Assessment of an instrumented reinforced-concrete bridge with fiber-reinforced-polymer strengthening

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Abstract. Field instrumentation is investigated on an in-service highway bridge over a 2-year period. Extrinsic Fabry-Pérot interferometric (EFPI) strain sensors provide a permanent health-monitoring capability. The bridge is a reinforced-concrete (RC) structure that was repaired and strengthened using fiber-reinforced-polymer (FRP) wraps. A sensor network monitors the load-induced strain in the FRP reinforcement and the steel rebar. Colocated electrical resistance strain gauges and a finite element analysis are used for comparison. Both dynamic and static load characteristics are analyzed for a near-capacity truck. The fiber optic measurements are generally consistent with the comparison measurements and the analytical results; and they show no failure or degradation as opposed to the electrical resistance gauges. We demonstrate the implementation and the performance of in situ EFPI sensors in a long-term field environment. Embedded fiber optic sensors can provide the required information for the intelligent management of a transportation infrastructure. © 2007 Society of Photo-Optical Instrumentation Engineers.

Subject terms: metrology; fiber optic sensors; nondestructive testing; smart structures; strain analysis.

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1 Introduction

Smart structures technology can provide required capabilities for the management, maintenance, and inspection of civil engineering structures. Whether a new or aging infrastructure, the intelligent assessment of conditions for instrumented structures can be performed quickly and efficiently. Embedded hardware and software can integrate sensing, processing, and intelligence. Possible applications include identifying specific structural health conditions, regular reporting from on-site decision-making software, and acquiring raw data for future processing or reference. Also, management of the instrumentation must be considered. Aspects in the decision making on the structure include failing structural elements, failing sensors, low battery or power outages, and other such information to enable notification that the structure or instrumentation is in need of repair. Progress toward intelligent smart structures involves component technologies, integrated systems, management needs, installation protocols, and field validation.

Health monitoring technology for civil engineering infrastructures was developed during the last decade. Smart structures instrumentation consists of permanent, integral sensors to measure key parameters such as strain and deflection. Benefits include enhanced understanding of in-service conditions, verification of repair or upgrades, and improved management of service life. A current area of research is field validation of sensing techniques using in-service structures. Fiber optic strain sensors can provide field measurements related to load performance and structural health and have advantages of environmental ruggedness, low profiles, and high sensitivity. Such fiber optic sensors offer excellent sensitivity, durability, and embedability and are virtually immune to electromagnetic (EM) interference. Embedded fiber optic sensor networks are good candidates for long-term installations where durability, extended life expectancy, and long term accuracy are required. The Fabry-Pérot cavity gives a highly responsive and noise-free sensing device that performs well in field conditions. Fiber optic sensors have been used in fiber-reinforced-polymer (FRP) composite, reinforced-concrete (RC), and steel structures to monitor internal strains for selected locations and components.

In a multi-year study sponsored by the Missouri Department of Transportation (MODOT), an aging RC bridge was strengthened with FRP, carbon fiber rebar, and steel-reinforced polymer. This in-service bridge is designated P-0962. Monitoring the bridge includes smart sensing systems and other nondestructive testing (NDT) methods to assess the strengthening as the bridge ages in field conditions. The sensing network also demonstrates developments and protocols for field instrumentation.

This paper presents the field assessment of the strengthened MODOT P-0962 bridge using the in situ sensing network. The performance of extrinsic Fabry-Pérot interferometric (EFPI) fiber optic strain sensors was investigated. The sensor network detected flexural strain at point locations in the steel rebar and in the FRP reinforcement. Colo-
cated electrical strain gauges were used for comparison. Periodic load testing per H-20 truck American Association of State Highway and Transportation Officials (AASHTO) standards\(^{29}\) showed both dynamic and static behavior of the sensing network and the bridge. Loads during the tests were applied for all three lanes of the bridge. Also, a finite-element analysis (FEA) is given for comparison. The fiber optic measurements were generally consistent with the comparison gauges and the FEA results. The fiber optic sensors displayed no significant aging effects. None of the fiber optic sensors failed since the network was installed in 2003. The measurements demonstrate the capabilities of \textit{in situ} health monitoring and give insight into the behavior of the strengthened bridge. The instrumentation provides a field test bed for future smart monitoring studies. The paper addresses the feasibility of long-term EFPI monitoring networks, a field comparison of EFPI and traditional strain gauges, and approaches for tracking structural health.

2 Bridge Description

Bridge P-0962 is a three-bay structure serving a rural two-lane highway in MODOT District 8 close to Interstate 44. The 49-yr-old monolithic reinforced concrete construction was showing signs of deterioration and was selected for rehabilitation. Structural strengthening enabled the bridge to meet increased use and current traffic loadings without the roadway interruption and cost of new construction.

Repair and upgrade procedures used for the structure were conventional repair (including steel rebar and concrete patches), FRP and SRP reinforcement (both wraps and rods), and health monitoring with \textit{in situ} sensors and other NDT methods\(^{20,25–28}\) Figure 1 shows the structure after instrumentation and repair work was completed. FRP reinforcement wraps are visible on the support girder.

The \textit{in situ} strain-sensing network was installed for postrehabilitation monitoring for the central span of the structure. This permanent network facilitated load testing for the life of the bridge and evaluation of the smart sensing technologies. The sensors targeted strain in the steel rebar and the FRP wrap of the center span. This span used only the FRP wrap reinforcement. Note that the sensor network was embedded in the structure and was permanently covered by FRP fabric and concrete.

3 Assessment Program

Structural assessment of the bridge involved both the sensing network of the fiber optic and electrical resistance strain gauges (ESGs) and the theoretical estimate of the strain performance using FEA. Scenarios are given for the load testing along with data-filtering and processing details.

3.1 Strain Monitoring Network

An EFPI sensor array alongside an array of ESGs was installed on P-0962 in summer of 2003. Use of this sensor network is primarily intended for confirmation of the structural strengthening and for monitoring performance over time. Research issues include comparison of the EFPI sensors to the ESGs, comparison of the EFPI sensors to the FEA results, accommodation for the field application (for example, aging and failure), and processing necessary for intelligent monitoring.

Figures 2–4 show the sensing network locations. The sensing network monitors strain at the midspan in the center bay of the bridge. The bridge runs north-south and the network was installed on the western side of the bridge. Two sensor locations are on the longitudinal beam. Two EFPI sensors and an ESG measure longitudinal strain within the layers of the FRP wrap (location designated LW). Colocated sensors measure longitudinal strain in the FRP wrap reinforcement.
internal steel rebar at the bottom of the beam (location designated LR). Two additional sensor locations are on the underside of the road deck midway between the longitudinal beams. Colocated sensors measure transverse strain within layers of the FRP wrap (location designated TW). Sensors measure transverse strain in the internal rebar at the bottom of the deck (location designated TR). In each sensor location, the two EFPI sensors and the ESG are colocated within three linear inches of each other. Sensor installation was attempted on the concrete underneath the paved wear surface of the bridge; these sensors behaved unreliably (for example, tensile and compressive strain for similar loadings and complete ESG unresponsiveness by the second load test) and are omitted from the tests results. All sensor cables were embedded and terminated in a control box, as shown in Fig. 3. Instrumentation for the EFPI sensors and the ESGs were patched through the control box during the load tests.

Preparation of the surfaces and the installation area was critical for proper installation. Removal of concrete was performed in areas to expose rebar along with marking and cutting of the cable lays, as shown in Fig. 4, to embed the sensor leads. Strain gauge application surfaces were cleaned to ensure good bonding between the sensors so the material stresses were properly transferred for detection. The leads for the ESGs were insulated with a mastic material to ensure that no shorting occurs to compromise their operation. Next the leads were placed in the cable lays using the same mastic material for temporary positioning until the leads were covered by a concrete epoxy. EFPI installation was similar, excluding the step of attaching leads to the sensor. EFPI sensors have a specified length of fiber attached to the sensing head. The fiber was laid into cable lays with the ESG leads and then the epoxy was applied to seal the sensor leads from weather and vandalism.20

3.2 Strain Instrumentation

Both EFPI sensors and ESGs were employed. These systems have good directional properties and are effectively point sensors. The fiber optic sensors can be embedded within concrete or FRP materials with more ease and longevity than the ESGs. The EFPI sensors have a smaller profile and can be embedded into tighter places. EFPI systems provide much more accuracy and potential measurement bandwidth than the ESG systems as well. The accuracy and bandwidth of ESG sensors are dependent on the construction of the amplifier and filtering circuitry. EFPI systems also are dependent on electronic circuitry; however, the actual sensor resolution and bandwidth are dependent on fiber optic parameters. Resolution, therefore, is dependent on the wavelength of the fiber optic systems. The EFPI bandwidth is dependent on the processing speed of the instrumentation. The normal processing required for these systems is digital-signal-processing (DSP)-based and therefore the bandwidth will be dependent on the speed and bit resolution of the DSP subsystem.

In an EFPI system, an optical interferometric relationship inside of an air cavity is related to the strain of the sensor. Figure 5 illustrates the physical construction and the detection instrumentation, respectively. In the sensor construction, a cavity is formed between two polished fiber end faces. Multiple-beam interference in the reflected optical signal is modulated by changes in the cavity length $\Delta d$ through the associated strain.15,21,30–35 The gauge length is roughly the length of the capillary alignment tube $L$. The sensor instrumentation consists of a broadband light source, the optical fiber for transmission, and the detector. The absolute strain can be demodulated by observing the reflected irradiance signal as a function of source wavelength. Equation (1) uses the parameters of the reflected irradiance $I(d)$, cavity length $d$, and an operator that tracks the nonlinear periodic modulation function $f(\cdot)$. The strain is

$$\varepsilon_{\text{FOSS}} = \frac{\Delta d}{L} = f[I(d)].$$

A commercial fiber optic sensor system from Luna Innovations (model Fiber Pro) was used to demodulate the sensor.
readings for the EFPI sensors. The Fiber Pro is capable of 1000-Hz sampling and submicrostrain resolution. Through a computer interface, the information is captured to a text file for further processed with Excel or MATLAB. In this paper, Model AFSS high-finesse EFPI sensors from Luna Innovations were used with a wavelength of 830 nm and a gauge length of approximately 8 mm.

In ESG systems, the strain is detected by deformation in a resistive grid element. This technology is mature and an ESG is typically interrogated with a Wheatstone bridge and amplifier circuitry. For this project, a dedicated UMR multichannel system was used. The PC-based National Instruments data acquisition operated at a 1000-Hz sampling rate on each channel. Posttest signal processing was done on the recorded strain, for example, filtering. The ESGs used in the bridge are Model EA-06-250BG-120 electrical resistance gauges from Micro-Measurements Company. These gauges have a nominal resistance of 120 Ω and gauge length of 6.35 mm.

### 3.3 FEA Modeling

An FEA was performed to model the expected performance of the strengthened bridge for a static load test (see Ref. 27 for details on the bridge geometry). The model consisted of 3D solid elements that represented the deck, girders, and parapets acting as a composite structure. The FEA was implemented by using the commercial software Abaqus version 6.4. A linear elastic analysis was performed for the load configuration of the static load test performed on the middle bay of the bridge. The finite element used for the simulations was the so-called “C3D8,” which is a continuum (solid) 3D element that has eight nodes with three active degrees of freedom (DOFs) per node. The active DOFs of this element are the translations along each of the global coordinate axes the (x, y, and z axes). The concrete of the deck, girders, and parapets was modeled as having an $f_c'$ equal to 6850 psi. The modulus of elasticity of the concrete was defined by Eq. (2). The Poisson ratio ($v$) was assumed to be 0.20.

$$E_c = 57,000 \sqrt{f_c'}.$$  

The FEA loads were modeled as concentrated loads per the experimental truck locations. Figure 6 shows the dimensions of the H-20 trucks used for load testing bridge P-0962. Longitudinal and transverse strains were determined at the LW, LR, TW, and TR locations for the beam and the deck, respectively. Figure 7 shows the resulting contour plot of the longitudinal strain (direction parallel to the axis of the girders) of the bridge superstructure.

### 3.4 Load Tests

Two loading scenarios were used for this study: a dynamic rolling load and a static load. After bridge strengthening and sensor network installation in 2003, the load tests were conducted on November 10, 2004, and October 20, 2005. Load tests were conducted using absolute strain values relative to an immediate zero-load reading, thus removing the necessity for temperature compensation. Dynamic rolling loads were performed with a full-size pickup as well as a full-scale H-20 load test vehicle. These dynamic tests were used to check the operability and sensitivity of the sensor network as well to provide a baseline of the dynamic behavior of the bridge. Static loadings were performed using a full-scale loading vehicle with the heavy axle over mid-span, as described in the AASHTO standard. This type of load is used to evaluate the strength of the structure with respect to time, that is, to assess the strengthening and aging.

#### 3.4.1 Dynamic loading

Initial rolling load tests where performed with a full-size pickup. These tests showed that the EFPI sensor network was performing at 11.25% of the bridge load rating (16,014 N, 3600 lb). The EFPI sensors showed clear patterns for peak strains less than 10 μ strain.

Dynamic loading consisted of passing a weighted H-20 vehicle over each lane of traffic in the proper direction and then over the center lane in both directions. Figure 8 shows the lanes of travel on the structure where WL-SB is the west lane south bound, EL-NB is east lane north bound, and CL is the center lane of travel. The sensor network captured the strain profile that was generated by the rolling loads in each lane of travel. Figure 9 shows the full-scale loading vehicle that was used for both the static and rolling load tests.

Rolling load tests were performed by passing the test vehicle over the structure at a slow speed to ensure that sufficient strain detail was sampled. This test enabled assessment of dynamic structural behavior. This type of information was more realistic than that of a static load, but it is difficult to capture when a vehicle is moving at a normal...
For realistic velocities, the sampling rate on the EFPI system must be greater than that available at the time of this work.

In Fig. 10, the initial rolling load test of rebar strain taken at a low sampling rate shows that the sensor array was active and sensitive to light loads, for example, 11.25% of the AASHTO load. The EFPI instrumentation had a minimum resolution of 1 μstrain and the ESG system data were filtered, as discussed in the next subsection. Figure 11 shows the rebar strain for a transversely mounted sensor from the full-scale H-20 vehicle. The signature shows both the front and rear axle contributions to the vehicle in the two strain peaks.

3.4.2 ESG data filtering

Filtering of ESG sampled data was employed to remove unwanted noise due to oversampling or outside interference. A second-order discrete time Butterworth filter was calculated and used to filter the data during postprocessing. The transfer function of the filter is shown in Eq. (3). Examples of strains before and after filtering are shown in Figs. 12 and 13 for the sensors at location LR. Note that the longitudinal beam did not show the double-peak pattern. In Fig. 12, ESG results are shown with no postprocessing performed on the data and the EFPI system results. Figure 13 shows the filtered results alongside the EFPI system results. Note that no filtering was required for the EFPI data.

\[
H(s) = \frac{s^2 - 1.956s + 0.9565}{0.0002414s^2 + 0.0004827s + 0.0002414}.
\] (3)

4 Results and Analysis

The load-induced strains were measured using selected EFPI and ESG sensors. Note that all EFPI sensors were checked after initial installation. In a few cases, one sensor...
did not survive installation and there remained only one EFPI sensor in a particular location. Colocated EFPI sensors were in close agreement and a primary EFPI sensor was selected for use during testing. Some ESG sensor data was unavailable due to faulty interconnections in early tests and due to apparent sensor failure in later tests. Due to EFPI instrumentation and load-test logistical limitations, only the selected EFPI sensors were used in these tests. Long-term comparisons of the redundant sensors are the subject of future testing.

Load testing of the structure consisted of dynamic and static loading scenarios. Data for the November 10, 2004, and October 20, 2005, load testing sessions are shown to illustrate information derived from the field activities. Table 1 shows the H-20 truck load ticket information for various load tests. This information was used in conjunction with the peak strains for determination of the health of the structure. Note that the front and rear axle weights differ by as much as 133.447 kN (30,000 lb). This nonpoint loading can also be seen in Fig. 11 by the difference in the two peaks that were due to the separate axles. The temperature was somewhat different for the two days.

### Table 1 Load ticket information and temperature from testing.

<table>
<thead>
<tr>
<th>Date</th>
<th>Front Axle [kN (lb)]</th>
<th>Rear Axle [kN (lb)]</th>
<th>Gross Weight [kN (lb)]</th>
<th>Temperature [°C]</th>
</tr>
</thead>
<tbody>
<tr>
<td>11/10/04</td>
<td>74,374 (16,760)</td>
<td>196,611 (44,200)</td>
<td>271,164 (60,960)</td>
<td>65.5</td>
</tr>
<tr>
<td>10/20/05</td>
<td>75,620 (17,000)</td>
<td>218,497 (49,120)</td>
<td>294,116 (66,120)</td>
<td>71.1</td>
</tr>
</tbody>
</table>

### 4.1 Rolling Load

Results of dynamic loading for the P-0962 structure are shown in Table 2. Peak strains were determined for the EFPI and filtered ESG signals. For the transverse deck which produced a double-peak pattern, the largest peak strain was tabulated. The rolling loads were produced for the three lanes CL, EL, and WL for either a south-bound (SB) or a north-bound (NB) H-20 truck. In the second test, the selected ESG signals increased in noise and contained multiple outlier points.

### 4.2 Static Load

Static load results for averaged and filtered strains over approximately 30-s intervals are shown in Table 3. The H-20 AASHTO truck was positioned with the heavy axle over the midspan in the center bay. The ESG at location TR completely failed between the first and second load tests and the ESGs at locations LR and TW produced questionable values for all measurements in the second load test. The EFPI sensors that were operating after installation gave reasonable results in all tests.

### 4.3 Discussion of Load Tests

The load tests gave insight into the behavior of the bridge and performance of the sensor network. The bridge has been strengthened by the FRP wrap and has a higher load rating. Comparison of EFPI strain measurements between the 2004 and 2005 tests shows that the strains in the longitudinal beam were more consistent than the strains in the transverse deck and that the static strains were more consistent than the dynamic strains. The deck was presumably more sensitive to load position than the longitudinal beam and the load placement was much less certain for the dynamic tests than for the static tests. Also, the rebar locations do show some variation even for the EFPI sensors, but this behavior was not unexpected due to load redistribution as the concrete cracks with time. Structural health can be quantified using the strain monitoring. For instance, a significant change in static strains would indicate structural degradation.

The EFPI sensors show consistent performance during the two years of testing. All EFPI measurements were reasonable. No EFPI fiber optic sensor that was installed successfully failed during the course of the study. However, the ESG sensor network has started to fail, and the performance degradation was not associated with only sensors experiencing high strain levels. Only the ESG at LW gave reliable results during the second test and the ESG at TR was completely unresponsive. The LR and TW ESGs pro-
duced erratic and inconsistent measurements. The failure cause was not clearly understood, but it seems to be associated with aging.

4.3.1 Health coefficient calculations

In this paper, health coefficients were proposed to track the condition of the structure over time. Health coefficients were calculated to compare the performance among load tests. Using the load ticket information, the strain information was normalized. Equation (3) shows the calculation of health coefficients $\Gamma$. For both dynamic and static values, $\varepsilon_{\text{peak}}$ was the peak strain observed, and $W$ was the rear-axle weight in kilonewtons. The peak strain values for dynamic cases were taken from the maximum peak in double-peak patterns. The ESG peak strain values for both cases were taken after the data was filtered. Tables 4 and 5 show the health coefficients for the dynamic and static loading cases, respectively.

$$\Gamma = \frac{|\varepsilon_{\text{peak}}|}{W}. \quad (3')$$

This health coefficient equation is not ideal, however, we believe it is sufficient to track structural degradation. It normalizes the strains to the rear-axle weight. The weights of the rear axles were directly over the sensor locations for peak strain. However, the strain values are influenced by the entire weight. The exact normalizing “weight” is presumably a complicated function of the type of test and the sensor location. For instance, the dynamic sensor values in the longitudinal beam show only one peak (per the entire weight) and the values for the transverse deck show two semiindependent peaks from the front and rear axles. The development of a more accurate health coefficient equation is beyond the scope of this field work.

4.3.2 FEA comparison

Table 6 shows the FEA results corresponding to the static tests from the November 10, 2004, and October 20, 2005, truck load tests. The strains in the top of the bridge at each designated location in the longitudinal beam and the transverse deck are also given. FEA results as compared to the EFPI and ESG readings show that the structure is a stiffer structure than before strengthening. Note that the predicted top strains are negative and small; the unreliable results of the top sensors may be due to faulty sensor bonding or to some complex interaction in the road surface. This question should be explored in future research. The FEA results are directly compared to the EFPI results in Table 7. The percent difference was calculated as

$$\% \text{diff} = \left( \frac{\varepsilon_{\text{EFPI}} - \varepsilon_{\text{FEA}}}{\varepsilon_{\text{FEA}}} \right) \times 100. \quad (4)$$

5 Conclusions

A sensor network and instrumentation were demonstrated in a field repair and upgrade application. Field assessment using intelligent, permanent sensors verified the strengthening of an in-service RC highway bridge with additional FRP composite reinforcement. (This work was a comple-

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### Table 2 Dynamic load test results.

<table>
<thead>
<tr>
<th>WL-SB Peak</th>
<th>CL-SB Peak</th>
<th>EL-NB Peak</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>November 10, 2004</td>
<td></td>
</tr>
<tr>
<td>LR (Rebar)</td>
<td>TW (FRP)</td>
<td>LR (Rebar)</td>
</tr>
<tr>
<td>$\mu\varepsilon$</td>
<td>$\mu\varepsilon$</td>
<td>$\mu\varepsilon$</td>
</tr>
<tr>
<td>ESG</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>EFPI</td>
<td>81.17</td>
<td>51.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>October 20, 2005</td>
<td></td>
</tr>
<tr>
<td>ESG</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>EFPI</td>
<td>58.93</td>
<td>63.03</td>
</tr>
</tbody>
</table>

$^a$Questionable values

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### Table 3 Static load test results.

<table>
<thead>
<tr>
<th>Longitudinal Beam</th>
<th>Transverse Deck</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>November 10, 2004</td>
</tr>
<tr>
<td>LW (FRP) (\ume)</td>
<td>LR (Rebar) (\ume)</td>
</tr>
<tr>
<td>ESG</td>
<td>53.35</td>
</tr>
<tr>
<td>EFPI</td>
<td>48.43</td>
</tr>
<tr>
<td></td>
<td>October 20, 2005</td>
</tr>
<tr>
<td>ESG</td>
<td>69.88</td>
</tr>
<tr>
<td>EFPI</td>
<td>51.39</td>
</tr>
</tbody>
</table>

$^a$Questionable values
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### Table 4 Table of dynamic health coefficients.

<table>
<thead>
<tr>
<th></th>
<th>WL-SB Peak</th>
<th>CL-SB Peak</th>
<th>EL-NB Peak</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>November 10, 2004</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LR (Rebar) (με/kN)</td>
<td>0.413</td>
<td>0.264</td>
<td>0.179</td>
</tr>
<tr>
<td>TW (FRP) (με/kN)</td>
<td>0.294</td>
<td>0.250</td>
<td>0.195</td>
</tr>
<tr>
<td>LR (Rebar) (με/kN)</td>
<td>0.179</td>
<td>0.316</td>
<td>0.153</td>
</tr>
<tr>
<td>TW (FRP) (με/kN)</td>
<td>0.216</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ESG X X</td>
<td>0.499</td>
<td>0.143</td>
<td>0.083</td>
</tr>
<tr>
<td>EFPI 0.270 0.288</td>
<td></td>
<td>0.280</td>
<td>0.252</td>
</tr>
</tbody>
</table>

*aQuestionable values

### Table 5 Table of static health coefficients.

<table>
<thead>
<tr>
<th></th>
<th>Longitudinal Beam</th>
<th>Transverse Deck</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>November 10, 2004</td>
<td>October 20, 2005</td>
</tr>
<tr>
<td>LW (FRP) (με)</td>
<td>0.271  0.386</td>
<td>0.286</td>
</tr>
<tr>
<td>LR (Rebar) (με)</td>
<td>0.246  0.217</td>
<td>0.304</td>
</tr>
<tr>
<td>TW (FRP) (με)</td>
<td>0.572  0.110</td>
<td></td>
</tr>
<tr>
<td>TR (Rebar) (με)</td>
<td>0.483a  0.045a</td>
<td>X</td>
</tr>
</tbody>
</table>

*aQuestionable values

As permanent instrumentation, the EFPI fiber optic system displayed uniform performance during annual dynamic and static load tests. This sensor type and system displayed excellent longevity, sensitivity, and accuracy. This work provides field validation of these characteristics.

The EFPI sensors and system demonstrated better operating characteristics than the comparison ESGs. Once operating after installation, the EFPI sensors did not fail during this 2-yr test and show consistent performance with age, while the ESG network showed sensor failures. EFPI systems can provide increased accuracy as their least-count measurement is determined by the operating wavelength. EM noise on the EFPI systems is also unlikely to be introduced in the cabling. ESGs are inherently noisier due in part to antenna loops in the runs of copper cabling and they require more postprocessing then their EFPI counterparts, especially in a field application.

The load-induced strain measurements from the EFPI sensors showed general agreement with the comparison sensors and showed similar trends with the FEA results. The measurements were very close, considering differences in load placement and the nonzero distances between colocated sensors. In particular, the proposed EFPI static health coefficients were consistent for similar locations for the two tests (see Tables 3 and 5). Also, the FEA results were uniformly higher than EFPI measurements with one rebar exception. FEA analysis verified that the structure was strengthened after rehabilitation since EFPI and ESG readings showed strains corresponding to a stiffer structure. Hence, the bridge showed greater stiffness than the analytical predictions and the EFPI sensors tend to measure strain at a similar fraction of the corresponding analytical predictions. Consequently, all EFPI results appear to be reasonable and have the potential to effectively quantify bridge performance.

Installation of the field sensor network provided insight into practical issues. The sensors and their location must be chosen with regard to expectations, initial modeling, or preliminary testing. The sensors should be tailored to the applications, for example, with regard to range and sensitivity. Embedding of the sensor network and its cabling into the structure prevents environmental degradation and vandalism to the network. A primary access point to the sensor

1. The bridge behavior per the fiber optic measurements seemed to show more repeatable results in the longitudinal beam than the transverse deck locations. The cause may be more sensitivity to load placement and the environment.
2. A proposed health coefficient showed promise as a single measure of load performance, but requires more development related to loading details and correlation with bridge condition and aging as future tests are performed.
3. The dynamic strain signature may possess more detailed information of the bridge condition. A permanent sensing network gives the potential of future investigations of dynamic performance and health indicators.
network must be established with enough security in terms of weather and tampering. For this work, an access point was established at an elevated position at the midspan of the structure. If embedded data acquisition and processing hardware is added, electrical power and remote transmission capabilities must also be considered.

Future work in this field study will involve the continued long-term monitoring of P-0962 and the testing of related smart technology. The correlation among EFPI sensor data and other NDT results and the aging of the sensor hardware are of concern. The application of embedded processing hardware to perform intelligent analysis of the structure in situ are planned. Questions raised by the research include quality assurance of sensor field installation, reasonable variation in strain monitoring due to environmental factors, and development of improved health coefficients. The health coefficients proposed in this paper should be studied with future tests to ensure they are a valid measure of structural health. Such advances are required to facilitate the implementation of smart structures technology and to gain acceptance among the end users.

**Acknowledgments**

The authors acknowledge the support of the Missouri Department of Transportation and the Center for Infrastructure Engineering Studies at the University of Missouri-Rolla.

**Table 6 Static strain from FEA.**

<table>
<thead>
<tr>
<th>Date</th>
<th>LW (FRP) (με)</th>
<th>LR (Rebar) (με)</th>
<th>Top (με)</th>
<th>TW (FRP) (με)</th>
<th>TR (Rebar) (με)</th>
<th>Top (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11/10/2004</td>
<td>89.0</td>
<td>73.0</td>
<td>−24.0</td>
<td>39.0</td>
<td>27.0</td>
<td>−20.0</td>
</tr>
<tr>
<td>10/20/2005</td>
<td>96.0</td>
<td>84.0</td>
<td>−34.0</td>
<td>42.0</td>
<td>29.3</td>
<td>−27.0</td>
</tr>
</tbody>
</table>

**Table 7 Comparison of FEA and measured EFPI results.**

<table>
<thead>
<tr>
<th>November 10, 2004</th>
</tr>
</thead>
<tbody>
<tr>
<td>LW (FRP) (με)</td>
</tr>
<tr>
<td>FEA</td>
</tr>
<tr>
<td>EFPI</td>
</tr>
<tr>
<td>Percent difference</td>
</tr>
<tr>
<td>October 20, 2005</td>
</tr>
<tr>
<td>FEA</td>
</tr>
<tr>
<td>Percent difference</td>
</tr>
</tbody>
</table>

We also thank Eli Hernandez for providing the FEA analysis. Finally, we thank Erik J. Timpson and Jason Cox for help during field work.

**References**


36. American Concrete Institute, Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary, American Concrete Institute Farmington Hills, MI (2002).

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