel rods on the south side of the girders in Span D eastbound

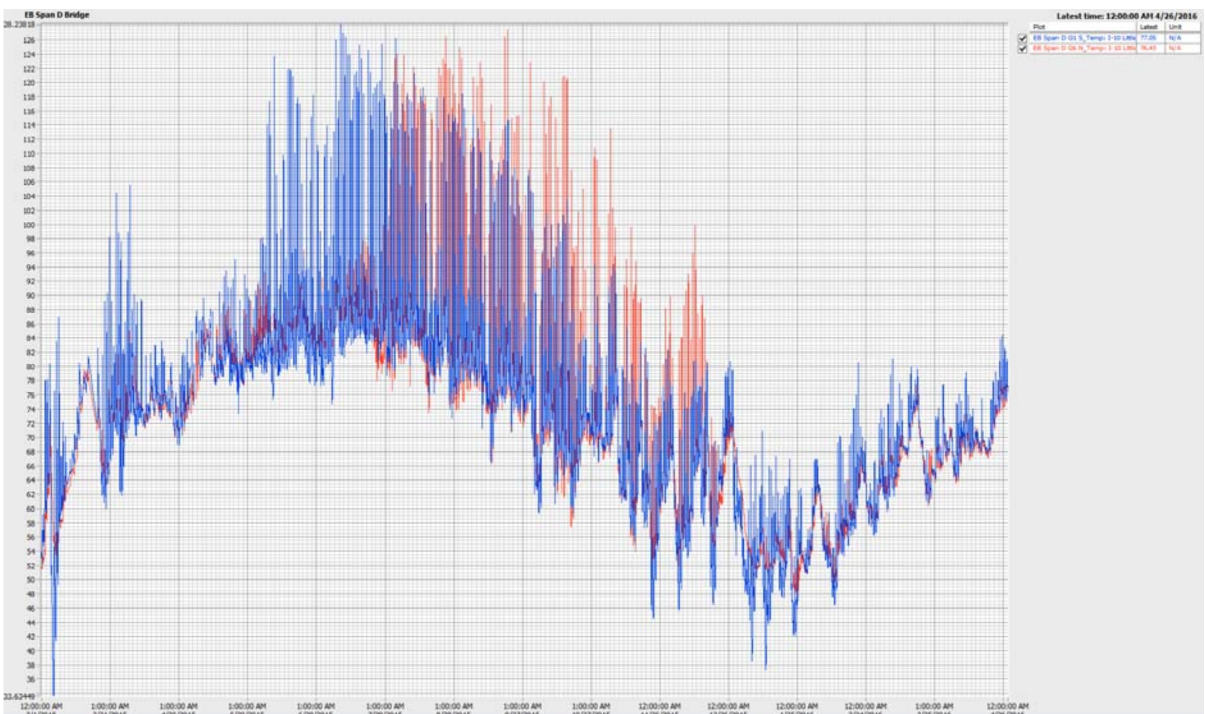

Figure 14(c)
Temperature of the external steel rods in Span D eastbound

Tensions on the Reinforcing Elements

The forces on the external reinforcements are the direct indication of the performance of the repair regardless of the temperature readings. Slow changes or sudden jumps or drops in tensions can indicate creep, deterioration, or sudden changes in the structural cross sections of the bridge element monitored. Creep can occur even without significant observable changes in the structural behavior within the study period, due to the short monitoring time frame. Gradual changes in tensions may be an indicative of creep or slow deteriorations and will likely to have insignificant effect on the short-term bridge load carrying capacity. With careful projection, it may be possible to project the rates of change to determine the likely life of the structure so that a bridge replacement project may be planned. The daily tension variations are most likely due to the result of the ambient and water temperature fluctuations. While the daily variations are very important to the understanding of the behavior of the repaired structure, the primary concerns are the long-term performance and safety of the bridge and the effectiveness of the external reinforcement. Both the long-term and short-term data trends are presented in this section. The implications of these trends are also discussed.

The tension forces on the reinforcing rods were measured using strain gages mounted on two separate locations on each of the steel rods. The original plan was to mount the two gages at the opposite side of the post-tension rods at the same location so that any bending can be properly evaluated. However, the contactor installed the gages at two different locations separated by two intermediate supports. The friction and possible bending at the two intermediate supports resulted in some deviations in tension forces measured on the same external steel reinforcing rod. The frictional forces made the attempts in inferring the results problematic. Therefore, only general observations that have significant consequences are discussed in the following.

Figure 15 shows the tensile force measurements of the Girder 1 Span A (westbound) from March 1, 2015, to April 26, 2016, the same period used for temperature comparison. The yellow and blue lines represent the tensile forces measured from the two strain gages on the steel reinforcing bar at the south side of the girder and the other two lines are the forces recorded from the reinforcement at the north side of the girder. It is clear that the tension forces of the steel bar at the south side are strongly influenced by the ambient conditions similar to the temperature fluctuations (black line) presented in Figure 14(a). The range of the tension force swing within a 24-hour period reaches 7.0 kips, about 30% of the applied tension. Another interesting observation is that the observed tension forces from these two gages converges with time. The tension force difference in early March 2015 was about 1.5 kips. By the end of April in 2016, the gap had narrowed to about 0.5 kips. The north side reinforcing rod was never exposed to the ambient conditions. As such, the temperature fluctuations thus the daily tension force swings measured are much smaller, mostly fluctuating within a range of about 1 kip.

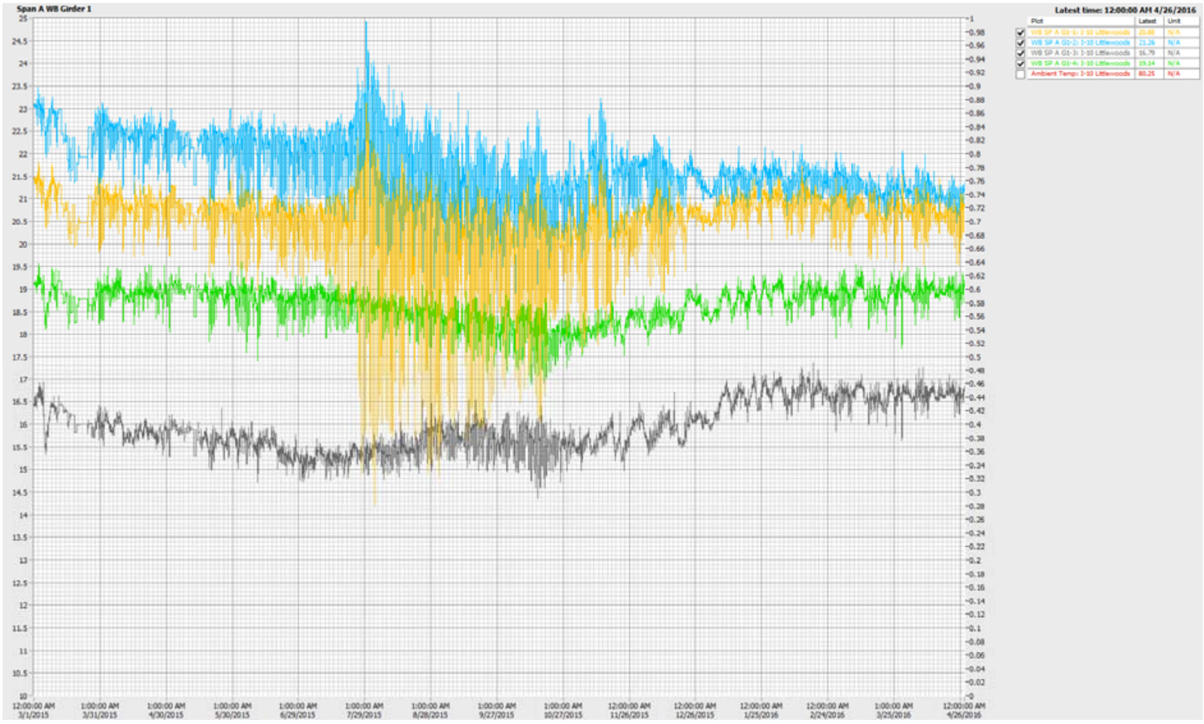

Figure 15
Tensions of the reinforcing steel rods for girder 1 Span A westbound

To further enhance the understanding of the structural response to temperature, the ambient temperatures are superimposed on Figure 15 with the ambient temperatures. The red line presented in Figure 16 shows the ambient temperatures during the monitoring period. It appears that during the first few months of observations (March 2015 to July 2015) the tension forces were relatively stable with only slight daily fluctuations. A sudden increase in tensions was observed on the south reinforcing rod in late July 2015. The tension increase coincided with the water level drop in the Lake Pontchartrain (Figure 17). It is suspected that the water level at the site dropped below the external reinforcing rods and exposed the outside reinforcement to the ambient conditions that in turn caused the large fluctuations of the tension forces as well as the spike in the tension observed in late July 2015. Once exposed, the observed tension forces steadily declined until late September 2016. This period coincides with the water level drop perfectly. Note that the exposure affected the south side of the girder much stronger than the north side. The hypothesis is that the lowering of the water level exposed the girder to the environment and cause the girder bend along the weak axis slightly. Some of this bending recovered once the girder became submerged again as evident by the tension recovery of the south reinforcement. Only small fluctuations were observed on the north side of the girder. The small fluctuation is likely due to the reinforcing rod being under the shade of the bridge deck and experiencing only minimum effect from the ambient temperatures. Based on the Lake Pontchartrain water level observations by the US Army Corps of Engineer at the Lakefront Airport in New Orleans, a portion of the girders submerged again in the water after later September 2015 *[6]*. The time-history plot (Figure 16) of the tensions reflected that change. There were two short periods that the water level dropped to below the girders in February and March 2016. The durations of these two periods appear to be too short to cause significant changes the temperature responses of the girders.

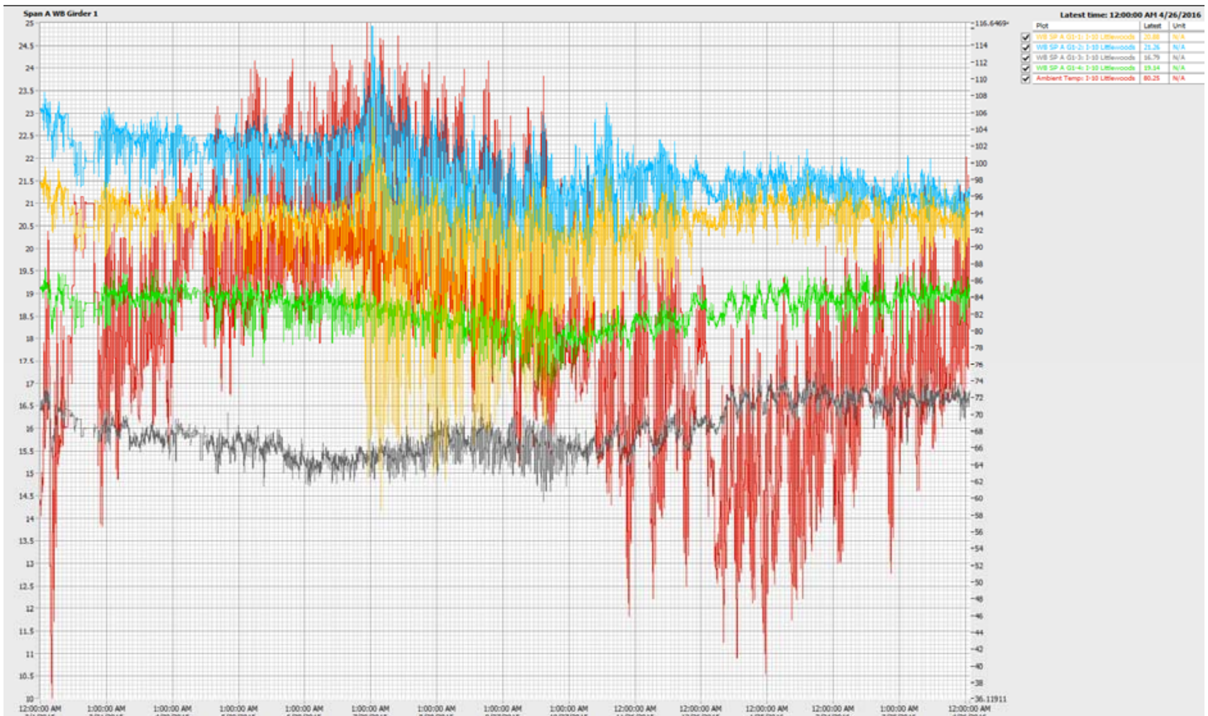

Figure 16
Tensions and temperature of the reinforcing steel rods for girder 1 Span A westbound

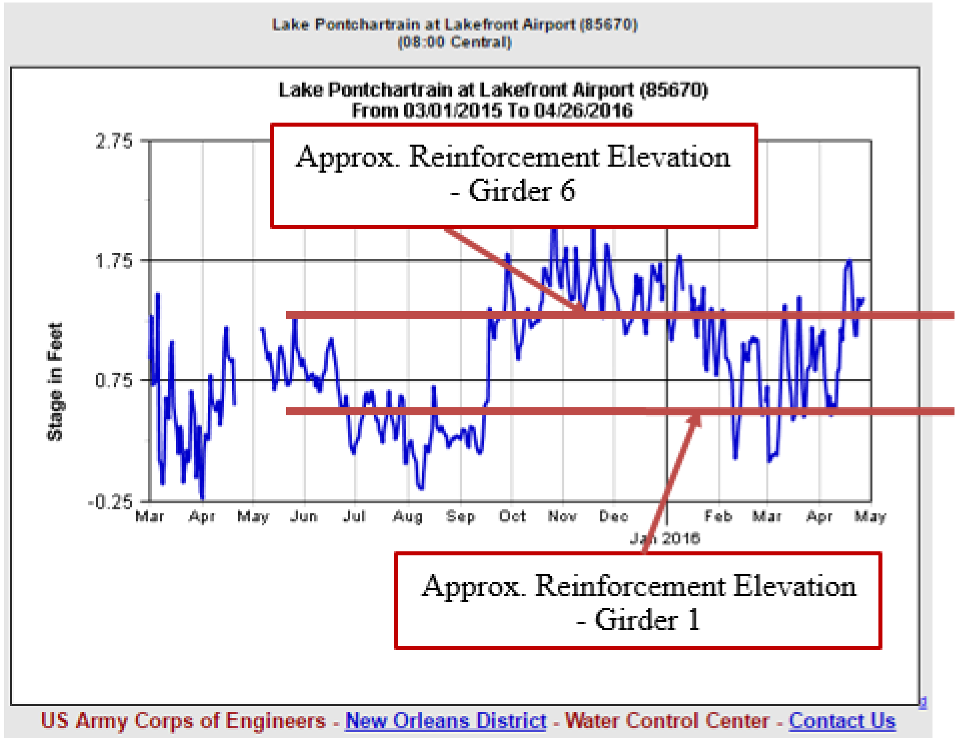

Figure 17
Lake Pontchartrain water level at Lakefront Airport (USCOE)

The next step is to examine the behavior of the northern most girder. Again the tension forces recorded from the external reinforcing of Girder 6 (northernmost girder) of the Span B westbound bridge are presented in Figure 18. The yellow and blue lines represent the south side reinforcing bar and the other two show the tension measurements from the reinforcing bar on the north side. Similar to the behaviors observed from Girder 1, the exposed side (in this case, north) experienced greater tension fluctuations. The magnitude of the daily tension change is about 7 kips, greater the than 50% of the initial tension. Since the deck slopes at 3/16 in. per foot southward, Girder 6 is about 7 in. higher than Girder 1. As such, the north side of the reinforcing bar was mostly above water during the monitoring period except between early October 2015 and February 2016. This is evident from the much longer period of large daily tension swings recorded. It is interesting that the tension forces between the two gages on the south side of the girder diverged starting sometime in July 2015, which coincided with the low water level. The initial difference between the two gages was slightly less than 1.5 kips; by January 2016, the difference increased to about 3 kips, about 20% of the initial tension force. Note that the two gages were installed on the same reinforcing bar. In theory, these two values should be the same if no friction or bending exist. It is possible that friction resulted from the contacts between the reinforcing bar and the intermediate supports (see Figure 19) contributed to a portion of the tension differential. It is likely that bending of the cables also caused the tension difference. It is speculated that the divergence was a result of the stress redistribution due to high temperature deviation between the inside and outside of the exterior girders causing the girder to warp slightly and resulting the increased friction between the support and external steel rods.

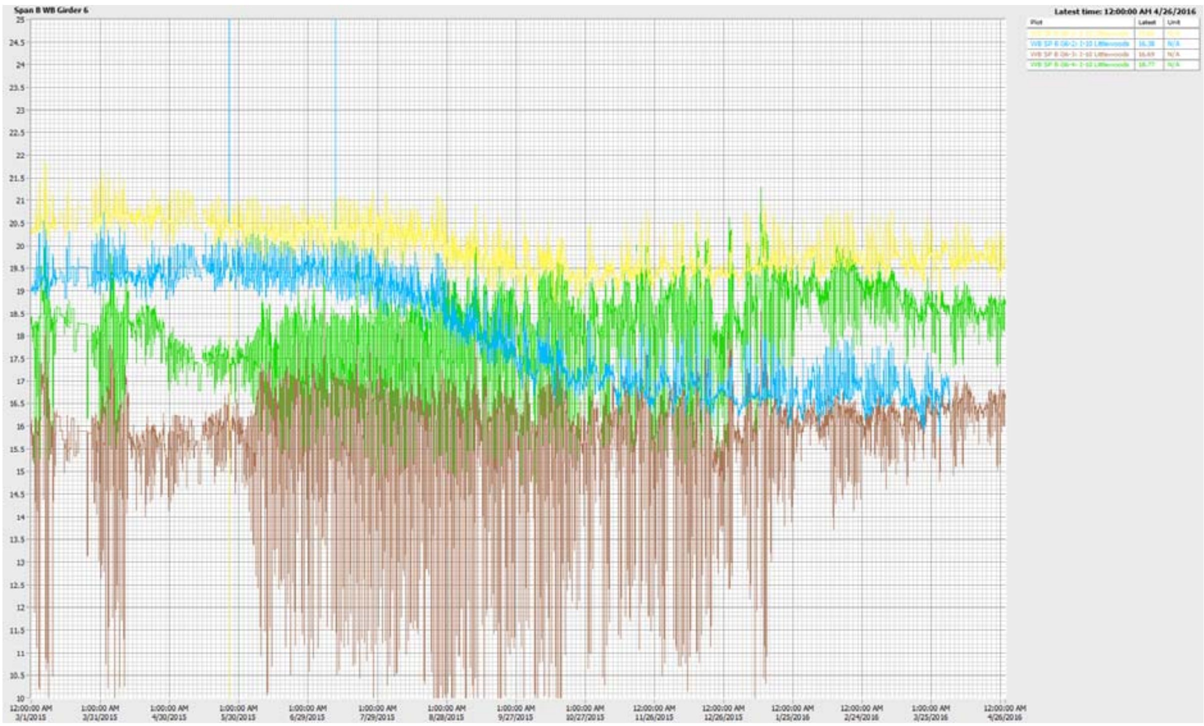

Figure 18
Tensions of post-tensioning rods for girder 6, Span B westbound

The above discussions are based on the observations from the tension force measured from the same side of the girder. No significant changes in tensions observed on the north side of the girder even with large fluctuations of the tensions during the summer months.

Also of interest is the tension difference between two sides of the same girder. Figures 15, 16, and 18 show the tension measurements of the Spans A and B of the westbound bridge. As can be observed in these figures, the long-term trends of the tension differences for these spans indicated the narrowing of the tension gaps between the two sides of the girders, also an indication that the stress is be distributed to the entire bridge structure.

Figure 19
Intermediate external reinforcement supports and strain gage locations

Figure 20 shows the tension difference between two sides of the same girders of the Span A of the westbound bridge. As discussed previously, the two exterior girders showed large tension fluctuations due to exposure to the ambient conditions. Both exterior girders (blue and pink lines) also show a reduction in tension differences with time. The interior girder (black line) show only minimum daily fluctuations. However, the long-term trend of the tension difference on this interior girder appears to be increasing at rate of about 1 kip/year. If the rate of deterioration remains the same, the girder should be sufficient the resist the lateral bending before the scheduled bridge replacement.

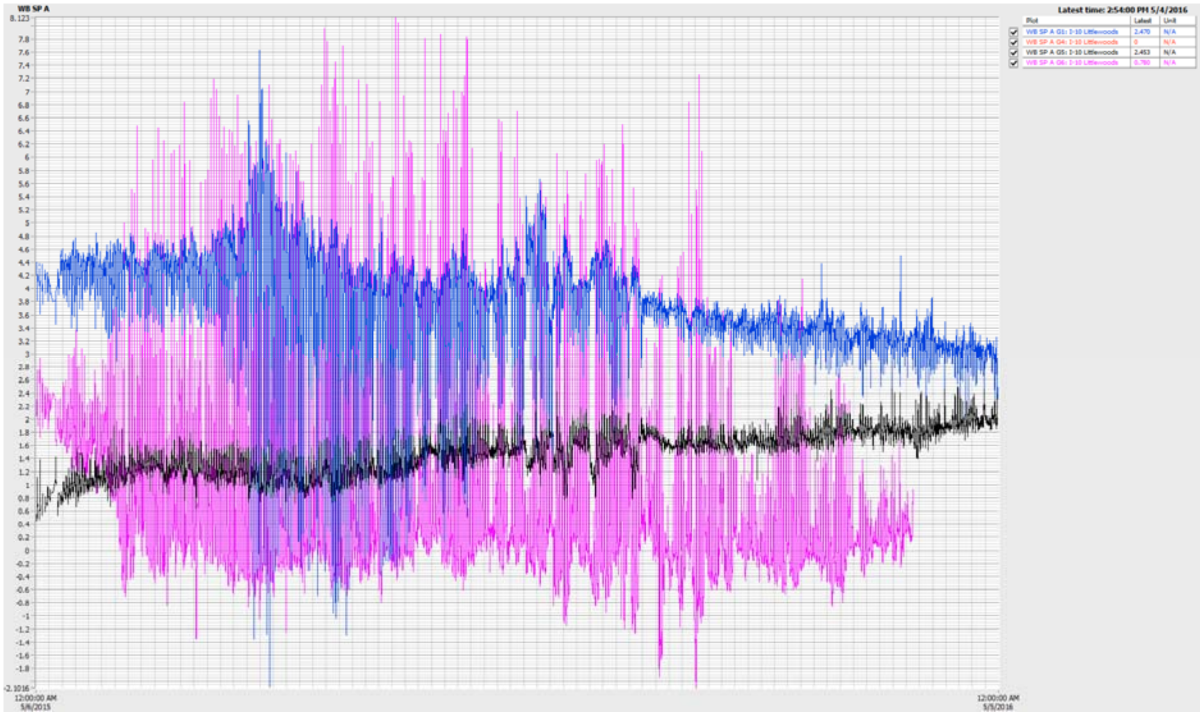

Figure 20
Tension differential between the sides of girders 1, 4, 5, and 6, Span A westbound

Similar behaviors have been observed from the other steel reinforced spans. For example, the tension differential of Girders 5 and 6 of the Span B eastbound bridge are shown in Figure 21. While the trends of the differentials showed slight changes, the long-term differentials are still much smaller than the daily fluctuations. This trend implies that the daily fluctuations are more critical than the long-term trend observed to date.

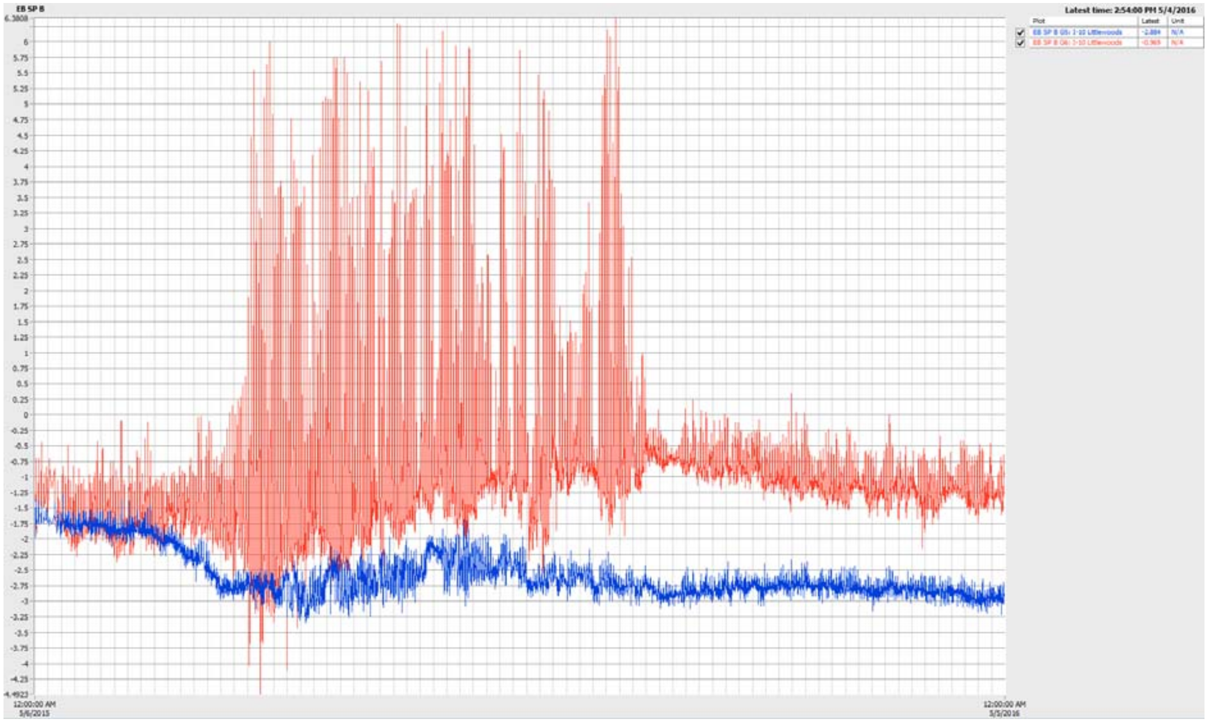

Figure 21
Tension differential between the sides of girders 5 and 6, Span B eastbound

The previous discussions are from the observations of the behavior from the steel rods. In order to provide a comparison, the discussions pertaining to the CFCC reinforced span (Span C eastbound) are presented next.

It should be noted that the tension force and temperatures measurements were made from the load cells installed at the east end of the girders only. In addition, the CFCCs are insulated from the ambient conditions and are less susceptible to temperature changes even for the exterior girders. This phenomenon can be illustrated by comparing the temperatures at the load cell locations of the exterior girder in Figure 22. It is very clear that the temperature of the CFCCs on both sides of the girders were almost identical even though the outside of the girder was exposed to the ambient conditions when water level dropped below the CFCC during the summer months. Another interesting fact is that, while the CFCCs are insulated, the load cells that measure the tension forces and temperatures are not. Even with the exposure, both the temperature and tension force fluctuations are much smaller than those observed from steel. Note that the small temperature effect on the CFCCs does not mean the temperature responses of the concrete girders would also be small. In fact, the differential thermal responses between the CFCCs and concrete could in theory generate stresses in the girders that are totally different from steel reinforcement. Unfortunately, the uncertainties of the structure conditions made the comparison difficult to positively identify the causations of some observations. As an example, Figure 23 shows the tension forces of the CFCCs for Girder 1 of the CFCC reinforced span. It is clear that once the water level dropped to below the reinforcement level, the force in the CFCC on the exposed side (blue line) started decline. As soon as the water level recovered, the force also gradually recovered. The CFCC on the protective side appears to respond differently. The force reduction was much smaller and gap of tension forces between the two sides never recovered.

Figure 22
Temperature of CFCC for girder 1, Span C eastbound

The daily maximum tension force fluctuation was only about 0.4 kips. This is in contrast to the 7 kips daily fluctuation of the steel rods. The long-term rate of tension increase on an annual basis is about 2 kips. This is very significant. The implication that the internal prestressed tendons are transferring equivalent of the 2 kips of prestress force to the external reinforcement every year. The force transferring could be the result of bonding loss of the prestressed strands due to concrete deterioration or steel corrosion.

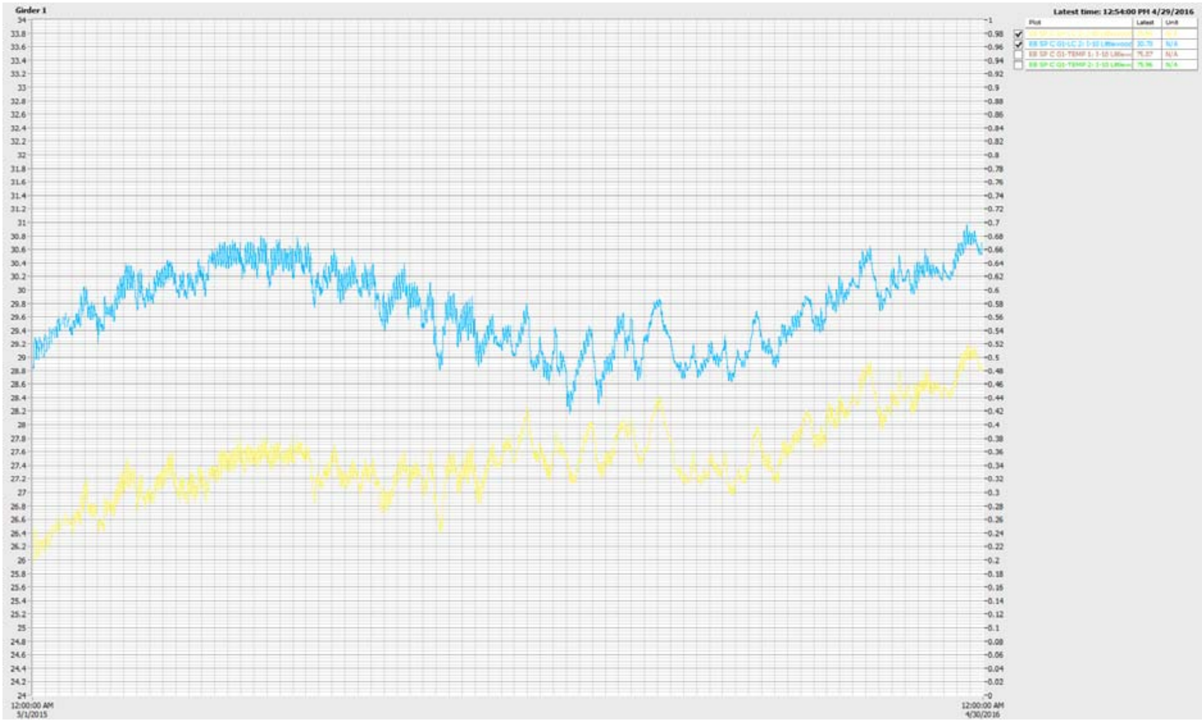

Figure 23
Tension forces of CFCC for girder 1, Span C eastbound

All girders of the Span C Eastbound Bridge experiences some deterioration. Table 4 summaries the load transferred to the CFCCs. Generally, the load transfer rates range from 2 to 3 kips per year.

Table 4
Load transferred rate to CFCC

Girder No.	Load Transferred to CFCCs (kips/yr)
G1	2.0
G2	2.8
G3	5.5
G4	2.7
G5	2.6
G6	3.0

It is obvious that Girder 3's behavior differs significantly from the others within the same span with a load transferring rate of 5.5 kips/year, nearly double the rates of the other girders. The high rate of load transfer indicates a potential problem for that girder.

In addition to the rate of load transfer to the external reinforcement, the force differentials between two sides of the girders are also of concern since they are the symptoms of uneven deterioration. They also cause warping or bending of the girder along the weak axis. Figure 24 shows the differential forces of the CFCCs between two sides of the girders for the CFCC reinforced span. It is interesting to observe the two external girders' (blue color – Girder 1 and green color – Girder 6) behavior. They are almost mirror images of each other. Girders 4 (pink) and 5 (tan) generally remained consistent throughout the monitoring period except for an event occurred on March 20, 2016, that caused a small jump (0.3 kips) in tension differential. It is postulated that the sudden change was due to redistribution of the stress within the girder from the prior stress build up. Due to the small magnitude of the jump, it should not have any material impact on the safety of the bridge. The data from Girders 2 and 3 presented the most significant change in the girder performance. Between mid-July and early November 2015 during the period of low water level, there observed a significant change in the tension differentials between the two sides of these girders. The large change suggests the girder exposure sped up the girder deterioration. The combination of the high warping force mentioned above and the high rate of load transfer from the internal prestressed strands to the external reinforcement of Girder 3 could potentially disrupt the bridge service and reduce the potential bridge life.

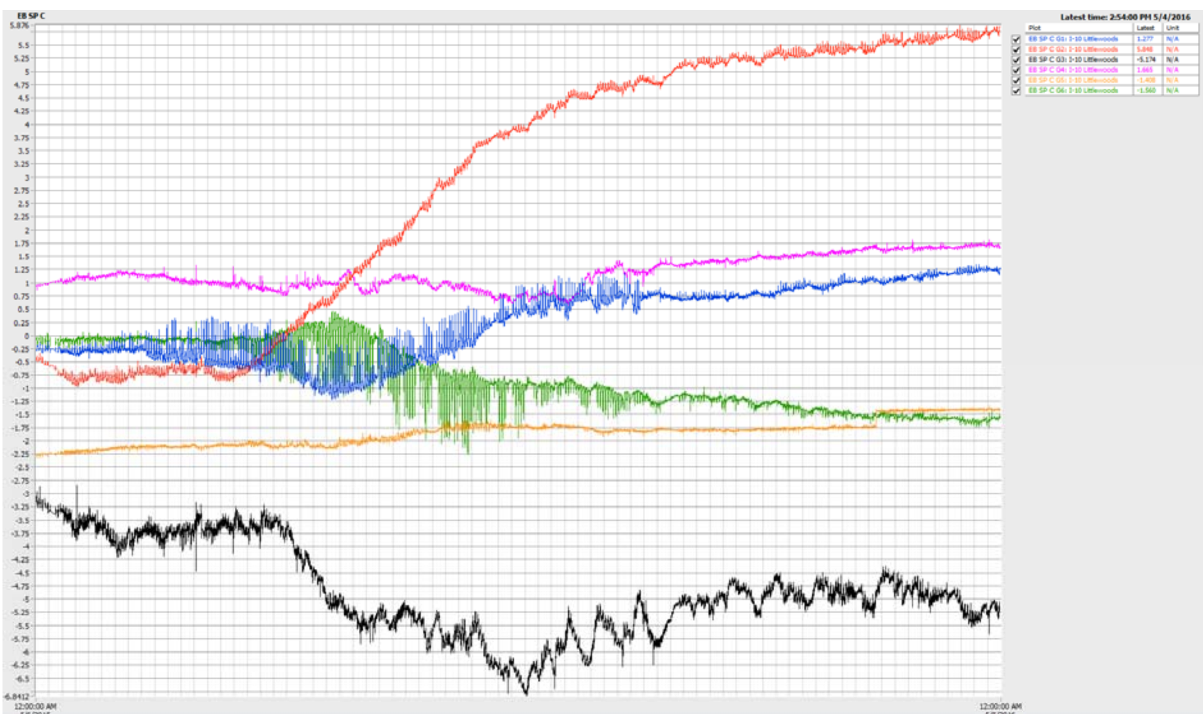

Figure 24
Tension force differential of the CFCC reinforced girders

CONCLUSIONS

This project explored using only external post-tensioning elements to extend the life of a heavily deteriorated bridges. To study the effects on the external reinforcement, two types of material were used: steel and CFCCs. The findings can be separated into two categories: structural safety and material behavior differences.

The foremost concern of this project is the structural safety. Based on the observations from slightly one year's worth of data, the following conclusions can be drawn:

- The current state of the bridge appears to be safe for the traveling public.
- Even after the repair, the bridge structures continued to deteriorate.
- The rates of deterioration for the steel reinforced girders are insignificant.
- There is no indication that the external steel reinforcing bars have experienced any corrosions with the monitoring period.
- Based on the data, minor warping (bending not along the direction of the maximum resistance) has been observed in some girders.
- It appears that some girders at CFCC reinforced span deteriorated at a higher rate than the other monitoring girders. It is possible that the difference between the CFCC reinforced girders and steel reinforced girders is due to the pre-existing conditions.
- Most severe warping was also observed on Girders 4 and 5 of the CFCC-reinforced span.

Due to the highly variable conditions of the bridges, it is not possible to attribute some of the behavior differences to any specific material. However, the following conclusions can be drawn based on the data recorded.

- With the two layer of insulations for the CFCCs, the temperature effects on the CFCCs are insignificant.
- The external steel reinforcing bars are strongly affected by the ambient temperatures.
- The behavior differences between the steel and CFCC may be attributable to the temperature responses and structural conditions.
- There is no evidence of corrosion of the external reinforcing steel during the monitoring period. Note that, as of time of this report, the reinforcement system had only been installed for less than 2 years. The long-term effectiveness of the galvanization on the steel cannot be determined with such a short-term observation period.
- The relative short observation period precludes the determination of the suitability of either material.

RECOMMENDATIONS

Again, the primary objective of this project was to monitor the bridge performance to ensure the safe operation of the bridge. A second objective was to offer performance comparisons between the two external reinforcement materials: steel and CFCC.

Based on the data collected, the bridge appears to be safe at the current state. However, indications of continuing deterioration have been observed. It is prudent that the performance be continuously monitored so the public can be warned if any significant condition changes occur and DOTD can prepare a contingency plan to ensure the safety of the traveling public.

A major concern of the external steel reinforcement is the corrosion. As previously discussed, there appears to be no observable corrosion. For long-term performance of the galvanized steel rods, continuing monitoring is recommended.

The CFCCs appear to be functioning as designed. Due to the double insulations and varying deterioration rates of the girders, it is not possible to compare the performance with the steel rods directly. However, based on the performance observed, the CFCCs have effectively provided the reinforcement needed to support the structure. Because of their lack of flexibility in the anchoring, it is recommended that the future adoption of CFCCs should involve the manufacturer early in the design process to avoid construction problems.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
CFCC	Carbon Fiber Composite Cable
DOTD	Louisiana Department of Transportation and Development
FHWA	Federal Highway Administration
ft.	foot (feet)
in.	inch(es)
LTRC	Louisiana Transportation Research Center
lb.	pound(s)
m	meter(s)
USACE	US Army Corps of Engineers
US	United States

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