FRP INTERNATIONAL

The Official Newsletter of the International Institute for FRP in Construction

Volume 3, Issue 3 ***September 2006***

Welcome to the third issue of IIFC for 2006. As this goes to press the organizing committee for CICE is busy finalizing the program for what promises to be a very interesting set of activities this December in Miami. In addition to the conference itself there will be a number of meetings related to IIFC itself, including the awarding of a number of prestigious awards. I encourage all readers, whether they are IIFC members or not to attend this conference which promises in the Olympic tradition to be the "best ever."

We continue to seek submissions from our readers on new applications of FRP in construction, forthcoming conferences and workshops, or even general items that may be of interest to the worldwide community. Material can be submitted directly to me at vkarbhari@ucsd.edu or to any of the members on the advisory or editorial boards.

Please also feel free to write to me or to the President of IIFC, Prof. J.G. Teng (cejgteng@polyu.edu.hk), with any ideas you may have for the newsletter and for IIFC, itself.

Vistasp M. Karbhari, Editor-in-Chief
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In this issue

FRP INTERNATIONAL

The Official Newsletter of the International Institute for FRP in Construction

Volume 3, Issue 3 **September 2006**

Reports From Around the World

Dynamic Fatigue Response of a Truss Bridge with Fiber Reinforced Polymer Decks

by
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Fiber reinforced polymer (FRP) decks are gaining popularity among bridge owners as an alternative to replace old heavy concrete bridge decks to increase the live load capacity of old steel superstructures. Bentley Creek Bridge, the first FRP deck employed on a state owned highway system, is located in the New York State. The old deteriorated concrete deck, overlaid several times during its life span, was replaced few years ago with a lightweight FRP deck for increasing the live load capacity of the bridge.

One of the most important aspects that may be overlooked in situations such as above is the effects of rehabilitation process on dynamic response of the structure and fatigue life of the bridge. By reducing the self-weight of the deck, dynamic characteristics of the structure would be altered. Hence, a study was initiated to investigate the dynamic fatigue response of this Truss Bridge with Fiber Reinforced Polymer Deck.

Three-dimensional (3D) finite element analysis (FEA) was employed to perform dynamic fatigue simulations of two deck systems, i.e., FRP and concrete decks. The entire bridge, including steel trusses, were modeled and analyzed using a general-purposed finite element analysis package *ABAQUS*. The FEA model was verified using the data from load-test results (see Figures 1 and 2) conducted by the New York State Department of Transportation, immediately after the rehabilitation, to verify some of the design assumptions used [Ref. 1].

Figure 1. Bentley FRP Deck during the Load Testing

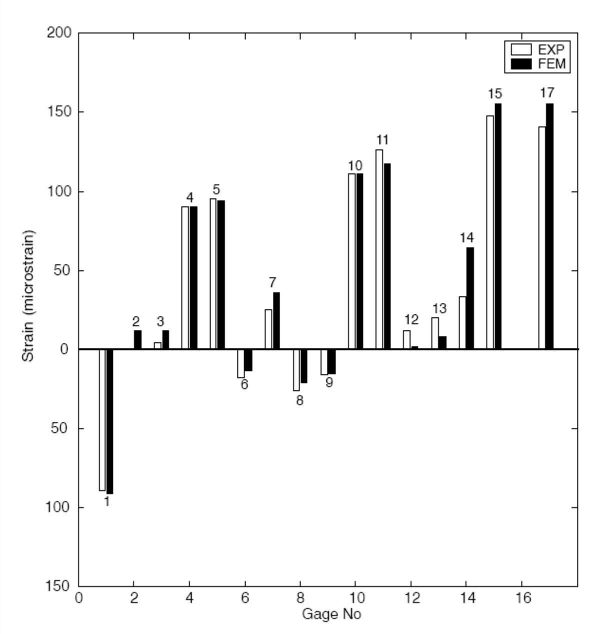

Figure 2. Verification of the FEM Model Using Field Test Data

Dynamic characteristics of FRP and concrete decks were obtained from finite element models by performing eigenvalue/frequency analyses. The first five fundamental frequencies of the bridge were extracted and used to determine Rayleigh damping parameters. In dynamic fatigue simulation, a nominal AASHTO fatigue truck was used. By assuming maximum allowable speed on Bentley Creek bridge of 50 mph, it would take approximately 2 seconds for a truck to travel across the bridge. A moving truck was simulated at eight discrete time instants (0.25 seconds time interval). An impact time duration at each time instant was assumed to be one-thousandth of a total traveling time, i.e., 0.002 seconds.

Dynamics analyses were conducted on both FRP and concrete decks. Fatigue life of each deck system was determined using AASHTO-LRFD fatigue resistance formulae [Ref. 2]. For the given ADTTs of 275 and 384 in years 2002 and 2020, respectively, the results show that trusses and steel floor-beams would be able to sustain infinite number of cycles without introducing any fatigue fracture or failure. Most importantly, the results show great improvement in fatigue life of the bridge after the replacement of concrete deck with FRP deck. The fatigue life of FRP deck system almost doubles when compared with the concrete deck system.

For more information on the testing and analysis reported in the article, please contact Dr. Sreenivas Alampalli of NYSDOT at salampalli@dot.state.ny.us .

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Simplified Finite Element Model for Honeycomb Fiber Reinforced Polymer Deck-on-Steel Girder Bridge Structures

by

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Honeycomb Fiber Reinforced Polymer (FRP) decks are a viable alternative for the rehabilitation of deteriorated reinforced concrete bridge decks in the U.S. These FRP decks have inherent high strength-to-weight ratio and low stiffness [Ref. 1]. Consequently, design of FRP honeycomb decks is governed by serviceability rather than strength. Furthermore, with the application of the existing AASHTO interim 2005 [Ref. 2] static deflection serviceability criteria, design of honeycomb FRP deck bridges tends to be uneconomical.

Due to the aforementioned inherent characteristics of the decks and the discrete nature of the deck-to-girder connections, which do not provide any composite action, the AASHTO serviceability criteria, intended to eliminate undesired vibrations, are not applicable to honeycomb FRP deck-on-steel girder bridges [Ref. 3]. The deck properties and connector characteristics generate a vibration response different from that of a reinforced concrete slab-on-steel girder bridge.

An alternative serviceability criterion [Ref. 3] for honeycomb FRP deck-on-steel girder bridges, based on the maximum peak vertical acceleration response to transient traffic loads and limited to human tolerance to vibrations [Ref. 4], is proposed. For development of this alternative criterion, the finite element (FE) method is utilized for predictions of acceleration response [Ref. 3]. As a first step, an appropriate FE model for honeycomb FRP deck-on-steel girder bridge structures is developed [Ref. 5].

A typical honeycomb panel fabricated by Kansas Structural Composites, Inc. (KSCI) is composed of several 305 mm wide units assembled together with two 12 mm thick top and bottom face sheets and 2 mm thick sinusoidal and flat honeycomb-type core elements extending along the longitudinal direction. An extra 6 mm thick wrap layer increases the strength and stiffness of each unit [Ref. 6]. The panel is constituted of E-glass fibers and either polyester or vinylester resins. On a bridge, several panel sections, with length equal to the bridge width, are placed transverse to the traffic.

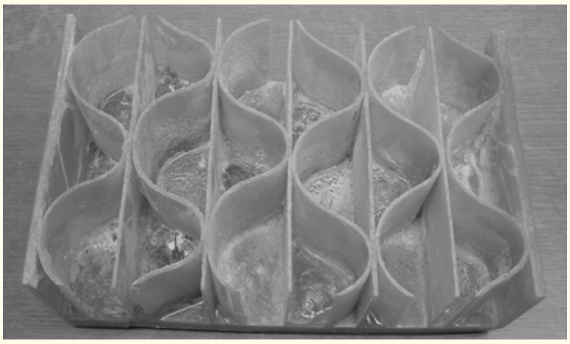

Figure 1. Honeycomb FRP structure

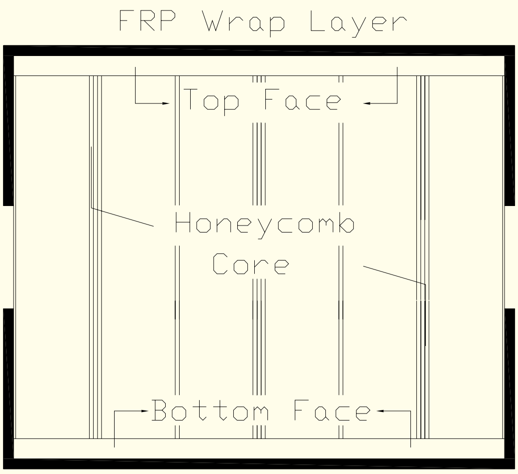

Figure 2. Cross-section of a Log

Figure 1 shows a unit honeycomb FRP structure, while Figure 2 shows the cross-section of one unit.

The deck-to-girder connections are placed at every 2.44 m, coinciding with the width of each panel usually shipped to the field by the manufacturer [Ref. 6]. As can be seen from Figure 1, honeycomb FRP decks have a very complex geometry, which results in an extremely large number of degrees-of-freedom (DOFs). This large number of DOFs produces a very computationally intensive finite element model. In order to decrease the number of DOFs, a simplified FE model is proposed [Ref. 5]. This simplified model maintains the original stiffness and mass, essential for modal and transient analyses of the complex honeycomb FRP cross-section. The simplified honeycomb FRP cross-section is represented as a box. The box top and bottom face thicknesses include the unit structure top and bottom face and wrap thicknesses, resulting in 19 mm. Each one of the box lateral walls contains one half of the sinusoidal and the flat core components, also including the wrap. An equivalent flat core thickness is determined using the formulation presented by Ugural [Ref. 7] for the stiffness of corrugated plates, resulting in 21 mm. The entire box is modeled using a 4-node and 6-DOF shell element, with rigid link connections between the top and bottom face nodes, except for those nodes coinciding to those of the core. The objective for use of rigid links is to avoid local deformation of the top faces.

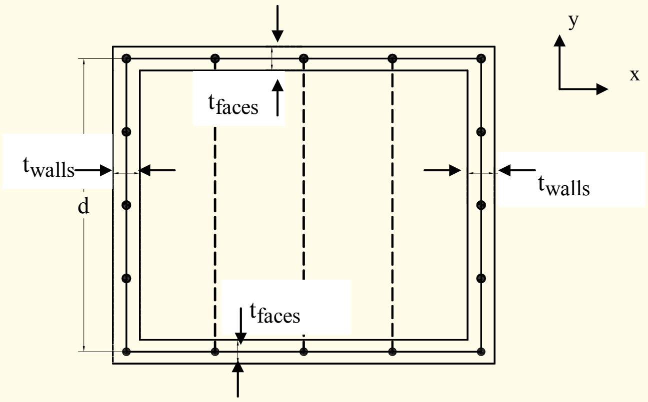

Figure 3. Simplified Honeycomb FRP Box

Figure 3 shows a box cross-section; rigid links are represented by dashed lines. The FRP material properties are given in [Ref. 5].

For the rest of the bridge, an eccentric beam model is used to represent the bridge girders, the guardrails, and the pier supports. The girder top flange, web, and bottom flange are modeled with 2-node and 6-DOF beam elements. The girder top flange nodes are connected to the deck bottom face nodes using linear springs in the X (traffic), Y (vertical), and Z (transverse) directions at every 2.44 m, representing the deck-to-girder connections [Ref. 5]. Between connections, linear springs in the Y (direction) connect the girder top flange nodes to the deck bottom face nodes. Meanwhile, the same 4-node and 6-DOF beam elements are used to model the guardrails. Rigid links represent the guardrail posts and connect the guardrail nodes to the deck top face nodes. Finally, the pier support is modeled using 1-node and 6-DOF mass elements. Rigid links connect pier support nodes to the girder and deck nodes [Ref. 5].

For validation of the proposed simplified FE model for honeycomb FRP deck-on-steel-girder bridges, two different experimental specimens are utilized: a honeycomb FRP beam and a reinforced concrete deck-on-steel girder bridge. The FRP beam is subjected to a static deflection test and a modal test, while the reinforced concrete deck-on-steel-girder bridge is also subjected to a modal test [Ref. 4]. The premises are: first, if the simplified FE model can mimic the results of an FRP beam, it can also mimic the results of an FRP deck; and second, if the FE eccentric beam model can mimic the results of a reinforced concrete deck-on-steel girder, it can also mimic the results of a honeycomb FRP deck-on-steel-girder bridge, if the correct stiffness and mass are input [Ref. 2]. These tests are simulated with the developed FE simplified model. The FRP beam is also modeled in detail (including the original honeycomb core component) for comparison with the simplified box cross-section. Table 1 gives the results of the FRP beam static and modal analyses, whereas Table 2 gives the results of the reinforced concrete deck-on-steel girder bridge modal analysis.

The results from the FRP beam analysis show that the simplified model represents very well the complex geometry of honeycomb FRP decks. Both the detailed and simplified models of the FRP beam mimic the experimental static deflection, with errors of 1.29% and 0.86%, respectively. Both the detailed and simplified models of the FRP beam also mimic the first three modal frequencies with errors of 0.41% and 0.62%, 0.33% and 7.89%, and 4.85% and 13.47%, respectively. Moreover, the simplified FRP beam model also represents well the numerical detailed FRP beam deflection and modal frequencies, with errors of 2.12%, 1.04%, 8.25%, and 8.22%, respectively.

In terms of number of DOFs, the detailed model has 145,580 DOFs, whereas the simplified model has only 4,560 DOFs.

Table 1. Comparison of the results of detailed and simplified models against experimental data – KSCI FRP beam

	Expt.	Detailed	Simplified
	1	2	3
Δ_y (mm)	59	60	58
f_1 (Hz)	48	47.8	48.3
f_2 (Hz)	152	151.5	164
f_3 (Hz)	297	311.4	337

Table 2. Comparison of the first three bending modes of concrete deck-on-steel girder bridge

Mode (Hz)	Experimental	FE
First	6.85	6.85
Second	9.25	9.21
Third	11.30	11.59

The results from the reinforced concrete deck-on-steel girder bridge modal analysis show that the eccentric beam FE model also represents the bridge response very well. The numerical results of the first three bending modal frequencies mimic the experimental values with errors of 0%, 0.43%, and 2.59%, respectively.

The results from both FRP beam and reinforced concrete deck-on-steel girder bridge analyses show that the proposed simplified FE model is a viable tool for the development of the alternative acceleration-based serviceability criterion for deck-on-steel girder bridges.

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Sandwich FRP Decks for Pedestrian Bridges in Washington, DC

by
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As part of the effort to revive the Anacostia Waterfront neighborhood in Washington DC, an 11-mile bicycle trail network that will extend from the Potomac River to the Maryland border is being designed and built. Part of the project was funded to demonstrate the use of composite materials for bridge structures. Two 1000 feet long pedestrian bridges will use Fiber Reinforced Polymer (FRP) materials. Historically, the primary use of composite materials in bridge infrastructure are FRP decks. This project continues that approach, but also expands FRP usage to the railing system.

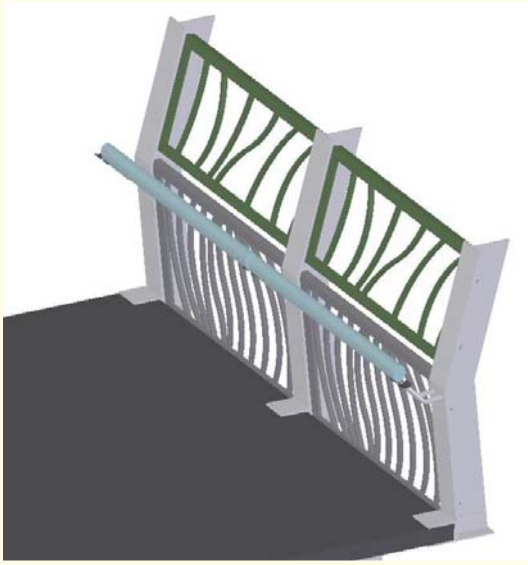

Figure 1. Long Fiber Thermoplastic (LFT) Railing System

The National Composite Center (NCC) leads composite implementation effort for these two bridges. The first part of the project was the design and manufacture of the FRP molded sandwich deck similar to FRP vehicular decks. To go one step further than other bridge deck projects, NCC teamed with architects from KPS (Kid Plosser Sprague) Group and the School of Engineering at University of Alabama at Birmingham (UAB) to develop a composite railing system that uses a new technology: Long Fiber Thermoplastic (LFT) compression molding. Composite railing system addition to the FRP deck means that more elements will be maintenance free.

To manufacture 2000 feet of pedestrian decking NCC teamed up with WebCore Technologies, supplier of a fiber reinforced foam core product called TYCOR, and Composite Advantage, a molder of large structural panels. Both companies are involved in FRP bridge decks. WebCore demonstrated performance of TYCOR® core for bridge applications on Hebble Creek Bridge at Wright Patterson Air Force Base where 32 feet wide and 17 feet long vehicular bridge was installed in July of 2001. Composite Advantage is currently molding a 4000 sf vehicular bridge deck for Summit County, Ohio.

High volume production of standard FRP bridge decks seemed to be the best way to lower the premium cost of the product. The composite bridge manufacturers went with high-volume manufacturing process called pultrusion. Pultrusion based FRP decks were mainstream of most FRP installations for past few years. However there is range of bridge projects that are unique and would benefit from custom design flexibility of FPR deck. Molded sandwich construction and TYCOR® core easily allow for customization of the bridge deck when needed. Changes in laminate thickness and core shear strength are easily made to meet the exact bridge requirements.

TYCOR® core is made by WebCore's unique patented process that integrates dry glass fiber reinforcements in a three-dimensional lattice through the thickness of low-cost, low-density foam to achieve z-directional reinforcement (Figure 2). TYCOR® materials are designed for use in liquid molding processes where the porous reinforcements act as resin channels during molding and co-cure with the skin layers. The result is an integrated sandwich panel with improved structural performance. The dry fiber rovings are infused with resin during molding. When cured, they form the structural members. Proprietary skin attachment features greatly enhance the core-to-skin bond in the sandwich panel and result in high strength and excellent damage tolerance.

Figure 2(a). TYCOR® Foam Strip Wound With Fiber

Figure 2(b). TYCOR® Consolidated FRC Core Preform Board

The molding process implemented to manufacture FRP deck panels for Anacostia bridges is a hybrid process combining

Structural RTM Light and WebCore Infusion Process (WIP). Traditional RTM Light technology typically uses high permeability reinforcements that allow resin to flow quickly at low pressures. Matched composite tooling is used to generate a two-sided surface finish and to minimize the use of consumables. The WIP employs a series of infusion grooves that are internal to the core. This internal configuration allows the panel to be highly structural, while allowing resin to flow quickly. This approach can ensure complete fiber wet-out during processing because the grooves are in the center plane of the panel so the part fills with resin from the center of the panel outward to the surface skin. By using the WIP vacuum infusion process, Composite Advantage gains fast fill time, high fiber content and the elimination of many consumable supplies. By using RTM Light technology, Composite Advantage gains fast cycle time, low consumable usage, greater consistency and a two-sided surface finish.

Figure 3. Fiberglass fabric layers for the lower skin are placed in the composite tooling bed.

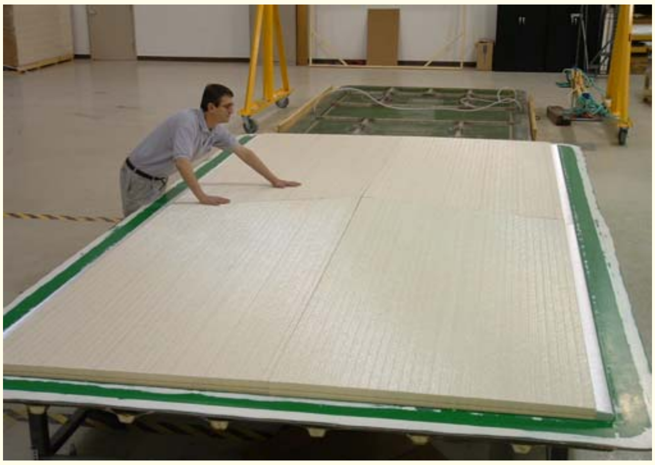

Figure 4. The fiber reinforced core is placed in the center of the sandwich. The longitudinal lines are the fiberglass shear webs.

There are additional benefits of using TYCOR® and WIP process. Since core is manufactured using automated process, the quality of the sandwich panel is improved by having cores with tight dimensional tolerances on core depth. Previous problems with sandwich FRP decks were caused by skin delamination from core. The "weak link" was bond between the core and skins. Since TYCOR® consists of fiberglass on core facings in addition to the fiberglass webs, the bond is substantially improved. An automated manufacturing process also creates core in high volumes at low cost. The core may be custom designed to handle different levels of punch-through shear stresses caused by wheel loads. The WIP infusion process aids in inspection of the end product. Since resin is injected starting in the mid-plane of the core and is delivered through core webs to skins, any imperfections in infusion are going to manifest themselves in the outermost skin of the sandwich panel. Fully infused skins assure fully infused core, which is otherwise hard to determine without using high-tech NDE methods.

The FRP deck for the first Anacostia Bridge is currently being built by Composite Advantage. It consists of 100 panels that are 195" x 100". Forty have been made so far at a rate of one deck panel per day. Both bridges will be installed in 2007.

Figure 5. Bridge Deck Panels for Pedestrian Bridge in Washington, DC.

Composite Advantage is currently building FRP deck panels for an HS25 vehicular bridge in Summit County, Ohio. This deck uses the same materials including TYCOR® as the Anacostia bridges. Composite Advantage is using the same unique molding process for the vehicular deck. The combination of materials and processes will be used on future vehicular and pedestrian projects currently in design.

A Low Power Wireless Sensor Network for Structural Health Monitoring

by

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Traditional monitoring systems have a star like topology with a data logger unit as center. Various sensors deployed over a structure are connected via long cable runs to that logging unit. These systems are very time consuming to install and therefore cost-intensive. The long cable runs are sensitive to electromagnetic interferences with the effect of reducing the accuracy of the acquired measurements and claiming for expensive high quality cables. Moreover, these cables are susceptible to mechanical damage involving additional maintenance effort. The monitoring system presented in this paper aims to solve this problems by introducing wireless data communication techniques.

The monitoring system is composed of three subsystems. Figure 1 gives an overview. The first subsystem is a wireless sensor network (WSN) installed on the structure. A WSN is a network made up of many tiny intercommunicating computers equipped with one or several sensors (Culler et al. 2004). Each tiny computer represents a node of the network. These nodes are called sensor nodes or motes. The motes are self-contained units typically consisting of a power supply, a radio transceiver, a micro controller and one or more sensors. The motes are discussed in more detail later in this section. The second subsystem, the control center, makes the information originating from the sensor network available to users or operators. It implements the data visualization, the long term data storage, and the remote monitoring and configuration tools of the sensor network. The third subsystem forms the link between the sensor network on the structure and the control center. This link is established using standard communication technologies (Internet or UMTS) and will not be described here.

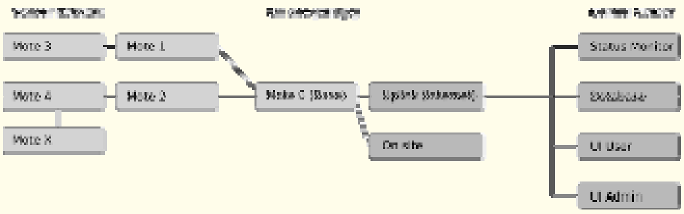

Figure 1. Overview of the monitoring system showing the wireless sensor network (left), the control components (right) and the link in between

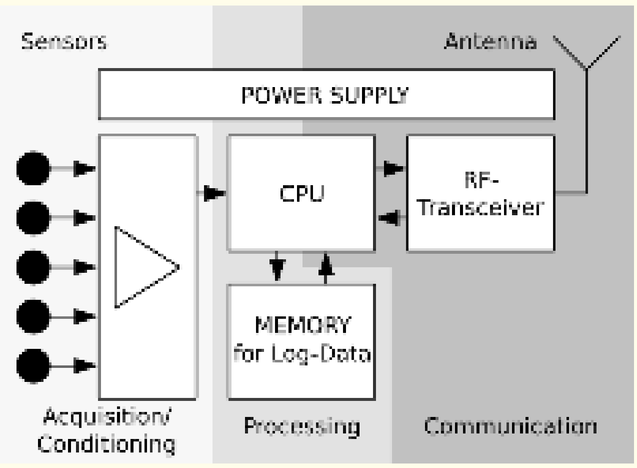

Figure 2. Basic components of a mote

The elimination of the cables connecting the sensors to the central logging unit solves the problems related to cabling but introduces new challenges concerning the power supply since the network has to be powered by an autonomous source like batteries or solar cells which are severely limited in capacity. The limited energy resources on each mote present the most restricting factor in designing and implementing wireless sensor networks. Therefore, measurement as well as communication strategies have to be designed taking into account these limited energy resources.

One option to save energy is to reduce the acquired raw data before transmitting it over the wireless link. This is often feasible since the raw data contains a redundant or even irrelevant information which can be discarded. An example for this kind of data reduction is given later in this section. Additional energy can be conserved considering the fact that the power needed to transmit data over a distance d is proportional to d^{α}, α ranging from 2 outdoors to approximately 4 indoors. Hence, in order to minimize power consumption, it is preferable to send the data over a chain of several short hops to the target instead of transmitting it directly over the entire distance. Therefore, each mote acts as data source as well as relaying station, forwarding data of other motes (multi hop network).

Motes are the fundamental components of the wireless sensor network. The functionality of a mote can be subdivided into four categories:

1. Sensing tasks including signal conditioning for different sensors, data acquisition and temporary storage of the acquired data.
2. Data processing, i.e. data reduction, analysis, diagnosis, detection of critical conditions and alarm rising.
3. Mote configuration functionality, i.e. dynamical configuration of sampling rates, query rates etc.
4. Communication functionality, i.e. data transmission, reception and forwarding of data from adjacent motes.

A mote is composed of one or more sensors, a signal conditioning unit, an analog to digital converter, a processing unit with memory, a radio transceiver and a power supply. Figure 2 illustrates these components. Various mote hardware platforms are available. This prototype network is based on the Tmote Sky platform (Polastre et al. 2005). It features a 6 channel ADC with a resolution of 12 bit, a 16 bit processor with 10 kB RAM and 48 kB program flash memory, and a radio transceiver operating in the 2.4 GHz ISM band with a raw data rate of 250 kbps.

A wireless sensor network is essentially a distributed computing system. The software running on each mote establishes the network, organizes the communication between the motes, acquires the measurements according to a predefined time schedule, performs the data processing and analysis and generates alerts when particular conditions are met.

The present prototype allows to collect information about the structural condition based on temperature, humidity, strain and acceleration measurements. Moreover, it permits to receive information about the internal state of each mote as well as the communication parameters of the sensor network. The energy constraints apply also to the software components. Therefore, all modules are implemented using energy-aware approaches in order to maximize the lifetime of the monitoring system.

The native operating system of the Tmote Sky platform is TinyOS (Levis et al. 2005); a component-based operating system designed for sensor networks and tailored to fit the memory constraints of the motes.

Each mote in the network can be equipped with different sets of sensors and, therefore, the software that is loaded on each mote needs to be configured appropriately. The parameters which characterize the mote configuration can be static or dynamic depending if they have to be set at compile time or if they can be modified at runtime. The present prototype allows the user to dynamically configure various parameters like sampling rates, query rates, number of values sent per data packet etc. Furthermore, it is possible to dynamically change the conditions for the triggering of alarm messages.

All the data received from the sensors is organized by attributes which are stored in the attributes pool on the mote and can be queried. If several queries have to be answered at the same time, they are merged together. There exist two types of attributes:

- Simple Attributes. These can be raw readings from sensors such as temperature, humidity or strain values which are acquired at low sampling rates. This group also includes the information describing the internal state of soft- and hardware as well as statistical information about the network configuration, e.g. number and IDs of neighbor motes, battery voltage, etc.
- Complex Attributes. These are physical quantities which need to be sampled with high sampling rates and require a in-mote post-processing step for reducing the amount of data.

The routing protocol is responsible for reliable and efficient packet delivery. This prototype uses multi-hop communication which forces several intermediate motes between source and destination to participate in data forwarding. Since radio communication is the most energy consuming task, the efficiency of the routing module influences considerably the lifetime of the whole sensor network. In addition, there is a trade-off between energy consumption and transport reliability, because if a packet is lost due to a too weak radio signal, it requires additional energy for its retransmission. A promising routing algorithm has been developed and implemented which accounts for link quality and energy consumption (Saukh et al. (2006). Since monitoring applications on civil structures typically do not require any mobility of the motes, the routing algorithm has been optimized for static networks.

Time synchronization is of utter importance in various monitoring applications. Furthermore, communication, coordination and power management also depend on the availability of global time. Due to this, the motes are kept synchronized using a Flooding Time Synchronization Protocol (Maróti et al. 2004). This protocol uses low bandwidth and is robust against node and link failures. The synchronization error per hop is below 0.1 ms.

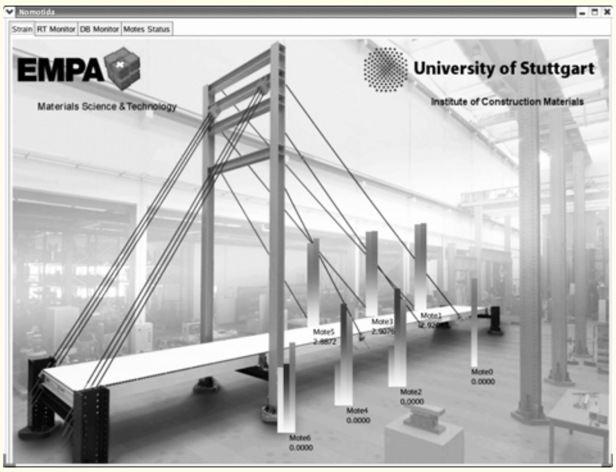

Figure 3. Screenshot showing the graphical user interface. The bars visualize different parameters acquired from the cable stays.

Signals which need to be sampled at high sampling rates produce a huge amount of data. For such complex attributes, the in-mote pre-processing is essential to reduce dramatically the number of data items which have to be transmitted. Often it is possible to extract a few parameters out of the raw measurement data stream which characterize the physical process adequately. This approach preserves considerable amounts of energy.

The data obtained from the sensor network has to be accessible for the system user or operator in a suitable way. The data items generated by the sensor network have to be stored and made accessible for further analysis steps. Various software components provide this accessibility and an operating system independent implementation of the software ensures flexibility and portability.

For the long term storage of the collected data, a PostgreSQL database is used. The applications access the database through JDBC (Java Database Connectivity), which provides methods for querying and updating data in a relational database. This connectivity component interfaces to a variety of databases and offers a high degree of flexibility. All events and attributes received from the sensor network are logged to the database. Furthermore, the data sent to the motes is also stored in the database, i.e. commands which contain network configuration data. Therefore, it is always possible to retrace a specific event.

The network itself and the motes can be configured and administrated remotely. This allows the operator of the monitoring system to remotely enable or disable the different measurements and change the attribute characteristics on a specific mote, calibrate various mote parameters, set up the detection of exceptional conditions and reset the mote. Furthermore, the motes can be reprogrammed remotely. This mechanism allows for a newly developed analysis algorithm or a completely new firmware to be easily installed on the motes.

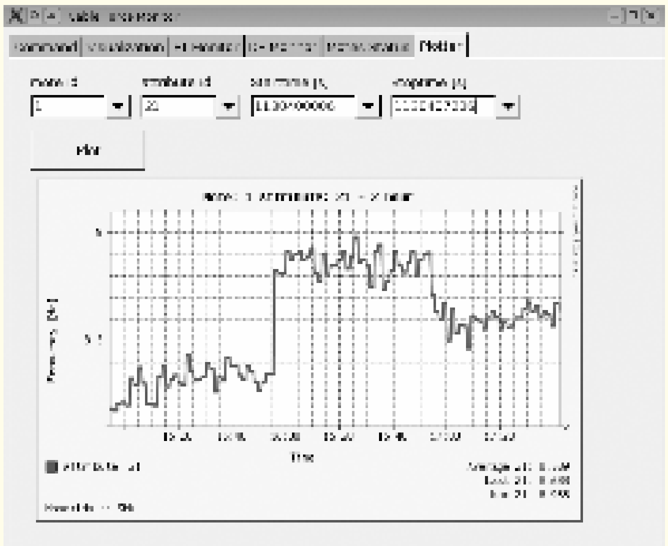

Figure 4. Screenshot of the graphical user interface. The graph window shows a plot of the estimated natural frequency of one cable stay. The visible steps in the plot correspond to three different loading states of the bridge deck.

The user interface integrates various highly modularized components. It enables the user to access the database and interact with the wireless sensor network. The basic access components are command line tools. These modules provide the low level access for the user. This level is the most suitable if the tools are used in highly automated monitoring setups and for system administration purposes. The graphical user interface is located one level above the command line tools. It is basically a framework that integrates the graphical interfaces of the various command line tools. Each command line tool has its own graphical user interface which can be executed standalone. This allows the administrator to assemble personalized user interfaces tailored to the needs and rights of the individual user (Figure 3). A third way to examine the structural behavior is to access the data using a web browser. The user interface components are mostly written in Java what enables a high interoperability on different platforms and operating systems.

A potential application of wireless sensor networks is cable tension force monitoring of cable bridges based on vibration measurements and natural frequency estimation. The natural frequencies are usually estimated by using frequency spectra computed with an FFT algorithm. However, these methods are too expensive in terms of memory usage to be applicable to wireless sensor networks. In the present monitoring application, a simple autoregressive model which requires significantly less memory has been used to estimate natural frequencies. The method has been successfully implemented and tested on the frp cable stay laboratory bridge at Empa. A detailed description of the setup and the implemented algorithm can be found in Feltrin et al. (2006).

The motes with the accelerometers were mounted on six cable stays. The bridge deck was excited with an electro-magnetic shaker driven by a broad band, stochastic signal ranging from 1 to 80 Hz. The accelerations of the cables have been simultaneously recorded with the motes and with a high precision data recorder. The natural frequency of each cable was estimated with a data block of 400 samples. Each data block was subdivided into 10 segments of 40 samples (0.4 seconds). With a clock speed of 8MHz, it took the microprocessor about 10s to complete the computation on a data block of 400 items. Figure 4 shows one natural frequency in a plot window of the graphical user interface. The changes of natural frequency are due to loading and unloading the bridge deck with additional mass. The natural frequency estimations show significant scattering. This scattering is due to the shortness of the data segments (0.4 seconds). The averages of the natural frequency of the three loading states are 8.51, 9.08 and 8.80 Hz. These values fit well with the results computed using the data recorded with the high precision data acquisition device: 8.47, 9.18 and 8.88 Hz. The latter figures were obtained by evaluating a 10 minutes record what corresponds to an accuracy of approximately 1%.

The presented prototype monitoring system has been successfully implemented and tested on the FRP laboratory bridge at Empa. The prototype shows that advanced monitoring tasks based on wireless sensor networks are feasible and that appropriate algorithms and strategies can be implemented which fit the limited memory and computation resources of the motes and provide accurate results. Additional laboratory and field tests are necessary to analyze the robustness of the soft- and hardware as well as measurement and analysis methodologies.

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ACKNOWLEDGMENTS

This work was performed within the FP6 Integrated Project "Sustainable Bridges: Assessment for Future Traffic Demands and Longer Lives". The authors acknowledge the European Commission and the Swiss Federal Office for Education and Science for their financial support.

Hybrid Bonding of FRP Laminate to Concrete Structures

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Up to date, three typical methods of bonding FRP reinforcement onto the concrete structures have been developed. The first method is by adhesively bonding FRPs to the external surfaces of reinforced concrete (RC) members, a method known as externally-bonded FRP or EB-FRP. The second one is the near-surface mounting (NSM) that increases bond strength but increase the risk of cutting existing reinforcement bars. The third one is the mechanically-fastened FRP (MF-FRP) recently developed at the University of Wisconsin, which employs a special type of FRP strip SafStrip^{TM} that has a significant bearing strength. Although significant effort has been made in the FRP community, premature debonding failure remains the critical failure mode that undermines the effectiveness of the FRP applications.

An alternative and new techongology of bonding FRP to concrete structures has recently been developed at City University of Hong Kong. This new technology combines the EB-FRP system with the MF-FRP system, namely the hybrid bonded FRP (HB-FRP) system. However, the mechanical fasteners in the HB-FRP system work in a very different way from that in the MF-FRP system. No bearing resistance in the FRP is required and, therefore, it is applicable to any existing commercially available FRP laminates. The application of the HB-FRP system consists of two steps. The first step involves the same procedure as that for EB-FRP system, by adhesively attaching the FRP onto the surface of the concrete. After hardening of the epoxy adhesive for EB-FRP, another coat of epoxy resin is applied on top of the FRP strip and the special mechanical fastener, as shown in Figure 1(a), is then installed along the longitudinal direction of the FRP reinforcement at a specified spacing, as shown in Figure 1(b). The installation of each mechanical fastener involves drilling two small holes in the concrete and driving the concrete nails into the two pre-drilled holes through the steel capping plate with a hammer, so that the capping plate firmly covers the FRP strips.

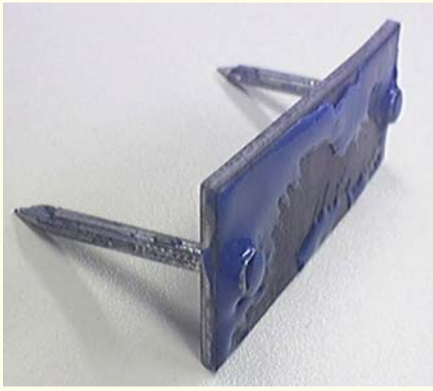

Figure 1(a). HB-FRP system: The mechanical fastener

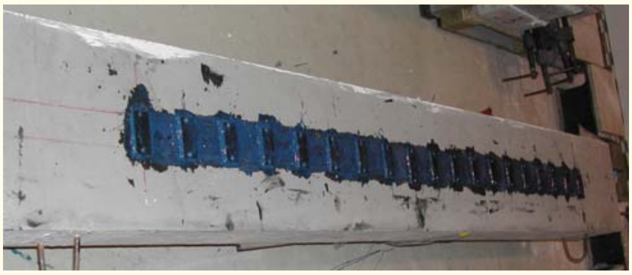

Figure 1(b). Test specimen after installation of the HB-FRP system

Experimental testing has demonstrated that this HB-FRP technique is highly effective in increasing the bond strength. Figure 2 gives the results of the three point bending tests for specimens with the dimensions of 150 mm (depth) × 300 mm (width) × 2500 mm (length). The test setup is shown in Figure 2(a) and the response curves are given in Figure 2(b). The lowest curve in Figure 2(b) gives the response of the member strengthened by the conventional EB-FRP, with 2 plies of 0.165 mm thick CFRP fabric. The other three response curves are for members strengthened with the HB-FRP system, with 2, 4 and 6 plies of CFRP fabric, respectively.

The EB-FRP system increased the flexural strength from the un-strengthened member of about 8 kN to 17 kN. The failure was debonding and complete detachment of the FRP strip from the bottom of the member, which indicated that the bond strength was smaller than that of the tensile break strength of the 2-ply CFRP strip. Both the members with the HB-FRP of 2 and 4 plies of CFRP failed due the rupture of the CFRP strip, as shown in Figure 2(c). This clearly indicated that the bond strength with the HB-FRP system was greater than that of the material tensile strength of the 4-ply CFRP strip and hence resulted in the rupture of the CFRP strip. The member with 6-ply CFRP failed due to debonding of the strip as shown in Figure 2(d). In this case, the bond strength of the HB-FRP system reached the maximum value, indicating that the tensile strength of the 6-ply CFRP strip was greater than that of the bond strength.

From the test result of the conventional EB-FRP strengthened member, it can be seen that the strength increment due to the EB-FRP system was about 9 kN (from 8 to 17 kN). In other words, the bond of the EB-FRP system contributed 9 kN of the flexural strength. The ultimate strength of the HB-FRP system was 70 kN. Taking away 8 kN contributed by the tensile strength of the steel bars, the strength due to the HB-FRP system was 62 kN. This was about seven times (62/9) that contributed by the conventional EB-FRP system. The effectiveness of the HB-FRP system is clearly demonstrated by these test results.

Figure 2(a). Experimental test results: Test Setup

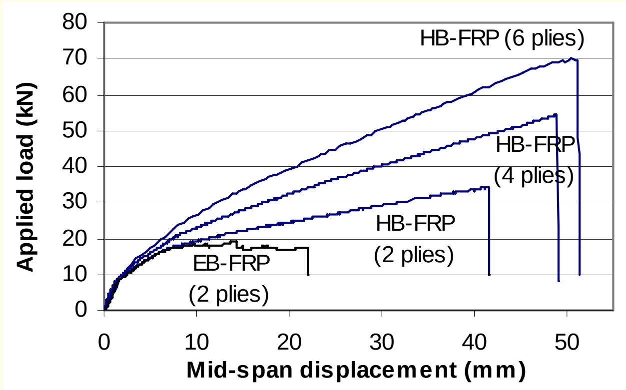

Figure 2(b). Experimental test results: Response Curves

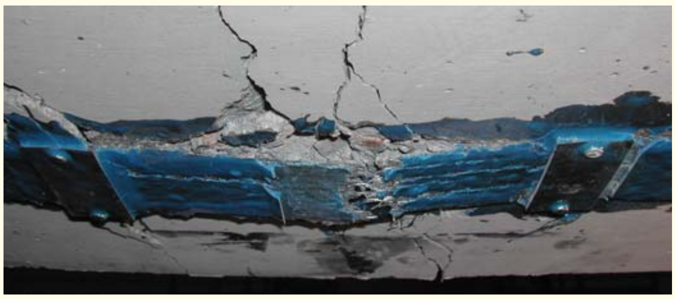

Figure 2(c). Experimental test results: Failure mode of FRP Rupture

Figure 2(d). Experimental test results: Failure mode of Debonding

Monitoring the Disbond of Externally Bonded CFRP Composite Strips for Rehabilitation of Bridges

by
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While more than 40% of the over 500,000 bridges nationwide are either structurally deficient or functionally obsolete mainly because of aging, rehabilitation of these bridges becomes the only feasible solution. Among a variety of rehabilitation technologies available, externally bonded Carbon Fiber Reinforced Plastic (CFRP) composite strips have been demonstrated to be one of most promising method for repairing and/or reinforcing damaged structures in a large scale to prolong their service lives and avoid loss of lives and assets [Ref. 1 to 5].

However, the reliability and integrity of the CFRP strips, particularly disbond between the strips and hosting structure, are still remained a major concern for their practical applications. Although fiber optic sensors and piezoelectric sensors have been used to monitor the health status of bridges rehabilitated with FRP [Ref. 6 to 8], there is a need to develop a viable and cost-effective diagnostic system to actively monitor the structure condition and detect damage while the structures are in service.

On the other hand, Structural Health Monitoring (SHM) is a revolutionary method of determining the integrity of structures [Ref. 9]. Recent advances in sensor technology, damage modeling, and system integration have enabled new developments in structural evaluation and inspection technologies to overcome the shortcomings of existing inspection systems. The SMART Layer technology, which is based on a built-in network of actuators and sensors, has proven to be a cost-effective potential technique to monitor the health of aerospace and civil structures [Ref. 10 to 12].

In this study, integrated active Structural Health Monitoring (SHM) systems have been developed as nondestructive evaluation tools for detecting the disbonds between composite repair patches and deck slabs of concrete in bridge rehabilitation.

The active SHM systems developed at Acellent consist of three components: (1) a sensor network (2) diagnostic hardware and (3) diagnostic software – integrated together and ready to use to perform in-situ monitoring, data collection, signal processing, real-time data interpretation and information management. The sensor network is permanently mounted on a thin layer of polyimide substrate material, called SMART Layer [Ref. 10]. The novelty of the SMART Layer lies in its networking capabilities with any type of sensor enhancing its monitoring capabilities and eliminating the need to place each type of sensor individually on the structure. Currently, there are two active modes for SHM: one is based on piezoelectric actuators and sensors, the other uses hybrid piezoelectric actuators/fiber optic sensors.

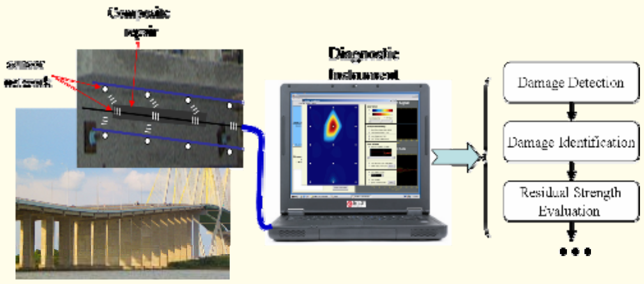

Figure 1. Schematic of active diagnostic system used to monitoring composite repair of bridge.

As shown in Figure 1, the scheme of an active system uses the piezoelectric actuators to input a controlled excitation to the structure and the neighboring sensors, either piezoelectric sensors or fiber optics sensors, to capture the corresponding structure response, which can then be interpreted in terms of damage location and size or material property changes within the structure. The methodology used in the diagnostic process is based on comparing the current sensor responses to previously recorded "baseline" sensor responses. The differences between the two sets of signals contain information about structural changes [Ref. 13].

The capability of the piezoelectric actuators and sensors based SHM system for detecting disbonds between the composite strips and the deck slab was demonstrated on a concrete bridge model in the laboratory.

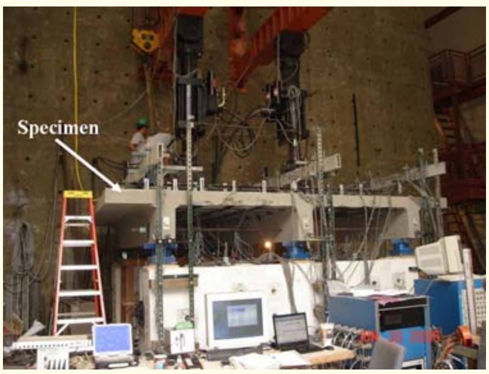

Figure 2. Specimen and test setup

As shown in Figure 2, the test specimen was a three-girder two bay bridge deck. The specimen was cast in place with 27.6 MPa (4000 psi) concrete keeping the construction procedure as close to field practices as possible. The specimen was placed on load cells at the two ends of each longitudinal girder, spaced at 3.2 m, to simulate simply supported conditions in the vertical and longitudinal directions. To restrict the movement in transverse directions, we simulated the effect of diaphragms between girders. Two hand tightened tie-rods were run across the transverse length through holes at the ends of the girders. The specimen was tested under monotonically increasing loads applied by two actuators, with the load cycled at predetermined levels to check for structural stability and to perform NDE. The load from the actuators was transferred to the test specimens through load pads having a footprint area of 508 mm x 254 mm representing the design wheel load contact areas specified by AASHTO (2004). The c/c distance between the load contact areas was 1.83 m, simulating the design axle distance of a permit truck.

The test was carried out in three phases. Phase 1 involved loading to initiate cracking in the slabs and causing them to reach 75% of punching shear capacity. This load level was deemed to be representative of deterioration in the deck slab that would warrant strengthening of the slabs with externally bonded FRP composites to prevent further degradation. Phase 2 involved loading to initiate shear capacity in the middle longitudinal girder, defined as 75% of yield in the internal steel stirrups, strengthened with externally bonded FRP composite stirrups. Phase 3, involved further loading of the test specimen until the strengthened slabs reached flexural capacity governed by debond of the composite strips and ultimate failure of the specimen due to local punching shear failure of the deck slabs. The progression of damage in the deck slabs and the girder was assessed using both visual and NDE techniques at beginning and end of each phase, as well as at intermediate load levels.

Six SMART Layers manufactured by Acellent Technologies, Inc. were integrated with two composite repair strips on the deck slab of the concrete bridge model after phase I of the test. There are eight piezoelectric disks, each having a diameter of 6.35 mm and thickness of 0.25 mm, on each layer with a sensor pacing of 178 mm (7 inches). For the three layers with each composite repair patch, two layers were bonded onto the top surface of the patch, and the other was embedded at the interface between the composite repair patch and the deck slab of the concrete bridge model. Figure 3 shows the external bonded CFRP composite repair patches with SMART layers on the deck slab.

Figure 3. CFRP composite repair patches with SMART layers.

After the CFRP composite repair patches implemented with SMART layers were bonded to the pre-cracked deck, baseline data was taken for further comparison purposes. When the load applied to the structure reached 90, 130, 150, 170, 190 and 210 kips (1 kips=6.9 MPa) respectively, the structure was then unloaded for the SHM system to take date. Sensor signals were recorded at both 0 and 24 kips.

The Damage Index (DI), which was developed to extract features in sensor signals related to damage in the structure [Ref. 13 and 14], was calculated for each actuator-sensor path. In this case, the Damage Index (DI) is defined as the relative ratio of the energy of scatter contained in a selected time window to the baseline energy contained in the same window. The Damage index is given as follows:

$$DI = \left[\frac{\int_{t_1}^{t_f} \left| S_{sc}(\omega_0, t) \right|^2 dt}{\int_{t_1}^{t_f} \left| S_b(\omega_0, t) \right|^2 dt} \right]^n$$

where S_{sc} denotes the amplitude of scatter signal; S_b denotes the amplitude of baseline signal; ω denotes the selected driving frequency; t_f and t_t denote the upper bound and lower bound of signal in time domain, respectively; n was set to 1 for this case. The scatter is the difference between the signals related the damaged structure and baselines. Based on the damage index on each path, a damage image can be generated by ACESS [Ref. 13]. When multiple paths are affected by a debonding, which results in big damage indices for these paths, their effects add up to show a heightened intensity of colors. This display technique can be used as a fast imaging method to help visualize the approximate location and extent of damage.

The images of damage at different loading stages are given in Figure 4. The images match the results of thermography and visual observation well. It is clear that the damage evolution in both composite patches was detected.

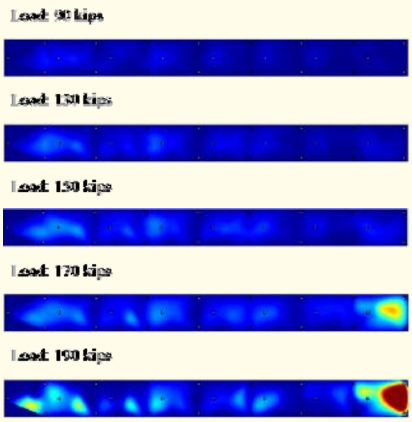

Figure 4(a). Damage images for two CFRP composite repair patches (1 kips=6.9 MPa): Composite Strip I.

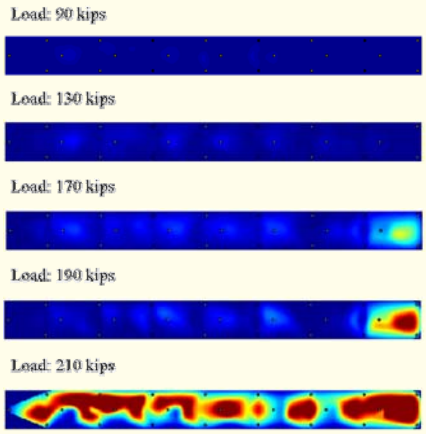

Figure 4(b). Damage images for two CFRP composite repair patches (1 kips=6.9 MPa): Composite Strip II.

Besides the piezoelectric sensors, Fiber Bragg Grating (FBG) sensing systems are being deployed in a wide variety of applications for structural health monitoring. To take the advantages of both piezoelectric transducer and fiber optic sensor, an active hybrid piezoelectric/fiber optic SHM system has been developed at Acellent [Ref. 15]. The hybrid system uses the piezoelectric actuators to input a controlled excitation to the structure and the fiber optic sensors to capture the corresponding structural response, as shown in Figure 5.

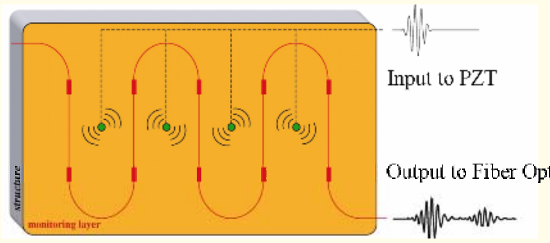

Figure 5. Hybrid piezoelectric actuators and fiber optic sensors network.

Similar to the piezoelectric actuators/sensors SHM system, the capability of the hybrid SHM System for damage detection has been demonstrated. As shown in Figure 6, the impact damage of approximate 100 mm^2 in the composite panel was successfully detected by using the hybrid SMART Layer.

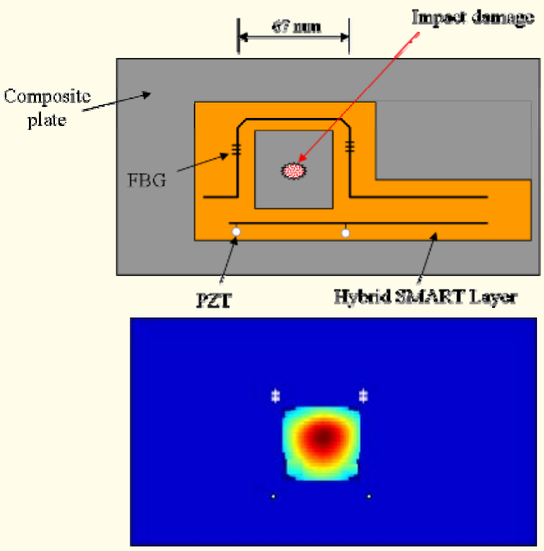

Figure 6. Hybrid sensor network used for damage detection (a) Hybrid layer mounted on the surface of a composite panel (b) image of impact damage.

The display technique used in ACESS [Ref. 14] was also employed in the hybrid SHM system. A heightened intensity of color on Figure 6(b) clearly indicates the presence of the impact damage. The major advantages of the hybrid SHM system include: 1) active damage detection, 2) the best actuator/sensor decoupling (minimum interference between actuation input signal and sensor output signal) and 3) few transmission losses during its optical signal nature.

As described in literature [Ref. 16], there is a major challenge in the networking of a multitude of piezoelectric sensors applied to physically large structures because of a large number of connection wires and big signal noise from long distance communication. The hybrid system could be a potential solution for the applications of SHM on large structures, such as the health monitoring of bridge and long pipeline structures. Besides much lese wires used and much less long distance signal transmission loss in the hybrid system, The fiber optic sensors in the system can also be used for temperature sensing, deflection measurement of the structure.

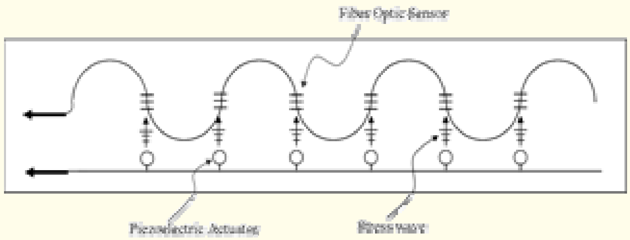

Figure 7. A typical design of hybrid piezoelectric/fiber optic sensor network.

Figure 7 shows a typical design of hybrid piezoelectric/fiber optic sensor network used in the SHM system. In the network, all piezoelectric actuators are connected to a diagnostic instrument through a smart cable. The distributed Fiber Bragg Gratings are used as sensors. The piezoelectric actuators can be simply connected together in series or in parallel, and then generate stress waves in the structure simultaneously. By using a set of control wires, each piezoelectric actuator could also be used to generate stress wave in the structure individually.

The smart cable with piezoelectric actuators and the optic fiber with distributed gratings can be placed on a thin carrier film, and then will be permanently bonded on the composite or metal structure to be monitored. The changes caused from the degradation of material properties (or corrosion) around all diagnostic paths will be identified. The design is particularly good for global damage monitoring of large composite or metal structures, such as bridges with composite repairs and metal pipelines.

Two active modes for Structural Health Monitoring are presented in the paper. The piezoelectric actuators and sensors based SHM system was demonstrated on a concrete bridge model in the laboratory. Test results showed that the system could detect the disbonds between the composite strips and the deck slab at different load stages well. The concept of the hybrid piezoelectric and fiber optic SHM system and its potential application in the rehabilitation of bridge were described. The piezoelectric SHM system has better sensitivity and is good for local damage monitoring, while the hybrid system can be used for global damage monitoring because only a few wires are used and long distance signal transmission loss is limited in the system.

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Global Focus: Iran

In this issue we highlight activities being conducted in the area of FRP in the construction industry in Iran.

We hope that this will be a continuing feature and invite future summary articles from our members worldwide. As can be seen from the list of authors, this also highlights international collaboration between scientists, engineers and practioneers across borders.

FRP Materials in Iran

by
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The use of FRP materials in the construction industry is a relatively new application worldwide. The development of this technology for construction started approximately 30 years ago, in a limited number of countries. In Iran, the use of FRP materials began a decade ago and can therefore still be categorized as an innovative material that is manufactured by modern technologies.

The application of composite materials was first investigated at the Iran University of Science and Technology. At this research center, research has been performed studying FRP composite properties, their durability and performance, as well as different production methods. As a result of these research activities, the Iran Composites Association (IRCOMAS) was established on 2004, which is a technical, non-governmental organization that aims to protect the interests of both suppliers and consumers in all types of composites industries. In order to disseminate information and to ensure a strong voice throughout the industry, IRCOMAS publishes a quarterly journal entitled "Composites". Each issue focuses on a particular subject, a more recent focus being the First International Composites Exhibition that took place in Iran in January 2005. Exhibitors and visitors from the 24 countries attended the First International Composites Exhibition, simultaneous to which IRCOMAS arranged a technical seminar with ten speakers.

At the University of Tehran, one of the top universities in the Middle East, significant research has been carried out on the properties, application, and also testing of composites materials. For example, the course on the "Application of Composites Materials in Structural Engineering" is given each semester by Prof. Masoud Motavalli, the Iranian member of International Institute for FRP in Construction (IIFC), and attended eagerly by many graduate students. This course covers topics such as an introduction to FRP material properties as well as their application in the retrofitting of existing structures. At the Construction Materials Institute of Civil Engineering Department, University of Tehran, many experimental and analytical research projects are conducted on the reinforcement of RC structures using FRP composites in the framework of PhD and Masters theses. Similar research activities are also being carried out at the Departments of Civil and Environmental Engineering at Sharif Universities of Technology and Amirkabir Universities of Technology.

In order to help spread the latest information on FRP materials among engineers and students, the Building and Housing Research Center of the Ministry of Housing and Urban Development held a conference on the "Application of FRP Composites in Construction and Rehabilitation of Structures" in May 2004. This event aided in the exchange of information among specialists and to further familiarize the consulting engineers of Iran with the technical specifications of fiber-reinforced polymers.

In addition to the above mentioned research activities, the application of FRP materials has been seen in a few real construction projects in Iran. The incomplete south tower of Esteghlal Hotel in Tehran was one of these projects (Figure 1).

Figure 1. South tower of Esteghlal Hotel, Tehran, Iran; south view and retrofitted beams and columns of the structure using FRP sheets

The tower was partly constructed 20 years ago, and, in order to complete the building, the client asked for a reevaluation and redesign of the building against seismic forces based on recent seismic codes. Through detailed consideration of the existing structure, a seismic retrofitting solution using FRP materials was chosen as the most suitable way for upgrading the current building in order to achieve the acceptable behavior against the earthquakes, the most common natural disaster in Iran. After preparation of seismic rehabilitation documents and drawings, the execution of the retrofitting project was awarded to an Iranian construction contractor through an international tender and bidding process. During the subsequent execution, many students, engineers, and experts visited the project in order to learn about the practical application methods used during seismic retrofitting of the tower with FRP materials.

The successful completion of the hotel project encouraged other clients to consider the application of FRP materials as one of the practical choices for the seismic rehabilitation and upgrading of existing buildings. Following the south tower of the Esteghlal Hotel, four other tall buildings including the east and west tower of the same hotel, the 28-floor tower of the Azadi Hotel in Tehran and the Shahid Mohammadi Hospital in Banddar Abbas (southern of Iran) were subjected to seismic evaluation resulting in a proposal to retrofit, among other techniques, using FRP materials.

Figure 2. Retrofitting of a typical Iranian masonry building, Dr. Ali Shariati's house, using FRP X-bracing.

Furthermore, the approximately 40-year old house of Dr. Ali Shariati, a famous Iranian intellectual, is an example of a typical Iranian masonry buildings that was upgraded using FRP materials following a comprehensive seismic analyses (Figure 2).

Official Iranian organizations have also made significant efforts to introduce FRP materials codified as well as providing the necessary guidelines for their testing and application. One of these governmental organizations is the Iranian State Management and Planning Organization, which plays a major role in advancing FRP materials. For example, the Technical Affairs, Criteria Codification and Earthquake Risk Reduction Bureau of the aforementioned organization, responsible for providing Iranian codes and guidelines, has published a comprehensive guideline entitled “The Guideline for Design Specifications and Construction of Strengthening RC Buildings using FRP Materials”. The document includes special requirements compatible with the Iranian codes and construction practices and provides recommendations for the selection, design, and installation of FRP systems for the external strengthening of concrete structures. Information on material properties, design requirements, installation, quality control and maintenance of FRP systems used for external strengthening is also presented in this document. The information can, for example, be used to select an FRP system for increasing the strength and stiffness of reinforced concrete beams, or the ductility of columns and other applications. Uniquely, the guideline includes a section that provides typical drawings for the retrofitting of existing RC structures using FRP materials. These drawings are certified and can be used for different upgrading aims such as joint strengthening as well as increasing the axial compression, shear, flexural (beams and slabs), or torsion capacity. At present, the publication of the proposed guideline is the final preparation stage and it will be published soon.

Since Iran is located in a region of high seismicity, occurrence of moderate and strong earthquakes is quite common. A program has therefore been organized by the Technical, Criteria Codification and Earthquake Risk Reduction Affairs Bureau of Management and Planning Organization to retrofit the important public and state buildings, infrastructures facilities and lifelines. One of the most emphasized parts of the program is using advanced technologies and materials such as FRP.

In the field of FRP materials production, some measures have also been taken in Iran. The production of glass fibers (GFRP) was initiated several years ago. The production of carbon fibers (CFRP) has also been started by the Advanced Fiber Development Corporation. The nominal production capacity of this factory is eight 15-ton units of producing carbon fibers.

In conclusion, although the use of FRP materials in the construction industry in Iran only started a few years ago, significant research activities have already been carried out on these advanced materials in most of the Iranian universities and research institutes. Many experimental tests as well as analytical studies have been conducted on different aspects of FRP materials including structural and non-structural characteristics, and, as a result, several books and papers have been published by Iranian researchers. All these facts point towards a promising future for FRP materials in Iran.

CICE Update

With the recent release of the preliminary program, the 3rd International Conference on FRP Composites in Civil Engineering (CICE 2006) promises to be an outstanding conference. The conference will be held in Miami, FL from December 13-15, 2006. Thirty-six sessions with papers from a wide range of leading international personnel will be held over the space of three days. Two special pre-conference workshops (FRP Repair for concrete bridges and buildings, and FRP repair for steel structures) will be conducted on December 12, 2006.

Keynote speakers from around the world will discuss prognosis of FRP in construction from the point of view of the research community, funding agency, repair industry, manufacturer, and government users.

In addition, the prestigious IIFC Lifetime Achievement Awards will also be presented to leading luminaries from the IIFC.

For additional information, please visit the conference website: http://www.iifc-hq.org/cice2006/

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