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Sensor enriched infrastructure system

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Abstract. Civil infrastructure, in both its construction and maintenance, represents the largest societal investment in this country, outside of the health care industry. Despite being the lifeline of US commerce, civil infrastructure has scarcely benefited from the latest sensor technological advances. Our future should focus on harnessing these technologies to enhance the robustness, longevity and economic viability of this vast, societal investment, in light of inherent uncertainties and their exposure to service and even extreme loadings. One of the principal means of insuring the robustness and longevity of infrastructure is to strategically deploy smart sensors in them. Therefore, the objective is to develop novel, durable, smart sensors that are especially applicable to major infrastructure and the facilities to validate their reliability and long-term functionality. In some cases, this implies the development of new sensing elements themselves, while in other cases involves innovative packaging and use of existing sensor technologies. In either case, a parallel focus will be the integration and networking of these smart sensing elements for reliable data acquisition, transmission, and fusion, within a decision-making framework targeting efficient management and maintenance of infrastructure systems. In this paper, prudent and viable sensor and health monitoring technologies have been developed and used in several large structural systems. Discussion will also include several practical bridge health monitoring applications including their design, construction, and operation of the systems.

Keywords: bridge health monitoring system; real-time monitoring; crack opening cisplacement (COD); temperature effect; traffic effect; cable force measurement; magneto-elastic sensor (EM); cable-supported bridge.

1. Introduction

Bridges play an essential part of a highway network. They are open to traffic, resistant to natural disaster, and undaunted by millions of loading cycles per year. However, this fortitude is quite expensive to maintain and do occasionally fail (DeWolf *et al.* 1989). The fact that many bridges are carrying greater average loads than predicted during their design has significantly increased the need to monitor bridge performance over the past few years. To effectively manage bridges today, there is a great need to monitor the real-time conditions of bridges, and the deterioration rates of their components, so that efficient and pro-active measures can be taken (Carder 1937, Catbas *et al.* 2000, Cheung *et al.* 1997, Ashkenazi and Roberts 1997). By using the latest state-of-the-art technologies, it is possible to utilize health monitoring systems on highway bridges to determine their behavior and condition, and promote a response to maintenance and inspection needs (DeWolf *et al.* 2002, Taljsten *et al.* 2007). Bridge health monitoring systems have historically been implemented for the

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purpose of understanding bridge behavior under various loads and environmental effects (Bampton *et al.* 1986, Barr *et al.* 1987, Brownjohn *et al.* 1994, Lau and Wong 1997, Leitch *et al.* 1987, Macdonald *et al.* 1997).

The Kishwaukee Bridge is a five-span pre-cast post-tensioned segmental concrete box girder bridge crossing the Kishwaukee River in Rockford, Illinois. The structure has been under increasingly stringent inspection since extensive cracking adjacent to the piers in the webs was observed (Nair and Iverson 1982, Shiu and Russell 1983, Wang *et al*. 2001, Lloyd *et al*. 2003b, Halvonik 2002). To this end, the author have developed instrumentation which continuously monitors the bridge on the basis of local strain, displacement, and temperature measurements, and global frequency measurements (Lloyd *et al*. 2003a, Lloyd *et al*. 2003b).

The ultimate focus of the instrumentation is the measurement of permanent deformations in the web at key locations that may indicate yielding of the steel shear reinforcement and consequent changes to the load carrying capacity. This is accomplished through local and global monitoring, for maximum redundancy. The local monitoring relies on strain gage pairs installed on the web faces in the middle of each main span to estimate the loads on the bridge, and by two LVDT displacement-strain rosettes that were installed at location identified during prior static load tests as having the most severe reinforcement stresses. The global measurements are derived from ambient vibration accelerometer records.

In another example, tension monitoring in steel cables and tendons for pre-stressed concrete structures and cable-stayed bridges is extremely valuable during construction and subsequent structural monitoring for cable supporting systems. Measurements need to be rapid, at low cost, portable at a remote site, and with little or no field preparation. With theoretically unlimited service life, EM sensor can monitor the tension in steel cable with high-density polyethylene cover and make contact-free stress monitoring feasible with high accuracy at a comparatively low cost. Therefore, these features provide an alternative for force monitoring of steel cables, tendons, and reinforcement used in infrastructures. Several examples using this technology will be included.

This paper provides a supplement to the current research on bridge health monitoring and improves the understanding of bridge behavior using state-of-the-art technologies.

2. Kishwaukee bridge monitoring system

2.1 Backgrounds

Kishwaukee River Bridge (Rockford, IL) is a post-tensioned precast segmental concrete box–girder bridge opened to traffic in 1980. The bridge has five spans with lengths of 170 ft + 3 × 250 ft + 170 ft (51.8 m + 3 × 76.2 m + 51.8 m). As the first-generation of segmental structures, the Kishwaukee Bridge engineers chose the design of a single shear-key joint usually located close to the center of gravity of the cross section. These joints are quite vulnerable especially during polymerization of the epoxy.

Problem arose during and after the completion of the bridge. The epoxy applied between segments was not hardened properly in some joints. The epoxy was unable to carry fully the shear stress and was instead acting as a lubricant that caused reduction of shear resistance capacity. Therefore a substantial part of the shear force was concentrated at the shear keys (Wang *et al.* 2001). As shown in Fig. 1, the inclined cracks went through only for the length of one segment but not continuously

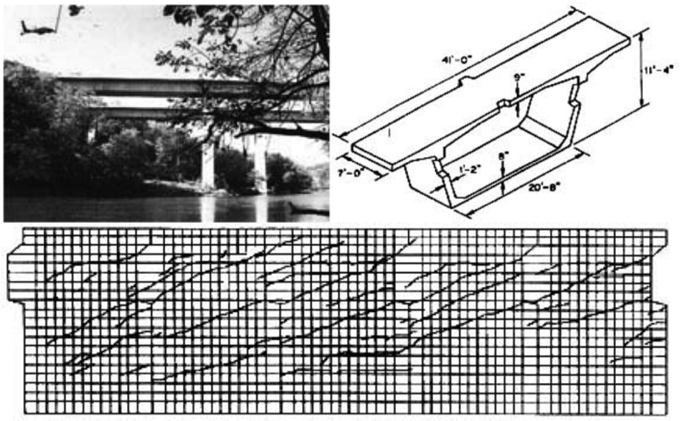


Fig. 1 Shear cracks on the webs of Southbound Kishwaukee Bridge

to the next segment. Many steel pins were inserted to the webs between segments to stop the propagation of shear cracks. It proved to be effective and successful slowing done the progress of cracking. However, there is a need to know current shear carrying capacity as well as the extent of propagation of cracks. A long-term monitoring program is therefore imperative for structural surveillance of Kishwaukee Bridge to provide continuous health records for the bridge management department (Lloyd *et al*. 2004a).

Since the major problem of Kishwaukee Bridge is shear cracks on the webs, a specifically designed structural health monitoring system was installed on the Kishwaukee Southbound Bridge in 2003 (Lloyd *et al*. 2004b). This system monitors and records the strain, crack opening displacement, and acceleration of the bridge, as well as the temperature outside and inside the bridge girder. Using shear strain measured through a rosette of three crack opening displacements across cracks, its corresponding shear resistance capacity can be predicted. Thus, the operation and management for this bridge is transformed into a more objective and quantitative process. This process provides for optimal integration of experimental, analytical, and informational system components (Lloyd *et al.* 2004c). In addition, the outcome of the process provides valuable information for current evaluation of structural integrity, durability, and reliability (Lloyd and Wang 2003). Using this information, composed from the sensory system, data acquisition system, and health assessment system, the bridge owner and maintenance authorities can make rational decisions in assigning the budgets for both maintenance and repair.

2.2 Static load test

In order to determine actual health status of Kishwaukee Southbound Bridge and set up the baseline for the following long-term health monitoring, we did two static load tests in 2000. The bridge was tested for service loading conditions. The weight of the trucks was comparable with a weight of the design truck defined in AASHTO LRFD Bridge Design Specifications. The design weight of the truck (72 kips) was amplified by a dynamic factor of $\delta = 1.320$. Four different positions of the trucks were proposed on the bridge, as shown in Fig. 2.

Visual inspection of the shear cracks determined the most damaged webs in the bridge. Web SB2-N4-E was chosen for in-situ measurement of deformation due to shear forces. Three linear variable displacement transducers (LVDT) were installed at the interior surface of the web, close to the

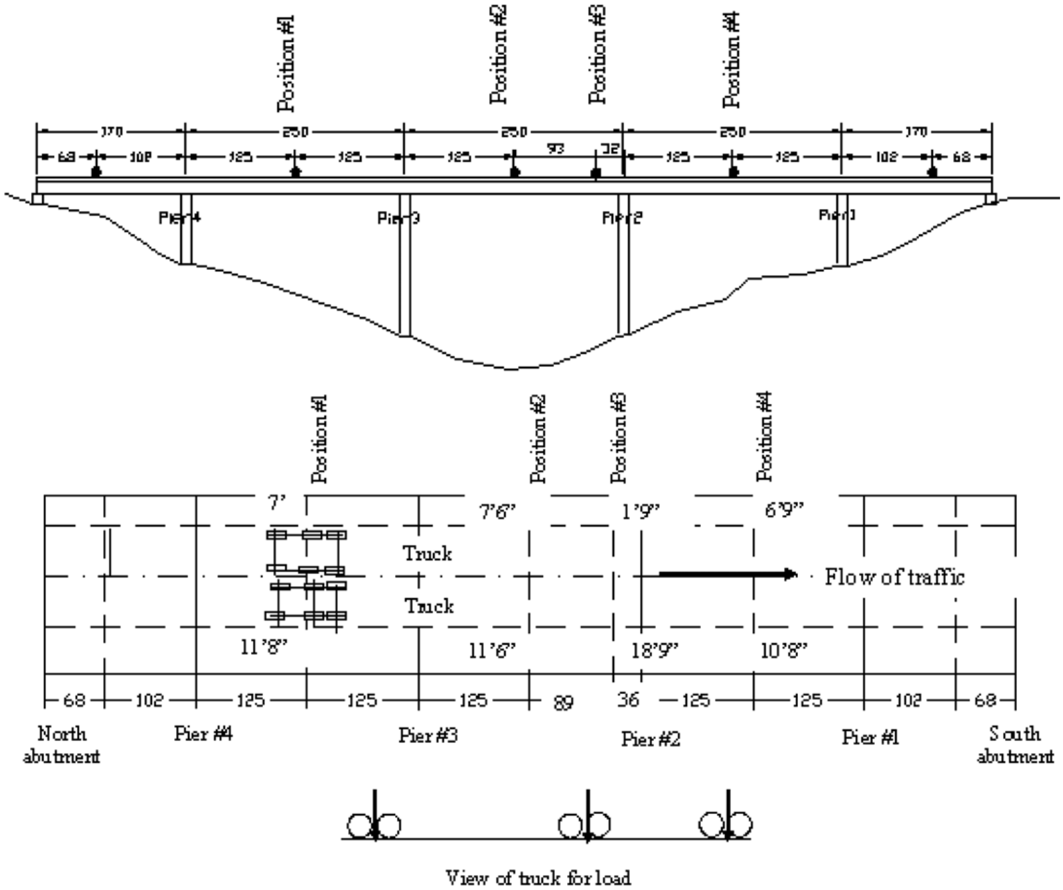


Fig. 2 Static diagnosis load test on the southbound Kishwaukee Bridge

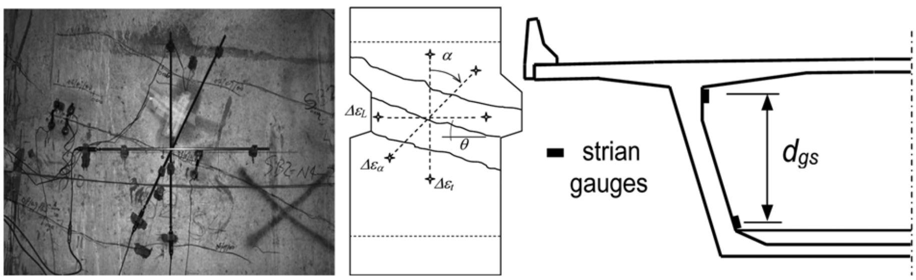


Fig. 3 LVDT sensors and strain gauges installed on the inner surface of the web

neutral axis, as shown in Fig. 3. Measured displacements were used for estimation of the web's shear stiffness. The load stage using trucks located in position #2 (second load) was selected for assessment, because neither transverse bending moments nor vertical axial stresses accompanied the imposed load in the web of segment SB2-N4. Shear forces generated by trucks located in position #3 (second load) were used for calculation of the steel stress increment in reinforcement.

Average shear strain, $\Delta\gamma_{Lt}$, and shear stiffness, $(GA)_{ir}$, of the cracked web can be assessed by Eqs. (1) and (2)

$$\Delta\gamma_{Lt} = \frac{2[\Delta\varepsilon_{\alpha} - \Delta\varepsilon_{L}\sin^{2}(\alpha) - \Delta\varepsilon_{t}\cos^{2}(\alpha)]}{2\cos(\alpha)\sin(\alpha)} \tag{1}$$

$$(GA)_{ir} = \frac{\Delta V}{\Delta\gamma_{Lt}} \tag{2}$$

Where, $\Delta\varepsilon_a$, $\Delta\varepsilon_L$, and $\Delta\varepsilon_t$ are average strain in inclined, vertical, and lateral direction, respectively. The shear force in the web ΔV was calculated with shear force and torsional moment due to trucks. To determine the global flexural stiffness of the bridge, strain gauges were installed at the webs of segments located next to the closures. Measured concrete strains were used for calculation of curvature in Eq. (3) and evaluation of the modulus by Eq. (4)

$$\Delta\chi_{meas} = (\Delta\varepsilon_{upper} - \Delta\varepsilon_{lower})/d_{gs} \tag{3}$$

$$E_c = \Delta M/(\Delta\chi_{meas}\, I_i) \tag{4}$$

Bending moment ΔM was determined analytically for known position and weight of the trucks. The assessed value of modulus was still in the range from 35,000 MPa to 40,000 MPa. This value is very similar to the design value, while the tangent shear modulus has been reduced about 50-55% by shear cracks. Therefore the change of bridge shear stiffness has little influence on its flexural stiffness. Dynamic tests and FEM simulation also corroborate the negligible effect of shear stiffness on flexural stiffness.

2.3 Half-scale experiment of concrete girder

In order to determine the shear carrying capacity of Kishwaukee Bridge, a half-scale I-beam model was cast in the Laboratory. The following parameters were measured in this experiment: deflection of the end of cantilever; flange strains; web strains; reinforcement strains; prestressing force; elongation of the strands.

Fig. 4 presents the arrangement of the half-scale experiment. The three half-scale I-beam segments were put between the anchoring block and the dispersal segment. The friction between the joints of three segments was reduced to a certain degree to simulate the unhardened epoxy problem as observed on the southbound Kishwaukee Bridge. The prestressing reinforcement was low relaxation strands with a high yielding point of 1800 MPa. The upper prestressing force N1 was applied with eight 3-strand tendons on the top flange while four 4-strand tendons were used to apply the lower

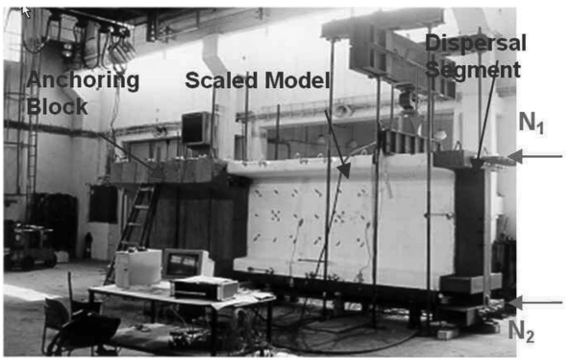


Fig. 4 Arrangement of half-scale experiment

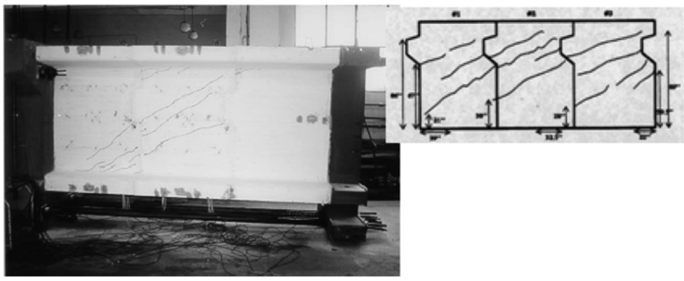


Fig. 5 Propagation of shear cracks on the specimen

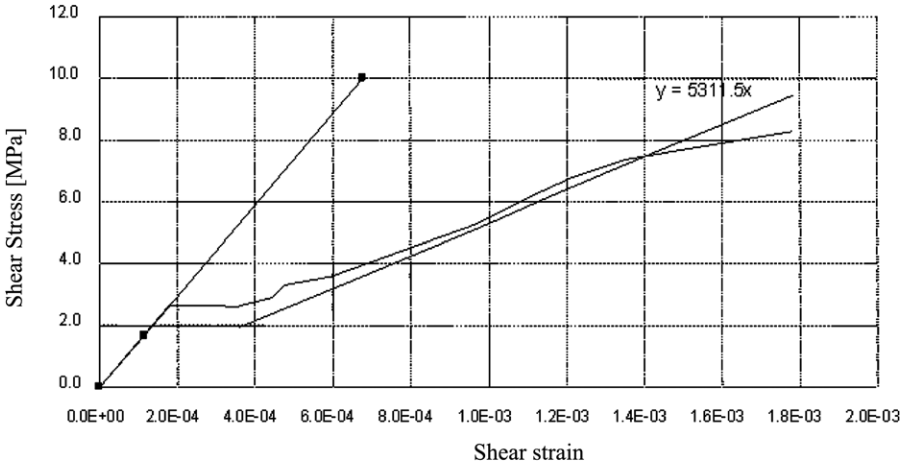


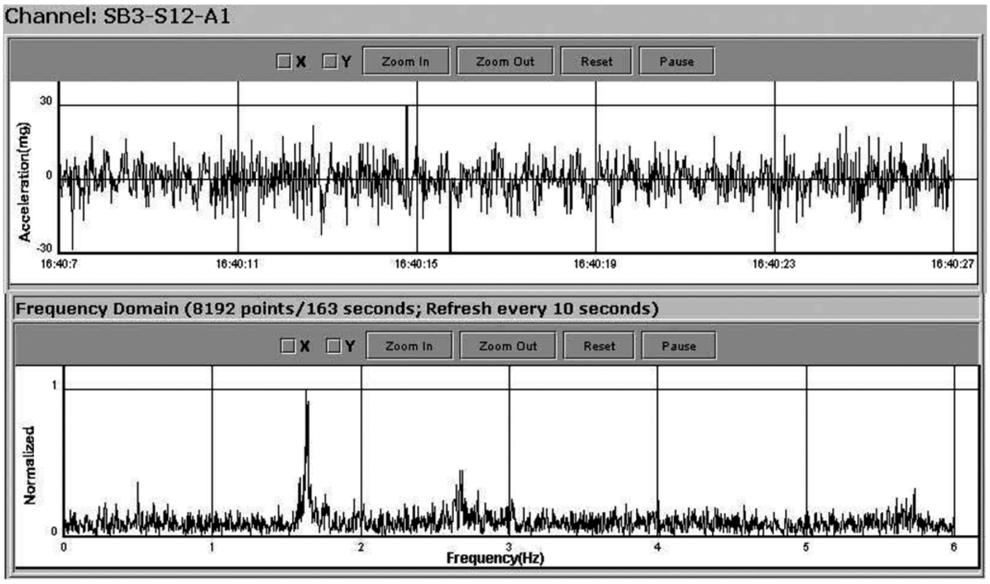
Fig. 6 Relationship of shear stress and shear strain

prestressing force N2 on the bottom flange. The shear force V was applied with a hydraulic jack.

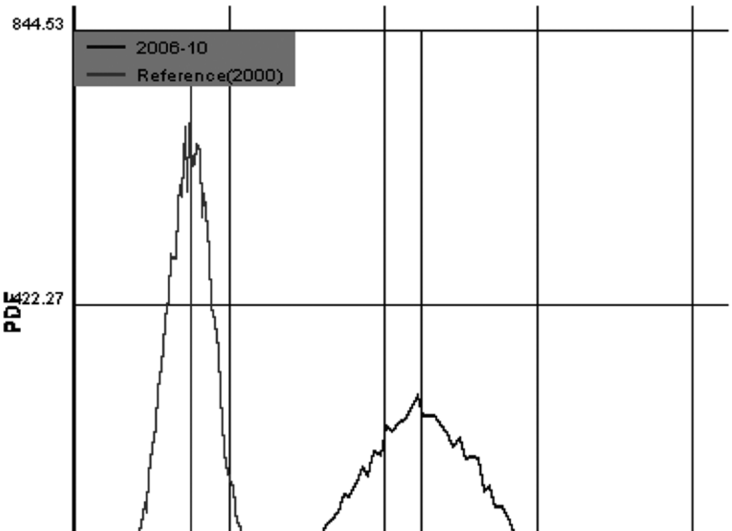
The propagation of shear cracks on the I-beam webs is shown in Fig. 5. According to the visual inspection records, the type and inclination of these shear cracks were similar to the actual cracks on the Kishwaukee Bridge as shown in Fig. 1. It indicated the experiment successfully simulated the actual damage condition of Kishwaukee Bridge. The relationship of shear stress and shear strain is shown in Fig. 6. After the linear elastic phase, the concrete cracked and the steel reinforcement carried much of the shear force itself, which is evident from the graph. About the yielding point of shear reinforcement, the reduced tangent shear modulus was roughly 5300 MPa. According to the analysis of the static load test, the current tangent shear modulus is approximately 6740 MPa at the worst location (SB2-N4) of the southbound Kishwaukee Bridge. This means the shear steels are still in the safe region of stress-strain curve. However, more strict regulations should be encouraged so that no overweight truck passes over the bridge which might induce further damage.

2.4 Real-time global and local health evaluation

Kishwaukee Bridge monitoring system has been collecting and processing data and generating evaluation and health reports for five years. The system can analyze the frequency distribution, crack opening displacement, shear strain in the web, and traffic information in real time. Automated warning/alarm system is in effect to warn against any further local structural damage on the bridge,



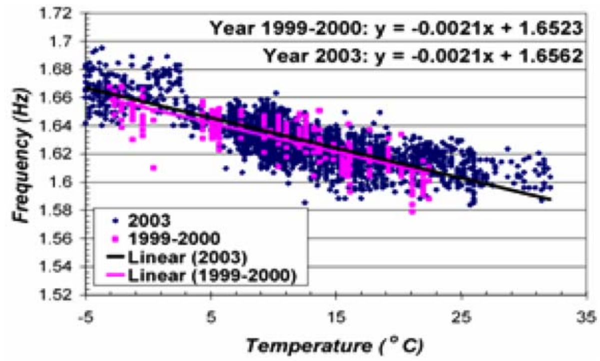
(a) Real-time acceleration & frequencies



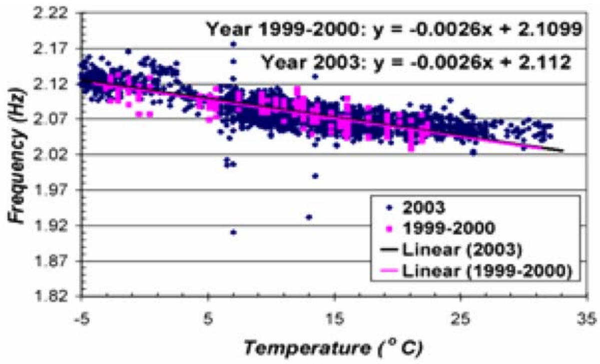
(b) Bootstrap distribution of frequencies

Fig. 7 Pre-processing and post-processing of acceleration and frequencies

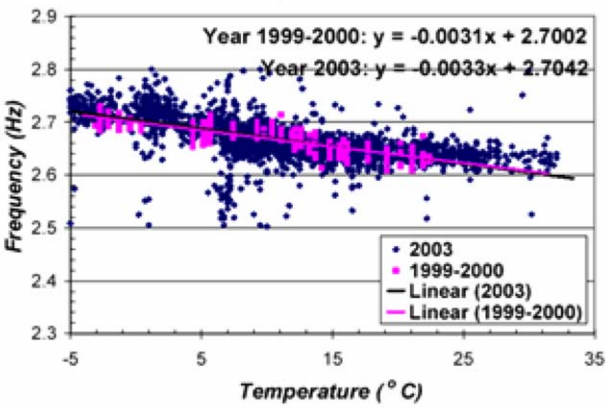
system problems, sensor dysfunction, and data errors. After establishing the baseline of bridge health, a long-term monitoring system can be used to provide the continuous health information of a bridge. For concrete bridges, cracks, especially the shear cracks act as the main role of local damages. And the global health information of a bridge can be represented with the bridge stiffness. Both global and local conditions of bridges need be evaluated in order to determine their in-service behavior and justify rehabilitation and repair plans. The global health information can be provided by dynamic measurement, while the local damage can be captured by sensors such as strain gages, linear variable displacement transducer (LVDT), etc. The raw acceleration data are collected and preprocessed to obtain the natural frequencies in a sensor substation. Then the acceleration and frequency data are transferred into the database server via the internet in real time, as shown in Fig. 7(a). After that, the application server will analyze the data to get the hourly bootstrap mean and its confidence intervals (Lloyd *et al.* 2003b), as shown in Fig. 7(b).



(a) 1st Modes

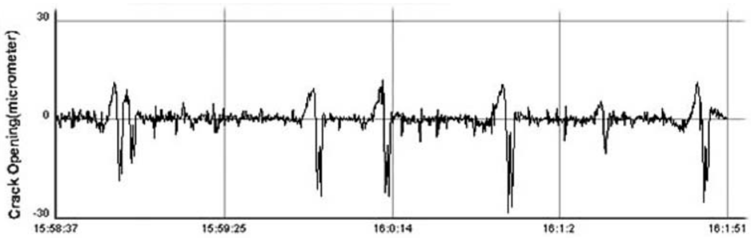


(b) 2nd Modes

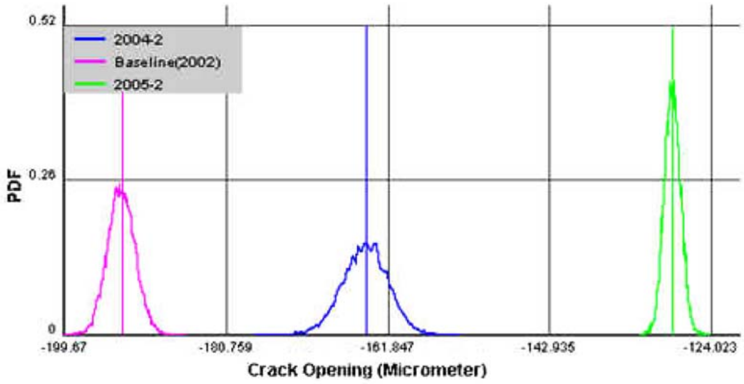


(c) 3rd Modes

Fig. 8 Relationship between temperature and first three modes



(a) Real-time crack opening displacement



(b) Bootstrap distribution of COD

Fig. 9 Pre-processing and post-processing of crack opening displacement

Based on the five dynamic tests from 1999-2000, the global dynamic characteristics of the bridge are obtained from the acceleration data with the related temperature values. Those structural parameters are setup as the baseline of global health assessment. Temperature has a significant influence on the natural frequencies of a structure. Hence it is important to derive the relationship between temperature and natural frequencies, i.e., how much the frequency will change due to the variation of 1^{o}C. To a certain degree, this relationship can represent the change of the bridge bending stiffness due to the temperature variation. It is important to compare the bootstrap means of frequencies under the same temperature.

Fig. 8 provides the temperature-related regression curves and parameters of the first three modes from year 1999 to year 2003. As shown in the figure, the changes of the first two modes due to the variation of 1^{o}C during 2003 are almost the same as the baseline (1999-2000). However, the analysis about mode 3 during year 2003 shows a little increase in the frequency change due to the unit temperature variation.

According to the theory of dynamic analysis, the changes of higher modes usually reflect the development of local damages. In order to verify the result of global health assessment and inspect the state of local damage, it is necessary to carry out the specific local health evaluation.

The raw data of crack opening displacement (COD) are preprocessed in sensor substation and transferred into the database server via the internet, as shown in Fig. 9(a). Then based on the hourly record of crack opening displacements, the application server will analyze those data to obtain their bootstrap distribution and confidence intervals. The result is shown in Fig. 9(b).

In order to find out whether the shear cracks propagated during year 2004, it is necessary to

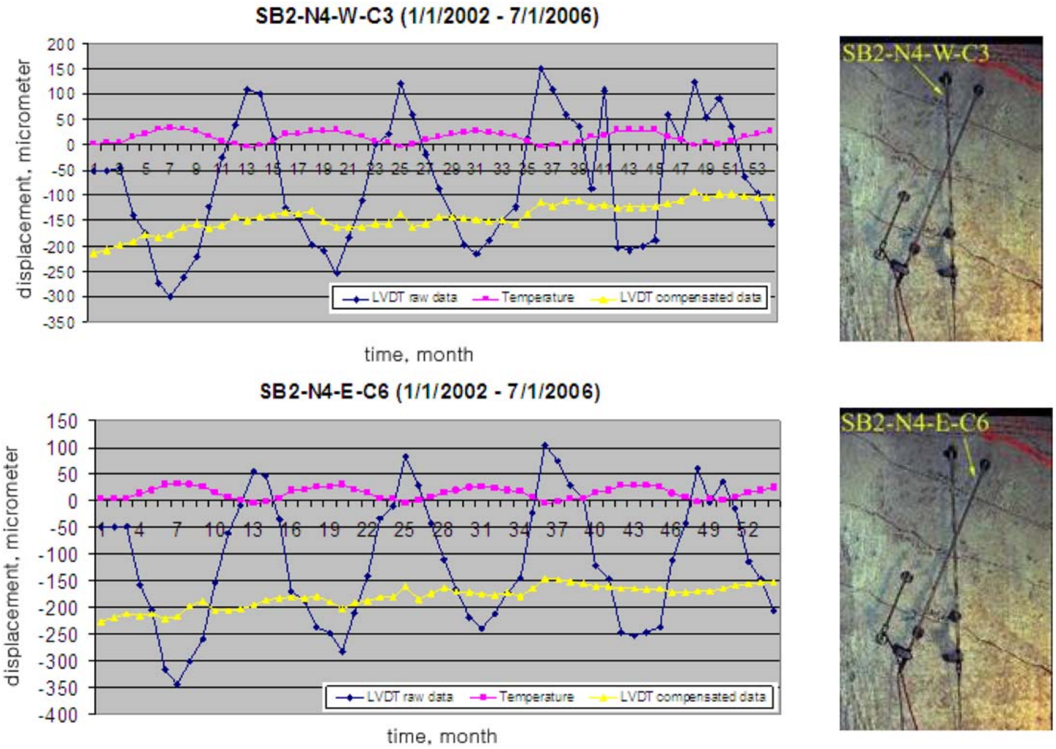


Fig. 10 Relationship between crack opening displacement and temperature, west web (top), east web (bottom)

analyze the relationship between temperature and crack opening displacements, i.e., to find how much the crack opening displacement will change due to a variation of 1°C. Again, Bootstrap means of crack opening displacement has to be compared based on the same temperature. It is evident that crack is progressing.

Based on the data of 5 years, the analysis result of crack opening displacements (LVDT raw data) is given in Fig. 10 according to their locations. In five years, the crack opening displacements (LVDT compensated data) excluding the temperature effect show some difference between the west web and the east web. As shown in the graph, on the west web, the total accumulated crack opening displacement is about 120 micrometer. But on the east web of SB2-N4, the corresponding COD is about 75 micrometers. Both values indicate accumulation of damage in terms of increases in crack opening displacement. The difference between the west web and the east web is possibly due to the heavier traffic on the west side of the south-bound bridge. This result conforms to the visual inspection of Illinois Department of Transportation. According to the analysis on crack opening displacement, we can evaluate the average shear strain of the cracking webs.

Based on the global and local measurements, the rule-based expert system gives the shear carrying capacity of the bridge with ductile mode of failure. Fig. 11 presents the shear stress-shear strain curve of the cracked web at Segment SB2-N4. As shown in the graph, the monthly maximum shear strain is over the baseline of the static load test in 2000. However, this value is still far below the yielding point. It indicates that the most damaged segment (SB2-N4) is still working in the nonlinear-elastic zone.

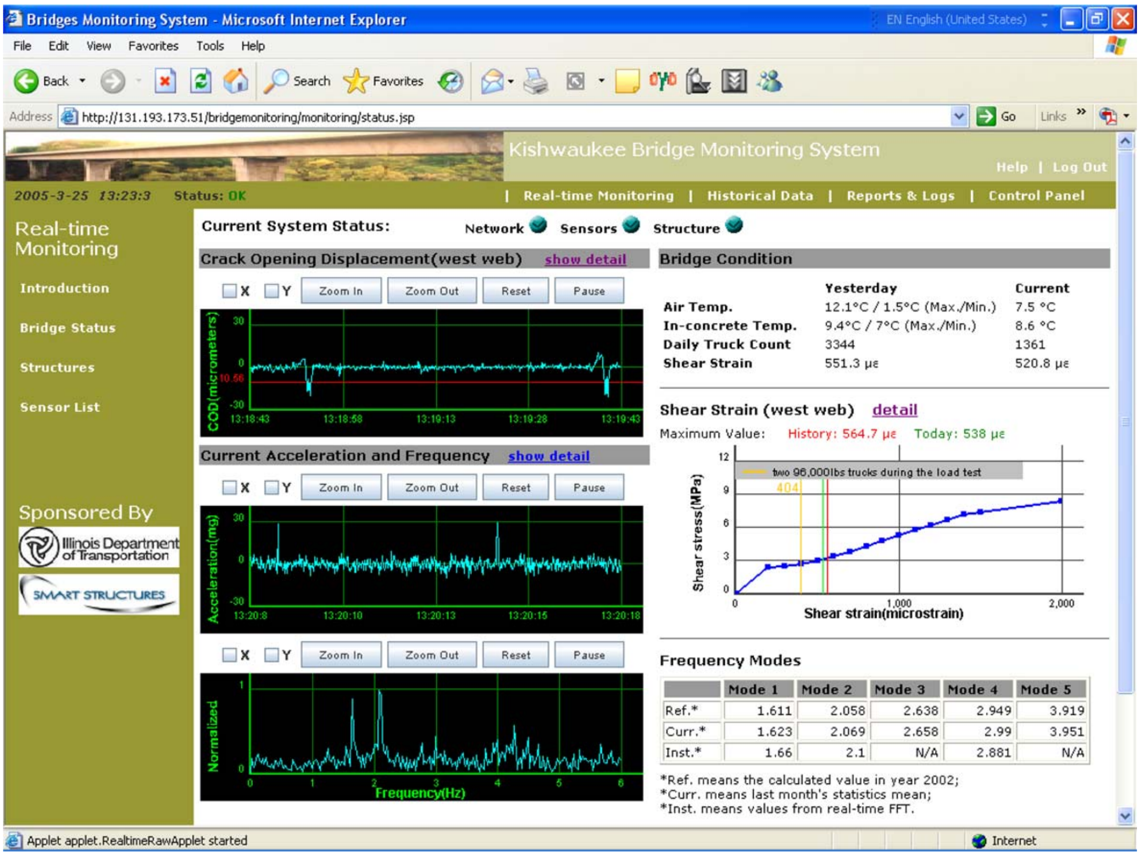


Fig. 11 Main interface page

2.5 Temperature effect

Measurements obtained from the monitoring system are available continuously and with greater precision; data studied in this paper are given in Fig. 12. The statistical nature of natural frequencies is clearly demonstrated even at the same temperature. A probability density function is therefore necessary to represent its behavior and used for comparison in determining the change of frequency due to damage. Bootstrap Method was used to determine the mean and its corresponding probabilistic property centered at the same temperature as shown in Figs. 7 and 9. It is too small of a difference in term of frequency for Kishwaukee Bridge to definitely conclude the extent of damage of the bridge. However, the accumulation of damage due to cracking is obvious as shown in Figs. 9 and 10.

2.6 Traffic effect

In this study we have measured crack opening displacement in real time. The average crack opening displacement subjected to a truck loading was about 25 micrometers as shown in Fig. 9(a) of a real time plot. The average crack opening displacement due to temperature in a day was about 40 - 50 micrometers which is only about 2 times of the displacement due to truck loading. For tram

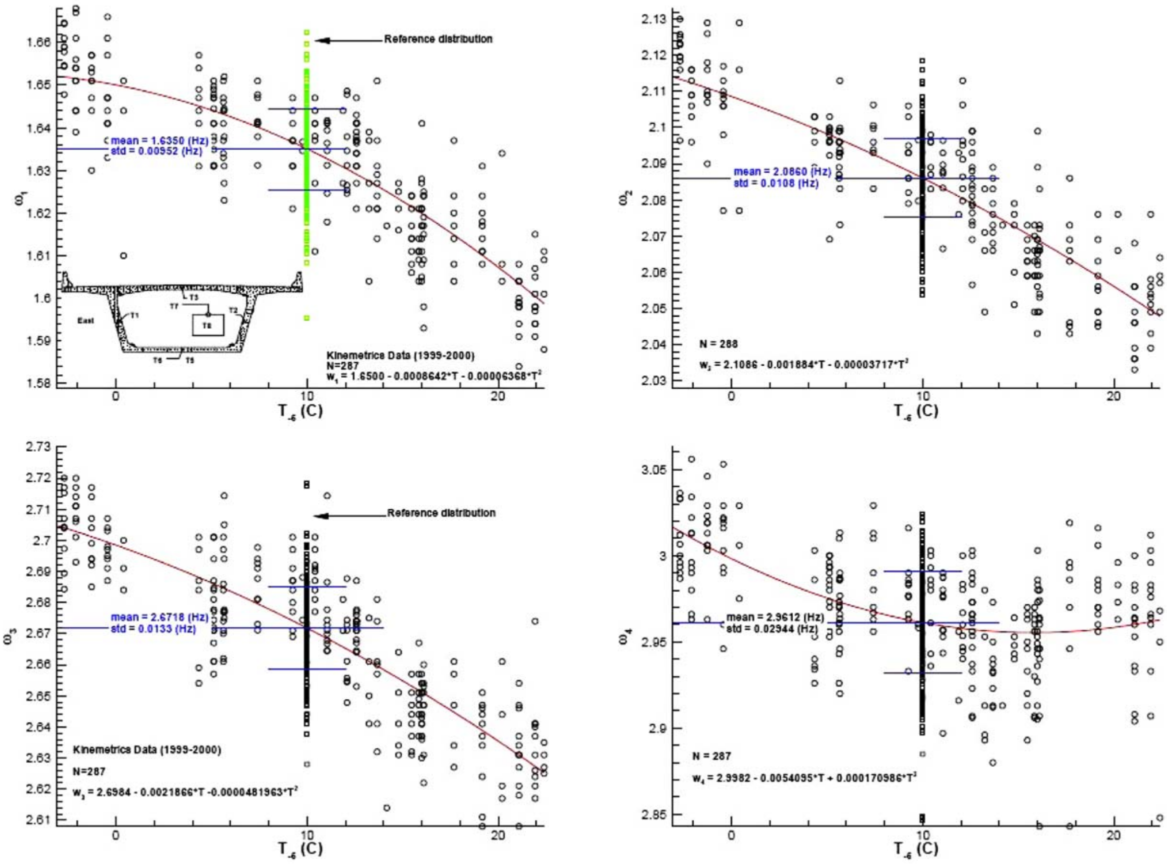
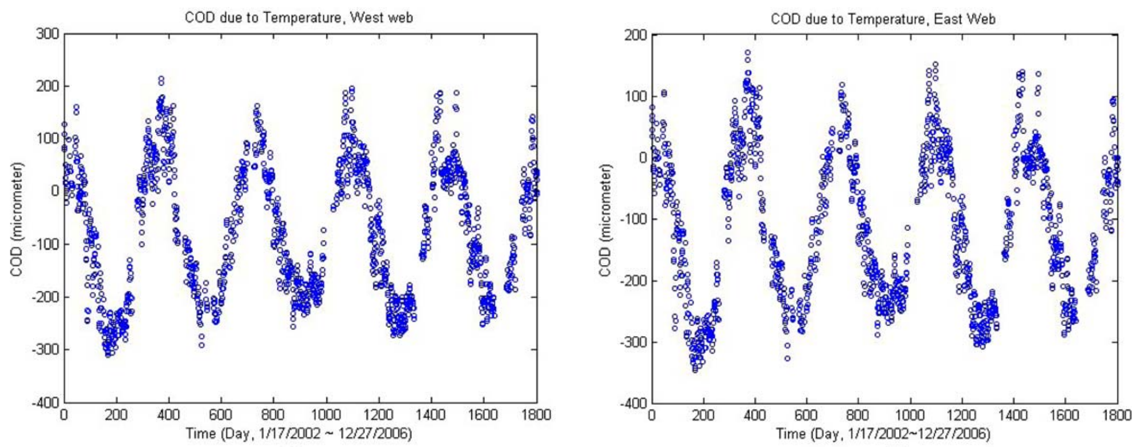
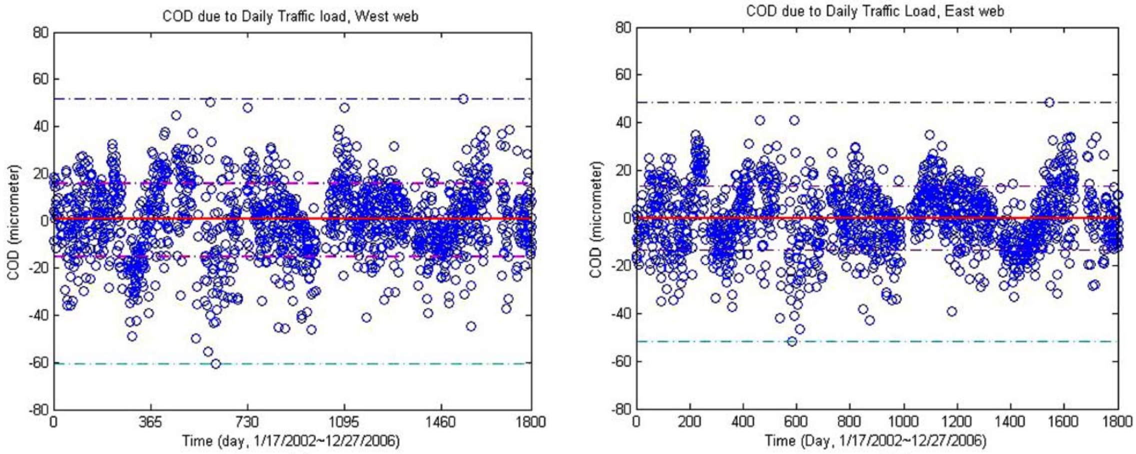


Fig. 12 Open circles denote reference frequency data obtained 1999-2000 for the south-bound Kishwaukee Bridge. Deterministic temperature bias inferred from T_5 (delayed five hours) was identified using least squares regression, and all frequency estimates were adjusted to a common reference temperature, $T_{ref} = 10(^{o}C)$



(a) (b)

Fig. 13 Crack opening displacement due to temperature



(a) (b)

Fig. 14 Crack opening displacement due to traffic load

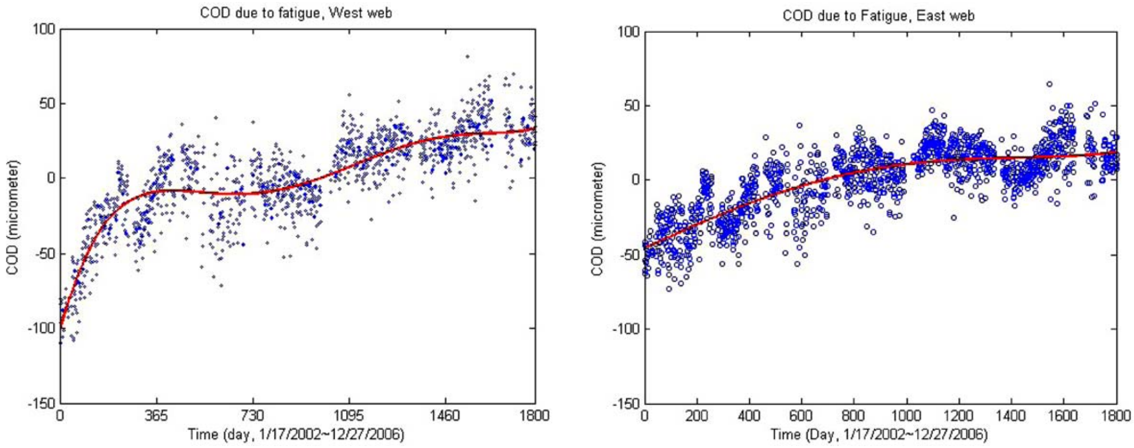


Fig. 15 Crack opening displacement due to fatigue

traffic, temperature effect could be 10 times more than the traffic effect in another study (Taljsten *et al*. 2007). However, it is difficult to separate the accumulation of damages that are due to traffic or temperature cycling. Using real time data, we estimated that the passing daily truck count was about 3000. Cracks open and close once a day due to temperature fluctuation. However, cracks open and close 3000 times every day due to trucks. Figs. 13(a) and 13(b) show the yearly displacement cycles due to temperature. Note that the average crack opening displacement due to temperature in a year was about 500 micrometers, which is about 20 times of the displacement due to truck loading. Figs. 14(a) and 14(b) show the estimated traffic effects. These are obtained by subtracting the temperature effect and permanent accumulation of damage due to fatigue as shown in Fig. 15 from the original data. We knew overall accumulation of damage in terms of crack opening displacement was about 120 micrometers in five years from 2002 to 2006. Therefore, one can concluded, although daily temperature effect is about 2 times of traffic effects, the accumulation of damage may be attributed mainly to truck loadings because of its sheer number of cycles over the years. Although overweight trucks are usually admitted by State Highway Department, unexpected loadings which produced very large crack opening displacement are infrequent. Upon the inspection of all data gathered, it is a fact that the cracks are active and progressing in a slow pace. Overall, the shear stiffness has been reduced by 50%. But, bending stiffness and frequency have not shown the same trend and proportion. On the contrary, frequency has shown a negligible effect due to continuing local shear cracking.

3. Monitoring of cable stresses for long span bridges

The magnetoelastic technology is applied to monitor cable forces in pre-stressed structures and bridge cables. This technology overcomes the drawbacks of the current available NDT while still inhabits the advantages of normal NDT methods. It measures the internal stress in steel tendons and cables directly. Only small size of sample (6 feet long) is needed for laboratory calibration of the material being used in structure. Easy installation for new structure and in-situ installation adaptability for existing structure are one of the key characteristics of this method. Other attractive includes compact, easy to operate, accurate, and boasts a theoretically unlimited service lifetime. Magnetoelastic sensors (EM) have been implemented in many structures for more than a decade (Wang *et al.* 1999, Wang *et al*. 2003, Polar *et al*. 2004, Sumitro *et al*. 2005, Lloyd and Wang 2003). Stresses in steel tendons and cables are monitored with specifically designed magnetic sensors which is being developed and used in structures in the United States, Japan, and China. Where other NDT technologies have failed, EM sensors have succeeded, blazing the trail in the science of structural health monitoring today and for the future.

3.1 Operating principle

The magnetoelastic(EM) sensor is essentially a solenoid consisting of two separate windings; the primary coil receives the input current that produces the magnetic field, and the resulting induced voltage is received by the secondary coil and output to the measuring device (See Fig. 16).

The magnetoelastic sensor can be applied in-situ by wrapping the primary and secondary coils around a specimen itself. The primary and secondary coils must be wrapped around a non-metallic core that does not exhibit ferromagnetic properties. Plastics and other polymers are ideal materials

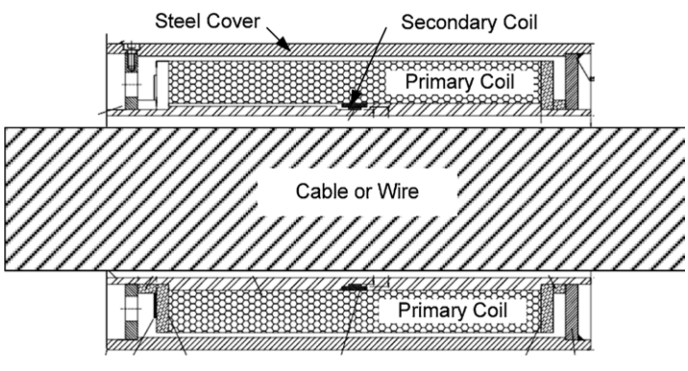


Fig. 16 EM sensor profile

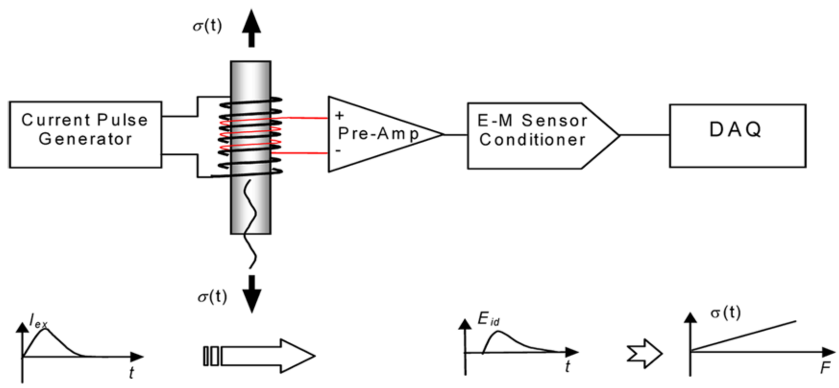


Fig. 17 Schematic of measurement principal

for the core, which acts as an "insulating" layer between the specimen and the sensor coils and protects the coils from being damaged. Because of the large number of turns in the sensor, there are usually many layers of windings. These layers must be insulated from each other in order to control the heat generated by the current pulse. One layer of masking tape is sufficient for these insulating layers. For any application outside the laboratory, the sensor must be encased by an external shell and sealed in order to protect the coils inside from moisture and other elements presented in the field.

Magnetoelastic (EM) stress sensors function by utilizing the direct dependence of the magnetic properties of structural steels on the state of stress. These properties are measured by subjecting the steel to a pulse or periodic magnetic field, which can be accomplished without any contact. Changes in magnetic flux in the steel allow those magnetic properties to be sensed and deduced through Faraday's law as depicted in Fig. 17.

By examining the sensor schematic structure, the sensory durability depends on a few parts, i.e., copper wire, potting compound, steel cover and connecting cable. Copper wire has a very high durability. When sheathed with special insulations, copper wire may sustain for hundreds of years. Potting compound also protects the copper wire. The steel cover may be plated, embedded in concrete in order to be protected against corrosion. The weakest point of the EM sensor is the connecting cable. Taken all the durability factors in the industrial environment into consideration, the minimum lifetime of the EM sensor is predicted to be 50 years. The measuring unit is not critical since the measuring conditions are precisely defined and the damaged measuring unit parts can be easily replaced. Even after the long period of operation, the current measuring unit can be replaced by a new one. Although the electronics parts, used in the measuring unit will no longer be available after decades, technology era will continue to grow 50 years from now. Both the physical principle and the measured steel cable can be applied in the same way in the future since the configuration of the windings and number of turns do not change over time.

3.2 Application examples

3.2.1 Penobscot Bridge, Maine, USA

Penobscot Bridge in Maine, USA, is a cable-stayed bridge. Advanced multi strand cable systems from DSI were used as stayed cables.

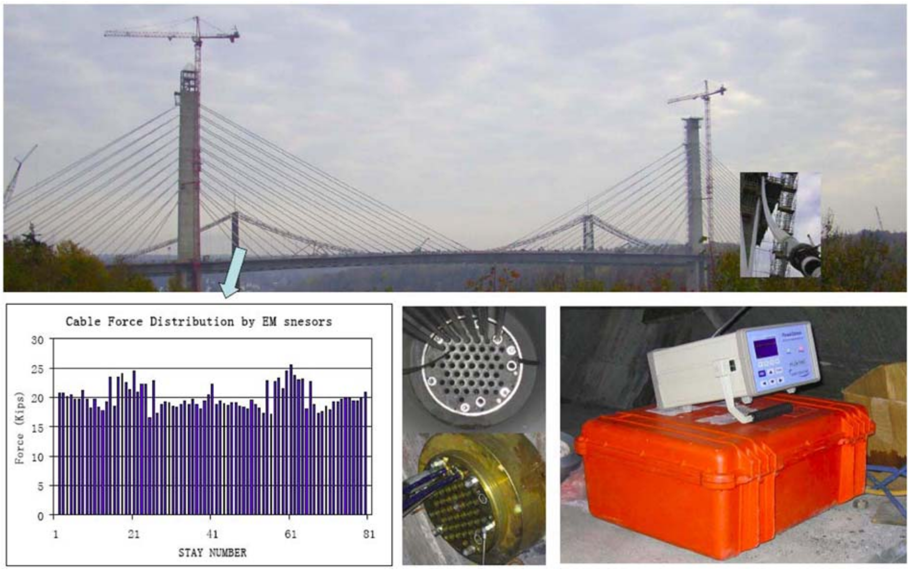


Fig. 18 Application of EM sensor on Penobscot River Bridge

Fig. 19 Strands in the cable in Penobscot River Bridge

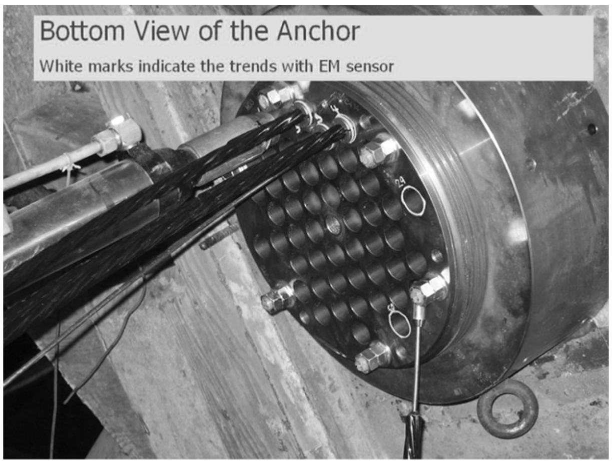


Fig. 20 Bottom view of the anchor in Penobscot River Bridge

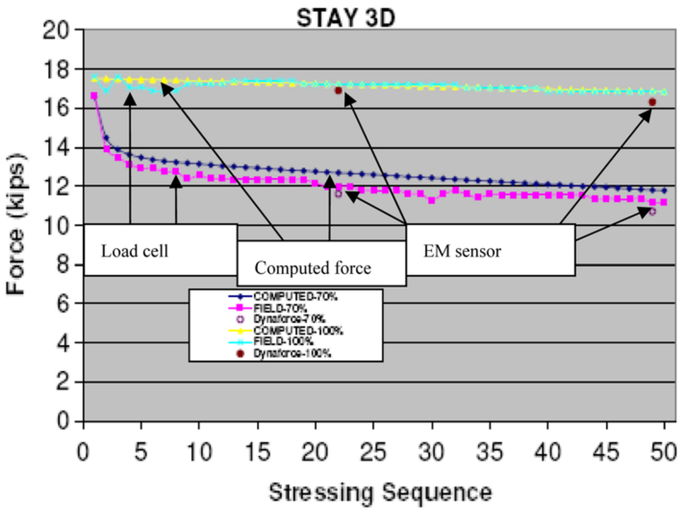


Fig. 21 Tension monitoring results from EM sensor used in Penobscot River Bridge during construction

Fig. 22 Completion of QJ Arch Bridge

In each anchor, three EM sensors were installed inside the anchor to monitor the cable force during and after construction as seen in Figs. 18-20. Prediction of cable forces were used during construction for 70% and 100% post-tensioning as shown in Fig. 21. The consistency between load cell and EM sensor measurements revealed the load relaxation in the cables after construction.

3.2.2 Qianjiang fourth bridge, China

QJ No. 4 Bridge under construction in Hangzhou of China is the longest double-deck and multi-arch bridge in China, the upper deck is a six-lane highway, while the lower one is for railway with sidewalk for pedestrians. To guarantee the reliability of its service, a state-of-the-art health monitoring system was built during the bridge construction, including the EM stress sensors on the hangers and post-tensioned tendons as seen in Figs. 22 and 23.

The EM sensors on the hanger cables were calibrated with the load cell in laboratories, while those on the post-tensioned cables were calibrated in construction fields. Tension measurement on the post-tensioned cable during construction and truck loading testing is shown in Fig. 24. Post-construction load relaxation and traffic-induced load increase can be observed. The tension in the

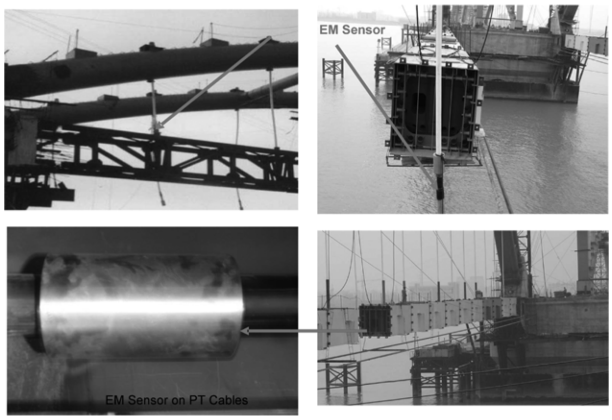


Fig. 23 EM sensors on hanger cables and box girder

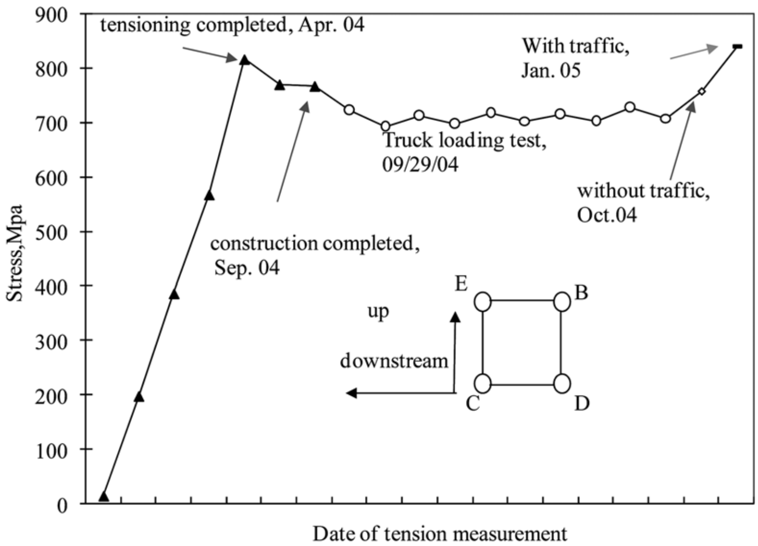


Fig. 24 Tension in post-tensioned cable (D) in the box-girder of south big arch

hangers on the big arches was continuously measured when the bridge began its service, as shown in Fig. 25.

3.2.3 Zhanjiang bay bridge, China

Zhanjiang bay bridge in Zhanjiang city, Guangdong Province of China is one of the longest cables stayed bridge in China. The main span of bridge is of 480 meters long with a navigational clearance of 48 meters. 14 EM sensors, as principle sensors in its state-of-the-art health monitoring system, were installed during the bridge construction. The EM stress sensors are used to monitor the cable forces as seen in Fig. 26.

The prefabricated EM sensors were calibrated in the factory before shipping to the construction site as shown in Figs. 27 and 28. Linear behavior after several repeated stressing is obvious and promising. Salient feature of continuous monitored data of cable force for the bridge are shown in Figs. 29(a) for 24 hours and 29(b) for two weeks, respectively. Continuous line depicted the temperature

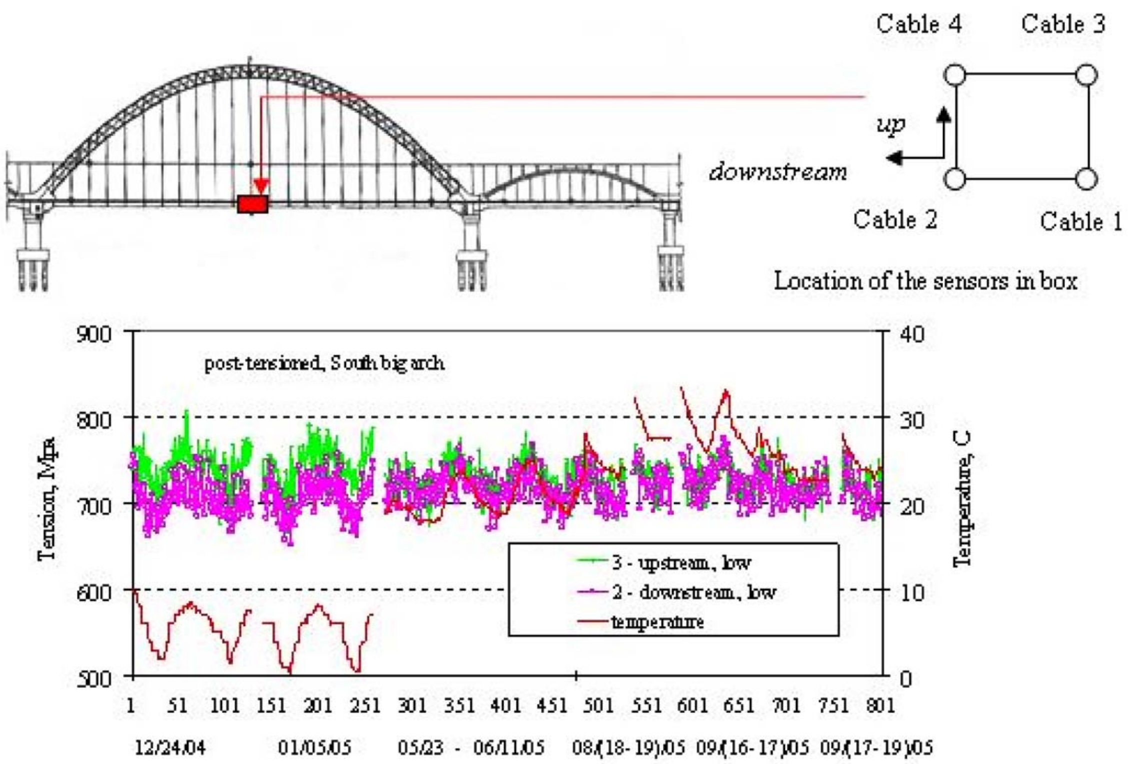


Fig. 25 Continuous tension measurements for the post-tensioned cables on the bridge

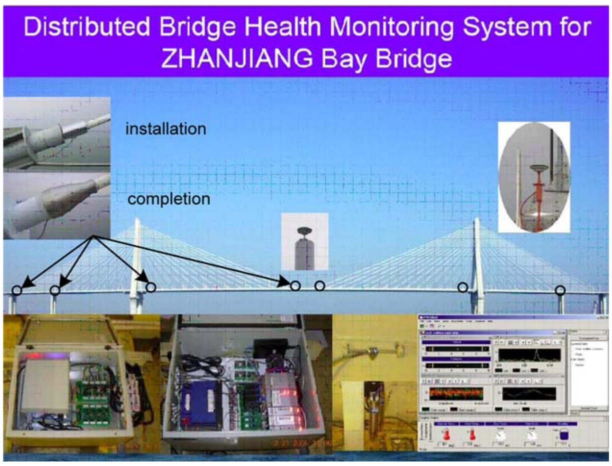


Fig. 26 Real time health monitoring system in Zhanjiang Bay Bridge

variation during the period of time. The system is stable. The measured cable forces are within the design range. Temperature variation does not seem to produce much effect on cable forces.

3.2.4 Kamikazue viaduct, Japan

Kamikazue viaduct is a double box girder PC bridge that is formed by connecting two single box

Fig. 27 EM sensor calibrations for Zhanjiang Bay Bridge

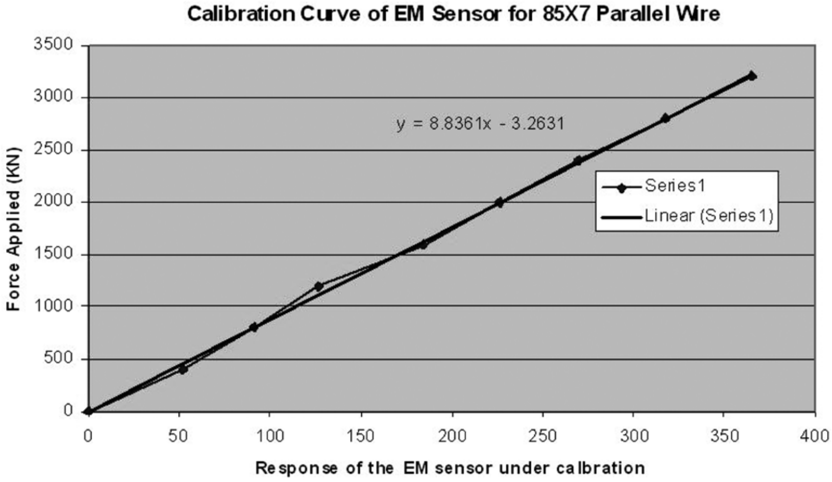
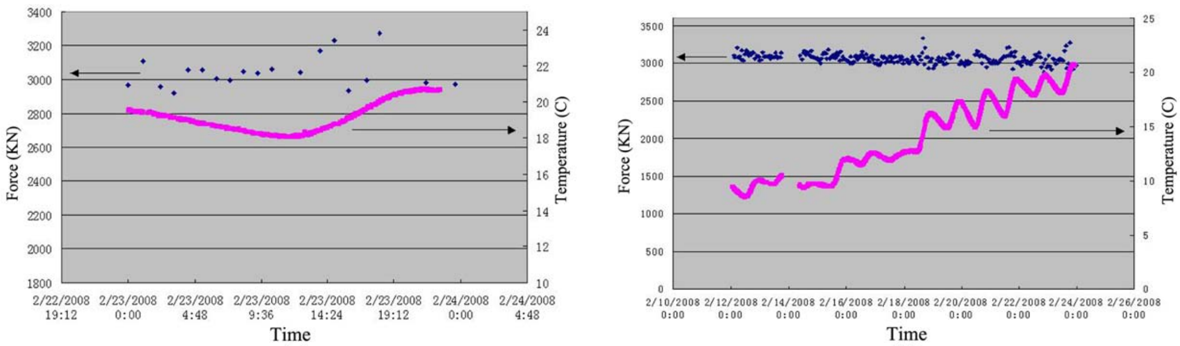


Fig. 28 Calibration of steel cable on Zhanjiang Bay Bridge



(a) (b)

Fig. 29 Cable force history in Zhanjiang Bay Bridge within (a) 24 hours and (b) 14 days

girder pre-cast segments. This bridge consists of 1040 segments in 17 span continuous box-girders, has a length of 630 m and width of 16 m. The segments were erected by span-by-span method using movable false-work. The segments are jointed using epoxy resin and post-tensioning with external

Fig. 30 EM sensor system calibration

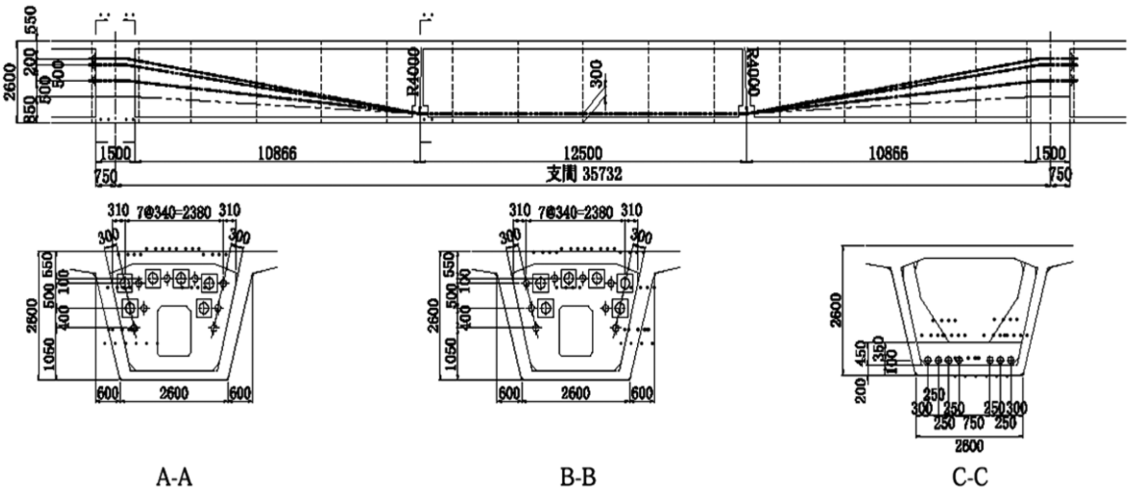


Fig. 31 External cable of a PC box girder bridge

tendons in each span as shown in Fig. 31. The construction of this viaduct by utilizing precast segmental construction and post-tensioning to provide large capacity external tendons was a construction technology challenge due to rare experience for such a construction type. Therefore, a full-scale model test was carried out to verify the safety performance of the anchorage blocks and deviators. Besides verifying the safety of the individual members, the frictional effect between the deviator and the post-tensioned tendon was also investigated. The verification test was conducted by measuring the tendon forces at either side of the deviator. Then the friction coefficient was calculated from the difference stresses of the post-tensioned tendon between these symmetrical adjacent sides of deviator. The most commonly used method to measure the post-tensioning tendon stress is by attaching strain gages to strands that form tendons. However, this method was not possible in this case since epoxy coated strands are used for external tendons. Therefore, EM sensor sensory technology was adopted. A verification test and calibration was conducted as shown in Fig. 30 to verify magnetic property change with respect to stress and temperature change. EM sensors were installed as shown in Fig. 32. The stress loss due to set-loss at the fixation stage, relaxation, creep and shrinkage, and elastic deformation effect were adequately observed through several months of continuous monitoring as shown in Fig. 33.

Fig. 32 Sensor configuration

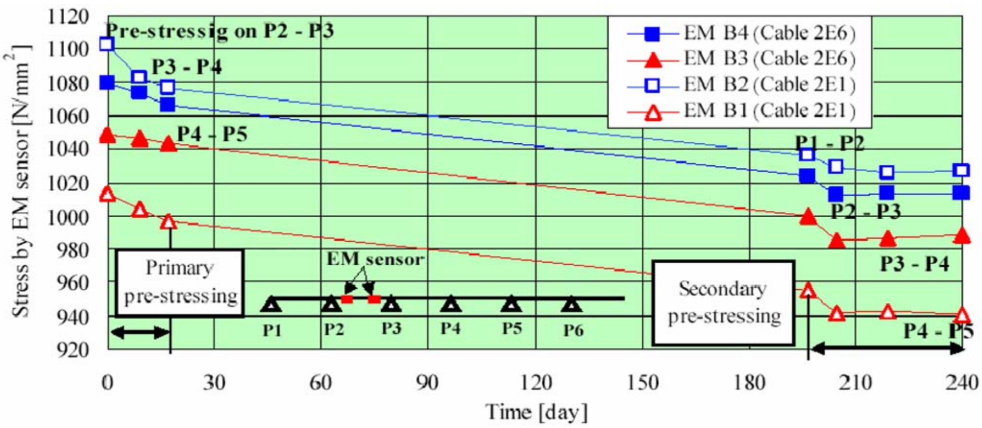


Fig. 33 Tendon stress history during and after the construction

4. Conclusions

A real-time bridge monitoring system includes a real-time data acquisition, a real-time data analysis, and a health reporting system. It should provide the current health status of the bridge in real-time. It should be able to determine the current strength and resistant capacity of a structure. The key point is to use the minimum number of sensors to collect, process, and analyze the real-time dynamic and static data from the most critical positions of the bridge.

A large-scale real-time monitoring system can generate huge amount of data every day. The excessive information will overwhelm and decrease the productivity of the bridge engineers if there is no any integrated program of data preprocessing and health diagnosis in the monitoring system. All the real-time raw data shall be pre-processed in the bridge to increase the speed of data transmission, save the capacity of database, and improve the efficiency of health assessment.

Automatic measurements should not be considered to be the be-all and end-all of bridge health monitoring. Its place is firmly entrenched in assisting engineers to conveniently carry out the damage detection, analysis, and evaluation of bridges. A retrofit mechanism was recommended for Kishwaukee Bridge. Retrofit is underway.

Any potential sensor development should consider its final use for reliability, durability, and user friendly design. In the course of EM sensor study and its final uses in many long-span bridges, we have observed the true experiences learned from many projects by understanding views from the designer, constructor, and the owner. Health monitoring design must be with a goal that is constrained by financial and technological limitations.

Acknowledgments

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