International Journal of Distributed Sensor Networks

# Sensing Methodologies and Sensor Networks for Health Monitoring of Civil Infrastructures

Guest Editors: Hong-Nan Li, Gang-Bing Song, Jeong-Tae Kim, and Ting-Hua Yi

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 358286, 2 pages
doi:10.1155/2012/358286

*Editorial*

# Sensing Methodologies and Sensor Networks for Health Monitoring of Civil Infrastructures

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Received 24 December 2012; Accepted 24 December 2012

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Large and complex civil infrastructures are especially susceptible to random vibrations, whether they are from high ground accelerations, strong wind forces, or abnormal loads such as explosions. Thus, it is imperative that the robust and continuous health monitoring system needs to be designed and implemented so as to meet the demanding goals of increasing structural safety and reliability, while reducing its operating and maintenance costs. The efficiency of a structural health monitoring system (SHMS) relies on the sensitivity of the acquired data to structural changes that may be obtained by an extended sensor network on structures as shown in "*Methodology developments in sensor placement for health monitoring of civil infrastructures*" and [1]. Recent advances in sensing technologies have enabled the use of large numbers of sensors for health monitoring of civil infrastructures. The sensors utilized in an SHMS are required to monitor not only the structural status including the stress, displacement, and acceleration but also those influential environmental parameters, such as the wind speed, temperature, and quality of its foundation. Since a large sensor network needs to be involved in an SHMS, how to deploy it in a complicated, spatially extended structure so that the data acquired from those locations will result in the best identification of structural characteristics will be a challenging task. On the other hand, a high cost of data acquisition systems (sensors and their supporting instruments) and accessibility limitation constrain in many cases the wide distribution of a large sensor networks on a structure. Especially, many civil structures have to be tested under operational conditions, in which the sensors are not easily amenable to be removed or changed [2, 3].

It is, therefore, of increasing interests to seek rational placement and design a method of sensor networks that contain as much information as possible about the health state of a structure. This special issue aims to aggregate the latest research efforts contributing to theoretical, methodological, and technological advances in the integration of various aspects of SHMS applications within a broad range of disciplines. Some research topics towards this target include intelligent and efficient algorithms in the design of sensor networks; evaluation criteria for different sensor networks placement methods; inherent relationship between different sensor networks placement methods; uncertainty, sensitivity, reliability, and redundancy in sensor networks; application of large scale sensors for civil infrastructures and others related.

A total of 26 peer-reviewed papers have been presented in this special issue. These include identifying the optimal number and location of sensors, as well as the robustness, which aims at maintaining the stability of the sensor network when some sensors malfunction, and so forth. Without a doubt, the papers reflect the state-of-the-art researches and developments of this subject. The Guest Editor would like to express his sincere appreciation to all the authors who have contributed their work to this special issue.

## Acknowledgments

The Guest Editor would also like to express his whole-hearted thanks to the reviewers around the world for their contributions in shaping this. This work was jointly supported by the Science Fund for Creative Research Groups of the NSFC (Grant no. 51121005), the National Natural Science Foundation of China (Grant no. 51178083, 51222806), and

the Program for New Century Excellent Talents in University (Grant no. NCET-10-0287).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 804394, 18 pages
doi:10.1155/2012/804394

*Research Article*

# Integrated Optical Fiber Sensing System by Combing Large-Scale Distributed BOTDA/R and Localized FBGs

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Received 5 July 2012; Accepted 3 December 2012

Academic Editor: Gangbing Song

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Structural health monitoring (SHM) has been regarded as a significant tool for the safety of civil infrastructures. Local fiber Bragg grating (FBG) sensor and distributed Brillouin optical fiber sensor have been successfully applied in civil engineering fields. Unfortunately, neither single FBG nor single Brillouin sensing technique can satisfy the requirements of simultaneously positioning full-scale structural damages and accurate local damage details. So it still matters to establish balance between localized high-precision measurement and large-scale distributed measurement with lower accuracy. This paper introduces an integration system by combining distributed Brillouin and local FBG sensors on one single-mode optical fiber. The temperature self-compensation and the data processing method of the integration system have been investigated. The integrated FBG/Brillouin sensor has been further packaged by glass fiber-reinforced polymer to be used as a robust and smart structural component. Finally the proposed integrated sensing system has been applied in three case studies, including investigation of reinforced concrete beam, smart cable, and smart steel strand to verify the feasibility. The test results illustrate that the integrated system can simultaneously provide accurate strain measurement at critical points by FBGs and the rough distributed strain measurement by Brillouin sensor, which can be further applied to various applications.

## 1. Introduction

In the last two decades, a large amount of critical infrastructures, characterized as high cost, large scale and required long service life, have been built all over the world. These critical structures are expected to survive after a long-term service and during or even the rare events such as earthquakes and fires. To secure the survivability of the critical structures in a long run, their health conditions need to be monitored and assessed in real time. Thus, structural health monitoring (SHM) technology has attracted considerable interests worldwide. SHM systems have been successfully deployed to understand the loads, environmental changes, and structural behaviors, which have also been applied to a number of case studies, for example, structural behavior monitoring of Guangzhou TV Tower, Water Cube, and Tsing Ma Bridge [1–6].

In general, damages are randomly distributed through the structure, and their locations are usually hidden. Also, the structural damages are time dependent and interrelated with various factors. Thus, a cost-effective SHM system is desired to provide damage distribution and current local damage assessment at the same time. Optical fiber sensor, with its unique advantages of immunity to electromagnetic interference, absolute measurement, high durability, distributed sensing capability, and locally accurate measurement, potentially provides a promising solution towards these issues. Among various optical fiber sensors, fiber Bragg grating (FBG) sensor shows excellent sensitivity to the strain and temperature [7–16], leading broad applications in the civil, marine, and aerospace engineering. However, the FBG sensors can only provide the strain measurement at localized locations, missing the damages away from the FBG sensors.

On the other hand, Brillouin optic time domain analysis/reflectometry (BOTDA/R) is a fully distributed optical sensing technique, which attracted worldwide attentions in recent years. BOTDA/R can provide strain or temperature information along the whole length of a single-mode fiber. The BOTDA/R sensing technique has been employed to a large number of applications, including bridges, dams,

pipeline, and tunnels [17–23]. Unfortunately, BOTDA/R also possesses its own disadvantages such as low sampling rate, low accuracy, and poor spatial resolution, which violates the measurement requirements of high accuracy, high frequency real-time monitoring capability for the structures.

Therefore, taking the advantages of both FBGs and BOTDA/R technique, a sensing system by combining FBGs and BOTDA/R technique together may yield a possible solution to simultanously provide distributed damage distribution and localized damage details. The integrated sensing technique has been investigated for strain and temperature measurements [24, 25]. However, the signal interference between the FBGs and BOTDA/R technique has not been fully studied. In addition, its temperature compensation for strain measurement still opens. In the past few years, a number of temperature compensation have been proposed, including multiple parametersensing, employing inverse expansion materials, applying polymer package, combining with other sensing techniques, and absolute temperature compensation [26, 27]. However, the compensation temperature sensors used in these methods do not measure the same temperature as that of the strain sensor. Besides, extra spaces are needed to install these additional temperature sensors, which may not be satisfied in some cases.

In an effort of providing an SHM system with both accurate localized and distributed measurements, in this paper, an integrated optical fiber sensing system is proposed by combining distributed BOTDA/R sensor and localized FBGs in one single optical fiber. The proposed integrated optical sensing system is then called BOTDA/R-FBGs. The system setup of BOTDA/R-FBGs has been investigated. With measurement signal from these two different sensing principles, the proposed integrated optical sensing system owns the capability of temperature self-compensation. In addition, the data processing method to distinguish the influences from the FBGs to the Brillouin sensing signal has also been investigated. The proposed integrated optical sensor has been packaged by GFRP and further applied into several structural components, which are not only robust but also smart for SHM purpose. At last, the smart components have been employed to several case studies to verify their feasibilities, including reinforced concrete beams and smart cables. The test results show that the proposed integrated BOTDA/R-FBG system works well and can be further applied to other associated applications.

## 2. System Integration of the BOTDA/R-FBG Sensing System

*2.1. Configuration of the BOTDA/R-FBG Sensing System.* Figure 1 shows the operational principle of the proposed optical sensing system. In this BOTDR-FBG system, the FBG sensors and the BOTDR sensor share the same single-mode optical fiber, which function as both sensing and transmitting unit. For the FBG system, the FBGs in the optical fiber are used as local measuring points, while the optical fiber itself is used as distributed sensor for the BOTDR system.

Figure 1: Operational principles of the BOTDR-FBG system.

Figure 2: Configuration of the integrated BOTDA/R-FBG system.

Figure 2 shows the schematic configuration of the BOTDR-FBG system. Here, we can use a fiber loop to design testing circuit at the end of OF sensor, so the BOTDA-FBG system can also be constructed. In the BOTDA-FBG system, light switch is used to separate the Brillouin and the FBG signal to avoid the damage of the FBG sampling system. In the following, this two hybrid sensing systems (BOTDA-FBG and the BOTDR-FBG systems) are named as BOTDA/R-FBG system. Light comes out from the light source, comes up to the light switcher, and travels through the single-mode optical fiber. When light encounters the FBGs, they get reflected back and the reflected signals are recorded on the FBG and BOTDA/R interrogators. With the light switcher, the light signals from BOTDA/R and FBGs can be recorded in time division independently. In this paper, an SI-720 instrument from Micron Optics Inc. has been used as the FBG interrogator and a DiTeSt STA 202 from Omnisens has been adopted as the BOTDA/R interrogator. On the single-mode optical fiber, several FBGs have been written to form an OF-FBGs sensor. To protect the OF-FBG sensor from external damage, it has been packaged by the highly durable material, such as polypropylene (PP) and fiber reinforced polymer (FRP). In field applications, the OF-FBGs sensor is installed along the structure member with the FBGs located at the potential damage locations. The actual damage locations and their spatial distribution will be identified from the BOTDA/R measurements while the detailed damage information can be acquired by the FBGs, respectively.

*2.2. Temperature Self-Compensation for the BOTDA/R-FBG Sensing System.* Without considering the cross-sensitivity of

strain and temperature, both the Bragg wavelength shift of FBG and the Brillouin frequency shift of BOTDA/R show excellent linearity to the strain and temperature as expressed below in binary matrix:

$$\begin{pmatrix} \Delta\lambda_B \\ \Delta\nu_B \end{pmatrix} = \begin{pmatrix} \alpha_\varepsilon & \alpha_T \\ C_\varepsilon & C_T \end{pmatrix} \begin{pmatrix} \Delta\varepsilon \\ \Delta T \end{pmatrix}, \quad (1)$$

where $\Delta\lambda_B$ and $\Delta\nu_B$ are the changes of the Bragg wavelength and the Brillouin frequency; $\Delta\varepsilon$ and $\Delta T$ are the changes of strain and temperature; $\alpha_\varepsilon$ and $\alpha_T$ are the strain and temperature sensitivity of FBGs, which equal to 1.2 pm/$\mu\varepsilon$ and 10.8 pm/°C, respectively [28]; $C_\varepsilon$ and $C_T$ are the strain and temperature sensitivity of BOTDA/R, which have values of 0.05 MHz/$\mu\varepsilon$ and 1.0 MHz/°C, respectively. So (1) can also be expressed as follows:

$$\begin{pmatrix} \Delta\varepsilon \\ \Delta T \end{pmatrix} = \begin{pmatrix} \alpha_\varepsilon & \alpha_T \\ C_\varepsilon & C_T \end{pmatrix}^{-1} \begin{pmatrix} \Delta\lambda_B \\ \Delta\nu_B \end{pmatrix}. \quad (2)$$

With measured changes of Bragg wavelength ($\Delta\lambda_B$) and Brillouin frequency shift ($\Delta\nu_B$), the changes of strain ($\Delta\varepsilon$) and temperature ($\Delta T$) then can be predicted accordingly. Thus, the temperature effect to the BOTDA/R-FBG system can be canceled out and self-compensated.

Test has been set up to verify the self-temperature compensation as shown in Figure 3(a). One unpackaged OF sensor, one OF-FBGs sensor, and one unpackaged FBG have been placed into a water tank with a gauge length of 1 m as can be seen in Figure 3(b). The unpackaged OF and the unpackaged FBG are set without mechanical loads and used as temperature sensors for absolute temperature compensation. On the other hand, both mechanical and thermal loads have been applied on the OF-FBGs sensor by the tensile instrument and the water tank. All the measurements from the three sensors have been recorded by the SI720 instrument for FBGs and DiTeSt STA200 instrument for BOTDA/R. Figure 4 shows the test results of self-temperature compensation and Table 1 lists the measured strain and temperature from the OF-FBGs sensor by using (2). The strain measured by the OF-FBGs sensor with self-temperature compensation agrees well with the reference strain calculated from absolute temperature compensation by OF or FBG, with a maximum relative error of 9%. Some of the measured strains or temperatures have large deviation with the references because of the temperature instability of water tank. However, taking the advantage of the convenience of the simultaneous measurement of strain and temperature, the OF-FBGs sensor can be a promising and cost-efficient sensing technique for field practices.

The self-temperature compensation test has been demonstrated for the GFRP packaged OF-FBG sensors as well. A static load is applied to the GFRP packaged OF-FBGs sensor for a short period. Figure 5 compares the variance of the strain measured by GFRP packaged OF-FBGs sensor with and without self-temperature compensation for two temperature states of the test. It can be seen that compared to the strain measured by the OF-FBGs sensor without the self-temperature compensation, the corresponding strain with

(a) Test profile

(b) Sensor layout

Figure 3: Test setup of self-temperature compensation verification.

the self-temperature compensation is much smaller than the actual strain. The self-temperature compensation property of the OF-FBGs sensor needs no additional temperature sensors, providing a promising temperature compensation solution for the sensing system for various applications.

## 3. Signal Interactions and Processing

FBGs use the broadband light source, but BOTDA/R uses a light source with a certain wavelength of 1550 nm. Thus the potential signal interactions exist between the BOTDA/R and FBGs signal, including interaction of the reflected light from different optical light sources. In addition, FBGs are somehow regarded as damages on the single-mode optical fiber. Their structures, typically the Bragg period, change with various mechanical and thermal loading, resulting in a complex Brillouin gain spectrum at the locations of FBGs, and the Brillouin gain curve may turn complex under complex external force within the spatial resolution.

*3.1. Interactions between BOTDA/R and FBG Signals.* Two GFRP packaged OF-FBG sensors, named as GFRP-OF-FBG1 and GFRP-OF-FBG2, had been tested for strain sensing under tension loads, and the two GFRP-OF-FBG sensors are connected to each other by using a free optical fiber. Each GFRP-OF-FBG sensor has two FBGs. GFRP-OF-FBG1 has a gauge length of 40 m, and the two FBGs, $FBG_1$ and $FBG_2$, have initial center wavelengths of 1525 nm and 1527 nm, respectively, while GFRP-OF-FBG2 has a gauge length of 70 m, and the initial center wavelengths of its two FBGs, $FBG_3$ and $FBG_4$, are 1540 nm and 1545 nm, respectively. Figure 6(a) shows the strains measured by GFRP-OF-FBG1 under various tension loads. With the initial

Figure 4: Test results for self-temperature compensation verification of OF-FBG.

Table 1: Simultaneous temperature and strain measurement by the OF-FBGs sensor.

| Reference temperature (°C) | Reference strain ($\mu\varepsilon$) | OF-FBG (°C) | OF-FBG ($\mu\varepsilon$) |
|---|---|---|---|
| 0 | 0 | 0 | 0 |
| 0 | 455 | −1.58 | 476 |
| 9 | 455 | 7.62 | 497 |
| 8.5 | 910 | 11.96 | 899 |
| 15 | 909 | 13.43 | 996 |
| 15 | 909 | 13.51 | 960 |
| 14.5 | 1136 | 9.78 | 1209 |
| 14.5 | 1136 | 8.66 | 1180 |

Figure 5: Field application of temperature intercompensation.

wavelengths of the $FBG_1$ and $FBG_2$ far away from 1550 nm, the strains measured by the BOTDA/R agree well with the strain measured by $FBG_1$ and $FBG_2$. From Figure 6(a), it has been seen that FBGs do not affect the Brillouin signal much if the center wavelength of FBGs is away from the wavelength of BOTDA/R instrument light source, 1550 nm. Figure 6(b) shows the strains measured by GFRP-OF-FBG2 under various tension loads. The obtained distributed strain of the BOTDA/R has a huge intensity loss at the locations closed to $FBG_3$ and $FBG_4$, which have initial center wavelengths close to 1550 nm. The more the wavelength of FBG is close to 1550 nm, the more the intensity loss of the BOTDA/R signal will be. In the two figures, there is a wave trough between the two FBGs at each load, which is the measuring value of the free optical fiber.

*3.2. Signal Processing of the OF-FBGs Sensor.* In an effort to eliminate the interactions of the FBGs to the Brillouin signal in the condition when the FBGs have initial center wavelengths around 1550 nm, in this paper, several signal processing techniques have been investigated and employed to denoise Brillouin signal, including wavelet analysis and curve fitting algorism.

*3.2.1. Wavelet Denoising for the OF-FBGs Sensor.* Denoising is a filter process that can get rid of the high-frequency noise and reconstruct the low-frequency signals. It uses the signal property that the Lipschitz index of noise is negative. The maximum modulus of noise decreases as the scale increases,

(a) Test results for GFRP-OF-FBG1

(b) Test results for GFRP-OF-FBG2

Figure 6: Strain measured by GFRP packaged OF-FBGs.

(a) Measured strain

(b) Strain after denoising

Figure 7: Measured Brillouin signal from OF-FBG sensor.

but the opposite applies for the actual signal. Various denoising processes exist, including fast Fourier transfer method and wavelet analysis. Wavelet analysis is an effective signal processing method to denoise signal in field application [29, 30]. The capability of multiscale enables the wavelet analysis to comprehensively show various abnormalities and changes of signal at different scales. In this paper, the nonsymmetric Daubechies wavelet (dbN) is used to denoise the measured Brillouin signal. Experiments had been employed to verify the signal processing based on wavelet analysis by installing one GFRP packaged OF-FBGs sensor on a structure under various tensile loads. The GFRP-OF-FBGs sensor has a gauge length of 20 m and the initial wavelengths of the two FBGs are 1525 nm and 1527 nm, respectively. Figure 7(a) shows the strain distribution of the GFRP-OF-FBGs at various loads. The measured strain spectrum shows a number of burrs or noise, caused by the interactions between FBGs and BOTDA/R and the instability of the measuring equipments such as the FBG and Brillouin instruments themselves and the loading devices. To eliminate the effect from these interactions, a db5 wavelet was selected to denoise the measured signal. Figure 8 shows the decomposition of the measured signal by using the db5 wavelet. The maximum modulus of the high-frequency components, denoted as D1, D2, and D3, decreases as the scale factor increases, as shown in Figure 8(a). On the other hand, the maximum

(a) High frequency of D4, D5 and low frequency of A5

(b) High frequency of D1, D2, and D3

Figure 8: Signal reconstruction for 5-scale one-dimensional wavelet decomposition.

modulus of other high-frequency components, named as D4 and D5, indicates good correlation with low-frequency component, named as A5, as shown in Figure 8(b). So the noise signal can be reconstructed from D3, D2, and D1, while the actual signal can be reconstructed by combining the low-frequency component of the measured signal, A5, and the high-frequency components, D5 and D4. Figure 7(b) shows the obtained strain after denoising by wavelet dB5. The strains flow smoother than the initial measured signals.

One strain gauge was installed at the end of GFRP packaged OF-FBGs to validate the strain measurement of the GFRP-OF-FBGs sensor. Figure 9(a) illustrates the strain measurement comparison of the GFRP-OF-FBGs sensor before and after db5 wavelet denoising to that from the strain gauge. Figure 9(b) shows the corresponding relative error. With the wavelet denoising, both the strain measured by the BOTDA/R and that by the FBGs agree well with that from the strain gauge. The maximum relative error of the GFRP-OF-FBGs sensor is 18% before denoising and 4% after denoising. The results indicate the feasibility of the wavelet denoising method for the data processing of OF-FBGs sensors.

*3.2.2. Brillouin Gain Spectrum Curve Fitting Algorithm for Complex Brillouin Gain Curve.* Brillouin distributed optical fiber sensing technology is based on the assumption that Brillouin frequency shift (BFS) obtained from Brillouin gain

(a) Strain measurement by various sensors

(b) Relative measurement error

Figure 9: Strain measurement comparison before and after denoising.

Figure 10: 3-D mapping of Brillouin gain spectrum.

spectrum linearly depends on the environmental temperature or applied strain on optical fiber. With an axial tensile load on the optical fiber, the Brillouin gain spectrum has a Lorentz profile, while it becomes complex for complex stress condition or damage within the spatial resolution. Figure 10 shows the comparison of the 3-D mapping of one GFRP packaged OF-FBGs' Brillouin gain spectrum to that of an unpackaged OF at certain temperature. Brillouin gain spectrum for the GFRP packaged OF-FBGs sensor turns to be more complex than that of the unpackaged OF, especially at the locations of FBGs. This complexity is caused by the initial stress induced by the packaging process and the interaction between the FBGs and the Brillouin spectrum. Typically, the algorithm to calculate the Brillouin frequency shift is not flexible, which cannot meet the required measurement accuracy of temperature or strain. Thus, in this paper, local Lorentz single-peak and localized least square fitting algorithms are proposed to fit the Brillouin gain spectrum. By applying optimization, a selected bandwidth of 50 MHz has been employed to fit the measured Brillouin gain spectrum with the proposed algorithms.

Experiments had been set up to verify the two proposed spectrum fitting algorithms, as shown in Figure 11. One GFRP packaged OF-FBGs sensor had been placed into a water tank with various temperature measurements. The test condition includes heating process and cooling process.

Figure 11: Setup of temperature test.

The temperature of the water tank is ranged from 23°C to 64°C. Figure 12 shows the Brillouin gain spectrum and the corresponding fitted curve by using the local Lorentz single-peak fitting algorithm at FBG location at various temperatures. Figure 13 illustrates the obtained Brillouin gain spectrum by using the localized least square fitting algorithm. Figure 14(a) shows the comparison of measured temperatures by BOTDA/R after using these two fitting algorithms. Figure 14(b) shows the corresponding relative error. The maximum errors from local single-peak Lorentz, localized least square fitting algorithms, and BOTDA/R measurement are 3.7%, 6.9%, and 7.5%, respectively. The test results indicated that, with the proposed curve fitting algorithms, the interaction of the FBGs to the Brillouin gain spectrum of OF-FBGs sensors can be eliminated.

## 4. Experimental Validation of GFRP-OF-FBGS Sensors

Considering the fragility of the optical fiber, proper packaging method is required for various practical applications. GFRP has the advantages of high strength, corrosion and fatigue resistance, and linear elasticity. More importantly, GFRP is naturally compatible with the optical fiber. Thus, GFRP is a potential packaging candidate for OF-FBGs sensors. In this paper, the fabrication of the GFRP packaged OF-FBGs (GFRP-OF-FBGs) sensors, their feasibility, and applications had been investigated as follows.

*4.1. Fabrication of GFRP-OF-FBGs Sensor.* In an effort to meet various layout requirements of the installation in different applications, two types of GFRP-OF-FBGs sensors are proposed, including the GFRP-OF-FBG-based rod and the GFRP-OF-FBG-based belt as shown in Figures 15(a) and 15(b), respectively. The GFRP-OF-FBG-based rod is able to be embedded into various structures for internal strain measurements. On the other hand, the GFRP-OF-FBG-based belt can be easily bonded to the surface of structures for surface strain measurements. Figures 16(a) and 16(b) show the fabricated prototypes of these two types of sensors. The anchors on both ends of the sensor are designed to improve the bonding between the host matrix and the sensors. The length of these anchors can be designed based on the requirements of the dynamic range of the strain measurements. In addition, with the GFRP packaging layer, several OF-FBGs sensors can be packaged into one GFRP layer for multiple parameters sensing.

Figure 12: Fitted BFS spectrum by Lorentz local single-peak fitting.

*4.2. Experimental Verification of the OF-FBG-Based Sensors for Strain Sensing.* The feasibility of the single-line OF-FBG sensors had been verified by series of experiments. One unpackaged OF-FBG sensor, one unpackaged OF sensor, and one GFRP-OF-FBG sensor were tested under static tension loads. One FBG had been placed on each of the OF-FBGs and the GFRP-OF-FBG sensor. The initial wavelengths of FBGs of the unpackaged OF-FBG and the GFRP-OF-FBG sensor were 1555 nm and 1566 nm, respectively. The gauge length of the unpackaged OF-FBG and OF sensors was 4.5 m. Considering the small diameter of the unpackaged OF and OF-FBG sensors, their tensile test had been set up as shown in Figure 17(a). The sensors were attached to two stages by using epoxy resin. One of the two stages was automatically controlled for axial movement. Thus, the reference strain

Figure 13: Fitted BFS spectrum by localized least square fitting.

Figure 14: Comparison of sensing temperature from various fitting methods.

applied to the attached sensors can be calculated by the relative movement of the stages. Figure 17(b) compares the measured strains by the unpackaged OF and OF-FBG sensors with the reference strain. The maximum relative error of the unpackaged OF-FBG is 6.5%, indicating that the measured strains by the unpackaged OF-FBG sensors agree well with the reference strain. The GFRP-OF-FBG sensor has a gauge length of 0.7 m. It had been tested by a tension machine as shown in Figure 18(a). The reference strain of the GFRP-OF-FBG sensor can be obtained by the attached extensometer as also shown in Figure 18(a). Figure 18(b) shows the strains measured by the GFRP-OF-FBG sensor compared to the reference strains. Comparing to the reference strains, the maximum relative measurement error of the GFRP-OF-FBG sensor is 11% application. The large error is mainly caused by the spatial resolution of the BOTDA/R measurement for the GFRP-OF-FBG sensor.

*4.3. Strain Measurement of Concrete Beam Based on the GFRP-OF-FBG Sensors.* The proposed GFRP-OF-FBG sensors had been applied to measure the strain distribution of a referenced concrete (RC) beam to prove their feasibility of field applications. The RC beam had a dimension of 2700 mm × 200 mm × 400 mm and it was loaded by a two-point loading. Loads were supplied by a hydrostatic machine with

(a) The GFRP-OF-FBG-based rod (b) The GFRP-OF-FBG-based belt

Figure 15: Types of GFRP-OF-FBG-based sensors.

(a) The GFRP-OF-FBG-based rod (b) The GFRP-OF-FBG-based belt

Figure 16: Sensor prototypes of the GFRP-OF-FBG-based sensors.

(a) Experimental setup (b) Measured strain distribution

Figure 17: Strain measurement by unpackaged OF-FBG sensor.

(a) Experimental setup

(b) Measured strain distribution

Figure 18: Strain measurement by the GFRP packaged OF-FBG sensor.

Figure 19: Schematic of the test setup and sensor layout.

Figure 20: Test scene of sensor installation.

a load step of 6 kN. In each load step, a static load had been maintained for 5 min for the strain measurement purpose. Figure 19 shows the schematic test setup and Figure 20 shows the test scene of the sensor installation. Various sensors had been placed on the bottom surface of the RC beam, including commercial electrical resistance strain (ERS) gauges, two unpackaged OF sensors, one unpackaged FBG sensor, one GFRP-OF sensor, and one GFRP-OF-FBG sensor. The GFRP-OF and GFRP-OF-FBG sensors had the same gauge length of 120 cm and they were fabricated in the shape of the rod. Both ends of the GFRP-OF and GFRP-OF-FBG sensors were welded onto the preembedded steel block on the RC beam to ensure perfect bonding conditions. The GFRP-OF-FBG, GFRP-OF, and unpackaged OF sensors were parallel to each other, and the ERSs were located at middle of the RC beam. The initial wavelength of FBG of the GFRP-OF-FBG-based rod is 1565 nm, which is far away from 1550 nm.

Figures 21(a) and 21(b) show the experimental results of strain measurements of the RC beam from various sensors mentioned above, for loading and unloading cycles. Strains measured by the GFRP-OF-FBG sensor agree with that from the GFRP-OF, unpackaged OF, and reference ERS sensors for the loading-unloading cycles. The maximum relative error between strain measured by the BOTDA/R and that by the FBGs is of 6%. Figure 22 shows strain distribution of the GFRP-OF-FBG sensor measured by the BOTDA/R system. The sensed strain from BOTDA/R system at the middle of the RC beam agrees well with that measured from the FBG technique. Figure 23 shows the comparison of dynamic range of the strain-to-load measurement for various sensors at the middle of the RC beam. The measured strain under load of 45 kN was away from the initial strain-load curve, indicating the occurrence of potential damages. Visual inspection found out that cracks with a length of 0.3 mm were viewed at the middle bottom of the RC beam. As the load applied on the

Figure 21: Comparison of the strain of the RC beam by various sensors in the loading-unloading cycle.

Figure 22: Measured strain distribution by the GFRP-OF-FBG sensor.

Figure 23: Strain-load measurement of failure of the RC beam from various sensors.

RC beam increases, cracks expand and the measured strain-load curves from various sensors began to separate from each other. When load went beyond 160 kN, the measured strain from the GFRP-OF-FBG sensor dropped quickly with the increase of the load, while that from other sensors increased. Visual inspection found that slippage occurred on the anchor

Figure 24: Configuration of a smart steel strand.

Figure 25: Tension test of the proposed smart steel strand.

Figure 26: Measured strain distribution of smart steel strand by the GFRP-OF-FBG sensor.

of the GFRP-OF-FBG sensor, resulting in the sudden increase of the measurement error. The slippage indicated that the dynamic range of the GFRP-OF-FBG sensor for strain measurement was highly dependent on the bonding between the sensor and the host matrix. The comparison of the GFRP-OF-FBG sensor to other types of sensors indicates that with proper sensor installation the proposed GFRP-OF-FBG sensor is able to measure strain accurately. Thus, the proposed GFRP-OF-FBG sensor is a promising solution for simultaneous localized and distributed strain measurement in practical applications.

## 5. Smart Structural Components Based on the GFRP-OF-FBG Sensors and Their Applications

*5.1. Smart Steel Strand Based on the GFRP-OF-FBG Sensors.* A smart steel strand has been proposed based on the GFRP-OF-FBG sensors to monitor its health condition during its service life. Figure 24 shows the schematic of

Figure 27: Layout of the smart stay cable.

Table 2: Comparison of the calculated internal force compared to the applied force.

| Load (kN) | BOTDA (kN) | Absolut error (%) | FBG (kN) | Absolute error (%) |
|---|---|---|---|---|
| 0 | 0 | 0 | 0 | 0 |
| 3.67 | 3.17 | 13.62 | 3.21 | 12.53 |
| 6.67 | 6.23 | 6.60 | 6.63 | 0.60 |
| 10 | 9.19 | 8.1 | 9.13 | 8.7 |
| 13.33 | 12.58 | 5.63 | 13.22 | 0.83 |
| 16.67 | 16.00 | 4.02 | 16.06 | 3.66 |
| 20 | 19.03 | 4.85 | 19.69 | 1.55 |
| 23.33 | 22.59 | 3.17 | 22.63 | 3.00 |
| 26.67 | 26.48 | 0.71 | 26.46 | 0.79 |
| 30 | 29.38 | 2.07 | 29.56 | 1.47 |

the configuration of the proposed smart steel brand. It consists of a GFRP-OF-FBG-based rod and six steel wires. So the internal force of the smart steel brand consists of the force applied on the steel wire and the FRP rod. The GFRP-OF-FBG-based rod is located in the middle of the smart strand and it has a diameter of 5 mm. As the strand deforms, the GFRP-OF-FBG-based rod will deform together with the six steel wires. With the real-time measurement of the strain change of the GFRP-OF-FBG-based rod, the deformation of the steel strand can be directly obtained. The health condition assessment of the steel strand can then be employed based on these measured strain information. In the practical application, the installation of the proposed smart steel strands is the same as the traditional strands.

With the measured strain from the GFRP-OF-FBG-based rod, the internal force of smart steel strand can be obtained as follows:

$$F_s = E_s A_s \varepsilon_s + E_{\mathrm{FRP}} A_{\mathrm{FRP}} \varepsilon_{\mathrm{FRP}}, \tag{3}$$

$$\varepsilon_s = \varepsilon_{\mathrm{FRP}}, \tag{4}$$

where $\varepsilon_s, \varepsilon_{\mathrm{FRP}}, F_s, A_s$, and $A_{\mathrm{FRP}}$ are the strain of steel wires, the strain of the GFRP-OF-FBG-based rod, internal force of steel strand, cross-section of the six steel wires, and the cross-section of the GFRP-OF-FBG-based rod, respectively.

Figure 28: Fabricating process of the proposed smart stay cable.

Figure 29: Schematic diagram of the real-time monitoring system for the proposed smart stay cable.

Figure 30: Static test of the smart stay cable.

One proposed smart steel strand with a length of 3 m was fabricated and tested. The initial wavelength of the FBG on the GFRP-OF-FBG sensor was 1527 nm. The smart steel strand was tested under tension as shown in Figure 25. Figure 26 shows the measured strain distribution of the smart steel strand under various loads. The GFRP-OF-FBG-based rod not only provided accurate strain in local FBG location, but also gave out the strain distribution along the whole GFRP-OF-FBG sensor. The strain distribution measured by the GFRP-OF-FBG sensor agrees well with that measured by the localized FBG. Thus, the internal force distribution along the smart steel brand can be obtained based on (3). Table 2 illustrates the comparison of the calculated internal force compared to the applied force. The maximum error between the BOTDA/R measurement and the FBG is 8.7%, indicating that the proposed smart steel strand is able to assess the health condition of the steel strand.

*5.2. Smart Stay Cable Based on the GFRP-OF-FBG Sensor.* A smart stay cable is proposed based on the combination of the GFRP-OF-FBG sensors with the stay cable wires. Two GFRP-OF-FBG-based rods were symmetrically placed in the stay cable together with other steel cable wires as shown in Figure 27. Two ends of the GFRP-OF-FBG-based rods and the steel cable wires were anchored together by the mixed epoxy and iron sand in the anchor cup. With torsion-induced frictions between the steel wires and the GFRP-OF-FBG-based rods, the GFRP-OF-FBG-based rods are supposed to deform cooperatively with the steel wires. Thus, the behavior of the stay cable can be monitored by the GFRP-OF-FBG sensors. Figure 28 shows the fabrication procedures for the proposed smart stay cable. Firstly, two GFRP-OF-FBG-based rods with the same length of the cable wires were symmetrically laid along with the cable wires as shown in Figures 28(a) and 28(b). The smart stay cables were then protected by polyethylene tube, as shown in Figure 28(c). Secondly, the GFRP-OF-FBG-based rods were placed in series and were fusion-spliced together. The splicing point was protected by a brass tube as shown in Figure 28(d). Finally, a mixture of epoxy and iron sands was infused in the anchor and solidified by heating, as shown Figures 28(e) and 28(f).

Figure 29 shows the schematic diagram of the real-time monitoring system for the proposed smart stay cable.

(a) Cable force distribution (b) Average cable force distribution

Figure 31: Distribution of measured cable force.

(a) Measured average cable force (b) Relative error of cable force

Figure 32: Cable force comparison by various sensing techniques.

The FBGs are located at critical locations, including the middle and the anchorage zone of the smart cable. With the strain information measured from the GFRP-OF-FBG sensors embedded in the smart cable, the cable fore ($F$) can be calculated as follows:

$$F = F_s + F_{\mathrm{FRP}} = n \cdot E_s \cdot \varepsilon_s \cdot A_s + E_{\mathrm{FRP}} \cdot A_{\mathrm{FRP}} \cdot \varepsilon_{\mathrm{FRP}}, \quad (5)$$

where $F_{\mathrm{FRP}}$ and $F_s$, $E_s$ and $E_{\mathrm{FRP}}$, $A_s$ and $A_{\mathrm{FRP}}$, $\varepsilon_s$ and $\varepsilon_{\mathrm{FRP}}$ are the forces, elastic modulus, area of the cross-section, and strain of the steel wires and the GFRP-OF-FBG-based rods, respectively; $n$ is the number of wires in the steel cable.

Based on the calibrated strain sensitivities of FBGs and Brillouin technique, the obtained force sensitivities of four types of smart stay cables were calculated and listed in Table 3. It can be seen that when the number of the steel wires increases, the force sensitivity of the GFRP-OF-FBG sensor decreases significantly.

To verify the feasibility of the proposed smart stay cable for strain measurements, a series of experiments had been

Table 3: Cable force sensitivity of the proposed smart cables with various numbers of steel wires.

| Number of wires ($n$) | 61 | 73 | 85 | 109 |
|---|---|---|---|---|
| FBG cable force sensitivity (c: nm/kN) | $2.43 \times 10^{-3}$ | $2.03 \times 10^{-3}$ | $1.74 \times 10^{-3}$ | $1.36 \times 10^{-3}$ |
| BOTDA/R cable force sensitivity (c: MHz/kN) | $10.1 \times 10^{-2}$ | $8.47 \times 10^{-2}$ | $7.27 \times 10^{-2}$ | $5.67 \times 10^{-2}$ |

employed. A smart steel cable with two GFRP-OF-FBG-based rods, (named GFRP-OF-FBG$_1$ and GFRP-OF-FBG$_2$, resp.) had been fabricated. The initial wavelengths of the two FBGs in the two FRP-OF-FBG sensors were the same, with values of 1525 nm and 1527 nm, which were far away from 1550 nm. The smart cable had a length of 20.3 m. It had 61 steel wires. Each wire had a diameter of 7 mm and an elastic modulus of 210 Gpa. The fracture force of the smart cable, $P$, was expected to be of 3990 kN. An ultratension test was conducted on a horizontal tensile testing machine at the JuLi Group Ltd. in China, as shown in Figure 30. The tension machine has a loading capacity of 1000 T. The load sequences of test were 878 kN, 1000 kN, 1317 kN, 1756 kN, 2195 kN, 1756 kN, 1317 kN, 1000 kN, and 878 kN. Two pressure sensors were applied on both ends of the test machine to control the applied force. The applied force was kept for 3 min at each load step for strain measurements.

Figure 31(a) shows the measured cable force distribution at various loads by the two installed GFRP-OF-FBG sensors and Figure 34(b) shows the average measured cable force by the two sensors. The variance of frictions along the cable induced the fluctuation of measured cable forces along the cable length under axial tension. Here, the measuring signals were denoised by the wavelet above mentioned.

Figures 32(a) and 32(b) show the comparison of the measured cable force and the relative error by the distributed GFRP-OF-FBG sensor and the localized FBGs on the sensor to that from the commercial presser sensor. The cable forces measured by the proposed smart stay cable agree well with the applied load measured by commercial pressure sensors with a relative error of less than 5%. Therefore, the proposed smart stay cable is applicable for the practices.

*5.3. Prestress Monitoring of a Steel Frame Using the Proposed Smart Steel Strand.* The prestress loss of a steel frame structure was monitored by the proposed smart steel strand. Figure 33 shows the dimension of the monitored steel frame and the setup of the prestress. The steel frame was made up by three 1.0-meter-long sections and was prestressed by a hydraulic jack. The jacking force was measured by a pressure sensor. A seven-wire smart steel strand was placed along the steel frame to monitor the prestress loss. After the jacking force reached a predetermined level, the smart steel strand was anchored. Other than the GFRP-OF-FBG sensor, a GFRP-OF sensor was also installed in the smart steel strand. The prestress loss in the smart steel strand mainly resulted from the elastic deformation of the steel frame and the initial slippage of the anchorage after the jacking force had been transferred to the frame structure.

Figure 34 illustrates the changes of prestress and the corresponding prestress loss at the middle of the steel strand as a function of time over a period of 100 hours. It can be seen that the prestress loss occurs mainly at the beginning, induced by the transfer of a jacking force to the steel frame. The measured prestress loss from the proposed smart steel strand agrees well with that from the pressure sensor and the FBG sensor, indicating that the proposed smart steel strand is able to apply in prestress loss for structural practices. Figure 35 shows a temporal-spatial prestress force distribution of the steel strand with various time periods. Figures 34 and 35 indicate that the stress in the steel strand remains almost constant after the initial prestress loss.

Figure 33: Setup of a steel frame for prestress loss monitoring by the proposed smart steel strand.

## 6. Conclusions

This paper presents a novel optical integrated sensing technology by combing distributed fiber-optic Brillouin sensing technology (BOTDA/R) and local fiber Bragg grating (FBG) sensing technique, named the BOTDA/R-FBG sensing technique. Based on the discussion of principles, signal processing methods, self-temperature compensation, packaging, and applications, the following conclusions are obtained.

(a) In the BOTDA/R-FBG system, FBGs do not affect the Brillouin signal unless the FBGs' wavelengths are close to 1550 nm. Wavelet denoising method and local single-peak Lorentz fitting algorithm can effectively eliminate the signal interaction between FBGs and the BOTDA/R technique, when the FBGs' wavelength is close to 1550 nm. Thus, the strain and temperature measurement accuracy can be improved.

(b) The measured strain by self-temperature compensation agrees well with that by the absolute temperature compensation. Considering that the self-temperature compensation can reduce the cost and complexity of sensor installation significantly, the proposed integrated sensing technique is cost efficient.

(a) Measured stress from various sensors

(b) Measured prestress loss from various sensors

Figure 34: Monitoring of prestress loss by the proposed smart steel strand.

Figure 35: Stress distribution along the steel strand within 100 hours.

(c) To protect the OF-FBG sensor from potential frugality, GFRP material is proposed to be used as packaging materials. With comparisons to various sensing techniques, the GFRP-OF-FBG sensor had been validated for strain measurement by experiments.

(d) Smart steel strand and smart stay cable based on the GFRP-OF-FBG sensors had been investigated. The feasibility of these smart components for strain and force monitoring had been verified by experiments and practical application case studies.

Based on all the above investigation, it is confident to draw the conclusion that the proposed BOTDA/R-FBG collinear technique introduced in this paper is a promising method to meet the need of the large-scale strain measurements for practical application, such as airports, large-scale dams, and highways.

## Acknowledgments

The authors are grateful for the financial support from the National Natural Science Foundation of China (NSFC) under Grant nos. 50978079, 61040031, and 51108065, China Postdoctoral Science Foundation under Grant no. 20100481233, the National Science-Technology Support Plan Projects (2011BAK02B02), the National Program on Key Basic Research Project (973 Program) (2011CB013700), and the Ministry of Education Program for New Century Excellent Talents.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 414692, 12 pages
doi:10.1155/2012/414692

*Research Article*

# Performance Evaluation of PPP-BOTDA-Based Distributed Optical Fiber Sensors

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Received 30 August 2012; Accepted 16 November 2012

Academic Editor: Jeong-Tae Kim

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The basic sensing behaviors of Pulse-Prepump Brillouin Optical Time Domain Analysis- (PPP-BOTDA-) based distributed fiber optic sensors (DFOSs) with a spatial resolution on the order of 0.1 m are first experimentally studied. Second, the sensing results for different types of optical fibres are compared. Based on these investigations, the performance of PPP-BOTDA—based DFOSs for crack monitoring is evaluated. The experimental results show that in the interior of an optical fibre, slippage between the bare fiber and the coating materials occurs and that different types of optical fibres have different degrees of slippage. Point fixation bonding methods can produce effective and correct measurement results when the gauge length of the optical fibre is longer than the critical effective sensing length (CESL), whereas the overall bonding method is still not feasible for monitoring localized deformation quantitatively. Finally, the differences between Brillouin Optical Time Domain Reflectometry (BOTDR) and BOTDA are discussed.

## 1. Introduction

As a novel and advanced sensing technique, fiber optic sensors (FOSs) have been extensively researched for structural health monitoring (SHM) purpose due to their distributed sensing, rapid data transmission, small dimension, ease of installation, and immunity from electromagnetic influence. Among them, the Brillouin-based sensing is a one type of the most useful and important sensing techniques [1–8]. Investigations on the application of optic fiber sensing techniques for SHM have been widely carried out and significant progress has been made in the recent years [3–14]. In addition to the aforementioned advantages, the Brillouin scattering-based distributed fiber optic sensors show some potential merits over general FOSs for SHM as well, which can be characterized by the following advantages. The optical fiber used as strain sensors can be installed in several and even scores of kilometers. Along the optic fiber, the sampling point can be taken every 5 cm, and thus the strain distribution along the entire length of optical fiber can be obtained in a continuous and distributed way. The sufficient measurement can solve the problem of instability including nonuniqueness and discontinuity of solutions that many damage detection algorithms have encountered. Through the point fixation (PF) installation of the optical fiber, there will be formed a uniform strain distribution between two bonding points, the interval between these two bonding points can vary from scores of centimeters to several meters, and the measured strain is the uniform strain between these two bonding points. Thus, the measured stain is less susceptible to local stress/strain concentration and hence more representative of the entire structural member, which is in favor of monitoring the global behavior of structures.

One of the most applicable approaches for the Brillouin scattering-based distributed optical fiber sensing technology method is the Brillouin Optical Time Domain Reflectometry- (BOTDR-) based technique as demonstrated by Horiguchi et al. and Bao et al. [1–5]. Some investigations

Figure 1: Principle of BOTDA.

on the application of BOTDR techniques for SHM have been carried out [1–6] and significant progress has been made. However, the spatial resolution of BOTDR technique is 1 m with a strain measuring accuracy of $\pm 50\,\mu\varepsilon$. Consequently, the BOTDR sensing technique is difficult to directly monitor the localized deformation (fine crack) of civil structures. In our previous investigation, loop installation of Optical fiber is a good countermeasure [6], but this method is still not convenient for monitoring the practical structure. Recently the newly developed Pulse-Prepump Brillouin Optical Time Domain Analysis (PPP-BOTDA) sensing technique improves largely the spatial resolution (10 cm), and some applications of PPP-BOTDA-based DFOSs for structural health monitoring have been carried out [6, 7].

The improvement in the spatial resolution significantly increases the sensing performance of DFOSs for structural health monitoring. In this paper, to make a good use of this powerful tool, the basic sensing behaviors of PPP-BOTDA-based DFOSs with a spatial resolution on the order of 0.1 m are experimentally studied, and the sensing results from different types of optical fibers are compared. Based on these investigations, the performance of PPP-BOTDA-based DFOSs for crack monitoring is evaluated. Finally, the differences between BOTDR and BOTDA are discussed.

## 2. PPP-BOTDA-Based Distributed Sensing Technique

*2.1. Measurement Principle of BOTDA [8, 9].* The PPP-BOTDA technique is based on stimulated Brillouin backscattering. Two laser sources are introduced into the optic fiber from different ends of the fiber. One is a pulse laser (pump laser) source, and the other is a continuous laser source. When the frequency difference between the two lasers is equal to the Brillouin frequency shift, the Brillouin backscattering will be stimulated and energy transfer will be generated between the two lasers as well, as shown in Figure 1.

The Brillouin frequency shift $\nu_B$ changes in proportion to the variation of the strain or temperature, and the linear relationships between the Brillouin frequency shift and the strain or temperature are as follows:

$$\begin{aligned} \nu_B(T_0, \varepsilon) &= C_\varepsilon(\varepsilon - \varepsilon_0) + \nu_{B0}(T_0, \varepsilon_0), \\ \nu_B(T, \varepsilon_0) &= C_T(T - T_0) + \nu_{B0}(T_0, \varepsilon_0), \end{aligned} \tag{1}$$

where $C_\varepsilon$ and $C_T$ are the strain and temperature coefficients, respectively, and $T_0$ and $\varepsilon_0$ are the strain and temperature that correspond to a reference Brillouin frequency $\nu_{B0}$. Spatial information along the fiber length could be obtained through the time-domain analysis by measuring the propagation times for light pulses traveling in the fiber. Thus, continuous temperature and strain distributions along the fiber can be obtained.

*2.2. Spatial Resolution.* The key to improving the spatial resolution of Brillouin scattering sensing is to shorten the pulse width of the laser pulses. However, for the normal BOTDA sensing technique, when the laser pulse width is shorter than 28 ns, the phonons cannot be fully stimulated, and the stimulated Brillouin gain is also decreased. As a result, the measuring accuracy deteriorates abruptly. To overcome this difficulty, a pre-pump technique has been developed in which two laser sources are introduced into the optical fiber, as shown in Figure 1. The main difference is the pump laser source, which actually includes two types of pulses. One is a prepump laser (PL) that is applied to fully stimulate phonons, and the other is a detection pump (DP) used as a detecting laser. Due to the existence of the PL, the phonons can be fully stimulated before the arrival of the DP. As a result, the pulse width of the DP can be decreased to 1 ns without influencing the stimulation of the Brillouin scattering and the stimulated Brillouin gain. Accordingly, a spatial resolution of approximately 10 cm has recently been realized.

*2.3. The Measurement Device.* Significant progress has been made in the development of distributed Brillouin scattering-based optic fiber sensors for improving spatial resolution and measurement accuracy and stability. A strain/loss analyser (Neubrexcope-6000, Neubrex Co. Ltd. in Japan.) based on

(a) Cross-section of the optic fiber

(b) Layout of fiber optic sensors

(c) Steel specimen.

Figure 2: Measurement behavior of fiber optic sensors.

Figure 3: Strain profile along fiber optic sensors.

the PPP-BOTDA technique is used for continuous strain distribution measurement with an optic fiber sensor. Table 1 summarizes the specifications of the Neubrexcope-BOTDA system.

## 3. Measurement Behavior of Fiber Optic Sensors

The nominal error of the Neubrexcope is $\pm 25\,\mu\varepsilon$, and the minimum spatial resolution of the Neubrexcope is 0.1 m. However, these values are the measured results of non-strained FOSs in a homoisothermal environment. To account for the influence of different factors such as hardware settings, slippage in fiber optic sensors, and fluctuation of the environmental temperature, the actual measurement error may be larger than the nominal value and the measured strain profile will be different from the actual strain profile of the host structure. Therefore, it is necessary to investigate the measured behaviour of BOTDA-based distributed optical fiber sensors experimentally so that the measurement accuracy of BOTDA-based DFOSs can be evaluated.

*3.1. Experimental Investigation of the Basic Measurement Behaviour of BOTDA-Based DFOSs.* The strain measurement behaviour of FOSs had been examined through basic tests. In an experiment conducted by the authors, 13 segments with different sensing lengths (0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0, 1.2 and 1.4 m) of one piece of optical fiber (type layout of 1; its cross-section is shown in Figure 2(a)) were adhered on the surface of a uniform tensile steel specimen by the overall bonding method. In addition, different sensing segments were separated from each other by an interval of 2 m of free fiber, as shown in Figure 2(b). The applied strain level of the steel specimen was controlled by a strain gauge.

### 3.2. Measurement Behaviour of a Type 1 Optical Fiber

*3.2.1. Strain Profile along a Fiber Optic Sensor.* BOTDA monitors the strain profile along the optical fiber with a 5 cm sampling interval. Figure 3 shows the average strain profile of 10 repeated measurements along the whole optical fiber under loading levels of 300, 800, and 1500 $\mu\varepsilon$.

Figure 4: Measurement values of optical fibers with different sensing lengths.

Figure 5: Sensing behaviour of optical fibers.

Table 1: Specifications of Neubrexcope systems.

| Sampling resolution | 5 cm | | | |
|---|---|---|---|---|
| Average count | $2^5$–$2^{23}$ times | | | |
| Pulse width (ns) | 1 | 2 | 5 | 10 |
| Spatial resolution (m) | 0.1 | 0.2 | 0.5 | 1.0 |
| Dynamic range (dB) | 1 | 2 | 3 | 5 |
| Max. measurement distance (km) | 1 | 5 | 10 | 20 |
| Strain measurement accuracy ($\mu\varepsilon$) | ±25 | ±25 | ±25 | ±25 |
| Repeatability ($\mu\varepsilon$) | ±50 | ±50 | ±50 | ±50 |
| Temperature test accuracy (°C) | ±1 | ±1 | ±1 | ±1 |

According to Figure 3, the lengths of the measured strain distributions for different strained FOS segments remain constant at different load levels.

*3.2.2. Measurement of the Strain Value of Optical Fibers with Different Sensing Lengths.* Under loading levels of 300, 800, and 1500 $\mu\varepsilon$, the 10 measurement strain values of optical fibers with different bonding lengths are shown in Figure 4.

Despite the nominal accuracy of strain measurement is ±25 $\mu\varepsilon$; according to the test results in Figure 4, the maximum difference among 10 times measurements is up to about 100 $\mu\varepsilon$, therefore, in order to monitor actual structure, the strain measurement accuracy of BOTDA needs a substantial increasing.

Based on Figure 4, one important phenomenon should be noted. Despite the spatial resolution of 10 cm (nominal value), if the length of the strained FOS segment is shorter than 0.4 m, the measured strain value of BOTDA is still less than can be detected with the strain gauge. To explain this phenomenon, the strain ratios between the measured BOTDA value and the strain gauge for different load levels are analyzed, as shown in Figure 5.

(a) Length of strained segment less than the CESL (b) Length of strained segment less than the CESL

Figure 6: Strain redistribution in FOS.

(a) (b)

Figure 7: Sensing behaviour of the optical fiber (type 2).

According to Figure 5, the larger the applied strain, the smaller the strain ratio of the measured value of BOTDA and the strain gauge. Therefore, slippage between the bare fiber and the coating materials may occur. Subsequently, the strain in optical fibers will redistribute, as shown in Figure 6(a), and this strain redistribution lowers the measured strain of BOTDA and results in a measured value of BOTDA below the correct value. At present, the detailed strain distribution of FOS still remains an unresolved problem. Nevertheless, based on the aforementioned analysis, to obtain the correct measured value by using this type of FOS, the length of the strained FOS should be longer than 0.4 m. This length is defined as the critical effective sensing length (CESL).

If the length of the strained FOS segment is longer than the CESL, after strain redistribution caused by slippage, there still exists upwards of 10 cm (spatial resolution) of uniform strain distribution whose value equals the applied strain value to ensure that the BOTDA system can obtain a correct measurement, as shown in Figure 6(b).

*3.3. Measurement Behaviour of a Type 2 Optical Fiber.* To check the internal slippage of the FOS, another type of optical fiber (as shown in Figure 7) was adhered to the same tensile steel specimen, and the same experimental procedure was performed. Eight segments with different lengths (0.1 m–0.16 m, 0.18 m, and 0.2 m) are set in series. Under a loading level of 800 $\mu\varepsilon$, the 10 measured strain values of the different strained FOS segments are shown in Figure 7.

According to Figure 7, if the length of the strained segment is longer than 0.16 m, the error associated with the measurement is smaller than 40 $\mu\varepsilon$ (5% of the applied strain level), and the measurements of BOTDA are close to the correct value. Obviously, the CESL of a type 2 FOS is 16 cm.

### 3.4. Measurement Behaviour of a Bare Optical Fiber

*3.4.1. CESL of a Bare Fiber.* For an FOS, if the length of the strained segment of the FOS is shorter than the CESL, the measured strain of BOTDA is less than that of the strain gauge. The actual spatial resolution of BOTDA is greater than 10 cm, which needs to be investigated further. To completely prevent slippage of the FOS, the UV coating of the type 2 FOS is peeled away, and the bare optic fiber is used to directly carry out the same experiment. The experimental results are illustrated in Figure 8.

From the experimental results depicted graphically in Figure 8, if the sensing length of the bare FOS is longer than 13 cm, BOTDA can produce the correct measurement. Thus,

Figure 8: Sensing behaviour of the optical fiber (bare fiber).

Figure 9: Gap between the sampling point and the starting point of the strained fiber.

the CESL of a bare optical fiber can be considered to be 13 cm.

When the length of the strained segment is only 0.1 m, the measurement is exceptionally undervalued, which can be explained. During the measurement, the sampling interval of PPP-BOTDA is 5 cm; generally, the 0.1 m uniform strain distribution is rarely considered exactly within a certain sampling point measurement by the PPP-BOTDA with a 0.1 m spatial resolution. There is a gap between the sampling point of BOTDA and the starting point of the strained segment, as shown in Figure 9. The range of the gap is considered to be from 0 to 2.5 cm. Thus, only the length of the strained bare FOS is longer than 0.125 m, to ensure that at least one sampling point can measure a 10 cm completely uniform strain distribution, and this sampling point can obtain the correct measured value. Therefore, the PPP-BOTDA sensing technique has a spatial resolution on the order of 10 cm.

Based on the aforementioned experimental result, different types of FOSs have different CESLs, and the CESL parameter can be used to evaluate the anti-slippage property of FOSs. Obviously, if the FOS has a worse anti-slippage property, the deformation of the strained segment will extend into the range of the adjacent non-strained FOSs. To measure a uniform strain distribution whose value equals the applied strain and is more than 10 cm in length (spatial resolution), the longer strained length is necessary so that the BOTDA system can obtain the correct measurement. Therefore, the longer the CESL is, the worse the anti-slippage property becomes [7].

*3.4.2. Measurement Behaviour of BOTDA When the Strained Length of FOS Is Less Than the Spatial Resolution.* In our previous investigation on BOTDR [6], when the strained length of FOS is less than the spatial resolution (1 m), the BOTDR device can obtain certain strain values. This characteristic can improve the ability of BOTDR to monitor localized deformation. In this section, an experiment similar to that described in Section 3.1 is used to investigate the measurement behaviour of BOTDA in the case in which the strained length of the FOS is less than the spatial resolution. A bare optical fiber is used in this experiment. The measured results are illustrated in Figure 10.

Based on the experimental results, at different load levels when the strained length of the FOS is less than the spatial resolution, the measured results of BOTDA are close to 0 and there is a large variance among the 10 repeated measurements. Consequently, if no internal slippage occurs, the BOTDA system is not able to directly monitor localized deformation whose strained range is less than the spatial resolution.

## 4. Performance Evaluation of PPP-BOTDA for Crack Monitoring

### 4.1. Test Method

*4.1.1. Specimen.* In this study, standard experiments are proposed to calibrate the crack measurement performance of the BOTDA-based distributed optical fiber sensors and to evaluate the accuracy of the measurement. The schematic illustration of the applied standard specimen is shown in

Figure 10: Strain redistribution in the FOS.

Figure 11: OB and PF installation of the FOS.

Figure 12(a). Two rectangular concrete blocks are connected with two strips of FRP sheets, and the small gap between the two blocks is utilized to simulate an ideal concrete crack. The specimen is axially loaded under a load control mode. Upon loading, the gap becomes wider, and its width is measured by BOTDA-based DFOSs. Simultaneously, the width of the gap is measured by means of displacement transducers (DTs) for comparison.

*4.1.2. Bonding Method of the FOS.* For different measurement purposes, two installation methods for attaching the optic fiber sensor to the surface of the specimen are proposed and studied. One is termed the overall bonding (OB) method. In the OB method, the necessary measured length of the fiber optic is bonded completely to the surface of the specimen with resin. The OB method is suitable for monitoring strain distribution over a large area. The second installation method is termed the point fixation (PF) bonding method. In the PF bonding method, the two ends of the fiber optic sensor are bonded to the specimen to form a uniform strain distribution within the two bonding points. The measured value of this uniform strain distribution area can be regarded as an average strain value of the gauge length. Consequently, if the gauge length is longer than the spatial resolution, the PF-installed FOS can be effectively used to monitor and measure the crack width of civil engineering structures. Figure 11 schematically demonstrates the OB and PF installation.

*4.1.3. Placement of the FOS.* In the experiment carried out by the author, the two types of optical fiber described previously were used. For the type 1 optical fiber, the gauge lengths of the PF bonding method are 0.2 m, 0.3 m, and 0.4 m, respectively, and for the type 2 optical fiber, the gauge lengths for the point fixation bonding method are 0.1 m, 0.15 m, 0.2 m, and 0.3 m, respectively (Figure 12).

*4.2. Performance Assessment.* To assess the performance of BOTDA for crack monitoring, the measured DT data are

(a) Side view

(b) Layout of optic fiber (type 1)

(c) Layout of optical fiber (type 2)

Figure 12: Experiment specimens.

Figure 13: DT measurement results.

used as a baseline to check the accuracy of BOTDA. The measured DT results for different gauge lengths and different load levels are shown in Figure 13.

According to Figure 13, two DTs whose gauge lengths are equal to 10 cm give almost the same measured value, and the measured values of the DTs whose gauge lengths equal 20 cm are bigger than the DTs whose gauge lengths equal 10 cm. Thus, this type of deformation can be used to simulate an ideal concrete crack while taking the elastic deformation of concrete block into consideration. The strain on the surface of a concrete structure can be expressed by the following equation:

$$\varepsilon = \frac{\Delta}{l} = \frac{\Delta_{\mathrm{cr}} + \Delta_c}{l}, \tag{2}$$

where $\Delta_{\mathrm{cr}}$ is the simulated crack width, $\Delta_c$ is the deformation of concrete, and $l$ is the gauge length.

According to Figure 12, the total deformation over the gauge length of 0.1 m, 0.15 m, 0.2 m, 0.3 m, and 0.4 m can be calculated. Moreover, these deformations over different gauge lengths can be converted to the corresponding strain values and then used to check the test accuracy of BOTDA.

*4.2.1. Assessment of the PF Bonding Method.* The measured results of type 1 FOS are shown in Figure 14.

With reference to Figure 14, for type 1 optical fiber with a gauge length less than 0.4 m, the measured strain value of BOTDA is smaller than the converted strain of DT; the smaller the gauge length is, the smaller the ratio of the measured strain of BOTDA to the converted strain of the DT becomes. The conclusion for Section 3.2.2 is also applicable here. However, the PF bonding method can only provide the total deformation over the gauge length; the exact position of cracks and the number of cracks cannot be adequately identified.

The measured results for type 2 optical fiber are shown in Figure 15.

Based on Figure 15, if the sensing length of the type 2 optical fiber is longer than 0.2 cm and the measurement of BOTDA agrees with the DT very well, the CESL of the type 2 optical fiber should be longer than 10 cm and less than 20 cm. This conclusion is practically the same as that of Section 3.3.

*4.2.2. Assessment of the OB Method.* The measured results of the two types of FOSs applied with the OB bonding method are illustrated in Figure 16.

According to Figure 16, for a preset crack width of 0.6 cm (a value much less than the 10 cm spatial resolution), the overall bonding method can reflect the propagation of the crack. Because the sensing region of the optical fiber sensor is bonded completely to the surface of the specimen with epoxy resin, the strain distribution generated in the concrete is thus directly given by the bonded optical fiber. For any occurrence of and propagation of cracks in concrete structures, there must be a debonding between the epoxy resin and the cracked concrete surface, with localized strain occurring on the fiber close to both sides of the crack, as shown in Figure 17(a). In the case of perfect bonding, the fiber would never survive such a large localized strain. Except for the slippage between the optic fiber and the epoxy resin, the optical fiber located at the cracked position is strained and elongated. Moreover, this localized strain peak also causes internal slippage of the optical fiber, and the localized strain peak is distributed near the optical fiber segment. Consequently, the strain distribution of the sensing fiber should be similar to Figure 17(b). However, performing detailed strain measurements close to cracks is still an unresolved problem, and no stable relationship is

Figure 14: Measurement results for the optical fiber (type 1).

Figure 15: Measurement result for the optical fiber (type 2).

found between the measured strain from the overall bonding and the converted strain of the DT. Thus, the overall bonding method is still not feasible for monitoring the crack width quantitatively.

According to Figure 17, for a type 2 FOS, because of the good antislippage property, the measured result for the optic fiber installed with the overall bonding method is much smaller than that for type 1 FOS. Thus, for the overall bonding method, the slippage of the FOS is favorable to monitor cracking of the concrete structure. If slippage of the FOS can be completely prevented, the overall bonding method may lead to the rupture of optic fibers and hence lose its ability to reflect the existence of cracks.

## 5. Comparisons with BOTDR

In our previous investigation [6], the measurement behaviour of BOTDR was discussed and the performance

Figure 16: Measurement result for an optical fiber installed by the overall bonding method.

Figure 17: Strain redistribution in optical fibers.

of BOTDR for crack monitoring was evaluated with the same experiment. The PPP-BOTDA sensing technique has a significantly improved spatial resolution; therefore, it is necessary to compare these two distributed fiber optic sensing techniques.

*5.1. Monitoring Localized Deformation.* The nominal spatial resolutions of BOTDR and PPP-BOTDA are 1 m and 0.1 m, respectively. The improvement in the spatial resolution is good for detecting the localized deformation. For the BOTDA technique, an optical fiber installed with the PF method can be used to monitor cracking or localized deformation. For the BOTDR technique, the optic fiber should be installed with the loop installation method.

*5.2. Relation of Truly Measured Strain Distributed Range and Actual Strain Distributed Range (Sensing Length).* For the BOTDR technique, the length of the actual strain distribution can be calculated from the length of the measured strain distribution by using the following equation [6]:

$$l = L_t - L, \tag{3}$$

where $l$ is the length of the truly measured strain distribution, and $L$ is the spatial resolution, $L_t$: the length of the actual strain distribution.

The correctness of (3) was verified by our experiments which are similar as the experiments carried out in Section 3.1. The actual length of the fiber optic sensor installed in the point fixation method is 1.4 m. According to (3), the truly measured length of the optic fiber can be calculated: $l = L_t - 1 = 0.4$ m, which agrees with the experimental result (0.45 m), as shown in Figure 18.

For the BOTDA technique, based on the experimental result of Section 3.2, the length of the actual strain distribution also can be calculated from the length of the truly measured strain distribution by using (3); consequently, the truly measured length of optic fiber can be calculated: $l = L_t - 1 = 1.3$ m, which agrees with the experimental result, as shown in Figure 19.

## 6. Conclusions

(1) Based on the experimental results for the bare fiber, the PPP-BOTDA sensing technique has a 10 cm spatial resolution.

Figure 18: Distributed strain data along an optic fiber measured by BOTDR (sensing length = 1.4 m).

Figure 19: Distributed strain data along an optic fiber measured by BOTDA (sensing length = 1.4 m).

(2) Despite the 10 cm spatial resolution, if the sensing length of the FOS is less than the CESL because of slippage between the bare fiber and the coating materials of the optic fiber, the BOTDA measurement result will not be correct. Different types of FOSs have different CESLs, and the longer the CESL is, the worse the antislippage property becomes.

(3) Point fixation installation methods can give effective and correct measurement results for monitoring localized deformation when the gauge length of the optical fiber is longer than the CESL.

(4) The overall bonding method can reflect the existence of cracking or localized deformation, but it is still unfit for monitoring crack width quantitatively.

(5) A higher number of repeated measurement counts and smaller scanning steps can improve the measurement accuracy, but these settings can increase the measurement time significantly.

(6) An effective way to reduce the error level of BOTDA is to adopt the average values of several time measurements.

## Acknowledgment

The authors would like to thank there viewers for their detailed comments that have helped to improve the quality of the paper. This work is supported by the Major Programs of the Chinese Academy of Sciences during the 12th Five-Year Plan Period (no. 2011BAK02B03); the National Natural Science Foundation of China (no. 51208113); Natural Science Foundation of Hebei Province of China (no. E2013210); The Foundation for Development of Science and Technology of Fuzhou University (no. 2012-XQ-33).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 914638, 14 pages
doi:10.1155/2012/914638

*Research Article*

# Intelligent Monitoring of Multistory Buildings under Unknown Earthquake Excitation by a Wireless Sensor Network

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Received 7 July 2012; Revised 7 October 2012; Accepted 8 October 2012

Academic Editor: Ting-Hua YI

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It is necessary to investigate techniques for monitoring structures under unknown earthquakes. For this purpose, an algorithm is proposed in this paper for the identification of structures and excitation of multi-story shear buildings with limited measurements of structural responses. The equation of motion of a multi-story shear building under ground motion is established in the absolute co-ordinate system, while the multi-story building is decomposed into substructures. A novel two-step Kalman estimator approach, which is not available in the previous literature, is proposed for the identification substructures and unknown ground motion with less computational effort. Then, for the purpose of intelligent structural monitoring, a new wireless sensor network is developed in this paper. The designed wireless sensor network has a two-level cluster-tree architecture. Hardware designs of the sensor unit and the cluster head are presented; especially the cluster head contains a low power digital signal processor with strong computing capacity. Thus, the wireless sensor network has the unique feature offers distributed computing at group level. Finally, the proposed algorithm is embedded into the wireless sensor network for intelligent structural monitoring and an experimental test shows the technique is effective for intelligent monitoring of multi-story buildings under unknown earthquakes.

## 1. Introduction

In the past decades, many structural identification and structural damage detection algorithms have been proposed, for example, see [1, 2]. Since some structural external excitations such as earthquakes or wind forces cannot be accurately measured under actual operating conditions, it is necessary to investigate algorithms for structural identification and damage detection under unknown earthquake excitation. Also, in structural health monitoring, the knowledge of external excitation is very useful for the safety evolution of structures.

Identification of structural parameters and unknown external excitation has been investigated by some researchers, for example, Wang and Haldar [3] developed iterative least squares approaches to identify simultaneously the structural parameters and ground motion of an earthquake. Chen and Li [4] proposed a statistical average algorithm for simultaneous estimation of structural parameters and earthquake-induced ground motion. Yang et al. presented several methods including extended Kalman filter with unknown excitation inputs (EKF-UI) [5] and the sequential nonlinear least square estimation (SNLSE) [6] for the identification of structural parameters as well as the unmeasured excitation inputs. However, the aforementioned identification approaches are based on the equation of motion established in the relative coordinate system for earthquake-excited structures. Since the absolute structural responses other than the relative structural responses are often measured in practice, there is a limitation for these approaches to be implemented in practice if the ground motion is unknown. Thus, Zhao et al. [7, 8] proposed hybrid identification methods for multistory buildings with unknown ground motion, in which the equation of motion of an $n$-story shear building subject to unknown ground motion is established in the absolute coordinate system. However, their methods request the measurements of all structural responses including the displacement, velocity, and acceleration responses. In practice, it is highly desirable to deploy as few sensors as possible [9, 10]. So, it is essential

to explore efficient algorithms which utilize only a limited numbers of measured responses of structures subject to unknown (unmeasured) earthquake excitation.

Extended Kalman filter (EKF) has been studied and shown to be a useful tool for structural identification with limited measurements of structural response [11, 12], but EKF approach requires that excitation inputs are measured [5]. Also, in the extended state vector, unknown structural parameters are included as an augmented vector. Hence the size of the extended state vector and the corresponding state equation are quite large [13]. If the order of the EKF is $2n + m$ ($n$ = number of structural DOFs, $m$ = number of unknown structural parameters), the computational effort of the EKF is on the order of $(2n + m)^3$ [14]. An EKF with high order suffers from not only long computational time and may lead to divergent behavior for a large number of unknown parameters as identification is an inversed problem. It has been demonstrated that EKF is sensitive to initial estimates of the parameters and exhibit divergent behavior for large numbers of parameters to be estimated [15]. Also, such estimation requires large computation effort and storages, which can hardly be implemented by the microprocessors in the wireless sensor network. To remove the above drawbacks of the current EKF approaches, a two-step Kalman estimator approach, which is not available in the previous literature, is proposed in this paper. The numbers of unknown parameters to be estimated in each step are reduced. Then, unknown external excitations are estimated via the least squares estimation. So, the proposed algorithm identifies structural parameters and unknown excitation in a sequential manner, which simplifies the identification problem and reduces both the computational effort and storages compared with other approaches [5, 6].

For the identification of a large number of unknown parameters in large size structural systems, the computational efforts increase tremendously. Consequently, substructural identification approaches are used, in which a large size structure is decomposed into smaller size substructures with fewer unknown parameters [16–19]. The proposed identification algorithm is extended to large size multistory shear buildings by dividing buildings into small size substructures. In the absolute coordinate system, the unknown ground-motion input is applied on the 1st floor of the building as an unknown external force. For each substructure above the 1st floor, substructure is identified by the two-step Kalman estimator approach; for the substructure containing the 1st floor, Zhao et al. [7, 8] demonstrated that this substructural parameters and the ground motion cannot be uniquely identified. Herein, an algorithm is proposed in which substructural responses and parameters are identified by the proposed two-step Kalman estimator and the structural eigenvalue equation. Then, the unknown ground motion is estimated subsequently by least squares estimation and the Newmark-$\beta$ method.

Therefore, the proposed algorithm enables distributed identification of substructures in multistory shear buildings with less computational effort and storage compared with other existing algorithms. It is suitable for the purpose of intelligent structural monitoring as it can be implemented into the computation core of a wireless sensor network (WSN) which has limited computation storage and capacity. So far, some innovative WSNs have been developed in recent years for intelligent structural monitoring [20–26]. Besides the merit of avoiding the problem of the extensive lengths of wires compared with the traditional wire-based monitoring system, another important virtue of a WSN is that it offers distributed data processing and computation capacity, which is particularly useful for the implementation of algorithms for intelligent structural monitoring [20, 27–29]. Since wireless communication consumes a large amount of energy, approaches which require transmission of long-time history records to the central server should be avoided. One of the efficient strategies is to embed data processing and analysis algorithms in the wireless sensor's microprocessor and then just transmit small amount of useful results wirelessly. The embedded algorithms enable a WSN autonomously analyze data, which grants the network with "intelligent" characteristics. So far, some intelligent WSNs have been established [20, 21, 26–29], but most of data processing and analysis are designed at sensor level. Moreover, many WSNs require deploying a large number of sensors. Hierarchical clustering is considered as an efficient way to facilitate the operation of such networks and minimize the energy consumption [30–32]. In this paper, a hierarchical wireless sensor network is established [33, 34]. The distributed sensors are grouped into clusters, in which a cluster head is assigned to each cluster to control the communication in the allocated cluster. A cluster head not only serves as a router of the network but also possesses computational capabilities with the data collected from the sensor nodes in the cluster. Hardware designs of the sensor unit and the cluster head are presented, especially each cluster head has a low power digital signal processor (DSP) with strong computing capacity. This is the distinguishing unique feature of the proposed WSN different from other WSNs. Therefore, the proposed WSN provides parallel computation resources at group level, which is particularly useful for the implementation of substructure identification as the clusters in WSN match with substructures.

Finally, based on the advantages of the proposed distributed structural identification algorithm and the designed new WSN, intelligent monitoring of multistory buildings under unknown earthquake excitation can be conducted by embedding the proposed algorithm into the cluster heads of the designed WSN. To evaluate the performance of the proposed technique, experimental examination is demonstrated by shake table test of a scaled 8-story shear building under unknown ground motion.

## 2. Algorithm for the Identification of Building under Unknown Earthquake Excitation

Consider an $n$-story shear building subject to earthquake-induced ground motion as shown in Figure 1(a). It is well known that the equation of motion of the building with relative to the ground motion structure can be written as

$$\mathbf{M}\ddot{\mathbf{x}}(t) + \mathbf{C}\dot{\mathbf{x}}(t) + \mathbf{K}\mathbf{x}(t) = -\mathbf{M}\mathbf{I}\ddot{\mathbf{x}}_g(t), \tag{1}$$

Figure 1: An $n$-story shear building under earthquake excitation. (a) In the relative coordinate system. (b) In the absolute coordinate system.

where $\mathbf{x}(t)$, $\dot{\mathbf{x}}(t)$, and $\ddot{\mathbf{x}}(t)$, are vectors of displacements, velocity, and acceleration responses, respectively; $\ddot{\mathbf{x}}_g(t)$ is the unknown earthquake-induced ground motion, $\mathbf{M}$, $\mathbf{C}$, and $\mathbf{K}$ are the mass, damping, and stiffness matrices of the building, respectively, and $\mathbf{I}$ is an unit vector. Usually, the mass of a building can be estimated with accuracy based on its geometry and material information.

*2.1. Equation of Motion in the Absolute Coordinate System.* In practice, it is hard to accurately measure earthquake-induced ground motion under actual operating conditions, so $\ddot{\mathbf{x}}_g(t)$ in (1) is unknown. As structural dynamic responses are usually measured by deploying some accelerometers, only limited absolute acceleration responses of the building can be measured. Then, (1) can be rewritten in the absolute coordinate system as

$$\mathbf{M}\ddot{\mathbf{x}}_a(t) + \mathbf{C}\dot{\mathbf{x}}_a(t) + \mathbf{K}\mathbf{x}_a(t) = \mathbf{C}\mathbf{I}\dot{\mathbf{x}}_g(t) + \mathbf{K}\mathbf{I}\mathbf{x}_g(t), \tag{2}$$

where $\ddot{\mathbf{x}}_a(t)$, $\dot{\mathbf{x}}_a(t)$, and $\mathbf{x}_a(t)$ are the absolute responses of structural acceleration, velocity, and displacement, respectively; $\dot{\mathbf{x}}_g(t)$ and $\mathbf{x}_g(t)$ are the velocity and displacement of ground motion, respectively.

Without the loss of generality, damping matrix of the building can be assumed as

$$\mathbf{C} = a_1\mathbf{M} + a_2\mathbf{K}, \tag{3}$$

where $a_1$ and $a_2$ are two Rayleigh damping coefficients which depend on structural damping ratios and frequencies.

For a shear building, its stiffness matrix $\mathbf{K}$ is in the form as

$$\mathbf{K} = \begin{bmatrix} k_1+k_2 & -k_2 & 0 & \cdots & 0 & 0 & 0 \\ -k_2 & k_2+k_3 & -k_3 & \cdots & 0 & 0 & 0 \\ 0 & \cdots & \cdots & \cdots & 0 & 0 & 0 \\ 0 & \cdots & -k_i & k_i+k_{i+1} & -k_{i+1} & 0 & 0 \\ 0 & 0 & 0 & \cdots & \cdots & \cdots & 0 \\ 0 & 0 & 0 & 0 & -k_{n-1} & k_{n-1}+k_n & -k_n \\ 0 & 0 & 0 & 0 & 0 & -k_n & k_n \end{bmatrix}, \tag{4}$$

where $k_i$ $(i = 1, 2, \ldots, n)$ is the $i$th story stiffness of the building. Then, (2) can be written as

$$\begin{aligned} &\mathbf{M}\ddot{\mathbf{x}}_a(t) + (a_1\mathbf{M} + a_2\mathbf{K})\dot{\mathbf{x}}_a(t) + \mathbf{K}\mathbf{x}_a(t) \\ &\quad = a_1\mathbf{M}\mathbf{I}\dot{\mathbf{x}}_g(t) + \begin{Bmatrix} a_2k_1\dot{\mathbf{x}}_g(t) + k_1\mathbf{x}_g(t) \\ \mathbf{0}_{(n-1)\times 1} \end{Bmatrix}. \end{aligned} \tag{5}$$

As each value in the vector $a_1\mathbf{M}\mathbf{I}\dot{\mathbf{x}}_g(t)$ is much smaller than the term $a_2k_1\dot{\mathbf{x}}_g(t) + k_1\mathbf{x}_g(t)$, contribution from $a_1\mathbf{M}\mathbf{I}\dot{\mathbf{x}}_g(t)$ can be neglected. Thus, (5) becomes

$$\begin{aligned} &\mathbf{M}\ddot{\mathbf{x}}^a(t) + (a_1\mathbf{M} + a_2\mathbf{K})\dot{\mathbf{x}}^a(t) + \mathbf{K}\mathbf{x}^a(t) \\ &\quad = \begin{Bmatrix} a_2k_1\dot{\mathbf{x}}_g(t) + k_1\mathbf{x}_g(t) \\ \mathbf{0}_{(n-1)\times 1} \end{Bmatrix}, \end{aligned} \tag{6}$$

that is in the absolute coordinate system, the building is excited on the 1st floor by an external force due to ground motion as shown in Figure 1(b). Since the ground motion is not measured, the external force is unknown.

*2.2. Substructure Approach.* If the story number $n$ is not small, the number of unknown parameters is quiet large. Direct identification of the whole building requires a large computation effort and storages. Moreover, as an inverse problem, direct identification of a large number of unknown parameters can lead to the problems of ill-condition and computational divergent. Consequently, substructural identification approaches are used, in which a large size structure is decomposed into smaller size substructures with fewer

unknown parameters [16–19]. The multistory building is divided into $m$ substructure as shown in Figure 2(a).

*2.3. A Two-Step Kalman Estimator for the Identification of Substructure under Known Excitation.* For substructure $m$ (Sub. $m$) at the top of the building as shown in Figure 2(a), the equation of motion of the substructure with relative to the motion of its bottom floor ($r$th floor) can be written as

$$\begin{aligned}\mathbf{M}_m\ddot{\mathbf{x}}_m^*(t) + (a_1\mathbf{M}_m + a_2\mathbf{K}_m)\dot{\mathbf{x}}_m^*(t) + \mathbf{K}_m\mathbf{x}_m^*(t)\\ = -\mathbf{M}_m\mathbf{I}_m\ddot{\mathbf{x}}_{\mathrm{ar}}(t),\end{aligned} \tag{7}$$

in which $\ddot{\mathbf{x}}_m^*(t)$, $\dot{\mathbf{x}}_m^*(t)$, and $\mathbf{x}_m^*(t)$ are the relative acceleration, velocity, and displacement response vector of substructure $m$, respectively, $\mathbf{M}_m$ and $\mathbf{K}_m$ are the mass and stiffness matrices of the substructure, respectively, $\ddot{\mathbf{x}}_{\mathrm{ar}}(t)$ is the measured absolute acceleration response of the $r$th floor, and $\mathbf{I}_m$ is a $m$-dimension unit vector as shown in Figure 2(b).

In (7), $\ddot{\mathbf{x}}_{\mathrm{ar}}(t)$ is measured, so substructure $m$ is excited by a known excitation. Introducing the state vector $\mathbf{X}_m^* = [\mathbf{x}_m^* \quad \dot{\mathbf{x}}_m^*]^T$, one can transform (7) into a state equation, that is,

$$\begin{aligned}\dot{\mathbf{X}}_m^* &= \left\{\begin{matrix}\dot{\mathbf{x}}_m^* \\ -\mathbf{I}\ddot{\mathbf{x}}_{\mathrm{ar}}(t) - \mathbf{M}_m^{-1}\left[(a_1\mathbf{M}_m + a_2\mathbf{K}_m)\dot{\mathbf{x}}_m^*(t) + \mathbf{K}_m\mathbf{x}_m^*(t)\right]\end{matrix}\right\}\\ &= \mathbf{g}(\mathbf{X}_m^*, \boldsymbol{\theta}_m, \ddot{\mathbf{x}}_{\mathrm{ar}}),\end{aligned} \tag{8}$$

in which $\boldsymbol{\theta}_m$ denotes the stiffness and damping parametric vector of the substructure.

Some sensors are deployed on the substructure to measure the response signals. Usually acceleration signals are measured and the observation vector of the substructure can be expressed in the discretized form as

$$\begin{aligned}\mathbf{y}_m[k] &= \mathbf{D}_m\ddot{\mathbf{x}}_m^*[k] + \mathbf{v}_m[k]\\ &= \mathbf{h}_m(\mathbf{X}_m^*[k], \boldsymbol{\theta}_m[k], \ddot{\mathbf{x}}_{\mathrm{ar}}[k]) + \mathbf{v}_m[k],\end{aligned} \tag{9}$$

in which $\mathbf{y}_m[k]$ the is observation vector (measured relative acceleration responses) in the $m$th substructure at time $t = k \times \Delta t$ with $\Delta t$ being the sampling time step, $\mathbf{X}_m^*[k]$, $\boldsymbol{\theta}_m[k]$ and $\ddot{\mathbf{x}}_{\mathrm{ar}}[k]$ are the corresponding discretized values at time $t = k \times \Delta t$, $\mathbf{D}_m$ is the matrix associated with the locations of accelerometers in the concerned substructure, $\mathbf{v}_m$ is the measured noise vector, and

$$\begin{aligned}&\mathbf{h}_m(\mathbf{X}_m^*[k], \boldsymbol{\theta}_m[k], \ddot{\mathbf{x}}_{\mathrm{ar}}[k])\\ &\quad = \mathbf{D}_m\{-\mathbf{I}\ddot{\mathbf{x}}_{\mathrm{ar}}[k] - \mathbf{M}_m^{-1}[(a_1\mathbf{M}_m + a_2\mathbf{K}_m)\dot{\mathbf{x}}_m^*[k] + \mathbf{K}_m\mathbf{x}_m^*[k]]\}.\end{aligned} \tag{10}$$

Extended Kalman filter (EKF) has been shown to be useful for structural identification with limited response outputs [11, 12], but in the EKF approach, the extended state vector includes the augmented unknown structural parameters. Structural state vector and the parametric vector are estimated simultaneously [13, 14]. Such estimation requires large computation effort and storages, which can hardly be implemented even by the DSP of a cluster head in the designed wireless sensor network. To remove these drawbacks of the EKF approaches, a two-step Kalman estimator approach, which is not available in the previous literature, is proposed in this paper.

In the first step, substructural response state vector is considered as an implicit function of the structural parameters. Then the discretized observation equation can be rewritten as

$$\mathbf{y}_m[k] = \mathbf{h}_m(\mathbf{X}_m^*(\boldsymbol{\theta}_m[k]), \boldsymbol{\theta}_m[k], \ddot{\mathbf{x}}_{\mathrm{ar}}[k]) + \mathbf{v}_m[k]. \tag{11}$$

Assume $\hat{\boldsymbol{\theta}}_m[k \mid k-1]$ is the estimated value of $\boldsymbol{\theta}_m[k]$ at time $t = (k-1)\Delta t$, $\mathbf{h}_m(\mathbf{X}_m^*(\boldsymbol{\theta}_m[k]), \boldsymbol{\theta}_m[k], \ddot{\mathbf{x}}_{\mathrm{ar}}[k])$, which is a nonlinear function of unknown parametric vector $\boldsymbol{\theta}_m$, can be linearized around $\hat{\boldsymbol{\theta}}_m[k \mid k-1]$ through Taylor expansion, that is,

$$\begin{aligned}&\mathbf{h}_m(\mathbf{X}_m^*(\boldsymbol{\theta}_m[k]), \boldsymbol{\theta}_m[k], \ddot{\mathbf{x}}_{\mathrm{ar}}[k])\\ &\quad = \mathbf{h}_m\left(\mathbf{X}_m^*\left(\hat{\boldsymbol{\theta}}_m[k \mid k-1]\right), \hat{\boldsymbol{\theta}}_m[k \mid k-1], \ddot{\mathbf{x}}_{\mathrm{ar}}[k]\right)\\ &\qquad + \mathbf{H}_m[k]\left(\boldsymbol{\theta}_m[k] - \hat{\boldsymbol{\theta}}_m[k \mid k-1]\right),\end{aligned} \tag{12}$$

where $\mathbf{H}_m[k]$ is derived based on the chain rule of partial differentiation as

$$\mathbf{H}_m[k] = \mathbf{H}_{m,\boldsymbol{\theta}}[k] + \mathbf{H}_{m,\mathbf{X}}[k]\mathbf{X}_{m,\boldsymbol{\theta}}[k], \tag{13}$$

$$\begin{aligned}\mathbf{H}_{m,\boldsymbol{\theta}}[k] &= \left.\frac{\partial \mathbf{h}_m}{\partial \boldsymbol{\theta}_m}\right|_{\boldsymbol{\theta}_m = \hat{\boldsymbol{\theta}}_m[k|k-1]};\\ \mathbf{H}_{m,\mathbf{X}}[k] &= \left.\frac{\partial \mathbf{h}_m}{\partial \mathbf{X}_m}\right|_{\boldsymbol{\theta}_m = \hat{\boldsymbol{\theta}}_m[k|k-1]};\\ \mathbf{X}_{m,\theta}^*[k] &= \left.\frac{\partial \mathbf{X}_m}{\partial \boldsymbol{\theta}_m}\right|_{\boldsymbol{\theta}_m = \hat{\boldsymbol{\theta}}_m[k|k-1]}.\end{aligned} \tag{14}$$

Then, the recursive solution for the parametric vector can be estimated based on Kalman estimator, which has been widely used in the field of structural control, as

$$\begin{aligned}&\hat{\boldsymbol{\theta}}_m[k+1 \mid k]\\ &\quad = \hat{\boldsymbol{\theta}}_m[k \mid k-1] + \mathbf{K}_{m\boldsymbol{\theta}}[k]\\ &\qquad \times\left[\mathbf{y}_m[k] - \mathbf{h}\left(\mathbf{X}_m^*\left(\hat{\boldsymbol{\theta}}_m[k \mid k-1]\right), \hat{\boldsymbol{\theta}}_m[k \mid k-1], \ddot{\mathbf{x}}_{\mathrm{ar}}[k]\right)\right],\end{aligned} \tag{15}$$

in which $\mathbf{K}_{m\boldsymbol{\theta}}[k]$ is the Kalman gain matrix for $\boldsymbol{\theta}_m$ given by

$$\mathbf{K}_{m\boldsymbol{\theta}}[k] = \mathbf{P}_{m\boldsymbol{\theta}}[k]\mathbf{H}_m^T[k]\left(\mathbf{H}_m[k]\mathbf{P}_{m\boldsymbol{\theta}}[k]\mathbf{H}_m^T[k] + \mathbf{R}_{\boldsymbol{\theta}}[k]\right)^{-1}, \tag{16}$$

and $\mathbf{P}_{m\boldsymbol{\theta}}[k]$ is given by

$$\mathbf{P}_{m\theta}[k] = (\mathbf{I} - \mathbf{K}_{m\theta}[k]\mathbf{H}_m[k])\mathbf{P}_{m\theta}[k-1]. \tag{17}$$

Figure 2: Substructure approach of the multistory building. (a) Building with $m$ substructures. (b) Substructures with external excitation.

In the second step, recursive solution for the substructural state vector is estimated by Kalman estimator after the recursive solution for the substructural parametric vector $\boldsymbol{\theta}_m$

$$\begin{aligned}\hat{\mathbf{X}}_m^*[k+1\mid k] &= \tilde{\mathbf{X}}_m^*[k+1\mid k]+\mathbf{K}_{m\mathbf{X}}[k]\\ &\quad\times\left[\mathbf{y}_m[k]-\mathbf{h}_m\left(\hat{\mathbf{X}}_m^*[k\mid k-1],\hat{\boldsymbol{\theta}}_m[k\mid k-1],\ddot{\mathbf{x}}_{\mathrm{ar}}[k]\right)\right],\end{aligned} \tag{18}$$

where

$$\tilde{\mathbf{X}}_m^*[k+1\mid k]=\hat{\mathbf{X}}_m^*[k\mid k-1]+\int_{k\Delta t}^{(k+1)\Delta t}\mathbf{g}\left(\hat{\mathbf{X}}_m^*,\boldsymbol{\theta}_{\mathbf{m}},\ddot{\mathbf{x}}_{\mathrm{ar}}\right)dt. \tag{19}$$

$\mathbf{K}_{\mathbf{m}\mathbf{X}}[k]$ is the Kalman gain matrix for state vector $\mathbf{X}_m^*$.

For the estimation of the partial differentiation $\mathbf{X}_{m,\theta}^*[k]=\partial\mathbf{X}_m^*/\partial\boldsymbol{\theta}_m\big|_{\boldsymbol{\theta}_m=\hat{\boldsymbol{\theta}}_m[k|k-1]}$, the direct differentiation method (DDM) is utilized in this paper. Differentiating both sides of (8) with respect to $\boldsymbol{\theta}_m$, one can derive the equation for $\mathbf{X}_{m,\boldsymbol{\theta}}$ as

$$\dot{\mathbf{X}}_{m,\theta}^*=\frac{\partial\mathbf{g}\left(\mathbf{X}_m^*(\boldsymbol{\theta}_{\mathbf{m}}),\boldsymbol{\theta}_{\mathbf{m}},\ddot{\mathbf{x}}_{aj}\right)}{\partial\boldsymbol{\theta}_m}=\bar{\mathbf{g}}\left(\mathbf{X}_{m,\theta}^*,\mathbf{X}_m^*(\boldsymbol{\theta}_{\mathbf{m}}),\boldsymbol{\theta}_{\mathbf{m}}\right). \tag{20}$$

Then,

$$\begin{aligned}\mathbf{X}_{m,\theta}^*[k+1\mid k] &= \mathbf{X}_{m,\theta}^*[k\mid k-1]\\ &\quad+\int_{k\Delta t}^{(k+1)\Delta t}\bar{\mathbf{g}}\left(\mathbf{X}_{m,\theta}^*,\mathbf{X}_m^*(\boldsymbol{\theta}_{\mathbf{m}}),\hat{\boldsymbol{\theta}}_m\right)dt.\end{aligned} \tag{21}$$

Thus, the substructural relative responses and parameters in substructure $m$ can be identified by the proposed two-step Kalman estimator approach.

For the next substructure $m-1$ (Sub. $m-1$) of the building in Figure 2(a), the equation of motion of the substructure with relative to the motion of its bottom floor ($j$th floor) can be written as

$$\begin{aligned}&\mathbf{M}_{m-1}\ddot{\mathbf{x}}_{m-1}^*(t)+(a_1\mathbf{M}_{m-1}+a_2\mathbf{K}_{m-1})\dot{\mathbf{x}}_{m-1}^*(t)+\mathbf{K}_{m-1}\mathbf{x}_{m-1}^*(t)\\ &\quad=-\mathbf{M}_{m-1}\mathbf{I}\ddot{\mathbf{x}}_{aj}(t)+k_{j+1}\mathbf{x}_{m,1}^*(t)+a_2k_{j+1}\dot{\mathbf{x}}_{m,1}^*(t),\end{aligned} \tag{22}$$

in which, $\ddot{\mathbf{x}}_{m-1}^*(t)$, $\dot{\mathbf{x}}_{m-1}^*(t)$, and $\mathbf{x}_{m-1}^*(t)$ are the relative acceleration, velocity, and displacement response vector of substructure $m-1$, respectively, $\ddot{\mathbf{x}}_{aj}(t)$ is the measured absolute acceleration response of the $j$th floor in the substructure, $\mathbf{x}_{m,1}^*(t)$ and $\dot{\mathbf{x}}_{m,1}^*(t)$ are the relative displacement and velocity responses of the 1st floor in substructure $m$ in Figure 2(b), that is, the relative displacement and velocity

responses between the $(r+1)$th and $r$th floors in the whole building in Figure 2(a).

As $k_{j+1}$, $\mathbf{x}^*_{m,1}(t)$, and $\dot{\mathbf{x}}^*_{m,1}(t)$ have been indentified from the above identification of substructure $m$, substructure $m-1$ is also excited by a know excitation as shown in Figure 2(b). Therefore, the above two-step Kalman estimator approach can be used for the identification of the substructure state and parameters.

Analogously, for other substructures, above substructure 1 (Sub. 1) in Figure 2 can be identified sequentially by the proposed two-step Kalman estimator for the substructure under known excitation.

Thus, for the identification of substructure under known excitation, a two-step Kalman estimator is proposed in this paper. Recursive solutions for the substructural parameters and state vector are derived sequentially. Such solutions are not available in the previous literature. The numbers of unknown parameters to be estimated in each step are greatly reduced, which not only saves computational effort and storage but also avoids the difficulty of computational divergent in the inverse problem.

*2.4. Identification of Substructure under Unknown Excitation.* For substructure 1 (Sub. 1) of the building in Figure 2, the equation of motion can be extracted from (6) as

$$\begin{aligned}&\mathbf{M}_1\ddot{\mathbf{x}}^a_1(t)+(a_1\mathbf{M}_1+a_2\mathbf{K}_1)\dot{\mathbf{x}}^a_1(t)+\mathbf{K}_1\mathbf{x}^a_1(t)\\&\quad=k_{e+1}\mathbf{x}^*_{2,1}(t)+a_2k_{e+1}\dot{\mathbf{x}}^*_{2,1}(t)+\begin{Bmatrix}a_2k_1\dot{\mathbf{x}}_g(t)+k_1\mathbf{x}_g(t)\\ \mathbf{0}_{(n-1)\times 1}\end{Bmatrix},\end{aligned}\tag{23}$$

in which $\ddot{\mathbf{x}}^a_1(t)$, $\dot{\mathbf{x}}^a_1(t)$, and $\mathbf{x}^a_1(t)$, are the absolute acceleration, velocity, and displacement response vector of substructure 1, respectively, $\mathbf{x}^*_{2,1}(t)$ and $\dot{\mathbf{x}}^*_{2,1}(t)$ are the relative displacement and velocity responses of the 1st floor in substructure 2, that is, the relative displacement and velocity responses between the $(e+1)$th and $e$th floors in the whole building.

As $\dot{\mathbf{x}}_g(t)$ and $\mathbf{x}_g(t)$ are unknown, substructure 1 is excited by both known an excitation on the $e$th floor and an unknown excitation on its 1st floor. Zhao et al. [7, 8] demonstrated through a multistory shear building that the 1st story stiffness $k_1$ and the unknown ground motion cannot be uniquely identified when the absolute forced structural response time histories are used directly. Therefore, the term containing $k_1$ in (23) is shifted to the right side and (23) is rewritten as

$$\mathbf{M}_1\ddot{\mathbf{x}}^a_1(t)+\left(a_1\mathbf{M}_1+a_2\mathbf{K}'_1\right)\dot{\mathbf{x}}^a_1(t)+\mathbf{K}'_1\mathbf{x}^a_1(t)=\mathbf{B}_1\mathbf{f}_1+\mathbf{B}^u_1\mathbf{f}^u_1,\tag{24}$$

where $\mathbf{K}'_1$ is the stiffness matrix of the substructure 1 without $k_1$,

$$\mathbf{f}_1=k_{e+1}\mathbf{x}^*_{2,1}(t)+a_2k_{e+1}\dot{\mathbf{x}}^*_{2,1}(t),\tag{25a}$$

$$\mathbf{f}^u_1=a_2k_1\dot{\mathbf{x}}_g(t)+k_1\mathbf{x}_g(t)-a_2k_1\dot{\mathbf{x}}_{a1}(t)-k_1\mathbf{x}_{a1}(t),\tag{25b}$$

$\dot{\mathbf{x}}^a_{11}(t)$ and $\mathbf{x}^a_{11}(t)$ are the absolute velocity and displacement at the 1st floor level, respectively, and $\mathbf{B}_1$ and $\mathbf{B}^u_1$ are the location vector of the known and unknown external forces, respectively.

For the identification of the unknown external excitation on the 1st floor of the building due to unknown ground motion, as shown in Figure 2(b), it is necessary to deploy a sensor on the 1st floor to measure its absolute acceleration response. The discretized observation equation in substructure 1 can be written as

$$\mathbf{y}_1[k]=\mathbf{h}_1\left(\mathbf{X}^a_1(\boldsymbol{\theta}_1[k]),\boldsymbol{\theta}_1[k]\right)+\mathbf{G}_1\mathbf{f}_1+\mathbf{G}^u_1\mathbf{f}^u_1,\tag{26}$$

in which

$$\begin{aligned}&\mathbf{h}_1\left(\mathbf{X}^a_1(\boldsymbol{\theta}_1[k]),\boldsymbol{\theta}_1[k]\right)\\&\qquad=-\mathbf{D}_1\mathbf{M}_1^{-1}\{(a_1\mathbf{M}_1+a_2\mathbf{K}_1)\dot{\mathbf{x}}^a_1(t)+\mathbf{K}_1\mathbf{x}^a_1(t)\};\\&\mathbf{G}_1=\mathbf{D}_1\mathbf{M}_1^{-1}\mathbf{B}_1;\qquad \mathbf{G}^u_1=\mathbf{D}_1\mathbf{M}_1^{-1}\mathbf{B}^u_1,\end{aligned}\tag{27}$$

and $\mathbf{D}_1$ is the matrix associated with the locations of accelerometers in substructure 1.

Substructural state vector and parameters above the 1st floor can be identified by the proposed two-step Kalman estimator approach. Then, recursive solution for the unknown external excitation $\mathbf{f}^u_1$ can be obtained by least squares estimation approach as

$$\begin{aligned}&\hat{\mathbf{f}}^u_1[k+1\mid k+1]\\&\quad=\left[\mathbf{G}^u_1(\mathbf{G}^u_1)^T\right]^{-1}(\mathbf{G}^u_1)^T\\&\qquad\times\{\mathbf{y}_1[k+1]\\&\qquad\quad-\mathbf{h}_1\left(\hat{\mathbf{X}}^a_1\left(\hat{\boldsymbol{\theta}}_1[k+1\mid k]\right),\hat{\boldsymbol{\theta}}_1[k+1\mid k]\right)\\&\qquad\quad-\mathbf{G}_1\mathbf{f}_1[k+1]\}.\end{aligned}\tag{28}$$

In order to estimate $k_1$, frequency spectra of the measured acceleration response are analyzed to obtain the vibration frequencies of the building. With an estimated value of the frequency of the building and its frequency equation as follows:

$$\left|\mathbf{K}-\boldsymbol{\omega}^2_i\mathbf{M}\right|=\mathbf{0},\tag{29}$$

$k_1$ can be estimated based on the expansion of the determinate of matrix $\mathbf{A}=\mathbf{K}-\boldsymbol{\omega}^2_i\mathbf{M}$ and the formulation of the stiffness matrix $\mathbf{K}$ by (4), that is,

$$\det(\mathbf{A})=\left(k_1+k_2-\boldsymbol{\omega}^2_i m_1\right)\mathbf{A}_{11}-k_2\mathbf{A}_{12}=0,\tag{30}$$

where $\mathbf{A}_{11}$ and $\mathbf{A}_{12}$ are the two determinates of the complementary submatrices (cofactors) of the two element in the 1st row of matrix $\mathbf{A}$, respectively.

With the recursive estimation of unknown external excitation $\mathbf{f}^u_1(t)$ and the estimated value of $k_1$, the unknown ground motion can be estimated from (25b) utilization the Newmark-$\beta$ method for solving a first-order differentiation

Figure 3: A two-level cluster-tree wireless network topology.

equation [8]. The following recursive estimations can be derived by the Newmark-$\beta$ method:

$$\ddot{\mathbf{x}}_g[k+1] = \frac{1}{\beta\Delta t^2}\left(\mathbf{x}_g[k+1] - \mathbf{x}_g[k]\right) - \frac{1}{\beta\Delta t}\dot{\mathbf{x}}_g[k] - \left(\frac{1}{2\beta} - 1\right)\ddot{\mathbf{x}}_g[k], \tag{31a}$$

$$\dot{\mathbf{x}}_g[k+1] = \frac{\gamma}{\beta\Delta t}\left(\mathbf{x}_g[k+1] - \mathbf{x}_g[k]\right) + \left(1 - \frac{\gamma}{\beta}\right)\dot{\mathbf{x}}_g[k] + \left(1 - \frac{\gamma}{2\beta}\right)\ddot{\mathbf{x}}_g[k]\Delta t. \tag{31b}$$

Inserting (31b) into (25b), one can obtain

$$\begin{aligned}\left(\frac{a_2k_1\gamma}{\beta\Delta t} + k_1\right)\hat{\mathbf{x}}_g[k+1 \mid k+1] &= \hat{\mathbf{f}}_1^u[k+1 \mid k+1] + a_2k_1\hat{\mathbf{x}}_{11}^a[k+1 \mid k] + k_1\hat{\mathbf{x}}_{11}^a[k+1 \mid k] \\ &\quad + a_2k_1\left\{\frac{\gamma}{\beta\Delta t}\hat{\mathbf{x}}_g[k \mid k] - \left(1 - \frac{\gamma}{\beta}\right)\dot{\mathbf{x}}_g[k \mid k] - \left(1 - \frac{\gamma}{2\beta}\right)\ddot{\mathbf{x}}_g[k \mid k]\Delta t\right\}.\end{aligned} \tag{32}$$

Thus, the recursive estimated values of the unknown ground motion are obtained by (31a),(31b), and (32).

It is noted that the proposed identification algorithm for substructure under unknown excitation is based on the Kalman estimator approach instead of the conventional Kalman filter approach in other identification algorithms. In the sequential Kalman estimator for the state vector and least squares estimation of the unknown excitation inputs, state vector at time $t = (k+1) \times \Delta t$ is initially estimated given the observation signals $(\mathbf{y}[1], \mathbf{y}[2], \ldots, \mathbf{y}[k])$. Then, the unknown excitation at time $t = (k+1) \times \Delta t$ is estimated. On the other hand, in the Kalman filter approach, estimation of extended state vector at time $t = (k+1) \times \Delta t$ needs the observation signals $(\mathbf{y}[1], \mathbf{y}[2], \ldots, \mathbf{y}[k+1])$, which contain the unknown excitation at time $t = (k+1) \times \Delta t$. Superiority of the algorithm based on Kalman estimator is obvious over the conventional Kalman filter approach for the identification of structural parameter under unknown excitation, which is the advantage of the proposed algorithm.

In summary, the proposed algorithm can identify structural parameters and unknown excitation in a sequential manner, which simplifies the identification problem and reduces both the computational effort and storages compared with other existing work.

After the identification of structural parameters of the building, structural damage can be detected from the changes of structural parameters, especially from the degradation of the identified stiffness parameters.

## 3. The New Wireless Sensor Network

Recently, a new wireless sensor network (WSN) has been designed by the authors. Some lab and in field experiment tests on the accuracy of data acquisition, time synchronization of measurement data and other capabilities of the wireless sensor units validated that the designed wireless sensor network possesses favorable performances of data collection, transmission, and distributed computation [34].

*3.1. Topology of the New WSN.* Many sensor networks require deploying many sensors and the transmission of recorded data via long distance. Hierarchical clustering is generally considered as an efficient and scalable way to facilitate the management and operation of such networks and minimize the total energy consumption for prolonged lifetime [33, 34]. In this paper, a two-level cluster-tree network topology is proposed for the wireless sensor network as shown in Figure 3. A large-size structure can be divided into substructures. The distributed sensor units deployed in a substructure are grouped into a cluster. A cluster head is assigned to each cluster to coordinate the sensors in its cluster and to collect data from them during monitoring. Communication between the distributed sensor units with their corresponding cluster head forms the lower level and the network of cluster heads forms the upper level as shown in Figure 3 In wireless communication, the relationship between power consumption $E$ and communication distance $d$ is

$$E = kd^n, \tag{33}$$

(a) Hardware structure of the wireless sensor unit

(b) Prototype of the sensor unit

Figure 4: Wireless sensor unit in the sensor network.

in which $2 \leqslant n \leqslant 4$. This means with the increase of communication distance, power consumption has a sharp increase. A cluster head not only serves as a router of the network messages but also possesses computational capabilities with the data collected from the sensor nodes in the cluster. Sensors in a cluster transmit recorded data in a single hop communications to the cluster head. Then, only calculation results by the cluster head are transmitted via the upper level routing. This avoids the transmission of raw data from a sensor node over long distance, which is of great help to decrease power consumption of sensor units as shown by (33).

In the designed WSN, wireless communication is based on the Zigbee communication protocol built on the IEEE802.15.4 wireless communication standard [35]. IEEE802.15.4 standard is intended to provide the most energy efficient wireless communication protocol available. It also offers a standardized wireless interface for wireless sensor networks, thereby ensuring compatibility between wireless sensor platforms with different designs and functionalities. Thus, while an immediate benefit of IEEE 802.15.4 is its low power consumption, a potentially greater benefit is that it offers a common protocol for wireless sensor networks.

The proposed wireless senor network topology provides parallel computation between the substructures, which is useful for the implementation of computational algorithms for structural health monitoring. This is the unique feature of the proposed WSN different from other WSNs currently available. Finally, a center node combines the function of a cluster head with additional computational capabilities can be used for the final decision on structural health monitoring.

*3.2. Hardware Design.* Figure 4(a) shows the overall hardware design of the wireless sensor unit. The sensor unit mainly consists of six functional modules: (1) sensor interface; (2) signal conditioning; (3) sensor signal digitization; (4) computational core; (5) wireless communication; (6) battery management. To package the selected hardware components into a compact wireless sensor prototype, a two-layer printed circuit board was designed and fabricated. As shown in Figure 4(b), all electrical components are surface mounted to the printed circuit board. Most of the designs of the above six main modules are similar to those in other wireless sensors. Only the distinguishing features in

(a) Hardware structure of a wireless cluster head

(b) Prototype of a wireless cluster head

Figure 5: Wireless cluster head of the sensor network.

the hardware structure of the proposed wireless sensor are introduced herein.

Compared with other commercial wireless sensors, the Chipcon CC2430 is selected as the wireless transceiver in this design. The chip, named CC2430, released by Texas Instrument Corporation and is a true System-on-Chip (SoC) solution specifically tailored for 2.4 GHz IEEE 802.15.4 and ZigBee applications [36]. The CC2430 combines the excellent performance of the leading CC2420 RF transceiver with an industry-standard enhanced 8051 MCU, 32/64/128 KB flash memory, 8 KB RAM and many other powerful features. It has various operating modes and features of extra low consumption. When the microcontroller runs at 32 MHz, the CC2430 only consumes 25 mA for receiving and 27 mA for transmitting, while 0.3 $\mu$A current consumed in stand-by mode. Its short transition time between operating modes further ensure low power consumption. All these features make CC2430 highly suitable for the applications where ultra low power consumption is required.

In the computation core, a low-cost, low-power 8-bit Atmel AVR microcontroller (ATmega 128) is selected for the sensor unit. The microcontroller has 128 KB of ROM and 4 kB of SRAM is integrated with the microcontroller; however, this amount of SRAM is insufficient to store all the collected data. An additional 128 kB of SRAM (Cypress CY62128B) is interfaced with the microcontroller for the storage of sensor data. This microcontroller, together with certain internal and external memories, provides the capability of onboard data interrogation at the sensor level.

The hardware structure of a cluster head in the designed wireless sensor network is similar to that of a sensor unit except that its computational core is replaced by a TMS320C5409 digital signal processor (DSP) as shown in Figure 5(a). The attractive feature of TMS320VC5509A is the extra low power consumption compared with its strong computing ability. The TMS320VC5509A, a 16-bit fixed-point digital signal processor (DSP) produced by Texas Instrument Corporation, is based on the TMS320C55x DSP generation CPU processor core. The C55x CPU provides two multiply-accumulate (MAC) units, each capable of 17-bit $\times$ 17-bit multiplication in a single cycle. The clock rate of this chip is up to 200 MHz and its power consumption is only 0.5 mW/MIPS in full-speed operation while in lowest power mode is 0.05 mW/MIPS. The TMS320VC5509A also provides rich on-chip resources and peripheral set, such as 128 k* 16-Bit on-Chip RAM, 64 k Bytes ROM, Two 20-Bit

Figure 6: WSN for intelligent monitoring of a multistory building.

Timers, 3 Multichannel Buffered Serial Ports (McBSPs), and so forth. Thus, it provides the capability of interrogating the large amount of data collected from the sensing nodes in the cluster with low power consumption. This design wireless sensor network provides parallel computation resources at group level, which is a unique feature compared with other sensor networks. Figure 5(b) shows the prototypes of a cluster.

## 4. Intelligent SHM of Multistory Building

*4.1. The Proposed Technique.* The proposed algorithm can identify structural parameters, unknown earthquake excitation and structural damage based on substructure approach and a proposed a two step Kalman estimator. The algorithm enables distributed identification of substructures in multistory shear buildings with less computational effort and storage compared with other existing algorithms. The designed hierarchical sensor network has a two-level cluster-tree architecture, especially the cluster head consists of a low power DSP with strong computing capacity. The new WSN provides parallel computation resources at group level, which is a particularly useful feature for the implementation of computational algorithms for structural health monitoring. Based on the unique advantages of the proposed algorithm and the wireless sensor network, a technique for intelligent monitoring multistory buildings under unknown earthquake excitations can be implemented by embedding the proposed algorithm into the new wireless sensor network. The proposed algorithm for distributed structural identification is coded in C language and embedded into the cluster heads and sensor nodes in the wireless sensor network, which grants the wireless sensor network with "intelligent" characteristics.

As shown in Figure 6, a multistory shear building is divided into $m$ substructures in such a manner each substructural interface responses are measured by accelerometers. A corresponding WSN with $m$ clusters is established in which sensors (s) and cluster heads (ch) are used to record some acceleration responses of the building under unknown earthquake excitation. Intelligent identification starts from substructure $m$ at the top of the building. Sensors (s) in the substructure $m$ transmit reordered acceleration responses to its cluster head (ch) in which the proposed two-step Kalman estimator algorithm is embedded to identify the substructural conditions and send the calculation results of structural parameters and substructural interaction force to the following substructure $m-1$. Then, substructural state vector and parameters in substructure $m-1$ are identified by the embedded algorithm in the cluster head (ch) and the identification results are sent to the lower substructure cluster head (ch). Such identification progress in performed sequentially till the 2nd substructure/cluster head. In the 1st substructure, each sensor node is embedded with the peak-picking (PP) algorithm to estimate the natural frequencies of the building based on the frequency response functions (FRFs) of recorded data by executing the embedded fast Fourier transform (FFT) algorithm [28, 29]. Then, each sensor node transmits both the recorded acceleration time history and the estimated natural frequencies of the building to the cluster head, where the embedded algorithm for Identification of substructure under unknown excitation is executed to identify substructural parameters above the 1st floor, the unknown excitation on the 1st floor, the 1st story stiffness and the unknown ground motion based on the algorithm in Section 3.2. Finally identification results of structural parameters and unknown earthquake excitation are transmitted to the central PC, where a final decision on the monitoring of the building can be made, for example, structural damage is detected by tracking the degrading of the identified values of story stiffness parameters.

It is noted that in the proposed technique by the WSN, each sensor node transmits its recorded data to its cluster head nearby instead of direct to the central PC. Also, each cluster head just transmits its calculation results to its adjacent cluster head. Thus, the transmission of recorded data over long distance is avoided, which greatly reduced the power consumption in data transmission.

*4.2. Experimental Validation.* To assess the performance of the proposed technique for intelligent monitoring of multistory building under unknown earthquake excitation by the wireless sensing network, an eight-story shear type building in lab under shake table test is selected as an experimental example. As shown in Figure 7(a), the steel structure model behaves as a lumped mass shear building

(a) An 8-story shear building model

(b) Column-beam joint (upper), Two kinds of column (lower)

Figure 7: Experimental test of an 8-story shear building model.

(a) WSN for monitoring the structure model

(b) Sensor node (upper), central PC (lower)

Figure 8: Experimental test of intelligent monitoring.

because the building's floors are constructed by rigid beams. The connection of the column-beam is shown in Figure 7(b). The structure model is mounted on a small size shake table which induces 1940 N-S El-Centro earthquake ground motion to the structure, but this ground motion is not unmeasured. Six light PCB accelerometers are deployed on the 1st, 3rd, 4th, 5th 6th and 8th floors to measure structural absolute acceleration responses of at the corresponding floor levels, respectively. Structural damage is simulated by replacing the flexible columns with thinner ones as shown in Figure 7(b), which results in the reduction of corresponding story stiffness $k_i$ $(i = 1, 2, \ldots, 8)$. In the experiment,

Figure 9: Identification results of story stiffness of the undamaged building shown on PC.

Figure 10: Comparison of the estimated earthquake excitation with the real one.

structural damage is assumed to occur in the 5th story which leads to the reduction of $k_5$.

As shown in Figure 8, the building is divided into two substructures with floors 1–4 being the 1st substructure and floors 5–8 being the second one. The upper substructure is subject to the measured acceleration $\ddot{x}_{a4}(t)$ while the lower substructure is excited by both the interface force and the unknown excitation due to unknown ground motion. Topology of the wireless sensor network is also shown in Figure 8. In each substructure, the deployed sensor nodes are grouped into a cluster. Each cluster is assigned with a cluster head (ch) to collect data from the sensor nodes during vibration. Based on the proposed technique described in above section, the identification results of story stiffness parameters and the unknown earthquake excitation are sent to the central server (PC).

Figure 9 indicates the results of the identified story stiffness parameters of the undamaged building shown in the PC server while the identified earthquake excitation results (solid line) are compared with the actual ground motion (dotted line) in Figure 10. For the ease of replacement of columns, each column is connected to the rigid beam by two screws at each ends, as shown in Figure 7(b). Thus, the structure model is not an ideal shear-type building, so there is certain difference between the identified story stiffness values and those of the analytical shear building model. From the comparison in Figure 9, it is noted that the proposed technique can identify unknown earthquake excitation with good accuracy.

Figure 11: Identification results of story stiffness of the damaged building shown on PC.

Figure 11 indicates the identified story stiffness parameters of the damaged building shown in the central PC. By comparing the identification results of the story stiffness parameters shown in Figures 9 and 11, a final decision of structural damage detection can be made in the central PC. It is clearly shown that the proposed technique can autonomously detect structural damage based on the degrading of identified values of element stiffness parameters of $k_5$.

Although a small size 8-story structure model is used as an experimental test, the experiment results demonstrate the strategy and performance of the purposed technique.

## 5. Conclusions

In this paper, a technique is proposed for intelligent monitoring of multistory building under unknown earthquake excitation by a new wireless sensor network. Original contributions of the paper include: first, an algorithm is proposed for distributed structural identification detection with limited output measurements. It requires fewer structural output measurements compared with previous approaches. The algorithm is based on a proposed two step Kalman estimator for the identification of structural parameters and least squares estimation for unknown excitation. It cannot only identify structural parameters and unknown excitation in a sequential manner but also reduce both computational effort and storages compared with other existing work. Such straightforward derivation and analytical solutions are not available in the previous literature. Then, a new wireless sensor network with a two-level cluster-tree architecture is proposed. The distributed sensors are grouped into a cluster, in which a cluster head consists of a low power DSP with strong computing capacity. Thus, it provides parallel computation resources at group level, which is a particularly useful feature for the implementation of algorithm for intelligent structural health monitoring compared with other WSNs. Finally, intelligent monitoring of multistory building under unknown earthquake excitation is implemented by embedding the proposed algorithm into the new WSN. Such a technique is not available in the previous literature. A lab experimental test demonstrates that the proposed technique

is effective for intelligent identification of structural parameters, excitation and structural damage of multistory shear buildings with limited measurements of structural responses.

## Acknowledgments

This research has been partially supported by the National Natural Science Foundation of China (NSFC) through the Grant no. 51178406, the China National High Technology Research and Development Program 2007AA04Z420, and the Natural Science Foundation of Fujian Province through the Grant no. 2010J01309.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 597546, 9 pages
doi:10.1155/2012/597546

*Research Article*

# Field Implementation of Wireless Vibration Sensing System for Monitoring of Harbor Caisson Breakwaters

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Received 25 September 2012; Accepted 7 November 2012

Academic Editor: Ting-Hua Yi

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A wireless sensing system for structural health monitoring (SHM) of harbor caisson structures is presented. To achieve the objective, the following approaches were implemented. First, a wave-induced vibration sensing system was designed for global structural health monitoring. Second, global SHM methods which are suitable for damage monitoring of caisson structures were selected to alarm the occurrence of unwanted behaviors. Third, an SHM scheme was designed for the target structure by implementing the selected SHM methods. Operation logics of the SHM methods were programmed based on the concept of the wireless sensor network. Finally, the performance of the proposed system was globally evaluated for a field harbor caisson structure for which a series of tasks were experimentally performed by the wireless sensing system.

## 1. Introduction

According to a report of the Intergovernmental Panel on Climate Change [1], global warming is unambiguous. As sea surface temperature increases, water vapor in the lower troposphere also increases, and so does the energy of typhoons. The relationship between global warming and increased typhoon activity and intensity has been verified, reported by studies based on analyses of historical records and simulation results [2–4]. It was also reported that typhoons in the western North Pacific (WNP), including Korea, have become more intense as measured by their frequency in hurricane categories 4 and 5 [5] or the power dissipation index [3]. A strong typhoon usually brings strong winds and heavy rains, causes severe surge and floods, and then results in significant loss of life and property. Accordingly, the increase of typhoon intensity and frequency makes severer damages over the Pacific Ocean from the Philippine Islands to Taiwan, Japan, Korea, and the southwestern coast of China.

In general, coastlines, interfaces between ocean and land, are facing a severe state of typhoon than inland; thus, the safety of coastal structures becomes more significant under storm waves. The safety can be checked in a regular or irregular time scale, depending on the state of the target coastal structures and occurrence of extreme loading conditions such as typhoons. The safety is usually done by structural health monitoring (SHM) to identify existing damages in the target structures, gravity type or pile type, and finally to make an assessment of their healthy status. In the gravity-type coastal structures, the damages can be classified into settlement, overturning, or sliding. Local defects such as scouring and disturbance in foundation can also occurr and these defects tend to propagate into severer damages, that is, the global damages, such as partial settlement, under extreme wave forces. Thus, it is imperative to capture the local defects in time, at least just before these defects become a real threat.

In South Korea, most coastal or harbor structures have been constructed as gravity type and recently, thanks to the advances in concrete and towing and laying technology, very large caissons, a gravity type, became popular for constructing breakwaters, quays, and so forth. Considering the report [6, 7], significant damages of harbor caissons

are mostly attributed to foundation-structure interface that is, foundation mound. Goda and Takagi [8] and Takahashi et al. [9] also reported that sliding at the foundation-structure interface is the primary damage of caissons. It is known that cavity in foundation mound or backfill makes harbor caissons weaker against extreme loads such as storm waves, mainly occurred from typhoon. Thus, SHM for gravel mounds for existing caisson structures is absolutely demanded.

Since as early as the 1980s, many researchers had studied SHM techniques in the field of civil engineering [10, 11] by mainly performing vibration-based damage monitoring of civil structures [12–14]. Also, many researchers have worked on developing damage detection methods such as the modal sensitivity method, modal flexibility method, genetic algorithm, and neural network [15–20]. Related to the damage detection, sensor placement techniques have been studied and evaluated by applying to the tower structure [21, 22].

Up-to-date research studies, however, have focused mostly on inland structures but only few research efforts have been made to harbor structures, which include vibration response analyses of soil-structure or fluid-soil-structure interactions in harbor caisson structures [23, 24] and SHM of a cylinder caisson using fiber grating sensing technology [25]. Considering harbor caisson structures, there exist research needs to monitor vibration responses with limited access and to identify sensitive vibration methods of a row of caisson units with respect to damage in caissons foundation and/or damage in caisson's interlocking members.

The demand to test full-scale structures has led structural engineers to develop new methods to perform such experiments, that is, input-output (forced vibration) and output-only (ambient vibration) modal identification methods. Since it is difficult to excite large and massive civil structures in a controlled manner, the performance of output-only modal identification tests became an alternative in the field of civil engineering. Bridge structures have been the representative target to verify the efficiency of output-only modal identification [26–30]. Also, other structures have been tested through the identification [31–34]. These studies show that ambient tests have been successfully applied to a number of large bridges, towers, storage tanks, and buildings. Thus, in this study, we applied vibration-based SHM on massive caisson structures that cannot be excited by forced vibration. In other words, instead of using forced vibrations, we used water wave-induced vibrations for SHM, which utilized by ambient vibration modal identification methods.

Output-only modal identification methods can be classified into two groups: parametric methods in time domain and nonparametric methods essentially developed in the frequency domain. The frequency domain methods are initiated by the construction of power spectral densities (PSDs) involving all of the measurement points in the test program. In this study, we also used PSDs and, consequently, correlation coefficient (CC) and lower control limit (LCL) to alarm the damaged states. Namely, the occurrence of damage was indicated when the standard deviation of CC values is beyond the bound of LCL. In addition, the frequency domain decomposition (FDD) method was used to extract modal parameters such as natural frequency and mode shape. These modal parameters were used to determine a reference state and, accordingly, distinguish the healthy states of the target caissons.

Recent development in wireless smart sensor node (SSN) technologies has motivated various applications in the frequency domain-based SHM of massive engineering structures. So far, numerous field works have demonstrated that wireless smart sensors can be used to construct a reliable and accurate SHM system [35–40]. In addition to the relatively lower cost than wired SHM systems, one of the great advantages for using wireless sensors is that autonomous operations for SHM can be implemented by embedding advanced system technologies. In the study, we also proposed a vibration-based wireless sensing system comprising an accelerometer, wireless data logger, wireless receiver, and laptop. Moreover, an autonomous SHM scheme was developed utilizing CC and modal parameter-based method, which were operation logics of the proposed SHM.

In summary, we present a wireless sensing system for structural health monitoring of harbor caisson structures. To achieve the objective, the following approaches were implemented. First, a wave-induced vibration sensing system was designed for global structural health monitoring. Second, global SHM methods which are suitable for damage monitoring of caisson structures were selected to alarm the occurrence of unwanted behaviors. Third, an SHM scheme was designed for the target structure by implementing the selected SHM methods. Operation logics of the SHM methods were programmed based on the concept of the wireless sensor network. Finally, the performance of the proposed system was globally evaluated for a field harbor caisson structure for which a series of tasks were experimentally performed by the wireless sensing system.

## 2. Wireless Monitoring System for Harbor Caisson

*2.1. Vibration-Based Wireless Sensing System.* Figure 1 shows the proposed vibration-based wireless sensing system. One system is comprised of accelerometers, wireless data loggers, a wireless receiver, and a laptop, as shown in Figure 1. Rechargeable battery, which supplied electric power to the data logger itself and to the accelerometer through the BNC cable, was embedded in the data logger. Electric power for receiver was supplied by external portable battery. In the system, as shown in the figure, the $i$th accelerometers captured the signals and transported the captured to the $i$th data loggers by the BNC cables, then the transported signals were wirelessly transmitted to the receiver, and finally the signal arrived to the laptop via the LAN cable. To acquire the wave-induced, micro-vibration signals, we used PCB393B03 model for the accelerometers, which have the measurable range of $\pm 0.5$ g and sensitivity of 10 V/g. In the wireless communication, 2.4 GHz Bluetooth of transmitting frequency was used to range its communication to 1.2 km. The wireless data loggers had a programmable low pass filter

Figure 1: Schematic of wireless vibration sensing system.

Figure 2: An autonomous SHM procedure for PSD-based method.

Figure 3: An autonomous SHM procedure for modal parameter-based method.

Figure 4: Oh-Ryuk-do caisson-type breakwater.

from 10 Hz to 1 kHz and resolution of 16 bits. Also, the available sampling frequency ranged from 1 Hz to 1 kHz and the accuracy of synchronization was in 10 Ms.

The wireless sensing system utilized the vibration-based SHM methods PSD–based method and modal parameter-based method. Moreover, an autonomous operation was carried out in the wireless sensing system by the operation logics based on correlation coefficient of power spectral density and modal parameter-based method. The brief introduction to the SHM methods and autonomous operation are briefly explained in the following sections.

### 2.2. Vibration-Based SHM Methods

*2.2.1. Power Spectral Density-Based Method.* Assume there are two acceleration signals, $x(t)$ and $y(t)$, measured before and after a damaging episode, respectively. The corresponding power spectral densities (PSDs), $S_{xx}$ and $S_{yy}$, are calculated from Welch's procedure as [41]

$$S_{xx}(f) = \frac{1}{n_d T} \sum_{i=1}^{n_d} |X_i(f,T)|^2, \tag{1a}$$

$$S_{yy}(f) = \frac{1}{n_d T} \sum_{i=1}^{n_d} |Y_i(f,T)|^2, \tag{1b}$$

where $X$ and $Y$ are the frequency response transformed from correspondent acceleration signal; $n_d$ is the number of divided segments; $T$ is the data length of divided segment.

The correlation coefficient (CC) of PSDs represents the linear identity between the two PSDs obtained before and after a damage event, as follows:

$$\rho_{XY} = \frac{E\left[S_{xx}S_{yy}\right] - \mu_{S_{xx}}\mu_{S_{yy}}}{\sigma_{S_{xx}}\sigma_{S_{yy}}}, \tag{2}$$

where $E[\cdot]$ is the expectation operator and $\sigma_{S_{xx}}$ and $\sigma_{S_{yy}}$ are the standard deviations of PSDs of acceleration signals measured before and after damaging episode, respectively. If any damage occurs in target structure, its acceleration responses would be affected and, consequently, the decrement of CC can be a warning sign of the presence of damage.

For damage alarming, control chart analysis is also performed to discriminate damage events from the CC values. The alarming threshold is determined by the lower control limit (LCL) as follows:

$$\mathrm{LCL}_\rho = \mu_\rho - 3\sigma_\rho, \tag{3}$$

where $\mu_\rho$ and $\sigma_\rho$ are the mean and the standard deviation of the CC values, respectively. The occurrence of damage is indicated when the CC values are beyond (i.e., less than) the bound of the LCL; otherwise, there is no indication of damage occurrence.

*2.2.2. Modal Parameter-Based Method.* Frequency domain decomposition (FDD) method [42, 43] is used to extract modal parameters such as natural frequency and mode shape. The singular values of the PSD function matrix $\mathbf{S}(\omega)$ are used to estimate the natural frequencies instead of the PSD functions themselves as follows:

$$\mathbf{S}(\omega) = \mathbf{U}(\omega)^T \mathbf{\Sigma}(\omega)\mathbf{V}(\omega), \tag{4}$$

where $\mathbf{\Sigma}$ is the diagonal matrix consisting of the singular values ($\sigma_i$'s) and $\mathbf{U}$ and $\mathbf{V}$ are unitary matrices. Since $\mathbf{S}(\omega)$ is symmetric, $\mathbf{U}$ becomes equal to $\mathbf{V}$. In this FDD method, the natural frequencies can be determined from the peak frequencies of the singular value, and the mode shape from

Figure 5: Test layout of Oh-Ryuk-do breakwater.

Figure 6: Schematic of wireless vibration sensing system.

any of the column vectors of $\mathbf{U}(\omega)$ at the corresponding peak frequencies. Generally the first singular value $\sigma_1(\omega)$ among $\sigma_i$'s ($i = 1, \ldots, N$) is used to estimate the modal parameters except in some special cases such as with two or more identical excitations.

### 2.3. Autonomous SHM Scheme

*2.3.1. Power Spectral Density-Based Method.* An autonomous SHM procedure for PSD-based method is designed as shown in Figure 2. The detailed procedure is as follows: (1) transmit "start command" by remote control server to the Acc-SSN (acceleration-based smart sensor node); (2) acquire acceleration signals and store the measurement data in the Acc-SSN; (3) transmit the acceleration signal by the Acc-SSN into the remote control server; (4) compute power spectral density in the remote control server using the embedded FFT algorithm; (5) calculate correlation coefficient in the remote control server; (6) determine if damage is occurred by performing the control chart analysis.

*2.3.2. Modal Parameter-Based Method.* An autonomous SHM procedure for the modal parameter-based method is designed as shown in Figure 3. The detailed procedure is as follows: (1) transmit "start command" by remote control server to the Acc-SSN (acceleration-based smart sensor node); (2) acquire acceleration signals and store the measurement data in the Acc-SSN; (3) transmit the acceleration signal by the Acc-SSN into the remote control server; (4) compute power spectral density in the remote control server using the embedded FFT algorithm; (5) calculate natural frequency and mode shape using singular value decomposition (SVD) in the remote control server.

## 3. Field Evaluation

*3.1. Target Caisson Structure.* A real caisson-type breakwater locating in Busan, South Korea, Oh-Ryuk-do breakwater was selected to verify the applicability of the proposed vibration-based SHM technique, as shown in Figure 4. The breakwater consisted of 50 caissons and its total length was 1,004 m. Each caisson is covered by cap concrete to the height of 4 m. Parapet structure, which has 5.3 m height and 8.8 m width, is located above the caisson units except 1–3 and 48–50. The parapet structure increases the crest height of breakwater and gives higher calmness of harbor. All of the caissons were submerged in seawater and only their cap concretes were exposed in air. Three target structures, which have the same geometry with the parapet, were selected to compare the relative vibration responses by wireless vibration sensing system. Vibration tests were mainly performed on target caissons denoted by 18, 19, and 20 as shown in Figure 5. The size of each caisson is 20 m (width) × 20 m (length) × 16.78 m (depth).

(a) Grinding concrete surface (b) Installed sensors for three measuring points

Figure 7: Sensor installation on the caisson.

(a) Sensor 2-$y$ (b) Sensor 2-$z$

Figure 8: Acceleration signals obtained from sensor 2-$y$ and 2-$z$ of caisson 18.

*3.2. Wave-Induced Ambient Vibration Test.* In general, it is not easy to acquire forced vibration responses from heavy structures like Oh-Ryuk-do breakwater. To get the response, we may need a tugboat, which was not a possible option in the test. Instead, we measured ambient vibration signals without using any artificial impact load by utilizing the fact that the caisson structures were under incident waves and wind loads.

The measurement was made by the wireless vibration sensing system as schematically shown in Figure 1. As explained earlier, wireless vibration sensing system consisted of accelerometers, wireless data loggers, a wireless receiver, and a laptop. A system of 12 acceleration channels was employed to get the vibration responses of three target caisson units at the same time. Four channels were assigned to each caisson unit (18, 19, and 20) along the direction of the wave incident ($y$-direction) and upward vertical direction ($z$-direction), as shown in Figure 6. The sensors are denoted by 2-$y$, 3-$y$, 1-$z$, and 2-$z$, respectively. Here the first notation (1, 2, or 3) indicates the location of the sensors and the second notation ($y$ or $z$) denotes the direction of acceleration. The orientation of sensors on the three caisson units was set as same as each other. Before installing the sensors in the precast steel blocks, the surfaces of the top concrete caps were ground down as shown in Figure 7(a). Figure 7(b) shows the components of local node and installed local node for three measuring points. Representative received signals, obtained from 2-$y$ and 2-$z$ of 18, were shown in Figure 8. As shown, the acceleration of the sensor 2-$y$ has a bigger acceleration level than that of sensor 2-$z$ because the wave-induced, ambient vibration was of the same direction as the $y$-axis.

*3.3. Vibration-Based Damage Monitoring.* For the three target caissons, damage monitoring was performed by the proposed methods. In the first step, six sets of acceleration measurements were acquired from each caisson and PSDs corresponding to the sets were calculated. The PSDs corresponding to the first set of each caisson were shown in Figure 9, respectively. We can observe from Figure 9 that there are two peaks in the frequency range from 0 to 5 Hz

Figure 9: Power spectral densities of caissons 18–20.

Figure 10: Planar mode shapes (mode 1) of caissons 18–20 and their relative motion.

and the $y$-direction responses are clearer than the $z$-direction responses. This result comes from the incident wave motion along with the $y$-axis on the caisson response. The first peak is located at the fixed value of 1.51 Hz for all of the caissons while the second peak is located approximately at 2.5 Hz with the variances such as 2.69 Hz in 18, 2.49 Hz in 19, and 2.54 Hz in 20. The maximum difference of the second peak values is about 8%, which means structural abnormality is sensitive to the second peak. In the second step, planar mode shapes (mode 1) were extracted to examine the relative motions between interfaces, as shown in Figure 10. Frequency domain decomposition (FDD) algorithm [43] was used to extract the mode shapes corresponding to the first frequency 1.51 Hz. As shown, the relative motion of 19 is much larger than those of 18 and 20. Between 18 and 20, the relative motion of 18 is slightly smaller than that of 20; thus, the caisson 18 was selected as the reference and then the states of 19 and 20 were compared with that of the reference caisson. In the final step, CC values were calculated and, accordingly, LCL values were determined as

Figure 11: Correlation coefficient of PSD for caissons 18–20.

shown in Figure 11. This figure illustrates the relative gaps in the healthy states among the caissons. The correlation between 18 and 19 is bigger than that between 18 and 20, which means the healthy state of 19 is worse than that of 20. This observation becomes clearer in the results from 1-$z$ and 3-$y$ because the gaps between the caissons are extinct.

## 4. Conclusion

In this study, a wireless sensing system for monitoring of the caisson structures of Oh-Ryuk-do breakwater was presented. The following approaches were implemented to achieve the objective. Firstly, a wave-induced vibration sensing system was designed for global structural health monitoring. Secondly, global SHM methods, utilizing correlation coefficient and modal parameters, were selected to alarm the occurrence of unwanted behaviors. Thirdly, autonomous SHM scheme, implementing correlation coefficient and modal parameter-based method, was designed for the target structure. Operation logics of the SHM methods were programmed based on the concept of the wireless sensor network. Finally, the performance of the proposed system was globally evaluated the caisson structures for which a series of tasks were experimentally performed by the wireless sensing system.

From the field test, we found that the proposed wireless sensing system can make an assessment of the healthy states of the target caissons by implementing the value of correlation coefficient with respect to the bound of the lower control limit. For the implementation, we need to establish the reference state, which can be possibly identified by comparing the relative motions of the target caissons. It should be also noted here that the direction of accelerometer is quite significant to get a strong signal. In this study, we found that the wave-induced, ambient vibration gave a strong signal when accelerometers were along with wave direction. Moreover, it is found that the clear peaks were established in the power spectral densities obtained from the sensors located in the $y$-direction. This fact supports the correlation coefficients extracted from the sensor 3-$y$, which distinguish the healthy states of the three caissons. However, the correlation coefficients from the sensor 1-$z$ also gave a distinct assessment. It seems that the sensor location is also an important factor to get a promising test result, along with the sensor orientation. Thus, the distributed sensor system should be addressed with promising damage detection methods, schemes, and wireless networks.

This study shows a successful wireless monitoring procedure for the harbor caisson structures. Thus, we believe that the proposed wireless SHM method and process provide invaluable information for designers, construction crews, and field engineers when they design, construct, and inspect caissons of breakwaters and quays, which are widely used in connection with men and goods transportation from the mainland to an island, and from a nation to other nations.

## Acknowledgment

This work was supported by Basic Science Research Program through the National Research Foundation of Korea (NRF) funded by the Ministry of Education, Science and Technology (2011-0004253). The authors also would like to acknowledge the financial support of the project "Development of inspection equipment technology for harbor facilities" funded by Korea Ministry of Land, Transportation, and Maritime Affairs.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 743673, 9 pages
doi:10.1155/2012/743673

*Research Article*

# Design of a Wireless Multi-Radio-Frequency Channels Inspection System for Bridges

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Received 21 June 2012; Revised 10 September 2012; Accepted 24 October 2012

Academic Editor: Gangbing Song

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This paper presents a wireless multi-radio-frequency channels inspection system (WMCIS) based on wireless sensor networks (WSN) for the sake of bridge structural vibration inspection. The designed system consists of a wireless controller and eight wireless nodes with acceleration sensing units. The whole system has 64 collection channels, so every wireless node has eight 24-bit A/D channels. Integrated with acceleration sensing units, an embedded low-power microprocessing unit, a wireless transceiver unit, a large-capacity power unit, and a GPS time synchronization unit, the wireless node has the functions such as data collection, initial analysis, data storage, and wireless data transmission. The wireless controller, equipped with a high-performance computation unit, a wireless transceiver unit, a mobile power source, and an embedded data analysis software, can totally control multiple wireless nodes and receive and analyze data for parameter identification. To verify the performance of the WMCIS, experiments are made on the Caihong Bridge and Jinzhou Bridge in China for inspecting the bridge structural vibration. Experimental results show that the system has the advantages such as fast arrangement, high sampling rate, high resolution, capacity of low-frequency inspection, and GPS clock project for time synchronization and, thus, has good application prospects and practical value.

## 1. Introduction

With continuous development of the bridge-building technology, the bridge structure has become more and more complex and more attention has been paid to the quality and life of a modern bridge. After long-term use, the bridge structure will inevitably be damaged. It is necessary to inspect and monitor the bridge structure for the safety of a bridge. How to conduct a quality and safety test to the bridge structure has become a hot spot in domestic and international engineering field [1]. Bridge structural damage in the course of use can be found for many reasons, mainly, the improper design and maintenance, car accidents, man-made accidents, environmental factors, and so on. No matter ancient or modern, there are various examples of bridge collapse, causing great losses and huge disaster. Due to the design flaws which led to the actual carrying capacity of the bridge being much lower than the designed carrying capacity, the Quebec Bridge collapsed for the second time on September 11, 1916. Its first collapse was in 1907, resulting in the death of 95 people (Figure 1). At 7:00 on Nov. 7, 1940, Tacoma Narrows Bridge burst into sudden fluctuations and continued for more than three hours. Around 10:30 the bridge began to fracture into multiple pieces, at 11:00 the bridge collapsed. The main factor leading to the falling is the "stall flutter" phenomenon in aerodynamics which causes the sympathetic vibration of the structure. Although the crash of the bridge didnot result in heavy casualties, it caused enormous economic loss of the country.

In recent years, China also has several examples of the bridge collapse, for example, the collapse of the Caihong Bridge and the Gaoping Bridge (Figure 2). Caihong Bridge in Chong Qing collapsed on January 4, 1999. One of the reasons resulted in the sudden collapse of the bridge is the poor engineering quality. The compressive strength of the concrete in steel tube was below the design standard by one-third

Figure 1: Quebec Bridge collapsed for the 2nd time on September 11, 1916.

Figure 2: Caihong Bridge in Chong Qing collapsed on January 4, 1999.

because of serious flaws. On August 27, 2000, Gaoping Bridge which is the important channel to connect Kaohsiung and Pingtung in Taiwan province suddenly broke down, causing traffic disruption and 16 vehicles falling into the river and 22 people injured. There was no information about whether lesions occurred in the body of the bridge or whether the aging structure was damaged before the tragedy happened. Many examples show that the health of the bridge must attract more attention.

Because of the occurrence of many accidents in recent years, many countries have paid more attention to the safety inspection of bridge structure, like the Faroe Bridge in Denmark, Tampico Bridge in Mexico, and Skamsundet Bridge in Norway. A lot of tests have been done on the bridge to collect the displacement, strain, acceleration, and other physical parameters and environmental factors of the whole structure and components. In China, for example, the Hong Kong Tsing Ma Bridge, Guangzhou Humen Bridge, Shanghai Xupu Bridge, Jiangyin Yangtze River Bridge, Nanjing Yangtze River Bridge, and Hunan Aizai Bridge, relevant monitoring equipment were installed during the construction period to conduct real-time inspection to the structure in operation. This work is done to ensure safe operation and extend the service life of the bridge and also to save maintenance costs and improve the effectiveness of the integrated use of the bridge through early inspection and monitoring [2–5]. Since new bridges are needed in many places and the functions of the bridges become versatile, it is further requested that the inspection system has the characteristics of quick installing and convenient detecting for ensuring safety.

One important element for inspection system is the transmission of the measurement data from the sensors to the processing terminal; conventionally, and most popularly, wired networks are used for this task. However, with a great number of sensors for a large structure, a huge amount of wires is needed. As a result, installation time and costs can be very high, and this may also affect the reliability of the data transmission. Moreover, there may be cases in which wires cannot be placed in certain locations of a structure. Fortunately, with the development of technologies in sensing, wireless communication, and microelectromechanical systems, wireless sensor network (WSN) technique has been developed rapidly and is being used gradually in structural health monitoring for civil engineering structures in an attempt to install quickly and inspect conveniently and lower the high capital costs associated with wire-based structural monitoring systems [6–10]. There exists a limit of power consumption for WSN working in some circumstances where there are no conditions of supplying power continually, so energy optimization is a basic problem for wireless sensor network [11]. For a wireless sensor, energy is usua lly provided by solar power [12], wind energy, structural vibration [13], chemical batteries, or lithium batteries. But the above energy methods cannot meet the requirement of long-term monitoring, so the WSN may be more suitable for rapid testing in short term. Otherwise, time synchronization of WSN is very important for further data analysis.

Based on the above information a system is developed for quickly inspecting the bridge in real-time and wireless way, the proposed wireless node may connected with not only the traditional high-resolution sensing units but also the MEMS chips with low resolution aroused by its machining technique. The paper includes system overview, hardware development, software design and on-site verification tests.

## 2. System Architecture

The system architecture of WMCIS is shown in Figure 3. The designed system consists of one wireless controller and eight wireless nodes. The whole system has 64 collection channels, that is, every wireless node has eight 24-bit A/D channels. Every wireless node have an individual data sensing frequecy

Figure 3: System structure.

channel and a common control frequency channel at 2.4 GHz wireless frequency band, so the wireless controller has nine frequency channels. Data is transmitted from wireless node to wireless controller at every data sensing channel; information orders of the wireless controller are sent to every wireless node by the same and stable frequency band.

## 3. Design of Wireless Node

Studies show that it is possible to get safety assessment of the bridge based on the modal information by making global dynamic inspection. Structural global inspection technology of bridges can overcome the limitations of local inspection. The global inspection technology can verify design assumptions, monitor construction quality, and make real-time safety status assessment; especially, it can work without external auxiliary excitation based on dynamic inspection. So the global inspection technology has great advantages. In this design, acceleration is selected as the global parameters, the structure of wireless node is shown in Figure 4. The wireless node which is modular designed consists of acceleration sensing units, a sensor interface unit, a microprocessing unit, wireless units, a storage unit and a power management unit [14].

*3.1. Selection and Analysis of Sensing Unit.* In the designed system, force-balance sensor is used for the low-frequency vibration sensing unit because bridges usually belong to low-frequency vibration structures. Force-balance acceleration sensor is a closed-loop one which converts data to force or moment of force firstly, then adjusts the balance system with feedback force. Force-balance acceleration sensor with huge dynamic range and high measurement precision is used in low-frequency and low-g measurement and is the crucial unit in inertial navigation system. With the good performance

Figure 4: Structure of wireless node.

in ultralow frequency, force-balance acceleration sensor can meet the needs of low frequency vibration inspection. Also, the ICP sensing units, integrated with traditional piezoelectric acceleration sensor and charge amplifier, are considered to complete the acceleration measurement.

*3.2. Sensor Interface.* Wireless nodes support the acceleration sensing unit which needs ±12 V power supply, and outputs ±2.5 V or ±5 V analog voltage signal. In this design, multi-way switch chip MAX4051 is used to process 8 channels' selection firstly, and then 24-bit high accurate ADC chip ADS1248 is selected to convert from analog signal to digital signal.

*3.3. Microprocessor and Storage Unit.* TI high performance 16 bit MSP430F5438 is used for microprocessor. The series microprocessor has the lowest operating power consumption. The performance is up to 25 MIPS at 1.8 V–3.6 V operating voltage range and can meet the design needs of

wireless sensor in low power consumption and high rapid data processing. Storage unit uses NAND large capacity flash memory.

*3.4. Wireless Units.* Wireless units include wireless transceiver and GPS module.

Wireless transceiver unit for data wireless transmission consists of such Zigbee wireless chip as CC2520 RF chip and enlarge front CC2591. CC2520 RF chip is 2.4 GHz license-free ISM band Zigbee/IEEE 802.15.4 second generation RF transceiver of TI Company. CC2591 is 2.4 GHz RF front end unit for low power consumption and low voltage wireless application, can improve the transmission power and receiving sensitivity, and increases the wireless signal strength and transmission distance.

The GPS module is mainly designed to achieve the precise time synchronization for the wireless sensor nodes. Lassen IQ module uses the trimble's breakthrough First-GPS technology. It can output reliable information about position, velocity, time (PVT) to the user in low power. Lassen IQ module uses three common standard protocols, but its volume is only equivalent to the size of the stamps. The adopted protocol is TSIP, TAIP (trimble standard interface protocol), and the NMEA0183. The module is placed inside a metal enclosure which plays the role of protection and shielding.

*3.5. Power Management Unit.* In the system, 24 V battery is selected as wireless node's power source for rapid measurement and continued power supply. Because each unit in the system needs different power supply, such as ±15 V ±12 V, +3.3 V, and analog circuits require a higher voltage ripple, so energy design has two-stage transformation structure. The first stage uses DC/DC chips for the transformation from +24 V to ±15 V and +3.3 V. The second stage uses LDO chips 7812 and 7912 for the transformation from ±15 V to ±12 V.

The wireless node, shown in Figure 5, is integrated using the above units.

(a) Wireless node package

(b) Wireless node internal circuit

Figure 5: Wireless node.

Figure 6: Hardware structure of wireless controller.

## 4. Integration of Wireless Controller

The main functions of wireless controller designed by module include the management of the wireless nodes, data collection, data storage, and data analysis. The wireless controller, shown in Figure 6, comprises computer units (PC), wireless modular (data channel, control channel), and serial server based on PCI bus. A detailed description about these modules is presented as follows.

*4.1. Computer Unit.* Computer unit is the center of wireless controller. In order to control the wireless nodes and analyze and process the measurement data, the intelligent controlling software is transplanted into computer units.

*4.2. Serial Server.* The serial server is a media used to connect computers with wireless modules. It is connected to a wireless module via serial port and connects to the PC through the PCI bus. The role of the serial server is to transmit the data received by the serial port to the PC through the PCI. The serial server used in this system is MOXA-C218 with cache settings function. And its serial cache can be set to overcome the problem of insufficient cache according to actual needs.

*4.3. Wireless Modules.* The wireless units are divided into eight receiving modules and a transmitting one, namely eight data channels and one control channel. The transmitting module (control channel) controlled by the embedded software on the PC is used to send commands to the WSNs nodes which can decide whether to sample data. The eight receiving modules (data channel) are used to receive the sampled data sent by the corresponding WSNs node

Figure 7: The package of wireless modules.

respectively and to transmit it to the PC through the serial server.

The wireless module's band should be matched with the wireless sensors' band to realize two functions.

(1) The embedded software of computer units sends the control orders to wireless vibration sensor nodes in order to control their behaviors.

(2) Receiving the vibration data collected by wireless vibration sensor nodes and sends them to computer unit through serial server.

The integrated wireless terminal controller is shown in Figure 7.

## 5. Software Design

The software of wireless acquisition system consists of two parts: embedded program of wireless node and the wireless controller software.

*5.1. Embedded Program of Wireless Node.* The embedded program, integrated with ADC driver, storage driver, wireless driver, preliminary data analysis and diagnosis, and so on, can control circuits and receive command from the host for parameters setting, data acquisition, processing, storage, and transmission. Every wireless node's workflow is as follows: the wireless node, powered up in the wireless receiving state, does not work until receiving the command from host. Among acquisition, host firstly finishes the parameters setting to all wireless nodes, then sends start or stop command to control data acquisition. The wireless nodes can collect the vibration data of the offshore platform for inspection.

*5.2. Host Acquisition Software of Wireless Controller.* The host acquisition software mainly completes the parameter setting of the wireless node, data collection, data export, data storage, and data analysis and process. The software consists of the basic setting, real-time waveform monitoring, history waveform playback, data export, help document, and other modules. The acquisition software's modules are shown in Figure 8.

Table 1: Wireless nodes and sensor type.

| Wireless node 1 | Channel 1 | Channel 2 | Channel 3 |
|---|---|---|---|
| Sensor type | ICP | Servo | ICP |
| Wireless node 2 | Channel 1 | Channel 2 | Channel 3 |
| Sensor type | ICP | ICP | ICP |

Basic setting is used to set the state of wireless nodes, wireless transmission channels, sampling frequency, file storage path, and other parameters on demands. Real-time waveform monitoring can display the collected data's real-time waveform and write the data in database. History waveform playback may show a playback of history waveform based on the data collected. Data export can export the data collected from a specific wireless node at certain channel and time to a certain file for further analysis and process. Help document provides some help information for the users who use this acquisition software.

## 6. Experiments

*6.1. Natural Vibration Measurement on Caihong Bridge Using WMCIS.* Caihong Bridge is a suspension bridge with the length of about 25 meters and located in the West Campus of Dalian University of Technology. In order to detect the vibration of the structure of the Caihong Bridge, the acceleration sensing units were placed on each suspension on one side of the bridge. Three acceleration sensing units share a wireless node to communicate and exchange data with wireless controller. While the bridge is closed to traffic, a rapid detection of the natural vibration of the bridge is carried out.

In the detection system, two wireless nodes, a wireless controller and six acceleration sensing units are used. And the acceleration sensing units consist of a force balance sensing unit (servo) and five ICP sensing units. The sensing units configuration project as shown in Table 1. The system is laid close to the bottom location of the suspension, and the layout scheme was shown in Figure 9.

The sensing units are used to measure the acceleration of the vertical direction of the bridge, then the sampled data of the wireless nodes are sent to the wireless controller via wireless transmission for the user to observe and analyze (see Figure 10). In addition, a test about the wireless communication distance of the system is conducted in the west campus. In the case of no obstacles the communication distance of wireless nodes can reach 400 meters.

The typical vibration frequency spectrum is shown in Figure 11, by the analysis of the sampled data, the natural frequency of the bridge is about 3.964 HZ–3.986 HZ. The test result is very close to the theoretical values (4 HZ). All these show that the *WMCIS* can measure well the natural vibration of the bridge.

*6.2. Vibration Test on Jinzhou Bridge Using WMCIS.* Large bridge structure is an important part of the transport system which plays an important role in maintaining the sustained and stable economic development. Due to environmental

Figure 8: Host acquisition software's modules.

Figure 9: Experimental layout program.

Figure 10: A wireless node in field experiment layout.

degradation, materials aging, and long-term effects of load and other factors together, the structural capability in resisting to natural disaster, or even normal environmental effects will inevitably decline. Therefore, the bridge structural health monitoring has practical significance and potential applications.

The monitoring object is the West Bridge (shown in Figure 12) in Jinzhou City, Liaoning province. The bridge was built in 1939 and completed in 1941. With the economic development and traffic volume demand, it was expanded in 2003. The total length is 1033.5 m, of which the main bridge is 501.9 meters long, 14 meters wide, and 7026.6 square meters area.

Cross-bridge vibration is possibly characterized by low frequency and large signal dynamic range. So the requirement to the sensing unit for inspecting the bridge's vibration is relatively high. The wireless monitoring test selects a servo (force balance) acceleration sensing unit, five piezoelectric (ICP type) acceleration sensing units. The sensing units' models and sensitivities used by the program are shown in Table 2.

(a) The frequency spectrum of channel 2, node 1

(b) The frequency spectrum of channel 3, node 2

Figure 11: Typical frequency analysis.

Figure 12: The West Bridge of Jinzhou.

Table 2: Type and sensitivity of sensing units.

| Sensor | ICP | ICP | ICP | ICP | Servo |
|---|---|---|---|---|---|
| Model | sno776 | snr168 | sno775 | snr165 | 2188 |
| Sensitivity (mv/g) | 2003 | 2017 | 2010 | 2016 | 2635 |

Table 3: Vibration frequency.

| | Cross-section 1 | Cross-section 20 | Cross-section 22 |
|---|---|---|---|
| Natural frequency (Hz) | 5.865 | 5.848 | 5.895 |
| Incentive frequency (Hz) | 2.871 | 2.878 | 2.869 |

Table 4: WMCIS' performances.

| | |
|---|---|
| Wireless frequency band | 2.4 GHz |
| Wireless distance | 400 m |
| Sensitivity | 0.06 mg |
| Power (DC) | 12–24 V |
| Wireless transmission | 100 Kbps |
| Sample channels | 64 |
| Measurement range | ±2 g |
| Frequency response | DC-120 Hz |
| Working | −20°C–+70°C |
| A/D bits | 24 bits |

According to the requirements of the on-site test, the cross-sections 1, 20, and 22 are chosen as a sample, respectively, to carry out experiments on the inspection of bridge vibration.The layout scheme of the 1st cross-section is shown in Figure 13.

The bridge span length is about 21 m, of which 5 sensing units were placed at equal interval location. And the sensing units layout of the other cross-section is the same as one of the first cross-section.

The test was carried out in the midnight on October 4, 2011 (see Figure 14). The inspection experiments included the natural vibration test and driving incentive vibration test. Typical experimental results are shown in Figures 15 and 16, and the corresponding vibration frequency of the bridge is shown in Table 3.

The above analysis and comparison show that (1) the bridge is a low-frequency vibration structure; (2) the bridge has a large dynamic range of vibration signal. While vehicles and other heavy objects are moving on the bridge, the vibration range becomes significantly larger.

## 7. Conclusions

In this paper, based on the current energy methods of WSN not suitable for long-term monitoring, the *WMCIS* is proposed for rapid testing in short term. The *WMCIS* including wireless nodes, wireless controller and responding

Figure 13: Experimental layout program.

Figure 14: On-site layout program.

(a) Time history (b) Frequency

Figure 15: Experimental results of natural vibration of cross-section 20.

(a) Time history (b) Frequency

Figure 16: Experimental results of driving incentive vibration of cross-section 20.

software is designed for fast inspection of bridge structure's global characteristics. The *WMCIS*' performances are shown in Table 4. The experimental results show that the designed *WMCIS* with high reliability, no wiring, low-frequency vibration inspection, and GPS clock project for time synchronization is suitable for vibration inspection of bridges and has broad application prospects.

The design is preliminary and more tests need to be carried out before it can be of practical use.

## Acknowledgments

Financial support for this study provided by Grants from National Natural Science Foundation of China (Project no. 51108060, 50921001, 90815022), The natural science foundation of Liaoning Province (201202039), National Key Technology Research and Development Program during the Twelfth Five-Year Plan Period (Project no. 2011BAK02B01), the National Hi-tech Research and Development 863 program of China (Project no. 2008AA092701), and the Fundamental Research Funds for the Central Universities (Project no. DUT12JR13) are gratefully acknowledged.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 560161, 17 pages
doi:10.1155/2012/560161

*Research Article*

# Feasibility of Output-Only Modal Identification Using Wireless Sensor Network: A Quantitative Field Experimental Study

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Received 2 August 2012; Accepted 27 September 2012

Academic Editor: Ting-Hua Yi

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Although significant advancements have been achieved in wireless sensor network (WSN) in the past decades, there is still a strong need for in-depth analysis on wireless time synchronization and data quality during field testing. In this paper, an academic WSN is adopted for output-only modal identification of two full-scale bridges in Xiamen, China. This paper first assesses the accuracy of time synchronization performed by the WSN through a centralized beacon signal and then studies wireless data quality using two quantitative performance indexes. The relationship between vibration amplitude and quality of wireless sensor data is investigated. With ambient bridge accelerations acquired by wireless sensing units, modal properties of the two bridges are identified using the stochastic subspace identification (SSI) method and the enhanced frequency domain decomposition (EFDD) method. Accuracies of the modal identification results are evaluated through comparison with modal properties identified from tethered sensing data. The comparison shows close match in natural frequencies and mode shapes, although relatively large difference in damping ratios exists between results from wireless and tethered sensing data. An overall close match between the wireless and tethered results demonstrates the feasibility of using WSN for the ambient vibration testing of full-scale bridges.

## 1. Introduction

Structural monitoring or testing is an important precaution for the safety of civil structures. Traditional tethered data acquisition systems (DAS) require extensive lengths of cables to transfer recorded data from sensors to a centralized data repository. Installation and maintenance of such tethered DAS on large scale civil structures is time consuming and expensive. In addition, tethered DAS would need cables spread all over the structure, which can disturb everyday functionality of the structure. Using wireless sensor network (WSN) to overcome the shortcoming of tethered DAS is not a recent idea. Early in 1998, Straser and Kiremidjian [1, 2] proposed to use WSN for monitoring civil structures. Since then, numerous innovative academic and commercial wireless sensing systems have been developed [3–11]. WSN is regarded as a promising technology for structural monitoring or testing, due to its low cost, easy installation, mobility, and embedded computational capability [12]. This emerging technology overcomes many drawbacks arising from traditional tethered DAS, which includes high installation cost, disturbance to everyday functionality of the structure, high maintenance cost, and very limited number of sensor nodes. As a result, WSN can bring significant advantages to structural health monitoring (SHM). Densely deployed WSN has been exploited to achieve high spatial resolution for accurately characterizing structural dynamic behaviors [13–15]. In this way WSN can provide more accurate structural damage localization than traditional tethered sensor network that is usually sparsely deployed. High-density WSN, to some extent, relieves the fundamental challenge caused by the sharp contrast between smaller number of sensor nodes and large number of degree-of-freedoms that is typical in finite-element models used for SHM. Moreover, the microprocessor in a wireless sensing unit can perform embedded computing to achieve on-board and distributed

system identification and damage detection. Hereby wireless sensor has evolved into "smart" sensor [3, 6, 12].

Despite significant advancements being achieved in WSN over the past decade, more validation studies on the performances of WSN are essential before they can serve as economical and trustworthy substitutes to traditional tethered DAS. Field validation is necessary to accurately assess the performance of WSN within the complex and harsh environments posed by large-scale civil structures [6, 16]. Especially, although field experimental studies on WSN performance in civil structural applications have been reported [3, 7, 8, 14–18], quantitative research about data quality is still lacking. In addition, time synchronization is another major concern for applying WSN on structural vibration testing. Asynchronous wireless sensing data usually results in errors in mode shape identification [19]. Lynch et al. [15] investigated the time synchronization problem of a single-hop WSN deployed at Geumdang Bridge located in Icheon, Korea. To assess the time synchronization accuracy of WSN, the researchers proposed a classical method that performs time step shifting to achieve the minimum error norm between wireless and tethered sensor data. Despite this initial study, time synchronization of wireless sensor data can be evaluated through more extensive vibration tests using new analysis methods.

Vibration modal parameters are important dynamic properties of structures, which are determined by structural properties (such as mass and stiffness) and may reflect potential damage [20]. To perform vibration testing on a large-scale civil structure in the field, it is generally difficult to excite structural vibrations in a systematic and effective manner. Therefore, output-only modal identification, which only requires structural vibration response measurements, is regarded as a practical way to determine the dynamic characteristics of civil structures. Benefiting from low cost and easy installation, WSN has been adopted for dynamic testing that aims to identify structural modal parameters [8, 14, 15]. However, the accuracy of modal identification results associated with wireless data quality should be carefully investigated.

This paper presents field experimental studies using a dense WSN for the output-only modal identification of two bridges and evaluates the effect of wireless sensor data acquisition on modal identification accuracy. Time synchronization error and data quality of WSN will be assessed using quantitative indexes, through comparison with tethered sensing data. Two bridges, Baicheng Footbridge and Wuyuan Bridge in Xiamen, China, are selected as a test-bed for short-span and long-span bridges, respectively. The academic prototype wireless sensing unit developed by Wang et al. [16] is employed in the field experiments. Since coaxial cables provide a relatively reliable communication link, a tethered DAS is also installed on the bridge for comparison. Time histories of acceleration responses collected by the WSN are compared with those obtained by the tethered system. Based on the synchronized tethered sensor data, the accuracy of a wireless beacon-based time synchronization approach is evaluated using normalized correlation function. Furthermore, the errors of wireless sensing data are quantitatively assessed with regard to signal waveform and magnitude, through two data quality indexes. Using acceleration data measured by the WSN, the natural frequencies, damping ratios, and mode shapes of the two bridges are identified using the stochastic subspace identification (SSI) and enhanced frequency domain decomposition (EFDD) algorithms. The modal parameters identified using wireless sensing data are compared with these identified from tethered sensing data. A short discussion is provided regarding the effect of wireless sensor data quality on modal identification accuracy.

## 2. Wireless Sensing Units and Wireless Sensing Network

Figure 1 shows the academic prototype wireless sensing unit developed at University of Michigan and Stanford University [15, 16]. The wireless sensing unit consists of three functional modules: sensor signal digitizer, computational core, and wireless communication module. The main component of the sensor signal digitization module is a 4-channel 16-bit analog-to-digital (A/D) converter (Texas Instruments ADS8341), which offers a maximum sampling frequency up to 100 kHz. Each wireless sensing unit can accommodate signals from a variety of structural sensors, as long as their outputs are analog voltages from 0 to 5 V. The wireless sensing unit can remotely measure various physical parameters, such as acceleration, velocity, displacement, stain, and temperature. In addition, the 16-bit resolution A/D converter is sufficient for most applications in civil structural monitoring. However, it is worth noting that the resolution of A/D converter would only determine data quality if the smallest measurable voltage is larger than a product between the sensor noise floor and amplification factor of associated signal conditioning circuit.

The computational core of the wireless unit is responsible for executing embedded software instructions for engineering analyses. A low-cost 8-bit microcontroller (Atmel ATmega128) is selected as the principal component of the computational core. The wireless sensing unit is designed to be operable with two wireless transceivers: 900 MHz MaxStream 9XCite and 2.4 GHz MaxStream 24XStream. Although the 9XCite module consumes significantly less amount of power, the 24XStream transceiver is selected as the wireless communication module in this study according to public radio frequency regulation in China. The 24XStream transceiver can offer up to 5 km line-of-sight communication range [21]. During transmitting, the transceiver consumes a maximum of 150 mA current and emits a 50 mW electromagnetic signal. During receiving, the current consumption is 80 mA. Consequently, adding the current consumption of other components in the wireless sensing unit, the overall current consumption of the unit is about 180 mA while transmitting, and 110 mA while receiving.

As shown in Figure 1, before feeding the sensing signals into a wireless sensing unit, a sensor signal conditioning board can be adopted. Three major functions of the circuit board are offsetting, filtering, and amplification. Figure 2(a)

Figure 1: Functional modules of the prototype wireless sensing unit.

(a) Signal conditioning circuit (b) Fully assembled wireless sensing unit

Figure 2: Photos of the prototype wireless sensing unit.

Figure 3: Single-hop wireless sensor network.

shows printed circuit board (PCB) of the signal conditioning circuit. The signal conditioning circuit provides a band-pass filter with the passing frequency band set at 0.014–25 Hz, which is suitable for modal vibration tests with most civil structures. Three amplification factors (5, 10, and 20) are available with the signal conditioning board. Figure 2(b) shows a fully assembled wireless sensing unit.

As illustrated in Figure 3, a single-hop WSN is employed, with one central server directly communicating with all wireless sensing units. Another approach to address the transmission range issue is through multihopping with relatively short-range wireless transceivers. Compared with single-hopping with relatively long-range transceivers, both approaches have pros and cons. Under scenarios where significant obstruction cannot be penetrated by the wireless signal, multi-hopping through relay units can potentially circumvent the obstruction. On the other hand, reliable multi-hopping requires more complicated middleware implementation on the wireless sensing units; relaying data through multiple wireless sensing units also causes greater communication latency. Furthermore, all relaying nodes along the hopping path have to consume some amount of battery power for transmitting a single packet. For this

(a) Signals $x(t)$ and $y(t)$

(b) Cross-correlation function

Figure 4: Two signals and the corresponding cross-correlation function.

(a) Picture

(b) Elevation drawing

Figure 5: Picture and elevation of Baicheng Footbridge.

Figure 6: Locations of sensors on bridge deck.

Figure 7: Tethered and wireless sensors installed on bridge deck.

study, benefiting from the long communication distance of the 24XStream transceiver, the single-hop WSN is sufficient for the ambient vibration tests with the two test-bed bridge structures, particularly when the deployment is divided into several test setups with the aid of a reference point.

## 3. Mathematical Tools

*3.1. Time Synchronization Assessment.* Time synchronization for data collected by multiple wireless sensor nodes is an important concern in wireless sensor network. When the central server broadcasts a beacon signal to all the wireless

Figure 8: Side view of Wuyuan Bridge.

Figure 9: Measurement locations during ambient vibration test at Wuyuan Bridge.

sensing units, the units would not receive this beacon at precisely the same time. The timing difference can usually lead to synchronization error at around tens of microseconds [16]. Furthermore, the wireless sensing units cannot share a common data repository clock as tethered DAS, and the crystal clocks of wireless sensing units gradually diverge from each other [22]. The unavoidable time shift in the crystal clocks of wireless sensing units would accumulate to a large time synchronization error over time. As a result, time synchronization error should be considered in WSN application for structural vibration monitoring. The time synchronization error possesses certain innate randomness, which makes it challenging to completely eliminate in data after processing. Although asynchronous data would oftentimes not affect the identification of damping ratios and natural frequencies, such data usually result in errors in identified mode shapes [19, 23]. Therefore, the time synchronization accuracy of WSN should be carefully assessed through field experimental studies.

In this paper, cross-correlation function is employed for identifying the time synchronization error of the single-hop WSN in field experimental studies. The cross-correlation function quantifies the waveform similarity or correlation between two time series (or signals) as a function of a time shift (i.e., the time delay or time-lag) applied to one of them [24]. It is well known that cross-correlation function has been employed to estimate the travel time in the time-of-arrival (TOA) method for GPS positioning or locating wireless nodes. Typically, the TOA is estimated by the time shift corresponding to the maximum cross-correlation value between a received signal and a known transmitted signal [25]. In a similar way, cross-correlation function is evaluated between a time-shifted wireless signal and a reference (benchmark) tethered signal, in order to identify the actual time-lag between these two signals. The cross-correlation function between two continuous time series $x(t)$ and $y(t)$ is defined as

$$\begin{aligned} R_{xy}(\tau) &= \lim_{T\to\infty}\frac{1}{T}\int_0^T x(t-\tau)y(t)dt \\ &= \lim_{T\to\infty}\frac{1}{T}\int_0^T x(t)y(t+\tau)dt, \end{aligned} \tag{1}$$

where $\tau$ is the lag or time shift. For numerical computation, the cross-correlation function between two discrete time series $x(i)$ and $y(i)$ can be estimated by

$$R_{xy}(\tau_k) = \frac{1}{N}\sum_{i=0}^{N-k-1} x(i)y(i+k). \tag{2}$$

In addition, the standard or normalized cross-correlation function between two time series $x(i)$ and $y(i)$, sampled by time step $\Delta t$, can be estimated by [26] as follows:

$$r_{xy}(\tau_k) = \frac{(1/N)\sum_{i=1}^{N-k}\left(x(i)-\mu_x\right)\left(y(i+k)-\mu_y\right)}{\left[(1/N)\sum_{i=1}^{N}\left(x(i)-\mu_x\right)^2\right]^{1/2}\left[(1/N)\sum_{i=1}^{N}\left(y(i)-\mu_y\right)^2\right]^{1/2}}, \tag{3}$$

where $\tau_k = k\Delta t$ is the lag or time shift, $N$ is the number of data point in time series $x(i)$ or $y(i)$, and $\mu_x$ and $\mu_y$ are the mean values of time series $x(i)$ and $y(i)$, respectively. Following the definitions above, the normalized cross-correlation function has a value between $-1$ and 1. Figure 4(a) shows an example signal $x(t) = 5\cos(1.4\pi t - 0.25\pi) + 4\cos(4\pi t - 0.125\pi) + 3\cos(5.6\pi t + 0.125\pi) + 5\cos(8.4\pi t - \pi/12) + 8\cos(10\pi t + 0.2\pi)$. Signal $y(t)$ shares the same waveform with $x(t)$ and is simply $x(t)$ time-delayed by 1 second. Figure 4(b) shows the cross-correlation function between $x(t)$ and $y(t)$, and the peak of $r_{xy}$ clearly identifies the time lag (100 time-steps with $\Delta t = 0.01$ second), which corresponds to 1 second time delay.

As demonstrated by numerous studies in wireless structural health monitoring, a well-designed wireless sensing unit usually provides very close waveform to a reference tethered signal (collected side-by-side at the same location). Hence, cross-correlation function between a wireless sensor signal and the reference tethered signal can be calculated to find the time lag between them, which is simply identified from the time shift $\tau_k$ that corresponds to the peak value of the cross correlation function. In addition, for this study, tethered sensor signals at different locations can be regarded as "perfectly" synchronized. Time lags can be calculated for different pairs of wireless and tethered signals, potentially measured at different locations and collected by different wireless and tethered sensing units/nodes. If approximately the same amount of time lag is identified among different

(a) Baicheng Footbridge (time synchronization assessment test 1)

(b) Wuyuan Bridge (time synchronization assessment test 4)

Figure 10: Normalized cross-correlation function between wireless and tethered sensing data.

(a) Acceleration Responses

(b) Fourier spectra

Figure 11: Comparisons of acceleration time history and frequency spectra at sensor location number 4 (Baicheng Footbridge Test).

(a) Acceleration Responses

(b) Fourier spectra

Figure 12: Comparisons of acceleration time history and frequency spectra at sensor location number 1 (Wuyuan Bridge Test).

(a) CC index of the 3rd data set of Baicheng Footbridge test

(b) CC index of the 5th data set of Wuyuan Bridge test

Figure 13: Correlation coefficient (CC) index variation with vibration magnitude.

(a) PRD index of the 3rd data set of Baicheng Footbridge test

(b) PRD index of the 5th data set of Wuyuan Bridge test

Figure 14: Percent RMS difference (PRD) index variation with vibration magnitude.

(a) Wireless

(b) Tethered

Figure 15: Singular values of the PSD matrix of acceleration responses measured by WSN and cable-based DAS (Baicheng Footbridge).

(a) Wireless (b) Tethered

Figure 16: Stabilization diagram of SSI algorithm based on acceleration responses measured by WSN and cable-based DAS (vertical axis: system order, Baicheng Footbridge).

(a) Mode 1 (b) Mode 3

(c) Mode 4 (d) Mode 5

Figure 17: Identified mode shapes of Baicheng Footbridge based on wireless sensing data (Top view; Side view +X; Side view +Y; 3D view).

pairs, the wireless data collected by different wireless sensing units are determined to be well synchronized, or to be more precise, and the time synchronization error is determined to be less than one sampling time-step. On the contrary, if different amount of time lags are identified for different pairs of wireless and tethered signals, the time synchronization error among wireless data can be estimated using the different time lags.

*3.2. Data Quality Index.* Data quality of WSN, to a certain extent, determines its suitability for ambient vibration tests and SHM. It can be challenging to obtain high-fidelity and accurate wireless sensing data in ambient vibration tests and SHM applications, because of limited capacity of on-board A/D conversion and on-board signal conditioning [7, 15, 16]. Lynch et al. [15] pointed out that the effective resolution of an onboard A/D converter is usually slightly lower than

its nominal value, which poses a significant challenge in low-magnitude vibration measurement. In addition, the performance of classical amplifier and passive filter without power supply is expected to provide performance inferior to that of more powerful counterparts used in tethered DAS. Third, data loss during wireless transmission is also a concern. All these factors would results in data quality problem, so the data quality of WSN needs to be carefully investigated by quantitative indexes and through field tests. As mentioned before, although a number of field tests on WSN performance in full-scale structural applications have been reported [3, 7, 8, 14–18], quantitative research about data quality is rarely found up to date. Therefore, this paper adopts two data quality indexes for wireless data quality assessment. The conditions for high data quality and low data quality are identified, respectively, and a discussion follows on their influences on output-only modal identification.

The aforementioned cross-correlation function can measure the waveform similarity between two signals. Because tethered sensing data are expected to be more accurate than wireless sensing data, the tethered data is used as a reference signal to examine the quality of wireless sensing data. This paper adopts the correlation coefficient as an indicator for the quality of wireless sensing data, after shifting the wireless data according to the lag. Correlation coefficient is a special case of the cross-correlation function when time lag $k$ is equal to zero. The correlation coefficient (CC) between wireless data $x(i)$ and tethered data $y(i)$ can be derived by (3), which is given by

$$r_{xy}(0) = \frac{\sum_{i=1}^{N}(x(i)-\mu_x)\left(y(i)-\mu_y\right)}{\left[\sum_{i=1}^{N}(x(i)-\mu_x)^2\sum_{i=1}^{N}\left(y(i)-\mu_y\right)^2\right]^{1/2}}. \quad (4)$$

A correlation coefficient $r_{xy}(0)$ equal to 1 shows that the two signals share identical waveforms or shapes, but their magnitudes may be different. Therefore, another index, the classical percent root-mean-square difference (PRD), is employed to examine the data quality as a supplement to correlation coefficient index. The PRD index is given by [27] as follows:

$$\mathrm{PRD} = 100 \times \left[\frac{\sum_{i=1}^{N}(x(i)-y(i))^2}{\sum_{i=1}^{N}(y(i))^2}\right]^{1/2}. \quad (5)$$

*3.3. Output-Only Modal Identification.* Two popular methods for output-only modal parameter identification are the enhanced frequency domain decomposition (EFDD) method in frequency-domain [28] and the stochastic subspace identification (SSI) method in time domain [29, 30]. A short and brief summary of the methods is provided herein. The original references by Brincker et al. [28], van Overschee and de Moor [29], and Peeters and de Roeck [30] are strongly recommended for interested readers. In addition, the modal assurance criterion (MAC) is employed to examine the consistency of mode shape identification based on wireless and tethered sensing data.

*3.3.1. Enhanced Frequency Domain Decomposition (EFDD).* In the EFDD method, the relationship between the unknown input $x(t)$ and the measured response $y(t)$ can be expressed as

$$\mathbf{G}_{yy}(\omega) = \mathbf{H}^*(\omega)\mathbf{G}_{xx}(\omega)\mathbf{H}^T(\omega), \quad (6)$$

in which $\mathbf{G}_{xx}$ is the input power spectral density (PSD) matrix, $\mathbf{G}_{yy}$ is the output PSD matrix, $\mathbf{H}(\omega)$ is frequency response function (FRF), and superscripts "$*$" and "$T$" denote the complex conjugate and transpose, respectively. If assuming the unknown input as white noise, that is, $\mathbf{G}_{xx}$, is a constant matrix, the FRF matrix can be solved by the output PSD matrix based on (6). EFDD method employs singular value decomposition (SVD) to calculate the singular values and vectors of $\mathbf{G}_{yy}$, which correspondingly provides the natural frequencies and mode shapes [28].

*3.3.2. Stochastic Subspace Identification (SSI).* In state-space formulation, the system input vector can be formulated to contain various white noise vectors, including force excitation, process errors, and measurement noises. As a result, a multi-degree-of-freedom linear structure can be represented as a stochastic state-space model, which is given by

$$\begin{aligned}\mathbf{X}_{k+1} &= \mathbf{A}\mathbf{X}_k + \mathbf{w}_k,\\ \mathbf{Y}_k &= \mathbf{C}\mathbf{X}_k + \mathbf{v}_k,\end{aligned} \quad (7)$$

where $\mathbf{X} = [\mathbf{x} \quad \dot{\mathbf{x}}]^T$ is the state vector; $\mathbf{w}_k$ and $\mathbf{v}_k$ are white-noise random vectors; $\mathbf{C} \in \mathbb{R}^{l\times n}$ is the output matrix. The measured data $\mathbf{Y}$ is used to identify the system matrix $\mathbf{A}$ using the SSI algorithm [29]. Based on system matrix $\mathbf{A}$, it is straightforward to solve the natural frequencies, damping ratios, and mode shapes through eigenvalue decomposition.

*3.3.3. Modal Assurance Criterion (MAC).* Modal assurance criterion (MAC) is widely used to provide an additional confidence evaluation in mode shape identification. The function of the MAC is to provide a measure of consistency (degree of linearity) between estimated mode shape vectors [31]. The MAC between different mode shapes identified based on wireless and tethered sensing data is given by

$$\mathrm{MAC}(\psi_{\mathrm{iw}}, \psi_{\mathrm{it}}) = \frac{\left|\psi_{\mathrm{iw}}{}^T \cdot \psi_{\mathrm{it}}\right|^2}{\left(\psi_{\mathrm{iw}}{}^T \cdot \psi_{\mathrm{iw}}\right)\cdot\left(\psi_{\mathrm{it}}{}^T \cdot \psi_{\mathrm{it}}\right)}, \quad (8)$$

in which $\psi_{\mathrm{iw}}, \psi_{\mathrm{it}}$ is the mode shape vector identified based on wireless and tethered sensing data, respectively.

## 4. Experimental Setup

In order to validate and assess the performance of WSN, two full-scale bridges located in Xiamen, China, were selected as the test-beds. The WSN and traditional tethered DAS were installed in the two test-beds for measuring ambient vibration.

*4.1. Test-Bed 1: Baicheng Footbridge.* Baicheng Footbridge, a one-side cable-stayed steel footbridge located at Baicheng beach, is on southeast part of the campus of Xiamen University in China. Figure 5 shows the picture and elevation drawing of the footbridge. The total length of Baicheng Footbridge is 72.2 m, which consists of three spans (12.1 m + 48.8 m + 12.1 m). The steel box girder of the bridge is a space-curved beam, consisting of one circular arc in elevation and three independent circular arcs in plan view. Due to its complex shape, the Baicheng Footbridge possesses somewhat complicated dynamic characteristics. In addition, this footbridge is susceptible to pedestrian loading and occasionally suffers excessive vibration. Hence, this Footbridge was selected as a test-bed for evaluating the performance of the WSN.

To investigate the performance of the WSN, a traditional DAS is also installed in parallel. Both the wireless and tethered systems employ accelerometers to measure the ambient vibration in the vertical and radial directions of the bridge deck. Optimal sensor placement is a critical issue in SHM, many methods have been proposed in the last decade [32–36]. Some widely accepted methods intrinsically concentrate on positions with high energy content (or positions with large vibration magnitude), resulting in sensors quasi-uniformly spaced and symmetrically deployed [32]. Consequently, a uniform and symmetric sensor placement scheme is employed. Figure 6 shows that the accelerometers are mounted in locations marked as $\#1, 2, \ldots, 18$ along the bridge deck. At each measurement point, two accelerometers, one connected to a wireless sensing unit and one with the tethered DAS, are aligned side by side (Figure 7).

Table 1 shows the test setups of Baicheng Footbridge ambient vibration test. Due to the limitation of bandwidth capacity, three sets of vibration tests (setup number 1∼3) are first conducted to collect data from the sensors with a sampling frequency of 50 Hz. For time synchronization error assessment, another two scenarios (setup numbers 4 and 5) are deployed with a sampling frequency of 50 Hz and 100 Hz, respectively. Both WSN system and tethered DAS (INV306) use the low pass filter with the same cut-off frequency of 25 Hz. Each test setup was repeated several times, depending on the data quality. The duration of each testing is 300 seconds. Table 2 shows the accelerometers adopted in the Baicheng ambient vibration test. The 891-II type accelerometer was used to measure vertical accelerations, while the piezoelectric type accelerometer was adopted to measure radial accelerations.

*4.2. Test-Bed 2: Wuyuan Bridge.* Wuyuan Bridge, located in the island ring road in Xiamen, China, is the first sea-crossing steel box arch bridge constructed in China (Figure 8). The bridge construction was finished in 2003. The total length of Wuyuan Bridge is 810 m and the width of the bridge deck is 34 m. As shown in Figure 8, the 3-span bridge (main span is 210 m long with two 58 m-long side spans) is a half-through X-style arch bridge consisting of steel-reinforced concrete girders and steel arch ribs. The arch has a ratio of rise to span equal to 1/4, with a second-degree parabolic shape. There are 9 strut rails on the arch to enhance the stability of the bridge in transverse direction.

In the Wuyuan Bridge ambient vibration test, vertical and transverse responses of the bridge deck and arch ribs were measured using both wireless and tethered accelerometers. For comparing the data quality of these two bridges tests, the same accelerometers were used in the ambient tests, as shown in Table 2. A signal conditioning circuit board is included in each wireless sensing unit to offset the sensor signals to have mean value of 2.5 V and to filter and amplify the weak and noisy ambient responses of the bridge. Amplification factor 20x is used for each wireless sensing unit. Due to the limitation of bandwidth capacity of 24XStream wireless transceiver, only 12 sensing units can simultaneously transmit data to the laptop (central server) when the sampling frequency is set to be 50 Hz. Five sets of vibration tests are carried out to collect data from the sensors locations in Figure 9. For assessing the data quality of wireless sensing data, acceleration responses at the same locations shown in Figure 9 were simultaneously measured by INV306 DAS with a sampling frequency of 50 Hz also. Different from the WSN, the tethered DAS used a 100x amplification factor. The low-pass filter cut-off frequency of tethered DAS is 25 Hz, which is the same as WSN. Table 3 shows the specific test setups of the Wuyuan Bridge ambient vibration test. In each testing, a total of 18,000 data points were measured with the duration of 360 seconds for both wireless and tethered systems, and each test setup was repeated several times for ensuring the quality of modal identification.

## 5. Results

This section discusses the testing results, with respect to the time synchronization of wireless sensing units, wireless sensing data quality assessment, and the output-only modal identification of Baicheng Footbridge and Wuyuan Bridge.

*5.1. Time Synchronization Assessment.* A limitation of WSN is the absence of a centralized clock; therefore, time synchronization is an important issue for WSN. The prototype wireless sensing unit used in this study is designed to synchronize its internal clock to a beacon signal broadcast from the centralized data repository. To validate the accuracy of wireless synchronization within a realistic field environment, the time lags between wireless sensing data and cable-based sensing data are investigated.

Three test scenarios for wireless synchronization assessment were carried out in Baicheng Footbridge test, as shown in Table 4. The sampling frequency is 50 Hz for test 1, and 100 Hz for test 2 and 3. Since the analysis resolution for time synchronization error is bounded by sampling interval, the wireless and tethered raw data were first up-sampled to 1 kHz sampling rate through interpolation. Figure 10(a) shows the normalized cross-correlation function of the two acceleration responses (wireless and tethered) recorded at measurement point number 4 of Baicheng Footbridge. With the aid of cross-correlation functions, a consistent time difference around 8.554 seconds (8554 sampling intervals

Table 1: Setups for ambient vibration tests at Baicheng Footbridge.

| Setup Number | Vertical measurement point | Radial measurement point | Objective |
|---|---|---|---|
| 1 | 3, 4, 5, 6, 9, 13, 14, 15, 18 | — | Modal identification |
| 2 | 1, 2, 7, 8, 10, 11, 16, 17 | — | Modal identification |
| 3 | — | 10, 11, 12, 13, 14, 15, 16, 17, 18 | Modal identification |
| 4 | 3, 4, 5, 7, 8, 12, 14, 16, 17 | — | Assessment of time synchronization |
| 5 | 5, 12, 14, 16 | — | Assessment of time synchronization |

Table 2: Parameters of accelerometer.

| Sensor type | Range (m/s$^2$) | Sensitivity (V/(m/s$^2$)) | Resolution (m/s$^2$) | Bandwidth (Hz), $^{+1}_{-3}dB$ | Application |
|---|---|---|---|---|---|
| 891-II | ±20 | 0.1 | $1 \times 10^{-5}$ m/s$^2$ | 0.5∼80 | Vertical measurement |
| Piezoelectric | ±100 | 0.05 | 0.0004 m/s$^2$ | 0.5–2500 | Radial measurement |

Table 3: Setups for ambient vibration tests at Wuyuan Bridge.

| Setup number | Vertical measurement points | Transverse measurement points | Reference point |
|---|---|---|---|
| 1 | 2, 3, 10, 11, 12, 13 | 10, 11, 12, 13 | 1 (V, T)* |
| 2 | 8, 9, 14, 15, 16 | 14, 15, 16 | 1 (V, T) |
| 3 | 4, 5, 6, 7, 8 | 5, 7 | 1 (V, T) |
| 4 | — | 2, 3, 4, 6, 8, 9 | 1 (V, T) |
| 5 | 17, 18, 19, 20, 21, 22, 23, 24, 25; | — | 1 (V, T) |

*V: vertical; *T: transverse.

Table 4: Estimation of time synchronization error for the entire measurement duration.

(a)

| Sensor location (Baicheng) | Test 1 (Baicheng) (ms) | Test 2 (Baicheng) (ms) | Test 3 (Baicheng) (ms) |
|---|---|---|---|
| #3 | $< \Delta t$ | — | — |
| #4 | $< \Delta t$ | — | — |
| #5 | — | $< \Delta t$ | 2 |
| #7 | $< \Delta t$ | — | — |
| #8 | $< \Delta t$ | — | — |
| #12 | $< \Delta t$ | $< \Delta t$ | $< \Delta t$ |
| #14 | $< \Delta t$ | 1 | 3 |
| #16 | $< \Delta t$ | 1 | 2 |
| #17 | $< \Delta t$ | — | — |

(b)

| Sensor location (Wuyuan) | Test 4 (Wuyuan) (ms) |
|---|---|
| #1 | $< \Delta t$ |
| #18 | 1 |
| #19 | $< \Delta t$ |
| #22 | $< \Delta t$ |
| #23 | 1 |
| #24 | $< \Delta t$ |

at 1 kHz) between wireless and tethered data is identified at all nodes of test 1. This result illustrates that the time synchronization error of the single-hop WSN is all less than one sampling interval ($\Delta t = 1$ ms) in test 1. In test 2 and 3 of Baicheng Footbridge, the time differences among the wireless sensing nodes have been found as 1 ms∼3 ms, as shown in Table 4. The maximum time synchronization error of the single-hop WSN is 3 ms in Baicheng Footbridge ambient vibration test, for sensor number 14 during Test 3.

Similar experimental results are achieved in Wuyuan Bridge test. Figure 10(b) shows the normalized cross-correlation function between wireless and tethered data recorded at measurement point #1 of Wuyuan Bridge. Again, a consistent time difference between wireless and tethered data is observed at all sensor locations except at measurement point #18 and 23. The time synchronization error among the wireless sensing nodes is found to be not more than 1 ms, as shown in Table 4. Overall, the two field experimental studies demonstrate that the time synchronization errors of the single-hop WSN are bounded by 3 ms, for the entire duration of the measurement. Therefore, beacon-based time synchronization algorithm in the single-hop network is quite effective and can provide highly accurate synchronized sensing.

*5.2. Quality of Wireless Sensing Data.* After being shifted according to the wireless-tethered time lag shown by correlation function, the acceleration time history collected by WSN is compared with the tethered baseline signals. Figure 11(a) shows the acceleration time histories recorded by the wireless sensing units at location #4 of Baicheng Footbridge, which matches well with those by the traditional cable-based DAS. The minor differences between amplitudes of the signals are due to several factors. Generally, the major error sources of wireless sensing include the relatively low resolution of onboard A/D converter, the limited abilities of prototype

Table 5: Modal identification of Baicheng Footbridge by EFDD.

| Mode | Wireless | | Tethered | | Frequency difference (%) | Damping difference (%) | Descriptions of mode shape | MAC |
|---|---|---|---|---|---|---|---|---|
| | $f_w$ (Hz) | $\xi_w$ (%) | $f_t$ (Hz) | $\xi_t$ (%) | | | | |
| 1 | 2.291 | 0.620 | 2.272 | 0.74 | 0.84 | 16.2 | Deck 1st vertical and radial bending | 0.999 |
| 2 | 4.067 | 0.88 | 4.058 | 0.95 | 0.22 | 7.4 | Deck bending and bridge tower bending | — |
| 3 | 5.323 | 0.29 | 5.320 | 0.33 | 0.06 | 12.1 | Deck 2nd vertical bending | 0.998 |
| 4 | 7.446 | 0.89 | 7.445 | 0.71 | 0.01 | 25.4 | Deck 2nd radial bending | 0.989 |
| 5 | 9.440 | 0.88 | 9.435 | 0.94 | 0.05 | 6.4 | Deck 3rd vertical and radial bending | 0.999 |

Table 6: Modal identification of Baicheng Footbridge by SSI.

| Mode | Wireless | | Tethered | | Frequency difference (%) | Damping difference (%) | Descriptions of mode shape | MAC |
|---|---|---|---|---|---|---|---|---|
| | $f_w$ (Hz) | $\xi_w$ (%) | $f_t$ (Hz) | $\xi_t$ (%) | | | | |
| 1 | 2.292 | 0.54 | 2.260 | 1.24 | 1.42 | 56.5 | Deck 1st vertical and radial bending | 0.997 |
| 2 | 4.042 | 0.53 | 4.067 | 0.88 | 0.61 | 39.8 | Deck bending and bridge tower bending | — |
| 3 | 5.319 | 0.41 | 5.323 | 0.40 | 0.08 | 2.5 | Deck 2nd vertical bending | 0.998 |
| 4 | 7.431 | 0.95 | 7.446 | 0.89 | 0.20 | 6.7 | Deck 2nd radial bending | 0.892 |
| 5 | 9.478 | 0.67 | 9.440 | 0.88 | 0.40 | 23.9 | Deck 3rd vertical and radial bending | 0.999 |

signal conditioning circuit, imperfect filtering, and variant sampling interval. Figure 11(b) shows the Fourier spectra of wireless and tethered sensing data, which are also in good agreement. Similar close match has also been found in the Wuyuan Bridge test, as shown in Figure 12.

Furthermore, for assessing the wireless data quality quantitatively, two data sets (wireless and tethered data, sampling frequency: 50 Hz) from the Baicheng Footbridge test and the Wuyuan Bridge test are selected to calculate the data quality indexes. First, each data set is divided into many short data segments (5 seconds, i.e., 250 data points per segment). Then, (4) and (5) are used to calculate the data quality indexes, including correlation coefficient (CC) and percent root-mean-square difference (PRD). Figure 13 shows the CC change with the vibration magnitudes. When the (root-mean-square) RMS vibration amplitude is larger than 1 mg (accelerometer output voltage changes by 0.98 mV), the CC index increases from 80% to 97%, which suggests a high waveform similarity between wireless and cable-based sensing data. However, the CC index sharply declines when the RMS vibration amplitude is less than 1 mg in both cases. Figure 14 shows the relationship between PRD and vibration amplitude. The PRD is usually in the range from 10% to 22%, when the vibration level is larger than 1 mg RMS acceleration.

As indicated by the CC and PRD indexes, the wireless and tethered data do not so "perfectly" match each other even in high vibration amplitudes, because the wireless and cable-based sensing both have different noise sources, that is, different data acquisition systems generate different measurement noises even their signal-to-noise ratios (SNR) are identical. Secondly, both CC index and PRD index are amplitude-dependent. When the vibration amplitude is larger than 1 mg, or equivalently, the analog sensor signal change is larger than 1 mV, high quality wireless sensing data can be expected. However, it is challenging to guarantee the data quality when the vibration amplitude is less than 1 mg RMS acceleration, or when the analog sensor signal change is less than 1 mV. Several factors may account for the measurement errors of wireless sensing unit at low vibration amplitude, for instance, resolution of A/D converter, inaccuracy of amplification and filtering circuit, circuit noise of sensor board, and the resolution of accelerometer. Higher resolution of A/D converter and more powerful signal conditioning circuit are helpful to achieve better performance with wireless data for extremely low-magnitude measurement. The prototype wireless sensing unit can perform well during the ambient vibration test when average RMS acceleration is larger than 1 mg. The peak vertical accelerations of the two bridges are 20 mg (Baicheng Footbridge) and 14 mg (Wuyuan Bridge) in the two field tests, respectively. Consequently, the data collected by the single-hop WSN in the two bridges are of high quality and sufficient for the output-only modal identification.

*5.3. Results of Output-Only Modal Identification.* The EFDD and SSI are employed for the identification of Baicheng Footbridge and Wuyuan Bridge based on the ambient acceleration responses collected by the wireless sensing units. In addition, the modal identification using cable-based sensing data are also conducted for examining the accuracy of the modal identification results from wireless measurements.

*5.3.1. Modal Identification Results of the Baicheng Footbridge.* Output-only modal identification usually demands lots of data for more precise analysis. A total length of 780 seconds of acceleration data, with 39,000 wireless sensing data points at each channel, are collected for modal identification of the Baicheng Footbridge. Meanwhile, a total length of 409.6 seconds of acceleration data, with 20,480 data points per channel, is measured by the cable-based DAS to estimate the modal parameters.

Figure 15 shows the singular values of the power spectral density (PSD) matrix based on the acceleration responses

Table 7: Modal identification of Wuyuan Bridge by EFDD.

| Mode | Wireless | | Tethered | | Frequency difference (%) | Damping difference (%) | Descriptions of mode shape |
|---|---|---|---|---|---|---|---|
| | $f_w$ (Hz) | $\xi_w$ (%) | $f_t$ (Hz) | $\xi_t$ (%) | | | |
| 1 | 0.774 | 2.14 | 0.780 | 1.99 | 0.77 | 7.5 | 1st vertical bending (antisymmetric) |
| 2 | 1.210 | 1.03 | 1.211 | 0.89 | 0.08 | 15.7 | 1st lateral bending (symmetric) |
| 3 | 1.255 | 0.79 | 1.258 | 1.05 | 0.24 | 24.8 | 2nd vertical bending (symmetric) |
| 4 | 1.631 | 0.27 | 1.638 | 0.87 | 0.43 | 69.0 | 1st torsion (anti-symmetric) |
| 5 | 1.705 | 0.57 | 1.699 | 0.99 | 0.35 | 42.4 | 3rd vertical bending (symmetric) |
| 6 | 1.897 | 0.62 | 1.887 | 1.29 | 0.53 | 51.9 | 2nd torsion (symmetric) |
| 7 | 2.440 | 0.54 | 2.443 | 0.69 | 0.12 | 21.7 | 3rd torsion (symmetric) |
| 8 | 2.700 | 0.74 | 2.730 | 0.65 | 1.10 | 13.8 | 4th vertical bending (anti-symmetric) |
| 9 | 3.296 | 0.28 | 3.317 | 0.76 | 0.63 | 63.2 | 5th vertical bending (symmetric) |

Table 8: Modal identification of Wuyuan Bridge by SSI.

| Mode | Wireless | | Tethered | | Frequency difference (%) | Damping difference (%) | Descriptions of mode shape |
|---|---|---|---|---|---|---|---|
| | $f_w$ (Hz) | $\xi_w$ (%) | $f_t$ (Hz) | $\xi_t$ (%) | | | |
| 1 | 0.775 | 1.93 | 0.775 | 1.56 | 0 | 23.7 | 1st vertical bending (antisymmetric) |
| 2 | 1.230 | 1.36 | 1.232 | 1.37 | 0.16 | 0.7 | 1st lateral bending (symmetric) |
| 3 | 1.254 | 0.64 | 1.251 | 1.32 | 0.24 | 51.5 | 2nd vertical bending (symmetric) |
| 4 | 1.603 | 1.52 | — | — | — | — | 1st torsion (anti-symmetric) |
| 5 | 1.707 | 0.91 | 1.712 | 1.71 | 0.29 | 46.8 | 3rd vertical bending (symmetric) |
| 6 | 1.898 | 1.63 | 1.867 | 1.06 | 1.66 | 53.8 | 2nd torsion (symmetric) |
| 7 | 2.429 | 1.97 | 2.425 | 2.31 | 0.16 | 14.7 | 3rd torsion (symmetric) |
| 8 | 2.872 | 2.38 | 2.683 | 1.50 | 7.04 | 58.7 | 4th vertical bending (anti-symmetric) |
| 9 | 3.295 | 1.59 | 3.313 | 1.70 | 0.54 | 6.5 | 5th vertical bending (symmetric) |

measured by WSN and cable-based DAS, respectively. The five peaks are very clear and well separated from each other. The identified natural frequencies and damping ratios of the first five modes of the footbridge are summarized in Table 5. For evaluating the accuracy of the modal identification using wireless sensing data, the results identified by EFDD using tethered data are also presented in Table 5. It can be observed that the identified natural frequencies of wireless system match well with those of tethered system. The relative differences of natural frequencies identified by the two types of data vary from 0.01% to 0.84%, with an average value of 0.236%. A close match between wireless and tethered results is also found in the identified mode shape. MAC values shown in Table 5 are quite high, from 0.989 to 0.999 with an average value of 0.996.

Figure 16 shows the SSI stabilization diagrams of the acceleration responses measured by WSN and cable-based DAS, respectively. The first five modes are well identified as shown in Figure 16. Table 6 shows the natural frequencies, damping ratios and mode shape descriptions, for the first five modes of the footbridge using SSI algorithm. The relative difference and MAC values between wireless and tethered results clearly demonstrate that the WSN can offer sufficiently accurate data for identifying the natural frequencies and mode shapes. However, the identified damping ratios are more different between wireless and tethered systems, with an average value of 13.5% (EFDD) and 25.9% (SSI). These significant differences in identified damping ratios may be attributed to the sight difference in acceleration amplitude measurements by wireless and tethered system, as shown in Figures 11 and 12. Nevertheless, damping ratios are notoriously uncertain in modal analysis, even just comparing among wired measurements. In general, variations up to 200% are commonly encountered [37]. Damping ratio estimation is constrained by dynamics modeling, identification algorithm, and vibration magnitudes, and structural damping is intrinsically nonlinear. As a result, when the structural dynamic behavior is represented by a linear model, such as in EFDD and SSI algorithms, the estimated equivalent damping ratios unavoidably demonstrate large variations. Damping ratio identification is still a widely known challenge in the research community. Consequently, the large difference in damping ratios listed in Tables 5 and 6 are primarily due to the difficulty of damping ratio identification in general, rather than being caused by the difference in wireless and tethered data. A similarly big difference in identified damping ratios between wireless and cable-based systems is also reported in the literature [37].

Figure 17 shows the mode shapes of Baicheng Footbridge. The 2nd mode shape is absent because the vibration

Figure 18: Stabilization diagram of SSI algorithm using data from wireless setup 1 (vertical axis: system order).

of tower was not fully measured due to sensor installation difficulties. The modal properties of one-side cable-stayed steel footbridges are rarely reported in literature. Due to its special space configuration, its mode shapes also exhibit some special features. The vertical and radial mode shapes are coupled in the 1st and 5th modes, as shown in Figure 17. The fundamental frequency of the Baicheng Footbridge is 2.292 Hz, which is close to the footfall frequency of pedestrian walking (typically around 2 Hz) [38]. Therefore, pedestrians can induce excessive vibrations to the bridge by "resonance" effect. Overall, the modal identification results presented in this section illustrate a good performance of wireless sensor instrumentation at this short-span bridge.

*5.3.2. Modal Identification Results of the Wuyuan Bridge.* Although the WSN works well at the Baicheng Footbridge, the system should also be tested in more complex and harsh environments posed by long-span bridges. Hence, Wuyuan Bridge is selected as another test-bed for experimental study of the prototype wireless sensing system. A total length of 360 seconds acceleration data (18,000 data points per channel) are measured by the WSN and employed to identify the modal parameters of the Wuyuan Bridge. Meanwhile, 409.6 seconds of acceleration responses (20,480 data points per channel) measured by the cable-based DAS are also used to identify the bridge modal properties.

Table 7 shows the modal identification results by EFDD algorithm based on wireless and tethered data. The first nine modes, with very closely spaced resonance frequencies, are identified. Based on wireless data, the fundamental frequency and damping ratio of Wuyuan Bridge are identified as 0.774 Hz and 2.14%, respectively. A good agreement is found between wireless and tethered results, as shown in Table 7. Average relative difference between identified natural frequencies is 0.47% and the difference between identified damping ratios is 34.4%, both about roughly twice of that from Baicheng Footbridge testing. This is likely because the vibration amplitude of Baicheng Footbridge is larger than Wuyuan Bridge and thus the WSN in former test can provide higher data quality than the latter.

Figure 18 shows the stabilization diagram of SSI algorithm of wireless setup 1. The identified frequencies and damping ratios of the first nine modes of the Wuyuan Bridge using SSI are summarized. Excluding the large relative difference in natural frequency of 8th mode, the other identified frequencies based on wireless data are overall consistent with the tethered results. A large relative difference in identified damping ratios, averaging 32.05%, is also observed in Table 8. In addition, the fourth mode (1st torsion) is not identified based on tethered data by SSI algorithm.

Figure 19 shows the comparisons of identified mode shapes of Wuyuan Bridge based on wireless and tethered sensing data. A very good agreement between the identified mode shapes from wireless and tethered data is found in Figures 19(a)–19(f), which is also indicated by a set of high MAC values (averaged 0.956). It is recognized here that the MAC values are also lower than those of Baicheng Footbridge test, which may also be attributed to a higher data quality at Baicheng Footbridge test.

Figure 20 shows the 3D view of identified mode shapes of Wuyuan Bridge based on wireless sensing data. The identified first vertical bending mode shape of Wuyuan Bridge has an antisymmetric waveform, which is similar to the second mode shape of a simply supported beam. This identified result is consistent with the theoretical solution of the mode shape of arch bridges [39] and also consistent with the experimental results at another arch bridge in China [40].

All the identified results of natural frequencies and mode shapes based on wireless sensing data are consistent with those based on conventional tethered data. Larger differences have been found in the identified damping ratios from wireless sensing data. Considering the commonly occurring uncertainty in damping ratio identification, even using traditional tethered sensing data, the performance of WSN for output-only modal identification has been well demonstrated in these two field ambient vibration tests.

## 6. Conclusion and Discussion

This paper presents the quantitative experimental studies on time synchronization, data quality, and accuracy for output-only modal identification using a single-hop WSN system. Both short-span to long-span bridge applications are studied. The cross-correlation function between wireless and tethered signal is employed to assess the time synchronization accuracy of the WSN. The field experimental results illustrate that time synchronization errors are bounded by 3 ms for the entire duration of the data collection. Hence the time synchronization error of the single-hop WSN is regarded as negligible in output-only modal identification of civil structures.

In addition, this paper proposes two data quality indexes, correlation coefficient (CC) and percent root-mean-square difference (PRD), for assessing the accuracy of wireless sensing data. High-quality data (CC: 80% to 97%; PRD: 10% to 22%) was collected when ambient vibration amplitude is relatively large (RMS > 1 mg, i.e., accelerometer voltage change > 0.98 mV). However, the data quality indexes decline when the vibration amplitude is low (e.g., accelerometer

(a) 1st vertical bending mode shape (frequency = 0.776 Hz, damping ratio = 2.57%, by SSI)

(b) 2nd vertical bending mode shape (frequency = 1.242 Hz, damping ratio = 0.65%, by SSI)

(c) 3rd vertical bending mode shape (frequency = 1.698 Hz, damping ratio = 1.26%, by SSI)

(d) 1st torsional mode shape (frequency = 1.631 Hz, damping ratio = 0.27%, by EFDD)

(e) 2nd torsional mode shape (frequency = 1.837 Hz, damping ratio = 3.97%, by SSI)

(f) 3rd torsional mode shape (frequency = 2.431 Hz, damping ratio = 0.69%, by SSI)

Figure 19: Comparisons of identified mode shapes of Wuyuan Bridge based on wireless and tethered sensing data.

output < 0.98 mV). Potential approaches to further improve data quality include higher resolution sensors, more powerful signal conditioning circuit with larger amplification factors [41], and higher resolution A/D converters.

The output-only modal identifications of Baicheng Footbridge and Wuyuan Bridge were carried out based on the wireless and cable-based sensing data. The results demonstrate that the identified natural frequencies using wireless sensing data are very accurate, where average relative difference from the tethered results is 0.236% for Baicheng Footbridge and 0.47% for Wuyuan Bridge. The mode shapes are also reliably identified. The averaged MAC value between wireless and tethered data is 0.996 for Baicheng Footbridge, and 0.956 for Wuyuan Bridge. However, large relative differences (averaging 13.5% for Baicheng Bridge test and 34.4% for Wuyuan Bridge test) in identified damping ratios were observed between wireless and tethered data. However, such difference is primarily due to the difficulty of damping ratio identification in general, rather than being caused by the difference in wireless and tethered data. Because the vibration amplitude of Baicheng Footbridge is larger than that of Wuyuan Bridge, providing higher signal-to-noise ratio, modal identification results from the former is overall more accurate than the latter.

In summary, the field ambient vibration tests demonstrate the feasibility of output-only modal identification using WSN. The accuracy can be assured if the ambient vibration amplitude is larger than RMS 1 mg, or in this case, when the analog sensor output voltage changes by more than 1 mV. From our experience, in addition to extremely low-amplitude ambient vibration test, another two challenges facing WSN in ambient vibration tests are the bandwidth limitation of wireless communication and the power supply for wireless sensors. Low power wireless sensors and energy harvesting technologies [42, 43] should be developed for next generation wireless sensing systems. Wireless sensors, with the characteristics of low cost, mobile, easy installation, and increasingly more powerful embedded computation capability, are expected to keep evolving and to enable exciting future applications on civil infrastructure. WSN, a significantly valuable product from technology revolution in modern era, is highly promising for widespread deployment in the vibration testing and safety monitoring of civil structures.

(a) Mode 1 (Vertical mode 1) (b) Mode 2 (Lateral mode 1) (c) Mode 3 (Vertical mode 2)

(d) Mode 4 (Torsion mode 1) (e) Mode 5 (Vertical mode 3) (f) Mode 6 (Torsion mode 2)

(g) Mode 7 (Torsion mode 3) (h) Mode 8 (Vertical mode 4)

Figure 20: 3D view of identified mode shapes of Wuyuan Bridge based on wireless sensing data.

## Acknowledgments

The authors are grateful for the financial supports from the China National High Technology Research and Development Program through Grant No. 2007AA04Z420 and from State Key Laboratory for Disaster Reduction in Civil Engineering at Tongji University through Project No. SLDRCE10-MB-01. Y. Wang is supported by the US National Science Foundation CAREER Award CMMI-1150700. Any opinions, findings, and conclusions or recommendations expressed in this publication are those of the authors and do not necessarily reflect the view of the sponsors. The authors would like to thank Professor J. P. Lynch from University of Michigan, Ann Arbor, MI, USA for providing valuable support on this research. In addition, Assoc. Prof. Yu Song from Xiamen University is gratefully acknowledged for the assistance in modal identification and field testing.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 545148, 10 pages
doi:10.1155/2012/545148

*Research Article*

# Multisensors On-Site Monitoring and Characteristic Analysis of UHV Transmission Tower

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Received 19 June 2012; Revised 23 August 2012; Accepted 25 September 2012

Academic Editor: Gangbing Song

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It is important to clarify the variational rules between dynamic characteristics and gust response factors (GRFs) of an ultra high-voltage (UHV) transmission tower for the structural design. In this paper, the synchronous multisensors on-site monitoring has been carried out under field wind conditions, which includes field fluctuating wind velocity, acceleration responses, and base bending strains on parts of the just-built transmission tower. The dynamic characteristics of tested transmission tower were investigated by a modal identification method (MIM) and validated those results with a finite-element method (FEM). Similarly, the GRFs based on the top displacement or base dynamic bending stains are calculated and compared by means of a measurement data statistical method and some different quasistatic methods, respectively. The results have shown that the dynamic characteristics obtained from the measurement data analysis with the MIM agree with those from the FEM, which confirms that the FEM and the MIM for dynamic characteristics are consistent and correct. In turn, the GRFs form the measurement displacement analyses have some differences from those calculated with the quasistatic method, and it is necessary to further identify and verify their causations.

## 1. Introduction

Transmission tower-line system (TTLS) is a dynamic response-sensitive structure system under wind loading, which is sensitive with analytical model and load uncertainty as well as many unpredictable difficulties in the reliability evaluation of the used structures for designer [1]. Thus, it is important to clarify the variational rules between dynamic characteristics and GRFs of UHV transmission tower for the structural design. The present studies indicate that there are two reasons for this situation. The first one is that once such transmission-line system has been built and put into use, it becomes a bearing high electric energy system. In such situation, it's difficult to monitor its structural responses induced by the environmental load. The second one is that, there is a parameter limitation with a specific structure design process from different specifications or specific monitoring. As for the integrity of such a structure system and its uncertainty, it is very necessary to carry out a comparative study between experimental test analysis and structural design theory for such transmission-tower structures.

As a result of some economic and technical limitation for the on-site testing and monitoring of transmission tower, some pervious methods for its dynamic response analysis and structure design are mostly to choose the numerical analyses or wind tunnel tests [2–4] so that the measurement data and its corresponding analytical results are very limited. Based on the full-scale measurement data, Takeuchi et al. [5] studied aerodynamic damping properties of two towers under strong wind conditions. Meanwhile, they established a new method which was applicable to the response record of a multidegree-of-freedom system such as the coupled structure of transmission tower-and-conductors, resulting in that the wind speed dependency of the aerodynamic damping property of the coupled tower-and-conductors system had wide differing characteristics depending on the vibration mode. Lam and Yin [1] and Yin et al. [6] carried out a feasibility study of utilizing ambient vibration data measured from a limited number of sensors in the structural damage detection of transmission towers. The proposed methodology can identify the damaged substructure by estimating the "equivalent" stiffness reduction even in the presence of

Figure 1: The different measurement states of transmission tower-line system.

both measurement noise and modeling error. Momomura et al. [7] and Okamura et al. [8] took the leader on the field monitoring for the wind characteristics and dynamic responses of transmission tower of an TTLS in a mountainous area and then simulated the mountain area wind field in a wind tunnel. These researches found that the aerodynamic response values of acceleration and strain of the tower with conductors and tensile forces of conductors increased in proportion to the power of wind speed. Harikrishna et al. [9], in order to test the designed structural reliability of a communication tower, carried out a comparative study on the results from the measurement data analysis and the design methods of different specifications, respectively. Savory et al. [10] established a hurricane forcing calculation model and its corresponding failure analytical method for the TTLS damage in a middle latitude area caused by the extreme wind load of tropical storm. Paluch et al. [11] based on the statistical analysis of a transmission-crossing long-term field testing data, obtained the wind field properties and the structural response rules, and then analyzed their consistency with a numerical analytical method. Comparatively, the theoretical analysis combined with the field measurement for the TTLS is rarely developed, and making a fore-step advance in this aspect is very useful.

In this paper, taken a just-built transmission tower of a TTLS in Gaizhou city in Liaoning province, as an investigated object, its dynamic characteristics and structural parameters have been measured and analyzed with both of a with-line and a without-line working conditions. In turn, a method for solving the GRFs which related structural dynamic characteristics has been presented; then the various analytical results of the GRF obtained from the measurement data analytical method and different quasistatic structural design methods are comparatively analyzed. Therefore, multisensors on-site monitoring test, wind-induced vibration analyses, and quasistatic design method have been integrated as a system for solving the design problem of a TTLS.

## 2. Multisensors On-Site Monitoring Test for Transmission Tower

*2.1. Structure Feature of Transmission Tower.* A transmission crossing is located in Gaizhou City in Liaoning, which belongs to a part of a just-built 550 kV TTLS. The monitored main tower is a type of SZ21 straight line tower, with total height of 56.9 m (nominal height 33 m), tower base width of 8.84 m and tower top width of 2.0 m. There are three crossing arms, respectively, located on the top, middle, and bottom of the tower up-position, in which its minimum arm length is 8 m. The tower can be divided into two segments with uniform taper section on its nominal height and its section gradually shrinking to upwards. Its two adjacent spans are 426 m and 294 m, respectively. The structural diagram and composites of the transmission tower are shown in Figure 1 and Table 1, respectively.

*2.2. Multisensor Monitoring Arrangement.* In order to obtain some analytical contrast results from the measurement data, structure dynamic characteristics, and quasistatic design parameters of transmission tower, the field multisensor monitoring on the tower working conditions of with-lines and without-lines have mainly been measured for three transmission tower related terms, that is, the acceleration responses at the representative positions, the dynamic strains of the base leg members, and the wind velocity at the 10 m height recommended by wind load standards. Taking little difference of wind velocity variation within the height of transmission tower and anemometer measurement sensitivity into account, a wind velocity measurement point is arranged on the 10 m height position of tower. The wind velocities on other positions from the base to the top of the tower can calculate its mean values according to wind velocity index profile distribution property, and wherein its turbulence intensity is taken to be 0.145 as a result of its terrain characteristics, so that on different heights of

Table 1: The structural composites of the transmission tower and lines.

| Segment | Main members | Secondary members | Remarks |
|---|---|---|---|
| 1 | Q345-L160 × 14, L100 × 8 | Q235-L63 × 5, L50 × 5, L45 × 5 | Up-crossing bars |
| 2 | Q345-L90 × 7, L100 × 8, L160 × 14 | Q235-L63 × 5, L50 × 4, L45 × 4, L40 × 4 | Middle crossing bars |
| 3 | Q345-L160 × 4, L100 × 8, L80 × 7 | Q235-L56 × 5, L50 × 5, L45 × 4, L40 × 3 | Bottom-crossing bars |
| 4 | Q345-L100 × 8 | Q235-L90 × 7, L80 × 6, L63 × 5, L56 × 4, L40 × 3 | Top segment of tower |
| 5 | Q345-L140 × 10, L100 × 8, L80 × 6 | Q235-L90 × 7, L75 × 6, L63 × 5, L50 × 4, L45 × 4, L40 × 3 | Middle segment of tower |
| 6 | Q345-L160 × 10, L110 × 8 | Q235-L75 × 6, L70 × 6, L40 × 3 | Middle segment of tower |
| 7 | Q345-L160 × 12, L140 × 10, L100 × 8 | Q235-L90 × 7, L80 × 7, L70 × 5, L40 × 3, L50 × 4, L40 × 3 | Middle segment of tower |
| 8 | Q345-L180 × 14, L160 × 10 | Q235-L90 × 7, L100 × 8, L80 × 6, L70 × 5, L50 × 4, L45 × 5 | Middle segment of tower |
| 9 | Q345-L200 × 14, L180 × 12 | Q235-L90 × 7, L80 × 6, L56 × 4, L50 × 4 | Middle segment of tower |
| 10 | Q345-L200 × 14, L180 × 12 | Q235-L90 × 7, L90 × 8, L80 × 7, L75 × 6, L56 × 5, L50 × 5, L45 × 4 | Middle segment of tower |
| 11 | Q345-L90 × 7, L200 × 14, L180 × 14, L160 × 12, L100 × 8 | Q235-L90×7, L80 × 6, L75 × 5, L63 × 5, L56 × 4, L50 × 4, L45 × 4, L40 × 3 | Bottom leg of tower |
| 12 | Conductors | 4 × JGJ-400/35 | |
| 13 | Ground lines | JGJ-95/55 | |

the tower, the measured wind velocities (or wind loads) are diverse. The acceleration monitoring is used with a telemetry data receiving system (TDR03) of which sampling frequency is set to 50 Hz and provides 8 channels for 8 representative positions of the transmission tower and measures the acceleration data on the windward direction. The dynamic strains of the main leg members are measured by means of an optical fiber strain sensor monitoring system of which sensors are arranged on the side of the 4 main leg members above 2.0 m of the tower foundation.

All accelerometers, strain sensors, and an anemometer are connected to their respective data receiving systems as well as acquiring the data for 10 min at the meantime. The concrete sensor instruments and installation positions of the multisensors monitoring equipments are shown in Figure 2. The anemometer is arranged on the position with approximately 10 m height to the foundation level of transmission tower around which is a terrain of open flat farmland. The typical measurement wind velocity time history at the height 10 m is shown in Figure 3.

Herein, in order to compare the consistency between the measurement statistical results and numerical analytical results from the finite-element model of transmission tower structure, the dynamic characteristic parameters are determined and compared through a parameter identification of response mode, which excited by the measurement loads and analyzed by the FEM, respectively.

The modal identification method (MIM) is a method that derives the structural characteristics from the obtained excitation and responses, in the light of the number of excitation, which can be divided into both approaches of single-point excitation and multiple-point excitation. As the complexity of transmission tower is a high-rise structure with large flexible characteristics and during the monitoring wind loading energy is not large enough, and the MIM is conducted for multiple reference points in frequency domain, that namely, using multiple-point excitations and multiple-point responses of the measurement signals, accurately estimating a frequency response function, the modal parameters are directly identified form the relationship between the frequency response functions and modal parameters. The node displacement values should take the mean values from the nodes of a transmission tower in a high level. The details about this theory and method can be referred to in the literature [12–14]. The excitations for the parameter identification can take the measurement wind velocity time histories, and those of responses correspondingly take the acceleration time histories.

In order to analyze the influence of transmission lines on dynamic characteristics of transmission tower, the single tower model (without-lines tower model) and tower-lines model (with-lines model) have been built. In the former, the main members of transmission tower are simulated by the beam elements, and its secondary support members are substituted by the spatial bar elements. In the latter, the tower-lines structural system is truncated as "three towers and two spans" finite-element model, in which the lines are simulated as cable elements, and the tower members are the same as these of the single tower model. Under the conditions of without-lines and with-lines on tower, the dynamic characteristics of the structure from the MIM and the FEM, respectively, are shown in Tables 2 and 3. Meanwhile, in Figure 4, the first-order vibration modes in both orthogonal directions are presented. From Tables 2 and 3 and Figure 4, it can be seen that the first three-order natural frequencies of the transmission tower in in-plane are comparatively close to those in out-of-plane, and the in-plane nature frequencies are slightly larger than those in out-of-plane, which may be

Figure 2: The transmission tower structure and measuring instruments installation.

Figure 3: The typical measured wind velocity time history.

associated with the mass differential distribution of cross-arms of tower in two orthogonal directions that indicates the differential damping-mass effect. The results obtained from the FEM are much better in agreement with those of the MIM, the relative deviation between them is about 5%, which indicates that the finite-element numerical model and its results are credible.

Figure 4: Vibration types of the first mode: (a) out-of-plane; (b) in-plane.

Table 2: The natural frequencies of transmission tower without lines (Hz).

| Nature frequency | MIM | | | FEM | | |
|---|---|---|---|---|---|---|
| | The first-order mode | The second-order mode | The third-order mode | The first-order mode | The second-order mode | The third-order mode |
| *XOZ* plane (out-of-plane) | 1.611 | 4.847 | 6.471 | 1.525 | 4.660 | 6.370 |
| *YOZ* plane (in-plane) | 1.622 | 4.909 | 6.588 | 1.554 | 4.848 | 6.549 |

Table 3: The natural frequencies of transmission tower with lines (Hz).

| Nature frequency | MIM | | | FEM | | |
|---|---|---|---|---|---|---|
| | The first-order mode | The second-order mode | The third-order mode | The first-order mode | The second-order mode | The third-order mode |
| *XOZ* plane (out-of-plane) | 1.512 | 4.568 | 6.185 | 1.487 | 4.479 | 6.095 |
| *YOZ* plane (in-plane) | 1.544 | 4.647 | 6.235 | 1.503 | 4.522 | 6.163 |

## 3. Gust Response Factor from Measurement Data

*3.1. Gust Response Factor Based on Displacement.* Because the displacement sensor for measuring directly the displacement is usually expensive, such as a laser displacement sensor, but the usage of normal displacement sensor needs to set a reference structure, it is very difficult to directly measure the displacement response on various parts of transmission tower. In relative terms, the measurement technology of acceleration time history is relatively mature and practical. Taking into account the acceleration response in combination with the wind velocity time history, the corresponding displacement response curves can be obtained from the quadratic numerical integration of the motion equations as well as the displacement signal filtering out of their lower-frequency components. For this method, the precision can fully meet the dynamic characteristic analysis of transmission tower. The power spectral density curves of the measurement displacement at the tower top are shown in Figure 5, and the GRF of the measurement displacement can be defined by the ratio of the peak value versus mean value of the displacement. As for the mean wind effects from different wind velocity profiles, the relevant tower top displacements can be calculated by the finite-element model established in the second section. In this way, the limitation of the average displacements obtained from the accelerometer data analysis can be overcame. In analysis, it should be paid attention to which theoretically analytical conditions must be consistent with the measured wind velocities and their directions. The GRF based on the displacement can be expressed as follows:

$$G_x = 1 + \frac{g_{ex}\sigma_{ex}}{\overline{x}_t}, \tag{1}$$

where $\overline{x}_t$ is the mean displacement response form the theoretical calculation, $\sigma_{ex}$ denotes a root mean square (RMS) value of measurement displacement response which is obtained from the integral of acceleration response, and $g_{ex}$ means a statistical peak factor derived from the response process that is assumed as the random Gaussian process, namely,

$$g_{ex} = \sqrt{2\ln(\gamma_{ex}T)} + \frac{0.5772}{\sqrt{2\ln(\gamma_{ex}T)}}, \tag{2}$$

where $T$ denotes an observation duration 600 s and $\gamma_{ex}$ is an effective zero crossing frequency, that is,

$$\gamma_{ex} = \left[\frac{\int_0^\infty n^2 S_x(n)dn}{\int_0^\infty S_x(n)dn}\right]^{1/2}, \tag{3}$$

where $n$ represents the frequency (Hz) and $S_x(n)$ means a displacement power spectral density function.

Under the wind excitations with different turbulence intensities, the GRF for the displacement calculation obtained from the measurement acceleration integral is shown in Table 4. From the data analysis, it can be concluded that as the increasing of the mean wind velocity and turbulence intensity, the dynamic displacement response at the tower top is strengthened, and the GRF of displacement also increases gradually. The reasons for the phenomena can be attributed to two aspects; on one hand, the increase of turbulence intensity means the RMS deviation of the measurement displacement increased; on the other hand, the displacement mean value increasing can be regarded as the increase of mean wind velocity inducing windward force on the tower enhancing. Thereby, the rules reflected in Table 4 conform to the dynamic response characteristics under the fluctuating wind load.

*3.2. GRF Based on Bending Moment.* For the reason of field monitoring of transmission tower, it is impossible to control natural wind velocity to zero for balancing a resistor bridge of strain gauge; so only the fluctuating effect of dynamic strains

Table 4: GRF values for top displacement from acceleration measurements.

| No. | Mean wind velocity $\overline{V}_{56.9}$ (m/s) | Turbulence intensity | Displacement peak factor $g_{ex}$ | Displacement RMS $\sigma_{ex}$ (cm) | Theoretical displacement mean values $\overline{x}_t$ (cm) | Displacement GRF |
|---|---|---|---|---|---|---|
| 1 | 8.70 | 0.06 | 4.039 | 0.986 | 4.887 | 1.815 |
| 2 | 9.48 | 0.75 | 4.046 | 1.253 | 5.904 | 1.858 |
| 3 | 10.36 | 0.10 | 4.057 | 1.584 | 7.210 | 1.891 |
| 4 | 11.44 | 0.11 | 4.089 | 2.158 | 9.021 | 1.978 |
| 5 | 12.09 | 0.13 | 4.120 | 2.537 | 10.153 | 2.029 |
| 6 | 13.21 | 0.14 | 4.082 | 3.216 | 11.575 | 2.134 |
| 7 | 14.21 | 0.16 | 4.095 | 3.749 | 12.637 | 2.215 |
| 8 | 15.03 | 0.15 | 4.110 | 4.305 | 13.429 | 2.318 |
| 9 | 13.65 | 0.13 | 4.107 | 2.934 | 10.768 | 2.119 |
| 10 | 11.90 | 0.11 | 4.072 | 2.083 | 8.952 | 1.947 |
| 11 | 9.36 | 0.09 | 4.065 | 1.632 | 6.733 | 1.985 |
| 12 | 7.45 | 0.08 | 4.052 | 1.038 | 5.071 | 1.829 |
| Mean value | | | | | | 2.010 |

Figure 5: The power spectral density of top displacement: (a) without lines; (b) with lines.

has been taken into account for the measured data analysis. The power spectral density curves of the measurement strains of base-leg members are shown in Figure 6. The GRF based on the bending moment is defined by a ratio of the strain peak value versus its mean value of the main leg members at corresponding measure points. The strain peak factor and its RMS deviation can be obtained from the measurement data analysis under various wind velocities, and the mean strains of measurement points near to the tower foundation of main leg members can be calculated through the FEM from the corresponding wind velocities. As the measurement strains are obtained from the four main legs of transmission tower, those values are invariably different under the conditions of different wind velocities and directions, sometimes, even not in the same magnitude. Thus, the theoretical analysis should be consistent with monitoring conditions. Similarly, the GRF based on the bending moment can be expressed as the following formula:

$$G_\varepsilon = 1 + \frac{g_{e\varepsilon}\sigma_{e\varepsilon}}{\overline{\varepsilon}_t}, \tag{4}$$

where $\overline{\varepsilon}_t$ is the mean strain response from theoretical calculation, $\sigma_{e\varepsilon}$ denotes the RMS value of measurement strain response, and $g_{e\varepsilon}$ means the statistical peak factor derived from the response process that assumed as the random Gaussian process, of which calculation method is the same as formula (2).

Taking into account various factors including wind directions, velocities, and turbulence intensities, the representative measurement strain data of the main legs are selected, and then the GRF based on bending moment is calculated as shown in Table 5. From Table 5, it can be seen that the law of

Table 5: GRF values for base bending moment from strain measurements.

| No. | Mean wind velocity $\overline{V}_{56.9}$ (m/s) | Turbulence intensity | Strain peak factors $g_{e\varepsilon}$ | Strain RMS $\sigma_{e\varepsilon}$ | Theoretical strain mean values $\overline{\varepsilon}_t$ | Bending GRF |
|---|---|---|---|---|---|---|
| 1 | 8.70 | 0.06 | 3.479 | 6.16 | 11.239 | 2.907 |
| 2 | 9.48 | 0.75 | 3.798 | 6.86 | 13.611 | 2.914 |
| 3 | 10.36 | 0.10 | 3.771 | 8.79 | 15.856 | 3.091 |
| 4 | 11.44 | 0.11 | 3.590 | 13.72 | 23.363 | 3.108 |
| 5 | 12.09 | 0.13 | 3.737 | 14.37 | 25.795 | 3.082 |
| 6 | 13.21 | 0.14 | 3.651 | 19.95 | 32.329 | 3.253 |
| 7 | 14.21 | 0.16 | 3.725 | 22.45 | 35.694 | 3.343 |
| 8 | 15.03 | 0.15 | 3.803 | 23.59 | 37.127 | 3.417 |
| 9 | 13.65 | 0.13 | 3.698 | 14.085 | 26.045 | 3.000 |
| 10 | 11.90 | 0.11 | 3.643 | 13.88 | 23.761 | 3.128 |
| 11 | 9.36 | 0.09 | 3.821 | 7.38 | 14.134 | 2.995 |
| 12 | 7.45 | 0.08 | 3.577 | 6.59 | 12.253 | 2.923 |
| Mean value | | | | | | 3.097 |

Figure 6: The power spectral density of strain at the base bottom: (a) without lines; (b) with lines.

main dynamic strain variation which reflected the bending moment GRF is similar to that of displacement dynamic response at the tower top, but the difference is that the bending moment GRF is larger than the displacement GRF, which may be caused by the larger displacement background component and instead of the relatively smaller strain background component.

## 4. Guasi-Static Method of GRF

*4.1. Australian Load Code Method.* The simplified Holmes GRF method for the lattice tower is applied in the Australian load code [15]. This approach assumes that only the first-order modal response of tower is considered and known the random load, such as the wind power spectrum density function. GRFs for the loading effect on the bending moment and shear are consistent and varying with altitudes. Usually, the wind loading duration is adopted from 10 min to 1 h. The GRF formula can be expressed as

$$G_s = 1 + rH\sqrt{g_B^2 B_s + g_R^2\left(\frac{SE}{\zeta}\right)}, \tag{5}$$

where $r = (2/M_t)(\sigma_V/\overline{V})$, $(\sigma_V/\overline{V})$ is the turbulence intensity on the tower height $h$, $M_t$ means the terrain factor of mean wind velocity, $H$ represents the factor for height, $H = 1 + 0.2(s/h)^2$, $s$ denotes the height of the peak moment or shear, $h$ stands for the transmission tower height above ground, and $B_s$ represents the background factor, that is,

$$B_s = \left(1 + \sqrt{\frac{36(h-s)^2 + 64w_s^2}{L_h}}\right)^{-1}, \tag{6}$$

where $w_s$ is the average structural width between $h$ and $s$, $L_h = 1000(h/10)^{0.25}$, $g_B = g_u[1 + 0.925r\sqrt{B_s}]$, $g_u = 3.7$,

Table 6: GRF values calculated from various quasi-static methods.

| No. | Mean wind velocity $\overline{V}_{56.9}$ (m/s) | Turbulence intensity | AD method (based on bending moment) | Holmes method (based on bending moment) | Holmes method (based on displacement) |
|---|---|---|---|---|---|
| 1 | 8.70 | 0.06 | 2.278 | 2.269 | 2.238 |
| 2 | 9.48 | 0.75 | 2.747 | 2.700 | 2.650 |
| 3 | 10.36 | 0.10 | 3.348 | 3.204 | 3.130 |
| 4 | 11.44 | 0.11 | 4.373 | 4.378 | 4.241 |
| 5 | 12.09 | 0.13 | 4.471 | 4.521 | 4.366 |
| 6 | 13.21 | 0.14 | 4.692 | 4.762 | 4.591 |
| 7 | 14.21 | 0.16 | 4.559 | 4.618 | 4.456 |
| 8 | 15.03 | 0.15 | 3.279 | 3.246 | 3.172 |
| 9 | 13.65 | 0.13 | 3.150 | 3.116 | 3.051 |
| 10 | 11.90 | 0.11 | 2.907 | 2.861 | 2.804 |
| 11 | 9.36 | 0.09 | 2.662 | 2.608 | 2.562 |
| 12 | 7.45 | 0.08 | 2.434 | 2.382 | 2.347 |
| Mean value | | | 3.408 | 3.389 | 3.301 |

$g_R = \sqrt{2\ln(Tn)}$, $T$ denotes the duration, usually taken to be 600 s, $n$ means the nature frequency of the first order mode, and $S$ is the scale factor for resonant response, that is,

$$S = \left\{\left[1 + \left(\frac{3.5nh}{\overline{V}_h}\right)\right]\left[1 + \left(\frac{4nw_0}{\overline{V}_h}\right)\right]\right\}^{-1}, \tag{7}$$

where $w_0$ means the average structural width between $h/2$ and $h$, $\overline{V}_h$ denotes the mean wind velocity on the calculation height $h$, and $E$ represents the energy factor of gust, that is,

$$E = \frac{0.47N}{(2 + N^2)^{5/6}}, \tag{8}$$

where $N = nL_h/\overline{V}_h$ is the conversion frequency and $\zeta$ means the limit damping ratio, for the bolted transmission tower, which is $\zeta = 0.05$.

On those conditions of various turbulence intensities and mean velocities of wind, the GRF based on the bending moment is calculated by means of the quasistatic method of Australian standard and the measurement data, and those results are shown in Table 4.

*4.2. Holmes Quasistatic Method.* The Holmes quasistatic method is a GRF one that is recommended by the Australian standard, which was proposed by Holmes in 1995 for the lattice tower design [16, 17]. This method can be used for calculating the GRFs based on the bending moment or shear which is varied with tower height and independent on the displacements at the tower top. The GRF at any height $s$ of the tower is defined as the ratio of the peak value to the average, of which detailed formulas can be found in the literature [16].

*(1) The Bending Moment Gust Factor.* The bending moment GRF of the tower at any height is given by

$$G_m(s) = 1 + \frac{r\left[g_B^2 B_s F_7 + g_R^2 (SE/\zeta) F_3 F_4 F_8\right]^{1/2}}{F_6}, \tag{9}$$

where the denotations of $r$, $g_B$, $g_u$, $g_R$, $S$, and $E$ have the same meanings with the Australian standard method.

$$B_s = 2\left[\frac{{}^zL_u}{(h - s)}\right] + 2\left[\frac{{}^zL_u}{(h - s)}\right]^2\left[e^{\{(s-h)/{}^zL_u\}} - 1\right],$$
$${}^zL_u = 25\left(\frac{h}{10}\right)^{0.25}, \tag{10}$$
$$\zeta = \zeta_s + \zeta_a,$$

where $\zeta_s$ means the structural damping, and $\zeta_a$ denotes the aerodynamic damping which can be expressed as

$$\zeta_a = \left[\frac{\rho C_d \delta \overline{V}_h h}{4\pi n_1 m_0}\right] F_9, \tag{11}$$

where $\rho$ means the aerial density, $C_d$ is the resistance coefficient, $\delta$ denotes the ratio of real area, $\overline{V}_h$ represents the mean velocity at tower top, $m_0$ means the mass per unit length at the tower top, and $F_3$, $F_4$, $F_6$, $F_7$, $F_8$, and $F_9$ are the simplified formula denotations, respectively, in which detailed expressions can be found in the literature [16].

*(2) The Displacement GRF at the Tower Top.* The displacement GRF at the tower top can be calculated by the following formula:

$$G_x = 1 + \frac{r\left[g_B^2 B_0 F_{11} + g_R^2 (SE/\zeta) F_3 F_4 F_{12}\right]^{1/2}}{F_{10}}, \tag{12}$$

where when $s = 0$, the expression of $B_0$ is fully same as $B_s$; also, $F_3$, $F_4$, $F_{10}$, $F_{11}$, and $F_{12}$ are the simplified formula denotations, respectively, in which detailed expressions can be found in literature [16].

The GRFs based on the bending moment and displacements are shown in Table 6, which are calculated by the Holmes quasistatic method. From Table 6, it can be seen that

the GRF obtained from the Australian specification is consistent with the Holmes quasistatic method. It is not obvious to the analytical deviation for neither bending moment nor displacement as a computational input, in which the displacement GRF is slightly smaller, but the deviation between them is not more than 5%. In addition, compared with the aforementioned analytical results of measurement data, the bending moment GRF is approximately near to that of the quasistatic method, but the displacement GRF then marches in the opposite direction, which may be caused from the displacement background response.

## 5. Conclusions

Based on the field monitoring of transmission tower, in the parallel and orthogonal directions with the transmission line, respectively, the measurement data of fluctuating wind velocity with different turbulence intensities, various accelerations from the parts of tower, and dynamic strains of the main upright legs near the tower foundation are measured and collected. The dynamic characteristics and GRFs of the transmission tower are analyzed from the modal identification method, the FEM, and the quasistatic method, respectively. The results have been shown as follows.

(1) An approach based on multisensors on-site monitoring test, wind-induced vibration analyses, and quasistatic design for TTLS have been presented, which can provide a simplified method for TTLS design and vibration analyses.

(2) The dynamic characteristics of transmission tower determined by the modal identification approach with the measured data are consistent with those of the FEM, and the calculated deviation between them is about 5%, which shows that both analytical methods have the better compliance and validity.

(3) The GRFs from the measured data are very close to the bending moment GRFs analyzed by the two kinds of quasistatic methods, and those calculated values of the quasistatic method are slightly larger, but the deviations between them are within 5%.

(4) As for the displacement GRF, the calculated values from the measured data are less than those of quasistatic method, which may be caused by the larger background components of displacement response.

## Acknowledgments

This research work is jointly supported by the Science Fund for Creative Research Groups of the National Natural Science Foundation of China (Grant no. 51121005), the National Natural Science Foundation of China (Grant nos. 51078051, 51178083, and 51222806), and the Program for New Century Excellent Talents in University of China (Grant no. NCET-10-0287).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 454530, 7 pages
doi:10.1155/2012/454530

*Research Article*

# Research on Optimal Sensor Placement Based on Reverberation Matrix for Structural Health Monitoring

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Received 28 June 2012; Revised 28 September 2012; Accepted 29 September 2012

Academic Editor: Hong-Nan Li

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Adequate sensor placement plays a key role in such fields as system identification, structural control, damage detection, and structural health monitoring (SHM) of large-scale civil infrastructures. Many optimal sensor placement (OSP) methods have been developed for general optimized solution searches. Due to the limitations of equipment facilities and cost, the number of sensors to be installed in a structure is relatively few. It is very important to determine the necessary number of sensors to be installed and where to deploy these sensors. Taking into account energy attenuate during the signal propagation, combined with classic reverberation matrix method, a two-step method is proposed to determine the sensors arrangement using the scattering matrix in this paper. First, calculate the utmost distance of wave propagation on a special structure by the principle of elastic wave propagation and determine the preliminary number of sensors; second, the utmost distance and number of sensors are applied to a sensor optimization algorithm named Effective Independence Driving-Point Residue method. In the bridge benchmark model case study, it shows the validity of proposed method under a special detection system.

## 1. Introduction

A typical SHM system includes three major components: a sensor system, a data processing system (including the data acquisition, transmission, and storage), and a health evaluation system [1–3]. The most common used methods of data acquisition and analysis are based on vibration test (active excitation or ambient excitation). The number and location of sensors directly affect the accuracy of the identified modal parameters. Thus, the sensors must be judiciously placed in order to provide adequate information for the identification of the structural behavior. On the other hand, using too many sensors as well as accessorial facilities will also cause exorbitant cost.

In the past years, many methods have been developed for general optimized solution searches such as effective independence method [4], kinetic energy method [5], and some modern-type methods [6–10]. These methods solve the sensor optimization position through, as much as possible, information getting from the data acquisition system based on the analysis of finite element model (FEM). Another approaches range from applying the constraints on the objective function (evaluation criteria) to streamline the optimization process to applying the advanced artificial analysis techniques such as the genetic algorithms and simulated annealing.

As illustrated in [11], damage detection techniques based on modal parameters represent the "traditional" approach to the problem. This approach could appear convenient when a complete characterization of the structure is strictly required, but, in general, it may present some critical issues. For example, the modal-based identification procedures are almost never completely and often not universal, but rather specifically related to the facility under examination. Moreover these techniques are not efficient in tracing the micro damages that could early occur in the process of damage, as illustrated by Peeters and Roeck [12]: modal parameters variations can be caused both by the presence of damage, whether as a result of changes in environmental conditions. Therefore making certain optimal number of sensors and placing sensors on appropriate locations are especially important.

But up to now, a few works were studied about the optimal number of needed sensors especially for data acquisition validity and micro damage detection. In fact, the effectiveness and the amount of information are equally important to the subsequent data analysis. The energy of wave (the strength of signal) will attenuate during the propagation. On the other hand, there are additional frequency component of wave at the damage position, the higher the frequency is, the rapid the wave attenuates. So even if the sensor got the signal, it cannot be judged if there is really damage when the energy of wave does not match the requirement of data acquisition system. The results of measurement are unreliable under this condition.

Reverberation matrix method is put forward in recent years. It is an effective way to analyze the transient dynamic response of structure based on wave propagation and classic Timoshenko beam theory [13, 14]. It has been proved that there is an obvious effect of inertia due to the shear deformation at high frequencies on the dynamic properties of structure [15]. Based on reverberation matrix method, a two-step method is hereby proposed in this paper to determine the sensors arrangement using the scattering matrix in this paper. First, calculate the utmost distance of wave propagation on a special structure by the principle of elastic wave propagation and determine the preliminary number of sensors; second, the utmost distance and number of sensors are applied to a sensor optimization algorithm named Effective Independence Driving-Point Residue method. In this way, the number and location of sensors can be optimized simultaneously.

## 2. Basic Theory

*2.1. Analysis of Antinoise Capacity of Data Acquisition System.* In actual engineering, different data acquisition system required different precision of the sensor based on the system's antinoise capacity. It is needed to analyze the system's antinoise capacity in order to ensure the validity and reliability of the measurement. The noise level that the system can tolerate must be estimated and peak signal to noise ratio (PSNR) must be calculated.

It is assumed that the given instrument's measurement range is $\delta$, precision is $\mu$, so the absolute deviation threshold is $\delta\mu$. And the value $P$ of PSNR is

$$P = 10 \times \lg\left(\frac{a_{\text{Sig}}^2}{a_{\text{Noi}}^2}\right) = 10 \times \lg\left(\frac{a_{\text{Sig}}^2}{(\delta\mu)^2}\right). \tag{1}$$

$a_{\text{Sig}}$ represents acceleration amplitude of signal, $a_{\text{Noi}}$ represents acceleration amplitude of noise.

According to (1), the needed signal minimum amplitude of acceleration $a_{\min}$ can be calculated:

$$|a_{\min}| = 10^{0.05P}\delta\mu. \tag{2}$$

*2.2. The Utmost Distance of Sensor Arrangement.* The lateral motion equation of modification Timoshenko beam taking account of the shear displacement is:

$$\begin{aligned} \kappa AG\frac{\partial^2 v_s}{\partial x^2} &= \rho A\frac{\partial^2(v_b+v_s)}{\partial t^2}, \\ \kappa AG\frac{\partial v_s}{\partial x} &= \rho I\frac{\partial^3 v_b}{\partial x\partial t^2} - EI\frac{\partial^3 v_b}{\partial x^3}, \end{aligned} \tag{3}$$

where $\kappa$ is section shear coefficient, $A$ is the area of section, $G$ is shear module, $\rho$ is density, $I$ is the moment of Inertia. $v_s$, $v_b$ are lateral displacement caused by shear and moment, respectively, and the lateral displacement $v(x,t) = v_s(x,t) + v_b(x,t)$.

After Fourier transformed, the solutions of (3) are

$$\begin{aligned} \hat{v}_b(x,p) &= a_1(p)e^{s_1x} + a_2(p)e^{s_2x} \\ &\quad + d_1(p)e^{-s_1x} + d_2(p)e^{-s_2x}, \\ \hat{v}_s(x,p) &= \alpha_1 a_1(p)e^{s_1x} + \alpha_2 a_2(p)e^{s_2x} \\ &\quad + \alpha_1 d_1(p)e^{-s_1x} + \alpha_2 d_2(p)e^{-s_2x}, \end{aligned} \tag{4}$$

the " ^ " indicates a variant in the frequency domain, $a_1$, $a_2$, $a_3$, and $a_4$ are undetermined constants depend on arrival wave, $d_1$, $d_2$, $d_3$, and $d_4$ are undetermined constants depend on departure wave, $p$ is the Laplace transform complex variable, $e$ is 2.718 28, $k = (\omega/c_1)\sqrt{((1+\eta_1)+(-1)^{j+1}\sqrt{(1+\eta_1)^2-4\eta_1-4c_1^2A/(I\omega^2)})/2}$ represents the number of wave, $c_1 = \sqrt{E/\rho}$ is the speed of wave, $\alpha_{1,2} = I(\rho p^2 - Es_{1,2}^2)/AkG$, $\eta_1 = E/\kappa G$, $\omega$ is frequency, $j$ is 0 or 1, and $s_{1,2} = (p/c_1)\sqrt{((1+\eta_1)\pm\sqrt{(1+\eta_1)^2-4\eta_1-4c_1^2A/(Ip^2)})/2}$.

The moment and shear can be expressed as follows in the frequency domain:

$$\begin{aligned} \widehat{M} &= EI\frac{d^2\hat{v}_b}{dx^2}, \\ \hat{V} &= i\kappa GAk_1\alpha_1\left[a(\omega)e^{-ik_1x} - d(\omega)e^{ik_1x}\right]. \end{aligned} \tag{5}$$

So the solutions of (5) are

$$\begin{aligned} \widehat{M} &= EI\{s_1^2[a_1(p)e^{s_1x} + d_1(p)e^{-s_1x}] \\ &\quad + s_2^2[a_2(p)e^{s_2x} + d_2(p)e^{-s_2x}]\}, \\ \hat{V} &= kGA\{s_1\alpha_1[a_1(p)e^{s_1x} - d_1(p)e^{-s_1x}] \\ &\quad + s_2\alpha_2[a_2(p)e^{s_2x} - d_2(p)e^{-s_2x}]\}. \end{aligned} \tag{6}$$

In reverberation matrix method, node scattering matrix is defined as $S^{X_i}$, the departing wave amplitude matrix $d^{X_i}$ can be calculated by arriving wave amplitude matrix $a^{X_i}$ at node $X_i$:

$$d^{X_i} = S^{X_i}a^{X_i} + s^{X_i}, \tag{7}$$

Figure 1: Benchmark model of bridge structure and its finite element model (FEM).

Figure 2: Wave acceleration peak value attenuation.

where $S^{X_i}$ and $s^{X_i}$ are local scattering matrix and local source matrix. The $a^{X_i}$ and $d^{X_i}$ are local arriving wave amplitude matrix and local departing wave amplitude matrix separately. For the whole structure, it is

$$d = Sa + s, \tag{8}$$

where $S = -d^{-1}a$ is global scattering matrix, $s$ is global source matrix, and $a$ and $d$ are global arriving wave amplitude matrix and global departing wave amplitude matrix. The global scattering matrix $S$ is a constant matrix depends on special structure. Equation (8) reflects the scattering relationship between departing wave amplitude and arriving wave amplitude under external force.

The $x$ in (6) represents the distance of wave propagation. The data acquisition system can correctly identify the signal only if the peak value of wave signal is greater than $|a_{\min}|$. The attenuation degree of peak value of response signal can be determined by matrix $S$. When it satisfies $S^{D_{\max}}a = |a_{\min}|$, $D_{\max}$ becomes the utmost distance of wave propagation, that is, the utmost distance of sensor arrangement.

*2.3. Effective Independence Driving-Point Residue (EFI-DPR) Method.* As mentioned above, previous works addressing the issue of optimally locating a given number of sensors in a structure have been carried out by several investigators. And the comparisons of different sensor placement methods have been done [2]. Because this paper mainly discusses the number of needed sensors taking into account data acquisition validity and micro damage detection sensitivity, so we just select a common method to optimize the location of sensors. In fact, a method named Effective Independence Driving-Point Residue (EFI-DPR) method is adopted in the aspect of location optimizing process.

The effective independence method (EFI method) was proposed by Kammer at 1991 [4]. The main idea of the EFI method is to gradually eliminate the degree of freedom which contributes the smallest to linearly independent of target mode vector from all measured points and to achieve the goal of that as much as possible modal parameters information is collected with limited number of sensors. The elimination process is implemented by the computing of effective independence distribution vector $E_D$:

$$E_D = \Phi\left[\Phi^T\Phi\right]^{-1}\Phi^T, \tag{9}$$

where $\Phi$ represents the modal shape matrix of reduction finite element model (FEM) of the structure to be detected based on all candidate measurement points.

The other optimal sensor placement method is Kinetic energy method (KEM). The basic idea of KEM is that the node's response is relatively large with larger modal strain energy, and then the sensors are installed on these nodes according to modal strain energy distribution map of every modal shape [16].

The step to optimize sensors location of KEM is similar to effective independence method (EFI), but the KEM method is highly dependent on the mesh level of finite element model. The sensor will be distributed more far away if the mesh is divided more coarsely, vice versa.

The definition of Kinetic energy matrix $K_E$ is

$$K_E = \Phi^T M \Phi. \tag{10}$$

A residue coefficient $C_{\mathrm{DPR}}$ of Effective Independence Driving-Point Residue (EFI-DPR) method is defined based on the theory of Kinetic energy:

$$C_{\mathrm{DPR}_i} = \frac{K_E}{K} = \frac{\Phi^T M \Phi}{K} = \frac{\Phi^T \Phi}{\omega^2} = \sum_{j=1}^{n} \frac{\Phi_{ij}^2}{\omega_{ij}^2}, \tag{11}$$

Table 1: Material parameters of structure.

| $E$ | $\rho$ | $G$ | $I$ | $\kappa$ | $\eta$ |
|---|---|---|---|---|---|
| $2.08e11$ | 7800 | $7.6923e+010$ | $1.04e-6$ | 3.8747 | 0.06 |

where $K$ represents stiffness matrix of structure, $K_E$ represents unit stiffness modal kinetic energy, $M$ represents mass matrix, $\omega$ is frequency, and $n$ is the total number of nodes of structure.

To multiply (9) by $C_{\mathrm{DPR}}$, the independence distribution vector of EFI-DPR is shown in

$$E_{\mathrm{DPR}} = \Phi\left[\Phi^T\Phi\right]^{-1}\Phi^T C_{\mathrm{DPR}}. \tag{12}$$

The optimal process of EFI-DPR method is similar to the classic effective independence method. The performance of EFI-DPR becomes a little bit excellent through combining the advantages of the two traditional methods (EFI and KEM) [17]. (For more information, please see the reference [17].)

## 3. Process of Optimal Sensor Placement

*3.1. The Optimal Minimum Number of Needed Sensors.* First, utmost distance $D_{\max}$ of sensor arrangement is calculated based on the theory of elastic wave propagation. Then the minimum number $N$ of sensors needed is estimated by $N = (L/D_{\max})$, where $L$ is the longest size of the structure ($L$ is not the size only in one dimension, it means the longest path of wave propagation through the whole structure.) and it is assumed that the sensors are arranged uniformly at first.

*3.2. The Optimal Position of Sensors.* First, the optimal location of sensors is obtained using Effective Independence Driving-Point Residue (EFI-DPR) method according to the $N$. Then the distance between every two sensors will be checked after the sensors are placed by EFI-DPR. The number of sensors will be added if there is any distance longer than the ultimate distance $D_{\min}$ mentioned above. This process will continue until all the distances are satisfied. The final optimal sensor placement plan will be get after the iteration is finished.

## 4. Case Study

*4.1. Model Analysis and Utmost Distance Calculation.* This paper considers the optimal sensor placement for a bridge benchmark model structure as shown in Figure 1. The length, width, and the height of model are 12 m, 2 m, and 1 m separately. All beams of the structure are joist steel, the elastic modulus is $2.08e11$ N/m$^2$, mass density is 7800 kg/m$^3$, and Poisson's ratio is 0.31.The parameters are shown in Table 1. All the joints are connected by bolts. The nine columns are fixed on the ground. The data acquisition system is Intelligent Signal Acquisition Analyzer INV306U-5160 produced by Beijing Orient Institute of Noise & Vibration. Its measurement range $\delta$ is $\pm 10$ m/s$^2$, precision $\mu$ is 0.001. According to (2), the needed minimum signal acceleration peak value is that $|a_{\min}| \geq 0.0056$ m/s$^2$ (the $P$ is $-5$ db according to the statistics result).

Figure 3: Optimal sensor placement of Benchmark model.

Figure 4: Values of mean square deviation (MSD).

In fact, there are various frequency components of wave at the damage position due to different damage type. According to the results of simulation of the damage due to stiffness decrease, a wave with 2000 Hertz and 0.13 m/s$^2$ acceleration was taken as an example data. The attenuation curve of wave is shown in Figure 2.

The utmost distance of sensor arrangement was calculated using the proposed method. The results indicate that the distance of wave propagation is 5.86 meters when the peak value of acceleration attenuates to 4.3% of the origin (from 0.13 m/s$^2$ to 0.0056 m/s$^2$). So the utmost distance of sensor arrangement is 5.86 meters.

*4.2. Final Placement Plan.* Based on the utmost distance $D_{\max} = 5.86$ m, the optimal sensor placement of the bridge Benchmark model was accomplished using EFI-DPR method with the given data acquisition system. The result is shown in Figure 3. (The arrows in Figure 3 represent the position and direction of the sensors.)

*4.3. Results Analysis.* In order to check the validity of the proposed method, the values of mean square deviation (MSD) and modal assurance criteria (MAC) between FEM and the measured data from the installed sensors are calculated. Results are shown in Tables 2 and 3 and Figures 4 and 5.

Table 2 shows that the mean square deviation (MSD) value is very small at the first several modes. Table 3 shows the value of modal assurance criteria (MAC) is close to 1 at the first three modes, six-order mode, and seven-order mode. MAC is a very good common tool for evaluating

Figure 5: Values of modal assurance criteria (MAC).

Table 2: Mean square deviation (MSD) value between FEM modal shape and curve fitting of test.

| Modal number | MSD |
|---|---|
| 1 | $3.271e-003$ |
| 2 | $9.5926e-003$ |
| 3 | $1.0417e-002$ |
| 4 | $1.7234e-002$ |
| 5 | $2.1884e-002$ |
| 6 | $2.2530e-002$ |
| 7 | $2.3381e-002$ |
| 8 | $2.7432e-002$ |
| 9 | $3.0603e-002$ |
| 10 | $3.1539e-002$ |

Table 3: Modal assurance criteria (MAC) value between FEM modal shape and curve fitting of test.

| Modal number | MAC |
|---|---|
| 1 | 0.9922 |
| 2 | 0.9468 |
| 3 | 0.9863 |
| 4 | 0.8633 |
| 5 | 0.8426 |
| 6 | 0.9771 |
| 7 | 0.9680 |
| 8 | 0.8544 |
| 9 | 0.8629 |
| 10 | 0.9534 |

the orthogonality of two vectors, and in general, the data of first several modes is enough for ordinary test.

At the same time, the mode shape comparison has been made between the simulation of FEM and the data of curve fitting based on the measured data from the installed sensors (the dashed line in Figure 6). Figure 6 shows that there is no significant difference between the mode shape of FEM and the one by curve fitting.

All the results show that the information measured from the installed sensors was consistent with the finite element model (FEM) at the first 1 to 6 modes, that is, the optimal sensor placement with proposed method is number enough and effective under the given data acquisition system.

## 5. Conclusions

This paper presents an alternative way to make effective optimal sensor placement especially for data acquisition validity and micro damage detection in structure health monitoring. Taking into account energy attenuate during the signal propagation, combined with classic reverberation matrix method, a two-step method is proposed to determine the sensors arrangement. The optimal number and location of sensors are calculated simultaneously through applying the utmost distance of sensors to a sensor optimization algorithm. The bridge benchmark model test is representative and the results show the validity of proposed method.

Although the study provides an alternative way for optimal sensor placement in structure damage detection and structure health monitoring, and this may make the system more economical and effective, there is still much more work need to do. The establishment of scattering matrix $S$ is more cumbersome. There are different $S$ for bar system and plate system. A little bit variation of $S$ exists even in the similar structure. On the other hand, more damage types must be considered for the selection of excitation wave in

Figure 6: Modal shapes between finite element model (FEM) and cubic spline.

large structures. And how to determine the order of the monitoring modes still needs further research.

## Acknowledgments

This research work was jointly supported by the National High Technology Research and Development Program ("863" Program) of China (Grant no. 2006AA04Z437) and the National Natural Science Foundation of China (Grant nos. 51278420 and 50878184). The authors would also like to thank the anonymous reviewers for their valuable suggestions.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 917849, 10 pages
doi:10.1155/2012/917849

*Research Article*

# A Flexible Network Structure for Temperature Monitoring of a Super High Arch Dam

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Received 29 June 2012; Revised 9 September 2012; Accepted 11 September 2012

Academic Editor: Hong-Nan Li

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The aim of the study presented in this paper is to develop a flexible network structure for temperature monitoring of a super high arch dam under construction period. The multiple channel temperature acquisition method collects and analyzes system including flexible and stable field bus for the sensors, communication between intelligent module and control unit is proposed. In this temperature monitoring system, a total of 3 kinds of networks which independently marked by *(1), *(2), and *(3) were proposed, with *(1) being the lowest priority and *(3) being the highest priority. The lowest priority is field bus (ITU Bus) which connects all the grouped sensors into different channels of different intelligent acquisition modules. The ITU network protocol is a star type, and the network structure is independent on the sensors. The bus can connect different types of sensors such as strainmeter, stressmeter, jointmeter, rock-deformeter, piezometer et al., the measurement scope could be extended wider. This study provides a measuring technique has been successfully implemented in monitoring of the Xiluodu arch dam construction, and ultimately solved the shortcoming of manual measurement technique.

## 1. Introduction

In recent times, sensory technologies have been utilized in monitoring health of structural engineering [1–5]. The sensors utilized in a super arch dam structural health monitoring system (DSHMS) are required to monitor not only the dam structural status including stress, displacement, and acceleration but also the affecting environmental parameters, such as wind speed, temperature, and seepage properties of foundation, especially in dam construction [2, 5, 6]. The DSHMS is a large sensory network that is deployed on a spatially extended structure so that the data acquired from the selected locations will result in the most optimal identification of the dam's structural characteristic changes.

A super high arch dam is divided into many blocks by contraction joints in the construction period. Heat of hydration develops in fresh concrete, associated with the exothermic chemical reactions of the cement [7]. Due to the poor conductivity of concrete, high temperature gradients may occur between the interior and the surface of the structural elements or between parts of an element with sequential concreting phases [8]. The temperature gradients may lead to tensile stresses and excessive tensile strength will result in the young concrete to crack [9]. A preemptive action can be taken to maintain the concrete temperature at the preferred design temperature, which in turn satisfies construction procedures and quality long-lasting lifetime. The other factors that play a major role are the heat of hydration temperature as a function of the construction plan and the quality of the materials used. Up to now, the proper curing methods are essentially empirical and rely on the experience of the contractor who usually considers the precooling (warming) of the materials and the use of cooling tubes, covering the insulation material or low-heat cements. For example, the 186 m high Kurobe Dam, a dome-type arch dam located at the Kurobe River, Japan, was completed in June 1963 [10]. Temperature evaluation requires not only thermometer readings but also theoretical calculations. The temperature data was collected by using vibrating cord thermometers and electrical resistance thermometers, which lacked in durability and precision. The Ertan arch dam is 240 m and is located in the Yalong River of China. The temperature of the concrete was measured by means of 3 sets of dial readouts using sufficient

Figure 1: The schematic of temperature acquisition, and collecting, analyzing system.

portable resistance thermometers, and maximum-minimum thermometers embedded in concrete [11]. The contractor provided all necessary assistance in obtaining weekly reports of the placing temperature of the concrete, direction of flow, pressure, and the temperature of the cooling water at the inlet and outlet of each individual cooling coil. Final average temperature checks were obtained by stopping the circulating water in an individual cooling coil for 96 hours and then measuring the water temperature. The 292.5 m Xiaowan arch dam was constructed in 2010 at the Lancang River of China [12]. For measuring internal-concrete temperature, the contractor buried infrared thermometers, and read data manually, since the existence of some errors in the concrete temperature leads to the occurrence of cracks in the dam body [13].

Above all, the current temperature data collection methods and control of manual records are very rudimentary due to thier poor accuracy and lack of data reliability. The main factors of accurate temperature control are real-time measurement, accurate collection of the real dam internal temperature data, inlet and outlet water temperature of the cooling pipe, and weather temperature data. Thus, development of a wall-mounted multisite environment temperature monitoring system for the super high arch dam has an important significance. In this paper, the methodology of temperature measurement based on the flexible network structure for temperature monitoring of super high arch dam is discussed in detail. The method entails collecting multiple channel temperature, analyzing system including flexible and stable field bus for the sensors, and communicating between intelligent module and control unit. An application case of temperature monitoring for the Xiluodu arch dam under construction period is discussed in detail.

## 2. Methodology and Technology

*2.1. Overview of Instrumentation System.* This system focuses mainly on providing a real-time, multipoint temperature logger that can automatically capture, transfer, and analyze the internal temperature distribution characters of water in the cooling pipe and mass concrete. Figure 1 shows a schematic of temperature acquiring, collecting, and analyzing system, including. (1) Digital temperature measurement devices installed in each inlet and outlet of the cooling pipe; see Figure 2(a); (2) digital thermometers embedded into the concrete block were used to detect internal temperature distribution of fresh concrete elements; the sensor is designed for underwater use and has a coaxial cable armored by stainless steel wires, so that it can be directly embedded in concrete blocks; see Figure 2(b). The temperature range of this sensor is approximately $-10^{\circ}$C–$60^{\circ}$C, working within an accuracy of $\pm 0.3^{\circ}$C. (3) Multichannel temperature logger for data acquisition was collected based on flexible network structure, shown in Figure 3. (4) Control server, the receiver temperature logger data.

*2.2. Flexible Network Structure.* It is very important to read the values from the several hundred sensors covering all pouring blocks and analyze them via computer temperature model system. A detailed solution method for the acquiring, collecting, and analyzing system of the temperature sensors on the dam site is detailed in the following.

In this temperature monitoring system, a total of 3 kinds of networks which are independently marked by *(1), *(2), and *(3) were proposed, with *(1) being the lowest priority and *(3) being the highest priority. The lowest priority is field bus (ITU bus) which connects all the grouped sensors into different channels of different intelligent acquisition modules. The intelligent modules are the interface between sensors and the PLC (Programmable Logic Controller) control system, which communicate with PLC via standard RS485 network (the medium network level marked by *(2)). The PLC polls all the modules in a certain time interval and gets updated temperature values circularly; the protocol is a standard RS485 ASCII. Using industrial Ethernet network (marked by *(3)) and monitoring PC, the data is collected from the PLC and stored in PLC data storage database, which in turn can be accessed by a remote PC via internet connection. For dam temperature control through internet or intranet router, the authorized engineer could make the right analysis or conclusion in real time.

*2.3. Flexible and Stable Field Bus for the Sensors.* The basic requirement for monitoring the dam temperature is to continuously collect and record values of the temperature measured by the sensors using a stable, fault tolerated field bus that must be used on site. The ITU bus is developed on the basis of 1-wire protocol (trade mark of Dallas Semiconductor) and the purpose is to communicate over one single twisted-pair cable. This network is defined with an open-drain master/slave multidrop architecture with resistor pullup to a nominal 5 V supply at the master. It is built up

(a) Digital internal temperature measurement devices of cooling pipe

(b) Digital thermometer for internal temperature measurement of mass concrete

Figure 2: Digital temperature measurement devices and digital thermometers.

Figure 3: Network structure for the temperature control system.

of 3 components: bus master, cable, and multiple slaves. The protocol can be described as follows.

(1) Using conventional CMOS/TTL logic levels (maximum 0.8 V for logic "zero" and a minimum 2.2 V for logic "one") with operation specified over a supply voltage range of 2.8 V to 6 V. (2) Both master and slaves are configured as transceivers permitting bit sequential data to flow in either directions, but only one direction at a time (half duplex); master initiates and controls all devices. (3) Data is byte sequential and bit sequential. Reading and writing least significant bit (LSB), first signal is transferred in time slots. (4) System clock is not required; each slave is self-clocked by an internal oscillator synchronized to the falling edge of the master. (5) Each 1-wire slave has stored in ROM a unique 64-bit serial number that acts as its node address, so the devices can be individually selected from among many that are connected to the same bus wire.

Data inquiring sequence and operations shown in Figure 4 include the following. (1) When the master polling sequence is triggered, the master first sends an inquiring command whose header contains the unique ID of slave 1; (2) the 1st slave is activated when the ID is called in the network, which will send the actual data to the master; (3) if the master finishes reading the data successfully, the 1st slave will stop the data transmitting at the falling edge of the master's reading action; (4) the master resets the slave devices and the slaves are ready for the next command.

The ITU bus is a key component utilized in this system and the most important flexible features are the following.

(1) The usual control cable acts as the communication carrier. The sensors (type: DS18B20) are adhered on the bus which supply the power to the sensors and fetch the signals from the sensors at the same time. The cable length can be extended as long as 200 meters. Briefly one single cable carries power and signals together; each cable is named as one channel, and each intelligent module can deal with 8 channels.

(2) The maximum sensors on one single cable is 64 (so one intelligent module is able to handle $8 \times 64 = 256$ sensors), and the sensors are distinguished from each other using a sensors coding system. Each sensor has its own unique ID which is read by the intelligent module. The module will sort the temperature values according to the IDs on the bus before it is sent to PLC. The main advantage of this architecture is that there are no drawbacks if some sensors are out of service (fault or removed).

(3) The bus can also connect other types of sensors such as strainmeter for measuring the strain the dam is under, stress meter for measuring the compressive stress the dam is under, jointmeter for detecting the opening of joints, rock deformeter, and piezometer. Moreover, the measurement scope of the contacts designed for this network specification could be extended wider. Figure 5 shows how the sensors connect with the ITU bus.

(4) The ITU network protocol is a star type. The master can access each slave directly. With global unique ID, each slave joins or quits the network freely taking into account the users' demand. One master can manage from 1 to 256 slaves including pressure transducers, stress meters, and flow. Since the unique ID is recognized by the master, it is possible that the program sorts the various values from filed sensors.

(5) The network structure is independent from the sensors. If one or several previously linked sensors failed, the master can still read data from other sensors. After replacement, the sensors can be accessed again, as the master collects the data on the whole network repeatedly.

*2.4. Communication between the Intelligent Module and Control Unit.* Using the standard industrial protocol, the intelligent module can easily interchange data with different control units such as PLC, MCU (Microcontrol Unit), or even PCs. Figure 6 illustrates how one intelligent module communicates with Simatic S7-300 CPU, the communicating processor on PLC side is CP 341-RS 422/485, and the protocol is ASCII driver which has a baud rate of 9600 bp, and a half duplex (RS485) two-wire mode interface.

PLC recognizes up to 128 modules which in turn connects many (maximum 256 for each module) sensors and captures them cycle by cycle. The PLC side is to poll all the modules/channels/sensors in predetermined order, identify all the modules/channels/sensors by unique IDs, and target each module/channel to get a response; this architecture can service 256 different temperature values in one module (32 sensors $\times$ 8 channels) in 12 seconds using an inquire/answer method.

Figure 7 shows the PLC's polling sequence, where there are 2 circularly actions: the inner cycle is for the 1st to 8th channels inside one single intelligent module and the outer cycle is for the 1st to maximum number 128 intelligent modules which are connected in one RS485 network. Theoretically, one Simatic CP341 module can access $32 \times 8 \times 128 = 32768$ sensors, which exceeds the requirement for temperature measurement of a super high arch dam.

The intelligent module has some powerful ASCII interface. For example, when PLC sends the command "−000CR" (Hex codes are 2A 30 30 30 0D), PLC inquires all the sensors' values of channel 0, module 0; subsequently the intelligent module will respond ">0032, ***...***" (Hex codes are likely 3E 30 30 33 32 ***...***); here the symbol "*" means the different values read out from the sensors: ">" means the inquiry is valid, "00" is the module address, and "32" means a total of there is 32 sensors in the channel. The PLC will acquire all the wanted valid values.

## 3. Case Analysis

*3.1. Outline of the Xiluodu Super High Arch Dam.* The Xiluodu hydroelectric power station [14, 15] is the second largest power generation outputting 13.86 million kW which is close to the Three Gorges hydroelectric power station in

Figure 4: The protocol schematic of ITU bus.

Figure 5: The link method for a maximum of 265 sensors in one intelligent module.

Figure 6: RS485 network for Simatic CP341 and 128 intelligent modules.

China. The project is located at the Jinshajiang River, in Leibo Country of Sichuan province. The principal structures consist of a double-curvature arch dam with a height of 285.5 meters, crest length of about 700 meters, spillway, underground powerhouse, and logway. Both sides of the arch dam are very steep with a good slope surface. Figure 8 illustrates a downstream view of the Xiluodu arch dam under construction.

*3.2. Requirement of Concrete Temperature Control.* Considering the arch structure of the stress and construction control difficulties, the basic control principle is ultimately small temperature difference, slow cooling, and early cooling [15]. The temperature control of pouring and cooling should satisfy the following requirements.

The original temperature of concrete for the Xiluodu arch dam was maintained at 7°C in hot seasons (from May to September) and 9°C in cold seasons (from October to April). The temperature of concrete upon entry into the silo was capped at 11°C in cold seasons and 9°C in hot seasons. The placement temperature was capped at 12°C all year round. The maximum concrete temperature was capped at 27°C with a tolerance of 1∼2°C, which is to take in to consideration the abnormal hot summer days. After the concrete was detached from the foundation constraint zone, the concrete temperature in the foundation constraint zones on the sections of the dam on both banks was capped at 25°C.

The cooling process for controlling the temperature of concrete at Xiluodu was carried out in three stages including the first stage being the medium phase and the second phase. Considering temperature changes due to body of the dam exposed to water, the two cooling supply systems were running at 8∼10°C and 14∼16°C, respectively. In the first stage, cooling was supplied for about 21 days at a stretch, where the maximum temperature of concrete was less than 27°C. The temperature reduction and cooling process lasted more than 45 days, 90 days and 120 days at a stretch for the medium phase, the second phase, and joint grouting phase, respectively. Table 1 lists the target temperature of dam concrete during medium-term cooling period. During the temperature control stage, concrete temperature in all phases was dropped down as close to the target temperature as possible, and temperature rebound was to be minimized.

The cooling process should be based on the temperature measurement results taking into consideration the concrete partition, pouring temperature, water pipes, density, and ambient temperature. The flow is adjusted in a timely manner to meet the targeted temperature requirements.

*3.3. Discussion on Temperature-Related Measurements.* During the construction period, digital thermometers were embedded in the dam to monitor temperature change of concrete blocks. Figure 9 is an illustration of 3D maximum temperature distribution of the Xiluodu arch dam under construction. The grey section represents the concrete blocks which will be poured later; meanwhile the color blocks means the temperature distribution across the dam. The results show that the maximum temperature of about 3.5 percent concrete blocks is greater than 27 degree which is the designed temperature extreme value.

The results of dam section 25–32, at EL.483 m, were discussed as a case. Nine thermometers were embedded

Figure 7: The PLC sequence of polling temperature values in various modules.

in spindle mode as seen in Figure 10 in dam section 25–32 (the $x$ axis points to the downstream direction). Internal temperature of dam concrete and entrance or exit temperature of cooling pipes were measured in real time with data collected two seconds apart.

Temperature fluctuation curves of cooling water were shown in Figure 11. The block's (dam section 25–32) average temperature change with time is shown in Figure 12. Figure 13 shows real temperature distribution field of dam section 25–32 with respect to different ages of concrete. These figures identify the following characteristics.

(1) In different temperature control phases, cooling water of different temperatures flows through concrete, absorbing heat generated by hydration in fresh

Table 1: The target temperature of dam concrete during medium phase of cooling.

| Position | Month | Joint grouting temperature | Dam section | |
|---|---|---|---|---|
| | | | Approximately 7–22 | Approximately 1–6, approximately 23–31 |
| Constraints zone | Approximately January–December | 12°C, 13°C | 16°C | 16°C |
| | | 14°C, 16°C | | 18°C |
| Free zone | Approximately January–March, November–December | 12°C, 13°C | 16°C | |
| | | 14°C, 16°C | 18°C | |
| | Approximately April–October | 12°C, 13°C | 16°C | |
| | | 14°C, 16°C | 18°C | |

Figure 8: The downstream view of the Xiluodu arch dam under construction.

Figure 9: The maximum temperature distribution of the Xiluodu arch dam under construction.

Figure 10: Schematic of thermometers distribution in dam section 25–32.

Figure 11: The entrance/exit of cooling pipe temperature curve of cooling pipe of dam section 25–32.

Figure 12: Concrete average temperature curve line of dam section 25–32.

concrete. Temperature gap between inlet and outlet of cooling pipe ranges from 0 to 3 centigrade.

(2) In order to ensure that concrete can be cooled evenly, the direction of flow is changed once daily, which could lead to periodic fluctuation of entrance and exit temperature of water as illustrated in Figure 11. Amplitude of the fluctuation can reach 2 centigrade.

(3) Block temperature of dam section 25–32 run in accordance with the designed cooling process. Temperature monitoring result shows that data is being obtained accurately and consistently. The temperature field distribution results, see Figure 13, show that, under concrete temperature increasing stage, the highest temperature mainly concentrates close upstream and downstream surface; see Figure 13(a).

(a) February 12th

(b) February 25th

(c) March 25th

(d) April 28th

(e) March 9th

(f) May 17th

(g) June 9th

(h) June 22nd

Figure 13: Temperature distribution field of dam section 25–32 in different cooling periods. (a) Temperature control stage in the first phase of concrete cooling; (b) temperature dropping stage of the first phase of concrete cooling; (c)~(h) temperature control stage in the middle phase of concrete cooling.

Large temperature gradient exists in arch axis zone and near upstream/downstream zone. As the concrete ages, cooling water technique takes away only some heat from accumulated hydration heat. Internal temperature field in the concrete also changes gradually. Internal temperature field of block transits gradually from the earlier spindle distribution to uniform distribution.

(4) After the concrete placement, the highest temperature in the section occurred about approximately 6–10 days later and is controlled below 27 centigrades

generally as shown in Figure 12. Concrete consists of different materials poured in different positions and different seasons, which require different times to win the highest temperature. Boundary temperature of the upstream and downstream faces approximately exhibits a constant value. The concrete surface warm-protection layer functions as a good thermal insulation. During summer and winter, temperature of the upstream and downstream surface can be 15 and 12 centigrades, respectively.

The monitoring results display that the flexible network structure was proven to be very useful for temperature monitoring of high arch dam under construction.

## 4. Conclusions

A flexible network structure for field temperature monitoring of a super high arch dam under construction period was developed based on the state-of-the-art intelligence control technology.

By employing flexible network structural architecture, the bus can also connect other types of sensors such as strainmeter for strain of dam, stress meter for compressive stress of dam, jointmeter for opening of joints, rock deformeter, and piezometer for seepage flow, or even contacts, which are designed for this network specification. Thus, the measurement scope can be utilized in a wider spectrum and in turn suffice temperature measurement requirement of a super high arch dam.

In this temperature monitoring system, a total of 3 kinds of networks which are independently marked by *(1), *(2), and *(3) were proposed, with *(1) being the lowest priority and *(3) being the highest priority. The lowest priority is field bus (ITU bus) which connects all the grouped sensors into different channels of different intelligent acquisition modules. The ITU network protocol is a star type, and the network structure is independent from the sensors.

A flexible network structure for temperature control system overcomes the shortcoming of manual measuring, recording, analyzing, and controlling, and, in turn, saves time, acquires high quality and accurate measurement, and responds with adjustment settings in a timely manner. The other benefits of utilizing real-time temperature precision control are mitigation of concrete cracking, upholding quality of mass concrete, and, as a result, progressing construction smoothly.

In conclusion, the flexible network structural architecture was proven to be very useful for the temperature control of high arch dam under construction. The proposed temperature monitoring method will be beneficial for the design and construction of controlling thermal stress of mass concrete in similar projects.

## Acknowledgments

This work was supported by the National Natural Science Foundation of China (no. 11272178), the National Basic Research Program of China (973 Program) Grant no. 2011CB013503, the Tsinghua University Initiative Scientific Research Program, the National Excellent State Key Laboratory Foundation of China (50823005), and the Special Foundation for Public Welfare Industry of the Ministry of Water resources (2010001035).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 164527, 15 pages
doi:10.1155/2012/164527

*Research Article*

# Mobile Multi-Agent Evaluation Method for Wireless Sensor Networks-Based Large-Scale Structural Health Monitoring

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Received 29 June 2012; Accepted 29 September 2012

Academic Editor: Hong-Nan Li

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Wireless sensor networks are increasingly adopted both in aircraft and civil structural health monitoring. However, due to the limited resources on a wireless sensor node, structural evaluation ability of the wireless sensor network is also limited, especially for large-scale structures. This paper presents a multi-agent cooperation system based on mobile agent technology for wireless sensor network-based structural health monitoring which deals with the evaluation of large-scale structure. Mobile agent with the signal processing or evaluation algorithm is designed to migrate among different sensor nodes and perform network self-organization, data processing, information fusion, and damage estimation. This method can reduce the request to the sensor nodes and achieve a better monitoring and evaluation performance through multiagent cooperation. The proposed mobile multiagent system is evaluated by a large dimension aerospace aluminum structure monitoring experiment in which strain distribution and joint failure are monitored by different sensors. Experimental results show that the system can significantly improve the monitoring performance for large-scale structures.

## 1. Introduction

Structural health monitoring (SHM) is an active area of research and practice in recent years [1–4]. When implementing a cable-based SHM system for large engineering structures, massive interconnections from sensors to the centralized data server require complex configurations of hardware systems which cause the heavy weight of the system and the decreasing of system reliability. To solve above problems, plenty of literatures have been published regarding applying wireless sensor network (WSN) technology to SHM, including various nodes development, such as strain, impedance, acceleration and active monitoring nodes [5–10], energy harvesting, or recharging WSN node methods [11, 12], evaluations of the WSN-based SHM systems on bridges and aircraft structures [13–17]. However, because of the limitations in terms of power, bandwidth, memory size and reliability, extensive research of WSN-based SHM technology is still being conducted.

To realize WSN-based SHM (WSN-SHM) system for large-scale engineering structures, the challenges include how to effectively manage the distributed sensor network in distributed positions and coordinate and fuse different information obtained by different sensors. Multiagent cooperation methods (MACMs) have been presented as a promising method to manage large-scale, complex, and distributed system, which is loosely coupled and heterogeneous [18].

Some literatures have been reported on the research of multiagent system (MASs) based SHM research. In 2002, NASA's Science and Technology Information Program (STI) was reported as a research on nonlife aerial vehicles using self-agent theory. The research focused on the coordination of multiagent system [19]. Esterline et al. put forward a MAS method for the vehicle health management system. They presented a contract net protocol-based method for the distributed problem-solving and implemented the MAS system using JADE platform [20]. The above researches have paid little attention to MAS for WSN-SHM. Yuan et al. initially proposed the distributed SHM system based on WSN-MAS. The detailed design and implementation of a WSN-MAS SHM system were given for a composite structure [21]. Zhao et al. presented a complete design method for

SHM based on MAS (SHM MAS) including ontology design, distributed database realization, facilitator design, and introduced the validation work of the case study in a large aviation aluminum plate in detail [22, 23]. Wu and Wu et al. presented a MAS design and evaluation for WSN-SHM to validate its efficiency [24].

Recently, mobile agent (MA) is an interesting field in MAS research. Different from stationary agents, MAs are able to migrate from one node in a network to other nodes and to be executed on any nodes in the network. MAs can be created dynamically at runtime and dispatched to destination systems to perform tasks with most updated code and algorithms [25]. Recently, few papers have been published on the application of MA in WSN-SHM [24, 26, 27]. For large-scale structural health monitoring, methods presented till now still have limitations. Firstly, the more the monitored subarea, the larger the data quantity transmitted in WSN. Hence, the station management node overburdens with the more computational tasks. Secondly, all the agents reside in the node or computer at the fixed physical location. If the monitored object is more complex, the task of the signal processing or evaluation agents is more onerous, and these agents consume largely the memory resource of their resided node or computer, whose CPU with high-performance is also required. Therefore, to solve the above problem in WSN-SHM, this paper combines MA's advantage with WSN-SHM, presents a mobile multiagent system for WSN-SHM, and gives the system implementation and experimental verification for the strain distribution change and joint failure. In the system, MA with the signal processing or evaluation method is migrated to the sensor nodes, processes with the database distributed in the node, routes and selects the returned path autonomously, self-organizes sensor network, returns the result to the station management node with the node's collaboration, aggregates and fuses the distributed sensor information, and coordinates to estimate the strain distribution change and joint failure position.

The organization of this paper is as follows. Section 2 provides the mobile multiagent coordination framework for WSN-SHM system. In Section 3, the proposed mobile multiagent system for the static distribution change and joint failure monitoring is described in a case evaluation for damages identification. At last, Section 4 gives a discussion and a brief comment.

## 2. Development of SHM Based on Mobile Multi-Agent System

The architecture of mobile multiagent system for WSN-SHM is given, which consists of the physical and software architecture. Two architecture perspectives are used to support collaborative SHM task allocation based on MA, network self-organization, sensor data aggregation and fusion, and damage location identification.

*2.1. Physical Architecture.* In the real application, the dense and distributed sensor network is laid on the large-scale structure to observe quantities as stress, stain, acoustic, or pressure. Wireless sensor node (SN) connected to these sensors might be deployed to clustered topologies, and report to a remote central station in a multihop fashion. In the clusters, Cluster Head (CH) can communicate with any other sensor nodes within its radio range to a star topology which might support synchronization simply. In the top layer, Station Management Node (SMN) has wireless links to the sensor nodes and communicates with the cluster heads to a cluster-star topology. The clustered topology framework based on MA can completely support low-power, multipoint, and heterogeneous operations with a distributed synchronization mechanism, and mobile agent migration in the wireless sensor network as shown in Figure 1 [24].

*2.2. Software Architecture.* Considering a typical SHM system, a large-scale structure can be divided into several subareas monitored. The cable-based sensors or wireless sensors are placed in or on the structure to acquire the data on the structural status parameters. The appropriate signal or information processing methods are adopted to analyze and extract the damage-sensitive features from the sensing data. The corresponding damage evaluation methods can obtain the structure health status using the key feature. According to the various functional components of SHM system, data monitoring, interpretation, and damage diagnostic layer can be defined to form MAS, and sensing agent (SA), signal processing agent (SPA) and damage evaluation agents (DEA) can be given correspondingly.

In the large-scale structure, each subarea is monitored by kinds of agents resided in their subsystem. In every subsystem, the district monitor agent (DMA) generates and transmits MA to obtain the important damage data in local subareas from SAs, and coordinates and delivers data and information between agents of each layer for damage evaluation. The Facilitator agent provides "yellow pages" services to every kind of agent. It is in charge of registering every agent services, such as feature extraction and damages assessment. Thus this is convenient for the agent to search the services and resources to achieve interaction and collaboration. In the subsystem, sharing information management agent (SIMA) is designed. It is a distributed database, and it allows the agent to publish its identity (ID) and address in order that other agents can easily find it, and it is beneficial to exchange the information between different DEAs in the same or different subsystems. Meanwhile, it saves and provides the DEA's parameter.

Taking the management and the coordination function into consideration, central coordination agent (CCA), central information fusion agent (CIFA), and user interface agent (UIA) are introduced to fuse the damage information from the local subareas and provide the whole information to the user in the system. CCA is responsible for the coordination among subsystems, such as conflict solving, time synchronization, resource distribution, and negotiation strategy, and so forth. CIFA is in charge of fusing the damage information from different subareas to give a global estimation of the whole structure. UIA provides information to the user and accepts the user's instruction. In our SHM system, Beliefs-Desires-Intentions (BDI) models of agents are adopted referring to [23]. Combined with the character of agent in each

Figure 1: The physical architecture of WSN topology structure based on mobile multiagent system.

layer, different agent architectures are utilized, such as, the reactive one of the data monitoring layer agent, the hybrid one of the data interpretation and damage diagnostic layer agent, and the deliberative one of information layer agent.

The whole structure's damage evaluation work can be realized through subsystem's cooperation. The software architecture of mobile multiagent system for WSN-SHM is shown in Figure 2 [22, 23]. SA is implemented by hardware and software, and the other agents are only implemented by software. SA includes a sensor or several sensors, for instance, a PZT sensor or four strain gauge sensors. It is in charge of sensing a subarea or a local site. SA resides in SN. CH is one of SNs. Hence, CN is also resided by SA. Each local SMN is connected to the central computer. It is the data transmit hardware interface of every subsystem in the central computer software which consists of SPA, DEA, Facilitator agent, and SIMA. Each DMA resides in its SMN. It is responsible for receiving the data from its CN, and transmitting and receiving MA for damage evaluation. SPA and DEA may also reside in each SN in a cluster. CIFA, CCA, and UIA are in charge of managing and coordinating the whole systems in the central computer.

## 3. Evaluation of SHM Based on Mobile Multiagent System

In order to verify the effectiveness of mobile multiagent system for WSN-SHM, in this paper, a large aerospace aluminum plate structure is studied as the experimental object. The wireless sensor array is adopted to monitor two typical structure damages such as strain distribution change and joint failure.

*3.1. System Setup.* In Figure 3, it gives the plate structure and the sensor distribution diagram, and the photo of the structure and the wireless sensor node are shown in Figure 4. The plate structural material is the aerospace hard aluminum LY12, whose basic dimensions and thickness are 120 cm × 200 cm × 0.25 cm. Around the structure, there are 64 M6-bolts which are used to fix the plate with bracket, and the bolt space is 10 cm. The bracket is put on the ground vertically, supporting the aluminum structure. The structure is divided into eight subareas, each of which is 49 cm × 45 cm except its edge. PZT sensors are laid on the vertices of each subarea. In the 3rd subarea, FBG sensors are arranged, and four strain gauge sensors are correspondingly laid on the reverse side of the structure, and in other subareas, the strain gauge sensors are laid on the right side of the structure. The finite element model and experiment experience are followed for putting these strain gauge and PZT sensors on the structural stress concentration positions. In addition, each subarea is divided into nine regional units.

In the paper, the presented mobile multiagent system for WSN-SHM is described in detail for the strain distribution change and joint failure monitoring as follows. It should demonstrate the advantage: the system can migrate the signal processing or evaluation method to different sensor networks, allocate the monitoring tasks, process with the database distributed in the node, route and select the returned path autonomously, self-organize sensor network, return the result to the station management node with the node's collaboration, aggregate and fuse the distributed sensor information, and coordinate to estimate the strain distribution change and joint failure position.

*3.2. Theory.* Strain distribution change and joint failure are two types of damages in SHM, which can be identified by the pattern recognition technology. Their principle is as follows.

The structural strain distribution change generated by static load is monitored by the strain sensor network [2]. The sensor outputs of the monitored substructure form a pattern to represent its strain distribution. Once a concentrated load applies on different positions of the structure, its strain distribution changes correspondingly. As a result, the output mode of the sensors in the monitored subarea also changes. The pattern recognition method can be used to classify different patterns to decide different positions of the applied static load or strain distribution change. Generally, the minimum-distance classification is adopted for the static load localization. If the distance between the monitored strain

Figure 2: The software architecture of mobile multiagent system for WSN-SHM.

Figure 3: System setup.

Figure 4: The photo of the structure and WSN.

pattern and a reference strain pattern in database is the minimum, the load position of the reference strain pattern is deemed to be the one of the monitored pattern.

The joint failure induced by bolt loosening is generally monitored by the active SHM method adopted [2]. The method is based on the structural vibration response and uses the piezoelectric sensor as the actuator or the sensor. For the large structure, the cycle-actuator multisensor scheme is adopted since the power of the actuator is finite, and numerous structure joint bolts and pattern overlapping exist. PZT sensors around the boundary act as actuators in turn. Each time the signals of two adjacent PZT sensors are sampled. Generally, a sine wave at a frequency is excited by PZT actuator to the structure, under which the vibration response of the structure is sensitive to the bolt loosening. The experiment proves the sensor signal varies before and after the bolt loosening [23]. Hence, the peak changes of the sensor signals are extracted to form the feature mode for pattern recognition. The neural network is often applied to identify the loosening bolt [23].

*3.3. Wireless Sensor Network and Mobile Agent System.* The entire hardware schematic diagram for the WSN-SHM system is shown in Figure 5. The entire system includes wireless strain sensor node and wireless PZT sensor node developed independently by the laboratory, stations management node, and central computer. For the system, the strain gauge sensor is connected to the wireless strain sensor node for strain measurement, and the signal of PZT sensor is obtained by the wireless PZT sensor node for joint failure active monitoring. These wireless sensor nodes communicate with each other by the wireless transmission based on ZigBee protocol. The stations management node communicates with the central computer by the Serial Port.

*3.3.1. Agent System Development.* In the multiagent system, all agents are implemented by software except SA which is implemented by the measurement hardware and software. The software is programmed with nesC language, LabVIEW 8.5, and MATLAB R2006a in the central computer.

In the software, there are data monitoring layer agents, data interpretation layer agents, damage diagnostic layer agents, and information layer agents. Each of the layers contains a number of agents performing different functions. The social ability and cooperation between the agents leads to the final damage estimation. The functionality and major composition principles of the agent within each layer are described below.

(1) Data Monitoring Layer Agents

*Strain Sensing Agent (Strain SA).* The static strain change generated by the concentrated load applied to the plate is measured by the sensing agent.

Generally, the strain gauge is adopted to measure the strain distribution. The strain gauge sensor is the most usual sensor to monitor the strain, and it has cheap price and stable performance. It is connected to the wireless sensor node through the bridge circuit, and condition circuit as shown in Figure 6 [24]. Figure 7 gives the schematic circuit diagram for the 3 V high-precision power supply and conditioning the strain gauge sensor signal. Strain SA consists of four sensors in a subarea, a wireless sensor node, and the node's acquisition and communication program implemented by nesC language.

For the wireless strain sensor, the relationship between the output voltage change $\Delta V$ and the strain parameter variation $\Delta\varepsilon$ is as follows: $\Delta V = K_1 K_2 V \Delta\varepsilon/4 \cdot K_1$ is the sensitivity coefficient of the strain gauge, $K_1 = 2 \cdot K_2$ is the amplify gain of the low-power instrumentation amplifier AD623, $K_2 = 151.5 \cdot V$ is the bridge voltage provided, $V = 3.3$ V. The TI MSP430F1611 MCU is a low-power and high performance 16-bit mixed signal processor. The sample frequency of the 16-bit analog-to-digital converter in the node's MCU is 32 Hz.

*PZT Sensing Agent (PZT SA).* The agent is responsible for monitoring the joint failure using the cycle-actuator multisensor method.

Generally, PZT element is adopted to actuate or sense the structural dynamic strain response, and it is connected to the wireless sensor node as the actuator or sensor. The active monitoring for bolt loosening can be finished by the wireless PZT sensor node [28]. As shown in Figure 8, the node integrates sine wave generator module, data acquisition module, and condition module. Figure 9(a) shows the schematic circuit design for a sinusoidal signals actuator circuit, and Figure 9(b) shows the schematic circuit design for a PZT signal conditioning circuit in the evaluation system. In the assessment experiment, fifteen PZT elements are considered, in which twelve ones are used to monitor the joint failure around the boundary on the frame. PZT SA consists of a PZT sensor and a wireless sensor node connected, and the node's acquisition and communication program developed by nesC language.

For the wireless PZT sensor, the output of excitation circuit with the timer TCL555 is the sine wave with 100 kHz, whose amplitude is 5 V. The magnification times $K$ of the two-stage amplifier OPA 340 circuit are tunable from 50 to 100. The TMS320F2812 DSP is a highly integrated low-power and high-performance 32-bit digital signal processor. Its sample frequency of the 12-bit analog-to-digital converter in the node's DSP is 10 MHz.

Figure 5: The schematic diagram for the wireless sensor system integration.

Figure 6: The connection of the strain gauge and its WSN.

(a) The schematic circuit design for the high-precision power supply

(b) The schematic circuit design for the strain gauge signals conditioning

Figure 7: The schematic circuit diagram for conditioning the strain gauge sensor.

Figure 8: The connection of the PZT and its WSN.

(a) The schematic circuit design for a sinusoidal signals actuator circuit

(b) The schematic circuit design for PZT signals conditioning

Figure 9: The schematic circuit design for PZT actuator and signals conditioning.

*District Monitor Agent (DMA).* The agent is in charge of managing strain SAs and PZT SAs. Each DMA supervises four neighboring strain SAs, which are in charge of monitoring Subarea 2, 3, 6, 7, and fifteen PZT SAs on the plate. It resides in SMN and is responsible for generating and transmitting MA and receiving the data from CN, and coordinates and delivers data and information between agents of each layer for damage evaluation. It is implemented by LabVIEW.

(2) Data Interpretation Layer Agents.

The layer agents have the LabVIEW work threads, and the communication protocol interaction thread, and their tasks are to extract the signal feature.

*Static Load Signal Processing Agent (SL SPA).* The agent uses the moving average method for the signal of Strain SA.

*Bolt Loosening Signal Processing Agent (BL SPA).* It extracts the peak value of the response from PZT SA.

(3) Damage Diagnostic Layer Agents.

The layer agents also consist of the work thread integrating with the MATLAB function, and the communication protocol interaction thread with LabVIEW. They begin the process of turning the data into information that is of greater use to the user. In addition, the agent in the layer uses the advanced intelligent system techniques coupled with codified knowledge problems and offers a prognosis.

*Static Load Damage Evaluation Agent (SL DEA).* The agent uses $k$ nearest neighbor ($k$-NN) classifier for the static load location identification. $k$-NN classifier is very simple and effective [29]. The $k$ nearest neighbors of the unidentified test pattern is searched within a hypersphere of predefined radius in order to determine its true class, which is the most class in the $k$ samples. If only one nearest neighbor is detected, $k$-NN is the minimum-distance classification.

*Bolt Loosening Damage Evaluation Agent (BL DEA).* The agent implements the bolt loosening position identification for support vector machine (SVM) classifier [30]. The classifier is based on statistical learning theory, and it can obtain the good recognition rate derived from fewer training samples than the neural network classifier [31]. Kernel function is a key parameter for SVM, which includes linear, polynomial, Gaussian RBF, and sigmoid.

(4) Information layer agents

The layer agents also integrate with the work thread and the communication protocol interaction thread with LabVIEW. They are responsible for fusing and coordinating the bottom layer agent.

*Facilitator Agent (FA).* Every subsystem should host a default FA. It manages the agent's ID and service name built with LabVIEW cluster array.

*Sharing Information Management Agent (SIMA).* A default SIMA should host in every subsystem and manage all the agent's IDs and addresses in the subsystem. The agent stores the agent's ID and address built with LabVIEW cluster array, which is set to be shared variable and is able to start up and stop other agents.

*Central Information Fusion Agent (CIFA).* The agent fuses different information from different local subsystems to obtain the most reliable and precise conclusion.

*User Interface Agent (UIA).* The agent provides the information to the user and accepts the user's instruction. UIA is realized with LabVIEW control.

*Central Coordination Agent (CCA).* The agent is responsible for coordinating the whole monitoring process. It collaborates DMA to generate and control MA with the algorithm and decides when to start the static load location process, when to deal with the joint bolt loosening and when to self-organize the sensor network.

MAS comprises a number of agents, which is to solve the complex problem not finished by a single agent and is the coordination network among agents. The coordination mechanism is the key issue in MAS research. In mobile multiagent system for WSN-SHM, the coordination principle is shown in Figure 10 for the static load and bolt loosening localization. Once CCA checks that there is PZT SA working at actuator status, wireless PZT sensor network is triggered to identify the joint failure. Otherwise, wireless strain network is activated to locate the static load. Since bolt loosening is an accumulated damage, the cycle monitoring is adopted, and

(a) The coordination program flow chart

(b) The scheme of MA migration

Figure 10: The work principle of the whole wireless sensor system.

the static load damage needs real-time monitoring. Hence, DMA periodically transmits MA to wireless PZT sensor network, drives the node to actuate the structure in turn, migrates from source to destination for obtaining the data and feature and publishes the feature vector to DMA for the damage evaluation. In the other most time, once the strain change crosses a threshold on the structure, DMA transmits MA to wireless strain sensor network, migrates from source to destination for acquiring the data and feature and identifying the damage location, and publishes the evaluation to DMA. The detailed principle and evaluation result are given as below.

*3.3.2. Collaborative Static Load Localization.* As shown in Figure 11, subareas 2, 3, 6, and 7 are monitored by 4 sensor nodes composed of 16 strain gauges that form the sensor network. In this paper, a loading equipment is utilized to change the strain distribution in the plate. 70 N is chosen in the demonstration.

The presented distributed wireless strain sensor network mode based on MA for strain distribution change monitoring is different from the traditional mode, and it need not to obtain the data from all the nodes, centralize to process, and finish the pattern matching as shown in Figures 12 and 13. The principle is given in detail as follows.

Every time when Strain SA reads the data of four strain variation of its subarea, it stores them in the temporary base of the node. A standard pattern database is also established in the node, which is a fixed base. If Strain SA finds the maximum change between fixed base and temporary base exceeds

Figure 11: The scheme of the strain distribution change monitoring based on WSN.

Figure 12: The traditional wireless strain sensor network monitoring mode.

Figure 13: The wireless strain sensor network monitoring mode based on MA.

Figure 14: The workflow of the multiagent implementation for the strain distribution change localization.

20 $\mu\varepsilon$, DMA controls SMN to route MA with SL SPA and SL DEA to all the SNs to deal with the strain distribution change localization, in which the first migrated node is defined to be CH. The threshold 20 $\mu\varepsilon$ is set according to the finite element modeling analysis based on MSC, Patran/Nastran software, and practical loading experiments. The multiagent implementation process for collaborative strain distribution change localization includes following steps. Figure 14 shows the workflow of the multiagent implementation process for the strain distribution change localization.

*Step 1.* In every subsystem, DMA sends an MA with the strain computation and judgment algorithm to each node for the initial damage location evaluation. Firstly, MA calculates the sum of four absolute values of the strain change ($\mathrm{Sum}(i) = |\Delta\varepsilon_{i1}| + |\Delta\varepsilon_{i2}| + |\Delta\varepsilon_{i3}| + |\Delta\varepsilon_{i4}|$, $i$ is Subarea ID, $i \in \{2, 3, 6, 7\}$) for the subarea of each node. Secondly, MA gives a judgment that whether the sum exceeds 280 $\mu\varepsilon$ for the subarea of its resident node. If so, the node's ID is transmitted to DMA, and MA departs. Otherwise, MA goes on to move to the next node, and its ID is sent to DMA, and then MA departs. The parameter 280 $\mu\varepsilon$ is also set by the finite element modeling analysis and practical experiments.

*Step 2.* If there is a sum value exceeding 280 $\mu\varepsilon$ for some node, DMA sends MA2 with SL SPA and SL DEA to the node according to its ID received for damage location evaluation. The final location result is transmitted to DMA, and MA2 departs.

*Step 3.* If there is no sum value exceeding 280 $\mu\varepsilon$ for all the nodes, MA3 is sent by DMA. MA3 with SL SPA and SL DEA is started to compute nine Euclidean distances in each node, and transmitting the minimal distance $d_i$ and the localization result of each subarea to the next node. Once MA3 migrates to the last node, it sorts the minimal distance $\{d_i\}$ of all the nodes. Here, the minimal distance and the second minimal distance is assumed to $d_{il}$, $d_{in}$ ($l, n \in \{2, 3, 6, 7\}$). If $d_{in}/d_{il}$ exceeds 10 $\mu\varepsilon$, the diagnostic result of the subarea with the minimum $d_{il}$ is the final result, which is transmitted to DMA, and MA3 departs. The ratio parameter 10 is determined by lots of experiments.

*Step 4.* If the ratio of two minimal Euclidean distances is less than 10, DMA sends MA4 to the node. MA4 deems that neither of the subareas develops successful localization, and the static load position may be on the boundary between two adjacent areas. Hence, the agent organizes the sensor network, and aggregates and transmits the data of the organized four sensors to DMA. DMA tells DEA in its subsystem to identify the load location. The utilized fixed base is in SIMA of its subsystem.

(a) Strain 3

(b) Strain 4

(c) Strain 7

(d) Strain 8

Figure 15: Sensor strain signal of subarea 2 of plate structure applied on by 70 N load.

In the experiment, twenty-one possible load positions are numbered in two subareas as shown in Figure 11. The length of the moving average is 70 for the strain gauge. For Strain SA, the strain changes of the four strain sensors are combined to be a feature vector with four dimensions. For each position, six samples are measured to train parameters of the classifiers. When 70 N load occurs in the twenty-one positions, the absolute sum of four sensors' strain of Strain SA 7 changes as shown in Table 1. The collected signal waveforms of Strain SA 2 are shown in Figure 15. Next, *1*-NN DEA with Euclidean distance is utilized to classify the calculated features of the strain distribution change. The classification accuracy is evaluated using a ratio of the number of the samples classified correctly to the total sample. When the static load appears at the boundary between Subareas 3 and 7 as shown in Figure 11, the corresponding strain variation in the four subsystems belongs to Step (3) above. MA finds the $d_{in}/d_{il} < 10$ by computing the minimum Euclidean distance in the four subsystems and organizes the new sensor network to get the result that the static load is on the boundary between subsystem 7 and 3. The accuracy of the static load localization is 80.95% in Table 2.

Table 1: The strain distribution of Subarea 7 of the plate structure applied on by 70 N load.

| Load location no. | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|
| Strain ($\mu\varepsilon$) | 348 | 204 | 290 | 138 | 54 | 21 | 10 |
| Load location no. | 8 | 9 | 10 | 11 | 12 | 13 | 14 |
| Strain ($\mu\varepsilon$) | 68 | 155 | 1531 | 106 | 51 | 28 | 9 |
| Load location no. | 15 | 16 | 17 | 18 | 19 | 20 | 21 |
| Strain ($\mu\varepsilon$) | 302 | 167 | 321 | 103 | 50 | 34 | 14 |

*3.3.3. Joint Failure Active Coordination Monitoring.* The cycle-actuator two-sensor scheme is adopted for joint failure monitoring. In Figure 16, through multiagent cooperation, the wireless sensor network nodes connected by PZT elements around the boundary are self-organized to act as the actuator in turn. The PZT node as actuator and its two adjacent PZT nodes as sensors constitute a cluster, and the actuator node is defined as CH of the cluster.

Every time when PZT SA reads the strain data of the local site of its subarea, it stores them in the temporary base of the node. Since the bolt loosening identification

Figure 16: The scheme of the joint failure monitoring based on wireless PZT sensor node.

Figure 17: The wireless PZT sensor network mode based on MA for the joint failure monitoring.

Table 2: The strain distribution change location identification results.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Load location no. | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Identification result | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Load location no. | 8 | 9 | 10 | 11 | 12 | 13 | 14 |
| Identification result | 8 | 2 | 10 | 11 | 12 | 13 | 6 |
| Load location no. | 15 | 16 | 17 | 18 | 19 | 20 | 21 |
| Identification result | 15 | 16 | 17 | 17 | 19 | 20 | 14 |

needs to fuse all the features from the whole area, a standard pattern database is only established in SIMA of each subsystem, which is implemented in the central computer connected with SMN resided by its DMA. Hence, its standard pattern database is also established in the computer. The multiagent implementation process for collaborative joint failure localization includes following steps.

*Step 1.* According to CCA's coordination request, DMA self-organizes the PZT sensor network and orderly routes MA1 to twelve nodes on the plate boundary as shown in Figure 17. Firstly, MA1 starts PZT SA1 (its sensor name is PZT1) as the first actuator. Then, its neighboring two PZT SAs acquire the data, number and store them in the temporary base of the node. Thirdly, MA1 migrates to the next node for finishing the scheme of the one-actuator two-sensor until it traverses to all the nodes. At last, MA1 is back to DMA and departs.

*Step 2.* DMA routes MA2 with BL SPA to these nodes on the border in turn for extracting the peak feature of its resident data in each node. Meanwhile, MA2 transmits the two features and their ID to DMA. Finally, MA2 returns SMN and departs.

*Step 3.* DMA waits and receives the features from all the nodes. Then, the agent sorts these features according to their ID and publishes the feature vector to BL DEA by Facilitator agent for the pattern location. The final result is sent to UIA by CIFA.

In the experiment of the joint failure, each time only one bolt is full loosening. The loosening of bolt 3, 5, 8, 9, 23, 27, 39, 50, and 54 are studied as shown in Figure 16. MA controls twelve PZT SA circularly and periodically to excite and sense the structure strain signal. Figure 18 gives the signal of PZT sensor 1 as actuator and PZT sensor 2 as sensor before and after bolt loosening of Bolt 5, and it gives that the signal of PZT sensor 1 as actuator and PZT sensor 4 as sensor

(a) PZT 2 sensor signal before Bolt 5 loosening

(b) PZT 2 sensor signal after Bolt 5 loosening

(c) PZT 4 sensor signal before Bolt 9 loosening

(d) PZT 4 sensor signal after Bolt 9 loosening

Figure 18: PZT sensor signal change before and after bolt loosening bolts.

before and after bolt loosening of Bolt 9. As shown in the figure, the bolt loosening causes the acquired strain signal to change since bolt loosening could cause the change of the prestress distribution in the structure and the bolt loosening can partly affect the scattering Lamb wave coupled with bolt. The number of sampled data is 600, and the measured time is 0.06 ms. Twenty-four acquired signal peak changes of the twelve sensors on the plate border for PZT SA are combined to be a feature vector as shown in Tables 3 and 4. Since PZT element as actuator in turn is PZT1, 2, 3, 6, 9, 12, 15, 14, 13, 10, 7, 4, the first and the second features in the feature vector are the peaks of two acquired signals of PZT 1 as the actuator and PZT2, 4 as the sensors, and are defined as 1-2/1-4. Hence, the third and the fourth features are defined as 2-1/2-3, and so forth. For chosen bolts, twelve samples are measured to train parameters of the classifiers. Table 5 gives the test accuracy of the SVM classifier agents, in which the kernel function $k(x, y) = 0.7x^T y + 1$ and the punish coefficient $C = 10$ are selected. It can be seen that the classification accuracy is 0.70.

Table 3: The feature vector before Bolt 5 loosening.

| Feature | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| Amplitude (v) | 1.538 | 2.188 | 2.170 | 7.152 | 4.829 | 0.578 |
| Feature | 7 | 8 | 9 | 10 | 11 | 12 |
| Amplitude (v) | 0.423 | 1.993 | 1.957 | 2.234 | 1.902 | 0.865 |
| Feature | 13 | 14 | 15 | 16 | 17 | 18 |
| Amplitude (v) | 1.007 | 1.031 | 0.594 | 0.996 | 1.793 | 0.585 |
| Feature | 19 | 20 | 21 | 22 | 23 | 24 |
| Amplitude (v) | 1.793 | 1.814 | 2.594 | 1.802 | 1.508 | 2.620 |

## 4. Conclusion

In order to build an efficient, robust, reliable, and autonomous large structural health monitoring system for wireless sensor network, a mobile multiagent system is applied to integrate distributed wireless sensor network. The proposed method is illustrated through a large dimension aerospace

Table 4: The feature vector after Bolt 5 loosening.

| Feature | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| Amplitude (v) | 2.104 | 3.783 | 3.746 | 8.842 | 5.619 | 0.905 |
| Feature | 7 | 8 | 9 | 10 | 11 | 12 |
| Amplitude (v) | 1.587 | 1.728 | 1.712 | 3.958 | 3.005 | 0.756 |
| Feature | 13 | 14 | 15 | 16 | 17 | 18 |
| Amplitude (v) | 1.175 | 1.545 | 1.259 | 1.549 | 1.484 | 1.248 |
| Feature | 19 | 20 | 21 | 22 | 23 | 24 |
| Amplitude (v) | 0.587 | 1.463 | 2.421 | 0.585 | 2.089 | 2.413 |

Table 5: The part identification result of bolt loosening.

| Bolt no. | 3 | 5 | 8 | 9 | 23 |
|---|---|---|---|---|---|
| Identification result | 8 | 5 | 54 | 9 | 23 |
| Bolt no. | 27 | 39 | 50 | 54 | Health |
| Identification result | 27 | 39 | 27 | 54 | Health |

aluminum structure monitoring experiment in which strain distribution and joint failure are monitored by different sensors. The result shows that the method can significantly improve the monitoring performance for the large-scale structure. In the future work, the validation work of mobile multiagent system and wireless sensor network on the large-scale composite material structure will be considered.

## Acknowledgments

This work was supported by European Union 7th Framework Program international cooperation project (Grant no. FP7-PEOPLE-2010-IRSES-269202), Natural Science Foundation of China (Grants nos. 60772072, 50830201, and 10872217), the Fundamental Research Funds for the Central Universities, and Funding of Jiangsu Innovation Program for Graduate Education (Grant no. CX10B_097Z).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 310797, 10 pages
doi:10.1155/2012/310797

*Research Article*

# Application of Multiscale Fiber Optical Sensing Network Based on Brillouin and Fiber Bragg Grating Sensing Techniques on Concrete Structures

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Received 19 June 2012; Accepted 27 August 2012

Academic Editor: Ting-Hua YI

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The paper reports the application of the distributed optical fiber sensing technology and the FBG sensing technology in bridge strain monitoring; the overall changeable characteristics of the whole structure can be obtained through the distributed optical fiber sensing technology (BOTDA), meanwhile the accurate information of local important parts of the structure can be obtained through the optical fiber Bragg grating sensor (FBG), which can improve the accuracy of the monitoring. FBG sensor has a high sensitivity, but it can only realize the measurement of local discrete points for the quasidistributed sensing. BOTDA can realize the long distance and distributed measurement, but its spatial resolution is not high. FBG and BOTDA were applied together in bridge monitoring in this test, taking full advantage of the distributed BOTDA on the overall strain measurements of the structure, as well as monitoring the key parts by the arrangement of FBG. The combined application of BOTDA and FBG can achieve the overall monitoring from point to line and then to the surface and, therefore, obtain more comprehensive information on the strain of the test structure.

## 1. Introduction

In china, with the continuous construction of large-scale infrastructure projects and facilities, the technology of the construction quality monitoring under construction and the health diagnosis in structural service are increasingly required. A key issue in structural health monitoring is how to monitor, identify, locate, and calibrate the force in the structural system effectively. Some researchers studied the optimal sensor installation method, in order to make the sensor networks work more efficiently. Li et al. and Yi et al. have systematically studied this issue proposed some meaningful optimal sensor installation methods and algorithms [1–4]. Brillouin optical time-domain analysis technology (BOTDA) is a distributed optical fiber strain sensing technology based on stimulated Brillouin backscatter principle, which breaks through the traditional concept of point sensing and realizes the continuous distributed monitoring in its true sense. The sensing gauge and the spatial resolution in the temperature and strain measurement can reach a higher level than other sensing technologies. So BOTDA has become a high-end sensing technology in the large civil engineering health diagnosis, which leads to the international advanced developed countries race to research. Scholars have been doing some research and applications on this technology. Xuan et al. had applied BOTDA technique to monitor the strain of reinforced concrete structure for monitoring steel bar corrosion according to the variation of strain distribution [5]. Qian et al. monitored the cracking development for asphalt concrete pavement [6]. He et al. studied the real-time reinforcement stress measurement ability of BOTDA technique [7]. FBG (fiber bragg grating) is a new type of optical components. Due to the higher monitoring accuracy and the mature technology, it has

been widely used in the monitoring of large structures. Sun reported the application in civil engineering systematically [8]. And other researchers had developed a monitoring technique for various parameters detection using FBG sensing technique [9–11]. Distributed fiber optic sensors have more advantages compared with conventional sensing methods as follows. Firstly, the measured spatial distribution state and time-varying information can be obtained simultaneously; secondly, FBG provides continuous measurement of the fiber environment parameters along the entire fiber; thirdly, theoretically the measured information can be a function of the fiber position, resulting in obtaining the resolution of any size; lastly, it is feasible to use an optical fiber to replace the traditional sensors array which consists of hundreds of dot matricies, which greatly reduce the cost of monitoring. FBG monitoring accuracy is high, but the cost is expensive; besides, the measuring point is relatively independent, and it is difficult to grasp the overall variation law of the structure. So it is reasonable to combine the two monitoring tools, combining distributed optical fiber and FBG, together with the structure, making full use of the advantages and avoiding the disadvantages. The distributed optical fiber sensors and FBG are used in the critical part of the structure at the same time to cross-check the two monitoring results. And only the distributed optical fiber sensors are used in the noncritical part of the structure to monitor the general trend of the structure. Experimental results demonstrate the good effect of the joint monitoring.

## 2. Basic Principle of the Fiber Bragg Gating and BOTDA

*2.1. Basic Principle of the FBG.* Fiber core is a photosensitive material, that is, the refractive index of the core varies with the intensity of the UV irradiation. Fiber Bragg grating makes use of this photosensitivity. By using this photosensitivity and ultraviolet writing technique to write the coherent field of the incident light to the core, refractive index of the core periodically changes and forms the FBG whose refractive index changes along the fiber length, as shown in Figure 1. The change of grating ambient temperature, strain, stress, or other test physical quantity will result in the change of the core refractive index or the grating period and then result in the wavelength change of fiber Bragg grating signal. By monitoring the changes of the grating wavelength $\lambda_B$, the change of physical quantities to be tested can be obtained [9].

Bragg grating center wavelength of the reflected light (Bragg wavelength) is called characteristic wavelength and satisfies

$$\lambda_B = 2n_{\text{eff}}\Lambda, \tag{1}$$

where $n_{\text{eff}}$ is refractive index, and $\Lambda$ is grating period. Equation (1) demonstrates that when $n_{\text{eff}}$ alters, $\lambda_B$ changes accordingly. Through the Bragg grating center wavelength detection, the structure strain and the changes of outside conditions such as temperature can be obtained. In the process of fiber Bragg grating sensor measuring the strain and temperature, the change of strain and temperature will lead to the change of reflected light center wavelength of Bragg grating, and the relationship between the offset of center wavelength and the change of strain and temperature is shown as as follows:

$$\Delta\lambda_B = \lambda_B(1-\kappa_\alpha)\Delta\varepsilon + \lambda_B(1+\zeta)\Delta T, \tag{2}$$

where $\Delta\lambda_B$ is the offset of center wavelength of the reflected light, $\Delta\varepsilon$ is the change of strain, $\Delta T$ is the change of temperature, $\kappa_\alpha$ is the elasto-optical coefficient of fiber, and $\xi$ is the thermo-optical coefficient of fiber.

Equation (2) shows that the wavelength of grating has dual sensitivity to the external strain and external temperature; the changes of temperature/strain will lead to changes in the wavelength of fiber Bragg grating sensors, which can be detected from the grating reflection spectrum. By comparing the grating wavelength before and after the motivation, calibrating the offset by the variation of wavelength, then the pressure and the strain values on different points can be obtained.

*2.2. The Principle of BOTDA.* The basic principle of the Brillouin optical time-domain analysis (BOTDA) is based on the feature of Brillouin-stimulated amplification. The spontaneous Brillouin scattering intensity increases as the pump light intensity increases, and when the intensity reaches a certain extent, the reverse transfer Stokes light and pump light will interfere, causing strong interference fringes, then the fiber-optic local refractive index greatly increases. Because of electrostrictive effect, it will produce acoustic waves, which inspire more Brillouin-scattered light, and the inspired scattered light also enhances the acoustic wave at the same time, and the interactions produce a very strong scattering, which is stimulated Brillouin-scattering (SBS) [12], as shown in Figure 2.

The value of Brillouin frequency shift is given by

$$\nu_B = \frac{2n\nu_a}{\lambda_0}, \tag{3}$$

where $\nu_B$ is a Brillouin frequency shift, $n$ is the refractive index of the fiber core, $\nu_a$ is the acoustic velocity, and $\lambda_0$ is the wavelength of incident light in vacuum. For common silica glass fibers, $n = 1.46, \nu_a = 5945\,\text{m/s}$, when $\lambda_0 = 1550\,\text{nm}, \nu_B = 11.2\,\text{GHz}$.

The Brillouin frequency shift of the incident light $\nu_B$ is related to the fibers' temperature and the strain $\varepsilon$; the relationship between them is shown as follows:

$$\nu_B(\varepsilon,\theta) = \nu_B(0,\theta_0) + \frac{\partial\nu_B}{\partial\varepsilon}\cdot\varepsilon + \frac{\partial\nu_B}{\partial\theta}(\theta-\theta_0), \tag{4}$$

where $\nu_B(\varepsilon,\theta)$ is the Brillouin frequency shift when the fiber strain is $\varepsilon$ under the temperature $\theta$, $\nu_B(0,\theta_0)$ is the Brillouin frequency shift under $\theta_0$ when the fiber strain is zero, $\partial\nu_B/\partial\varepsilon$ is strain coefficient, $\partial\nu_B/\partial\theta$ is the temperature coefficient, $\theta_0$ is reference temperature, and $\varepsilon$ is the fiber strain.

Figure 1: Schematic drawing of ultraviolet reads grating.

Figure 2: Stimulated Brillouin scattering produce process.

Figure 3: The pattern of laying method.

## 3. The Laying Method of Distributed Fiber and FBG

Two methods are used to lay distributed optical fiber sensor. One is comprehensive bonding, that is, the fiber optic sensor sensing area is fully bonded to the structure, whose characteristic is realizing an overall monitoring to the structure deformation; this kind of bond needs a specific placing technology; distributed optical fiber sensors and Bragg grating sensors are bounded to the structure surface according to the design route with an effective binder; then connecting the sensing fiber end to the monitoring instruments, the monitor signal will be fed back to the monitoring system through the optical fiber. Another way is the fixed-point contact laying, that is, the paste point is determined at intervals of 1 m or 2 m along the optical fiber and then pasted to the monitoring body, and the measured strain value is the average strain value between the pasted points. This method is always used on the distortion position, and usually the space distance between the fixed points is greater than the spatial resolution of BOTDA. The fixed-point contact laying method is roughly equivalent to the comprehensive contact laying method, but the difference is to select some point on the designed construction surface to bond. Figure 3 illustrates the schematic of comprehensive bonding and fixed-point contact. Different methods should be adopted to lay the monitoring optical fiber in actual application according to the structural characteristics and monitoring purpose.

In order to meet the monitoring requirements on different sizes, fixed-point contact laying method is adopted to increase the measuring range of the fiber Bragg grating

Figure 4: The way of pc FBG adhesive in different distance.

sensors, and FBG monitoring strain is the average strain between the two pasted points. In order to ensure that the fiber grating can monitor effectively, preload tensile stress of 300–400 microstrain to the grating to prevent grating relaxation, which will lead to monitor failure or get wrong monitoring results. Figure 4 gives the paste way of prestress FBG in different distance.

## 4. Experiment Introduction

*4.1. Test Beam.* This experiment is located in Development Zone of Zhenliang Road in Dalian. The length of the bridge is 2817.004 m (k7 + 080 K9 ~897.004), the width of single-standard cross-section of mainline bridge is 23.5 m; the section of the superstructure is single box and multicell. Of all the prestressed continuous concrete beam, in addition to the height of special-shaped beam is 1.6 m, the height of other beams and cantilever is 1.5 m. The thickness of the web in the midspan and the fulcrum position is 0.4 m and 0.6 m, respectively. Most of the bridge superstructure is a prestressed structure, which forms multiple prestressed concrete box girder connection, so isolation joint pouring is used, and wet joint pouring is used for the remaining joints in the construction sequence. The cross-slope of deck is adjusted by the cross-slope of coping flange plate.

The test bridge is located in a straight line segment, and beam length is 30 m. Superstructure is the simply supported transgenic continuous structure with prestressed concrete box girder. Substructure is the piers with circular cross-section, piers that match with different forms of the pier cap and the capping beams according to the superstructure. The capping beams are prestressed structures, whose cross-sections are small and in a beautiful form. Bearing plate bed is set on the top of part of the pipers. Bored pile is used in pipe foundation. Figure 5 shows the cross-section of beam.

According to the construction drawings provided by the commissioning party, the load under the ultimate limit state is equivalent to the uniformly distributed load according to the Highway Bridge Design General specification (Take the deadweight of the beam, deck pavement, vehicle load, and other loads into consideration. The worst load transversal distribution coefficient is 0.63, and the load effect combination is done combined under the limit state.). The design load is 130 tons. During the test, give the bending moment of the midspan as the control loading, and the load is equivalent to the uniformly distributed load; in the test, the maximum load of the beam is 87 tons.

*4.2. The Scheme of Multiscale FBG Network.* The arrangement of distributed optical fiber sensors and FBG along the T-box girder flange was shown in Figures 6 and 7. Theoretical analysis shows that the axial strain of simply supported beam is larger than other directions. The laying of monitoring optical fiber mainly measures the axial strain, so one-dimensional network is used, and the comprehensive bonding method was adopted to arrange the optical fiber along the beam length direction, and 20 m free optical fiber was reserved as a temperature sensor at the beam end. Five fiber-optic temperature sensors were set up on the beam for temperature compensation, and distributed fiber-optic sensor network was shown in Figure 6. Distributed optical fiber strain sensors connected to the BOTDA were divided into five regions which, respectively, are S1, S2, S3, S4, and S5; distributed optical fiber temperature sensors respectively were T1, T2, T3, T4, and T5. Three flexometers were placed at beam bottom, on fulcrum position, and midspan of the beam, respectively. Distributed optical fibers were fixed with 502 glue at the interval 1-2 m firstly and then were bonded comprehensively with the AB glue. The optical fiber needs to be tested with optical power meter or light source so as to check the on-off and loss of the optical fiber, then inspect and repair the abnormal point, and weld the breakpoint. In order to obtain better measurements and eliminate the end effect, a length of fiber should be set aside at both ends, generally greater than 50 m.

The fiber Bragg grating strain sensors on the beam were named as follows: L31, L32, L33, L34, and L35, and in order to increase the measurement range of FBG, two points pasting method were adopted, applied prestress about 200–300 microstrain on two paste points, and then fixed the two points with AB glue. L31, L32, and L33 were preloaded grating sensors, and the length is 30 cm. The strain measured by FBG is the average strain in the range of 30 cm; L33, L34, and L35 are directly attached to the beam, in which gauge length is 10 cm.

FBG was mainly arranged in the parts where force is considerable, such as the bottom of the midspan and quarter span of the structure. The arrangement of FBG was shown in Figure 7. Put more than one grating sensor in series, and use quasidistributed measurement to reduce the measurement workload. But it needs to be paid attention to distinguish different sensors in advance according to the wavelength before laying out FBG, and the wavelength of the sensors put in a line must contain certain interval, otherwise the instrument cannot recognize or distinguish the measurement points.

*4.3. Data Acquisition System.* The main instruments and equipment used in the experiment are as follows: BOTDA (STA100), fiber cleaver, fiber fusion splicer (FSM-50S), thermostatic bath (DC-2015), hot pyrocondensation pipe, fiber grating, weights, computer, multichannel fiber-optic grating demodulator (TFBGD-9000), fiber, and so forth. The

Figure 5: Cross-section of beam (unit: cm).

Table 1: BOTDA STA 100 performance index.

| | | | |
|---|---|---|---|
| Type | STA 100-C | Maximum number of measurement point | 100000 |
| Sensing structure | Loop | Measurement average | 1∼10000 |
| Channel | 2 | Optical fiber connector | FC-APC (E-2000) |
| Spatial resolution | 0.5∼20 m | Laser wavelength | 1550 nm |
| Range resolution | 0.2 m | Sampling time | 20 s (1-2 min) |
| Range of strain | −1.5%∼1.5% | Temperature range | −270∼700°C |

Figure 6: Beam diagram of optical fiber sensor network.

Figure 8: BOTDA (STA100).

Figure 7: Beam diagram of FBG network.

following mainly describes the basic parameters of the experimental fiber, the main performance indicators of Brillouin optical time-domain analysis instrument BOTDA (STA100), and multichannel fiber Bragg grating demodulator (TFBGD-9000).

(i) The introduction of BOTDA (STA100): the Institute of the Smart Structural Systems in Dalian University of Technology imported the BOTDA (STA100) from Switzerland. The appearance of BOTDA is shown in Figure 8, and the performance indicators are shown in Table 1; the basic features are showed as follows:

(1) distributed temperature, strain measurements, and long-distance measurement which can be up to 30 km;

(2) high spatial resolution, long-term stability, and reliability;

Table 2: TFBGD-9000 fiber grating demodulation devices performance indicator.

| | | | |
|---|---|---|---|
| Optical channel | 16~64 (can be extended arbitrarily) | Wavelength range | 1520~1570 NM |
| Wavelength resolution | 1 PM | Repeatability | 2 PM |
| Scanning frequency | 300 HZ (can be set by software) | Power supply mode | 220 V AC |
| Data storage method | USB | Communication interface | Ethernet, RS-232, USB |

Table 3: Numerical solution of beam.

| Location | Load | | |
|---|---|---|---|
| | 4.986 KN/m$^2$ | 7.978 KN/m$^2$ | 9.97 KN/m$^2$ |
| Quartile | 131 ($\mu\varepsilon$) | 162 ($\mu\varepsilon$) | 183 ($\mu\varepsilon$) |
| Midspan | 180 ($\mu\varepsilon$) | 223 ($\mu\varepsilon$) | 252 ($\mu\varepsilon$) |
| Quartile | 131 ($\mu\varepsilon$) | 163 ($\mu\varepsilon$) | 184 ($\mu\varepsilon$) |

Figure 9: TFBGD-9000 grating demodulation device appearance.

(3) strain accuracy: 20 $\mu\varepsilon$ (0.002%), temperature accuracy: 1°C;

(4) measurement modes: manual or automatic measurement;

(5) data analysis: measurement, analysis, comparison of multiple data analysis, and graphical zoom.

(ii) The introduction of multichannel fiber Bragg grating demodulator (TFBGD-9000): the instrument adopts a high-power scanning laser light source, which is suitable for distributed monitoring of large-scale projects; there is no need for expensive optical switch devices, and which can achieve multichannel synchronous acquisition in the true sense. It uses parallel data processing algorithm, and there is no delay between the channels, so it has excellent dynamic performance; channel scalability is extremely high, which can extend to 64 channels easily with a low cost of channel expansion; the scan rate is 300 HZ, so the scan speed can be set by software, meanwhile it can also be compatible with the static work. The fiber optic sensors have a high capacity; 64 channels are available for over 2000 FBG sensors; integration of data acquisition device and optical access module makes the structure simple and easy to install; the appearance; as shown in Figure 9 is the performance/indicators is as shown in Table 2.

*4.4. The Test Procedure.* The distributed fiber optic strain monitoring system BOTDA was used to test the fiber when optical fiber laying was completed and took the result as the initial value of periodic measurement. Since this experiment was only a short-term measurement, step loading was adopted to collect data. The first load was 43.5 t, the second load was 69.6 t, and the third load was 87 t; the corresponding uniformly distributed loads were 4.986 KN/m$^2$ , 7.978 KN/m$^2$, and 9.97 KN/m$^2$. Each loading lasted for 10 minutes and then was read and recorded after the strain and deformation were stable, after all loads were applied, lasted for 10 minutes was read, and then unloaded and collected data 10 minutes later.

Distributed optical fiber sensor adopted Brillouin optical time-domain analyzer (BOTDA) to collect the data, which is stored in the instrument directly, and only needs to name files. Fiber Bragg grating sensors used fiber Bragg grating strain demodulation devices to collect data which is dynamic acquisition.

## 5. Numerical Simulation and Analysis

*5.1. The Results of Finite-Element Numerical Model.* The finite-element model of prestressed box girder was established by the universal finite-element software ANSYS10.0. Solid element was adopted to establish the model. The two ends of the beam are hinge structure, using SOLID95 element, which is more advanced than the 3D 8-node solid elements SOLID45. The $y$-axis node strain contour under uniformly distributed load (9.972 KN/m$^2$) is shown in Figure 10. It can be seen from the figure that the pressure of the top of the midspan girder is the largest. The finite strain numerical solution at the bottom of the beam in midspan and quartile with all levels of load is shown in Table 3.

*5.2. Distributed Optical Fiber Sensor Monitoring Results.* S1 is the distributed strain sensor on the $T$-box girder flange, S2 is the strain sensor on the bottom of the beam, S3, S4, and S5 are distributed optical fiber sensors distant from the bottom of the beam are 32 cm, 66 cm, and 102 cm respectively, and T1~T5 are distributed optical fiber temperature sensors. The method of temperature compensation on strain sensor is as follows: calculate the temperature of each temperature sensor T1 ~ T5, because the ambient temperature is not uniform, so the average temperature of the five optical fiber temperature sensors can be the temperature compensation of the strain sensor. Figure 11 is monitoring data of the entire

Table 4: FBG strain of beam.

| Sensor number | Load | | | |
|---|---|---|---|---|
| | 4.986 KN/m$^2$ strain ($\mu\varepsilon$) | 7.978 KN/m$^2$ strain ($\mu\varepsilon$) | 9.97 KN/m$^2$ strain ($\mu\varepsilon$) | Complete unload strain ($\mu\varepsilon$) |
| L31 | 73.33 | 123.33 | 164.17 | 5.83 |
| L32 | 105 | 173.33 | 227.5 | 15 |
| L33 | 73.33 | 121.67 | 156.67 | 37.5 |
| L34 | −20.83 | −34.17 | −45 | 25 |
| L35 | −14.17 | −20.83 | −26.67 | 30.83 |
| L36 | 84.17 | 137.5 | 176.67 | 10 |

Figure 10: The node of $y$-axis strain contour.

Figure 11: Curve of distributed fiber sensor frequency-distance.

Figure 12: Curve of S1 distributed optical fiber sensor strain-location.

sensor network; the monitoring strain of five distributed optical fiber strain sensors after temperature compensation is as shown in Figures 12, 13, 14, 15, 16.

Figure 12 shows the strain variation of the bottom of box girder flange which is obtained by the monitoring of distributed optical fiber strain sensor S1. Four curves in the figure, respectively, are strain curves after loading three

Figure 13: Curve of S2 distributed optical fiber sensor strain-location.

Figure 14: Curve of S3 distributed optical fiber sensor strain-location.

Figure 15: Curve of S4 distributed optical fiber sensor strain-location.

Figure 16: Curve of S5 distributed optical fiber sensor strain-location.

times and unloading, and it can be seen from the figure that the bottom of the beam is compressed; with the load increases, the compressive strain has a trend to increase, but the results of monitoring are not stable. According to the stress characteristics of simply supported beam under uniformly distributed load, the maximum strain occurred in the midspan, but the peak strain of the monitoring result is not in the midspan.

Figures 13–16 show that with the load increases, the strain of the concrete beam increases gradually; with the increase of tensile strain, the measurement results of distributed optical fiber become more and more stable, the peak is apparently in the midspan. Distributed strain measurement results comply with the strain distribution characteristics of simply supported beam under uniformly distributed load, which form a parabolic distribution, gradually increasing from both ends towards the middle.

*5.3. FBG Monitoring Data.* Corresponding to the measurement points $(l, \ldots, n)$ of the test structure, according to reflection wavelength of different grating, quasidistributed fiber Bragg grating sensors feel the stress and strain of each point along the test structure, respectively, and make the wavelength of the reflected light change; the change reflected light transmitted from the measurement site though the transmission fiber; to detect the change of wavelength with fiber Bragg grating demodulator and convert it into electrical signals, calculate the stress and strain of the points of the test structure and the distribution of the entire test structure by the secondary instrument.

Each grating strain sensor measurement results after temperature compensation calculated are shown in Table 4, and the quasidistributed strain monitoring results of FBG are shown in Figure 17.

*5.4. Results Discussion.* Figure 18 is a Gauss fitting curve of distributed optical fiber; FBG and ANSYS are numerical solution strain results for the first time loading 4.986 KN/m$^2$; Figure 19 is a Gauss curve fitting of distributed optical fiber; FBG and ANSYS are numerical solution strain results for the second time loading 7.978 KN/m$^2$; Figure 20 is a Gauss curve fitting of distributed optical fiber; FBG and ANSYS are numerical solution strain results for the third time loading 9.972 KN/m$^2$.

Figure 17: Strain of grating strain sensor after temperature compensation.

Figure 18: Gauss fitting curve of the load 4.986 KN/m$^2$.

Figure 19: Gauss fitting curve of the load 7.978 KN/m$^2$.

Figure 20: Gauss fitting curve of the load 9.972 KN/m$^2$.

From the Gauss fitting curve in Figures 18–20, it is concluded that when load and structural strain are small, due to the law strain accuracy of Brillouin optical time-domain analyzer (BOTDA), the deviation between measurement results and theoretical values solution is greater. As the load increases, the distributed measuring results of fiber Bragg grating and numerical simulation solution are closer and closer. When the load was 7.978 KN/m$^2$, the structural strain was greater than 200 $\mu\varepsilon$, and the fitting curves of monitoring value almost meet FBG measured values and numerical solutions, and the maximum deviation of the fitted values is 7%. It can be also known from the figure that when measuring the small strain, the results of FBG are more accurate than those of the distributed monitoring, but with the increase of load, the results of deviation between distribution strain monitoring and fiber Bragg grating decreased gradually. Because the strain measurement precision of BOTDA is less than FBG's, the BOTDA monitoring result has shown a larger error than FBG's results. And when the load applied increased, the error will become relatively small compared with FBG results. BOTDA is not suitable for monitoring small strains, otherwise there would be a great error. Figure 21 shows the results contrast between BOTDA, FBG, and finite-element numerical solution of the beam bottom. Figure 22 shows the contrast of across-pressure zone strain of distributed optical fiber and FBG; the experiments show that the BOTDA is not suitable for monitoring structure with small compression strain; while as the load increases, the deviation of the monitoring results between the distributed and fiber grating reduces gradually.

## 6. Conclusion

More comprehensive information of the structure can be obtained by the full-scale distributed optical fiber sensing technology which is easy to capture the overall stress condition of the structure; fiber Bragg grating sensors monitor the local point, get more precise information of

Figure 21: Contrast of across-tension zone strain.

Figure 22: Contrast of across-pressure zone strain.

local structure, and focus on critical points. The combination of the distributed monitoring technology and the multiscale monitoring of fiber Bragg grating can obtain full information with local high precision and distributed high precision. The combination of these two methods can build more economical and reasonable multiscale sensor networks.

## Acknowledgments

The authors thankful for the financial supports of the National Basic Research Program of China (2011CB013702), Key Projects in the National Science & Technology Pillar Program during the Twelfth Five-Year Plan Period (2011BAK02B02), and the National Natural Science Foundation of China (5092100).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 641391, 7 pages
doi:10.1155/2012/641391

*Research Article*

# Industrial Pipe-Rack Health Monitoring System Based on Reliable-Secure Wireless Sensor Network

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Received 7 July 2012; Revised 26 September 2012; Accepted 27 September 2012

Academic Editor: Hong-Nan Li

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Energy and power industrial plants need to improve the health monitoring systems of their facilities, particularly high-risk facilities. This need has created a demand for wireless sensor networks (WSNs). However, for the application of WSN technology in large-scale industrial plants, issues of reliability and security should be fully addressed, and an industrial sensor network standard that mitigatesthe problem of compatibility with legacy equipment and systems should be established. To fulfill these requirements, this study proposes a health monitoring system of the pipe-rack structure using ISA100.11a standard. We constructed the system, which consists of field nodes, a network gateway, and a control server, and tested its operation at a large-scale petrochemical plant. The data obtained from WSN-based sensors show that the proposed system can constantly monitor and evaluate the condition of the pipe-rack structure and provide more efficient risk management.

## 1. Introduction

Energy and power plants, although a critical element of a national infrastructure, are also high-risk facilities. Accidents that occur in industrial plants cause significant loss of life and property, which threatens national economies. From the gas explosion accident at the Union Carbide pesticide plant in Bhopal, India to the recent massive explosion during the operation of a coal-fired plant in Connecticut, we know that accidents at industrial plants often have catastrophic results in the form of property damage and fatalities. Thus, to secure the safety of the industrial plant facilities, we should construct a health monitoring system of pipe-rack structures, which typically support cables and pipes conveying resource material between equipment, with reliable and secure detection and communication technologies. In addition, the pipe-rack structures require a continuous monitoring technique that can evaluate the performance and the soundness of the structure [1].

Structural health monitoring system has gradually become a technique for ensuring the health and the safety of civil infrastructures. Furthermore, some recent advances in wireless sensor technologies have greatly explored wireless sensors for structural monitoring of civil engineering structures, such as long-span bridges and high-rise buildings [2, 3]. Network monitoring systems composed of low-cost wireless sensors were successfully installed to monitor the dynamic response of the bridge structures [4, 5]. In particular, high-rise buildings have used the global positioning system (GPS), capable of wirelessly autonomous operation and straight-line independence between target points, for the structural health monitoring of build structures [6–8]. In addition, a modular wireless micro electromechanical inclination sensor system was developed to provide structural health monitoring of large-scale hook structures [9]. Such installations allow researchers to quantify the accuracy and robustness of wireless monitoring systems within the complex environment encountered in the field.

The evaluation techniques of the structural stability of pipe racks have also evolved greatly with the development of essential technologies such as sensors [10], measurements [11], and information technologies [12]. In particular, smart sensors such as optical fiber [13] and piezoelectric sensors [14] have paved the way for evaluating the stability of the

Figure 1: A case of the aging and corrosion of pipe racks.

Figure 2: A case of the excessive extension of pipe racks.

pipe-rack structures. Efforts to reduce the installation cost of the sensors and the maintenance expense of the pipe-racks are leading to the development of new sensors with the ability to simultaneously analyze the data obtained from the sensors. In addition, new research is being conducted to develop a WSN technique that detects the abnormalities of pipe racks using the real-time analysis and control of the measured data.

Recently, industrial plants have applied wireless sensor network (WSN) technology, which has been used for the control and monitoring of logistics management, water quality, indoor temperature and humidity, and so on, in their facilities. However, the application of current WSN technology in large complex industrial plants lacks reliability and security. In addition, because of the limitation of standardized technology for interoperability between existing equipment and related communication devices, the currently used WSN suffers from its practical application [15]. However, high-risk facilities such as energy and power plants emphasize a reliable health monitoring system based on wireless sensor technology.

In the past, industrial plants were reluctant to apply wireless technology because of safety and security issues. Despite their reluctance, engineers and researchers have persevered in their interest in applying reliable standardized wireless communication technology, and they have attempted to replace the wired systems they have faced difficulties with their maintenance and management. As a result of the industry demand for wireless technology, new communication standards for Wireless HART [16] and ISA 100 [17] in addition to Wireless Fieldbus and Modbus have emerged. Since ISA 100 places far greater emphasis on connectivity between devices than Wireless HART, it ensures compatibility among field devices, field routers, gateways, and system managers based on the 6 LowPAN frame structure [18]. With the recent development of highly reliable wireless communication standards, industrial plants have attempted to construct a WSN-based health monitoring system while reducing overall administrative and operating costs. WSN technology using IEEE 802.15.4 standards has been the focus of attention as the next generation of WSN technology applicable to the field of industrial plants requiring high reliability and security. Moreover, using the WSN-based health monitoring system, we can constantly monitor and evaluate the condition of facilities and provide more efficient risk management with ample reliable and accurate field data [19].

Thus, the present study presents a WSN-based safety monitoring system using highly flexible and reliable industry-standard communication standards for the safety of pipe-rack structures largely distributed in industrial plants. The WSN health monitoring system proposed in this study could contribute to the improvement of detection technology, the automation of management, and the increase in the efficiency of the automated system. The proposed integrated monitoring system could also have a great "ripple effect" on social security framework technology.

## 2. Motivation

Numerous plants currently operating at home and abroad experience serious deterioration of pipe racks (Figure 1), which requires a method of determining the optimal time to repair and upgrade these structures [20]. In addition, the industrial complex is expanding its number of pipe racks (Figure 2), which support pipelines directly responsible for its safety. Thus, without an accurate evaluation for the current condition of operating structures, damage to pipe racks (i.e., the aging and deterioration of pipe racks) will lead to serious accidents that lead to loss of life and property.

Furthermore, although industrial plants are subject to various administrative regulations, they have no established integrated management system that can monitor or promote information sharing regarding the status of equipment safety between related companies and organizations. To reduce overall administrative and operating costs, these entities require the development and application of WSN technology to the health monitoring and evaluation of industrial plant facilities. With the introduction of WSN technology to the health monitoring system of pipe racks, we could reduce the risk of serious accidents.

Using a reliable and secure WSN, we constructed a monitoring system of pipe-rack structures, a current large-scale petrochemical plant located at the Yeosu National Industrial Complex in Korea. We performed operating tests on the system for three months from March 2012 to June 2012. The

(a) ISA100.11a field device (b) ISA100.11a field network gateway

Figure 3: ISA100.11a standard-based network system developed in this study.

advantage of the monitoring system developed in this study is its capabilities of detecting and diagnosing abnormalities in pipe racks before they pose a risk. Detailed technology and installation of the integrated health monitoring system for the pipe racks will be explained in the following Section.

## 3. WSN-Based Health Monitoring System of Pipe Racks

One of our research goals is to construct a marketable WSN prototype of pipe-rack health monitoring network system. We develop the WSN system, which supports an ISA100.11a (ISA100.11a is an open wireless networking technology standard developed by the ISA. The official description is "Wireless System for Industrial Automation: Process Control and Related Applications.") standard for reliable and secure data transmission. The architecture of the ISA100.11a system can combine a variety of functional entities such as field device, field gateway, backbone router, and system manager and thus provide flexibility to various network topologies according to the requirements of the application.

*3.1. Field Device and Field Network Gateway.* In contrast to automation and monitoring applications originating from enterprise and home environments, those of industrial plants have specific requirements such as strict delay requirements, deterministic performance guarantees, and network security. To fulfill these requirements, we developed an ISA100.11a standard-based network system composed of a field device and a field network gateway, shown in Figure 3.

The field device platform, especially designed to meet the requirement of ISA100.11a standard, is equipped with an RF amplifier and antenna diversity functions to eliminate radio fading and high-precision RTC for the time division multiple access (TDMA) operation. The field device also consists of a small IC chip in which detection, signal processing algorithm, and data transmission modules coexist as built-in units. The field device, which calls signals and performs data acquisition and processes itself, is loaded with a low-power measuring device, a microprocessor, and an RF transmitter.

Table 1: Major characteristics of the field device.

| Characteristic | Unit | Min. | Type | Max. |
|---|---|---|---|---|
| Operating frequency | GHz | 2.4 | | 2.483 |
| Operating voltage | V | | 3 | |
| Operating temperature | °C | −40 | | +85 |
| Power supply current sleep | uA | | <10 | |
| Transmit mode (10 dBm) | mA | | 85 | |
| Receive mode | mA | 45 | | |
| Sensitivity for 1% PER | dBm | −107 | | |
| Maximum output power | dBm | 18 | | |
| Over the air data rate | Bps | 250 K | | 2 M |

The microprocessor controls all functions, including signal measurements, and executes the analysis algorithm. The measured signals and the analyzed results are transmitted to a remote server through RF transmission module IEEE 802.15.4 PHY. The block diagram of the filed device platform is shown in Figure 4, and the major characteristics are given in Table 1.

ARM Coretex-M3 [21, 22] is used as the main processor, and the RF transceiver consists of a four-wire SPI interface. Monitoring of the data and movement of each platform is performed through RS-232 communication. The interface is made for JTAG and SPI, which are used for programming the board of the field device. The MFC interface includes a two-pin connector designed to act as an A/D transformation port and execute GPIO through the ATmega128 setting so that the MFC interface can be used as an all-purpose interface. The Silicon Serial Number IC used for the ID of each board reads data only through a 1-wire interface.

The field network gateway is responsible for application layer connectivity between the field devices and the plant network. It also allows interaction between devices utilizing ISA100.11a protocols and system non-ISA100.11a (legacy or foreign) protocols. Our gateway system shown in Figure 3(b) operates on the Linux-based platform, and it connects WSN mesh networks to the Internet via Wi-Fi/Ethernet interface.

Figure 4: Block diagram of the field device.

Figure 5: Configuration of the WSN health monitoring system for pipe racks.

Figure 6: Locations of the sensors for the health monitoring of pipe rack.

*3.2. Transmission Protocol.* For wireless transmission, we adapted the ISA100.11a protocol, which consists of wireless field devices, field network gateways, and system manager. At the physical layer, the field network gateway and all the devices use an IEEE 802.15.4-compatible radio transceiver that uses the 2.4 GHz ISM band at a data transmission rate of 250 Kbits/s. The field network gateway has both a wired and a wireless connection. The wireless connection is used to communicate with the field devices while the wired connection of the gateway is used to communicate with the servers.

The medium access control (MAC) layer is based on the TDMA and channel hopping for reliable data transmission. The accelerometer, inclinometer, and strain gauge data are transmitted in a series of rounds, and each round has a fixed number of packets to transfer, called the "window size." For example, if we have 100 packets and a window size of 5, the 100 packets are divided into 20 rounds. Only one acknowledgment is transmitted back to the sender, and lost packets in each round are retransmitted by looking at the lost-packet information in the acknowledgment. Once the receiver has acquired every packet in the current round, the sender and receiver can move on to the next round. This prevents any packet collisions in the networks. Moreover, we used a specific parameter, an RF group ID, to avoid radio frequency interference from surrounding wireless devices or systems. When the data packet is transmitted, the gateway will check whether it contains the same group ID or not. The transmission protocol is designed to meet both reliable communication and power efficiency requirements.

## 4. Implementation of the WSN Monitoring System

Using WSN-based sensors and network modules, we constructed a pipe-rack health monitoring system at an industrial complex. The sensors include accelerometers, an inclinometer, a strain gauge, and a closed-circuit television (CCTV). For the reliable acquisition and transmission of the data obtained from the sensors, the proposed pipe-rack health monitoring system entails FD nodes and FNG gateways.

*4.1. Construction of the Proposed System.* We have established a WSN-based health monitoring system of the pipe-rack structures in the eighth segment district of the Yeosu National Industrial Complex. Figure 5 illustrates the configuration of the WSN health monitoring system applied to pipe rack. To monitor and detect the physical vibration and deformation of pipe-rack structures, this study installed WSN-based accelerometers, inclinometers, and strain gauge sensors, shown in Figure 6. Table 2 summarizes information pertaining to the installation and the measurement of sensors. The average communication distance of sensor nodes is

(a) Data-logger (b) Gateway

Figure 7: Data-logger and gateway of the pipe-rack health monitoring system.

(a) Accelerometer (b) Strain Gauge (c) Inclinometer (d) CCTV

Figure 8: Sensors used in the pipe-rack health monitoring system.

Figure 9: An example of variations in strain values at the bottom of the pipe-rack structure.

Figure 10: An example of variations in title angles at the top of the pipe-rack structure.

Table 2: Summary of the sensors installed at the pipe racks.

| Sensors | Numbers | Measurement frequency | Communications | Locations |
|---|---|---|---|---|
| Accelerometer | 3 | 256/1 sec. | Wibro | Top, Middle, Bottom |
| Strain gauge | 1 | 1/1 min. | ISA | Bottom |
| Inclinometer | 1 | 1/1 min. | ISA | Top |
| CCTV | 1 | Real time | Wibro | Top |

Figure 11: Acceleration and spectrogram data at the bottom of the pipe rack during a day on May 4, 2012.

approximately 300 meters, and power consumption is 60 mA and 25 mA for transmit and receive currents, respectively. The running time, which depends on transmission period, is estimated to be about 3 years with a battery of 34000 mAh in this study. A data-logger that receives signals from the sensors transmits the data to a main system through the gateway instrument, shown in Figure 7. In addition, a CCTV, installed at the top of the pipe rack, records the visual conditions and situations of the health monitoring system and the pipe-rack structures. The images recorded on the CCTV are also transmitted through the gateway and then stored along with the sensor signals in the main system.

*4.2. Specifications of the Sensors.* To monitor operational and abnormal vibration in the structure, we installed three Lance accelerometers at the top, the middle, and the bottom of the pip-rack structure. The accelerometers measure a range of −0.5 g to 0.5 g and represent an electrical signal of 10 mV per 1 g acceleration. At the bottom of the pipe-rack structure, we monitored variation in the strain, which represents the stress condition of the structure using a strain gauge (manufactured by Tokyo Sokki Kenhyuio Company Ltd.), which exhibited the highest level of stresses in the structural analysis. In addition, to measure changes in the angle of the structure, we installed an inclinometer at the top of the pipe rack. The inclinometer sensor, manufactured by Digital Advanced Sensors, can measure tilt angles in two directions at the same time. Then we installed a WonWoo Eng EWSJ-330 model CCTV with an optical 33-time zoom at the top of the pipe rack to facilitate an operator's visual inspection and improve the efficiency of the monitoring system. Figure 8 shows the sensors, accelerometer, strain gauge, inclinometer, and CCTV used in this study.

*4.3. Monitoring Data.* From the WSN-based accelerometer, inclinometer, and strain gauge sensors, we monitored ambient variations in the pipe-rack structure during the operation of the industrial equipment. Changes in the strain and the slope of the structure were measured using a strain gauge and an inclinometer, installed at the bottom and the top of the structure, respectively, at intervals of 60 seconds. Figures 9 and 10 present the strain and the tilt measurements obtained from the strain gauge and the inclinometer for the selected three days. As shown in the figure, the values of both strain and tilt angles increased as the plant began operation and the daily temperature rose. In particular, the highest values occurred around 3 PM when the daily temperature was the highest. The trends of daily changes in the values were similar on other days.

Figure 11 shows a variation in ambient acceleration measured at 256 Hz intervals and the spectrogram obtained from a WSN-based accelerometer installed at the bottom of the pipe rack. The amplitude of the acceleration exhibits larger values between around 8 AM and 8 PM when the plant is operating. The frequency of the structure during the working hours was in the range of 35 Hz to 50 Hz, depending on working conditions. During the night when the plant is not fully operating, the frequency was almost constant at around 49 Hz.

## 5. Conclusions

In this paper, we proposed a WSN-based health monitoring system applicable to industrial plants that require high reliability and security. For reliable and secure requirements in data acquisition and transmission, this study developed an ISA100.11a standard-based network system composed of a field device and a field network gateway. The proposed monitoring system, which was based on WSN-based sensors and network systems, was established and tested at a large industrial complex, where the system monitored the operational condition of the pipe-rack structure to detect abnormal conditions of the structure before they pose a risk. The system generated data that showed ambient conditions and variations in the pipe-rack structure triggered by environmental and working conditions. The findings from this research show that the WSN health monitoring

system is capable of monitoring and evaluating the health and/or abnormal conditions of facilities. In the future, with abundant reliable and accurate field data, we should be able to provide more efficient risk management during the operation of industrial plants and contribute to the improvement in detection technology and the automation of management.

## Acknowledgments

This work is a part of the projects "Smart Plant Safety Framework based on Reliable-Secure USN," supported by the IT R&D program of MKE/KEIT (no. 2010-10035310) and "Development of an Integrated Design Solution based on Codes and Field Data for the Advancement of the Plant Industry (no. 10040909)," supported by the Korea Government Ministry of Knowledge Economy.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 709208, 17 pages
doi:10.1155/2012/709208

*Research Article*

# Multiscale Acceleration-Dynamic Strain-Impedance Sensor System for Structural Health Monitoring

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Received 21 July 2012; Accepted 17 September 2012

Academic Editor: Ting-Hua YI

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A multiscale wireless sensor system is designed for vibration- and impedance-based structural health monitoring. In order to achieve the objective, the following approaches are implemented. Firstly, smart sensor nodes for vibration and impedance monitoring are designed. In the design, Imote2 platform which has high performance microcontroller, large amount of memory, and flexible radio communication is implemented to acceleration and impedance sensor nodes. Acceleration sensor node is modified to measure PZT's dynamic strain along with acceleration. A solar-power harvesting unit is implemented for power supply to the sensor system. Secondly, operation logics of the multi-scale sensor nodes are programmed based on the concept of the decentralized sensor network. Finally, the performance of the multi-scale sensor system is evaluated on a lab-scale beam to examine the long-term monitoring capacities under various weather conditions.

## 1. Introduction

Many researchers have developed novel sensing technologies for the practical structural health monitoring (SHM) applications. The SHM system for civil infrastructures mainly includes a number of sensors, a huge amount of signal transmitting wires, data acquisition instruments, and centralized data storage servers [1–3]. Also, the stored data in the centralized servers should be handled for offline information analysis. In order to reduce high-tech labors and costs associated with the wired SHM system, many researchers have attempted to adopt wireless sensors [4–10]. One of great advantages for using wireless sensors is autonomous operations for SHM, which can be implemented by embedding system technologies.

The development of wireless sensor nodes as much as the selection of embedding SHM algorithms are important topics for the autonomous SHM [11–15]. To date, many damage monitoring algorithms have been developed for detecting the location and the severity of damage in structures [16–20]. Most of those algorithms are dependent on structural types, damage characteristics, and available response signals that are related to external loadings and environmental conditions.

Since 1990s, several researchers have focused on using vibration characteristics of a structure as an indication of its structural damage [21–24]. Acceleration response of a structure is usually measured to obtain modal parameters such as natural frequency and mode shapes which are utilized for damage detection. It were demonstrated that curvature (or strain) mode shapes are sensitive to structural damage in beam structures [19]. However, computational differentiation of mode shapes with assumption in boundary condition is required to obtain curvature mode shapes from acceleration response. This may lead to less or more errors in estimation of curvature mode shapes. Alternatively, curvature mode shapes directly extracted from strain response are much more accurate. Nevertheless, using strain response, the curvature mode shapes are sensitive only to damage nearby sensor [25]. Therefore, in a hybrid concept, damage detection results would be more accurate and reliable by the combined usage of acceleration and strain responses [26].

Based on the previous works, however, vibration-based approaches cannot easily distinguish multiple damage types

unless the information on real damages is known. The pattern of one damage type is hard to be distinguished from the other since the change in vibration characteristics may be attributed to the damage types involved in the structure. Therefore, other nondestructive evaluation techniques which are complementary to vibration-based approaches should be sought. Recently, electromechanical impedance-based monitoring has shown the promising success to detect the minor incipient change in structural integrity at local subsystems [27–31]. Compared to vibration-based approaches, the impedance-based method has the capability of more precisely localizing damage. Moreover, its local monitoring does not characterize the entire structure, which means the global healthy state would not be easily captured to couple with the local monitoring information. Using those characteristics of the impedance-based methods, Kim et al. [32] first proposed a combined SHM system with global vibration-based techniques and local impedance-based techniques. Also, Kim et al. [20] proposed a serial hybrid SHM scheme using the global and local techniques for sequentially monitoring of damage in PSC bridges.

This paper presents a multiscale wireless sensor system which is designed for vibration- and impedance-based SHM. Firstly, smart sensor nodes for vibration and impedance monitoring are designed. In the design, Imote2 platform which has high performance microcontroller, large amount of memory, and flexible radio communication is implemented to acceleration and impedance sensor nodes. Acceleration sensor node is modified to measure PZT's dynamic strain along with acceleration. A solar-power harvesting unit is implemented for power supply to the sensor system. Secondly, operation logics of the multiscale sensor nodes are programmed based on the concept of the decentralized sensor network. Finally, the performance of the multiscale sensor system is evaluated on a lab-scale beam to examine the long-term monitoring capacities under various weather conditions.

## 2. Multiscale Wireless SHM System

An efficient SHM system must have the capability to monitor structural properties in different scales to guarantee the designated behaviors of the structure as well as specified structural components. Also, the change in structural characteristics due to environmental perturbation must be examined to distinguish environment effect from damage-induced effect. Additionally, the installation and operation of the SHM system should be convenient, cost-efficient, and enabled for long-term monitoring with minimized human engagement.

Considering those requirements, this study presents a multiscale wireless SHM system as schematized in Figure 1. The system is implemented with *wireless/autonomous operation unit* so that the structural responses are automatically measured, and the data are wirelessly transmitted to the base station. The sensor system can be alive for long time with *solar-power unit* which harvests solar energy and supplies power to sensors. Also, the environmental temperature

Figure 1: Schematic of multiscale wireless SHM system.

Figure 2: Schematic of multiscale sensor node.

is monitored with *temperature monitoring unit*. In order to examine structural health in different scales, three types of structural responses which are acceleration, dynamic strain, and electromechanical (E/M) impedance are monitored. Global structural changes are monitored using acceleration response since it represents global behavior of structure. Meanwhile, local structural changes are monitored using dynamic strain response since it is sensitive to local behavior of structure. For specified members, the E/M impedance responses are monitored by PZT sensors since E/M impedance is very sensitive to any mechanical change around the sensor.

## 3. Hardware Design of Multiscale Sensor System

*3.1. Schematic of Multiscale Sensor Node.* According to the concept of multiscale wireless SHM system, an acceleration-dynamic strain-impedance sensor node on Imote2 platform was designed as schematized in Figure 2. The high-performance sensor platform, Imote2, provided by Crossbow Technology [34] was selected to control the operation of the sensor node. For vibration monitoring, SHM-A, SHM-AS, and SHM-H sensor boards were selected. The SHM-A and SHM-H sensor boards were developed for acceleration measurement by University of Illinois at Urbana-Champaign (UIUC)

Figure 3: Prototype of multiscale sensor node on Imote2 platform.

Figure 4: Prototype of Imote2 sensor platform [34].

Table 1: Comparison of sensor platforms.

| Feature | Mica2 [33] | WSN [6] | Imote2 [34] |
|---|---|---|---|
| Clock speed | 7.4 MHz | 0 MHz–8 MHz | 13 MHz–416 MHz |
| Flash memory | 128 kB | 128 kB | 32 MB |
| SRAM | — | — | 256 kB |
| SDRAM | 4 kB | 128 kB | 32 MB |
| Radio frequency | 2.4 GHz | 900 MHz | 2.4 GHz |
| Data rate | 38.4 kbps | 57.6 kbps | 250 kbps |
| Outdoor range | 150 m | 300 m | 150 m |
| Radio power | 31 mW (TX) | 230 mW (TX) | 52 mW (TX) |
| | 22 mW (RX) | 145 mW (RX) | 59 mW (RX) |

[35, 39]. The SHM-H is utilized to measure low-amplitude acceleration response. For measuring higher amplitude acceleration response, the cheaper sensor board SHM-A is employed. The SHM-AS sensor board was modified from SHM-A sensor board in order to additionally measure PZT's dynamic strain signal. For impedance monitoring at critical structural components, impedance sensor board (SSeL-I) developed by Pukyong National University (PKNU) [36] was also selected. As shown in Figure 1, the solar-powered energy harvesting is implemented by employing solar panel and rechargeable battery. Figure 3 shows the prototype of the multiscale sensor node which consists of four layers as (1) X-bow battery board, (2) Imote2 sensor platform, (3) SHM-H board or SHM-A (AS) board, and (4) SSeL-I board.

*3.2. Imote2 Sensor Platform.* For the multiscale sensor node, a sensor platform should be selected based on the capabilities of microcontroller, memory, and wireless radio. Table 1 gives the comparison of three sensor platforms including Mica2 [33], wireless sensor node (WSN) [6], and Imote2 [34]. As summarized in the table, the Imote2 has high performance microcontroller and large memory as compared to Mica2 and WSN. Lynch et al. [6] selected a wireless radio using 9XCite-900 MHz which has the outdoor line-of-sight range up to 300 m. However, only radio frequency of 2.4 GHz is allowed to be used outdoor in Korea. Therefore, the Imote2 platform was selected for this study.

The prototype of Imote2 sensor platform is shown in Figure 4. It is built with 13–416 MHz PXA271 XScale processor. This processor is integrated with 256 kB SRAM, 32 MB flash memory, and 32 MB SDRAM. It is also integrated with many I/O options such as 3 × UART, I2C, 2 × SPI, SDIO, I2S, AC97, USB host, Camera I/F, GPIOs. Therefore, Imote2 platform is very flexible in supporting different sensor types, ADC chips, and radio options. A 2.4 GHz surface mount antenna which has a communication range of about 30 m

Table 2: Comparison of vibration measurement systems.

| Hardware | Feature | SHM-H | SHM-A (AS) | Commercial PCB |
|---|---|---|---|---|
| Accelerometer | Model | SD1221L-002 | LIS344ALH | PCB393B04 |
| | Sensitivity | 2 V/g | 0.66 V/g | 1 V/g |
| | Input range | ±2 g | ±2 g | ±5 g |
| | Output noise | 5 $\mu$g/$\sqrt{}$Hz | 50 $\mu$g/$\sqrt{}$Hz | 0.3 $\mu$g/$\sqrt{}$Hz |
| Data acquisition | Model | QF4A512 | QF4A512 | NI6036E |
| | ADC resolution | 16 bits | 16 bits | 16 bits |
| Signal conditioner | Model | QF4A512 | QF4A512 | PCB481A03 |
| | Filter type | Digital filter | Digital filter | Digital filter |
| Dimensions | | 57 × 37 × 8 mm | 45.8 × 45 × 8 mm | 483 × 413 × 135 mm |
| Weight | | 12 g | 10 g | 7.8 kg |
| Total cost | | $US 400 | $US 100 | $US 20,000 |

Figure 5: Schematic of SHM-H sensor board [35].

is equipped for each Imote2 platform. In addition, an SMA connector is soldered directly to the board for connecting to an external antenna in case of longer communication range is desired. The Imote2 platform connects with sensor boards and battery board by basic connectors.

*3.3. SHM-H Sensor Board for Acceleration Measurement.* For large civil infrastructures where their vibration is very small, highly sensitive vibration sensor must be employed to acquire structural response. For that, the high-sensitivity SHM-H sensor board was developed by Jo et al. [35]. Comparison of the SHM-H and a commercial PCB system is shown in Table 2. The sensor board has relative low input range and relative high noise density. But their dimensions, weight, and cost are much lower than the commercial PCB system.

As schematized in Figure 5 and listed in Table 2, the SHM-H includes several key components such as accelerometer, antialiasing filter and analog-to-digital converter (ADC). It employs a SD1221L-002 accelerometer [40] for high-sensitivity channel, which has input range ±2 g, sensitivity 2 V/g, and output noise 5 $\mu$g/$\sqrt{}$Hz. It also employs a LIS344ALH accelerometer for two normal channels, which has input range ±2 g, sensitivity 0.66 V/g, and output noise 50 $\mu$g/$\sqrt{}$Hz. In addition, it has a Sensirion SHT11 digital relative temperature and humidity sensor. A 4-channel 16-bit high-resolution ADC with digital antialiasing filters (QF4A512) is adopted to convert analog signal to digital data by 16 bit resolution (12 bit resolution is guaranteed through oversampling and averaging process). By adopting the customizable digital filters, the sensor board provides user-selectable sampling rates and cutoff frequencies that can meet a wide range of applications for civil infrastructure monitoring.

*3.4. SHM-A (AS) Sensor Board for Acceleration and PZT's Dynamic Strain Measurement.* For monitoring vibration responses of structures or structural components with large vibration magnitude, a cheaper acceleration sensor board with lower sensitivity, SHM-A, can be used. The SHM-A sensor board was developed by Rice et al. [39]. As listed in Table 2, the components of this sensor board are similar to those of the SHM-H sensor board, except the high-sensitivity accelerometer. For acceleration measurement, the SHM-A employs the triaxial LIS344ALH accelerometer of which its sensitivity is relatively lower and output noise is quite higher than the SHM-H.

In this study, a modified SHM-AS sensor board was designed to measure PZT's dynamic strain signals, additionally. The principle of PZT as the passive strain sensor is that electrical displacement (related directly to electrical current)

Figure 6: Schematic of SHM-A (AS) sensor board.

(a) Schematic (b) Prototype

Figure 7: Schematic and prototype of SSeL-I sensor board [36].

is induced since a mechanical stress (or strain) is applied to a PZT material. More detailed explanation of PZT's dynamic strain can be found in [41].

The schematic of SHM-AS sensor board is shown in Figure 6. This sensor board was modified from SHM-A sensor board. First, the dynamic strain signal from PZT sensor is passed through a signal conditioner circuit to produce an analog signal of 0–3.3 V. Then, the external channel (i.e., channel 4) on SHM-A sensor board is hooked up for measuring the analog signal which is processed by the ADC. Note that the input range 0–3.3 V is required to be maintained for the external channel on SHM-A sensor board.

*3.5. Impedance Sensor Board.* For damage detection of critical local region, electromechanical impedance of a structure is monitored. The changes of electromechanical impedance represent the changes of structural properties which are caused by damages [20, 27, 29]. In this study, the SSeL-I sensor board developed by Kim et al. [36] was selected for impedance-based SHM. The SSeL-I sensor board was designed on the basis of original impedance sensor nodes presented by Mascarenas et al. [42] and Park et al. [43]. As schematized in Figure 7, the sensor board consists of an AD5933 impedance converter, a connector to PZT patches, a temperature and humidity sensor SHT11, and two connectors to the Imote2 sensor platform. The microcontroller PXA271 and wireless radio CC2420 on the Imote2 platform are utilized for controlling impedance measurement and data transmission, respectively.

The core component of the sensor board, AD5933 impedance converter, has the following embedded multifunctional circuits: function generator, digital-to-analog converter, current-to-voltage amplifier, antialiasing filter, ADC, and discrete Fourier transform (DFT) analyzer. With measurable range from 1 kHz to 100 kHz, this chip converts real and imaginary of impedance signatures at a target frequency and transmits these values into the microcontroller [44]. As outlined in Table 3, the specifications of the SSeL-I sensor board is compared with those of the commercial impedance analyzer HIOKI3532. Note that the cost of the SSeL-I sensor board is much lower than the HIOKI3532.

*3.6. Solar Power Harvesting Unit.* The use of disposable battery is available for powering smart sensor nodes. However, it needs to be regularly replaced for long-term usage. In order to deal with power supply issue, especially for long-term operation of smart sensor nodes, energy harvesting is essential. Among the natural energy sources (i.e., solar energy, wind energy, and vibration energy), solar energy is a valuable selection for Imote2 platform. The solar power harvesting system consists of a solar panel and a rechargeable battery.

(a) (b)

Figure 8: Solar power harvesting components.

Table 3: Comparison of impedance measurement systems.

| Feature | Imote2/SSeL-I | Commercial HIOKI |
|---|---|---|
| Model | AD5933 | HIOKI3532 |
| Impedance range | 1 kΩ–10 kΩ | 10 kΩ–200 kΩ |
| Frequency range | 1 kHz–100 kHz | 42 Hz–5 MHz |
| Excitation voltage | 1.98 $V_{p\text{-}p}$ | 14 $V_{p\text{-}p}$ |
| Dimensions | 45 × 45 × 12 mm | 352 × 323 × 124 mm |
| Weight | 12 g | 6.5 kg |
| Cost | $US 300 | $US 15,000 |

Table 4: Power consumption (mW) of Imote2/SHM-A(AS)/SHM-H/SSeL-I.

| Prototype | Operation | | |
|---|---|---|---|
| | Deep sleep | Basic operation | Sensing |
| Imote2/SHM-A (AS) | 8 | 184 | 672 |
| Imote2/SHM-H | 8 | 216 | 704 |
| Imote2/SSeL-I | 8 | 212 | 248 |

To harvest solar energy, the original hardware of X-bow battery board must be modified as Miller et al. [37].

In order to select appropriate solar power harvesting components, power consumption of the three sensor nodes is examined. The power consumption of each prototype sensor node (i.e., Imote2/SHM-A (AS), Imote2/SHM-H, and Imote2/SSeL-I) is listed in Table 4. Among the three prototypes, Imote2/SHM-H is most consumed power. Rechargeable battery must be selected in order to supply power enough for long-time operation of the sensor nodes. Due to the requirement for solar panel integrated with the Imote2 platform such as output voltage range 4.6–10 V and output current range 115–1400 mA, SPE-350-6 solar panel (9 V, 350 mA) provided by SolarMade is a suitable selection. In addition, Powerizer Li-polymer rechargeable battery which has the capacity of 10000 mAh, normal voltage of 3.7 V which can be charged up to 4.2 V, and contains a protection circuit is also employed. Figure 8 shows SPE-350-6 solar power and Li-ion Battery used for the multiscale sensor node.

When solar energy is available, it is assumed that the solar panel harvests in an hour the minimum voltage of 4.6 V, minimum current of 115 mA, and minimum power of 529. This power is sufficient to operate the Imote2/SHM-A (AS), Imote2/SHM-H, and Imote2/SSeL-I in about 47 minutes, 45 minutes, and 128 minutes, respectively. Note that it takes only about 2 minutes for one impedance measurement with 501 sweeping points. When solar energy is not available, the full-charged battery can still supply enough power in 60 hours, 55 hours, and 160 hours to the Imote2/SHM-A (AS), Imote2/SHM-H, and Imote2/SSeL-I, respectively.

## 4. Embedded Software for Multiscale Sensor System

*4.1. Software for Solar Power Harvesting Unit.* For solar power harvesting, *ChargerControl* component from ISHMP Services Toolsuite [45] is employed. This component is developed to check battery voltage and to control charging process. If the battery voltage is less than 3.9 V or the charging voltage is adequate (more than 4.1 V), the charging mode will be initiated. If the battery voltage is sufficient, the Imote2 goes to sleep mode. The *ChargerControl* component works in conjunction with *SnoozeAlarm* component which frequently checks charging voltage and battery voltage for the efficient charging process. More details about the solar power harvesting system can be found in [37].

*4.2. Vibration-Based SHM Software.* As schematized in Figure 9, device drivers of vibration-based SHM are program-med for the Imote2/SHM-A(AS)/SHM-H according to UIUC ISHMP Service Toolsuite and PKNU SSeL (Smart Structure engineering Lab) SHM Tools [43]. The sensor nodes are embedded with the following key components: (1) *RemoteSensing* for synchronized vibration measurements, (2) *AutoMonitor* for autonomous operation, and (3) *VibrationMonitoring* for vibration-based damage detection.

After finishing vibration measurements, all the measured data from leaf nodes are transmitted to a gateway node. Then the data is processed to feature extraction and damage detection from SSeL SHM tools. The SSeL SHM tools include

Figure 9: Vibration-based SHM software.

Figure 10: Flowchart of *RemoteSensing* component [37].

a device driver for ADC and mathematical functions for damage monitoring such as power spectral density (PSD) and correlation coefficient (CC) of PSDs. Also, the modal identification method, stochastic subspace identification (SSI) algorithm, is embedded into the system to extract modal parameters such as natural frequencies and mode shapes.

The *RemoteSensing* provides a high level of flexibility in the choice of network and sensing parameters. Figure 10 shows the flowchart of the *RemoteSensing* component [37]. The application includes the following four major steps. The first step is network synchronization. The second step is sending measurement parameters from the gateway node to leaf nodes. The third step is data collection. The last step is transferring data back to the gateway node and saving the data on the base station.

The *AutoMonitor* is another advanced component of ISHMP Service Toolsuite that allows autonomous operation of sensor network by combining three components: *ThresholdSentry*, *RemoteSensing*, and *SnoozeAlarm* as shown in Figure 11. *ThresholdSentry* is a component that periodically wakes leaf nodes at predefined time to measure data with the *RemoteSensing* component. *SnoozeAlarm* is a component that puts the leaf nodes in a continuous sleep/wake cycle. The purpose of the *SnoozeAlarm* is power saving. The node uses less than 10% of the power when it is in the deep sleep mode than when it is in an idle awake mode. The interval time to execute the *AutoMonitor* component is defined by user. More detail about the operation of the *AutoMonitor* component can be found in [38].

The *VibrationMonitoring* component is programmed in SSeL-SHM Tools to extract vibration features and to detect

Figure 11: Flowchart of *AutoMonitor* component [38].

Figure 12: Impedance-based SHM software.

damages in structure. Firstly, the change in power spectral density (PSD) of vibration signal can be utilized to detect the change in structural properties. PSD is calculated from Welch's procedure as follows [46]:

$$S_{xx}(f) = \frac{1}{n_d T}\sum_{i=1}^{n_d} |X_i(f,T)|^2, \tag{1}$$

where $X_i(f,T)$ is the dynamic response transformed into frequency domain; $n_d$ is the number of divided segments in time history response; $T$ is the data length of a divided segment.

To estimate the change in PSD due to structural change, correlation coefficient (CC) of PSD is calculated as follows [36]:

$$\rho_{XY} = \frac{E\left[S_{xx}(f)S_{yy}(f)\right] - E[S_{xx}(f)]E\left[S_{yy}(f)\right]}{\sigma_{S_{xx}}\sigma_{S_{yy}}}, \tag{2}$$

where $E[\cdot]$ is the expectation operator; $S_{xx}(f)$ and $S_{yy}(f)$ are the PSDs of two time history signals before and after the change in structural properties, respectively; $\sigma_{S_{xx}}$, $\sigma_{S_{yy}}$ are the corresponding standard deviations of PSDs, respectively.

Secondly, the change in structural properties can also be detected using the change in modal parameters such as natural frequencies. In order to obtain natural frequencies, stochastic subspace identification (SSI) is first performed to extract natural frequencies and mode shapes [47]. Subsequently, the relative change of natural frequency or frequency shift (FS) is calculated as follows:

$$\frac{\delta f_i}{f_i} = \frac{f_i^* - f_i}{f_i} \times 100\%, \tag{3}$$

where $f_i$, $f_i^*$ are the $i$th natural frequency before and after the change in structural properties, respectively.

*4.3. Impedance-Based SHM Software.* As schematized in Figure 12, device drivers of impedance-based SHM are programmed for the Imote2/SSeL-I according to UIUC ISHMP Service Toolsuite and PKNU SSeL (Smart Structure engineering Lab) SHM Tools [43]. The sensor nodes are embedded with the following key components: (1) *Impedance* for impedance measurements, (2) *ImpAutoMonitor* for autonomous operation, and (3) *ImpedanceMonitoring* for impedance-based damage detection.

After finishing impedance measurements, all the measured data from leaf nodes are transmitted to a gateway node. Then the data is processed to feature extraction and damage detection from SSeL SHM tools. The SSeL SHM Tools include a device driver for impedance measurement and mathematical functions for damage monitoring such as root mean square deviation (RMSD) and correlation coefficient (CC) of impedance signatures.

Figure 13 shows the flowchart of the *Impedance* component. The application includes the following three major steps. The first step is sending measurement parameters from

Figure 13: Flowchart of *Impedance* component.

Figure 14: Flowchart of *ImpAutoMonitor* component.

the gateway node to leaf nodes. The second step is impedance measurement. The last step is transferring data back to the gateway node and saving the data on the base station.

The *ImpAutoMonitor* is a component of SSeL SHM Tools that allows autonomous operation of sensor network for impedance monitoring as shown in Figure 14. The component is the combining of *Timer* component and the *Impedance* component. *Timer* is a component that periodically wakes leaf nodes at predefined time to measure impedance data with the *Impedance* component. The interval time to execute the *ImpAutoMonitor* component is defined by user.

The *ImpedanceMonitoring* component is programmed in SSeL SHM Tools to extract impedance features and to detect damages in structure. In order to quantify the change in impedance signature due to the change in structural properties at critical region, root mean square deviation (RMSD), and correlation coefficient (CC) of impedance signatures are calculated as follows:

$$\mathrm{RMSD}(Z, Z^*) = \sqrt{\frac{\sum_{i=1}^{N} [Z^*(\omega_i) - Z(\omega_i)]^2}{\sum_{i=1}^{N} [Z(\omega_i)]^2}}, \tag{4}$$

where $Z(\omega_i)$ and $Z^*(\omega_i)$ are the impedances at the $i$th frequency measured before and after the change in structural properties, respectively, and $N$ denotes the number of frequency points in the sweep.

$$\mathrm{CC}(Z, Z^*) = \frac{E[Z(\omega_i)Z^*(\omega_i)] - \mu_Z \mu_{Z^*}}{\sigma_Z \sigma_{Z^*}}, \tag{5}$$

where $E[\cdot]$ is the expectation operation; $\mu_Z$, $\mu_{Z^*}$ signify the mean values of impedance signatures before and after structural change; $\sigma_Z$, $\sigma_{Z^*}$ signify the standard deviation values of

Figure 15: Experimental setup for lab-scale beam.

Figure 16: Temperature monitoring results.

impedance signatures before and after structural change. It is worth noting that RMSD of impedance is more sensitive to the change in impedance magnitude, whereas CC of impedance is more sensitive to the change in impedance frequency.

## 5. Performance Evaluation of Multiscale Sensor System

*5.1. Target Structure and Experimental Setup.* The performance of the multiscale sensor nodes is evaluated by open-air monitoring tests on a small-scale beam model. The test model is an aluminum cantilever beam with the dimension of 900 × 60 × 10 mm. It was placed outdoors on the third floor of a building so that it could be exposed to various weather conditions (e.g., sun-light and rain). Responses of the beam were measured under long-term ambient vibration condition. As shown in Figure 15, the multiscale sensor nodes, Imote2/SHM-AS/SSeL-I, were arranged on the beam at a location 400 mm distanced from the fixed end. It was connected with a solar panel for energy harvesting purpose.

Vibration signals (i.e., acceleration and dynamic strain) were recorded with sampling rate of 500 Hz (digital cut-off frequency was set of 200 Hz). As shown in Figure 15, for PZT's dynamic strain measurement, a PZT patch, FT-35T-2.8A1, was bonded on the beam at the same location with accelerometer. The PZT patch was connected to external channel (i.e., channel 4) on SHM-AS sensor board. For impedance measurement, a PZT-5A type sensor connected to the SSeL-I sensor board was also bonded on the beam. The impedance signatures between 40 kHz and 60 kHz were measured from the PZT sensor with 500 intervals. For temperature measurement, temperature sensor on the SHM-AS board (i.e., SHT11) was moved outside of the sensor box with extended lines and covered with filter cap making a waterproof. Temperature was also measured by wired thermometer with Kyowa EDX-100A Universal Recorder for comparison. Additionally, supply voltage and charging status were also recorded by the multiscale sensor nodes. The interval time to execute the *AutoMonitor* and *ImpAutoMonitor* was setup as one hour for autonomous operation.

*5.2. Performance of Temperature Sensing Unit.* During ten-day experiment (9th August to 18th August, 2011), temperature was recorded by the multiscale sensor nodes. Figure 16 shows the temperature monitoring results due to the change in weather condition. It is observed that temperature data measured by the sensor nodes shows relatively good agreement with those by Kyowa system. However, the temperature data show significant gap (e.g., 7°C) when temperature went up under sunlight (more than 35°C).

*5.3. Performance of Power Harvesting Unit under Various Weather Condition.* In order to evaluate the performance of

(a) Supply voltage

(b) Charging voltage

(c) Charging current

Figure 17: Power monitoring results.

power supply unit, some parameters such as supply voltage, charging status were recorded by the multiscale sensor nodes during the ten-day experiment. The charge status of solar power harvesting unit is shown in Figure 17. During ten-day experiment, the supply voltage remained around 4.0 V which satisfies the minimum required voltage for Imote2 operating of 3.7 V. As shown in Figures 17(b) and 17(c), the charging process of the solar power harvesting unit worked stably well for long-term monitoring, except on some cloudy days (i.e., 11th, 16th–18th August, 2011). On those days, the charging voltage was relatively low.

*5.4. Vibration Monitoring Performance.* Figures 18 and 19 show acceleration responses and PZT's dynamic strain responses, respectively, measured by the Imote2/SHM-AS sensor node. Under ambient vibration condition, vibration responses of the test beam were recorded for 200 seconds per every one hour. As shown in Figures 18(b) and 19(b), PSDs indicating two modes were calculated by (1) to estimate changes in frequency responses due to the temperature change. Note that, some noise peaks due to electrical effect were also obtained in the PSD of dynamic strain. Therefore, the improvement on sensor node design to eliminate electrical noise from PZT's dynamic strain should be remained as future study.

Figure 20 shows the correlation coefficients (CCs) of PSDs versus temperature changes during the ten-day test period. As shown in Figure 20, the CCs of PSDs measured by the accelerometer were good matching with those measured by the PZT sensor. The CCs of PSDs change little due to the change in weather conditions. However, during rainy times

(a) Time history response (b) Power spectral density

Figure 18: Acceleration responses measured by Imote2/SHM-AS.

(a) Time history response (b) Power spectral density

Figure 19: PZT's dynamic strain responses measured by Imote2/SHM-AS.

(on 9th, 11th and 13th August, 2011), CCs of PSDs were significantly reduced.

Next, stochastic subspace identification (SSI) algorithm was utilized to extract natural frequencies [47]. Figure 21 shows the SSI method's stabilization charts for modal identification of the test beam. Natural frequencies of two first modes are identified similarly by using acceleration (i.e., 9.964 Hz and 52.847 Hz) and PZT's dynamic strain (i.e., 9.964 Hz and 52.796 Hz). Since temperature variation causes the change in structural properties, the changes in natural frequencies due to temperature are monitored to detect structural changes (3). Figures 22 and 23 show the relative changes in natural frequencies versus temperature changes during the long-term period. Both modes 1 and 2 show sensitive changes due to the temperature changes. It is noticed that natural frequencies decrease as temperature goes up and vice versa. The monitoring results show that the PZT's dynamic strain can be used for vibration-based SHM applications.

*5.5. Impedance Monitoring Performance.* As shown in Figure 24, the impedance signals between 40 kHz and 60 kHz were measured from the PZT sensor by the Imote2/SSeL-I sensor node. The impedances in the range between 51 kHz and 52 kHz are sensitive to the temperature change. The real impedances shift to up and left side as the temperature increases. In order to evaluate that effect, the root mean square deviation (RMSD) and correlation coefficient (CC) of impe-dance signatures were calculated by (4) and (5). Ten-day impedance monitoring results are shown in Figure 25. The RMSD of impedance increases as the temperature increases, and vice versa. Meanwhile, the CC of impedance decreases as the temperature increases and vice versa.

Figure 20: Correlation coefficients of PSDs versus various temperatures.

Figure 21: SSI method's stabilization charts for modal identification.

## 6. Concluding Remarks

A multiscale acceleration-dynamic strain-impedance sensor system was developed for vibration and impedance based SHM. Firstly, acceleration-based and impedance-based smart sensor nodes were designed. The hardware and embedded software were designed for the multiscale SHM system. In the design, Imote2 platform is implemented to acceleration and impedance sensor nodes. For strain monitoring, the acceleration sensor node was modified to measure PZT's dynamic strain. A solar-power harvesting unit was implemented for power supply to the sensor system. Secondly, operation logics of the multiscale sensor nodes were programmed based on the concept of the decentralized

(a) Acceleration

(b) PZT's dynamic strain

Figure 22: Relative changes in the first natural frequency versus various temperatures.

(a) Acceleration

(b) PZT's dynamic strain

Figure 23: Relative changes in the second natural frequency versus various temperatures.

(a) Frequency range: 40–60 kHz

(b) Frequency range: 51–52 kHz

Figure 24: Real impedance signatures measured by Imote2/SSeL-I.

(a) RMSD of impedance signatures

(b) CC of impedance signatures

Figure 25: Impedance signatures versus various temperatures.

sensor network. Finally, the performance of the multiscale smart sensor system was evaluated on a lab-scale beam to examine the long-term monitoring capacities under various weather conditions.

From the evaluation tests, the following conclusions have been made. The solar-power supply worked stably well for long-term monitoring. Environmental temperature was accurately monitored by the multiscale sensor system. Global and local behaviors of beam structure with regarding to acceleration, PZT's dynamic strain, and impedance were well monitored by the sensor system under various weather conditions. As a summary, the multiscale smart sensor system showed good performance for the robust long-term vibration and impedance monitoring.

## Acknowledgments

This work was supported by Basic Science Research Program through the National Research Foundation of Korea (NRF)

funded by the Ministry of Education, Science and Technology (2011-0004253). The graduate student involved in this study was also partially supported by the Brain Korea 21 (BK21) program, Korea.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 972893, 11 pages
doi:10.1155/2012/972893

*Research Article*

# Long-Term Deformation Monitoring of Metro-Tunnel Airshaft Excavation during Construction Stage

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Received 2 August 2012; Accepted 24 August 2012

Academic Editor: Hong-Nan Li

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The project of Hangzhou Metro Line 1 is the first metro line of the urban rapid rail transit system in Hangzhou, China, which is one of the largest municipal projects of Hangzhou and is being constructed commencing from March 28, 2007 and expected to be completed by October 1, 2012. This metro line has a total length of 48 km and 34 stations, connecting Hangzhou downtown with the suburban area of the city. Owing to the complex geological condition, harsh construction situation, and immature computational methodology, construction of metro systems is often subjected to considerable sources of uncertainties. To ensure the safety of the adjacent building structures, it is a vital necessity to monitor deep excavations of metro tunnels at their in-construction stage. This paper introduces the instrumentation system for settlement monitoring of a metro-tunnel airshaft of the project of Hangzhou Metro Line 1 during the construction of deep excavation. The long-term settlement data monitored by the measurement markers installed at the surface ground and in the depth direction of the airshaft excavation construction site are analyzed and presented in detail. The obtained results indicate that the settlements at the instrumented locations of the construction site during different construction steps vary steadily in an allowable variation range.

## 1. Introduction

The subway networks have been recognized as the most efficaciously urban rapid rail transit system and are vital in alleviating the urban traffic congestion problem. In the past decade, a large number of underground railway transportation networks have been opened to operation or are being constructed in the metropolises of China. In the construction of metro systems, deep excavations with large excavation areas, great excavation depths, and complex shapes and geological conditions remain a challenging and high risky task in metropolitan regions, especially in soft soil areas [1, 2]. The deformation and stability of deep excavations are highly affected by the soil characteristics, groundwater variations, and surcharge conditions. In the past several years, a few cases of excessive deformation or instability in braced excavations of metro stations occurred in China. In this regard, field instrumentation and performance evaluation of deep excavations during construction stage are of great importance for quality control and safety assurance [3–5].

Health monitoring of civil and transportation infrastructures at both in-construction and in-service stages has gained increasing concerns and achieved rapid progress with the aid of advanced technologies in sensors and sensing networks, data acquisition and communication, signal processing, and data and information management. Numerous long-term monitoring systems have been designed and implemented for civil and transportation infrastructures worldwide such as high-rise buildings, long-span bridges, and high-speed rails [6–9], while the applications of health monitoring technology to underground structures during construction and operation have not been widely exploited and put into practice [10–12]. In this paper, the instrumentation system for settlement monitoring of a metro-tunnel airshaft of the project of Hangzhou Metro Line 1 during the construction of deep excavation is presented, and the long-term settlement data monitored by the measurement markers installed at the

Figure 1: River-crossing metro tunnel and airshafts.

Figure 2: Plan view of southern airshaft excavation.

Figure 3: Cross-section of southern airshaft foundation.

Figure 4: Plan view of deployment locations of settlement measurement markers.

Figure 5: Sectional profile of deployment locations of settlement measurement markers.

Figure 6: Supporting structures of southern airshaft excavation.

surface ground and in the depth direction of the airshaft excavation construction site are examined.

## 2. Deep Excavation of Metro-Tunnel Airshaft

As one of the largest municipal projects of Hangzhou, China, the project of Hangzhou Metro Line 1 is the first metro line of the urban rapid rail transit system in Hangzhou which is being constructed commencing from March 28, 2007 and expected to be completed by October 1, 2012. This metro line has a total length of 48 km and 34 stations, connecting Hangzhou downtown with suburban area of the city. It starts from the south at the Xianghu Station in Xiaoshan District, stretches northwards to the Binjiang Station adjacent to the Qiantang River, crosses beneath the Qiantang River to the Fuchun Road Station, passes through Hangzhou downtown, and ends in the Linping Station, with a branch line ending in Xiasha District which diverges from the main line at the Jiubao Station.

The 2nd construction segment of Hangzhou Metro Line 1 covers from the Binjiang Station to the Jiubao Station with a length of 25 km. In this construction section, a two-lane single-bore shield tunnel has been constructed under the Qiantang River to link the Binjiang Station and the Fuchun Road Station, with two airshafts being settled at both sides of the Qiantang River, as shown in Figure 1. In this study, the deformation monitoring of surrounding areas during the deep excavation construction of the southern airshaft will be presented. Figure 2 illustrates the plan view of the southern airshaft excavation with a depth of 29 m.

## 3. Instrumentation System for Settlement Monitoring

*3.1. Engineering Geological and Hydrogeological Conditions.* According to the geological characteristics of the construction site of the southern airshaft, the foundation of the southern airshaft is divided into 20 soil layers along the depth of the southern airshaft with the aid of the static cone penetration tests. Table 1 lists the physical and mechanical properties of different soil layers of the southern airshaft foundation. Figure 3 illustrates a typical sectional profile of the southern airshaft foundation in the depth direction. Shallow groundwater at the construction site of the southern airshaft is a layer of porosity diving, mainly existing in the surface soil layers and the soil layers ③$_1$ ~ ③$_8$ of the

(a) C2-1~C2-8

(b) C3-1~C3-8

Figure 7: Settlement time histories monitored by measurement markers at surface ground.

sandy silt. This layer of groundwater is supplied by the atmospheric precipitation and the surface water runoff, and the groundwater level is largely affected by seasonal changes and the water level of the Qiantang River. Additionally, the construction site of the southern airshaft has two layers of pore confined water, which are distributed in the soil layer ⑥$_3$ and the soil layers $(14)_1$ and $(14)_2$, respectively.

*3.2. Instrumentation.* To monitor the deformation of the southern airshaft excavation during the construction stage and secure the construction safety, 35 instrumented locations are specified for settlement measurement in parallel with the process of deep excavation construction. Among these monitoring points, 23 settlement measurement markers are located at the ground surface of the construction site, distributing on the circles by taking the central of the southern airshaft excavation as the centre of the circles. The remaining 12 settlement measurement markers are deployed in the depth direction of the southern airshaft excavation, and they are divided into 4 groups in the horizontal direction. The distances between each group of the settlement measurement markers and the underground diaphragm wall of the southern airshaft excavation are 13.5 m, 22.5 m, 40 m, and 78 m, respectively; each group has 3 settlement measurement markers in the depth direction of the southern airshaft excavation with the depths of 20 m, 30 m, and 46 m, respectively. Figures 4 and 5 show the plan view and sectional profile of the deployment locations of the settlement measurement markers distributed at the construction site of the southern airshaft excavation.

## 4. Analysis of Long-Term Settlement Monitoring Results

The settlements at the instrumented locations are measured during the construction of the southern airshaft excavation, and the settlement monitoring data from April 9, 2008 to October 9, 2008 are extracted and examined in this paper. The deep excavation construction of the southern airshaft excavation can be divided into 15 construction steps, that is, step 1—excavation of the fourth soil layer, step 2—construction of the fourth concrete ring-shape purlin, step 3—construction of the lining wall on the second floor underground, step 4—excavation of the fifth soil

Figure 8: Continued.

Figure 8: Evaluated settlements and rebounds of instrumented locations at surface ground during different construction steps.

layer, step 5—construction of the fifth concrete ring-shape purlin and supporting structures, step 6—construction of the lining wall on the third floor underground, step 7—concrete curing, step 8—excavation of the sixth soil layer, step 9—construction of the sixth concrete ring-shape purlin, step 10—concrete curing, step 11—excavation of the last soil layer, step 12—base-plate pouring, step 13—base-plate curing, step 14—lining pouring, and step 15—capping operation. Figure 6 illustrates the sectional profile of the supporting structures of the southern airshaft excavation during construction stage.

*4.1. Analysis of Monitoring Data at Surface Ground.* Figure 7 illustrates the settlement time histories monitored by the measurement markers C2-1∼C2-8 and C3-1∼C3-8 which are instrumented at the surface ground of the construction site of the southern airshaft excavation. It is seen from Figure 7(a) that the accumulated settlements at the locations monitored by the measurement markers C2-1∼C2-8 increase gradually during the whole construction stage of deep excavation of the southern airshaft. The accumulated settlements at the locations monitored by the measurement markers C2-1 and C2-2 are much larger than those at the other locations, and it is because these two settlement monitoring locations are much closer to the southern airshaft excavation than others. A further observation into Figure 7(a) reveals that the settlements during the steps 8 and 11 decline more sharply than those during the other steps, and therefore much attention should be paid to the construction safety control during the excavation of the sixth soil layer (step 8) and the excavation of the last soil layer (step 11).

It can be found from Figure 7(b) that the accumulated settlements at the locations monitored by the measurement markers C3-1∼C3-8 vary nearly in the same trend during the whole construction stage of deep excavation of the southern airshaft. It is also observed from Figure 7(b) that the settlements fluctuate greatly during the step 3 (construction of the lining wall on the second floor underground), the step 4 (excavation of the fifth soil layer), the step 7 (concrete curing), and the step 8 (excavation of the sixth soil layer); while the rebounds dominate the foundation deformation during the step 1 (excavation of the fourth soil layer), the step 2 (construction of the fourth concrete ring-shape purlin), and the step 12 (base-plate pouring). Figure 8 shows the evaluated settlements and rebounds of the instrumented locations at the surface ground of the construction site of the southern airshaft excavation during different construction steps.

*4.2. Analysis of Monitoring Data in Depth Direction.* Figure 9 shows the settlement time histories monitored by the measurement markers F1∼F4 which are installed in the depth direction of the construction site of the southern airshaft excavation. An insight into Figure 9 reveals that the accumulated settlements monitored by the measurement markers installed at the locations in the depths of 20 m and 30 m increase gradually during the whole construction stage of deep excavation of the southern airshaft, while the accumulated settlements monitored by the measurement markers installed at the locations in the depth of 46 m vary gently during the whole construction stage of deep excavation of the southern airshaft. Figure 10 illustrates the evaluated settlements and rebounds of the instrumented locations in the depth direction of the construction site of the southern airshaft excavation during different construction steps.

(a) F1-20~F1-46

(b) F2-20~F2-46

(c) F3-20~F3-46

Figure 9: Continued.

(d) F4-20~F4-46

Figure 9: Settlement time histories monitored by measurement markers in depth direction.

Table 1: Physical and mechanical properties of different soil layers of southern airshaft foundation.

| Number | Soil layer | Thickness of soil layer (m) | Physical and mechanical properties | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Unit weight (kN/m$^3$) | Specific gravity | Porosity ratio | Saturation (%) | Coefficient of permeability (cm/s) | Modulus of compressibility (MPa) |
| ①$_1$ | Miscellaneous fill soil | 0.70~3.10 | 17.5 | — | — | — | $5.0E-03$ | — |
| ①$_2$ | Plain fill soil | 0.30~6.00 | 17.8 | — | — | — | $8.0E-04$ | 6.5 |
| ③$_1$ | Sandy silt | 3.4 | 17.9 | 2.70 | — | — | $4.0E-04$ | 7.0 |
| ③$_2$ | Sandy silt | 3.50~9.70 | 18.4 | 2.70 | 0.87 | 94.2 | $7.0E-04$ | 8.5 |
| ③$_4$ | Sandy silt | 1.00~5.90 | 18.2 | 2.70 | 0.94 | 95.1 | $6.5E-04$ | 5.5 |
| ③$_5$ | Silty sand and sandy silt | 1.60~7.00 | 18.2 | 2.69 | 0.89 | 92.5 | $3.0E-03$ | 7.0 |
| ③$_6$ | Silt | 4.20~10.95 | 18.6 | 2.69 | 0.79 | 89.5 | $4.0E-03$ | 10.0 |
| ③$_7$ | Sandy silt | 0.80~6.20 | 17.9 | 2.70 | 0.95 | 91.0 | $2.0E-04$ | 5.5 |
| ③$_8$ | Silt | — | 18.3 | 2.69 | — | — | $4.5E-03$ | 10.5 |
| ④$_3$ | Silty soft clay | 2.60~6.70 | 17.2 | 2.73 | 1.25 | 96.0 | $3.0E-06$ | 2.6 |
| ⑥$_1$ | Silty soft clay | 1.50~3.70 | 17.2 | 2.73 | 1.16 | 92.2 | $2.0E-06$ | 2.7 |
| ⑥$_2$ | Silty soft clay | 1.00~8.30 | 17.4 | 2.72 | 1.11 | 92.3 | $5.0E-06$ | 2.8 |
| ⑥$_3$ | Silt | 0.50~3.85 | 18.1 | 2.69 | 0.82 | 82.8 | $3.0E-03$ | 8.0 |
| ⑧$_2$ | Silty soft clay | 0.80~8.20 | 17.3 | 2.72 | 1.09 | 90.4 | $4.0E-05$ | 3.0 |
| ⑧$_3$ | Silty-fine sand | 1.80~8.50 | 17.7 | 2.68 | 0.90 | 82.8 | $5.0E-03$ | 8.5 |
| ⑩$_1$ | Silty clay | 1.90~4.30 | 17.4 | 2.73 | 1.09 | 89.7 | $8.0E-06$ | 3.2 |
| ⑩$_2$ | Silty clay | 2.80~4.60 | 19.6 | 2.71 | 0.67 | 94.1 | $4.0E-06$ | 4.5 |
| (14)$_1$ | Medium sand | — | 19.9 | 2.68 | — | — | $6.0E-03$ | 11.0 |
| (14)$_2$ | Rounded pebble | — | 20.8 | 2.65 | — | — | $3.5E-01$ | — |

## 5. Conclusions

Metro-tunnel excavations bring a challenge to the civil engineering community and pose threat to the public safety in metropolitan regions due to the complexity and uncertainty inherent in excavation activities. The stability of deep excavations and adjacent buildings has gained major concerns during metro-tunnel construction. A viable and practical way to ensure the construction safety is by executing real-time monitoring strategy with the aid of advanced

Figure 10: Continued.

Figure 10: Evaluated settlements and rebounds of instrumented locations in depth direction during different construction steps.

sensing and signal processing technologies. In this paper, an instrumentation system for settlement monitoring of a metro-tunnel airshaft of the project of Hangzhou Metro Line 1 during the construction of deep excavation is presented. After examining the long-term settlement data monitored by the measurement markers installed at the surface ground and in the depth direction of the airshaft excavation construction site, the following specific conclusions are drawn: (i) the accumulated settlements at the locations monitored by the majority of the measurement markers at the ground surface increase gradually during the whole construction stage of deep excavation of the southern airshaft, and the rebounds dominate the foundation deformation during several construction steps; (ii) the accumulated settlements monitored by the measurement markers installed at the locations in the depths of 20 m and 30 m increase gradually, while the accumulated settlements monitored by the measurement markers installed at the locations in the depth of 46 m vary gently during the whole construction stage of deep excavation of the southern airshaft; (iii) the obtained results reveal that the settlements at the instrumented locations of the construction site during different construction steps vary steadily in an allowable variation range.

## Acknowledgments

This research work was jointly supported by the Science Fund for Creative Research Groups of the NSFC (Grants no. 51121005), the National Natural Science Foundation of China (Grants no. 51178083 and 51222806), and the Program for New Century Excellent Talents in University (Grant no. NCET-10-0287). The authors also wish to express their thanks to the Hangzhou Metro Group Co. Ltd. for permission to publish this paper.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 902613, 10 pages
doi:10.1155/2012/902613

*Research Article*

# Experimental Investigation of a Self-Sensing Hybrid GFRP-Concrete Bridge Superstructure with Embedded FBG Sensors

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Received 29 June 2012; Accepted 14 September 2012

Academic Editor: Hong-Nan Li

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A self-sensing hybrid GFRP-concrete bridge superstructure, which consists of two bridge decks and each bridge deck is comprised of four GFRP box sections combined with a thin layer of concrete in the compression zone, was developed by using eight embedded FBG sensors in the top and bottom flanges of the four GFRP box sections at midspan section of one bridge deck along longitudinal direction, respectively. The proposed self-sensing hybrid bridge superstructure was tested in 4-point loading to investigate its flexural behavior and verify the operation of the embedded FBG sensors. The longitudinal strains of the hybrid bridge superstructure were recorded using the embedded FBG sensors as well as the surface-bonded electric resistance strain gauges. The experimental results indicate that the embedded FBG sensors can faithfully record the longitudinal strains of the hybrid bridge superstructure in tension at bottom flanges and in compression at top flanges of the four GFRP box sections over the entire loading range, as compared with the surface-bonded strain gauges. So, the proposed self-sensing hybrid GFRP-concrete bridge superstructure can reveal its internal strains in serviceability limit state as well as in strength limit state, and it will have wide applications for long-term monitoring in civil engineering.

## 1. Introduction

A major concern for many bridge superstructures is the significant reduction in durability and life expectancy caused by the corrosion of the reinforcing steel and the corresponding deterioration of the concrete. These problems are accelerated by the application of deicing salts in cold area. The corrosion will eventually cause enough damage to warrant a superstructure replacement or retrofit [1–3]. It implies that it is imperative to develop and construct bridge systems that have long-term durability and require low maintenance. As one of the solutions to this problem, structural applications of fiber-reinforced polymer (FRP) composites have recently been attractive in the civil engineering community due to their superior material properties such as high corrosion resistance, high specific strength, and high specific stiffness [4]. In spite of all these advantages, FRP composites have higher initial costs and lower stiffness (for glass FRP, GFRP) comparing to conventional materials used in infrastructure applications. To overcome these drawbacks and make the best use of materials, hybrid FRP-concrete structures have been considered as the most effective composite form [5–7].

One of the main reasons preventing greater implementation of FRP composites into the civil infrastructure is the lack of performance data and accepted engineering standards. Using sensors and instrumentation, performance data can be obtained that will provide information on the behavior of the structure over a given time. The integrity of the structure can be established at any phase during the service life of the structure to compensate for other parts of it that are weakening. Smart (or self-sensing) structures will be employed to a greater extent in the future than they are currently [8]. In most designs, crushing of concrete in compression defines the flexural failure of hybrid FRP-concrete bridge superstructure. The failure will be sudden, and hybrid bridge superstructures did not show any ductility [9]. For load bearing elements, the brittle failure is not anticipated because there is no safety prewarning before

the structure failure. So, the monitoring of hybrid FRP-concrete bridge superstructure is essential. Now, structural health monitoring (SHM) has become a hot research topic in the civil engineering. For example, Yi et al. proposed some optimal methods to identify and evaluate civil engineering structure [10–14].

The fiber Bragg grating (FBG) technique is a new technology that proposed important applications in both the telecommunication and the sensing fields [15]. Compared with electric resistance strain gauge, FBG sensor has the advantages of electromagnetic insensitivity, wavelength multiplexing, capability, miniature size, high sensitivity, good long-term stability, and high reliability [16]. FBG sensors can also be easily bonded on the surface of the material or embedded into the structure to measure the strain, temperature, or other physical quantities without affecting the integrity of the structure itself [17].

However, there are some problems in the applications of FBG sensors because the bare FBG sensors are very thin and frangible. They should be packaged or protected before applications. FRP materials lend themselves as prime candidates for the rapidly expanding field of research of "smart (or self-sensing) structures." Smart (or self-sensing) structures are the structures which contain the built-in sensing device to continuously monitor the current state and serviceability of the structures. This is referred to as "passive" smart structures, which have many applications in civil engineering [18, 19]. FRP structures are good candidates for making smart (or self-sensing) structures because their fabrication techniques inherently allow for the embedding of FBG sensors and their communication lines.

In this paper, eight FBG strain sensors will be preembedded in a hybrid GFRP-concrete bridge superstructure during its fabrication process. Embedment of FBG strain sensors in the hybrid GFRP-concrete bridge superstructure leads the bridge superstructure to be a self-sensing bridge superstructure. The flexural behavior of the hybrid bridge superstructure will be investigated, and the monitoring performance of the embedded FBG strain sensors in the hybrid bridge superstructure will also be examined.

## 2. Proposed Hybrid GFRP-Concrete Bridge Superstructure System

Although many configurations of the hybrid FRP-concrete bridge superstructure are possible, a conceptual design of the hybrid bridge superstructure shown in Figure 1, where supports and substructures are not shown, was chosen. A prototype bridge was designed as a simply supported, single-span, one-lane bridge with a span length of 10 m. Geometric parameters of the cross-section of the bridge in Figure 1(b) were determined to meet the design limit (serviceability check and strength check) by numerous finite-element analyses. The detailed design procedures and finite-element analyses can be found in the literature [20]. The cross-section of the bridge is comprised of 8 GFRP box sections each of which has a layer of concrete in the compression zone.

Figure 1: The proposed hybrid bridge superstructure (units in mm).

The bridge design is based on the traditional plank bridge concept, in which a number of individual beams are placed side by side to create a bridge. The advantages of this concept include:

(1) corrosion resistance for steel-free design;

(2) initial costs reduction due to the effective use of concrete;

(3) lightweight;

(4) reduction of local deformation under concentrated loads that is found to be a problem in all-FRP bridge;

(5) no joints between decks and girders (the girders are the decks);

(6) significant understanding of the bridge behavior that can be obtained by testing of individual beams;

(7) the ability to be well understood by bridge engineers.

Four beams make up a bridge deck with a width of 2048 mm. The prototype bridge superstructure consists of two bridge decks. The 2048 mm wide bridge deck can be easily transported to site on a standard truck and assembled into a bridge using simple field joints.

## 3. Experimental Program

*3.1. Specimen Preparation.* The test specimen (described in Figure 2) is a one-third scale model of the 10 m hybrid GFRP-concrete bridge superstructure shown in Figure 1. As previously mentioned, the hybrid bridge superstructure consists of two individual hybrid bridge decks (shown in Figure 2(a)). The geometry of cross-section and the locations of the embedded FBG sensors (FBG) and surface-bonded electric resistance strain gauges (SGs) of one hybrid GFRP-concrete bridge deck (test specimen) are shown in Figure 3. Eight FBG sensors were designed to be embedded into the top and bottom flanges of the GFRP part at midspan section of one bridge deck along the longitudinal direction,

(a) Two individual bridge decks

(b) Bridge superstructure

Figure 2: The photographs of the test specimen.

respectively, to form the self-sensing hybrid GFRP-concrete bridge superstructure. The embedded FBG sensors were used to monitor the internal compressive and tensile strains of the bridge superstructure, respectively.

As a trial test, the GFRP part of the test specimen was fabricated by hand lay-up process at present. Two types of stitched bidirectional E-glass fabric with a weight of 600 g/m$^2$, (0°/90°), and (±45°) were selected as the primary continuous reinforcement in this study. In these fabrics, fibers in 0° and 90° or +45° and −45° directions have the same weight (300 g/m$^2$), respectively. Vinyl ester resin, Swancor 901, was selected as the matrix for the GFRP part. The vinyl ester resin was selected because of its high flexibility and impact resistance, its lower cost compared to other resin systems, and its good performance in harsh filed environments. Therefore, it is very suitable for civil engineering structures.

The fabrication process of the proposed self-sensing hybrid GFRP-concrete bridge superstructure is as follows.

Firstly, eight individual GFRP box beams were fabricated. The top flange and two webs of each GFRP box section had the same fiber architecture, which consisted of 12 layers of woven fabric reinforcement. However, the bottom flange of the GFRP box section consisted of 18 layers of woven fabric to increase the global stiffness of the section. Stacking sequences of the top and bottom flanges were $[(90°/0°)_3, (\pm45°)_2, (0°/90°)]_s$ and $[(90°/0°)_3, (0°/90°)_3, (\pm45°)_2, (0°/90°)]_s$, respectively.

Secondly, by combining two sets of four box beams each using vinyl ester resin and chopped strand mat (CSM), two sections with a width of approximately 680 mm were created.

Thirdly, the two sections were wrapped individually with a $[(90°/0°)_2]_s$ outer laminate (shown in Figure 3) to strengthen the transverse stiffness of the sections of the bridge deck. Eight FBG sensors were designed to be embedded into the outer laminate at the top and bottom flanges of GFRP part at midspan section of one bridge deck along the longitudinal direction (shown in Figure 3(b)), respectively. FBG sensors were located at the middle location of the top and bottom flanges of each GFRP box section. Eight electric resistance strain gauges were attached on the surface of GFRP part where FBG sensors were embedded (shown in Figure 3(b)) to investigate the strain sensing properties of the embedded FBG sensors. Stacking sequences of the outer laminate with FBG sensor were $[(90°/0°)_2, \{\text{FBG-}0°\}, (0°/90°)_2]$. It has also been widely documented that for optimum transfer of strain from host material to sensor, the sensor should be aligned parallel to the reinforcing fiber [21]. When sensors were embedded parallel to the direction of reinforcing fiber, resin-rich areas around the sensor gage were prevented. It was assumed that this placement minimized nonuniform stress fields around the sensor, which could reduce measurement accuracy [22]. Stitching/bonding FBG sensors to the fabric reinforcement was the least invasive method to prevent sensor slippages (shown in Figure 4).

Fourthly, concrete was cast on the top surfaces of the two sections, respectively, forming the scaled hybrid GFRP-concrete bridge decks (shown in Figure 2(a)). To develop a good bond between GFRP part and concrete, 5∼10 mm aggregate was applied to the top flange of GFRP part with epoxy adhesive. Applying too much or too little aggregate could create insufficient bond between GFRP and concrete for the two materials to act compositely. The aggregate distribution percentage was recommended to be 35% ∼45% to obtain the optimal bond [23].

Finally, the two bridge decks were assembled together by a simple method (shown in Figure 5), forming the proposed scaled hybrid GFRP-concrete bridge superstructure (shown in Figure 2(b)).

As previously mentioned, the FBG sensors were embedded into FRP parts. So, the embedded FBG sensors could suit for the extensive civil engineering construction without any protective measure. However, the outside optical fiber wire needs additional protective measure. Usually, the Kevlar fiber optic wire was used as a protective measure to ensure its optimal performance and avoid being broken.

The fiber volume fraction of the GFRP part is about 0.3. The material properties of the GFRP part are recorded in Table 1. Due to the size of the test specimen, coarse aggregates with maximum size of 10 mm were used in the concrete. The compressive strength of the concrete on the day of test is 51 MPa from 150 mm × 150 mm × 150 mm concrete cube. According to the Chinese concrete code (GB 50010-2010), Young's modulus of the concrete of 36.46 GPa is obtained by theoretical calculation with the concrete strength of 51 MPa. However, the true Young's modulus of

(a) Geometry of cross-section of one bridge deck

(b) Locations of FBG sensors and strain gauges (SG) in one bridge deck

Figure 3: The proposed self-sensing hybrid GFRP-concrete bridge deck (units in mm).

Table 1: Material properties of GFRP part.

| Type of material test | Module of elasticity/GPa | Poisson ratio | Strength/MPa |
|---|---|---|---|
| Longitudinal tension of bottom flange | 15.14 | 0.18 | 245 |
| Longitudinal tension of top flange and outer web | 14.44 | 0.19 | 240 |
| Longitudinal tension of inner web | 14.05 | 0.21 | 237 |
| In-plane shear of outer web | 3.36 | — | 120 |
| In-plane shear of inner web | 3.69 | — | 120 |

Figure 4: Bonding FBG sensors to the fabric reinforcement.

Figure 5: Connecting details between two hybrid bridge decks.

the concrete will be slightly smaller than the normal concrete due to the use of small size coarse aggregates.

*3.2. Strain Calibration.* The strain sensitivity coefficient of FBG sensors will be different because of the different fiber, writing, annealing, and packaging techniques for FBG sensors. So, the strain sensitivity coefficients of the embedded FBG sensors into GFRP should be determined through strain calibration experiments before their applications.

According to ASTM D3039 standard test procedures, two test specimens with embedded FBG sensors were fabricated and cut from two laminates. Stacking sequences of the two laminates were the same as those of the top and bottom flanges of the GFRP part mentioned in Figure 3, respectively. The dimensions and schematic view of specimen are shown in Figure 6. Aluminum tabs were attached on the specimen to prevent damage of the leads from gripping pressure during the uniaxial tensile test. Before testing, two electrical

Figure 6: Schematic illustration showing the relative positions of FBG sensor and strain gauges and specimen dimension (units in mm).

Figure 7: Relationship between FBG's wavelength and strain for the specimen from top flange.

Figure 8: Relationship between FBG's wavelength and strain for the specimen from bottom flange.

Figure 9: Experimental setup.

strain gauges were bonded side by side on the surface of each specimen where FBG sensor was embedded (shown in Figure 6). The test was conducted on a universal material experimental machine. The FBG interrogation system was provided by MOI Ltd. with model number si-720. In this test, the effect of temperature changes was not considered due to the fact that the experiment was conducted in the laboratory in a short time.

The results of the calibration test are shown in Figures 7 and 8. The strain sensing coefficients of the two test specimens are 1.16 pm/$\mu\varepsilon$ and 1.17 pm/$\mu\varepsilon$ for the top flange and bottom flange of the hybrid bridge superstructure, respectively, which correlate well with that of bare FBG sensors whose value is about 1.2 pm/$\mu\varepsilon$. Based on the precision of si-720 interrogation system, the strain sensing precision of the specimens is near 1 $\mu\varepsilon$. From the two figures, we can see that the shifts of center wavelength of FBG sensors are in a good linear relationship with the strains measured by the strain gauges. And the linear correlation coefficients of the two specimens all reach 0.9999.

*3.3. Test Setup and Loading Protocols.* The test setup is shown in Figure 9. Loads were applied vertically at four points on the top surface of the test specimen by three spreader beams and the screw jack supported by the top beam of a reaction frame. The load configuration simulates the two heaviest axles of the standard truck load (total 5 axles) specified in the Chinese bridge design specifications (JTG D6-2004). In the specifications, the design truck load is a live load and has 5 axles. And the two heaviest axles are specified as 140 kN each, where one axle is 1.4 m away from the other. Each axle has two tires that are 1.8 m apart center-to-center, and each tire area has a length of 0.6 m and a width of 0.2 m. For the 1/3 scale model, this design truck load becomes two axles of 31.12 kN, 0.467 m apart. Two tires of axle are 0.6 m apart, and each tire area is 0.2 m long and 0.067 m wide. The test specimen was simply supported on rollers with a span length of 3333 mm and a support length of 0.2 m at each end. The loading configuration of the flexural test is shown in Figure 10. To protect the bottom surface of the specimen from damage, rubber pads were placed between test specimen and the pin/roller at two ends of the specimen.

Figure 10: Loading configuration of the hybrid bridge superstructure.

As mentioned in Section 3.1, eight FBG sensors were embedded into the midspan section of one hybrid GFRP-concrete bridge deck. Eight surface-bonded electrical resistance strain gauges (shown in Figure 3(b)) were also used to check the accuracy of the output of the embedded FBG sensors. A four-point loading test was conducted to investigate the flexural behavior and the strain response measured from FBG sensors for the proposed self-sensing hybrid GFRP-concrete bridge superstructure. The strain sensing performances of the embedded FBG sensors were recorded by the Micron Optical Sensing Interrogator (MOI Ltd., si-720). And the data of the electric resistance strain gauges were gathered using the Static Strain Acquisition Device (DongHua Test Inc.). In this test, the change of temperature was not considered to simplify the analysis because the test time was short, the temperature of the laboratory almost did not change during the test, and the monitoring values were large (the maximum value was 0.857%, stated in Section 4.2). However, there will be large temperature variation in field application. Due to the fact that FBG senses both strain and temperature simultaneously, temperature compensation for FBG strain sensors of long-term field monitoring system is indispensable. And many solutions to temperature compensation have been developed [24].

The flexural test was performed by force control, and it was divided into two steps: Step I and Step II, shown in Figure 11, in order to evaluate its performance and meet the objectives of the research. In the first step (Step I), force history of a triangular shape with the load amplitude gradually increased was applied to examine the deflection stabilization. In the second step (Step II), force had been increased monotonically until the test specimen failed. In Figure 11, as previously mentioned, "Truck Load" is two axles load of 31.12 kN.

Figure 11: Loading protocols of the test.

## 4. Test Results and Discussions

*4.1. Load-Deflection Response and Failure Mode.* Figure 12 shows load-deflection responses obtained at central point of midspan section of the test specimen during Step I and Step II. No sound of cracking of either the concrete or GFRP was heard during Step I. Visual inspection after the test revealed no cracking in the surface of the test specimen. It can be concluded that the obtained load-deflection response is very much linear and has a good repeatability for the hybrid bridge specimen during Step I.

At the load of near 16 × truck load in Step II, there was a medium loud noise. It is regretted that visual inspection was not conducted in time. So, the noise may be caused by the cracking of the interface between GFRP and concrete or the cracking of concrete under the loading points. And at this point, the slope of load-deflection curve changes slightly. The load was 8.8 times the Chinese bridge design requirement of 1.4(1 + IM) × truck load for live loads in the strength limit state, where IM = dynamic load allowance, and has a value of 0.3 in this case. Since the dead load and other live loads were not considered appropriately in this test, 8.8 cannot be called the safety factor.

Figure 12: Load versus deflection at central point of midspan section.

Figure 12 shows that the loading path is nearly linear until the specimen failed. A global failure as shown in Figure 13 occurred at 28 × truck load, which is 15.4 times the 1.4(1 + IM) × truck load. Crushing of concrete in compression near the loading points defines flexural collapse of test specimen. However, the GFRP part was found to be intact. This can be described as follows. As concrete failed, large deflection occurred immediately due to the low stiffness of GFRP part in comparison to the hybrid section, along with instantaneous reduction of the load for the quick increase of deflection, and the combined compression capacity of the GFRP top flange and the residual strength of the concrete enable the specimen to carry the residual load. After unloading the residual load, the deflection of the specimen almost came back. The failure was sudden due to the nature of the concrete. However, the obtained failure mode is considered to be favorable because it did not lead to collapse of the entire bridge, and the bridge can be retrofitted easily by the replacement of the concrete layer.

As pointed out by the Chinese road authorities, trucks in excess of many times the legal limit have been detected in China. These extreme loads can initiate cracks or cause damage in a bridge structure, which may propagate under subsequent dynamic load, resulting in a significantly decreased life span. However, the proposed bridge model has significant reserve strength. It will be of great benefit for the ubiquity of overload operation in China.

There was no visual failure in the connection between GFRP and concrete before the failure of the bridge superstructure. This phenomenon reflects that using aggregate coating is an efficient method to ensure composite action between GFRP and concrete for the proposed hybrid bridge superstructure.

*4.2. Self-Sensing Performance.* The locations of the embedded FBG sensors and surface-bond strain gauges in the proposed self-sensing hybrid bridge superstructure are illustrated in Figure 3(b). Strains from eight FBG sensors

(a) Top view

(b) Side view

Figure 13: Failure of test specimen.

and eight strain gauges of the self-sensing hybrid bridge superstructure during Step I are shown in Figure 14. Strains from FBG sensors are obtained based on the strain sensing coefficients calibrated in Section 3.2. This figure shows that there is a good agreement between FBG sensor and strain gauge in tensile status at bottom flange of the GFRP part, as well as in compressive status at top flange of the GFRP part. And we can see that the repeatability and consistency of the embedded FBG sensors and surface-bond strain gauges are remarkable. From Figure 14, we can find that the outputs of FBG sensors are slightly smaller than those of strain gauges. The reason is that the locations of them are different in the cross-section of the self-sensing hybrid bridge superstructure (shown in Figure 3(b)). So, the outputs of them should be slightly different based on the assumption of plane section.

Figure 15 shows the measured strains from eight FBG sensors and eight strain gauges in the self-sensing hybrid bridge superstructure during Step II. There is an excellent agreement between the eight FBG sensors and the eight strain gauges in tensile and compressive status over the entire load range, respectively. Because of the same reason previously mentioned, the outputs of FBG sensors are slightly smaller than those of strain gauges. And the maximum tensile and

Figure 14: Strains from eight FBG sensors and eight strain gauges in the self-sensing hybrid bridge superstructure during Step I.

compressive strains recorded by the embedded FBG sensors are 0.857% (from FBG-5) and −0.186% (from FBG-1) at failure of the self-sensing hybrid bridge superstructure, respectively.

We can conclude that the embedded FBG sensors can monitor the internal strain of the proposed self-sensing hybrid GFRP-concrete bridge superstructure over the entire load range from zero to the failure of the bridge superstructure. It indicates that the embedded FBG sensors can monitor the internal strain of the composite bridge superstructure in normal service state as well as in limit state.

## 5. Conclusions

A self-sensing hybrid GFRP-concrete bridge superstructure was developed by embedding eight FBG sensors into GFRP part to monitor the internal longitudinal strains of the proposed hybrid bridge superstructure. Experiment for strain calibration of the GFRP laminates with embedded FBG sensors was conducted firstly on material test system, and the strain sensing property of the laminates is nearly the same as that of bare FBG sensors. Then, a 4-point loading test was carried out to assess the flexural behavior of the proposed self-sensing hybrid GFRP-concrete bridge superstructure and the feasibility of strain monitoring of the embedded FBG sensors. Compared with the surface-bonded electric resistance strain gauges, the FBG sensors can faithfully record the longitudinal internal strain of the hybrid bridge superstructure in tension and in compression over the entire load range. It indicates that the embedded FBG sensors can monitor the internal strain of the hybrid GFRP-concrete bridge superstructure in serviceability limit state as well as in strength limit state. The proposed self-sensing hybrid GFRP-concrete bridge superstructure has a good capability of monitoring its internal strain in its service life and is very suitable for the long-term monitoring.

Figure 15: Strains from eight FBG sensors and eight strain gauges in the self-sensing hybrid bridge superstructure during Step II.

## Acknowledgments

The authors gratefully acknowledge the financial support provided by the National Key Basic Research Program of China (Project no. 2012CB026203), the Natural Science Foundation of China (Project no. 50908030), the National Key Technologies R&D Program of China (Project no. 2011BAK02B03), and China Postdoctoral Science Foundation (Project no. 201003620).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 272190, 10 pages
doi:10.1155/2012/272190

*Research Article*

# Structural Health Monitoring of a Tall Building during Construction with Fiber Bragg Grating Sensors

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Received 29 June 2012; Accepted 22 August 2012

Academic Editor: Jeong-Tae Kim

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Fiber Bragg grating sensors demonstrate a great potential as a structural health monitoring tool for civil structures to ensure structural integrity, durability, and reliability. The advantages of applying fiber optic sensors to a tall building include their immunity to electromagnetic interference and their multiplexing ability to transfer optical signals over a long distance. In the work, fiber Bragg grating sensors, consisting of strain and temperature sensors, are applied to structural monitoring of an 18-floor tall building since the date of its construction. The strain transferring rate from host material to the fiber core is discussed and the calibration of packaged fiber Bragg grating sensor is presented. The main purposes of the investigation are monitoring temperature evolution history within concrete during the pouring and curing process, measuring variation of the main column strains on the underground floor while upper 18 floors were subsequently added on, and monitoring relative displacement between two foundation blocks. Fiber Bragg grating sensors have been installed and integrated continuously for more than five months. Monitoring results of temperature and strain are presented in the paper. Furthermore, temperature lag behavior between concrete and its surrounding air is investigated.

## 1. Introduction

Structural health monitoring attracts increasing interests in civil engineering because of its potential ability to alert managing staff of a forthcoming, dangerous state of a structure in advance. In addition, structural health monitoring helps to evaluate the extent of damage and to estimate, afterwards, the structure's remaining service life once a disaster occurred [1, 2]. Among all the sensors that can be used in structural health monitoring or smart structures in the future, fiber Bragg grating (FBG) sensors stand out as the most promising sensing element. In comparison to conventional electricity-based sensors, FBG sensors exhibit many advantages including: (1) electromagnetic inference immunity and low energy loss, which is preferable in the applications of long-distance health monitoring; (2) multiplexing ability to allow for quasi-distributive monitoring of strains by one fiber; (3) high sensibility over a wide measurement range; (4) slenderness and flexibility such that added mass and stiffness to the host structure can be neglected [1, 3]. Furthermore, FBG sensors exhibit tolerance to harsh construction conditions and excellent long-term performance [4].

The past decades have witnessed an intense research effort to embed FBG sensors that monitor many types of structures, especially bridges [1, 5]. However, tall buildings are not usually monitored, especially during its construction phase [6–8]. In current investigation, a monitoring system consisting of FBG sensors was designed to monitor: (1) temperature increase within concrete due to hydration process, (2) strain variation of the main column on the underground floor due to subsequent addition of upper 18 floors, and (3) relative long-term displacement between two sinking foundation blocks. For these purposes, FBG temperature and strain sensors were bonded on the rebars of the column and the 1st floor beam before concrete pouring. The FBG sensors have operated continuously for more than five months after installation. Concrete pouring process was precisely monitored in terms of hydration course. Strain variation in the main column was recorded when upper floors were sequentially built up along with the

Figure 1: Dongsheng Garden A5 building in construction (on October 30, 2008).

temperature records inside and outside concrete. Records of temperature were also used for compensation of FBG wavelength variations during strain monitoring. Such on-site temperature and strain monitoring data could provide valuable information for future building construction.

The paper is organized as follows: the instrumented building, the monitoring system along with packaged FBG sensors and their calibration tests are described first. Monitoring results of the building are then presented in three parts: the temperature and strain in the column before the underground floor was concreted, the concrete pouring and curing process, and the long-term strain monitoring while upper floors were built up in sequence. In the follows, the temperature lag behavior between inside concrete and ambient air, and its effects on structural health monitoring are discussed. Finally, results and contributions of the paper are summarized.

## 2. Description of the Under-Construction Building and Its Monitoring System

*2.1. Dongsheng Garden A5 Building: Description and Construction.* The project under investigation is the Dongsheng Garden A5 building, located in Fushan Bay Area (Qingdao, China). The A5 building is an 18-story frame-shear wall residential building with 54.85 meters above ground. It includes one underground floor for parking, where FBG sensors were installed. The A5 building began to be constructed on April 25, 2008. Figure 1 shows that the 12th floor was being under construction on October 30, 2008 and the red downward solid arrow points FBG sensors' locations, that is, the underground floor wall and the first floor beam.

*2.2. Fundamentals of Fiber Bragg Gratings.* A Bragg grating is a permanent periodic modulation of the refractive index in the core of a single mode optical fiber, which consists of three layers: fiber core, cladding, and coating. When light transmitted in a fiber impinges upon Bragg gratings, constructive interference between the forward wave and the contra-propagating wave leads to a narrowband back-reflection of light when the Bragg (or phase match) condition is satisfied. The Bragg condition can be expressed between the central wavelength of a Bragg grating and its period by the following relationship [9]:

$$\lambda_b = 2n\Lambda, \tag{1}$$

where, $\lambda_b$, $n$, and $\Lambda$ are the central wavelength of reflected light, effective refractive index, and period of a Bragg grating, respectively.

When a Bragg grating is exposed to mechanical strains, its period is altered and thus the wavelength of reflected light changes. In general, the wavelength of the reflected light varies with applied strain as follows:

$$\frac{1}{\lambda_b}\frac{\Delta\lambda_b}{\varepsilon} = 0.78 \times 10^{-6}/\mu\varepsilon. \tag{2}$$

Moreover, the Bragg grating is also sensitive to the variation of environment temperature as,

$$\frac{\Delta\lambda_b}{\lambda_b} = \left(\alpha_f + \xi\right)\Delta T, \tag{3}$$

where, $\alpha_f = (1/\Lambda)(d\Lambda/dT)$ is the thermoelongation coefficient and $\xi = (1/n)(dn/dT)$ is the photosensitivity coefficient of the fiber. The temperature coefficient in (3) is, in total, 10.3 pm/°C for Bragg gratings with wavelength around 1550 nm [4]. Hence, structural strain can be measured by the change of reflected light wavelength once the influences of temperature are eliminated based on (3).

*2.3. Calibration Tests of Packaged FBG Sensors.* The FBGs used in this work are inscribed in Ge-doped photosensitive optical fiber using the phase mask technique [10]. The gratings are annealed for more than 7 hours at 200°C to stabilize their performance for reliable long-term monitoring and then packaged for measurement applications since naked FBGs are fragile and too delicate to be handled [1]. Figure 2(a) illustrates a steel-tube-packaged FBG strain sensor with patented techniques developed in our laboratory.

A series of tests were performed to calibrate FBG strain sensors packaged in our laboratory. An FBG strain sensor and a strain gauge were first bonded on a steel plate in parallel with cyanoacrylate adhesive. Tensile tests of the steel plate were then carried out in the material testing system shown on the right in Figure 2(b). During the calibration tests, the steel plates were loaded continuously from 0 $\mu\varepsilon$ to 1000 $\mu\varepsilon$ and then unloaded six times to ensure repeatability. The results by calibration tests thus obtained are plotted in Figure 2(b), showing the relationship between Bragg wavelength shifts and strains measured by the strain gauge. Moreover, experimental strain sensitivity of the packaged FBG strain sensor is 0.5013 $\mu\varepsilon$/pm, which agrees well with the theoretical analysis [11, 12] in the following section.

*2.4. Strain Transferring Consideration of Packaged FBG Sensors.* Conventionally, the values measured by an FBG sensor were assumed to be actual structural strains [13]. In fact, the strain measured by an FBG is different from actual host structure strain because of the difference between

(a) Steel-tube-packed FBG sensor

(b) Calibration tests on a universal material testing system

Figure 2: Steel-tube-packed FBG sensor and calibration tests.

the optical fiber core modulus and the modulus of the fiber coating or the adhesive. Such strain difference depends on detailed packing measures of FBGs. Generally, there are three methods to integrate FBGs with host structures in terms of packing strategies: (1) direct integration, in which FBGs are directly embedded in or surface bonded to a host structure; (2) sensor-packaging integration, by which FBGs are first fixed in a small tube or bonded on the surface of a plate, and then the tube or the plate is anchored in the host structure; (3) clamping integration, in which an FBG is gripped at two ends by a bracket fixed on the host structure. No matter how the FBGs are packaged and integrated, the fiber core is brittle and has to be protected by adhesives or a coating layer in most sensing applications to avoid fiber breakage and to ascertain its long-term stability. However, such a protection results in inconsistency between the fiber strain and the structural strain. Such discrepancies are neglected in many applications of FBGs by simply assuming that the fiber strain is consistent with the host structural strain [3, 14]. Such an assumption gives acceptable measurement results for the FBGs with long gauge length, in which the peak host strain can be fully transferred into the fiber strain, but cannot provide good measurement strains for short gauge FBGs, for instance, FBG sensors, in which the effect of the bonded fiber length on strain transfer between the fiber and host structure is significant [15]. It is, thus, of primary importance to have a detailed knowledge of the relationship between the host structural strain and the fiber strain in order to correctly interpret structural strain from the strains sensed in the fiber. For the packaged FBG sensor used in our investigation, strain transferring loss between an FBG and its external layer can be approximately written as [16, 17]

$$\varepsilon_g(x) = \varepsilon_m\left(1 - \frac{\cosh(kx)}{\cosh(kL)}\right), \tag{4}$$

where $\varepsilon_g$ and $\varepsilon_m$ are fiber core and host structural strains, respectively. $k$ is a strain lag parameter, containing both effects of the geometry and the relative stiffness on the packaging components, and can be written as

$$k^2 = \frac{2G_c}{r_g^2 E_g \ln\left(r_m/r_g\right)} = \frac{1}{(1+\nu)\left(E_g/E_c\right) r_g^2 \ln\left(r_m/r_g\right)}, \tag{5}$$

where $E_g$ and $E_c$ are Young's modulus of the optical fiber and coating layer, respectively. $G_c$ is the shear Young's modulus

of the coating or outerlayer. $r_g$ and $r_m$ are radii of the optical fiber and coating layer, respectively. $\nu$ is the Poisson's ratio of the fiber core.

Furthermore, local enhancement may occur, which necessitates the need for calibration of FBG sensors to obtain actual structural strains as shown in Section 2.3. Previous investigations indicate that the strain distribution of the host material surrounding the fiber is influenced by the fiber embedment and that the characteristics of the host material have important effects on the strain transferring from host material to fiber. The mechanical properties of the host material must, therefore, be taken into account when exploring the strain transferring from host material to the fiber. Interested readers may consult the references [18, 19] which discuss related issues in further details.

*2.5. FBG Monitoring System: Design and Installation.* We had measured vibration of a pipeline model with steel-tube-packaged FBG sensors [11, 12] and performed investigations on health monitoring of an offshore platform (number CB271, located in the Bohai Sea, China) with packaged FBG sensors bonded on the circumference of the pillar [11, 12]. With the three objectives outlined in the Introduction and with reference to the experience gained from previous investigations, a monitoring system consisting of packaged FBG sensors with adequate protection measures was designed. The monitoring system involves initially seven FBG sensors.

In a later stage, another FBG temperature sensor was added. Among the eight FBG sensors, four FBG strain sensors are intended to measure column strain while the building was still in construction. Two of the four FBG strain sensors were bonded on the rebars of the upper part of the column, 40 cm below the beam as illustrated in Figure 3(a), and the other two on the lower part of the column, about 50 cm above the underground floor as shown in Figure 3(b). Another two FBG strain sensors were adhered to the horizontal beam as shown in Figure 3(c) to monitor relative displacements between two foundation blocks. As far as temperature is concerned, two FBG sensors were installed. One sensor (FBG 7#) was left freely in a plastic pipe between rebars to measure temperature variation inside concrete, whereas the other (FBG 8#) was laid in the junction box (not shown in the figure) to measure environment air temperature as shown in Figure 3(d). All the eight FBG sensors were located directly underneath the standing worker wearing a blue suit in Figure 4(a).

The eight FBG sensors were multiplexed serially into two single-mode optical fibers (Figure 3(d)), which were protected in plastic pipes as shown in Figure 4(b) and routed to the monitoring room around 100 meters away. The acquisition system was composed of a demodulation unit FBG-SI425 (Micronoptics Incorporated), a personal computer, and a digital interface PCMCIA card. A customized program running under LABVIEW (National Instruments) on the PC was developed for acquiring data every minute from the FBG sensors through FBG-SI425 and the PCMCIA card. The acquired data are then saved in separate files automatically. The sampling interval can be adjusted for desired frequency to accommodate the requirement of various construction phases.

Table 1: Daily temperature variation before concreting.

| Date (July, 2008) | 7.14 | 7.15 | 7.16 | 7.17 | 7.18 | 7.19 |
|---|---|---|---|---|---|---|
| Lowest temperature (°C) | 22.5 | 23.5 | 20.9 | 22 | 22.9 | 23.2 |
| Highest temperature (°C) | 25 | 25.2 | 26.7 | 27.7 | 24.6 | 26.3 |
| Maximum difference (°C) | 2.5 | 1.7 | 5.8 | 5.7 | 1.7 | 3.1 |

As pointed out in previous paragraphs, FBG sensors need to be protected from concrete aggregate during cement pouring. Details of FBG sensors' fixture to the rebars and their protection in the investigation are illustrated in Figures 3 and 4. Packaged FBG sensors were first bonded to the rebars with epoxy, and then wrapped with gauze patches to allow for solidification of the epoxy. In addition, steel hoses were presleeved to shield leadings of bonded FBG sensors (small silver steel hoses shown in Figure 3) and plastic pipes (large blue ones) were used to house optical fibers. Moreover, two junction boxes are placed in the wall to (1) protect optical connectors, (2) to check the integrity of each FBG sensor, and (3) to provide long-term monitoring once the building is completed. Furthermore, larger and harder pipes were used to protect communication fibers leading to the monitoring room as shown in Figure 4.

## 3. Dongsheng Garden A5 Building Monitoring Results

The monitoring of strain and temperature of the A5 building during construction can be categorized into three periods: the first is around one-week before concreting of the 1st floor; the second is immediately followed by concrete pouring and includes curing process which lasts approximately 10 days; and finally the long-term monitoring with upper 18 floors built up sequentially.

*3.1. One-Week Monitoring before Cement Pouring.* After one week of sensor installation, the monitoring of temperature and rebar strain started. Figure 5 shows the time history of two FBG strain sensors bonded on the upper rebars of the column (FBG 3# and FBG 4# as shown in Figure 3(a)). The blue solid curve and the red dotted curve illustrate the strain variation measured by FBG 3 and FBG 4, respectively. Both rebars belong to the same column and are supposed to bear similar loads. The first parts of the curves in Figure 5 demonstrate clearly that both FBG sensors were synchronously stressed with respect to the strains measured at 00:00:00 on July 14, 2008. However, FBG 3 sensor measured 15 $\mu\varepsilon$ compressive strains more than that measured by FBG 4 between July 18 and July 20. It is postulated that the workers may have unevenly laid construction materials on the column during the three days, and that the rebars were subjected to unequal loads due to loose connections in-between. The rebars display, therefore, asynchronous strain variations. Nevertheless, no visual evidence was available to confirm such an argument.

(a) Two rebars in the 1st floor horizontal beam bonded with FBG sensors; top: FBG 1#, 1529 nm; bottom: FBG 2#, 1564 nm

(b) Lower part of two rebars in the vertical underground column bonded with FBG sensors; left: FBG 5#, 1557 nm; right: FBG 6#, 1525 nm

(c) Junction box with two FBG sensors; FBG 7#, 1519 nm; FBG 8#, 1548 nm

(d) Upper part of two rebars in the vertical underground column bonded with FBG sensors; left: FBG 3#, 1534 nm; right: FBG 4#, 1562 nm

Figure 3: Installation of the FBG sensors, their position, and protection measures on site.

(a) Layout of the first floor and shear wall of the A5 building installed with FBG sensors (sensors bonded on the rebars underneath the standing worker)

(b) Plastic pile protecting the connection fibers leading to the monitoring room

Figure 4: Installation of the FBG sensors, their protection, and routing to the monitoring room.

Figure 5: One-week monitoring of column strains of the A5 building before concreting.

Figure 6: Strain and temperature variations in the beam rebar on the first floor before the concreting.

The strains in the 1st floor beam and the temperatures in the rebars before concreting were monitored as shown in Figure 6. The blue solid curve represents the change in temperature within 4°C. For the purpose of comparison, daily temperature records, which were obtained from the Qingdao Weather Bureau (http://qdqx.qingdao.gov.cn/), are listed in Table 1. It can be clearly observed that measured temperatures (shown in Figure 6) agree well with the records (Table 1). Furthermore, the temperature record on July 14 was also used to adjust relative temperatures measured by the FBG temperature sensor in the investigation. It should be noted that the rebars, junction boxes, and the protection

Figure 7: Strain and temperature variations during the concrete pouring and curing in the column.

pipes are hidden in the shadows. Therefore, the temperatures measured by the FBG sensor in Figure 6 are slightly lower than the local air temperature.

The red dashed curve in Figure 6 illustrates the strains measured by the FBG 1 sensor. Two obvious peaks appeared with maximum strains of 110 $\mu\varepsilon$ and 90 $\mu\varepsilon$. It was later verified that construction materials and working apparatus were placed on the horizontal beam temporarily, where the FBG 1 sensor is located. Unfortunately, a construction worker was unaware of the fragility of FBG sensors and accidentally broke one lead of the FBG 1 sensor during the installation, which resulted in the unavailability of the FBG 1 sensor in further measurements.

*3.2. Concrete Pouring and Curing.* During the pouring of concrete into the beam of the first floor and the column of the underground floor where optical sensors were located, FBG sensors were integrated every minute to ensure that variation of temperature and strain inside cement could be accurately traced with a proper frequency. Figure 7 illustrates temperature variations recorded by the optical temperature sensor during concrete pouring and initial curing with time. The horizontal axis denotes the date starting from July 20 to August 1, 2008. At 04:00:00, July 20, 2008, which is the starting time of Figure 7, environment air temperature was 24°C. The blue solid curve illustrates the temperature evolution ranging from the minimum of 10.435°C to the maximum of 40.1417°C. In the graph, three phases are clearly distinguishable: the first is the fresh concrete (phase I); followed by a quick-rising phase (phase II), and finally there is a temperature-decreasing trend and a long stabilization process (phase III).

The reason for the temperature decrease in the first phase may be attributed to the fact that the temperature of fresh cement with much water content was lower than that of embedded FBG sensors, which experienced originally environment air temperature. Once the optical temperature

Table 2: Concrete pouring date floor by floor.

| Floor number | Concreting date | Lowest temperature (°C) | Highest temperature (°C) | Maximum difference (°C) |
|---|---|---|---|---|
| 1 | 8.25 | 21.5 | 26.9 | 5.4 |
| 2 | 9.4 | 21.1 | 24.9 | 3.8 |
| 3 | 9.12 | 19.2 | 28.7 | 9.5 |
| 4 | 9.19 | 23 | 25.8 | 2.8 |
| 5 | 9.27 | 11.8 | 19.5 | 7.7 |
| 6 | 10.3 | 18.5 | 24.3 | 5.8 |
| 7 | 10.8 | 15.5 | 20.6 | 5.1 |
| 8 | 10.14 | 17.1 | 23.4 | 6.3 |
| 9 | 10.19 | 16.3 | 23.7 | 7.4 |
| 10 | 10.25 | 11.9 | 19.1 | 7.2 |
| 11 | 10.31 | 11.2 | 16.8 | 5.6 |
| 12 | 11.5 | 15.6 | 20.4 | 4.8 |
| 13 | 11.10 | 4.1 | 11.6 | 7.5 |
| 14 | 11.16 | 5.7 | 12.9 | 7.2 |
| 15 | 11.21 | 4.9 | 12.8 | 7.9 |
| 16 | 11.26 | 5.8 | 11.8 | 6 |
| 17 | 12.1 | 8.5 | 14.7 | 6.2 |
| 18 | 12.7 | 1.2 | 7.5 | 6.3 |

sensor was totally immersed in the cement while the pouring continued, its temperature decreased. As the concrete began to cure shortly after the pouring was finished, a rapid increase in the temperature was observed due to exothermic reaction of cement in the second phase, which lasted about 16 hours. The temperature rise was then followed by a gradual decreasing course in the third phase, which lasted more than 7 days. Finally, the temperature in the concrete stabilized at a constant level in the third stage, and the curing process ceased.

Strain variation during the pouring and curing of concrete in the steel-frame wall by a typical strain sensor (FBG 4) was presented by the red dashed line in Figure 7. Recall that the FBG 4 strain sensor is attached to the column rebar with a calibrated wavelength of 1562 nm as shown in Figure 3(a). The right vertical axis shows that strain variation of the FBG 4 sensor ranges from $-21.5\,\mu\varepsilon$ to $197.5\,\mu\varepsilon$. The trend of strain variation of concrete follows exactly the temperature changes, which indicates that the strain measured in concrete was mainly attributed to the thermal expansion and contraction of concrete, but not to external forces. The initial compressive strain measured by the FBG strain sensor was due to thermal contraction of the steel rebar immersed in fresh concrete.

It is well known that FBG sensors have cross sensitivities with both strain and temperature. In Figure 7, the green dash-dotted line represents the "absolute strain" measured by the FBG strain sensor due to external forces, that is, the strain induced by the temperature variations is already eliminated with the calibrated sensitivities (shortened for naming convenience, "total strain minus thermal strain" is more appropriate). It is interesting to note that tensile strains were measured by the FBG sensor and the rebar was elongated in the first phase. This may attribute to the flowing of cement below the FBG sensor and the rebar enduring temporary tensile stress. Immediately after that, compressive strain was measured. Apparently, the compressive strain was caused by the weight of the cement on the rebar. After the compression phase, tensile strains were recorded.

It is noteworthy that the period between 13:00 on July 23 and 16:00 on July 25, 2008 was due to lack of measurement data because of power breakdown onsite. The construction site was far from downtown and electric power facilities were not well maintained. Nevertheless, the unavailable measurement data shown in Figure 7 were compensated with straight lines by two nearby measurements without interpolation. In the following figures, the same rules are applied if not stated otherwise. Furthermore, although the workers were aware that the sensors are present in the framework, they did not take additional precautions during concreting, and poured directly and vibrated. All but two of the installed sensors were working properly after concreting. One sensor was lost at just the beginning of concreting, and the other was active for the first several hours, but lost its signal a little later.

*3.3. Long-Term Construction Monitoring with Added Building Floors.* Influences of the construction of upper building floors on the underground column were investigated in this subsection. The strains on the column rebars were continuously and remotely monitored in the monitoring room. The sampling interval is one hour, which was not as frequent as the previous section. Measurements obtained by the sensors are presented in Figure 8. The blue solid curve in Figure 8 illustrates measured temperature variation for a period of approximately half a year. In the graph, the dates with available monitoring data when the power was not broken down are indicated. The red dashed curve presents the strain variation in the concrete column, and it could be clearly observed that the strain follows roughly the same trend as that of temperature. The build-up of compressive strains was due to the construction of different building floors. Dates of concrete pouring of different floors were listed in the 2nd column of Table 2. The construction of the ground floor took place 35 days after the pouring of concrete into the underground wall. Concrete pouring began on August 25 and was completed on December 7, 2008. The 3rd and 4th columns show also the environment temperature, which will be used later in the subsequent section to compare the temperature difference inside and outside the concrete.

Furthermore, the column strains on pouring dates of the upper floors clearly show valleys, which was apparently caused by the adding weights of the poured concrete. On the other hand, strain anomalies were also detected, which could be explained by the short gauge length of the FBGs used for the optical sensors (approximately 10 mm) that could result in the measurement of localized strains at the sensor positions. In the future, long gauge FBG strain sensors may be developed to accommodate such demands [8].

Figure 8: Strain and temperature variations in the five-month's monitoring of the A5 building.

Figure 9: Environment lowest and highest temperature variations on the concrete pouring dates along with measured inside concrete temperature by the FBG 8 temperature sensor.

## 4. Temperature Lag between Inside the Concrete and Environment

*4.1. Validate Measured Temperature by the FBG Sensors.* To validate measured temperatures in concrete, environment air temperature of the local area, especially daily temperature records on concrete pouring dates of different floors, was obtained from the Qingdao Weather Bureau and listed in the last three columns of Table 2. The 3rd column of Table 2 lists the lowest temperature whereas the 4th column lists the highest temperature. Since the construction site is not far from the city center and actual on-site air temperatures were not measured, the temperature records in Table 2 can be reasonably regarded as the Dongsheng A5 building on-site air temperature without much deviation.

Figure 9 illustrates measured concrete temperature by the FBG 8 temperature sensor, which was located in the junction box on the wall of the underground floor as shown in Figure 3(c). The lowest temperatures are drawn by the green dash-dotted line whereas the highest temperatures by the red dashed line in Figure 9. Due to frequent breakdowns of power on the construction site, the measurements are not completely continuous. Nevertheless, it could be clearly observed that measured concrete temperatures by the FBG 8 sensor fall well within the temperature range. The FBG 8 sensor is, therefore, considered to be capable of monitoring concrete temperature adequately.

*4.2. Temperature Lag between Concrete and Environment.* It is known that the temperature inside concrete lags normally behind the temperature of its surrounding environment. Concrete takes a while to warm up or cool down due to thermal inertia effects [20, 21]. The lagging behavior will be of great importance in structural health monitoring since structural dynamic features used for damage identification will be heavily influenced by environment temperature. For meaningful interpretations of damage index changes, temperature effects have to be eliminated. Two FBG temperature sensors installed in the Dongsheng A5 building, one inside and the other outside concrete, allows us to study this interesting temperature lag behavior.

Figure 10: Comparison of inside and outside concrete temperature.

Figure 10 takes a closer look at measured temperatures by both FBG temperature sensors in a month: FBG 8 in the junction box to measure environment temperature while FBG 7 buried in the concrete to monitor inner concrete temperature. As indicated by "time lag A", "time lag B", "time lag C" and "time lag D" in Figure 10, time lags obviously

exist between the blue solid line (environment temperature by measured FBG 8) and the green dashed line (concrete temperature by measured FBG 7). The average time lag is approximately half a day, that is, about 12 hours. This delay period agrees with the investigations by [22]. It is noteworthy that such temperature lag is strongly influenced by the dimensions of the structure, especially the thickness of the concrete. In our case, the thickness of the concrete wall is 200 mm and the FBG temperature sensor is located around 80 mm below the wall surface. Moreover, the FBG 8 sensor was laid freely in the conjunction box with only a thin wood covering. Therefore, it is believed that the FBG 8 sensor could accurately measure underground air temperature of the Dongsheng A5 building. On the other hand, the FBG 7 sensor is packaged inside a steel tube and protected by a plastic pipe as illustrated in Figure 4(b). It is therefore conjectured that there may exist a pocket of air surrounding the FBG 7 sensor, which may further increase the lagging of sensed concrete temperature with respect to external environmental temperature.

Daily temperature fluctuations could be obviously seen in Figure 10 in a 24-hour base. However, it should be noted that day temperature did not differs much from night temperature since the measured concrete wall is in the underground and because Qingdao is a seaside city. As shown in Table 2, air temperature difference between day and night in Qingdao stays mostly within 5°C, less than 10°C in extremities. This explains why the temperature in Figure 10 varies only slightly in a day and night cycle as observed in a month from October 22 to November 22, 2008. Furthermore, the downward trend clearly shows seasonal temperature variation in autumn.

## 5. Concluding Remarks

Fiber Bragg grating (FBG) sensors were used to monitor strain and temperature of a tall building while in construction; namely, three periods are monitored: before the concrete pouring, during the pouring and curing of concrete, and the construction of subsequent upper floors of the building. For this purpose, a monitoring system consisting of packaged FBG sensors with patented design in our laboratory for high and accurate strain transferring was designed. Moreover, measures were taken to protect the FBG sensors and the communication optical fibers for successful sensing. The temperature changes in the concrete during pouring and curing exhibit clearly three phases. More than five months' monitoring of the Dongsheng A5 building construction in progress permitted us to have a thorough understanding of loading variations of the main column on the underground floor.

Furthermore, structural health monitoring has generally been considered as a reliable method for determining health state of a structure. However, experience has shown that in some cases the deviation between structural temperature and environment temperature can be quite significant. It appears that temperature lag affects the effectiveness of structural health monitoring and has substantial implications in actual applications. In this investigation, the temperature lagging between inside and outside the concrete is studied and the lag time is found to be around 12 hours, which may provide interesting clues for further investigations. Moreover, the strain-transferring mechanism from neighbouring packaging material to the FBG and the influence of host material is discussed.

## Acknowledgments

The authors are grateful for the assistance of the construction workers at the Dongsheng construction site and the support of the Science Fund for Creative Research Groups of the National Natural Science Foundation of China (Grant no. 51121005) and of the Fundamental Research Funds for the Central Universities (Grant no. DUT11LK35).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 810682, 7 pages
doi:10.1155/2012/810682

*Research Article*

# Application of Laser Vibrometer to the Measurement and Control of Cable Tensile Forces in Cable-Stayed Bridges

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Received 2 July 2012; Revised 20 August 2012; Accepted 31 August 2012

Academic Editor: Hong-Nan Li

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The tensile forces acting on the cable of long-span bridges are one of the most important factors since they reflect not only the structural stability of cables but also the overall quality of construction. Currently, indirect measurement using accelerometers attached to the surface of the cable is widely used to measure the natural frequency of cable. The frequency obtained from the accelerometer is converted to the tensile force of the cable. However, it sometimes requires many hazardous labors such as attaching the device on the surface of cable and wiring it with data logger, which could hinder the safety of workers during the erection of cables. In this study, a method using laser vibrometer is introduced to measure the tensile forces on cables at a distance. In addition, this study developed a unique postanalysis computer program that can calculate the tensile forces in real time. Compared with the values obtained from the accelerometers, the laser vibrometer system provided accurate and reliable matching.

## 1. Introduction

Long-span bridges have become more constructed since the market rapidly grew up from the late 1990s. During the construction of long-span bridges, various measurements such as acceleration, displacement, and tilt angle are performed to certify the safety of the constructions in stages. Among them, the tensile force on the cable of bridge is of the most importance since it reflects the overall structural stability of bridge under construction. Therefore, the measurement for tensile force on cable is required to be precisely measured and calculated for structural integrity when the construction is complete [1]. However, it is difficult to predict the structural displacements and the cable forces by erection simulation analysis because cable-stayed bridge has the characteristics of a slender, flexible, and nonlinear behavior. The cable forces have to be adjusted for the reduction of estimation errors in section modulus, dead load, fabrication, and installation during the construction and also for structural integrity when the construction is complete [2, 3].

Currently, for the calculation of tension force on cables, accelerometers attached to the surface of the cable are mainly used to measure the natural frequency of cable [4]. However, they sometimes require many hazardous works such as attaching the device on the surface of cable and wiring it with data logger, which could hinder the safety of workers in the middle of cable. It is sometimes even more dangerous when the bridge deck work is not yet finished. In recent studies, many wireless measuring instruments including GPS are used for health monitoring of high-rise buildings [5, 6] and bridges [7]. In particular, Yi et al. [8] measured the spectral displacements and the mode frequencies of a suspension bridge using the real-time kinematic (RTK) GPS, which showed consistent results compared to the ones from accelerometer and FEM analysis.

In this study, a measurement method using laser vibrometer and analysis software is developed to eliminate both the unsafe works and to expedite the construction. With the developed method, on-site engineers are able to figure out the tensile force of cable in no time while standing on the ground. In addition, compared with the previous method using accelerometers, the developed system is assumed to be accurate and reliable.

## 2. Development of the Noncontact Tension Measurement System

*2.1. Theory.* There are several ways to measure the tension of cables from the natural frequencies of the cable vibration using string vibration method, vibration method, and vibration equation [9]. As for the string vibration method, it is easy to apply using a simple equation but known to have a discrepancy because the cables are actually subjected to flexural rigidity and friction. The vibration method takes into consideration the flexural rigidity, sagging, and inclination angle of cables and thus provides more reliability than the string theory method [10]. However, this method contains conditional variables, which make the calculation complicated. Thus, we have applied the vibration equation method, which is more reliable and easy to apply, in this study.

The differential equation of motion considering the flexural rigidity of the $y$-direction is shown in (1) [11] and the equation for hinge boundary conditions at both ends is as follows as (2)

$$\frac{w}{g}\frac{\partial^2 y}{\partial t^2} - EI\frac{\partial^4 y}{\partial x^4} - T\frac{\partial^2 y}{\partial x^2} = 0 \tag{1}$$

$$T = \frac{4wf_n^2L_e^2}{n^2g} - \frac{(n\pi)^2EI}{L_e^2} \quad (n = 1, 2, \ldots), \tag{2}$$

where $y$ = coordinate of vertical direction due to vibration, $x$ = coordinate of longitudinal direction, $t$ = time (sec), $T$ = cable tension (kN), $w$ = cable unit weight (kN/m), $f_n$ = natural frequency of vibration at $n$th mode(Hz), $L_e$ = effective length of cable (m), $n$ = number of vibration mode, $g$ = gravitational acceleration (= 9.8 m/sec$^2$), and $EI$ = bending stiffness of cable(kN·m). The second item denotes the effect of the length of cable on the flexural rigidity. As shown in the equation, the shorter the cable is, the bigger the effect of flexural rigidity becomes. Generally, the measured values show the trend as shown in Figure 1 and can be expressed as primary regression equation. In the primary mode, once the values of slope "$a$" and "$b$" are defined, the tension $T$ and the bending stiffness $EI$ can be obtained as follows:

$$\left(\frac{f_n}{n}\right)^2 = \frac{Tg}{4wL_e^2} + \frac{EI\pi^2 g}{4wL_e^4}n^2 = b + an^2 \tag{3}$$

$$T = \frac{4wL_e^2}{g} \times b \tag{4}$$

$$EI = \frac{4wL_e^2}{\pi^2 g} \times a. \tag{5}$$

Then, the variables for "$a$" and "$b$" can be determined from the linear regression analyses between $(f_n/n)^2$ and $n^2$ as shown in Figure 1. When we take into account the sagging effect of the cable, we can obtain the tension of the cable using the following formula:

Figure 1: Regression analyses of the measured data.

$$\left[\frac{f_n}{n(1-\xi_n^2)}\right]^2 = \frac{g}{4wL_e{}^2}\left[1 + \frac{2\lambda^2}{\pi^4} - \frac{\left[1-(-1)^n\right]^2}{n^4}\right]T + g\frac{EI\pi^2}{4wL_e{}^4}n^2 = a + bn^2 \tag{6}$$

$$T = a\frac{4wL_e{}^2}{g\left[1 + (2\lambda^2/\pi^4) - \left(\left[1-(-1)^n\right]^2/n^4\right)\right]}. \tag{7}$$

Please note that $\xi_n$ is the damping ratio of cable and $\lambda^2$ the coefficient for the amount of sag. As the damping ratio of the cable is insignificant, we can ignore the term of the damping ration in (7). In addition, when we use only the even modes to take into account the sagging effect, the equation above can be expressed as follows:

$$T = a\frac{4wl^2}{g[1 + (2\lambda^2/\pi^4)]}. \tag{8}$$

*2.2. System Configuration.* In this study, we used a laser vibrometer PDV-100, manufactured by Polytec GmbH of Germany, to evaluate the tension of cables. This device is characterized by the followings. (1) It consists of a more precise laser and optic equipment and has the interferometer inside of the tool, so that it does not require additional setup time. (2) It does not have a homodyne interferometer, which does not show the directional information, but instead has the heterodyne interferometer, which shows the directional information. As it has its own decoder, it can check the speed up to the precision of 0.02 $\mu$m/s and provides not only the analogue information but also the digital information. (3) With a significantly lower noise floor, it helps us to observe the signal, which would be undetected if the analog analyzer is used. In addition, this device measures the minute vibration of the structures on site using a noncontact method.

The proposed system consists of laser vibrometer (PDV-100), USB DAQ, and notebook computer. The vibrometer basically measures the amplitude of vibration in the object from a distance since it uses laser beam projection. The measurement is basically the velocities and the maximum

(a) (b)

Figure 2: Configuration of the proposed measurement system and field setup.

range can be selected as one of 10 mm/s, 100 mm/s, and 500 mm/s with the frequencies from 0 up to 22 kHz. When the laser is projected onto the surface of the object, the laser reflection returns to the receiving sensor of the laser vibrometer. Therefore, the surface should be clean enough to reflect the laser beam. As shown in Figure 2, the device can be set away from the target cable. At this time, the vibrometer should be firmly fastened to tripod or order to minimize unnecessary noise. The working distance or sensing range can be up to 30 meters from the target to the vibrometer. The USB DAQ is used to convert the analog signals from the laser vibrometer into digital signal. The notebook computer is required to process the postdata to numerical analyses in the field. The configuration and field setup of system are shown in the Figure 2.

*2.3. Algorithm of the System.* We have reviewed how to convert the values measured from the portable vibrometer, PDV-100, into the tension for cables. In the United States, FHWA and CTL (Construction Technologies Laboratories, Inc.) jointly developed the laser-based portable cable tension meter in 1999 and have adopted it into the cable bridges for maintenance. It has been known that it measures 20 to 50 cables a day for their vibration as of 1999 (FHWA, 1999).

The values obtained from the device are velocity values, which can be converted into acceleration or sagging using the differential and integral calculus. In addition, either the acceleration or velocity data is used to obtain the number of vibration modes and relevant natural frequencies to get the tensional force. The basic processes for obtaining the tension are specified in Figure 3.

As for the application of the portable vibrometer in the field, it is recommended that the vibrometer is placed at the right angle to the cable for more accurate measuring. However, it is difficult to set the tool at the right angle with the cables. So, it is effective in terms of accurateness of measured values that the device is located within 15 degree from the right angle with the cable.

*2.4. Software.* In this study, we have developed a program based on the suggested algorithm. The developed software takes the velocity of the cable using the embedded DAQ computer and a portable laser vibrometer. In Figure 4, from (1) to (9), we define the basic information of DAQ device channel, sampling rate, and scale factor. Then, the velocity data are collected and saved at the same time. The windows 1 and 2 in Figure 4 show the real-time data obtained from the laser vibrometer and the periods per each vibration mode obtained from the FFT analyses, respectively. Window 3 shows the result of regression analyses and window 4 the variations in the cable tensions in real time. Once the laser vibrometer is set in a distance from the cable to be measured, the focus of the laser has to be manually set in order to obtain the maximum reflective rate. In general, the cable is subjected to ambient vibration. Therefore, intentional vibration or impact on the cable is not required.

Figure 3: Basic process of measuring tension.

## 3. Verification of the Proposed Measurement System

The tension measuring system is applied for a cable-stayed bridge which was constructed in Yeongduk, Korea. The structural type is a cable-stayed bridge with a steel-composite girder and single steel pylon. The span length is 205.0 m (90 m + 115.0 m) with the bridge width of 23.40 m. Figure 5(a) shows the elevation view and the overview of the Yeongduk cable-stayed bridge.

Using the developed tension-measuring system, the tensional forces acting on cables are measured at the final

Table 1: Comparisons of tension forces for the Yeongduk cable-stayed bridge.

| Cable number | | SC1 | SC2 | SC3 | SC4 | SC5 | SC6 | EC6 | EC5 | EC4 | EC3 | EC2 | EC1 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Cable length (m) | | 91.24 | 83.15 | 75.14 | 64.7 | 54.6 | 45.1 | 44.34 | 53.69 | 63.61 | 73.9 | 84.44 | 95.22 |
| Weight (kN/m) | | 1.128 | 1.128 | 1.128 | 1.128 | 1.128 | 1.128 | 1.128 | 1.128 | 1.128 | 1.128 | 1.128 | 1.128 |
| Design tension (kN) | | 6,387 | 7,074 | 7,663 | 7,664 | 7,564 | 7,564 | 6,384 | 6,582 | 7,268 | 7,368 | 7,370 | 7,273 |
| Measured tension (kN) | Accelerometer | 6,296 | 7,021 | 7,636 | 7,654 | 7,545 | 7,549 | 6,182 | 6,439 | 7,199 | 7,355 | 7,388 | 7,295 |
| | Laser vibrometer | 6,341 | 6,762 | 7,263 | 7,403 | 7,371 | 7,413 | 6,254 | 6,429 | 7,259 | 7,208 | 7,121 | 6,931 |
| Difference (%) | | 0.7 | −4.4 | −5.2 | −3.4 | −2.6 | −2 | −2 | −2.3 | 0.1 | −2.2 | −3.4 | −4.7 |

Note: SC means starting side of cable and EC means ending side of cable, see Figure 5.

Figure 4: Measurement and postprocessing.

Table 2: Main erection process for the Incheon new port cable-stayed bridge.

| Construction stages | Description |
|---|---|
| Step 1 | Erection steel girder |
| Step 2 | Cable 1st pulling |
| Step 3 | Remove bent |
| Step 4 | Slab casting |
| Step 5 | Slab activation |
| Step 6 | Cable 2nd pulling |
| Step 7 | Superimposed load |
| Step 8 | 10000 day |

stage of construction. For comparison and reliability analyses, the resultant tension forces measured from laser vibrometer and conventional accelerometers are given in the Table 1. As shown in the table, the differences between the tensional forces measured from both methods are less than 5% for most of the cases. Therefore, the method using laser vibrometer proved to be reliable. Furthermore, the developed method takes less than couple of minutes to measure tension on a cable, which is even more efficient than conventional method. In this study, we also measured the cable tensions at the stage of installment of steel girders. Measuring tension at the stage could have been a dangerous job for technicians because the space between girders is wide open without concrete deck slab. Thus, this method contributed to improve the safety of construction as well.

## 4. Application of the Proposed Measurement System to the Construction of a Cable-Stayed Bridge

Objectives of construction control are to adjust deck elevations and cable forces within management limitations using simulation analysis, field measurements, and verifications of structural system throughout the construction of cable-supported bridges. Construction control can be divided into two tasks: the collection (measurement) and the analysis of data. Since these two tasks are separately operated, data sharing is very important. While one of the inspection teams performs the verification of the measurements through a laser vibrometer, another team analyzes the data for adjustments of deck elevations and cable forces.

*4.1. Description of the Bridge.* A new port access road connecting an LNG terminal with a coastal road in Incheon, South Korea is planed to be opened in November 2012. The roadway consists of a twin-deck cable-stayed bridge with two spans of 105 meter. The decks are composed of concrete slabs on steel box girders carrying three lanes of traffic. A typical section of cable-stayed bridge is shown in Figure 5(b). The decks are supported by the total of 16 Parallel Wire Strand (PWS) cables and connected at 10 m intervals. Each cable has a bundle of 367 to 421–7 mm diameter parallel wires and is inserted to the dead anchorage of the single concrete tower. The cable force is developed at the live anchorage of the girder next to a center-hole jack. The single tower and the layout of the cables can be seen in Figure 5(b).

*4.2. Simulation Analysis of Construction Process.* Simulation analysis is one of the ways to calculate the structural displacements and cable forces of cable-stayed bridges having the characteristics of a slender, flexible, and nonlinear behavior. The erection process of the Incheon new port bridge is summarized in Table 2.

*4.3. Measurements of Cable Tension Forces.* During the erection of the cable-stayed bridge, we measured the tension of the stay cables, deck profile, and pylon deformation. There are some discrepancies between the simulation values and the actual responses of the bridge. Generally, structural engineers identify these discrepancies from stochastic sensitive analyses. In this study, we identified these discrepancies from the measurements of cable tensions because we could measure sixteen cable forces using a laser vibrometer within an hour. The global matrix of the cable-stayed bridge does

Figure 5: Dimensions and overviews of the bridges. (a) Yeongduk cable-stayed bridge. (b) Incheon new port cable-stayed bridge.

Figure 6: Camber diagram of the girder during erection.

not change after the composition of the bridge. Therefore, the influence matrix of cable forces obtained from the measurements during the 2nd pulling of cables can be used for the adjustments of the tension forces. Table 3 shows the measurements of the cable forces using the laser vibrometer system during the construction of Incheon new port bridge.

*4.4. Control of the Bridge.* During the construction of Incheon new port cable-stayed bridge, the first pulling of the stay cables started on March 17, 2012 and ended on April 9. For the second pulling of the cables, we measured and controlled the cable tension of the bridge. The results of deck elevations and camber diagram are illustrated in Figures 6 and 7.

## 5. Conclusions

In this study we have conducted research on how to obtain and control the tension of cables by using the portable laser vibrometer. This study has been the first trial to find out a tension measuring method using the portable vibrometer in Korea. We measured the cable tension of cable-stayed bridges using the portable vibrometer and compared the results with those obtained from accelerometers. As a result, the proposed noncontact tension measurement system using a laser vibrometer provided reliable values. The main findings are as follows.

(1) The tension of cables can be effectively obtained using the portable vibrometer and the program developed based on a suggested algorithm.

Table 3: Field measurement of the first tensioning for the incheon new port bridge's cables.

| Cable ID | | SC1 | SC2 | SC3 | SC4 | SC5 | SC6 | SC7 | SC8 | EC8 | EC7 | EC6 | EC5 | EC4 | EC3 | EC2 | EC1 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Unit weight (kN/m) | | 1.17 | 1.25 | 1.25 | 1.33 | 1.33 | 1.33 | 1.33 | 1.17 | 1.17 | 1.33 | 1.33 | 1.33 | 1.33 | 1.25 | 1.25 | 1.17 |
| Effective length (m) | | 100.51 | 90.23 | 80.41 | 70.54 | 61.00 | 51.74 | 42.79 | 34.75 | 34.74 | 42.79 | 51.74 | 61.00 | 70.52 | 80.42 | 90.23 | 100.51 |
| Allowable force (kN) | | 10614 | 11481 | 11481 | 11481 | 12176 | 12176 | 12176 | 11481 | 11481 | 12176 | 12176 | 12176 | 11481 | 11481 | 11481 | 10614 |
| SC8 EC8 | Simulation | | | | | | | | **5,034** | **4,949** | | | | | | | |
| | Accelerometer | | | | | | | | **4,909** | **4,950** | | | | | | | |
| | LVTMS | | | | | | | | **4,926** | **4,969** | | | | | | | |
| SC7 EC7 | Simulation | | | | | | | **4,645** | 4,589 | 4,580 | **4,598** | | | | | | |
| | Accelerometer | | | | | | | 4,541 | 4,716 | 4,603 | **4,645** | | | | | | |
| | LVTMS | | | | | | | 4,353 | 4,636 | 4,507 | **4,564** | | | | | | |
| SC6 EC6 | Simulation | | | | | | **3,586** | 3,679 | 3,943 | 3,950 | 3,657 | **3,534** | | | | | |
| | Accelerometer | | | | | | **3,476** | 3,796 | 4,488 | 4,240 | 4,033 | **3,513** | | | | | |
| | LVTMS | | | | | | **3,316** | 3,781 | 4,471 | 4,188 | 3,986 | **3,488** | | | | | |
| SC5 EC5 | Simulation | | | | | **3,605** | 2,988 | 3,186 | 3,648 | 3,656 | 3,175 | 2,956 | **3,561** | | | | |
| | Accelerometer | | | | | **3,594** | 2,864 | 3,456 | 4,282 | 3,992 | 3,715 | 3,066 | **3,551** | | | | |
| | LVTMS | | | | | **3,501** | 2,835 | 3,488 | 4,265 | 3,996 | 3,639 | 2,991 | **3,495** | | | | |
| SC4 EC4 | Simulation | | | | **3,581** | 3,339 | 2,743 | 3,003 | 3,546 | 3,549 | 2,989 | 2,714 | 3,307 | **3,561** | | | |
| | Accelerometer | | | | **3,566** | 3,205 | 2,662 | 3,167 | 4,024 | 3,780 | 3,356 | 2,689 | 3,204 | **3,533** | | | |
| | LVTMS | | | | **3,388** | 3,176 | 2,580 | 3,263 | 4,109 | 3,837 | 3,345 | 2,728 | 3,185 | **3,387** | | | |
| SC3 EC3 | Simulation | | | **3,507** | 3,429 | 3,204 | 2,646 | 2,944 | 3,520 | 3,531 | 2,945 | 2,634 | 3,184 | 3,410 | **3,501** | | |
| | Accelerometer | | | **3,508** | 3,420 | 3,221 | 2,562 | 3,086 | 4,054 | 3,924 | 3,356 | 2,684 | 3,132 | 3,252 | **3,484** | | |
| | LVTMS | | | n/a | n/a | n/a | n/a | n/a | n/a | n/a | n/a | n/a | n/a | n/a | n/a | | |
| SC2 EC2 | Simulation | | **3,735** | 3,024 | 3,007 | 2,869 | 2,439 | 2,853 | 3,507 | 3,514 | 2,861 | 2,447 | 2,874 | 3,005 | 3,014 | **3,731** | |
| | Accelerometer | | **3,531** | 3,044 | 2,945 | 2,925 | 2,404 | 3,191 | 4,091 | 3,815 | 3,244 | 2,535 | 2,826 | 3,008 | 2,985 | **3,434** | |
| | LVTMS | | **3,518** | 2,942 | 2,892 | 2,865 | 2,346 | 3,123 | 4,032 | 3,806 | 3,261 | 2,527 | 2,865 | 2,945 | 2,939 | **3,421** | |
| SC1 EC1 | Simulation | **3,729** | 3,356 | 2,666 | 2,713 | 2,651 | 2,312 | 2,803 | 3,506 | 3,505 | 2,802 | 2,311 | 2,647 | 2,708 | 2,658 | 3,364 | **3,729** |
| | Accelerometer | **3,720** | 3,180 | 2,682 | 2,681 | 2,741 | 2,299 | 2,992 | 3,713 | 3,711 | 2,827 | 2,509 | 2,740 | 2,757 | 2,656 | 3,082 | **3,662** |
| | LVTMS | **3,763** | 3,150 | 2,715 | 2,685 | 2,738 | 2,296 | 2,988 | 3,731 | 3,697 | 2,822 | 2,525 | 2,736 | 2,727 | 2,638 | 3,138 | **3,658** |

Note: LVTMS designates laser vibrometer test meter system.

Figure 7: Deck elevation of the girders after the first pulling of the cables.

(2) The portable vibrometer has been found to be an effective way to check the tension of the cable and the maintenance of the bridge in cable-stayed bridges.

(3) The resultant tensional forces on cables show little difference from the conventional way using accelerometers. In other words, the maximum discrepancy in tensile forces between the ones from accelerometer and laser vibrometer is within 5%.

In the application of this proposed system to the construction of real cable-stayed bridges, we successfully measured and controlled the tensile forces of cables in cable-stayed bridges before the second pulling through the error correction work. The error correction work includes the jacking force calibration, geometric control of cables, and cable tensile forces. It proved that the error correction can be performed by comparing the data of structural analyses

to the measurements using the laser vibrometer system. In addition, the influence matrix of cable forces obtained from the laser vibrometer system can be used for the adjustments of the tension forces for the second pulling of cables based on simulation analysis.

## Acknowledgments

The authors gratefully acknowledge the financial support of the Super-Long Span Bridge R&D Project from the Korea Ministry of Land, Transportation Maritime Affairs (MLTM). The authors also wish to express their gratitude to Dr. Jong-Han Lee, R&D Center at POSCO E&C, for his review and comments on this paper.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 282139, 7 pages
doi:10.1155/2012/282139

*Research Article*

# Fatigue Damage Evolution and Monitoring of Carbon Fiber Reinforced Polymer Bridge Cable by Acoustic Emission Technique

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Received 18 June 2012; Accepted 13 August 2012

Academic Editor: Ting-Hua YI

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Desirable properties of carbon fiber-reinforced plastic (CFRP) composites include their high strength, high rigidity, light weight, corrosion free, and fatigue resistance. CFRP composites are popularly applied in bridge engineering structures, but the causes of fatigue damage in CFRP bridges have not been thoroughly investigated. We adopt acoustic emission (AE) technology to monitor fatigue damage and failure of CFRP bridge cables. The relationship between AE signal characteristics and CFRP cable fatigue damage, as well as the pattern of AE signals during a fatigue test, is investigated. Results show that the failure models exhibit matrix and fiber-matrix interface failures at the initial stage of fatigue testing, followed by delamination and fiber rupture. The $b$ value, Kurtosis index, and RA value based on AE characteristic parameters are proposed to describe the different damage stage failure modes. Finally, the failure types of AE waveform are extracted and analyzed using wavelet transformation. The AE technique proved to be a potential means for evaluating the fatigue damage characteristics of CFRP cables.

## 1. Introduction

The stay cable, a high-strength steel wire, is an important component of stayed bridge construction. A large sample survey has shown that the corrosion of stayed cable steel wires is the main cause of failure mode. Many cables in China are prematurely replaced because of corrosion, even if the cables have only been in service for a period of ten years. The cost of cable replacement has reached 3 to 4 times more than the cost of a new cable. Thus, the durability of stayed cable should be improved. There are currently two methods of enhancing the durability of a stay cable, namely, by applying protective measures to improve corrosion resistance and the use of corrosion-resistant materials. The latter has a wide range of applications based on theoretical analysis, experimentation, and results of field implementation [1]. Structural health monitoring technique has been widely developed in recent years. For example, Yi and Li proposed some optimal methods to identify and evaluate civil engineering structure [2–4].

The carbon fiber-reinforced polymer (CFRP) has many remarkable advantages. This material is advantageous for cables because it is lightweight, rigid and has high strength-to-weight ratio. However, damage to cables is not visible and can be generated in the manufacturing or service processes, and these damages can seriously affect the durability of structures. Hence, the study of the failure process and breakage mechanism has emerged as a major issue for the structural development, design, and quality inspection of composite materials. To date, various testing methods, such as X-ray detection, ultrasound detection, acoustic emission (AE), and so on, are used to evaluate the damage of composite materials. The AE technique provides damage information and is an effective nondestructive method for examining dynamic failure [5]. The AE source is generated from a range of deformation and fracture mechanisms of materials produced under stress. For example, the failure mechanisms for matrix cracking, interface of fiber and matrix debonding, interface separation, and fiber fracture of composite material are important AE sources. The main key in the study of damage and failure of composite materials is to distinguish the relationship between material failure modes and AE signal characteristics.

AE analysis relies on conventional AE features. An amplitude distribution-based criterion has been applied in distinguishing fiber composite failure modes. High amplitude signals have been correlated with the fiber failure mode,

and lower amplitude signals with the matrix cracking mode [6, 7]. The frequency-peak magnitude distribution, event and energy versus duration, has shown clear differences in different damage stages and could explain different damage type mechanisms [8–10]. The average frequency shift from high to low values indicates the shift of the cracking mode from tensile to shear [11]. The coefficient denoted by $R_{AE}$ and the cumulative Benioff law can predict residual fatigue life [12]. Sometimes, however, the simple AE parameter analysis method becomes invalid because of the overlapping of different failure modes. The simultaneous study of various AE features has permitted researchers to obtain more reliable information for the identification of AE source mechanisms, especially when associated with pattern recognition algorithms [13]. The $b$ value is used to evaluate seismic activity and predict earthquake in earthquake engineering. The lower $b$ values in AE application are usually associated with discontinuous crack growth, and the high values can be caused by the plastic zone growth prior to crack propagation. Pattern recognition and neural networks represent a more advanced data analysis methodology that can be applied to AE data. The methods have been verified to identify composite damage sequence and types [13]. Wavelet analysis of AE waveform has undergone rapid development. The wavelet decomposition result shows that different frequency levels reveal different potential failure modes, for example, 300 kHz for fiber failure, 250 kHz for fiber-matrix debonding, and 110 kHz for matrix cracking [14].

The CFRP bridge cable fatigue damage was monitored in this study using the AE technique, and its evolution process was analyzed, as well as described, based on the AE monitoring data. The failure modes of the CFRP bridge cable such as matrix cracking, debonding, and fiber breakage were identified using the kurtosis and the $b$ values. Different damage type waveforms were obtained based on the analysis of the abrupt change points in the kurtosis and the $b$ value curves. The frequencies of AE signals were obtained using wavelet transformation and were used to identify various failure modes.

## 2. Experimental Procedure

We made two 2 m long cables, each cable inner including seven parallel CFRPs with a diameter of 7 mm. The CFRP cable model was fixed to the MTS fatigue testing machine. The load control mode was used to perform the fatigue test. The fatigue cycle was sinusoidal with a frequency of 5 Hz. The minimum load was 30 N, and the maximum load was 0.45 times the cable breaking force. Three R15 AE transducers with resonant frequency 150 KHz were employed. The AE transducers are made by American Physical Acoustics Corp. Two of which were installed on the anchor as alarm transducers to filter the signals generated from the friction of the anchor and clamp. The AE transducers were fixed using a magnetic mechanical clamp. The third AE transducer was installed in the middle of the specimen and was designed for receiving the signal generated by the cable fatigue damage of the cable. A ferrule was attached to the middle of the cable to fix the AE transducer, which was mounted on the ferrule. We

Figure 1: Schematic representation of the experimental setup for the AE.

performed the broken pencil experiment prior to the fatigue experiment to test the sensitivity of AE transducers and to ensure that the three transducers have the same sensitivity. Acoustic signals were monitored during the fatigue test by an AE measuring system (MISTRAS-2001, Physical Acoustic Corporation). The monitoring data were collected via the AEwin software. Instrument settings were set at a fixed threshold level of 45 dB, preamplifier gain of 40 dB, main amplifier gain of 20 dB, sampling rate of 5 MHz, and filter frequency range from 100 kHz to 1 MHz. The experimental device used during the test is illustrated in Figure 1.

## 3. Test Results and Discussion

*3.1. Total AE Activity of the CFRP Bridge Cable Fatigue Damage.* The AE parameter correlation method is simple, intuitive, and practical, making it the most widely used in practical projects. The entire fatigue damage process can be visually represented by the relationship of AE parameters.

Combining the description of AE signals for fiber composites in literature [15] with the characteristic signal of AE signals and SEM photograph shown in Figures 2 and 3, we conclude that AE signals were of low energy, low amplitude, and short duration in the initial stage of the fatigue test of CFRP bridges, although some high-energy and high-amplitude signals were occasionally recorded. The main sources of these irregular signals are the internal fraction of the CFRP cable and some defects possibly generated during the production process. Stress is concentrated in the pre-existing defects, leading to the cracking of the carbon fiber. The damage accumulates as the fatigue test continues. The interfaces between the fiber bundles, as well as fiber bundles and the base, begin to break and lead to crack propagation

(a) Time history of AE energy

(b) Time history of AE amplitude

(c) Time history of AE Duration

(d) Time history of AE count

Figure 2: CFRP bridge cable fatigue damage AE characteristic parameters.

Figure 3: The FRP breakage SEM photograph [15].

because the base materials and reinforcing materials exhibit different properties during elongation and fatigue. Moreover, the cracked fibers at the base have relatively high stress. The fiber breaks when the stress reaches the fatigue limit of the fiber, and higher amplitudes can be obtained from 55 dB to 80 dB. In addition, the energy signal generates a higher value and a longer duration compared with the initial stage of the fatigue test. A considerable amount of high-amplitude and high-energy signals are generated after the test, and the peak value of these signals reaches 100 dB. The damage that occurs at this stage results from the fracture of the fibers after reaching their limit, and many fibers are pulled out from the anchor.

We qualitatively assess the condition of the carbon fiber cables during the whole process of the fatigue test and evaluate the failure modes based on Figure 3. We also reach the conclusion that different types of damage always occur simultaneously, resulting in the superposition of AE signals. However, we cannot effectively evaluate the types of damage solely based on the characteristic parameters of the AE test.

### 3.2. CFRP Bridge Cable Failure Analysis

*3.2.1. CFRP Bridge Cable Fatigue Damage Evaluation with AE Parameters.* Aside from the direct analysis of AE parameters and their relationship using the AE signal time-domain analysis method, the statistical time indicators of AE parameters can be calculated to analyze comprehensively the evolution of

Figure 4: $B$ value distribution over time.

Figure 5: Kurtosis index distribution over time.

fatigue damage of carbon fiber cable and identify accurately the various stages of fatigue damage process. Two AE evaluation methods are proposed to determine the ability of these indicators to assess the damage levels in the evolution of CFRP bridge cable fatigue damage.

The $b$ value is proven to be an effective method in studying fracture mechanism. The $b$ values for different materials also show different ranges of values. This value is calculated in the AE technique as follows [16]:

$$\log_{10} N = a - b \times A_{\mathrm{dB}}, \tag{1}$$

where $N$ is an AE event, and $A_{\mathrm{dB}}$ is the peak amplitude of the AE event in decibels. $a$ and $b$ are empirical constants. The $b$ value is obtained by the least squares method.

Kurtosis is a measure of the impulsive nature of the signal. Thus, this condition can be used to evaluate potential defects and the service capabilities of the carbon fiber cable. Kurtosis is calculated as follows [17]:

$$\mathrm{Kurtosis} = \frac{(1/N)\sum_{n=1}^{N}\left(x(n) - (1/N)\sum_{n=1}^{N} x(n)\right)^4}{\left[(1/N)\sum_{n=1}^{N}\left(x(n) - (1/N)\sum_{n=1}^{N} x(n)\right)^2\right]^2}, \tag{2}$$

where $N$ is the length of each group AE characteristic parameter (energy) $x(n)$.

The AE characteristic parameters of the entire fatigue process are equally divided into 100 subintervals to simplify the calculations. The calculated results of the kurtosis index and the $b$ value are shown in Figures 4 and 5.

The kurtosis and the $b$ values have the same variation. The peak values in these figures show the failure generated, and the different amplitude values show the degree of fatigue damage.

The activity of the AE source is weak at the start of the test, and the main type of AE signal at this stage is a low-amplitude continuous pulse. The AE waveforms are generated by the rupture of the internal bubble of the CFRP as well as the friction between the anchor and specimens. Damage is accumulated during the fatigue test. The interface of the fiber and matrix begins to break, and a macrocrack is propagated. A few bursts with high amplitude are also generated in the range of 28485 s to 56970 s. The high-amplitude signals show that the activity of AE source is enhanced, and that the cable defects increase with each loading cycle, resulting in matrix separation from the fibers along the area leading to edge of the crack. The process eventually leads to interfacial debonding. The values exhibit substantial changes when the cable enters the stage during which the crack is propagated rapidly. The waveform generated by the interface of the carbon and matrix is damaged, and the carbon fiber is pulled out from the matrix. The essence of carbon fiber and matrix recombination is to form a complete interface upon the fiber surface to transfer stress, and fatigue load is transferred from the matrix to the reinforcement through this interface. Therefore, the structural integrity of the interface plays an important role in the overall performance of the material. The severe damage sustained by the interface will hasten the rate of damage of the carbon fiber cable. Therefore, microcracks expand in an unstable manner during this stage. The fiber pulls out from the fiber bundles, and a high-amplitude pulse waveform is generated.

*3.2.2. CFRP Bridge Cable Fatigue Damage Fracture Mode.* Different fracture modes produce different AE signals. For example, plastic deformation appears as a continuous waveform, whereas fracture appears as a burst waveform. Aggelis [18] found that a crack in a tensile mode has a short rise time and high-frequency AE waveforms. By contrast, shear-type cracks usually result in longer waveforms with lower frequency and longer rise time. The rise time/amplitude (RA, Figure 6) values are proposed to describe different fracture modes. Aggelis concluded that lower RA is exhibited during the early damage stage, but this value increases as the material approaches total failure. Thus, lower RA expresses tensile crack, and higher RA expresses shear crack.

Figure 6: Typical AE waveform.

Figure 7: Time history of CFRP bridge cable fatigue damage RA value.

RA value is defined as

$$\mathrm{RA} = \frac{\text{Rise time}}{\text{Amplitude}}. \tag{3}$$

The RA value is defined as the angle where the AE signal crosses the threshold to the maximum amplitude for the first time. The RA value of the entire fatigue damage process is shown in Figure 7.

The RA value gradually increases until the end of the experiment. The activity of the AE source is enhanced during the damage initiation stage, and the cable defects increase with each loading cycle. This phenomenon eventually leads to interfacial debonding. Furthermore, matrix material is easily damaged under fatigue load because of its low ductility. The cracking is due to the tensile load. Delamination occurs within the loading cycle, and large quantities of fiber are pulled out from the fiber bundles because of the interface separation during the late stage of fatigue damage, resulting in eventual carbon fiber fracture. These events resemble shear-type failure caused by the friction between the fibers and the surrounding matrix. The RA shift to higher values indicates that shear stresses gradually dominate the failure process although the cracks are initiated by tension. The same case has also been reported recently for vinyl fiber and steel fiber concrete as well as metal plate tensile fatigue fracture [18–20].

## 4. Failure Modes Classified by AE Waveforms

The damage waveforms were extracted based on the location of each high burst point presented in Figures 4 and 5, and the time-frequency method was used to analyze the waveforms. Wavelet functions are localized both in frequency (or scale) and time via dilations as well as translations of the mother wavelet. Wavelet transform offers a self-adjusted and localized method that automatically adjusts the time-frequency window to analyze different signals in the frequency domain, thus fulfilling our demand for practical analysis. The wavelet transformation theory has been reported in a number of documents [21, 22]. The equation corresponding to the wavelet transformation theory was not shown in this paper.

In this paper, the CFRP cable fatigue damage type was analyzed by AE waveform, that is, only a preliminary investigation and qualitative differentiation method. The waveforms and the time-frequency spectra for matrix cracking, interface separation, and fiber fracture are shown in Figure 8 to reveal the distribution features in the time-frequency spectrum for different fatigue damage waveforms.

The different damage mechanisms of the AE signal have different characteristics of waveform and frequency scale (Figure 8). The waveform of matrix fracture may be described as a low-amplitude, wide pulse and has a wide frequency range from 64 kHz to 512 kHz, mostly concentrated at 128 kHz. The frequency range of delamination ranges from 64 kHz to 1024 kHz, mostly concentrated at 256 kHz. The frequency scale of a fiber fracture is wide, with a minimum frequency less than 64 KHz and crest amplitude of frequency reaching over 1024 KHz, mostly concentrated from 256 kHz to 512 kHz.

## 5. Conclusions

The AE technique was used to monitor the damage process in CFRP cable, and the AE data of the entire fatigue test was analyzed to examine the fatigue damage evolution process. The results show what follows.

(1) The entire fatigue damage process in carbon fiber cable can be represented visually by the AE parameter correlation diagram. The stage of steady crack propagation has a long duration and takes up 88% of the total fatigue life. The AE parameters are fluctuating within a small range in this phase. However, the AE signal shows the features of high-amplitude and wide frequency once the crack reaches the unstable microcrack growth phase.

(2) The breakage evolution process in carbon fiber cable was analyzed using the kurtosis index, and $b$ and

(a) The AE waveform of matrix fracture

(b) The AE waveform of delamination

(c) The AE waveform of fiber fracture

Figure 8: The time-frequency analysis of CFRP cable fatigue damage AE signal.

RA values. The location and extent of fatigue failure in the entire process were reflected clearly from the changes in the indicators.

(3) The matrix fracture, delamination, and fiber fracture waveforms were extracted from large transformations of diagnostic criteria. These are the major damage types of a CFRP cable. Different waveforms have different frequency distributions according to the wavelet transformation time-frequency analysis. High-amplitude signals have been correlated with the fiber failure mode and lower amplitude signals with the matrix cracking mode.

## Acknowledgments

The authors are grateful for the financial support from National Natural Science Foundation of China (NSFC)

under Grant no. 50808030, Doctoral Fund of Ministry of Education of China under Grant no. 200801411102, and National Science and Technology Support Program of China no. 2011BAK02B04.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 167120, 12 pages
doi:10.1155/2012/167120

*Research Article*

# Temperature-Compensated Damage Monitoring by Using Wireless Acceleration-Impedance Sensor Nodes in Steel Girder Connection

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Received 27 June 2012; Accepted 23 August 2012

Academic Editor: Hong-Nan Li

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Temperature-compensated damage monitoring in steel girder connections by using wireless acceleration-impedance sensor nodes is experimentally examined. To achieve the objective, the following approaches are implemented. Firstly, wireless acceleration-impedance sensor nodes are described on the design of hardware components to operate. Secondly, temperature-compensated damage monitoring scheme for steel girder connections is designed by using the temperature compensation model and acceleration-impedance-based structural health monitoring methods. Finally, the feasibility of temperature-compensated damage monitoring scheme by using wireless acceleration-impedance sensor nodes is experimentally evaluated from damage monitoring in a lab-scaled steel girder with bolted connection joints.

## 1. Introduction

Steel structures have structural connections such as bolted joints. Potential damage types of the structural connections include fatigue cracks between bolt holes, the change in tensile force of bolts, and failures of connection components. Structural health monitoring (SHM) on those structural connections becomes an important topic since damage occurred in structural connections, which is not detected or remedied appropriately, may result in local failure, reduction of load carrying capacity, or catastrophic disaster [1].

Up to date, many studies have been focused on SHM of structural connections by using global and local dynamic characteristics [2–11]. These studies have been mainly focused on developing SHM methods by using a single physical quantity such as strain, acceleration, or electromechanical impedance. However, the reliability of SHM methods using a single sensing device is relatively low compared to the case of multiphysical quantities. Therefore, the SHM scheme using multiscale sensing mechanism can be an alternative approach [12–15].

Also, the high costs associated with wired SHM systems can be greatly reduced through the adoption of wireless sensors. An advantage of wireless sensor is that the automated operation can be implemented by embedded operation software. This fact leads to a new paradigm that adopts smart sensors for autonomous and cost-efficient SHM [16–22]. Straser and Kiremidjian [23] first proposed a design of a low-cost wireless modular monitoring system (WiMMS) for SHM applications. Lynch et al. [24] improved the performance of WiMMS in which acceleration-based damage monitoring algorithms are embedded. Nagayama et al. [25] used Imote2 sensor platforms from Memsic Co. [26] for acceleration-based SHM of truss structures. Park et al. [14] developed acceleration-based and impedance-based smart sensor nodes (Acc-SSN and Imp-SSN) which are modified from the sensor nodes by Lynch et al. [24] and Mascarenas et al. [27], respectively.

However, there still exist two issues that should be solved before real applications on SHM methods of steel girder bridges in the field. Firstly, the temperature-driven variability of structural response data should be quantified

in the determination of features such as modal parameters or impedance signatures. Therefore, an issue arises on how to distinguish the temperature-induced variability on feature extraction. Secondly, the detected output may be true, false-positive, or false-negative. Even in the true case, moreover, damage-localization error and severity-estimation error are inevitable due to the temperature effect [28, 29].

In this paper, temperature-compensated damage monitoring in steel girder connections by using wireless acceleration-impedance sensor nodes is experimentally examined. Firstly, wireless acceleration-impedance sensor nodes are described on the design of hardware components to operate. Secondly, temperature-compensated damage monitoring scheme for steel girder connections is designed by using acceleration-impedance-based SHM methods. The temperature compensation model selected the linear regression analysis. The acceleration-based SHM methods selected the correlation coefficient of power spectral density and frequency-based damage index. The impedance-based SHM method selected the correlation coefficient of impedance. Finally, the feasibility of temperature-compensated damage monitoring scheme by using wireless acceleration-impedance sensor nodes is experimentally evaluated from damage monitoring in a lab-scaled steel girder with bolted connection joints.

## 2. Wireless Acceleration-Impedance Sensor Nodes

A wireless acceleration-impedance sensor node can be defined as a sensor node to simultaneously measure multiple physical quantities from structures in different scales. For the SHM using acceleration and impedance measurements, a wireless acceleration-impedance sensor node on Imote2 platform is designed as shown in Figure 1. The Imote2 sensor platform was selected to control peripheral devices such as microcontroller, wireless radio, and memory. For acceleration measurement, a SHM-A sensor board developed by Rice and Spencer [30] was selected. For impedance measurement, an SSeL-I sensor board developed by Kim et al. [1] was selected. As shown in Figure 2(a), the prototype of the multiscale sensor node is consisted of four layers. The first and second layers are a battery board (IBB2400) and the Imote2 sensor platform (IPR2400), respectively. The third and fourth layers are the SHM-A acceleration sensor board and the SSeL-I impedance sensor board, respectively.

The Imote2 sensor platform incorporates a low-power X-scale process, PXA27x, and a wireless radio, CC2420. The microcontroller PXA27x runs for multiple tasks which include operation schedule, system control, and radio transmission. Also, the Imote2 has 256 kB of integrated SRAM, 32 MB of external SDRAM, and 32 MB of program flash memory. The memory repository will guarantee to store large amount of data measured by a group of accelerometers and PZT patches. The data processing speed of the Imote2 is faster enough to provide good computational capability and the transmitting distance can be expanded up to 125 m by using an external antenna.

Figure 1: Design of acceleration-impedance sensor nodes on Imote2-platform.

As shown in Figure 2(b), the SHM-A sensor board was selected for acceleration measurement. The SHM-A sensor board should have suitable capabilities for key components such as accelerometer, noise density, antialiasing filter, and analog-to-digital converter (ADC). The sensor board provides three axis acceleration sensor (LIS344ALH) with relative low noise level, light sensor (TSL2561), and temperature and humidity measurements (SHT11). Also, the 4-channel 16-bit high-resolution analog-to-digital converter (ADC) with digital antialiasing filters (QF4A512) is adopted. The ADC converts analog signal to digital data by 16-bit resolution but it guarantees 12-bit resolution. By adopting the digital filters, the sensor board provides user-selectable antialiasing filters and sample rates that can meet a wide range of application demands for infrastructure monitoring.

As shown in Figure 2(c), the SSeL-I sensor board was selected for impedance-based SHM. The SSeL-I is consisted of an impedance converter AD5933, two pull-up resistors for I2C communication, two capacitors for bypassing noises, a connector to a PZT patch and two connectors to the SHM-A sensor board, and the Imote2 sensor platform. Two pull-up resistors are utilized for I2C interface communication between the SSeL-I board, and the Imote2 platform. The microcontroller PXA27x and wireless radio CC2420 in the Imote2 platform are utilized for the impedance measurement. The AD5933 impedance converter has the following embedded multifunctional circuits: function generator, digital-to-analog (D/A) converter, current-to-voltage amplifier, antialiasing filter, A/D converter, and discrete Fourier transform (DFT) analyzer. The AD5933 converter outputs real and imaginary values of impedance signatures for a target frequency of interest and transmits the values into a microcontroller.

## 3. Temperature-Compensated Damage Monitoring Scheme

Kim et al. [1] proposed a hybrid damage monitoring scheme by using acceleration and impedance signatures. Based on this study, the temperature-compensated damage monitoring scheme is designed as schematized in Figure 3. It consists

(a) Acceleration-impedance sensor nodes on Imote2-platform

(b) SHM-A sensor board

(c) SSeL-I sensor board

Figure 2: Prototypes of acceleration-impedance sensor nodes on Imote2-platform.

of three phases: (1) global damage monitoring in overall structure and local damage monitoring in critical sub-structural points by temperature compensation, (2) damage-occurrence alarming and damage-type identification by temperature compensation, and (3) damage localization and severity estimation by temperature compensation of the identified damage.

In Phase 1, the damage occurrence is globally monitored by measuring acceleration responses from the target structure. At the same moment, the damage occurrence is monitored at local critical members by measuring changes in electromechanical impedance signatures. Firstly, a global damage-monitoring method using CC of PSD [1] and control chart analysis [31] is implemented to monitor damage occurrence in the entire structure. If any damage occurs in the structure, the acceleration responses would be affected and consequently the CC of PSD values would be decreased. Secondly, local damage monitoring method using CC of impedance [32] signatures and control chart analysis is selected to monitor the occurrence of damage at the local sensor-vicinity zone. The electromechanical impedance is sensitive to the sensor-vicinity zone but almost insensitive to the remaining structure, which makes it feasible to be indicative for the occurrence of damage in localized area.

In Phase 2, the occurrence of damage is alarmed and the type of damage is locally identified by recognizing patterns of the CC of PSD/impedance features [1]. The alarmed damage is classified (localized) as damage occurred at specific locations (i.e., prescribed damages: girder stiffness loss by bolt loosen) or elsewhere in the structure. There are four possible patterns of damage alarming situations: (1) no damage alerted either globally or locally, (2) damage alerted both globally and locally, (3) damage alerted globally but not locally, and (4) damage alerted locally but not globally. In the first pattern, no damage would be occurred in the structure if the acceleration-based monitoring does not alert damage occurrence in the entire structure and the impedance-based monitoring does not alarm damage occurrence at the local zone. In the second pattern, damage would be apparently occurred at the local critical zone if the acceleration-based monitoring alarms the occurrence of damage in the structure and the impedance-based monitoring also alarms the occurrence of damage at the local zone. In the third pattern, damage might be occurred elsewhere than the preselected local zones if the acceleration-based monitoring alarms the occurrence of damage in the structure but the impedance-based monitoring does not indicate the occurrence of damage at the local zones. In the final pattern, incipient small damage would be occurred at a local critical zone if the acceleration-based monitoring does not alert damage occurrence but the impedance-based monitoring indicates damage occurrence at the local zone.

In Phase 3, the location and severity of the identified damage by using frequency-based damage index method are estimated in details. The damage estimation by using frequency-based damage index method is performed by a few temperature-compensated natural frequencies of target structures.

*3.1. Temperature Compensation by Linear Regression Analysis.* Regression analysis gives information on the relationship between a response variable and one or more independent variables to the extent that information is contained in the data. The goal of regression analysis is to express the response variable as a function of the predictor variables. The duality of fit and the accuracy of conclusion depend on the data used.

Once a regression analysis relationship is obtained, it can be used to predict values of the response variable, identify variables that most affect the response, or verify hypothesized causal models of the response. The value of each predictor variable can be accessed through statistical tests on the estimated coefficients (multipliers) of the predictor variables. An example of a regression model is the simple linear regression model which is a linear relationship between

Figure 3: Temperature-compensated damage monitoring scheme for steel girder connections.

response variable, $y$, and the predictor variable, $x$, of the form

$$y = \alpha + \beta x, \tag{1}$$

where $\alpha$, $\beta$ are regression coefficients (unknown modal parameters). The error term has to be equal to zero on average. In statistics, simple linear regression is the least squares estimator of a linear regression model with a single predictor variable. In other words, simple linear regression fits a straight line through the set of $n$ points in such a way that makes the sum of squared residuals of the model (i.e., vertical distances between the points of the data set and the fitted line) as small as possible.

*3.2. Damage Alarming by CC of PSD.* Assume that two acceleration signals $x(t)$ and $y(t)$ are measured before and after a damaging episode, respectively, their corresponding power spectral densities $S_{xx}$ and $S_{yy}$ are calculated from Welch's procedure as [33]

$$\begin{aligned} S_{xx}(f) &= \frac{1}{n_d T}\sum_{i=1}^{n_d} |X(f,T)|^2, \\ S_{yy}(f) &= \frac{1}{n_d T}\sum_{i=1}^{n_d} |Y(f,T)|^2, \end{aligned} \tag{2}$$

where $X$ and $Y$ are dynamic responses transformed into frequency domain; $n_d$ is the number of divided segments in time history; $T$ is data length of a divided segment.

The correlation coefficient of PSDs (CC of PSDs) represents the linear identity between the two PSDs obtained before and after a damage event:

$$\rho_{XY} = \frac{E\left[S_{xx}(f)S_{yy}(f)\right] - E[S_{xx}(f)]E\left[S_{yy}(f)\right]}{\sigma_{S_{xx}}\sigma_{S_{yy}}}, \tag{3}$$

where $E[\cdot]$ is the expectation operator, and $\sigma_{S_{xx}}$ and $\sigma_{S_{yy}}$ are the standard deviations of PSDs of acceleration signals measured before and after damaging episode, respectively. If any damage occurs in the garget structure, its acceleration responses would be affected and, consequently, the indication by the CC of PSDs can be a warning sign of the presence of damage [1]. A control chart analysis is used to discriminate damage events from the CC values [31]. The lower control limit (LCL) is determined as

$$\mathrm{LCL}_\rho = \mu_\rho - 3\sigma_\rho, \tag{4}$$

where $\mu_\rho$ and $\sigma_\rho$ are the mean and the standard deviation of the CC values, respectively. The occurrence of damage is indicated when the CC values are beyond (i.e., less than) the bound of the LCL; otherwise, there is no indication of damage occurrence. To use the CC of PSDs as a damage-sensitive index, it would be better to get rid of measurement uncertainty induced by inconsistent excitation conditions.

*3.3. Damage Alarming by CC of Impedance.* The active material is described by its short-circuited mechanical

(a) Girder dimension and sensor layout

(b) Cross section

(c) Splice plates

Figure 4: Schematic of Bolt-connected steel girder.

impedance, which is powered by voltage or current. The host structure is modeled as the effect of mass, stiffness, damping, and boundary conditions. When a piezoelectric patch is surface-bonded to a structure, the electrical admittance (the inverse of electromechanical impedance) of the patch, $Y(\omega)$ (units Siemens or ohm$^{-1}$), is a combined function of the mechanical impedance of the host structure, $Z_s(\omega)$, and that of the piezoelectric patch, $Z_a(\omega)$:

$$Y(\omega) = \frac{1}{Z(\omega)} = i\omega \frac{WI}{t_c}\left[\left(\varepsilon_{33}^{T} - d_{3x}^{2} Y_{xx}^{\wedge E}\right) + \frac{Z_a(\omega)}{Z_a(\omega)+Z_s(\omega)} d_{3x}^{2} Y_{xx}^{\wedge E}\left(\frac{\tan KI}{KI}\right)\right], \tag{5}$$

where $Y_{xx}^{\wedge E}$ is the complex Young's modulus of the at zero electric field; $\varepsilon_{33}^{T}$ is the dielectric constant of piezoelectric wafer; $d_{3x}$ is the piezoelectric coupling constant in the $x$ direction at zero stress; $K$ is the wave number that depends on mass density and Young's modulus of the piezoelectric material; $W$, $I$, and $t_c$ are, respectively, the width, length, and thickness of the piezoelectric transducer.

Equation (5) indicates that the electrical impedance of the piezoelectric patch bonded onto a host structure is directly related to the mechanical impedance of the structure. The first term of the equation is the capacitive admittance of the free piezoelectric patch. The second term includes the mechanical impedance of both the piezoelectric patch and the host structure. When damage occurs to a structure, its mechanical impedance will be shifted. Hence, any changes in the electrical impedance signature (such as magnitude of admittance and resonant frequency) are attributed to damage or changes in the structure.

To quantify the change in impedance signature due to damage in the structure, the CC of impedance signatures measured before and after damage [32] is used in this study. The CC is calculated from impedance measurements before and after damage as

$$\rho_{ZZ^*} = \frac{E[Z(\omega)Z^*(\omega)] - \mu_Z \mu_{Z^*}}{\sigma_Z \sigma_{Z^*}}, \tag{6}$$

where $Z(\omega)$ and $Z^*(\omega)$ are impedance signatures of a frequency band measured before and after damage, respectively. Also, $\mu_Z$ and $\sigma_Z$ are mean and standard deviation values of impedance signals. Control chart analysis is used to alert damage occurrence from the correlation coefficients of impedance signatures. Lower control limit is determined similarly as described in (4).

*3.4. Damage Estimation by Frequency-Based Damage Index.* Kim et al. [34] proposed a frequency-based damage index algorithm for a structural system of NE elements ($j = 1, 2, \ldots, q, \ldots, \mathrm{NE}$) and a measured set of NM vibration modes ($j = 1, \ldots, m, n, \ldots, \mathrm{NM}$). The system yields the $i$th natural frequency $\omega_i$ and $i$th mode shape vector $\{\phi_i\}$. Next, assume that at some later time the structure is damaged (e.g., stiffness loss) in one or more locations of the structure. The resulting characteristic equation of the damaged structure yields $\omega_i^*$ and $\{\phi_i^*\}$. Note that the asterisk denotes the damaged state.

As the $i$th modal information is available for the system, a damage index (DI) for the $j$th location can be defined as follows:

$$\mathrm{DI}_j = \left[\sum_{i=1}^{\mathrm{NM}} e_{ij}^2\right]^{-1/2}, \tag{7}$$

where $0 \le \mathrm{DI}_j < \infty$ and the damage is located at element $j$ if $\mathrm{DI}_j$ approaches the local maximum point. Here, $e_{ij}$

(a) Free-free steel girder

(b) Bolted connection

(c) Interface washer

Figure 5: Experimental setup of Bolt-connected steel girder.

represents localization error for the $j$th location that can be measured by using the $i$th modal information:

$$e_{ij} = \frac{Z_m}{\sum_{k=1}^{\mathrm{NM}} Z_k} - \frac{F_{mq}}{\sum_{k=1}^{\mathrm{NM}} F_{kq}}. \tag{8}$$

The condition $e_{ij} = 0$ indicates that the damage is located at the $j$th location using the $i$th modal information. The term $Z_i$ is the fractional change in the $i$th eigenvalue due to damage, by neglecting changes in mass, which is given by

$$Z_i = \frac{\delta\omega_i^2}{\omega_i^2}, \tag{9}$$

(a) Temperature variation

(b) Acceleration variation

(c) PSD variation

(d) Impedance variation

Figure 6: Temperature, acceleration, and impedance variation during test.

(a) Time history

(b) Linear relationship

Figure 7: Time history and linear relationship between temperature and CC of PSD.

(a) Time history

(b) Linear relationship

Figure 8: Time history and linear relationship between temperature and CC of impedance.

where $\delta\omega_i^2 = \omega_i^{*2} - \omega_i^2$. The term $F_{ij}$ is the sensitivity of $i$th mode and $j$th element, which defines the fraction of modal energy for the $i$th mode that is concentrated in the $j$th element:

$$F_{ij} = \frac{\{\phi_i\}^T \left[C_j\right] \{\phi_i\}}{\{\phi_i\}^T [C] \{\phi_i\}}, \tag{10}$$

where $\{\phi_i\}$ is the $i$th mode shape vector, $[C]$ is the system stiffness matrix, and $[C_j]$ is the contribution of the $j$th element to the system stiffness. The term $Z_m/Z_n$ is the ratio of the fractional change in $m$th eigenvalue to the corresponding fractional change in $n$th eigenvalue. Also, $F_{mq}/F_{nq}$ is the ratio of the sensitivity for $m$th mode and $q$th element to the sensitivity of $n$th mode and $q$th element.

## 4. Experimental Setup and Damage Scenarios

*4.1. Target Structure and Sensor Layout.* A lab-scaled steel girder model with bolted connections was used to evaluate the performance of the temperature compensated damage monitoring scheme. As shown in Figures 4 and 5, the girder is H-section (H-200 × 180 × 8 × 100), 4.14 m span length, and free-free boundary condition. As shown in the figures, two girder sections (2.07 m length each) are connected by splice plates and bolts on the flanges. Seven Imote2/SHM-A sensor nodes (Loc. 1–7) were placed on the web of the girder with constant interval of 0.64 m. As shown in Figures 4(c) and 5(c), an interface washer was installed to impedance measurement with Imote2/SSeL-I sensor node (Loc. 4). Therefore, Loc. 4 location was placed with Imote2/SHMA/SSeL-I sensor node. Other locations were placed with Imote2/SHM-A sensor nodes. A PZT (10 mm × 10 mm, PZT 5A type) was placed on the interface washer. The interface washer was placed between the splice plate and Bolt 2. The measurable range of the wireless SSeL-I sensor board is much smaller than that of the wired commercial impedance analyzer. The disadvantage may interfere with wide applications of the SSeL-I sensor board into real structures. To overcome the disadvantage, an interface washer proposed by Park et al. [14] is employed as a complement device for high-sensitivity and fixed-range impedance measurement by the SSeL-I sensor board. By using the interface washer, it may provide the following benefits: (1) sensitive impedance features to the change in

(a) Mode 1

(b) Mode 2

Figure 9: Linear relationship between temperature and natural frequencies.

(a) CC of PSD

(b) CC of impedance

Figure 10: Temperature-compensated damage monitoring results.

structural system and (2) relatively constant frequency range independent of target structures.

The structure was tested in a lab (Smart Structure Engineering Lab) located at Pukyong National University, Busan, Korea. A series of tests were performed from 22 July, 2010 to 22 January, 2011. When these tests were performed, temperature varied between 5.7°C and 29.9°C as shown in Figure 6. Temperature, acceleration, and impedance data were measured and recorded on every one hour. Temperatures had been measured by using K-type thermocouple wires and KYOWA (EDX-100A) Temperature Logger.

*4.2. Temperature Compensation Results by Linear Regression Analysis.* The temperature-compensated damage monitoring of steel girder connections was performed after compensation between temperature and acceleration/impedance. The temperature compensation of temperature acceleration and temperature impedance was extracted by the linear regression analysis. In statistics, regression analysis includes many techniques for modeling and analyzing several variables, when the focus is on the relationship between a dependent variable and one or more independent variables. The linear regression analysis is the simplest approach to analyze the relationship between a response variable ($y$) denoted acceleration/impedance response features and a predictor variable ($x$) denoted temperature. And the linear regression analysis is classified into simple linear regression and polynomial regression. The simple linear regression is the least squares estimator of a linear regression model with a single predictor variable. The polynomial regression is a form of linear regression in which the relationship between the predictor variable and the response variable is modeled as $n$th order polynomial. Figures 7–9 show the results of linear regression analysis between acceleration/impedance response features and temperature by using simple linear and polynomial regression model. The linear relationship of temperature and acceleration and impedance features

Figure 11: Temperature-compensated damage estimation results.

was extracted from the reference data at the temperature of 29.9°C. As shown in Figure 7, CC of PSD presents about 3∼40% error between simple linear regression and polynomial regression on temperature variation. And CC of impedance presents about 8∼17% error in Figure 8. The natural frequencies present about less 1% error between simple linear regression and polynomial regression in Figure 9. Those results show that the natural frequencies show linear dependence with temperature variation although there is no clear linear dependence between CC of PSD and temperature. In despite of this error, the simple linear regression analysis is selected for providing good performance for wireless acceleration-impedance sensor node.

The temperature compensation between temperature and CC of PSD was extracted by the linear regression. Figure 7(a) shows the time history between CC of PSD and temperature by using the measured data during 6 months. Figure 7(b) shows the linear regression analysis results between temperature and CC of PSD. By these analysis results, CC of PSD was changed 0.0124 to values by temperature change of 1°C.

The temperature compensation between temperature and CC of impedance was extracted by the linear regression analysis. Figure 8(a) shows the time history between CC of impedance and temperature by using the measured data during 6 months. Figure 8(b) shows the linear regression analysis results between temperature and CC of impedance. By these analysis results, CC of impedance was changed to 0.0138 values by temperature change of 1°C.

The temperature compensation between temperature and natural frequencies was extracted by the linear regression analysis. Figure 9 shows the linear regression analysis results between temperature and natural frequencies. By these analysis results, the first natural frequency (Mode 1) was changed to 0.029 values by temperature change of 1°C. And the second natural frequency (Mode 2) was changed to 0.032 values by temperature change of 1°C.

*4.3. Damage Scenarios.* The bolt loosen was selected for general damage type of steel girder connections. Table 1 shows damage scenarios for bolt loosen of the steel girder connections. The damage scenarios include one reference and two bolt-loosen damage cases. The reference case is that all bolts are fastened by torque of 160 N-m. The damage cases are loosened to torque of 35 N-m by Bolt-1 and Bolt-2. The temperature variations ($\Delta T$) between Reference and Bolt-1 and Bolt-2 are 14.3 and 14.4°C.

## 5. Temperature-Compensated Damage Monitoring Results

*5.1. Damage Alarming Results.* The temperature-compensated damage alarming was performed by using acceleration-impedance-based SHM results. Firstly, the acceleration-based SHM results presented the change of global structural behavior by using the temperature-compensated CC of PSD in steel girder connections. Secondly, the impedance-based SHM results presented the change of identified local structural behavior (by the location of contacted PZT patch) by using the temperature-compensated CC of impedance.

For Reference and two damage cases (Bolt-1 and Bolt-2), acceleration and impedance signals up to twenty-four and four ensembles were measured from Loc. 4 sensor. As shown in Figure 10, twenty-four ensembles of the Reference (acceleration and impedance) were used to decide an LCL for damage alarming. An LCL mean (standard deviation) value is 0.9954 (0.0041) for CC of PSD. When the CC of PSD is dropped under LCL, the damage monitoring scheme makes a decision to make global damage in steel girder connection. An LCL mean (standard deviation) value is 0.9910 (0.0045) for CC of impedance. When the CC of impedance is dropped under LCL, the damage monitoring scheme makes a decision to make identified local damage in steel girder connection.

As the effect of temperature compensation, Bolt-2 was successfully alerted by both the wireless system before and after temperature compensation. These results apparently decided that damage type is classified Bolt-2 damage. The PZT patch is directly contacted Bolt 2. So, Bolt-2 should be detected as damage by LCL analysis of CC of impedance regardless of temperature compensation. The temperature compensation of this result shows only improvement of damage grade in alarmed Bolt 2 (compared before temperature compensation, CC of PSD/impedance had adjacent value on decided LCL). As secondly case, Bolt-1 apparently presents the effect of temperature compensation. In Bolt-1, the results before temperature compensation alarms damage occurrence by dropping under LCL of CC of PSD/impedance. Bolt-1 damage was not detected by CC of impedance (PZT patch is directly contacted Bolt 2). This result was shown to be false alarming result. Then, Bolt-1 damage was not alarmed by CC of impedance after temperature compensation. These results apparently decided that damage type is the damage of other location except classified Bolt-2 damage.

*5.2. Damage Estimation Results.* Although damage case Bolt-2 did not present the effects of temperature compensation, the two damage cases were apparently alarmed and

Table 1: Damage scenarios inflicted in bolt-connected steel girder.

| Damage case | Damage scenario | Temperature (°C) | | |
|---|---|---|---|---|
| | | Mean | SD | Number of data |
| Reference | All bolts fastened by 160 N-m | 6.4 | 0.4 | 24 |
| Bolt-1 | Bolt 1 loosened by 35 N-m (all others remained as 160 N-m) | 20.7 | — | 4 |
| Bolt-2 | Bolt 2 loosened by 35 N-m (all others remained as 160 N-m) | 20.8 | — | 4 |

Table 2: Natural frequencies before and after temperature compensation for steel girder connection.

| Case | Damage scenario | Temperature (°C) | | Natural frequency (Hz) | | | |
|---|---|---|---|---|---|---|---|
| | | | | Before compensation | | After compensation | |
| | | $T$ | $\Delta T$ | Mode 1 | Mode 2 | Mode 1 | Mode 2 |
| Reference | All bolts fastened by 160 N-m | 6.4 | — | 81.13 | 208.10 | 81.13 | 208.10 |
| Bolt-1 | Bolt 1 loosened by 35 N-m (all others remained as 160 N-m) | 20.7 | −14.3 | 80.59 | 207.59 | 81.0 | 208.05 |
| Bolt-2 | Bolt 2 loosened by 35 N-m (all others remained as 160 N-m) | 20.8 | −14.4 | 80.73 | 206.81 | 81.15 | 207.27 |

classified to correct damage types. Then, the temperature-compensated damage estimation was performed based on these results. The damage estimation performed to predict damage location and size by using frequency-based damage index method. The frequency-based damage index method needed initial natural frequencies and mode shape. The natural frequencies used extracted values of Reference and two damage cases. Also, the mode shape only used extracted values of Reference. Based on these facts, the frequency-based damage index method was important to extract accurate natural frequencies before and after damage sates. Modal parameters (natural frequency and mode shape) were extracted from acceleration signals measured at the seven sensor locations (i.e., Loc. 1–Loc. 7) by using the frequency domain decomposition method [35, 36].

For the Reference and the two damage cases, the corresponding natural frequencies measured by before and after temperature compensation are listed in Table 2. Then modal curvatures were analyzed from the postprocessed mode shapes, from which modal strain energies of girder elements were computed. As shown in Figure 11, the damage locations were predicted by using modes 1 and 2. In case of Bolt-1, the predicted damage location has some error. But, in case of Bolt-2, the predicted damage location has correctly predicted real damage location. The predicted damage size of Bolt-2 damage has big difference value (0.660) compared with real damage size (0.261). The difference considered that the method for calculating real damage size by using second moment of area ($I$) had error. In the future, we will investigate the study for calculating real damage size of bolt-connection damage.

## 6. Conclusions

In this paper, temperature-compensated damage monitoring in steel girder connections by using wireless acceleration-impedance sensor nodes is experimentally examined. To achieve the objective, the following approaches are implemented. Firstly, wireless acceleration-impedance sensor nodes are described on the design of hardware components to operate. Secondly, the temperature-compensated damage monitoring scheme for detecting bolt loosening, as typical damage type of steel girder connections, is designed by using acceleration-impedance-based SHM methods. The temperature compensation selected the linear regression analysis model to extract linear model between temperature and acceleration-impedance responses. The acceleration-based SHM methods selected the correlation coefficient of power spectral density and frequency-based damage index. These SHM methods performed damage alarming, classification, and damage estimation for detecting bolt loosening. The impedance-based SHM method is selected the correlation coefficient of impedance. This SHM method performed damage alarming and classification for detecting bolt loosening. Finally, the feasibility of temperature-compensated damage monitoring scheme by using wireless acceleration-impedance sensor nodes is experimentally evaluated from damage monitoring in a lab-scaled steel girder with bolted connection joints.

From the experimental results, damage monitoring results for bolt-connection damage show obvious improvement result by temperature compensation. Also, the wireless sensor nodes are applied to improve temperature-compensated damage monitoring scheme by considering

recently new paradigm in SHM. However, the damage estimation results to predict damage size have some error compared with real damage size. By solving this difference, the temperature-compensated damage monitoring scheme will improve to construct SHM system for steel girder connections.

## Acknowledgments

This work was supported by the National Research Foundation of Korea through a grant funded by the Korean Government (Ministry of Education, Science and Technology (NRF-2011-1-D00063)). The student involved in this research was supported by the Brain Korea 21 Program granted by Ministry of Education, Science and Technology of Korea.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 451879, 10 pages
doi:10.1155/2012/451879

*Research Article*

# Field Experiments for Wind Loads on a Low-Rise Building with Adjustable Pitch

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Received 29 June 2012; Accepted 25 August 2012

Academic Editor: Hong-Nan Li

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A wind engineering research field laboratory, which consists of a full-scale low-rise building and two towers, has been constructed by Tongji University near Shanghai Pudong International Airport to study the characteristics of near-ground wind field and wind pressure on low-rise buildings. The full-scale building, whose roof pitch could be adjusted ranging from 0° to 30°, is 10 m in length, 6 m in width and 8 m in eave's height. It is employed to study the wind pressure on the gable roof of low-rise building with different roof pitches. This paper explicitly and concretely discusses the filed facility, data measurement system, data acquisition system, and tap location to provide references for related researchers. Besides, two pieces of time-histories of ten-minute-length wind pressures are analyzed at 0° and 20° roof pitches respectively to compare with those of a wind tunnel test on a rigid model of 1 : 30 scale. The results show that the tendency for the mean and fluctuating wind pressure distributions between the two kinds of tests is nearly similar.

## 1. Introduction

According to the survey of wind hazard, the main reason resulting in casualties and property losses is the damage and collapse of the low-rise buildings in villages and small towns during tropical storm periods [1]. The low-rise buildings are generally not damaged due to structure vibration because of their high stiffness and low height, which is different from tall buildings [2, 3]. The investigation shows that the failure of low-rise buildings usually starts from the local failure of envelope. So it is very important to study how to prevent the failure of envelope for the wind resistance of low-rise buildings.

At present, the available method for this research is based mainly on wind tunnel test, while the field measurement is seldom employed for its high investment of resources and time taking. On the other hand, there is a gap between the results of field measurement and wind tunnel test due to reasons such as the inaccurate simulation in wind tunnel of the Reynolds effect and high turbulence at atmosphere surface layer. Therefore, field measurement has become the most reliable way to master the action mechanism of wind loads and is the most authoritative reference to modify the method of physical and numerical modeling [4].

In the last four decades, there have been a number of notable full-scale studies of wind loads on low-rise buildings, which permit researchers to make appropriate comparisons with wind tunnel measured data and numerical simulation results. In the early 1970s, the BRE (Building Research Establishment) in the UK commenced a program of full-scale measurements on a special constructed experimental building with two stories in Aylesbury, England [5]. In the late 1980s, another famous full-scale experiment on a low-rise building was set up in Lubbock, TX, USA [6, 7]. Almost at the same time, a new full-scale experiment was commenced in Silsoe, UK [8, 9]. It consisted of a steel frame building with variable eave types. At the beginning of twentieth century, a 6 m cube was constructed at the Silsoe Research Institute in an open country exposed position [10]. During the late 2000s, the researchers from Hunan University in China put forward the idea of a removable low-rise building and successfully developed field measurement

Figure 1: Measurement site.

Figure 2: The terrain of test building.

(a) Pitch of 0° (b) Pitch of 30°

Figure 3: Full-scale low-rise test building.

Table 1: Field measurement laboratories of low-rise buildings.

| Name | Site | Size (length × width × height) | Roof pitch | Feature |
|---|---|---|---|---|
| Aylesbury test | UK | 13.3 m × 7 m × 5 m | Variable | Adjustable roof pitch between 5° and 45° |
| TTU experiment | USA | 9.1 m × 13.7 m × 4 m | 2° | Rotatable building |
| Silsoe (steel frame) | UK | 24.3 m × 12.93 m × 4.14 m | 10° | Variable eave styles |
| Silsoe (cube) | New Zealand | 6 m × 6 m × 6 m | 0° | Adjustable cube pitch |
| HNU building | China | 6 m × 4 m × 4 m | 1.1° | Removable building |
| TJU building | China | 10 m × 6 m × 8 m | Variable | Adjustable roof pitch between 0° and 30° |

Table 2: Specifications of the pressure transducer.

| Specifications | CYG1220 | CYG1516 |
|---|---|---|
| Range | 0 ~ ±1 Kpa | 0 ~ ±2.5 Kpa |
| Response frequency | More than 20 Hz | More than 200 Hz |
| Input voltage | DC 24 V | DC 24 V |
| Output signal | 4 ~ 20 mA | 4 ~ 20 mA |
| Rated accuracy | Less than 0.5% FS | Less than 0.5% FS |
| Compensation temperature | 0 ~ 50°C | 0 ~ 50°C |

Figure 4: Elevating system.

Figure 5: Sliding guiding device.

of wind loads on that building during a typhoon [11, 12]. The information about these experiments is summarized in Table 1.

Another field measurement research in China was developed by Tongji University from the year 2008. The building used for field measurement was built at the coast of China near Shanghai Pudong International Airport. The plane size of the building is 10 m × 6 m and the eave height is 8 m. The main feature of this building is that the pitch of roof can be adjusted from 0° to 30° by using the lifting device. The architectural appearance of the pitch-adjustable building was designed according to the typical characteristics of the low-rise buildings in villages in South China.

## 2. Pressure Measurement System

Pudong New District in Shanghai is an area where strong wind, especially strong typhoons, frequently occurs each year. A field laboratory has been set up by State Key Laboratory of Disaster Reduction in Civil Engineering of Tongji University to study the turbulence characteristics of near-ground wind and wind loads on full-scale low-rise buildings. The field laboratory is located in a flat area close to the Yangtze River's estuary and near Shanghai Pudong International Airport (see Figure 1). It consists of a test building and two meteorological towers. The surrounding terrain can be considered as exposure category B according to the Chinese national load code (GB50009-2001) [13] and the exposures around the facility are of inhomogeneous roughness situations (see Figure 2).

*2.1. Test Building.* The steel structure test building has three single-story internal rooms, 3 m, 2.5 m, and 1.5 m in height for each story, respectively. The recording equipment is housed on the middle floor. The test building features an adjustable roof pitch from 0° to 30°. Figure 3 shows the full-scale building with 0° and 30° roof pitches.

Figure 4 shows the elevating system design. A hinge method is used for the connections between the gable roof and the ridge girder. In order to ensure the stability of the roof, a sliding guiding device is designed to control the motion of the side beam, which is shown in Figure 5. Three lifting devices, as shown in Figure 6, are mounted

(a) Active device (b) Passive device

Figure 6: Lifting devices.

(a) CYG 1220 (b) CYG 1516

Figure 7: Photos of two types of pressure sensors.

Figure 8: Pressure measurement system of CYG 1220.

Figure 9: Pressure measurement system of CYG 1516.

Figure 10: Photo of adapter.

on the columns. Of these, the one located on the middle column is the active device; the other two are passive ones. The active device is driven by a servo motor, which is the difference between the active and the passive devices. With the ridge being lifted by these devices, the roof pitch will change.

*2.2. Wind Pressure Measurement and Data Acquisition System.* Two types of pressure transducers are used to measure surface pressures of the test building's roof. They are all developed by Kunshan Shuangqiao Sensor Measurement Controlling Co. Ltd. Of these, 94 microdifferential pressure sensors, CYG1220, are mounted to study the wind pressure without rain, and 20 other diaphragm pressure sensors, CYG1516, are installed under the roof to study the wind-rain-induced effects on pressure. The photos of two kinds of transducers are shown in Figure 7, and specifications of the pressure transducers are given in Table 2.

Figure 8 shows the pressure measurement system of CYG1220. The transducers are mounted on the inside surface

Figure 11: Photo of industrial computer.

of the steel roof. For each transducer, a 20 cm length of tube extends down from the pressure tap on the roof and connects to a tee. A 30 cm length of tube extends from the tee to the active side of the transducer. A 1.5 m long tube extends down from the tee down to the floor and is sealed by a valve, which acts as a dam to collect rainwater. The tubes, extending down from the static side of the transducers, are connected together by tees. The static system extends to a box below ground outdoors through a long tube to obtain ambient atmospheric pressure for reference.

Figure 9 shows the pressure measurement system of CYG1516. The transducers' structures and diaphragms are made of stainless steel and can prevent rainwater from entering inside. The electric line at the end of each transducer is connected to the recording system, and then, diaphragm strain is converted to an analog output voltage proportional to the pressure signal.

An industrial computer with 2G RAM and 2.4 GHz coprocessor is used for data acquisition. Two conversion boards supplied by National Instruments Corporation are used to capture the incoming signals, and each one is an 80-channel and 16-bit A/D board. Cycling scanning has been adopted for the minimum requirement of dynamic responsibility, which is completely satisfied with the data collection for 160 channels. The sampling frequency of each transducer is 20 Hz. Zero calibration of all the transducers is checked before data collection. Figure 10 shows the photo of two adapters together with 160 channels, and Figure 11 shows the industrial computer.

*2.3. Tap Locations.* According to previous field measurement and wind tunnel tests, the extreme negative pressure usually occurs at the windward corner during oblique wind direction for gable roof buildings, and the gradient of pressure is larger than those at other locations. From the wind rose of Shanghai, shown in Figure 12, southeast wind is the prevailing wind direction in Shanghai [14]. Therefore, more transducers are put at the southeast corner. The locations of pressure taps are shown in Figure 13, and the photos of tap locations at the southeast corner roof are shown in Figure 14.

Figure 12: Wind rose of Shanghai.

Figure 13: Layout of pressure taps on the surface of roof (•: CYG 1220°: CYG 1516).

(a) Inside (b) Outside

Figure 14: Photos of tap locations at corner roof.

(a) 10 m tower (b) 40 m tower

Figure 15: Photos of towers.

Figure 16: The arrangements of anemometers (unit: m).

## 3. Wind Velocity Measurement System

*3.1. Field Facility and Instrumentation.* The field laboratory also consists of two self-supporting steel towers. The photos of the towers are shown in Figure 15. One tower is 10 m in height and located about 25 m to the east of the test building. It is used to support two anemometers installed on the top to collect wind velocity data. The wind velocity data can be converted to total reference pressure which is used to calculate the pressure coefficient of each tap on roof surface.

The other tower is 40 m in height and about 35 m far from the building in the north. To study the turbulence characteristics of strong wind near ground, eight anemometers were installed on the tower at the height of 10, 20, 30, and 40 m, respectively. The types of the anemometers are R. M. Young 81000, R. M. Young 85106, and R. M. Young 05305V, and their sampling frequencies are 20, 4, and 20 Hz, respectively. The arrangement of anemometers is displayed in Figure 16 and specifications of different types of anemometer are shown in Table 3.

*3.2. Data Acquisition System.* Another industrial computer type PXI-1031 is used for data acquisition by National Instruments Corporation, and the machine combines a 4-slot PXI backplane with a structural design that has been optimized for maximum usability in a wide range of applications. A DC power from National Instruments is used to provide the power through the DC input connector on the rear panel of the chassis. The PXI-1031 backplane is a 32-bit PCI, so the 64-bit compact PCI cards operate in 32-bit mode in this chassis.

(a) Case of field measurement (FM1)

(b) Case of wind tunnel (WT1)

(c) Case of field measurement (FM2)

(d) Case of wind tunnel (WT2)

Figure 17: Contours of mean wind pressure coefficients.

(a) Case of field measurement (FM1)

(b) Case of wind tunnel (WT1)

(c) Case of field measurement (FM2)

(d) Case of wind tunnel (WT2)

Figure 18: Contours of RMS wind pressure coefficients.

## 4. Recent Field Measurement Results and Comparison with Wind Tunnel Test

*4.1. Preparation for Analysis.* A wind tunnel test rigid model was in 1/30 scale based on the full-scale test building to compare the wind pressure on the roof between full-scale experiment and wind tunnel tests. The test was performed in TJ-2 Boundary Layer Wind Tunnel in Tongji University. The wind tunnel cross-section is 3 m in width, 2.5 m in height, and 15 m in length. Wind field condition corresponding to roughness exposure B in the Chinese code was simulated in the wind tunnel at a length scale of 1/30. Table 4 shows the parameters for cases of field measurement during strong wind of normal climate and the wind tunnel test.

The surface pressure on the body is usually expressed in the form of a nondimensional pressure coefficient [15]. A general time-varying pressure coefficient $C_{p_i}$ is as follows:

$$C_{p_i} = \frac{p_i - p_\infty}{p_0 - p_\infty}, \tag{1}$$

where $p_i$ is the pressure of tap $i$; $p_0$ is a total reference pressure; $p_\infty$ is a static reference pressure. In this paper, the

Table 3: Specifications of anemometers.

| Specifications | | R.M. Young 81000 | R.M. Young 85106 | R.M. Young 05350 V |
|---|---|---|---|---|
| Wind speed | Range | 0 ~ 40 m/s | 0 ~ 70 m/s | 0 ~ 50 m/s |
| | Resolution | 0.01 m/s | 0.1 m/s | — |
| | Accuracy | ±0.05 m/s (0 ~ 30 m/s) | ±0.1 m/s (0 ~ 30 m/s) | 0.2 m/s |
| Wind direction | Range (horizontal) | 0 ~ 360° | 0 ~ 360° | 0 ~ 360° |
| | Range (vertical) | ±60° | 0 | 0 |
| | Resolution | 0.1° | 1° | — |
| | Accuracy | ±2° (1 ~ 30 m/s) | ±2° | ±3° |
| Sampling frequency | | 4–32 Hz (20 Hz used) | 1 Hz | 20 Hz |
| Working temperature | | −50 ~ 50°C | −50 ~ 50°C | −50 ~ 50°C |

Table 4: Cases of field measurement (FM) and wind tunnel (WT) test.

| Parameters | FM1 | FM2 | WT1 | WT2 |
|---|---|---|---|---|
| Test type | Full-scale | Full-scale | Wind tunnel | Wind tunnel |
| Scale proportion | 1 : 1 | 1 : 1 | 1 : 30 | 1 : 30 |
| Roof pitch | 0° | 20° | 0° | 20° |
| Mean wind speed at eave (m/s) | 8.58 | 9.97 | 9.02 | 9.02 |
| Mean wind direction | 41° | 43° | 40° | 45° |
| Turbulence intensity | 0.233 | 0.221 | 0.247 | 0.241 |
| Sampling frequency (Hz) | 20 | 20 | 312.5 | 312.5 |
| Sample time and data length | 10 min | 10 min | 38.4 s | 38.4 s |
| | 12000 | 12000 | 12000 | 12000 |

mean wind speed was measured at the height of the eave of test building.

*4.2. Pressure Distribution on Roof Surface.* For comparison, Figure 17 shows the contour graph of mean wind pressure on the roof based on field measurement and wind tunnel tests, and Figure 18 displays the contour graph of RMS (root mean square) fluctuating wind pressure. The results indicate that the distribution patterns of mean and fluctuating pressure agree well with each other. From either field measurement or wind tunnel test results, it is observed that there are two pairs of conical vortices occurring at the windward of roof as well as the backside of ridge. In these regions, the negative pressure is larger than in other regions. However, there is an evident gap for magnitudes of mean and fluctuated pressure coefficients between field measurement and wind tunnel test results. The discrepancies are probably caused by inadequate simulation of turbulence intensities in atmospheric surface layer, small-scale turbulence content, inaccurate detail structure of wind tunnel model, the stationary state of wind flow, and the Reynolds and Jensen number effects. Further research is needed to interpret the deterministic reason for the discrepancy in the magnitudes.

## 5. Conclusions

Pudong New District in Shanghai is an area that strong wind, especially strong typhoons, frequently occurs each year. Therefore, a field laboratory was constructed by State Key Laboratory of Disaster Reduction in Civil Engineering of Tongji University to study the turbulence characteristics of near-ground wind field and wind effects on full-scale low-rise gable-roof buildings. The field laboratory described in this paper is the first project in China for wind effects based on the measurement on a fixed low-rise building. The full-scale building's roof pitch angle could be adjusted from 0° to 30°, which facilitates the study on the wind effects of low-rise building with different pitches. The details of the project are explicitly and concretely given to provide reference for related researchers. More importantly, results of the field measurement can allow researchers to make a comparison with those from physical and numerical modeling to provide guidance for the improvement of the wind tunnel test as well as computational fluid dynamic (CFD) technology.

## Acknowledgments

This project is jointly supported by the Ministry of Science and Technology of China (Grants no. SLDRCE09-B-06 and SLDRCE08-A-03), and the National Natural Science Foundation (nos. 51178352 and 90715040), which are gratefully acknowledged.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 893739, 11 pages
doi:10.1155/2012/893739

*Research Article*

# Field Measurement of Wind Characteristics of Typhoon Muifa on the Shanghai World Financial Center

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Received 29 June 2012; Accepted 26 August 2012

Academic Editor: Hong-Nan Li

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Field measurement is considered to be the most reliable method to obtain wind characteristics of typhoon, and it is of essential value of further understanding of wind-resistant design of super tall buildings. In this paper, the wind data were recorded atop the Shanghai World Financial Center (SWFC) with a height of 494 m during the whole course of Typhoon Muifa on August 6–8 in 2011. Detailed analysis of the field data is conducted to investigate the characteristics of the typhoon-generated wind including the mean wind speed and direction, turbulence intensities, gust factors, peak factors, turbulence integral length scales, and spectra of fluctuating wind speed. A comparative study between the measured wind characteristics and those previously obtained in the literature is also carried out. It is found that the average values of turbulence intensities, gust factors, and turbulence integral length scales under the condition of Typhoon Muifa are higher than those under monsoon condition atop the SWFC, and Von Karman spectra are suitable to describe of the turbulent kinetic energy distribution at Lujiazui of the Pudong district in Shanghai.

## 1. Introduction

As a destructive disastrous weather system, typhoon has been threatening China for thousands of years, especially in southern China. Typhoons that have landed in China account for about 35% of the total number that developed in Northwest Pacific area. Based on the statistical results on available typhoon records since 1949 by Shanghai Meteorological Administration, it is indicated that the intensity of tropical cyclones landing in China follows an upward trend, where the proportion of strong typhoon increases every year as well. Thus, it is of essential value to investigate wind characteristics for further understanding of wind-resistant design of tall buildings [1].

Although many advances in wind tunnel tests and numerical simulation techniques have been recognized, there are still many critical phenomena which can only be investigated by full-scale experiments. Field measurement is considered to be the most reliable method of evaluating wind characteristics of tall buildings, and it will be the fundamental approach for a long term. Some countries have developed local database of wind characteristics, such as Frøya database in Norway [2], offshore observation databases in Canada and the United Kingdom, Sparks database in the USA, and the extensive results observed by Kato et al. [3] in Japan. In Hong Kong and mainland China, several measurement results of wind effects have been contributed by Li et al. [4–6], Xu and Zhan [7], Gu et al. [8, 9], and Li et al. [10, 11]. However, field study of wind effects on super tall buildings under typhoon conditions is still far from sufficient.

At present, the lack of field data is an obstructive factor in the development of research of the wind characteristics. Some field measurements of typhoon effects on skyscrapers have been carried out in Southern China, but few were conducted in Shanghai due to the fact that typhoons rarely move to such high latitude. Thus, field measurements of typhoon-generated wind data in Shanghai are of great value for the understanding of wind characteristics in eastern China.

The wind data were recorded atop the 494 m high Shanghai World Financial Center during the whole course of Typhoon Muifa on August 6–8 in 2011. Detailed analysis of the field data was conducted to investigate the characteristics of the typhoon-generated wind including the turbulence

Figure 1: Storm path of Typhoon Muifa (from Wikipedia).

Figure 2: Overview of the SWFC.

intensities, gust factors, peak factors, turbulence integral length scales, and spectra of fluctuating wind speed.

## 2. Typhoon Muifa

Typhoon Muifa was a large, strong, and persistent typhoon. It was the ninth named storm of the 2011 Pacific typhoon season. Early on July 28, the tropical storm, named Muifa, entered the Philippine. The storm gradually drifted north over the next day while maintaining tropical storm strength. At 08:00 on July 30, Muifa was upgraded into a Severe Tropical Storm; the storm strengthened rapidly and was upgraded into a typhoon at 14:00 and a strong typhoon at 20:00. Muifa had strengthened to a super typhoon in the next day when the sustained wind speed reached 55 m/s and weakened to strong typhoon later in the day. The storm gradually moved north and was finally downgraded into a severe tropical storm on August 7. After weakening to a tropical storm, Muifa made landfall at North Korea on August 8. Early on August 9, Muifa weakened to a tropical depression in northeast China and became a low-pressure area later. During the passage of Typhoon Muifa, the minimum horizontal distance between the observation site and the typhoon center was 260 km. Figure 1 shows the storm path of Typhoon Muifa.

## 3. Overview of the Measurement

The Shanghai World Financial Center (SWFC), located in the Pudong district of Shanghai, is the tallest skyscraper in mainland China, whose structure height is 492 m, as shown in Figure 2. The building is surrounded by a large number of tall and super tall buildings, making the wind characteristics of atmospheric surface layer of the building extremely complex.

According to a preliminary CFD simulation of the wind field around the SWFC, the wind data are interfered by the parapet walls and the window clean machines assembled atop the SWFC. To obtain as many available data as possible, two spots for wind speed observation were located at two corners atop the building, namely, northeast side and southwest side, as shown in Figure 3. Two ultrasonic anemometers (Figure 4) were installed at each spot at a height of 494 m from the ground level. The horizontal distance between the ultrasonic anemometers is 71.6 m. In this study, $x$-axis, $y$-axis, and $z$-axis are defined as the north, west, and upward directions of the ultrasonic anemometer respectively, as illustrated in Figure 5. By this way, the wind direction increases with anticlockwise direction from the top view. For instance, the wind direction from the south is 0 degree, and from the east is 90 degrees. The measurement range of the ultrasonic anemometer is from 0.01 to 65 m/s, with the maximum dynamic response frequency of 40 HZ and the actual sampling frequency of 10 HZ. The real-time storage of the acquired data is achieved by CR1000 data acquisition system program produced by Campbell Scientific, Inc. Among the acquired data, those within a wind direction range of 45 degrees at each side atop the SWFC are considered as the available data, based on the CFD simulation.

## 4. Approach to Analysis of Wind Data

*4.1. Mean Wind and Direction.* The wind speed of $x$-axis, $y$-axis, and $z$-axis, denoted by $u_x(t)$, $u_y(t)$, and $u_z(t)$, respectively, can be measured by ultrasonic anemometers simultaneously. Since $u_x(t)$, $u_y(t)$, and $u_z(t)$ are time series of $x$-axis, $y$-axis, and $z$-axis, the longitudinal, lateral, and vertical components of wind speed can be obtained by means of vector decomposition method.

Making 10 min the averaging time interval, the mean horizontal wind speed $U$ and the mean horizontal wind direction $\varphi$ are

$$U = \sqrt{\overline{u_x(t)}^2 + \overline{u_y(t)}^2},$$
$$\cos(\phi) = \frac{\overline{u_x(t)}}{U}. \quad (1)$$

Figure 3: Sketch of the location of the ultrasonic anemometers.

Figure 4: WindMaster Pro ultrasonic anemometers.

Figure 5: Definition of $x$, $y$, and $z$ axes of the anemometer.

The direction of vertical wind speed is the same as that of $z$-axis; thus, the mean vertical wind speed $W$ is

$$W = \overline{u_z(t)}. \tag{2}$$

The longitudinal, lateral, and vertical components of fluctuating wind speed within a basic time interval, denoted by $u_x(t)$, $u_y(t)$ and $u_z(t)$, can be obtained as follows:

$$\begin{aligned} u(t) &= u_x(t)\cos\phi + u_y(t)\sin\phi - U, \\ v(t) &= -u_x(t)\sin\phi + u_y(t)\cos\phi, \\ w(t) &= u_z(t) - W. \end{aligned} \tag{3}$$

*4.2. Turbulence Intensity, Gust Factor, and Peak Factor.* The turbulence intensity ($I$) indicates the variation of wind speed as well as the relative intensity of the fluctuating wind. It is defined as the ratio of standard deviation of fluctuating wind speed within 10 min to the mean horizontal wind speed $U$, expressed as

$$I_i = \frac{\sigma_i}{U} \quad (i = u, v, w), \tag{4}$$

where $\sigma_i$ ($i = u, v, w$) is the standard deviation of fluctuating wind speeds $u(t)$, $v(t)$, and $w(t)$ within the analyzed time interval, respectively.

The gust factors demonstrate proportions of gust wind speed to mean wind speed. It is generally defined as the ratio of maximum mean wind speed during gust duration $t_g$, which is usually made as 3 s in structural wind engineering, to mean horizontal wind speed $U$ in 10 min. $G_u$ can be obtained by

$$\begin{aligned} G_u(t_g) &= 1 + \frac{\max\left(\overline{u(t_g)}\right)}{U}, \qquad G_v(t_g) = \frac{\max\left(\overline{v(t_g)}\right)}{U}, \\ G_w(t_g) &= \frac{\max\left(\overline{w(t_g)}\right)}{U}, \end{aligned} \tag{5}$$

where $\max(\overline{u(t_g)})$, $\max(\overline{v(t_g)})$, and $\max(\overline{w(t_g)})$ are the maximum mean wind speeds of longitudinal, lateral, and vertical directions during gust duration $t_g$, respectively.

Similar to the gust factor, the peak factor also features the instantaneous intensity of the fluctuating wind, which can be calculated from

$$g_u = \frac{\hat{U}_{t_g} - U}{\sigma_u}, \tag{6}$$

where $\hat{U}_{t_g}$ is the maximum mean wind speed within time interval $t_g$, and $\sigma_u$ is the standard deviation of $u(t)$.

*4.3. Turbulence Integral Length Scale.* The turbulence integral length scale ($L^x$) is one of the important parameters for describing the turbulent characteristics of wind flows, and it is generally interpreted as the length of the dominating

vortex scale. It varies from different analysis methods. Of the methods developed by numerous researchers, two effective approaches are usually adopted to calculate the turbulence scale: the first one is autocorrelation function integral method based on Taylor hypothesis [12], expressed by (7); the other one is Von Karman velocity spectrum method used to fit the measured velocity spectrum as the fitting parameter, expressed in (8). The fluctuating wind speed in the former method is assumed to be stable, while in the latter the velocity spectrum must fit Von Karman velocity spectrum. In this paper, the turbulence scale is calculated using the former method

$$L_i^x = \frac{U}{\sigma_i^2}\int_0^\infty R_i(\tau)d\tau \quad (i = u, v, w), \tag{7}$$

$$L_i^x = \frac{US_i(0)}{4\sigma_i^2} \quad (i = u, v, w). \tag{8}$$

In (7) and (8), $R_i(\tau)\,(i = u, v, w)$ is the autocorrelation function of fluctuating wind; $S_i(0)\,(i = u, v, w)$ is the fluctuating wind spectrum value when $f = 0$; $\sigma_i^2\,(i = u, v, w)$ is the variance of fluctuating wind speed.

*4.4. Power Spectral Density of Fluctuating Wind Speed.* The power spectral density of fluctuating wind speed describes the contribution of the kinetic energy of the vortices in different sizes to the whole turbulent kinetic energy. The values of spectrum in the frequency domain represent the distributive ratios of the kinetic energy of different vortices to the turbulent kinetic energy. Davenport Spectrum, Von Karman Spectrum, Simiu Spectrum, Kaimal Spectrum, and Harris Spectrum are some typical fitting power spectra, among which the Von Karman form has been widely recognized as a suitable representation of velocity spectra [13]. Equation (9) is the expressions of normalized Von Karman spectra [14]:

$$\begin{aligned} \frac{nS_\varepsilon(n)}{\sigma_\varepsilon{}^2} &= \frac{4nL_\varepsilon^x/U}{\left[1 + 70.8\left(nL_\varepsilon^x/U\right)^2\right]^{5/6}} \quad (\varepsilon = u), \\ \frac{nS_\varepsilon(n)}{\sigma_\varepsilon{}^2} &= \frac{4nL_\varepsilon^x/U\left[1 + 755.2\left(nL_\varepsilon^x/U\right)^2\right]}{\left[1 + 283\left(nL_\varepsilon^x/U\right)^2\right]^{11/6}} \quad (\varepsilon = v, w), \end{aligned} \tag{9}$$

where $n$ is the frequency; $L_\varepsilon\,(\varepsilon = u, v, w)$ is the turbulence integral length scales in the longitudinal, lateral, and vertical directions, respectively; $S_\varepsilon(n)$ $(\varepsilon = u, v, w)$ is the power spectral density of the fluctuating wind components in the longitudinal, lateral, and vertical direction, respectively.

## 5. Characteristics of Typhoon Muifa

*5.1. Mean Wind Speed and Direction.* Based on the preanalysis of the whole time histories of wind speed measured at both sides atop the SWFC when Typhoon Muifa passed Shanghai, a 42-hour time history measured at the northeast side, from 0:00 on August 6 to 18:00 on August 7, is chosen as research sample. Figure 6 shows the variations of mean wind speed and mean wind direction in each 10-min period at both sides atop the SWFC during the passage of Muifa. According to the Chinese Load Code for the Design of Building Structures [15], the research object is divided into 252 consecutive samples with an averaging time interval of 10 minutes. The maximum 10-min mean wind speed reaches 32.97 m/s, which illustrates that the strength of Muifa is intense, and can also be attributed to that the building site is 260 km far from the center of the typhoon. Figure 7 shows the signal time histories of the research sample along three axes of ultrasonic anemometer. In addition, with respect to the samples, variations of the mean wind speed and mean wind direction in each 10 min period as well as the time histories of longitudinal, lateral, and vertical fluctuating components are shown in Figures 8 and 9.

*5.2. Turbulence Intensity.* As an important parameter to describe the characteristics of fluctuating wind speed and determine design wind loads on structures, it is of significant value to investigate the turbulence intensity under typhoon condition. Before calculating turbulence intensities, unwanted data among those 252 samples caused by unfavorable effect of parapet walls and window clean machines on the anemometers are filtered out by means of the effective wind angle which is determined by CFD simulation of the flow field. Variations of the longitudinal, lateral, and vertical turbulence intensities $I_u$, $I_v$, and $I_w$ with 10 min mean wind speed are illustrated in Figure 10. The minimum, maximum, and average values are 0.05, 0.29, and 0.14 for $I_u$; 0.04, 0.26, and 0.13 for $I_v$; 0.04, 0.17, and 0.09 for $I_w$, respectively. It is observed that all the turbulence intensity components $I_u$, $I_v$, and $I_w$ remain almost constant with 10 min mean wind speed, among which, $I_u$ exhibits the most scattered character.

Comparison results of the ratios between the three average values of the turbulence intensities in the longitudinal, lateral, and vertical direction in this study with available field measurement results in the literature are listed in Table 1. Li et al. [4, 11] conducted field measurement atop the Jin Mao Building with a height of 420.5 m during the passage of Typhoon Rananim, and the average values of the longitudinal and lateral turbulence intensities with a time interval of 10 min were 0.562 and 0.437, respectively, while those measured in Hong Kong during the passage of Typhoon Sally were 0.216 and 0.206, respectively [5], all of which are higher than the results in this study. Xu and Zhan [7] found that the mean longitudinal turbulence intensity in 1 hr period atop the Di Wang Tower during the passage of Typhoon York was 0.117, which was slightly lower than the lateral one. Gu et al. [9] analyzed the wind data recorded atop the SWFC under monsoon condition, and the average values of longitudinal and lateral turbulence intensities were 0.085 and 0.075, which, compared with results in this study, shows that the turbulence intensities under typhoon condition are higher than those under monsoon condition at Lujiazui of the Pudong district in

Table 1: Comparison of the ratios between the three average values of the turbulence intensities in the longitudinal, lateral, and vertical directions.

| Reference | Type of wind field | Observation height (m) | $I_u : I_v : I_w$ ($I_u : I_v$) | Location |
|---|---|---|---|---|
| Solari and Piccardo [14] | / | / | 1 : 0.75 : 0.55 | / |
| Tieleman [16] | Monsoon | 5 | 1 : 0.80 : 0.39 | The Netherlands |
| Shiau and Chen [17] | Typhoon Babs | 26 | 1 : 0.78 : 0.50 | Taiwan (China) |
| Li et al. [5] | Typhoon Sally | 325 | 1 : 0.95 | Shenzhen (China) |
| | | 374 | 1 : 0.27 | Hong Kong (China) |
| Li et al. [4] | Typhoon Rananim | 420 | 1 : 0.78 | Shanghai (China) |
| Gu et al. [9] | Monsoon | 494 | 1 : 0.88 | Shanghai (China) |
| Cao et al. [1] | Typhoon Maemi | 10 | 1 : 0.83 : 0.56 | Japan |
| Pang and Ling [18] | Typhoon Jelawat | 20 | 1 : 0.54 : 0.23 | Shanghai (China) |
| | Typhoon Prapiroon | | 1 : 0.91 : 0.46 | |
| Wang et al. [19] | Typhoon Muifa | 10 | 1 : 0.67 : 0.50 | Shanghai (China) |
| | | 20 | 1 : 0.65 : 0.55 | |
| | | 40 | 1 : 0.73 : 0.47 | |
| This paper | Typhoon Muifa | 494 | 1 : 0.93 : 0.64 | Shanghai (China) |

Figure 6: Variation of 10 min mean wind speeds and mean wind directions (top) Shanghai World Financial Center.

Figure 7: Ultrasonic anemometer signal histories along three axes of the samples.

Figure 8: Variation of mean wind speed and mean wind direction.

Shanghai. This can be attributed to the differences of characteristics between typhoon and monsoon. Furthermore, of the same height, 0.115 is recommended by the Architectural Institute of Japan (AIJ) Recommendation for Loads on

Figure 9: Time histories fluctuating wind in the longitudinal, lateral, and vertical directions.

Figure 10: Variation of turbulence intensities with 10 min mean wind speed in three directions.

Figure 11: Variation of gust factors with 10 min mean wind speed in three directions.

Buildings [21] for longitudinal turbulence intensity under the similar type of site. Comparing the recommended value with the observed one in this paper, it is found that the former is slightly smaller than the latter, suggesting that the longitudinal turbulence intensity recommended by the Recommendation for Loads on Buildings [21] is no longer applicable under typhoon condition at a height of 494 m. Although there is no explicit equations for turbulence intensities in the Chinese Load Code for the Design of Building Structures [15], yet with respect to a fluctuating coefficient in the code and the definition of turbulence intensity, an equivalent value of 0.036 can be calculated, which is much lower than the measured value in this study and the recommended value in the AIJ-2004. Thus, the Chinese code is obviously conservative for turbulence intensities.

*5.3. Gust Factor.* Similar to turbulence intensity, the gust factor is also an important parameter to describe the characteristics of fluctuating wind and to determine wind-induced dynamic response of structures [4, 11]. By analyzing the effective samples, the longitudinal, lateral, and vertical gust factors with a gust duration of 3 s in 10 min period are obtained. Figure 11 presents the variation of longitudinal, lateral, and vertical gust factors $G_u$, $G_v$, and $G_w$ with 10 min mean wind speed. The minimum, maximum, and average values are 1.11, 1.64, and 1.28 for $G_u$; 0.10, 0.73, and 0.31 for $G_v$; and 0.10, 0.57, and 0.22 for $G_w$, respectively. It is shown that the gust factors in the three directions, $G_u$, $G_v$, and $G_w$ remain almost constant with mean wind speed. Gu et al. [9] found that the average values of longitudinal and lateral gust factors were 1.15 and 0.17 atop the SWFC under monsoon condition, which were slightly lower than those in this study, and there is a tendency for the gust factors to decrease with the increase of mean wind speed. This can be attributed to the same reason as discussed previously for the turbulence intensity. During the passage of Typhoon Muifa, a field study carried out by Wang et al. [19] at elevation of 10 m on an observation tower located in the Pudong district shows that the average values of longitudinal, lateral, and vertical gust factors are 1.48, 0.30, and 0.29 at 10 m, respectively, among which the longitudinal, and vertical gust factors are higher than those in the present study.

*5.4. Peak Factor.* The peak factor with a gust duration of 3 s in 10 min period is analyzed. Figure 12 shows the variation of the peak factor with 10 min mean wind speed. It can be seen that all the points lie approximately on a horizontal line,

Table 2: Comparison of the ratios between the three average values of the turbulence integral length scale in the longitudinal, lateral, and vertical directions.

| Reference | Type of wind field | Observation height (m) | $L_u : L_v : L_w$ |
|---|---|---|---|
| Kato et al. [3] | Typhoon | 55.7 | 1 : 0.33 : 0.17 |
| | | 86.0 | 1 : 0.50 : 0.17 |
| Gu et al. [8] | Monsoon | 494 | 1 : 0.90 : 0.42 |
| Hui et al. [20] | Monsoon | 50 | SW 1 : 0.36 : 0.098 |
| Wang et al. [19] | | | NE 1 : 0.46 : 0.19 |
| | Typhoon | 10 | 1 : 0.54 : 0.07 |
| | | 20 | 1 : 0.65 : 0.11 |
| | | 40 | 1 : 0.73 : 0.1 |
| This paper | Typhoon | 494 | 1 : 0.54 : 0.42 |

Figure 12: Variation of peak factor with 10 min mean wind speed.

and the minimum, maximum, and average values of peak factors are 1.37, 3.13, and 2.11, respectively. The average peak factors derived from the field study by Wang et al. [19] at elevation of 10 m, 20 m, and 40 m on an observation tower in Pudong district during the passage of Typhoon Muifa are 3.18, 3.16, and 3.05, respectively, which are slightly higher than the results in the present study.

*5.5. Turbulence Integral Length Scale.* Values for the longitudinal, lateral, and vertical turbulence integral length scales of the effective samples are obtained from integration of the autocorrelation function. Figures 13(a), 13(b), and 13(c) show the variation of the longitudinal, lateral, and vertical turbulence integral length scales with the 10 min mean wind speed. It is observed that the turbulence integral length scales in the longitudinal direction scatter with the increase of 10 min mean wind speed and no explicit trend can be found between them. While the turbulence integral length scales in the other two directions remain almost constant with the increase of 10 min mean wind speed. The minimum, maximum, and average values are 37.24 m, 706.77 m, and 217.60 m for $L_u$; 39.16 m, 459.14 m, and 117.98 m for $L_v$; and 18.74 m, 488.24 m, and 54.17 m for $L_w$, respectively. In addition, it is noticed that the average values of $L_u, L_v$ and $L_w$ measured atop the SWFC under monsoon condition were 127.98 m, 114.85 m, and 54.17 m [8], respectively; those obtained by Wang et al. [19] at elevation of 40 m on an observation tower in Pudong district during the passage of Typhoon Muifa were 150.31 m, 110.24 m, and 14.94 m respectively. All of them in the corresponding directions are lower than the measured values in the present study. On the other hand, as specified in the Recommendation for Loads on Buildings [21], the longitudinal turbulence integral length scale can be estimated by the following empirical equation:

$$L_u = \begin{cases} 100\left(\dfrac{Z}{30}\right)^{0.5}, & 30 < Z \le Z_G, \\ 100, & Z \le 30\,\mathrm{m}, \end{cases} \tag{10}$$

where $Z$ is the observation height; $Z_G$ is the height of the gradient wind. And the recommended value atop the SWFC determined by (10) is 405.79 m, which is much higher than the average observed value in the present study (217.60 m), suggesting that (7) employed in the Recommendation for Loads on Buildings [21] does not suit the estimation of the longitudinal turbulence integral length scale atop the SWFC during the passage of Typhoon Muifa.

Comparison results of the ratios between the three average values of the turbulence intensities in the longitudinal, lateral, and vertical directions in this study with available field measurement results in the literature are listed in Table 2. It is noteworthy that the ratio of $L_u : L_v$ in this study is consistent with the one from Wang et al. [19] at elevation of 10 m during the passage of Typhoon Muifa. Meanwhile, the ratio of $L_u : L_w$ in this study is obviously higher than those from Wang et al. [19]. The main reason for this discrepancy can be attributed to that the present field measurements of wind data atop the SWFC were conducted at a height of 494 m above ground level and the results provided in Wang et al. [19] were obtained near ground. Moreover, the ratio of $L_u : L_w$ in this study is in good agreement with the one from Gu et al. [8] atop the SWFC under monsoon condition, while the ratio of $L_u : L_v$ in this study is noticeably lower than the one from Gu et al. [8], which may be attributed to the differences between the turbulence structures of

Figure 13: Variation of turbulence integral length scales with 10 min mean wind speed in three directions: (a) longitudinal turbulence integral length scale, (b) lateral turbulence integral length scale, and (c) vertical turbulence integral length scale.

typhoon and monsoon. In view of this, the factors that may influence turbulence integral length scales lie in the type of atmospheric flow, height of observation, terrain conditions and wind speed, and so forth.

*5.6. Power Spectral Density of Fluctuating Wind Speed.* The power spectral density can significantly affect the wind-induced dynamic responses of tall buildings and it is, thus, worthwhile to identify the power spectral density functions based on the field measured wind speeds [5]. The normalized autopower spectral density functions of the fluctuating wind speed in the longitudinal, lateral, and vertical directions are obtained from the wind data measured atop the SWFC during the passage of Typhoon Muifa, as shown in Figures 14(a), 14(b), and 14(c). It is observed that the shapes of the spectra obtained from three different samples disagree to some extent. Within low-frequency range, the spectra of the different samples fit fairly well, while within inertial subrange, the value of the spectrum increases with the increasing 10 min mean wind speed. Thus, it is indicated that turbulent kinetic energy distributes mainly in the low-frequency region, and the dissipation rate of turbulent kinetic energy decreases with the increasing 10 min mean wind speed within inertial subrange.

The measured spectra in the longitudinal, lateral, and vertical directions and the corresponding Von Karman spectra using the measured-turbulence integral length scales are presented in Figures 15(a), 15(b), and 15(c) for comparison purposes. It can be observed that the measured spectral density functions of fluctuating wind speed in the longitudinal, lateral, and vertical directions fit fairly well with the Von Karman spectra using the measured turbulence integral length scales, indicating that the Von Karman spectra are suitable for the description of the turbulent kinetic energy distribution at Lujiazui of the Pudong district in Shanghai.

## 6. Conclusions

The wind data were recorded atop the 494 m high Shanghai World Financial Center during the whole course of Typhoon Muifa on August 6–8 in 2011. Detailed analysis of the

Figure 14: The normalized autopower spectral density functions of the fluctuating wind speed: (a) longitudinal spectra, (b) lateral spectra, and (c) vertical spectra.

field data was conducted to investigate the characteristics of the typhoon-generated wind including the turbulence intensities, gust factors, peak factors, turbulence integral length scales, and spectra of fluctuating wind speed. Some important conclusions are listed as follows.

(1) All the turbulence intensity components $I_u, I_v$, and $I_w$ remain almost constant with 10 min mean wind speed, among which, $I_u$ exhibits the most scattered character. The average values of $I_u, I_v$ and $I_w$ are 0.14, 0.13, and 0.09, respectively, which are higher than those obtained by Gu et al. [9] at the same site under monsoon condition. The longitudinal turbulence intensities recommended by the Recommendation for Loads on Buildings [21] and the Chinese Load Code for the Design of Building Structures [15] are no longer applicable for engineering purpose.

(2) The gust factors in the three directions, $G_u$, $G_v$, and $G_w$ remain almost constant with mean wind speed, and the average values of $G_u$, $G_v$, and $G_w$ are 1.28, 0.31, and 0.22, respectively, which are higher than those obtained by Gu et al. [9] at the same site under monsoon condition. The average values of longitudinal and vertical gust factors obtained by Wang et al. [19] at elevation of 10 m on an

Figure 15: Comparison between the measured spectra and Von Karman spectra: (a) longitudinal spectrum, (b) lateral spectrum, and (c) vertical spectrum.

observation tower located in the Pudong district during the passage of Typhoon Muifa are higher than those in this study.

(3) All the peak factors lie approximately on a horizontal line with the increasing 10 min mean wind speed, and the minimum, maximum, and averages value of peak factors are 1.37, 3.13, and 2.11, respectively. The average gust factors obtained by Wang et al. [19] at elevation of 10 m, 20 m, and 40 m are slightly higher than those in this study.

(4) The turbulence integral length scales in the longitudinal direction scatter with the increase of 10 min mean wind speed and those in the other two directions remain almost constant with the increase of 10 min mean wind speed. The average values of $L_u$, $L_v$, and $L_w$ are 217.60 m, 117.98 m, and 54.17 m, respectively, all of which are higher than those obtained by Gu et al. [8] and Wang et al. [19]. Equation (7) employed in the Recommendation for Loads on Buildings [21] does not suit the estimation of the longitudinal turbulence integral length scale atop the SWFC during the passage of Typhoon Muifa.

(5) The shapes of the spectra obtained from three different samples disagree to some extent. Within low-frequency range, the spectra of the different samples fit fairly well, while within inertial subrange, the values of the spectra increase with the increasing 10 min mean wind speed. The turbulent kinetic energy distributes mainly in the low-frequency region, and the dissipation rate of turbulent kinetic

energy decreases with the increasing 10 min mean wind speed within inertial subrange. The measured spectra fit fairly well with the Von Karman spectra using the measured turbulence integral length scales, indicating that the Von Karman spectra are suitable for the description of the turbulent kinetic energy distribution at Lujiazui of the Pudong district in Shanghai.

## Acknowledgment

The work presented in this study is supported by the National Natural Science Foundation (90715040), which is gratefully acknowledged.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 685054, 13 pages
doi:10.1155/2012/685054

*Research Article*

# Distributed Strain Sensor Networks for In-Construction Monitoring and Safety Evaluation of a High-Rise Building

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Received 14 July 2012; Accepted 9 August 2012

Academic Editor: Ting-Hua YI

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The New Headquarters of Shenzhen Stock Exchange (NHSSE), currently being constructed in Shenzhen, China, is a high-rise building with a height of 245 m. One salient feature of NHSSE is its huge floating platform with an overall plan dimension of 98 m× 162 m and a total height of 24 m, making it one of the largest cantilever structures in the world. In recognition of the uniqueness of the floating platform, a long-term structural health monitoring (SHM) system has been designed and implemented by The Hong Kong Polytechnic University for both in-construction and in-service monitoring of NHSSE. As part of this monitoring system, 224 vibrating-wire strain gauges have been permanently installed to measure the strain responses of key structural components of the floating platform. A wireless strain monitoring system by integrating local tethered data acquisition and long-range wireless data transmission has been developed for real-time strain monitoring and visualization. This paper presents the stress evolution of the floating platform during dismantlement of temporary supports on the basis of the real-time monitoring data and the statistical stress analysis and safety evaluation of the floating platform by use of the long-term monitoring data, as well as the effect of the welding residual stress on structural safety of NHSSE.

## 1. Introduction

Structural health monitoring (SHM) of building structures, especially for super-tall buildings and out-of-codes buildings, has been a cutting-edge technology and gained more and more attention in civil engineering community over the past decade [1–5]. With the increasing complexity of modern building structures, civil engineers have met a challenging task of ensuring not only the life-cycle safety of these building structures but also their robustness in resisting natural and/or man-made hazards such as earthquakes, typhoons, and fires, and so forth at both in-construction and in-service stages. Instrumented high-rise buildings with long-term SHM systems are able to provide the most authentic and practical information for real-time assessment of structural integrity, durability, and reliability as well as for decision-making on structural inspection and maintenance actions. A recent application of the SHM technology to high-rise buildings is the Canton Tower permanently instrumented with more than 700 sensors of sixteen types which were installed in parallel with the construction progress leading to a life-cycle monitoring architecture [6–9].

In the applications of SHM to building structures, static and/or dynamic strain monitoring of key structural components is essential since strain measurement data after being transferred into the corresponding stress data can be directly used to indicate the utilization extent of the resistance of materials as well as the safety margin or reliability of the structural components. The physical quantities of acceleration, moment, and torque also can be derived through careful design and implementation of densely distributed strain sensor networks on the targeted building structure, and further for data-driven health and safety condition evaluation. Additionally, several investigations demonstrate that strain measurement as a local structural response shall be more suitable for local behavior characterization or damage detection than vibration-based acceleration data. Real-world practice of strain monitoring of large-scale structures by use of electrical strain gauges, vibrating-wire strain gauges, or fiber optical strain sensors has been widely reported [10–12], and research efforts devoted to strain-based condition assessment of instrumented structures have been made by different investigators [13–19].

(a) (b)

Figure 1: New Headquarters of Shenzhen Stock Exchange (NHSSE).

The New Headquarters of Shenzhen Stock Exchange (NHSSE), which is planned as a financial center in Shenzhen, China, is a high-rise building with a height of 245 m. Its surrounding area is designed as a public space for festivals and gatherings, while the main tower will serve for various functions including trading floor, Shenzhen Stock Exchange offices, and so forth. A salient feature of this building exists in its huge floating platform, which is one of the largest cantilever structures in the world. In view of this uniqueness and the importance of NHSSE, a long-term SHM system has been devised and implemented by The Hong Kong Polytechnic University to continuously monitor the structural responses (strain, temperature, displacement, and acceleration) of the floating platform during its whole life cycle, including both construction and service stages [20]. This paper explores several key issues in developing and implementing distributed strain sensor networks on the floating platform commencing from its construction stage for long-term strain monitoring in real time and presents the statistical stress analysis and safety evaluation of the floating platform using the long-term strain monitoring data.

## 2. NHSSE and Its SHM System

*2.1. New Headquarters of Shenzhen Stock Exchange (NHSSE).* As illustrated in Figure 1, the New Headquarters of Shenzhen Stock Exchange (NHSSE), with 46 storeys above the ground and 3 basement storeys, is composed of 4 segments in the vertical direction of the building. The first segment is the reinforced concrete basement storeys. The second segment, with a rectangular cross-section of 54 m × 90 m, is a trussed tube structure from the 1st floor to the 6th floor of the main tower. The floating platform, from the 7th floor to the 10th floor of the building, constitutes the third segment with a rectangular plan of 98 m × 162 m. From the 10th floor of the main tower above, the fourth segment of the building is a tube-in-tube structure consisting of an inner reinforced concrete core and an outer shaped steel and reinforced concrete composite frame. The inner structure has a rectangular cross-section of 28 m × 32 m, and the outer structure has a 54 m × 54 m square cross-section.

As the most distinct feature of NHSSE, the huge floating platform is a mega-overhanging steel truss structure with a total height of 24 m. The floating platform consists of 14 crisscrossed steel trusses which are divided into 6 types. The main truss structures are assembled by lots of welded composite joints made of box-sectional members. The outrigger truss storeys have the outrigger components of 22 m in the south-north direction and 36 m in the east-west direction at a height of 36 m above the ground, which makes the floating platform one of the largest cantilever structures in the world.

*2.2. SHM System for NHSSE.* In recognizing the uniqueness of the floating platform, a long-term SHM system was designed and implemented by The Hong Kong Polytechnic University to monitor the structural responses (strain, temperature, displacement, and acceleration) of the cantilever trusses of NHSSE in both construction and service stages. The SHM system for NHSSE consists of six modules: Module 1—Sensory System (SS), Module 2—Data Acquisition and Transmission System (DATS), Module 3—Data Processing and Control System (DPCS), Module 4—Data Management System (DMS), Module 5—Structural Health Evaluation System (SHES), and Module 6—Inspection and Maintenance System (IMS). The integration of these six modules is shown in Figure 2. The SS and DATS are located in the structure, the DPCS, DMS, and SHES are inside the monitoring center room, and the IMS is a portable system.

Figure 2: Modular architecture of SHM system for NHSSE.

The SS system includes vibrating-wire strain gauges, temperature sensors, accelerometers, and a vision-based displacement tracking system. In this SHM system, the vibrating-wire strain gauge was adopted for strain measurement due to its advantage of antielectromagnetic interference in comparison with the electrical strain gauge, and its merit of cost-effectiveness compared with the fiber optical strain sensor. The Geokon Model 4000 vibrating-wire strain gauge was used to measure the strain and temperature simultaneously. With the measured temperature data, the strain data could be calibrated by taking into account the temperature effect on the resonant frequency of the tensioned steel wire of the vibrating-wire strain gauge. The DATS system is composed of 4 stand-alone data acquisition units (DAUs) and 2 wireless LAN bridges for data transmission in construction stage. The 4 DAUs are assigned at the 4 weak electricity wells which are located in the four corners of the core structure. They are used to collect the signals from the surrounding sensors, digitize the analogue signals, and wirelessly transmit the data into the central room. The DPCS comprises high-performance servers and data-processing software. This system is used for routine data processing, structural and system status monitoring and prewarning as well as display of the data. The DMS consists of a high-performance server, relational Oracle-based database system and geographic information system- (GIS-) based data management software. The SHES is composed of a high-performance server and structural health evaluation software. It is used for processing and analyzing the monitoring data and evaluating and diagnosing the structural condition. It emphasizes on control-oriented evaluation of structural status in construction stage and updating of the finite element model (FEM) for the purpose of structural damage detection. The IMS is a laptop-computer-aided portable system for inspecting and maintaining sensors, DAUs, and cabling networks.

## 3. Distributed Strain Sensor Networks

*3.1. Deployment Sections of Strain Gauges.* The structural strains of the cantilever trusses of NHSSE are of the utmost concern at the construction stage, especially during temporary supports' dismantlement of the floating platform. After examining the load transfer mechanism and load-carrying characteristics of the cantilever trusses of NHSSE, 56 critical cross-sections on key structural members of the cantilever trusses have been selected for strain monitoring and further stress analysis. All the instrumented cross sections are allocated on the upper chords, diagonal struts, and bottom chords of four main steel trusses (TR1, TR2, TR4, and TR5) and the internal diagonal bracings on the 7th floor, which are bisymmetrically distributed on the floating platform, as illustrated in Figure 3. The deployment sections of strain gauges are divided into four monitoring zones, that is, southeast zone, southwest zone, northeast zone, and northwest zone. Each monitoring zone contains 14 instrumented sections and each instrumented section has 4 vibrating-wire strain gauges. As a result, a total of 224 vibrating-wire strain gauges have been permanently installed on the floating platform. The strain sensor networks distributed in the northeast zone of the floating platform are illustrated in Figure 3 (the instrumented section number 6 is located at the diagonal bracing out of the floating platform and the instrumented section number 14 is located at the internal diagonal bracing on the 7th floor of the floating platform). Figure 4 shows the sectional view of the deployment sections of strain gauges on four main trusses and the diagonal bracings on the 7th floor of the floating platform.

*3.2. Deployment Locations of Strain Gauges.* The strain gauges are deployed in consideration of the type of cross-sections (rectangle or I section), loading kinds (axial force, bending or bidirectional bending, etc.), and limitations due to construction and inaccessibility. As illustrated in Figure 5, four vibrating-wire strain gauges are installed on the two webs and the bottom flange of the upper chords (the instrumented sections number 2, 3, 8, and 11), bottom chords (the instrumented sections number 4, 5, 10, and 13), and the internal diagonal bracings on the 7th floor (the instrumented section number 14). Four vibrating-wire strain gauges are installed on four surfaces of the diagonal struts (the instrumented sections number 6, 7, 9, and 12) and the corner columns (the instrumented section number 1). In Figure 5, the location of each strain gauge is denoted as "$n-m(o)$" where $n$ stands for the instrumented section, $m$ is the sensor number, and $o$ indicates the sensor orientation.

*3.3. Real-Time Monitoring and Wireless Transmission.* The measuring principle of a vibrating-wire strain gauge relies on the relationship between the resonant frequency of the tensioned steel wire and the strain of the targeted structure. The frequency output of the vibrating-wire strain gauge is immune to electrical noise and is capable of signal transmission of several kilometers without loss of signal. Therefore, the vibrating-wire strain gauge has been the

Figure 3: Deployment sections of strain gauges on floating platform.

Figure 4: Sectional view of deployment sections of strain gauges.

Figure 5: Deployment locations of strain gauges on instrumented sections.

(a) (b)

Figure 6: Installation and protection of strain gauges.

most common type of strain gauges being widely used for long-term static strain monitoring of large-span or high-rise civil infrastructure. In many applications, especially during in-construction stage, the robustness and effectiveness of the protective measures for the vibrating-wire strain gauges and signal transmission cables are essential to the success of a long-term strain monitoring system. In this project, all the vibrating-wire strain gauges and their cabling networks are protected by stainless steel covers which are screwed onto the surfaces of the steel truss members and sealed with a sealant to prevent water seeping into the sensor housings, as illustrated in Figure 6. By these means, the survival rate of the vibrating-wire strain gauges in this project reaches to 96.88% (only 7 out of 244 vibrating-wire strain gauges were damaged in construction stage).

(a) (b)

Figure 7: Wireless data acquisition and transmission system.

The sampling rate for each vibrating-wire strain gauge is set as one data per second in the process of dismantling the temporary supports and switches to one data per 90 seconds in normal service circumstance. It can be adjusted as required for different purposes. The signals from the vibrating-wire strain gauges are first transmitted to the data loggers inside the structure through coaxial wires and then sent to the server PC at a site office (outside the in-construction building) in a wireless way through a pair of wireless access points (2 wireless LAN bridges) for real-time monitoring in construction stage (refer to Figure 7). By doing so, the strain is real-time monitored and visualized remotely thus greatly facilitating the in-construction monitoring and control. For instance, this wireless real-time monitoring system has played an important role in instructing dismantlement of the temporary supports of the floating platform (will be discussed in details in Section 4).

## 4. Strain Monitoring during Dismantlement of Temporary Supports

*4.1. Unloading Process of Temporary Supports.* During the erection of the floating platform, tower cranes were used to hoist the steel framework, and a temporary brace-frame structure was used to support the cantilever trusses. As illustrated in Figure 8, two-tier connecting trusses were added to strengthen the temporary supports and ensure the transversal stability. After completing the erection of the floating platform, the temporary supports were dismantled by use of the sandbox unloading technology. A rational planning for the unloading process is deemed to be extremely significant because (i) the total weight of the cantilever trusses for unloading is up to 9,473 tons, and it is required to make each control point subside uniformly and synchronously to ensure the consistent deformation of the floating platform and main tower; (ii) there are in total 46 widespread temporary supports for unloading; therefore, it is necessary to carry out the dismantling work in a well-coordinated way; (iii) the minimum supporting force is about 102 tons, while the maximum one is around 305 tons.

Figure 8: Skeleton diagram of floating platform erection.

Recognizing the above, a stage-by-stage procedure for unloading temporary supports in batches was formulated to make the transition of the structural stress and configuration in a steady and successive way. The temporary supports were divided into 3 groups, that is, TJO, TJM, and TJI as illustrated in Figure 9. The unloading sequence of the temporary supports was from the outer positions (TJO), through middle positions (TJM), to the inner positions (TJI), and each step was executed simultaneously to prevent uneven deformation. A new structural system was generated upon the completion of dismantlement of all temporary supports. During the unloading process of the temporary supports, the SHM system is highly desirable to detect the potential anomaly such as sudden significant rising of the structural stress, and the unloading work would be suspended once such anomaly happens.

Table 1: Statistics of the maximum stress increments and strength utilization factors during dismantlement of temporary supports.

| Monitoring zone | Tensile stress increment (MPa) | SUF | Sensor location | Compressive stress increment (MPa) | SUF | Sensor location |
|---|---|---|---|---|---|---|
| Southeast (SE) | 20.03 | 6.8% | 2-1 (S) | 21.42 | 7.3% | 4–2 (BS) |
| Southwest (SW) | 19.65 | 6.7% | 2-2 (BS) | 18.73 | 6.3% | 7–4 (W) |
| Northeast (NE) | 18.59 | 6.3% | 2–4 (N) | 21.02 | 7.1% | 4-3 (BN) |
| Northwest (NW) | 19.27 | 6.5% | 2-3 (BN) | 19.23 | 6.5% | 4-3 (BN) |

Figure 9: Layout of temporary supports.

*4.2. Statistical Analysis of Monitoring Data.* The temporary supports were dismantled step by step (commences at 09:00 am on 3 April 2010 and ends at 16:00 pm on 3 April 2010), with the unloading weight being small at the beginning and getting larger and larger till the temporary supports were totally disconnected with the cantilever trusses. The strain data from all the deployed vibration-wire strain gauges were real-time measured during dismantlement of the temporary supports and wirelessly transmitted to the site office for visualization. The stress increments of all the instrumented structural components were automatically derived by simply multiplying the measured strain increments by the elastic modulus of steel in assumption of elastic strain. The strength utilization factor (SUF), which is defined as the ratio of the stress increment of each instrumented structural component to the design strength of steel (295 MPa in this case), was also obtained using the following formula:

$$\mathrm{SUF} = \frac{\Delta s}{f_{\mathrm{sd}}}, \tag{1}$$

where SUF is the strength utilization factor; $\Delta s$ is the stress increment of each instrumented structural component; $f_{\mathrm{sd}}$ is the design strength of steel.

Table 1 lists the calculated maximum stress increments and the strength utilization factors of the instrumented structural components within the four monitoring zones, that is, southeast zone, southwest zone, northeast zone, and northwest zone of the floating platform. From Table 1, it is seen that the maximum tensile stress increment during dismantlement of the temporary supports is 20.03 MPa with an SUF of 6.8%, happening at the structural component located at the southern web of the upper chord (the instrumented section number 2 as illustrated in Figure 4) of the main steel truss TR1 in the southeast zone of the floating platform, while the maximum compressive stress increment during dismantlement of the temporary supports is 21.42 MPa with an SUF of 7.3%, happening at the structural component located at the bottom flange of the bottom chord (the instrumented section number 4 as illustrated in Figure 4) of the main steel truss TR1 in the southeast zone of the floating platform. Both the maximum tensile stress increment and the maximum compressive stress increment during dismantlement of the temporary supports are much less than the allowable stress threshold which was set as 50 MPa.

Figure 10 illustrates the growth of stresses at the structural components at the instrumented sections number 2 and 4 of the floating platform in the temporary supports' dismantling process. In Figure 10, the location of a vibrating-wire strain gauge is denoted as "SXXNN-M," where XX represents the monitoring zone of the floating platform, for instance, SE stands for the sensors at the southeast zone of the floating platform; NN is the instrumented section; $m$ is the sensor number. It is observed from Figure 10 that the stresses of the instrumented structural components (that are bisymmetrically distributed on the floating platform) grew step-by-step synchronously with the unloading process. With the help of the real-time SHM system, the temporary supports' dismantling duration was shortened to less than six hours from the originally scheduled three days.

## 5. Long-Term Strain Monitoring and Safety Evaluation

*5.1. Long-Term Strain Monitoring Data.* After the completion of dismantlement of the temporary supports, a variety of construction dead-loads such as glass curtain-walls, concrete floor-slabs, electromechanical facilities, and so forth were gradually applied on the floating platform. Real-time monitoring of the stress growth of key structural components of the floating platform is vital to guiding the massive construction in a well-organized manner in terms of both construction sequence and progress. The long-term SHM system for NHSSE has been continuously operating since the commencement of the unloading of the temporary supports of the floating platform.

In this study, two-year (from 3 April 2010 to 3 April 2012) strain monitoring data from all the installed vibrating-wire strain gauges have been acquired for statistical stress analysis and safety evaluation of the floating platform. Figure 11 illustrates the stress growth of the structural components at the instrumented sections number 2 and 4 of the floating platform in the two-year monitoring period

Figure 10: Growth of stresses during dismantlement of temporary supports.

Figure 11: Two-year monitoring of structural stress growth.

Table 2: Statistics of the maximum stress increments and strength utilization factors after two-year monitoring.

| Monitoring zone | Tensile stress increment (MPa) | SUF | Sensor location | Compressive stress increment (MPa) | SUF | Sensor location |
|---|---|---|---|---|---|---|
| Southeast (SE) | 81.63 | 27.7% | 2-1 (S) | 71.66 | 24.3% | 4–1 (S) |
| Southwest (SW) | 69.45 | 23.5% | 9–3 (N) | 81.39 | 27.6% | 7–4 (W) |
| Northeast (NE) | 72.97 | 24.7% | 9–1 (S) | 74.40 | 25.2% | 4-4 (N) |
| Northwest (NW) | 73.98 | 25.1% | 9–1 (S) | 101.84 | 34.5% | 4–2 (BS) |

(the vibrating-wire strain gauge SNW04-4 was damaged in construction stage). It is seen from Figure 11 that the stresses of the bi-symmetrically distributed structural components are increased gradually and synchronously.

*5.2. Statistical Analysis of Stress Increments.* As listed in Table 2, the maximum stress increments and the strength utilization factors of the instrumented structural components within the four monitoring zones of the floating platform are derived by use of the two-year strain monitoring data. It is known from Table 2 that the maximum tensile stress increment after a two-year monitoring period is 81.63 MPa with an SUF of 27.7%. The structural component with the maximum tensile stress increment is located at the southern web of the upper chord (the instrumented section number 2 as illustrated in Figure 4) of the main steel truss TR1 in the southeast zone of the floating platform. The maximum compressive stress increment after a two-year monitoring period is 101.84 MPa with an SUF of 34.5%. The structural component with the maximum compressive stress increment is located at the bottom flange of the bottom chord (the instrumented section number 4 as illustrated in Figure 4) of the main steel truss TR1 in the northwest zone of the floating platform.

It appears from the data that both the maximum tensile stress increment and the maximum compressive stress increment after a two-year monitoring period are relatively low in comparison with the design strength. It is therefore concluded that the key structural components of the floating platform are safe with sufficient safety margins in recognition that the majority of the dead-loads of the floating platform have been imposed up to 3 April 2012. As illustrated in Figure 12, it was observed that the measured stress increment at the bottom flange of the bottom chord of the main steel truss TR1 in the northwest zone of the floating platform (monitored by the vibrating-wire strain gauge SNW04-2) on 2 December 2010 was −21.43 MPa, which is much larger than the stress increments at other three structural components bi-symmetrically distributed on the floating platform. After an in situ inspection, it was found that keel welding of the curtain-wall suspended ceiling was being executed adjacent to the vibrating-wire strain gauge SNW04-2 during that period of time, as shown in Figure 13.

*5.3. Experiment on Welding Residual Stress.* To identify the cause of the abrupt stress change of the above-mentioned structural component, a field experiment was conducted to investigate the features of the welding residual stress caused by the keel welding of the curtain-wall suspended ceiling.

Figure 12: Stress increment pattern at welding-affected zone and other areas.

Figure 13: Keel welding of curtain-wall suspended ceiling.

The field welding experiment was carried out by welding the keel base onto the ceiling on the lower surface of the 7th floor of the floating platform with the same welding process as keel welding of the curtain-wall suspended ceiling. Seven vibrating-wire strain gauges capable of simultaneously measuring the strain and temperature were deployed around the keel base, and then the keel base was welded onto the ceiling following the prespecified welding process as illustrated in Figure 14. In Figure 14, SG-X and TEM-X are denoted as the strain and temperature sensors, respectively, in which X represents the sensor number. Figure 15 shows the welded keel base and the deployed strain gauges. It is worth underlining that the strain sensor SG-3, which is 50 mm away from the weld toe, was installed in accordance

Figure 14: Deployment locations of strain gauges and welding process.

Figure 15: Photo of welded keel base and deployed strain gauges.

with the vibrating-wire strain gauge SNW04-2 in terms of the distance between the sensor and the weld toe as well as the sensor installation direction.

The welding process in this field experiment commenced at 13:26 pm on 14 April 2011 and ended at 14:22 pm on 14 April 2011. During the whole period of the welding experiment, the strain and temperature data were synchronously acquired from all the deployed sensors from 13:00 pm to 17:50 pm on 14 April 2011. Figure 16 shows the stress and temperature time histories of the seven deployment locations during the welding experiment. It is observed from Figure 16 that the stress variation is fairly complex during the welding process and becomes almost steady one hour after completion of welding. An insight into Figure 16 reveals that welding residual stresses are generated at the sensor deployment locations which can be derived by simply subtracting the initial value, which is treated as zero at the beginning of welding process, from the structural stress of each deployment location after the completion of welding.

Table 3 lists the evaluated welding residual stresses at the seven deployment locations by use of the strain monitoring data during the welding experiment. From Table 3, it is revealed that the welding residual stresses at the seven deployment locations vary dramatically in terms of the nature and amplitude of the welding residual stress. The

Table 3: Statistics of welding residual stress.

| Strain sensor | Distance to weld toe (mm) | Welding residual stress (MPa) |
|---|---|---|
| SG-1 | 160 | 26.13 |
| SG-2 | 50 | 15.24 |
| SG-3 | 50 | −23.12 |
| SG-4 | 150 | −5.30 |
| SG-5 | 300 | 3.27 |
| SG-6 | 160 | 7.37 |
| SG-7 | 50 | 48.69 |

welding residual stress measured by the strain sensor SG-3 is −23.12 MPa, which is in line with the stress increment measured by the vibrating-wire strain gauge SNW04-2 on 2 December 2010 during the period of keel welding of the curtain-wall suspended ceiling. It is therefore concluded that the abrupt stress change of the vibrating-wire strain gauge SNW04-2 on 2 December 2010 was induced by the keel welding of the curtain-wall suspended ceiling.

## 6. Conclusions

In this paper, a long-term SHM system implemented on NHSSE featured by its huge floating platform has been outlined. This system was devised to monitor the health and safety condition of NHSSE in both construction and service stages. As part of this system, densely distributed strain sensor networks comprising 224 vibrating-wire strain gauges were constructed, from which strain monitoring data were real-time acquired and visualized remotely by forming a framework of local tethered data acquisition and long-range wireless data transmission. The effectiveness of the monitoring system was fully demonstrated in the process of dismantling the temporary supports for the floating platform during construction, through providing accurate real-time information on the evolution of structural stress which in turn greatly benefits the progress control and decision-making on the unloading procedure (the temporary supports' dismantlement duration was shortened to less than six hours from the originally scheduled three days).

Based on the statistical analysis of the real-time strain monitoring data acquired during and after dismantlement of the temporary supports of the floating platform, the following specific conclusions are obtained: (i) during dismantlement of the temporary supports, the stresses at the instrumented structural components bi-symmetrically distributed on the floating platform evolve in a step-by-step manner synchronously with the unloading process, and the stress increments of the structural components are much less than the allowable stress threshold; (ii) by examining two-year strain monitoring data from all the deployed sensors, it is revealed that the strength utilization factor of each instrumented structural component, which is defined as the ratio of the stress increment to the design strength of steel, is relatively low, indicating that the structural components of the floating platform are safe with sufficient safety margins;

Figure 16: Stress and temperature time histories during welding.

(iii) the abrupt stress change of the deployed vibrating-wire strain gauge caused by the keel welding of the curtain-wall suspended ceiling is testified by the field welding experiment. The results of the field welding experiment reveal that the welding residual stresses at different sensor deployment locations vary dramatically in terms of the nature and amplitude of the welding residual stress.

## Acknowledgments

The work described in this paper was supported by The Hong Kong Polytechnic University under the Grant G-U845 and through the Development of Niche Areas Programme (Project no. 1-BB68). The authors also wish to thank the engineers of Shenzhen Stock Exchange Operation Centre Management Co., Ltd. for their support throughout the work.

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 296124, 10 pages
doi:10.1155/2012/296124

*Research Article*

# Beat Phenomenon Analysis of Concrete Beam with Piezoelectric Sensors

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Received 1 June 2012; Accepted 22 July 2012

Academic Editor: Ting-Hua YI

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The focus of this paper is to give a better understanding of beat phenomenon in the free vibration test of a concrete beam with piezoelectric ceramic sensors from the view of mathematics. The cause of beat phenomenon from piezoelectric ceramic sensors embedded in the concrete beam is illustrated and the influence factors of beat phenomenon are discussed. The results show that the beat phenomenon from piezoelectric ceramic sensors in the concrete beam is caused by the coupled responses with similar model frequencies in different directions. The influence factors of beat phenomenon due to damping effect, impact direction, sensor position and sectional dimension are discussed. As the damping ratios increased, the amplitude of beat signal will die out in an exponential decay. Meanwhile, the damping has a tiny influence on the beat frequency of system response, the amplitudes of beat signal both in the time and frequency domain are changed with the variation of impact direction. In addition, the amplitude of beat signal will be also changed with the position of sensors altered. The beat frequency will get more with the greater difference of sectional dimension.

## 1. Introduction

Piezoelectric materials have a broadly applicative prospect in structure health monitoring due to their characters of electromechanical coupling, simple structure, low cost, good reliability, and wide frequency response range. The piezoelectric transducers based on positive and negative piezoelectric effect have been applied as actuator and dynamic measurement in damage identification [1–3], impact force location [4, 5], and fatigue crack detection [6, 7] in composite structures. Besides, the piezoelectric transducers have been successfully utilized in civil engineering structures. Song et al. [8] have developed an overheight collision detection and evaluation system for concrete bridge girders using piezoelectric transducers. Li et al. [9] applied a new type of cement-based piezoelectric sensor to monitor the traffic flows and concluded that there is a good potential for the cement-based piezoelectric sensor in the engineering application for monitoring traffic flows in the field of transportation. Gu et al. [10] provided a method of piezoelectric-based strength monitoring, in which an innovative experimental approach is proposed to conduct the concrete strength monitoring at early ages. Song et al. [11] exploited smart aggregate (SA), an innovative multifunctional transducer, fabricated by embedding a wired, waterproof piezoelectric ceramic patch into a small concrete block. The proposed SA has been successfully utilized in the structural health monitoring of a full-size bridge girder [12], reinforced concrete shear walls [13], a two-story reinforced concrete frame subjected to progressive collapse [14], circular RC columns under cyclic combined loading [15], and circular reinforced concrete columns after seismic excitations [16].

The free vibration of undercritically damped system would be decayed exponentially, as shown in Figure 1 [17]. However, when conducting the free vibration test with piezoelectric ceramic sensors, instead of an exponential decay, the vibration tends to decrease or increase periodically as shown in Figure 2, which is characterized as the classical beat phenomenon. It will be difficult to evaluate the frequency or damage of structures from the beat signals. However, there is no definite conclusion about the cause and influence factors of beat phenomenon in piezoelectric transducers currently. In this paper, the cause of beat phenomenon in piezoelectric ceramic sensors of a simply supported beam is

Figure 1: Free-vibration response of undercritically damped system.

Figure 2: The classical beat phenomenon from piezoelectric ceramic sensors.

illustrated from the view of mathematics. In addition, the influence factors of beat phenomenon due to damping effect, impact direction, sensor position, and sectional dimension are discussed.

## 2. The Analysis of Beat Phenomenon from Piezoelectric Ceramic Sensor Embedded in the Simply Supported Beam

A simply supported beam embedded with piezoelectric ceramic sensors is taken as an example to analyze the cause of beat phenomenon. The hypothesis is put up that the calculative model is the ideal simply supported beam, and the material of structure is in elastic phase. The size of the beam is $B \times W \times L$, and the piezoelectric ceramic sensor is embedded with the electrode parallel to the beam axis at the length of $z_0$ to the end of the beam. The detailed illustration of calculative model is shown in Figure 3. When subject to an impact with initial velocity of $v$ on the middle of the beam from $\theta$-direction, the beam would oscillate as free vibration.

*2.1. The Stress Condition on the Surface of Sensor Dipole.* The electrode surface of piezoelectric ceramic sensor has only axial stress when subjected to the curvature movement. The initial velocity can be resolved into orthometric components $v_x$ and $v_y$, and the axis stress $\sigma_3$ is the composition of stress component $\sigma_{3x}$ and $\sigma_{3y}$ caused by $v_x$ and $v_y$, respectively. Consequently, the stress on the electrode surface is expressed by (1)

$$\sigma_3 = \sigma_{3x} + \sigma_{3y}, \tag{1}$$

where $\sigma_{3x}$ and $\sigma_{3y}$ can be analyzed, respectively. When subjected to impact force by $v_x$, the dynamic equation in undercritically damped system is given by [17]

$$\begin{aligned} E_b I_y \frac{\partial {u_x}^{(4)}(z,t)}{\partial z^4} + m\frac{\partial^2 u_x(z,t)}{\partial t^2} \\ + a_1 E_b I_y \frac{\partial {u_x}^{(5)}(z,t)}{\partial z^4 \partial t} + a_0 m \frac{\partial u_x(z,t)}{\partial t} = 0, \end{aligned} \tag{2}$$

where $u_x(z,t)$ is the displacement response of the beam in $x$-direction, $E_b$ is the modulus of elasticity, $I_y$ is the moment of inertia of the beam in $y$-direction, $m$ is the mass of the beam in unit length, and $a_0$ and $a_1$ represent the Rayleigh damping coefficient related to the mass and stiffness, respectively. Solving (2) with the method of separating variables, the solution is expressed by $Z_x(z)Y_x(t)$, which indicates that the free vibration motion is of a specific mode shape $Z_x(z)$ having a time-dependent amplitude $Y_x(t)$. Then (2) is transformed by

$${Z_x}^{(4)}(z) - a^4 Z_x(z) = 0, \tag{3}$$

$$\ddot{Y}_x(t) + 2\xi_x \omega_x \dot{Y}(t) + \omega_x^2 Y_x(t) = 0, \tag{4}$$

where $a$ is a parameter related to the frequency of the beam, in which the natural frequency in $x$-direction is conveyed as

$$\omega_x = a^2 \sqrt{\frac{E_b I_y}{m}}, \tag{5}$$

where $\xi_x$ is the damping ratio of the beam in $x$ direction given by

$$\xi_x = \frac{a_0}{2\omega_x} + \frac{a_1 \omega_x}{2}. \tag{6}$$

The general solution of (3) is

$$Z_x(z) = A_1 \cos(az) + A_2 \sin(az) + A_3 \cosh(az) + A_4 \sinh(az). \tag{7}$$

The boundary conditions of simply supported beam are shown in (8), where $M(z)$ is the bending moment at the length of $z$:

$$Z_x(0) = Z_x'(0) = M(0) = M(L) = 0. \tag{8}$$

Substituting (8) into (7), then the value $a$ is acquired by the vibration $Z_x(z)$, and natural frequency of each mode in $x$ direction can be calculated in each order. The first mode provides the greatest contribution to the vibration so as the $u_x(z,t)$ in 1st mode can be considered as the total displacement response approximately. The value of $a$ in 1st

Figure 3: The calculative model of the simply supported beam.

Figure 4: The positive direction of computation model.

Figure 5: The operating principle of charge amplifier.

mode is displayed in (9), and the natural frequency is given by (10)

$$a = \frac{\pi}{L}, \tag{9}$$

$$\omega_x = \pi^2 \sqrt{\frac{E_b I_y}{mL^4}}. \tag{10}$$

The initial condition 1st mode is calculated as (11) based on the mode shape orthogonality:

$$\dot{Y}_x(0) = \frac{\int_0^L m Z_x(z) v(z,0) dz}{\int_0^L m Z_x^2(z) dz} \cos\theta. \tag{11}$$

By solving (4) $Y_x(t)$ is obtained as (12), which is a harmonic signal decayed exponentially:

$$Y_x(t) = \frac{\dot{Y}_x(0)}{\omega_{Dx}} \sin(\omega_{Dx} t) \exp(-\xi_x \omega_x t), \tag{12}$$

where $\omega_{Dx}$ is the free vibration frequency of the damped system which is equal to $\omega_x \sqrt{1-\xi_x^2}$. The stress on the surface of sensor dipole caused by $v_x$ can be described as

$$\begin{aligned} s_{3x} &= \frac{x_0 E_b I_y Z_x''(z_0) Y_x(t)}{I_y} \\ &= x_0 E_b Z_x''(z_0) Y_x(t), \end{aligned} \tag{13}$$

where $z_0$ depicts the position of the piezoceramic sensor. $x_0$ is the distance between $x$-axis and the centroid of piezoelectric ceramic sensor axial section. $\sigma_{3x}$ is also a harmonic signal with natural frequency of $\omega_x$. Likewise, the stress on the surface of sensor dipole caused by $v_y$ is a harmonic signal with natural frequency of $\omega_y$, which can be conveyed as

$$\sigma_{3y} = y_0 E_b Z_y''(z_0) Y_y(t), \tag{14}$$

where $y_0$ is the distance between $x$-axis and the centroid of piezoelectric ceramic sensor axial section. The stress on the electrode surface is acquired by submitting (13) and (14) into (1).

*2.2. The Output Voltage of Piezoelectric Ceramic Sensor.* By means of the piezoelectric sensors and signal acquisition system, the changes of stress are transformed into voltage signal. Following assumptions are made to the computational model of piezoelectric ceramic sensors to facilitate the analysis according to the actual background and the need of the analysis. First, the piezoelectric ceramic can be considered as the ideal elastic material without free charge. In addition, the electrode is isopotential, which means that the electric field between both electrodes is uniform, and there is no electric field in any other directions. The piezoelectric equation can be formulated as (15) [18]:

$$\begin{aligned} D &= d^d \sigma + e^\sigma E, \\ \varepsilon &= s^E \sigma + d^c E, \end{aligned} \tag{15}$$

where $D$ is the electric displacement vector, which means the charge on unit area. $\varepsilon$ denotes the strain vector. $\sigma$ expresses

Figure 6: Illustration of the beat phenomenon with equal $A_x$ and $A_y$.

the stress vector, and $E$ is the electric intensity. $s^E$ is the elastic compliance matrix, and $e^\sigma$ is the dielectric permittivity matrix. The superscripts $\sigma$ and $E$ indicate that the quantity is measured at constant stress and constant electric field, respectively. $d^d$ and $d^c$ are piezoelectric coefficient matrix, where $c$ and $d$ have been added to differentiate the converse and direct piezoelectric effects. In practice, $d^d$ is equal to $d^c$. The positive direction of computation model is shown in Figure 4. Equations (16) and (17) are the matrix form of piezoelectric equation:

$$\begin{bmatrix} D_1 \\ D_2 \\ D_3 \end{bmatrix} = \begin{bmatrix} & & & & d_{15} \\ & & & d_{24} & \\ d_{31} & d_{32} & d_{33} & & \end{bmatrix} \begin{bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \\ \sigma_4 \\ \sigma_5 \\ \sigma_6 \end{bmatrix} + \begin{bmatrix} e_{11} & & \\ & e_{22} & \\ & & e_{33} \end{bmatrix} \begin{bmatrix} E_1 \\ E_2 \\ E_3 \end{bmatrix}, \tag{16}$$

$$\begin{bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \varepsilon_3 \\ \varepsilon_4 \\ \varepsilon_5 \\ \varepsilon_6 \end{bmatrix} = \begin{bmatrix} s_{11} & s_{12} & s_{13} & & & \\ s_{21} & s_{22} & s_{23} & & & \\ s_{31} & s_{32} & s_{33} & & & \\ & & & s_{44} & & \\ & & & & s_{44} & \\ & & & & & s_{66} \end{bmatrix} \begin{bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \\ \sigma_4 \\ \sigma_5 \\ \sigma_6 \end{bmatrix} + \begin{bmatrix} & & d_{31} \\ & & d_{32} \\ & & d_{33} \\ & d_{24} & \\ d_{15} & & \end{bmatrix} \begin{bmatrix} E_1 \\ E_2 \\ E_3 \end{bmatrix}. \tag{17}$$

When the poling of sensor is in 3-direction, the eternal electric field is zero, and (16) is expressed by

$$D_3 = d_{31}\sigma_1 + d_{32}\sigma_2 + d_{33}\sigma_3. \tag{18}$$

And the charge $Q$ is

$$Q = \iint D_3 dA_3, \tag{19}$$

where $A_3$ is the area of electrode.

Piezoelectric ceramic has very high impedance, in which the output current is weak. The output signal needs to be collected through charge amplifier which can provide low impedance to signal. Figure 5 is the operating principle of charge amplifier, where $C_P$ is the capacity of piezoelectric ceramic sensor, $C_C$ is the lead wire capacitance, $C_F$ is the feedback capacitance, and $K_F$ is the gain of charge amplifier. The total charge $(C)$ can be written as

Figure 7: Illustration of $V$ by rotation vector method.

$$C = C_C + C_P + (1 + K_F)C_F, \tag{20}$$

$C_F$ is far larger than $C_C$ and $C_P$, and the influence of accuracy by $C_C$ and $C_P$ is less than 0.1% [19]. Therefore, it can be considered that the output voltage $V$ only depends on $Q$ and $C_F$, just as

$$V = \frac{Q}{C_F}. \tag{21}$$

For typical piezoelectric sheet, the axial thickness is so thin that the shear stress in 1 and 2 directions can be neglected. Substituting (18) and (19) into (21) leads to

$$V = \frac{d_{33}\sigma_3 A_3}{C_F}. \tag{22}$$

The sensitivity of piezoelectric ceramic sensor can be defined as (23), which represents the relation between output voltage and stress on the surface of electrode, and $V$ can be depicted as (24):

$$S_q = \frac{d_{33}A_3}{C_F}, \tag{23}$$

$$V = S_q\sigma_3. \tag{24}$$

Table 1: The parameters of the simply supported beam.

| $B \times W \times L$ ($m^3$) | $m$ (kg/m) | $E_b$ ($10^{10}$N/$m^2$) | $z_0$ (m) | $\theta$ (°) | $v$ (m/s) | $S_q$ (V/MPa) |
|---|---|---|---|---|---|---|
| $0.1 \times 0.11 \times 1$ | 23.57 | 2.5 | 0.25 | 45 | 1 | 0.813 |

Substitute (1), (13), and (14) into (24), and the coupled response from piezoelectric ceramic sensors can be acquired as (25). Note that for the simply supported beam subject to eccentric impact, the rotating component has no influence on the output voltage of piezoelectric ceramic sensors, because the piezoelectric coefficient ($d_{36}$) in the direction of rotation about PZT axis is zero. Then, the output voltage $V$ can be expressed by

$$V = A_x \sin(\omega_{Dx} t) + A_y \sin(\omega_{Dy} t), \tag{25}$$

where

$$\begin{aligned} A_x &= S_q \frac{x_0}{\omega_{Dx}} E_b Z_x''(z_0) \dot{Y}_x(0) \exp(-\xi_x \omega_x t) \\ &= A_{cx} \exp(-\xi_x \omega_x t), \\ A_y &= S_q \frac{y_0}{\omega_{Dy}} E_b Z_y''(z_0) \dot{Y}_y(0) \exp(-\xi_y \omega_y t) \\ &= A_{cy} \exp(-\xi_y \omega_y t), \end{aligned} \tag{26}$$

where $A_{cx}$ and $A_{cy}$ are constants unrelated to time. To illustrate the cause of beat phenomenon, it is assumed the amplitude of voltages from the piezoceramic sensor has the same value, which means that $A_x$ equals $A_y$. Then, (25) can be derived as [20]

$$\begin{aligned} V &= 2A_{cx} \exp(-\xi_x \omega_x t) \cos\left(\frac{\omega_{Dy} - \omega_{Dx}}{2} t\right) \sin\left(\frac{\omega_{Dx} + \omega_{Dy}}{2} t\right) \\ &= 2A_{cx} \exp(-\xi_x \omega_x t) \cos(\omega_B t) \sin(\omega_A t), \end{aligned} \tag{27}$$

where $\omega_B = (\omega_{Dx} - \omega_{Dy})/2$ and $\omega_A = (\omega_{Dx} + \omega_{Dy})/2$. Equation (27) shows the coupled response is an amplitude-modulated harmonic function with the frequency equal to $\omega_B$ and amplitude varying function with the frequency equal to $\omega_A$, as illustrated in Figure 6. $\omega_B$ is defined as the beat frequency and $\omega_A$ is the average frequency.

To give a general understanding of beat phenomenon, one can consider the solution of system response by the way of rotate vector method. As shown in Figure 7, $\vec{A}_x$ can be seen as a vector rotating around $X$-axis with the length of $A_x$, where $\omega_{Dx}$ is the angular velocity of $\vec{A}_x$. Similarly, $\vec{A}_y$ is a vector rotating around $X$-axis with the length of $A_y$ and the angular velocity of $\omega_{Dy}$. $\vec{A}$ is the summation of $\vec{A}_x$ and $\vec{A}_y$, and the mathematical meaning of $V$ in (25) is the projective length of $\vec{A}$ in $Y$-axis and can be expressed by (28), in which $\omega' t$ is the phase given by (29), and $A$ is the length of $\vec{A}$ shown in (30). In this stage, the cause of beat phenomenon can be further examined in Figure 8. $A_x$ and $A_y$ are illustrated as the amplitudes of system responses caused by $v_x$ and $v_y$, respectively; $V$ can be described as a simple harmonic oscillation with the amplitude of $A$ and the phase of $\omega' t$. $A$ is the upper envelope of output signal, where the amplitude appears in a way of time-varying with the beat frequency of $|(\omega_{Dx} - \omega_{Dy})/2|$. Meanwhile, the phase is altered with time and $A_y/A_x$. The beat phenomenon will become easily observable if the beat frequency is in an appropriate scope:

$$V = A \sin(\omega' t), \tag{28}$$

$$\omega' t = \begin{cases} \arccos\left(\dfrac{1 + (A_y/A_x)\cos[(\omega_{Dy} - \omega_{Dx})t]}{\sqrt{1 + (A_y/A_x)^2 + 2(A_y/A_x)\cos[(\omega_{Dy} - \omega_{Dx})t]}}\right) + \omega_{Dx} t \\ \quad \text{if } (\omega_{Dy} - \omega_{Dx})t \text{ is in 1st and 2nd quadrant} \\ -\arccos\left(\dfrac{1 + (A_y/A_x)\cos[(\omega_{Dy} - \omega_{Dx})t]}{\sqrt{1 + (A_y/A_x)^2 + 2(A_y/A_x)\cos[(\omega_{Dy} - \omega_{Dx})t]}}\right) + \omega_{Dx} t \\ \quad \text{if } (\omega_{Dy} - \omega_{Dx})t \text{ is in 3rd and 4th quadrant,} \end{cases} \tag{29}$$

$$A = \sqrt{A_x^2 + A_y^2 + 2A_x A_y \cos[(\omega_{Dy} - \omega_{Dx})t]}. \tag{30}$$

From previous discussion, the beat frequency $\omega_B$ is the major parameter related to beat phenomenon. The zero value of $\omega_B$ means that beat phenomenon could not be observed. The beat phenomenon is characterized as

Figure 8: Anatomy of the beat phenomenon with unequal $A_x$ and $A_y$.

Figure 9: The upper envelope of output signal with equal $\xi_x$ and $\xi_y$ ($\xi_x = \xi_y = \xi$).

Figure 10: The upper envelope of output signal with unequal $\xi_x$ and $\xi_y$.

the amplitude modulated periodically. The exponentially decayed amplitude implies that there is be phenomenon in the system response. Hence, the beat frequency $\omega_B$ and upper envelope $A$ are researched further in the following.

## 3. The Influence Factor of Beat Phenomenon

In this section, the influences of beat phenomenon including damping effect, impact direction, the position of sensor and anisotropic properties are discussed. Unless mentioned otherwise, the damping ratio of calculative model is 0.01 in both orthometric directions, and other parameters are listed in Table 1.

*3.1. Damping Effect.* Figure 9 shows the upper envelope of output signal with equal $\xi_x$ and $\xi_y$. As the damping ratios increase, the output signal dies out in an exponential decay. In practice, the damping ratios in different modes are diverse more or less. Figure 10 shows the upper envelope of output signal in which $\xi_x$ is 0.01 and $\xi_y$ is unequal to $\xi_x$. With the increasing of $\xi_y$, the single response with modal frequency of $\omega_y$ decays rapidly, and the coupled response of sensor tends to the one with the modal frequency of $\omega_x$.

The damping not only leads to the attenuation of amplitude but also reacts on the beat frequency. Figure 11 depicts the change of beat frequency in which $\xi_y/\xi_x$ is in the range of 0.9 to 1.1, which shows that the difference of damper makes tiny influence on beat frequency.

Figure 11: The change of beat frequency with different $\xi_y/\xi_x$ ($\omega_B'$ is the beat frequency when $\xi_x$ equals $\xi_y$).

*3.2. Impact Direction.* With the variation of impact direction, the initial conditions in orthorhombic modes would be different, and the amplitude of coupled response of sensor and the magnitudes in each modal frequency are also changed. Figure 12 is the time history and frequency domain chart of output signals with the impact direction of 0°, 45°, 60°, and 90°. From the figures of time history, the beat

Figure 12: The time history and frequency domain chart of output signals.

phenomenon is observed obviously with the eccentric impact force. If the impact direction is parallel to the symmetric axis of the section, beat phenomenon would not happen. Also, from the figures of frequency domain, it is difficult to evaluate the structural frequency if beat phenomenon happened.

Inversely, the maximum and minimum of the upper envelope expressed as $A_{\max}$ and $A_{\min}$ can be observed from beat signal. The relation between $A_{\max}/A_{\min}$ and $\tan\theta$ is shown as (31), where $\beta_0$ is the $A_y/A_x$ when $\theta$ equals 45°. Then the impact direction can be determined, which can be a practical approach to determine impact direction, such as the collision monitoring of the ocean platform or bridge in a traffic accident:

$$\frac{\mathrm{A}_{\max}}{\mathrm{A}_{\min}} = \left|\frac{1+\beta_0\tan\theta}{1-\beta_0\tan\theta}\right|. \tag{31}$$

*3.3. The Sensor Position.* The stress on the surface of sensor dipole rather than the system frequency will be changed with the alteration of sensor position. Therefore, the position of piezoelectric ceramic sensor has no influence on beat

Figure 13: The upper envelope of beat signal with different $z_0$.

Figure 14: The $A_y/A_x$ with different $z_0$.

Figure 15: The upper envelope of beat signal with different $y_0$ ($x_0$ = 0.025 m).

Figure 16: The $A_y/A_x$ with different $y_0$ ($x_0$ = 0.025 m).

frequency but has influence on the amplitude of sensor response. Figure 13 represents the upper envelope of beat signal with various $z_0$. Figure 14 shows the ratio of single amplitudes ($A_y/A_x$) in orthometric directions with various $z_0$. Note that the change of $z_0$ has no influence on the proportion of response in each single mode. It can be concluded that the beat phenomenon can be observed more easily if the piezoelectric ceramic sensors are placed near to the middle of the beam.

Figure 15 is the upper envelope of beat signal with different $y_0$, and the $A_y/A_x$ with different $y_0$ is charted in Figure 16. For the system with similar orthometric modes, the dipole surface stress caused by each single mode is different with the variation of position in the same section, so the amplitude in single mode will be also changed, which can cause the alteration of the amplitude of coupled response and the proportion of response in each single mode.

*3.4. Sectional Dimension.* Sectional dimension can be regarded as a significant factor accounting for the beat phenomenon, for the natural frequencies in orthogonal directions of the beam are related to material properties. Figure 17 is the beat frequency in which the $B/W$ is from 0.9 to 1.1, and Figure 18 is the upper envelope of output signal with different $B/W$. It can be concluded that the beat frequency will get more with the greater difference of sectional dimension.

## 4. Conclusion

The beat phenomenon in the free vibration test of a simply supported beam is analyzed from the view of mathematics. The results show that the beat phenomenon of piezoelectric ceramic sensors is caused by coupled response with similar frequency in different directions. As the damping ratios increase, the amplitude of beat phenomenon will die out in an exponential decay. However, the damping effect has a tiny influence on beat frequency. With the variation of impact direction, the proportion of response in each single

Figure 17: The beat frequency with different $B/W$.

Figure 18: The upper envelope of output signal with different sectional dimension.

mode will be changed. Hence, the amplitude of coupled response of sensor and the magnitude in frequency domain are changed. The amplitude of system response can also be changed with the variation of sensor position in axial direction, which has no influence on the proportion of response in single mode. The location shift of sensors in the same section will change not only the coupled response but also the proportion of amplitude in both single modes. The sectional dimension is a vital factor for the beat frequency, which will get more with the greater difference of sectional dimension.

## Acknowledgments

The authors are grateful for the support from Science Fund for Creative Research Groups of the National Natural Science Foundation of China (Grant no. 51121005).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 637989, 6 pages
doi:10.1155/2012/637989

*Research Article*

# Design and Experiment on Single-Chip Microprocessor for MRD Coupling Sensing and Control

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Received 19 June 2012; Accepted 25 July 2012

Academic Editor: Ting-Hua Yi

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Magnetorheological (MR) damper, a kind of intelligent vibration mitigation device, can be used to reduce the vibration or dynamic responses of structures. Its damping forces can be adjusted in real-time by updating control currents of the MR damper according to the state parameters of the structure. In this paper, a single-chip microprocessor (SCM) for MR damper coupling sensing and control is developed. The SCM has three main functions: the first one is that it can collect the acceleration responses of the structure by using the acceleration sensor in real-time; the second one is that it can use the segmented control strategy to decide the control currents of the MR damper according to the collected acceleration information; the third one is that based on the PWM technology it can realize to accurately supply power for the MR damper. The developed SCM is tested through experiment. The analysis results of the simulation and experiment show that the SCM can collect the state information of the structure in real-time and realize to choose the currents of MR damper accurately to control the damping forces of the MR damper.

## 1. Instruction

The magnetorheological (MR) fluid belongs to the class of smart materials. The essential characteristic of MR fluids is that it can reversibly change from free-flowing, linear viscous liquids to semisolids having controllable yield strength in milliseconds when exposed to a magnetic field. This feature provides simple, quiet, rapid-response interfaces between electronic controls and mechanical systems [1]. The MR damper utilizing MR fluids to provide controllable damping forces is a kind of intelligent semiactive vibration mitigation device. In 1996, Carlson and Spencer [2] have been designed and built a 20t MR damper to study the salient features of large-scale MR devices. In 2001, the first full-scale implementation of MR dampers in building was realized in the National Museum of Emerging Science and Innovation of Tokyo. In this building, two 30t MR dampers built by the Sanwa Tekki Corporation were installed between the third and fifth floors; the MR fluids inside the damper are manufactured by the Lord Corporation [3]. In 2001, Ni et al. [4] presented a method to identify the characteristics of a MR damper for a cable bridge. In 2003, Fujitani et al. [5] developed a 40t MR damper for a real base-isolated building in Japan. In 2008, Xu and Guo [6] built a 26t MR damper to investigate its mechanical performances. In 2012, Li et al. [7] researched wind-induced vibration control of Dalian international trade mansion by tuned liquid dampers.

MR dampers are controllable devices by adjusting currents of the coils to change the magnetic field intensity to control their damping forces, so these are very important that how to choose the control currents, and then how to apply the control currents to MR dampers. Some control strategies and power supply methods of MR dampers have been developed by researchers. In 1996, Dyke et al. [8] proposed a semiactive clipped optimal control strategy based on acceleration feedback for the MR damper. In 1998, Chung et al. [9] also used the acceleration feedback control of seismic structures to successfully reduce vibration of structures. In 2003, Xu et al. [10] presented a neural networks control method. In 2005, Wang and Liao [11] presented an inverse neural network model for MR dampers to generate the control voltage. In 2006, Xu and Guo [12] developed

Figure 1: Schematic of MR damper.

a fuzzy control method for earthquake mitigation structures with MR dampers. In 2004, Yang et al. [13] developed a current-driver power supply to power a MR damper, and the current-driven power supply can substantially reduce the MR damper response time. In 2004, Lord Corporation and Motorola Incorporation [14] have designed controllers of MR dampers in the automobile field. In 2007, Liu [15] presented a controllable current amplifier for MR damper. In 2011, Yi et al. [16] present a method to place sensors for structure.

If MR dampers will be used to reduce the vibration successfully, except for the current-driver power supply, the information collection system is necessary. That is, the sensing system should be used to monitor state parameters of the structure building to realize the real-time vibration control of the structure. In this paper a single-chip microprocessor (SCM) for MR damper coupling sensing and control is developed to monitor the structure and control MR dampers. The SCM has three main functions: the first one is that it can collect the acceleration responses of the structure by using the acceleration sensor in real-time; the second one is that it can use the segmented control strategy to decide the control currents of the MR damper according to the collected acceleration information; the third one is that based on the PWM technology it can realize to accurately supply power for the MR damper. The developed SCM is tested through experiment. The analysis results of the simulation and experiment show that the SCM can collect the state information of the structure in real-time and realize to choose the currents of MR damper accurately to control the damping forces of the MR damper.

## 2. Mathematical Model of MR Dampers

MR dampers typically consist of a hydraulic cylinder containing micrometer-sized magnetically polarizable particles dispersed in hydrocarbon oil, as shown in Figure 1. The Bingham model [12] is often used to one of the mathematical model of MR dampers, and its relationship between the stress and strain rate can be expressed as

$$\tau = \eta\dot{\gamma} + \tau_y(H)\,\mathrm{sgn}(\dot{\gamma}), \tag{1}$$

where $\tau$ is the shear stress in fluid, $\eta$ is the Newtonian viscosity, independent of the applied magnetic field, $\dot{\gamma}$ is the shear strain rate and $\tau_y$ is the dynamic yielding shear stress controlled by the applied field, and $H$ is the magnetic field. $H$ can be controlled by updating the control current, and $\tau_y$ will be the function of the control current $I$ [12],

$$\tau_y = A_1 e^{-I} + A_2 \ln(I + e) + A_3 I, \tag{2}$$

where $A_1 = -11374$, $A_2 = 14580$, $A_3 = 1281$, and $e$ is mathematical constant.

## 3. Design of SCM for MR Damper Coupling Sensing and Control

A SCM for MR dampers coupling sensing and control, monitoring the structure and controlling MR dampers, is developed. Its design block diagram is shown in Figure 2. As shown in Figure 2, the SCM has three main functions: the first one is that it can collect the acceleration responses of the structure by using the acceleration sensor in real-time; the second one is that it can use the segmented control strategy to decide the control currents of the MR damper according to the collected acceleration information by using SCM; the third one is that based on the PWM technology it can realize to accurately supply power for the MR damper.

*3.1. Design of Sensing System.* In the process of the vibration mitigation control of structures with MR dampers, structural acceleration is one of important parameters will clearly show the vibration condition of the structure, and it is also very useful to decide control currents of MR dampers. This paper will use a three axis low-g acceleration sensor (MMA7260QT [17]) to collect acceleration responses of the structure in real-time. The MMA7260QT low cost capacitive micromachined accelerometer features signal conditioning, a 1-pole low pass filter, temperature compensation, and g-Select which allows for the selection among 4 sensitivities. Zero-g offset full scale span and filter cut-off are factory set and require no external devices. It includes a sleep mode that makes it ideal for handheld battery powered electronics. The device can measure both + and − acceleration, and transform acceleration to voltage signal. With no input acceleration, the output is at mid-supply. For positive acceleration, the output will increase above $V_{\mathrm{DD}}/2$ ($V_{\mathrm{DD}}$ is the supply voltage). For negative acceleration, the output will decrease below $V_{\mathrm{DD}}/2$. So the relationship between the acceleration input and the voltage output of the MMA7260QT is

$$V_{\mathrm{out}} = \frac{V_{\mathrm{DD}}}{8} A_{\mathrm{cc}} + \frac{V_{\mathrm{DD}}}{2}, \tag{3}$$

where $A_{\mathrm{cc}}$ is the acceleration input, $V_{\mathrm{out}}$ is the voltage output. In this paper, according to the MMA7260QT's features, it is used to collect the three-way acceleration responses of the structure in real-time, and then the voltage output signal will be send to the microcontroller. Figure 3 shows its connection diagram.

Figure 2: The block design diagram of the SCM for MR dampers coupling sensing and control.

Figure 3: The connection diagram of MMA7260QT.

*3.2. Design of Control Strategy.* In order to effectively reduce the vibration responses of the structure with MR dampers under earthquake or wind excitation, the control currents must be adjusted in real-time according to the structural state parameters collected by sensors. This paper uses a microcontroller, Atmega16, to receive the voltage output signal of the acceleration sensor, produce the control signal of the MR damper's current by using the segmented control strategy, and then output control current signal to the MR damper by using PWM technology.

The segmented control strategy is an easy method to be realized. It can be described as

$$\begin{aligned} I &= 2.40 & |A_{cc}| &\geq 1 \\ I &= |A_{cc}| + 1.4 & 0.4 \leq |A_{cc}| &\leq 0.8 \\ I &= 4|A_{cc}| + 0.6 & 0.05 < |A_{cc}| &< 0.4 \\ I &= 0.50 & |A_{cc}| &= 0.05 \\ I &= 0.00 & |A_{cc}| &\leq 0.01. \end{aligned} \tag{4}$$

The microcontroller cannot output the control current of the MR damper, so pulse-width modulation (PWM) technique is used. PWM [18] is a powerful technique for controlling analog circuits with a processor's digital outputs. PWM is employed in a wide variety of applications, ranging from measurement and communications to power control and conversion. As we all know, at any moment, the output of the digital circuit only has two states, ON and OFF. On the other hand, an analog signal has a continuously varying value, with infinite resolution in both time and magnitude. PWM is a way of digitally encoding analog signal levels. Through the use of high-resolution counters, the duty cycle of a square wave is modulated to encode a specific analog signal level. The PWM signal is still digital because, at any given instant of time, the full DC supply is either fully on or fully off. The voltage or current source is supplied to the analog load by means of a repeating series of ON and OFF pulses. The ON-time is the time during which the DC supply is applied to the load, and the OFF-time is the period during which the supply is switched off. Given a sufficient bandwidth, any analog value can be encoded with PWM. At the same time, one of the advantages of PWM is that the signal remains digital all the way from the processor to the controlled system; no digital-to-analog conversion is necessary. By keeping the digital signal, noise effects are minimized.

The microcontroller, Atmega16 [19], produced by Atmel Corporation, can output PWM control signal to the MR damper. Figure 4 is the Atmega16's connection diagram. It can be seen from Figures 3 and 4 that: (1) the Atmega16's PB0 and PB1 pins are connected with the MMA7260QT's g-select1 and g-select2 pins, respectively, to setup the sensor's working rage as ±4 g; (2) the Atmega16's PB2 pin is connected with the MMA7260QT's $\overline{\text{SleepMode}}$ pin to setup the sensor's sleep mode; (3) the Atmega16's PA0, PA1 and PA2 pins of are connected with the MMA7260QT's Xout, Yout and Zout pins, respectively, to receive the acceleration

Figure 4: The connection diagram of Atmegal16.

Figure 5: The SCM for MR dampers coupling sensing and control.

data of the structure collected by the sensor. After the Atmega16 received the acceleration of data the structural, it will calculate the control currents of the MR damper using the segmented control method, and then the decided currents will be outputted to the optocoupler through the Atmega16's PWM pin (the Atmega16's PD4 pin).

## 4. Experiment of SCM for MR Damper Coupling Sensing and Control

According to the above principle, the SCM for MR damper coupling sensing and control is developed, as shown in Figure 5. In order to grasp and analysis the performance of the SCM, a series of experiments are carried out, including static experiment and dynamic experiment, as shown in Figure 6.

In the process of the static experiment of the SCM, the SCM is connected with the MR damper, and it will be motionless. Because the SCM is motionless, the acceleration and the PWM waveform by produced by the microcontroller will be not change. It can be seen from Figure 6(a) that the direction of the $z$-axis of the acceleration sensor is the direction of the Earth's gravity field, so the three axis outputs of the acceleration are

$$\begin{aligned} X_{\text{out}} &= 0\text{g}, \\ Y_{\text{out}} &= 0\text{g}, \\ Z_{\text{out}} &= 1\text{g}. \end{aligned} \tag{5}$$

And the PWM waveform is shown on the oscilloscope. Similarly, the direction of the $x$-axis or $y$-axis of the acceleration sensor can be placed as the direction of the Earth's

Table 1: Acceleration comparison of excepted value and measured value.

| Acceleration (g) | Excepted value (A) | Measured value (A) | Relative error (%) |
|---|---|---|---|
| ≥1 | 2.40 | 2.42 | 0.82 |
| 0.8 | 2.20 | 2.19 | 3.69 |
| 0.6 | 2.00 | 2.04 | 1.96 |
| 0.4 | 1.80 | 1.80 | 0 |
| 0.2 | 1.40 | 1.41 | 0.71 |
| 0.1 | 1.00 | 1.03 | 2.91 |
| 0.05 | 0.50 | 0.48 | 4.17 |
| ≤0.01 | 0.00 | 0.00 | 0 |

Figure 6: The performance experiment of SCM (a) the static experiment of SCM and (b) the dynamic experiment of SCM.

Figure 7: The comparison cure of measured value and excepted value.

gravity field, and the same PWM waveform is shown on the oscilloscope. This indicate that in the static state the SCM can accurately measure the acceleration value and produce the PWM signal by using the program of segmented control strategy installed in the microcontroller.

During the dynamic experiment of the SCM, the SCM is fastened the top of the vibration machine, and it will vibrate under the different sine wave excitation. Figure 6(b) shows the PWM waveforms and values of the magnetic field intensity produced by the MR damper under the same sine wave excitation and at the different moment. At the same time, the currents of the MR damper are measured under the different acceleration, and compared with the expected values, as shown in Table 1 and Figure 7. It can be seen from Table 1 and Figure 7 that the measured values of the currents and the expected values are consistent, and the maximum relative error, 4.17%, meets the requirement of the vibration mitigation of the structure.

## 5. Conclusions

In this paper, a single-chip microprocessor (SCM) for MR damper coupling sensing and control is developed to monitor the structure and control MR dampers. It has three main functions: the first one is that it can collect the acceleration responses of the structure by using the acceleration sensor in real-time; the second one is that it can use the segmented control strategy to decide the control currents of the MR damper according to the collected acceleration information; the third one is that based on the PWM technology it can realize to accurately supply power for the MR damper. The developed SCM is tested by simulation and experiment. The following conclusions are obtained from this study.

The MMA7260QT is a three-axis low-g acceleration sensor. It integrates a low pass filter and can realize accurately measure the acceleration responses of the structure. Furthermore, it is easy to connect with the microcontroller and send the acceleration information to the microcontroller by voltage signals.

The static and dynamic experiments of the SCM indicate that the SCM can use the segmented control strategy to produce the accurate PWM signal according to the collected acceleration data. And the measured values of the currents and the expected values are consistent.

## Acknowledgments

This work was financially supported by the National Natural Science Foundation of China (61004064), NSAF of China (11176008), Jiangsu Province Brace Program (BE2010069), Jiangsu Province Postgraduate Research, and Innovation Project of the Universities in (CXZZ_0160).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 612726, 11 pages
doi:10.1155/2012/612726

*Review Article*

# Methodology Developments in Sensor Placement for Health Monitoring of Civil Infrastructures

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Received 16 April 2012; Accepted 7 August 2012

Academic Editor: Gangbing Song

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Optimal sensor placement (OSP) technique plays a key role in the structural health monitoring (SHM) of large-scale civil infrastructures. This paper outlines an overview of current research and development in the field of OSP problems in a perspective of both researchers and engineers. The paper begins with a definition of the model of sensor placement and provides the basic issues covering relevant methodologies. The primary evaluation criteria and main sensor placement methods are then discussed in details. Following that, the linkage between several influential sensor placement methods is described. Finally, existing problems and promising research efforts in the OSP problem of civil SHM are discussed.

## 1. Introduction

Today, there are many more large-scale civil infrastructures than in the past. These structures are susceptible to random vibrations in its long service period; whether it is from changes in temperature, severe wind gusts, torrential rain, strong earthquake tremors, or abnormal loads such as explosions [1–3]. The coupling effects between these natural or man-made factors make the problem even more complicated. Although the routine visual inspection is effective in some cases, its effectiveness in finding the possible defects in time is questionable. Therefore, it is imperative that a continuous structural health monitoring (SHM) system be developed. The SHM systems are generally envisaged to [4]: (i) validate the design assumptions and parameters with the potential benefits of improving design specifications and guidelines for future similar structures; (ii) detect anomalies in loading and response, and possible damage/deterioration at an early stage to ensure the structural and operational safety; (iii) provide the real-time information for the safety assessment immediately after disasters and extreme events; (iv) provide the evidence and instruction for planning and prioritizing the structural inspection, rehabilitation, maintenance, and repair; (v) monitor the repair and reconstruction with the view of evaluating the effectiveness of maintenance, retrofit and repair work; (vi) obtain the massive amount of in situ data for the leadingedge research in structure engineering, such as wind- and earthquake-resistant designs, novel structural types, and smart material applications.

A typical SHM system includes three major components: a sensor system, a data processing system (including the data acquisition, transmission, and storage), and a health evaluation system [5]. The sensors utilized in the SHM are required to monitor not only the structural status including stress, displacement, acceleration, but also those influential environmental parameters, such as wind speed, temperature, humidity, soil condition of its foundation. Generally, the more locations of sensors placed on a structure are, the more detailed information of the structure could be obtained. In addition, advances in sensing technology have also enabled the use of large numbers of sensors for the SHM. However, high costs of data acquisition systems (including development, purchase and maintenance costs for the sensors, as well as resource and communication costs) and accessibility limitations constrain in many cases the wide distribution of a large number of sensors on a structure. Especially, many structures have to be tested under operational conditions, in which the sensors are not easily amenable to be removed

or changed. In such cases, a practical question that naturally arises is how to select a set with a minimum number of sensor locations from all possibilities, such that the data collected can provide adequate information for the identification of the structural behavior. Otherwise, incomplete structural and environmental characteristics may be measured and an accurate SHM assessment will be impossible. Generally, the design of a sensor network for the SHM system excepts identifying the rational number and location of sensors in order to fulfill specified performance requirements within a set of system constraints. Another area of interest is sensor network robustness, which aims at maintaining the stability of the sensor network when some sensors malfunction [6]. The development and implementation of various kinds of sensor placement methods capable of fully achieving the above objectives and benefit are still challenging at present, and need well-coordinated interdisciplinary research for full adaptation of innovative technologies developed in other disciplines to applications in the civil engineering community. Actually, the optimal sensor placement (OSP) has been a subject of important international research in recent years. The research in this subject covers sensing, structural dynamics, information technology, optimization theory, and so forth [7, 8]. The current challenges for the OSP method are being identified as intelligent and efficient algorithms, evaluation criteria for different sensor placement methods, inherent relationship between different sensor placement methods, and uncertainty, sensitivity, reliability, and redundancy in sensor networks, and so forth.

In this paper, a brief overview of the OSP problem is given in Section 2; a detailed discussion of current status of evaluation criteria and sensor placement methods are provided in Sections 3 and 4, respectively; the comparison of several influential sensor placement methods is described in Section 5, and the concluding remarks are given in Section 6. This paper is not intended to be encompassing in terms of directly comparing the various techniques against each other and assessing their performance. Rather, the review is based on categorizing salient features of these approaches and the desirability of certain features when they are applied to SHM systems.

## 2. Objective of Sensor Placement for SHM System

The OSP issue is important in cases where the properties of a structure, described in terms of continuous functions, need to be identified using discrete sensor information. Thus, the sensor placement optimization is a kind of combinatorial optimization problem that can be generalized as "given a set of $n$ candidate locations, find $m$ locations, where $m < n$, which may provide the best possible performance." The number of all distinct sensor configurations involving $m$ sensors is given by the expression

$$C = \frac{n!}{m!(n-m)!}. \tag{1}$$

For a structure that has simple geometry, or smaller number of degrees of freedom (DOF), experience and a trial-and-error approach may suffice to solve the problem. For a large-scale complicated structure, whose finite element model may have thousands, tens of thousands, and even hundreds of thousands of DOFs, an exhaustive search would be extremely time consuming or even impossible. Thus, a systematic and efficient approach is needed to solve such a computationally demanding problem.

*2.1. Model for Sensor Placement in Structural Dynamics.* The sensor placement problem can be investigated from uncoupled modal coordinates of governing structural equations as follows [9]:

$$\ddot{\mathbf{q}}_i + \mathbf{M}_i^{-1} \cdot \mathbf{C}_i \cdot \dot{\mathbf{q}}_i + \mathbf{M}_i^{-1} \cdot \mathbf{K}_i \cdot \mathbf{q}_i = \mathbf{M}_i^{-1} \cdot \mathbf{\Phi}_i^T \cdot \mathbf{B}_0 \cdot \mathbf{u},$$
$$\mathbf{y} = \mathbf{\Phi} \cdot \mathbf{q} + \boldsymbol{\varepsilon}, \tag{2}$$

where, $\mathbf{q}_i$ is the $i$th modal coordinate and is also the $i$th element of the modal coordinate vector $\mathbf{q}$ in the 2nd equation; $\mathbf{M}_i$, $\mathbf{K}_i$, and $\mathbf{C}_i$ are the corresponding $i$th modal mass, stiffness, and damping matrices, respectively; $\mathbf{\Phi}$ means the mode shape matrix with its $i$th column as the $i$th mass-normalized mode shape; $\mathbf{B}_0$ simply denotes the location matrix formed by ones (with actuators) and zeros (without actuators), specifying the positions of the force vector $\mathbf{u}$; The superscripts $-1$ and $T$ represent the inversion and transpose of a matrix, respectively; $\mathbf{y}$ implies the measurement column vector indicating which positions of the structure are measured, and $\boldsymbol{\varepsilon}$ is the stationary Gaussian white noise with zero mean and a variance of $\sigma^2$, describing measurement uncertainties.

*2.2. Basic Issues Regarding Sensor Placement.* Sensor placement problem described in (2) is, essentially, divided into three aspects [10]. Firstly, what is the least number of sensors required to be installed in a structure for a successful dynamic testing? Secondly, where should these sensors be installed, including those additional ones if available? And if these additional sensors are available, should they be installed as redundant sensors or place them in other positions? Lastly, how could the effectiveness of different sensor placement methods be evaluated?

In fact, these three aspects are intervening with each other, and the first problem is partly resolved. It is known that the minimum number of sensors to be instrumented could not be less than the number of mode shapes to be identified, which is determined by the observability of the structure. Moreover, the practical number of sensors, which is limitedly preset before test due to the availability of instruments, is usually larger than the minimum number because of the requirement to visualize the identified mode shapes (i.e., the identified mode shapes are dependent on each other and one or more of them can be determined by a combination of others) [11]. The second problem is the core one that has attracted the majority of research interest, which depends largely, however, on the third aspect. For the limited number of sensors available, the problem is the development of a suitable sensor placement performance measure to be

optimized and the selection of an appropriate method with which it can be optimized. Some approaches require a single calculation to be performed, some are iterative, and many others take the form of an objective function to which an optimization technique must be applied. The third aspect includes several possibilities for assessing the performance of chosen sensor sets. If the economical issues were not considered, only after the last two aspects are clearly understood, it was then possible to know exactly the required number of sensors to be installed.

## 3. Criteria to Evaluate Suitability of Sensor Locations

It should be noted that a good method for sensor placement in a particular application is not necessarily good for another project. That means the effectiveness of a certain sensor placement method depends on the evaluation criteria to some extent. A criterion stresses one perspective whereas another pays more attention to another aspect. Compromises need to be made if one or several of the criteria are to be used in dynamic testing. In this section, some influential criteria are discussed both from historical points of view and from their impacts on practices and the development of sensor placement theory. Of course, bias and omissions could not be avoided for such a daunting task because of personal knowledge. Descriptions have been kept mathematically light intentionally; full descriptions of the criteria may be found in the references.

*(1) Modal Assurance Criterion (MAC).* As known, the measured mode shape vectors have to be as linearly independent as possible, which is a basic requirement to distinguish measured or identified modes. Moreover, the linear independency is also particularly important when the test results are to validate or to update the FE model. The larger space angles among the measured modal vectors should be guaranteed while choosing measuring points in order to keep the original properties of the structure if possible. Carne and Dohmann (1995) [12] thought that the MAC was an ideal scalar constant relating the causal relationship between two modal vectors

$$\mathrm{MAC}_{ij} = \frac{\left(\Phi_i^T \Phi_j\right)^2}{\left(\Phi_i^T \Phi_i\right)\left(\Phi_j^T \Phi_j\right)}, \tag{3}$$

where, $\Phi_i$ and $\Phi_j$ represent the $i$th and $j$th column vectors in matrix $\Phi$, respectively, and the superscript $T$ denotes the transpose of the vector.

The element values of the MAC matrix range between 0 and 1, where zero indicates that there is little or no correlation between the off-diagonal element $\mathrm{MAC}_{ij}$ $(i \neq j)$ (i.e., the modal vector easily distinguishable) and one denotes that there is a high degree of similarity between the modal vectors (i.e., the modal vector fairly indistinguishable). For an optimal (orthogonal) set the MAC matrix would be diagonal, thus the size of the off-diagonal elements could be an indication of optimal result.

*(2) Singular Value Decomposition Ratio (SVDR).* The singular value decomposition of mode shape matrix specified at a certain DOF provides another measure to the chosen sensor locations [13]. The method evaluates the ratio of the largest to the smallest singular value of the mode shape matrix:

$$\mathrm{SVDR} = \frac{\sigma_{\max}}{\sigma_{\min}}, \tag{4}$$

where, $\sigma_{\max}$ and $\sigma_{\min}$ denote the largest to the smallest singular value of the mode shape matrix, respectively.

The smaller the ratio, the better the choice of sensor locations. There are three reasons to adopt the SVDR criteria: namely, mode orthogonality, the condition for mode expansion, and the observability of the modes [14]. The lower limit of the SVDR is one in the case that the mode shapes are orthogonal, which is an ideal situation. This criterion can also be termed as the condition criteria since the SVDR of a truncated mode shape matrix is nothing else, but its condition number.

*(3) Measured Energy per Mode.* Since the kinetic energy of a structure is usually not evenly distributed into the modes of the structure, the measured DOFs are expected to capture a large part of the total kinetic energy of the structure, and the energy contained in the measured DOFs for each mode should be a significant portion of that mode to satisfactorily measure the modes. It is based on the traditional heuristic visual inspection, which is to visually inspect the response of a structure, to examine the mode shapes of interest, and to select locations with high amplitudes of responses. This criterion helps to select those sensor positions with possible large amplitudes, and to increase the signal to noise ratio, which is critical in harsh and noisy circumstances.

*(4) Fisher Information Matrix (FIM).* The criterion of the FIM results from minimizing the covariance matrix of the estimate error for an efficient unbiased estimator from the perspective of statistics. It relates also to the information contained in the measured responses from the viewpoint of information community:

$$\mathrm{FIM} = \Phi^T \Phi. \tag{5}$$

Therefore, the procedure using this criterion for selecting the best sensor placements is to unselect candidate sensor positions such that the FIM is maximized. In practice, three variants of the FIM are used, the determinant, the trace, as well as the minimum singular value of the FIM, which are maximized to increase the information or to decrease the uncertainties of the estimates. What need to be mentioned is that these three variants are just different norms of a matrix, that means, different sensor placement methods based on these variants of the FIM will yield similar, if not the same, results for most cases [15, 16].

*(5) Probability-Based Damage Detection Criterion.* Since any SHM issue is fundamentally a detection problem in its simplest execution, Flynn and Todd [17] developed a global

optimality criterion within a detection theory framework that seeks to optimize a given sensing network resource allocation for minimizing the expected appearance of type I or type II error.

The first global performance measure, which is called the global false alarm rate, is the expected proportion of the structural undamaged regions that will be incorrectly identified as damage, or render type I error. It can be expressed as

$$\overline{P}_{\mathrm{FA}} = \sum_{k=1}^{K} \frac{P(d_{k1} \mid h_{k0})P(h_{k0})}{\sum_{k=1}^{K} P(h_{k0})}, \tag{6}$$

where, $P(d_{k1} \mid h_{k0}) = \Pr(T[k] > \gamma[k]h_{k0})$ is the local detection rate for region $k$, $d_{k1}$ denotes $m_1$ is the local damage state, $h_{k0}$ represents $m_0$ is the true local damage state, $\gamma[k]$ means the cost ratio.

The second global performance measure, which is labeled the global detection rate, is the expected proportion of the structural damaged regions that will be correctly identified as damaged. This is equivalent to the complement of the proportion of the structure to exhibit type II error. It can be expressed as

$$\overline{P}_{D} = \sum_{k=1}^{K} \frac{P(d_{k1} \mid h_{k1})P(h_{k1})}{\sum_{k=1}^{K} P(h_{k1})}, \tag{7}$$

where, $P(d_{k1} \mid h_{k1}) = \Pr(T[k] > \gamma[k]h_{k1})$ denotes the local detection rate for region $k$, $h_{k1}$ represents $m_1$ is the true local damage state.

*(6) Mean Square Error (MSE).* For randomly vibrating structures, the problem of estimating the optimal sensor locations can be chosen to minimize the total, averaged over all prediction points, mean square error (MSE) of the response prediction [18]. The MSE $\sigma_i^2(r_1^{(o)},\ldots,r_N^{(o)})$ gives the uncertainty in the resulting response prediction at the $i$th prediction point located at position $r_i^{(p)}$. Thus, the problem of optimally placing the sensors in the structure for the purpose of obtaining predictions of the response involving the least uncertainty is formulated as a multiobjective optimization of finding the optimal locations of the observation points $(r_1^{(o)},\ldots,r_N^{(o)})$ which simultaneously minimize the $P$ objectives $(\sigma_1^2,\ldots,\sigma_P^2)$. The MSE $\sigma^2(r_i^{(p)};r_1^{(o)},\ldots,r_N^{(o)}) \equiv \sigma_i^2(r_1^{(o)},\ldots,r_N^{(o)})$ of the kriging prediction at location $r_i^{(p)}$ is given by

$$\begin{aligned}\sigma_i^2\left(r_1^{(o)},\ldots,r_N^{(o)}\right) &= E\left[\left\{X\left(r_i^{(p)};t\right) - X^*\left(r_i^{(p)};t\right)\right\}^2\right] \\ &= K\left(r_i^{(p)},r_i^{(p)}\right) - c_i^T K^{-1} c_i.\end{aligned} \tag{8}$$

The MSE depends on the location of the observation points $(r_1^{(o)},\ldots,r_N^{(o)})$ and the covariance characteristics of the response. The simple kriging estimator $X^*(r_i^{(p)};t)$ is unbiased.

*(7) Information Entropy (IE).* In experimental design, it is desirable to design the sensor configuration such that the resulting measured data are most informative about the structural model parameters selected for estimation. The information entropy (IE) as the measure of the uncertainty in the system parameters gives the amount of useful information contained in the measured data. The most informative test data are the ones that give the least uncertainty in the parameter estimates or, equivalently, the ones that minimize the IE. Thus, among all sensor configurations, the optimal sensor configuration can be selected as one that minimizes the IE [19–22]. That is

$$L_{\mathrm{best}} = \arg\min_{L} H(L;\underline{\theta}_0,\Sigma), \tag{9}$$

where, $H(L;\underline{\theta}_0,\Sigma)$ is given by (10) and the minimization is constrained over the set of $N_p$ measurable DOFs. The lower bound of the information entropy is then given by $H_{\min}(L_{\mathrm{best}};\underline{\theta}_0,\Sigma)$

$$H(L,\underline{\theta}_0,\sigma_0) = \frac{1}{2}N_\theta \ln(2\pi) - \frac{1}{2}\ln[\det Q(L,\underline{\theta}_0,\Sigma)], \tag{10}$$

where $\underline{\theta}_0 \equiv \hat{\underline{\theta}}(L,\Sigma,D)$ is the optimal value of the parameter set $\underline{\theta}$ that minimizes $J(\underline{\theta};L,D)$, which means the measure of fit between the measured and the model response time histories; $Q(L;\underline{\theta},\Sigma)$ is an $N_\theta \times N_\theta$ semipositive definite matrix defined as $NN_0\nabla_{\underline{\theta}}^T J(\underline{\theta};\Sigma,D)$.

*(8) Mutual Information (MI).* The MI gives a measure of how much information one sensor location, "learns" from another [23]. If there are two sets of measurement locations, $A$ and $B$, the amount of information learned by $a_i$ about $b_j$ is presented by the mutual information:

$$I\left(a_i,b_j\right) = \log 2\left[\frac{P_{AB}\left(a_i,b_j\right)}{P_A(a_i)P_B\left(b_j\right)}\right], \tag{11}$$

where $a_i$ and $b_j$ are the measurements from locations $A$ and $B$, respectively; $P_A(a_i)$ and $P_B(b_j)$ are the individual probability densities for $A$ and $B$, and $P_{AB}(a_i,b_j)$ is the joint probability density for $A$ and $B$.

If the measurement of $a_i$ is completely independent of the measurement of $b_j$, $I(a_i,b_j)$ becomes zero. The average mutual information between $A$ and $B$ is calculated by averaging over all the sensor locations, and the optimal sensor location determined by minimizing the mutual information between sensors.

*(9) Representative Least Squares Criterion (RLSC).* The RLSC derived from the representative least square method depends on both the characteristics and the actual loading situations of a structure [24]. The objective of the RLSC is to achieve the best identification of modal frequencies and mode shapes through almost unbiased estimation of modal coordinates. It selects sensor positions with the best subspace approximation of the vibration responses from the linear space spanned by the mode shapes.

$$J_{\mathrm{rls}} = (\hat{q}_s - \hat{q}_{\mathrm{OLS}})^T(\hat{q}_s - \hat{q}_{\mathrm{OLS}}), \tag{12}$$

where, $J_{\text{rls}}$ denotes the objective function of RLSC, $\hat{q}_s$ means a biased estimator with only partially $s$ components, $\hat{q}_{\text{OLS}}$ represents the ordinary least squares estimator with all $n$ components, respectively. The $\hat{q}_s$ is the best estimator which achieves the smallest $J_{\text{rls}}$ among all possible combinations including $s$ components drawn from total $n$ components. The smaller the RLSC, the better the identified modal parameters.

*(10) Visualization of Mode Shape.* Structural engineer must first visualize the mode shape vectors identified from modal experiments to have a first impression of the overall motion of the structure under consideration, as discussed by Carne and Dohmann [12]. This criterion has no concrete mathematical formulations not like the other criteria. It depends on the structure, and usually the points in the frame corner or middle are picked. As suggested by Pickrel [25], the number of sensors required to visualize the mode shapes is at least five times the number of mode shapes in order to provide a crude visual depiction of the shapes and avoid spatial aliasing. But modern structures composed of new materials can present unexpected modes, or complex couplings between substructures. Thus, it is not possible to design the best sensor set using this criteria prior to test because it would involve the vibratory behavior that should be already perfectly known.

## 4. Development of Sensor Placement Method

The aforementioned analyses have shown that the general sensor selection problems addressing diagnosability, or observability, reliability, and detectability are NP complete and are therefore computationally intractable. These types of issues require efficient searching solutions in order to generate acceptable results in reasonable time. Many methods have been developed for general optimized solution searches. These approaches range from applying the constraints on the objective function (evaluation criteria) to streamline the optimization process to applying the advanced artificial analysis techniques such as the genetic algorithms and simulated annealing algorithms. The point of this section is not to review each search solution but to identify the variety of algorithms available. Especially, the computational intelligent algorithms are reviewed in details due to their unique advantages for the OSP problem.

*4.1. Deterministic Optimization Methods.* A number of unconstrained (Newton methods) and constrained (linear and nonlinear programming) deterministic optimization methods can be used for the optimal sensor location. This includes countless methods based on the availability of gradients and Hessians. Simple deterministic techniques are sufficient for the local search, and the constrained optimization has a great degree of complexity [26]. For the structures with simple and regular shapes (like beam, plates, etc.), the optimal layout of the sensors can be obtained directly by the constrained deterministic optimization methods such as the recursive quadratic programming method, since their mode shapes and frequencies can be accurately described using the analytical expression. As aforementioned, in most of the practical problems, only discrete sites are available. The optimal selection of locations becomes an integer programming problem which is usually much more difficult and costly to solve than a continuous optimization problem. In this case, the discrete variables problem can be converted as continuous variables sometimes and then solved by the deterministic optimization methods, too. For example, Sepulveda et al. [27, 28] presented a control-augmented structural synthesis methodology in which the actuator and sensor placement is treated in terms of $(0, 1)$ variables. The combinatorial aspects of the mixed $(0, 1)$-continuous variable design optimization problem are made tractable by combining approximation concepts with the branch and bound techniques. Sunar and Rao [29] solved the thermopiezoelectric sensor placement problem for the cantilever beamlike structures well by using the quasistatic thermopiezoelectricity equations. The advantages of the continuous optimization techniques are maturer compared with other methods, but these techniques need to use the gradient of the objective function. Thus, they are easy to fall into local optimum.

*4.2. Sequential Sensor Placement Method.* The sequential sensor placement (SSP) algorithm is a relative systematic and computationally efficient approach for obtaining a good sensor configuration although it cannot be guaranteed to be the optimal one [30]. The basic steps of the SSP algorithm for a fixed number of $N_0$ sensors are: the positions of $N_0$ sensors are computed sequentially by placing one sensor at a time in the structure at a position that results in the highest reduction in the objective function. Specifically, the position of the first sensor is chosen as one that gives the highest reduction in the objective function for one sensor. Given the optimal position of the first sensor, the position of the second sensor is chosen as one that gives the highest reduction in the objective function computed for two sensors with the position of the first sensor fixed at the optimal one already computed in the first step. Continuing in a similar fashion, given the positions of $(i-1)$ sensors in the structure computed in the previous $(i-1)$ steps, the position of the next $i$th sensor is obtained as one that gives the highest reduction in the objective function for $i$ sensors with the positions of the first $(i-1)$ sensors fixed at the optimal ones already obtained in the previous $(i-1)$ steps. This procedure is continued up to $N_0$ sensors. The aforementioned algorithm is termed the forward sequential sensor placement (FSSP) algorithm. The SSP approach can also be used in an inverse order, starting with $N_d$ sensors placed at all DOFs of the structure and removing successively one sensor at a time from the position that results in the smallest increase in the objective function. This algorithm is termed as the backward sequential sensor placement (BSSP).

As known, to start the SSP algorithm, the initial set of DOFs should be selected to cover the structure and areas of special interest. Carne and Dohmann [12] used intuition to determine the initial measurement set of DOFs based on experience and requirements of structural topology for the visualization of mode shapes. The original algorithm

(called MinMAC) proposed by Carne and Dohrmann can be regarded as a FSSP algorithm. However, Li et al. [31] found that the maximum off-diagonal term was not monotonically decreasing with the number of sensors. The reason for such increasing contradiction is that a newly included or excluded sensor may conflict with other previously selected sensors, or even with the original intuition set. Mathematically speaking, the row vector determined at this newly included sensor position has strong linear relationship with the previous whole sensor set. To alleviate the contra-decreasing, they developed a forward-backward combinational MinMAC algorithm called the extended MinMAC algorithm. The mainly difference resides in the stopping criteria. The extended MinMAC algorithm computes further to obtain a sensor set consisting a certain number of sensors larger than the required one(s) where the original MinMAC stops. Yi et al. [32] suggested selecting the initial sensor assignment by the QR-factorization of the structural mode shape matrix. The underlying idea is to find the most linear independent rows of the modal matrix to minimize the off-diagonal terms of the MAC matrix. Papadimitriou [22] evaluated the accuracy of the sensor configurations provided by the SSP algorithms by comparing the corresponding information entropy values with those computed by genetic algorithms (GAs). He found that the BSSP and FSSP methods for predicting the maximum information entropy values depended on the number of sensors and observable modes. The performance of the two methods give approximately the same predictions for the minimum information entropy and the combined results provided by the SSP methods are in all cases better than ones provided by the GAs. In addition, the application of the SSP methods for 1 and $N_d$ sensors may provide predictions of the upper and lower bounds of the information entropy for sensor configurations involving any number of sensors ranging from 1 to $N_d$; while the GAs provide predictions for an optimal configuration involving a fixed number of sensors only.

*4.3. Combinatorial Optimization Method.* Conventional gradient-based local optimization methods are unable to handle efficiently the multiple local optima and may present difficulties in estimating the global minimum. They lack reliability in dealing with the optimization problem since convergence to the global minimum is not guaranteed. In the recent years, combinatorial optimization methods based on the biological and physical analogue have been extensively used for the optimization of OSP problems due to its many advantages over the classical optimization techniques such as it is a blind search method and highly parallel. There are many interesting approaches to tackle such problems, but one of the most powerful heuristics is based on the GAs.

*(1) Genetic Algorithms.* The GAs try to imitate natural evolution by assigning a fitness value to each candidate solution of the problem and by applying the principle of survival of the fittest [33]. Their basic components are the representation of candidate solutions to the problem in a "genetic" form, the creation of an initial, usually random population of solutions, the establishment of a fitness function that rates each solution in the population, the application of genetic operators of crossover and mutation to produce new individuals from existing ones and finally the tuning of the algorithm parameters like population size and probabilities of performing the prementioned genetic operators [34].

In the original GA, the minimization variables must be encoded into bit strings (i.e., chromosomes with "0" and "1"). Therefore, the variables are discretized and the range of the discretization corresponds to some power of 2 (e.g., 1024). For the OSP problem, the optimization variables are the sensor locations, which could be either the spatial coordinate or the node index number. The mapping between the physical minimization variables and the chromosomes is the big difficulty in the application of GAs in addressing the OSP. The traditional coding method is the simple one dimension binary coding method which is very simple and intuitionistic [35, 36]. However, the number of sensors will be changed in the crossover and mutation, which is impractical and must be avoided [37]. In addition, the binary coding method requires increased string length and computational time especially in the large-scale structures where possible combination sensors are large. Roy and Chakraborty [38] and Chow et al. [39] suggested using the integer coding method to get over the problem. Huang et al. [40] proposed a kind of dual-structure coding approach to overcome the problem. In this coding method, the individual chromosome is composed of two rows, the upper row denotes the append code and the lower row represents the variable code. The crossover and mutation only operate on upper append code and the lower variable value of offspring is fixed which means the number of sensors could be unchanged. Different genetic operators, such as force mutation [41], filter operator [42], partially matched crossover (PMX) [43], have also been used in order to overcome these faults although the process of these particular genetic operators are complex and the computational efficiency is relatively low. Kang and Xu [44] adopted the partheno-genetic operators to sensor placement which only changed the order of the chromosome but never changed the sensor numbers. This is quite suitable to the OSP because the sensor number should be invariable during the optimization process. Considering the number of the DOF is enormous in the large-scale structures, the requirement for the large storage space is increased to save the optimal solutions, Liu et al. [45] presented a decimal two-dimension array coding approach instead of binary coding method to code the solutions. Numerical results showed that the dissipative storage space of the decimal two-dimension array coding method was minimal among them compared to the binary coding methods.

It should be noticed that, due to the nature of the GA's method, the number of iterations required for a guaranteed convergence is usually dependent on the size of the population and the searching space being used [46, 47]. For a given population size, the larger the searching space, the more the number of iterations needed to reach a converged solution with confidence. In other words, the higher confidence usually requires the number of iterations large enough, or multiple runs of the GAs process [48]. Thus, another

drawback of GAs is that it may spend much time when meeting a relative large size of the searching space which is constrained by the number of nodes on the FEM mesh excluding the constrained nodes and the vibration nodes of the selected modes. Some attempts have been made to overcome this fault. For example, Javadi et al. [49] presented a hybrid intelligent GA which was based on a combination of neural network and GA. Hwang and He [50] used simulated annealing and adaptive mechanism to insure the solution quality and to improve the convergence speed. In order to improve the convergence speed and avoid premature convergence, some improved GAs also been adopted to sensor placement problems, such as the generalized genetic algorithms (GGA) [51, 52] which were based on some famous modern biological theories such as the genetic theory by Morgan, the punctuated equilibrium theory by Eldridge and Gould, and the general system theory by Bertalanffy, so it is superior in biologics to the classical GA, the elite genetic algorithms (EGA) [53], the virus evolutionary genetic algorithm (VEGA) [54], the coevolutionary genetic algorithm (CGA) [55], and the multiobjective genetic algorithm (MGA) [56].

*(2) Simulated Annealing Algorithm.* Simulated annealing (SA) is an optimization procedure analogue to the physical process in thermodynamics, specifically to the way that liquids freeze and crystallize, or metals cool and anneal [57]. While the SA is a general purpose optimization procedure, its actual implementation in the OSP issue is defined by the evaluation function, perturbation operator, constraints, and parameter settings [58]. The perturbation operator works on each decision variable. In each iteration of the algorithm each variable is randomly selected for perturbation. The operator then uses direction cosines for obtaining a random direction. Spatial constraints prevent solutions from leaving sensor regions. The parameters in the SA algorithm determine its performance both in speed and solution quality. Generally the better performance is obtained with highly random initial search, which means high temperature, followed by slow cooling.

Chattopadhyay and Seeley [59] developed a multiobjective optimization procedure based on a SA technique to include both discrete and continuous design variables for the design of intelligent structures. A numerical example using a cantilever box beam was carried out and the results demonstrated the SA could reduce computational effort when compared with the previous nonlinear programming technique. Chiu and Lin [60] defined the sensor placement issue as a min-max mathematical optimization model provided that either discrimination, or distance error, was the objective under cost and coverage constraints. Numerical experimental results showed that the SA algorithm could find the OSP under the minimum cost limitation and outperform the brute force approach no matter in the case of smaller or large sensor fields. The SA algorithm has a strong ability for finding the local optimal result, thus, it can be introduced into other method to avoid the problem of local minima. For example, Wang et al. [61] proposed a distributed particle swarm optimization and simulated annealing for the energy-efficient coverage in wireless sensor networks; Hwang and He [50] used the simulated annealing and adaptive mechanism to insure the solution quality and to improve the convergence speed of GA; Gou and Cui [62] developed a mathematical model of minimizing total energy for the sensor/actuator placement of simple beam active control based on the GA and SA algorithms. Experimental results showed that the hybrid algorithm had high efficiency and global convergence for the sensor/actuator placement problem.

*(3) Particle Swarm Optimization Algorithm.* Particle swarm optimization (PSO) is a population based stochastic search technique introduced by Eberhart and Kennedy in 1995 [63], inspired by the social behavior of bird flocking or fish schooling. It works in the same way as the GAs and other evolutionary algorithms. The PSO algorithm is initialized with a population of random candidate solutions, conceptualized as particles. Each particle is assigned to a randomized velocity and is iteratively moved through the problem space. It is attracted towards the location of the best fitness achieved so far by the particle itself and by the location of the best fitness achieved so far across the whole population (global version of the algorithm). The PSO algorithm has better performance than early intelligent algorithms on calculation speed and memory occupation, and has less parameter and is easier to realize [64, 65].

Many researchers have worked on improving its performance in various ways and developed many interesting methods for the OSP problems. For example, the computational time required to evaluate the objective function, coupled with the dimensionality problem renders the standard PSO impractical for real-time planning of large-sized sensor configurations. Even for configurations of manageable size, the PSO algorithm takes too long to converge. Ngatchou et al. [66] proposed a kind of modified PSO algorithm called Sequential-PSO (S-PSO) that shortened the computational run-time and yielded better convergence performance. The S-PSO iteratively optimizes the objective function over randomly selected subspaces of the parameter search space instead of the entire parameter space at once as in the standard PSO. This approach is advantageous since fewer particles are needed to solve the lower-dimension subproblems. Zhang and Vachtsevanos [67] combined the PSO algorithm with a heuristic search algorithm to develop a methodology for deciding the type, number, and location of sensors required to monitor accurately and robustly fault indications or signatures in a critical military or industrial system. In this method, the number of sensors at each location is rounded to integer numbers since the PSO method is originally designed for continuous variables optimization. Inspired by the binary PSO, Rapaić et al. [68] presented a novel modification of the PSO algorithm, called the discrete PSO (DSPO) to the OSP problem. Several examples demonstrated that the DSPO could be used to solve any combinatorial and integer programming problem. Rao and Anandakumar [69] put forward to a hybrid PSO algorithm by combining a self-configurable PSO with the Nelder-Mead algorithm for

arriving at an optimal number as well as locations of sensors on civil engineering structures for the structural system identification and health monitoring. Numerical studies indicated that the proposed hybrid PSO algorithm generated sensor configurations superior to the conventional iterative information-based approaches which had been popularly used for large structures. Further, the proposed hybrid PSO algorithm exhibits superior convergence characteristics when compared to other PSO counterparts.

*(4) Monkey Algorithm.* The monkey algorithm (MA) was firstly designed by Zhao and Tang [70] from the inspiration of mountain-climbing processes of monkeys. It assumes that there are many mountains in a given field (i.e., in the feasible space of the optimization problem), and in order to find the highest mountaintop (i.e., find the maximal value of the objective function), monkeys need to climb up from their respective positions. The algorithm mainly consists of climb process, watch-jump process, and somersault process in which the climb process is employed to search the local optimal solution, the watch-jump process to look for other points whose objective values exceed those of the current solutions so as to accelerate the monkeys' search courses, and the somersault process to make the monkeys transfer to new search domains rapidly.

To implement the MA in the OSP problem, it is necessary to make some changes since this algorithm is originally designed to solve global numerical optimization problems with continuous variables. Yi et al. [71] adopted the integer coding method to represent the solution and employed the hamming distance operator and the stochastic perturbation mechanism to improve the local and global search capabilities of the MA. A computational case of a high rise building was implemented to demonstrate the effectiveness of the improved MA. Results showed that the modifications in the MA could improve the convergence of the algorithm and the algorithm was effective in solving the OSP problem.

*(5) Ant Colony Optimization Algorithm.* The ant colony optimization (ACO) algorithm uses a colony of artificial ants that behave as cooperative agents in a mathematic space were they are allowed to search and reinforce pathways (solutions) in order to find the optimal ones [72]. The problem is represented by the graph and the ants walk on the graph to construct solutions. The solution is given by a path in the graph. After the initialization of the pheromone trails, ants construct feasible solutions, starting from random nodes, then the pheromone trails are updated. At each step ants compute a set of feasible movements and select the best one to carry out the rest of the tour.

Having established that the ACO algorithm properly worked on the TSP problem, some modification was required before it could be applied to the OSP problem. The first important difference is that one is looking for an optimum subset of the candidate sensor locations and this means that a tour does not visit all locations. The second major difference is that the ants are truly blind. The sensor distributions can only be evaluated when all the members of the tour are known, and there is no analog of visibility [73].

## 5. Comparisons of Different Sensor Placement Methods

Due to the existence of a wide number of optimum criteria and method, it is necessary to produce several sensor arrays using different formulations, and to compare the obtained configurations according to some performance indicators. Their difference and consistency are essential to the basis of theoretical considerations and to the development of other effective sensor placement methods. This is not a marginal question in the OSP, especially if a large number of candidate positions have to be examined. On the one hand, it will give some guidance in engineering applications; on the other hand, it can increase the confidence in the OSP methodology.

Meo and Zumpano [74] analyzed the difference between several criteria for the optimal sensor positioning on the Nottingham Bridge. Li et al. [31] carried out the comparison of nine sensor placement methods on a ladder structure. Marano et al. [75] performed comparisons among six different optimum criteria with reference to two examples of broadcasting antennas. A most significant and commonly cited OSP approaches called effective independence (EfI) method was developed by Kammer in 1991 [76–78]. The EfI method tends to maximize the trace and determinant and minimize the condition number of the Fisher information matrix corresponding to the target modal partitions. Li et al. [79, 80] found this strategy had some connections with other well-developed method, such as the modal kinetic energy (MKE) method [81], QR decomposition (QRD) method [82], and MinMAC method [12].

*(1) Connection between EfI and MKE.* For the case of identity equivalent mass matrix: the EfI requires iteration computations, but the MKE does not. In the following iterations of EfI, it redistributes the MKE into the retaining DOFs and recomputed their MKE index for the reduced system using the reorthonormalized mode shapes. Thus, the EfI is an iterated version of MKE with the reorthonormalized mode shapes.

For the case of nonidentity equivalent mass matrix: the MKE assigns more weights for the DOFs with the large mass concentration by left multiplying the EfI indices at the first iteration with the mass matrix. When the mass is only slight unevenly distributed throughout the whole structure, which is common in engineering practice, the relative sequence of the MKE indices will agree with that of the EfI very well. In this sense, the MKE can be regarded as a weighted EfI without iterations for structures with general mass distributions.

*(2) Relationship between EfI and QRD.* The QRD method selects sensor positions according to their row norms and linear independency in the row space, whereas the EfI computes the QR decomposition in the column space of the modal matrix. Both the EfI and QRD give similar results when the required number of sensors equals to the number of modes. When more sensors are needed, they will give different sensor combinations. Moreover, the difference between the EfI and QRD results from the reorthogonalization of the column

space or the row space of the original full modal matrix during the EfI or QRD computation, respectively.

*(3) Connection between EfI and MinMAC.* The EfI method is equivalent to the MinMAC algorithm in the global sense, but there are minor differences between them. The MinMAC algorithm tries to minimize its maximum off-diagonal terms by tracking every off-diagonal terms of the MAC matrix, while the EfI method minimizes all the off-diagonal terms in the sense of the whole MAC matrix. Namely, the minimization of the off-diagonal terms by the iterative EfI method in the norm sense could not always leads to the decreasing of the maximum off-diagonal terms defined to be minimized by the MinMAC algorithm. Secondly, the EfI method includes an implicit step of reorthonormalization during its iterations. This implicit reorthonormalization step results in small deviation from the directions of the reduced mode shapes, while the MinMAC algorithm sticks stubbornly to the directions of the reduced mode shapes.

## 6. Summary

The development and implementation of a large-scale SHM system using the rational number of sensors and positioning them appropriately on a structure may provide direct and full information for structural inspection and maintenance, and would most benefit structural owners with regard to asset management. The OSP problem is still a challenge at present, and needs well-coordinated interdisciplinary researches for full adaptation of innovative technologies developed in other disciplines to applications in the civil engineering community. Future methods are likely to be specific to the SHM methodology employed and should be developed alongside them. Wherever possible, successful numerical simulation should be followed by experimental study. When performing the sensor placement optimization for the SHM, it is beneficial to extend the experimental study to cover the results of the SHM, especially where comparisons can be made between the placement algorithms and exhaustive search results.

## Acknowledgments

This research work was jointly supported by the Science Fund for Creative Research Groups of the National Natural Science Foundation of China (Grant no. 51121005), the National Natural Science Foundation of China (Grant no. 51178083, 51222806), the Program for New Century Excellent Talents in University (Grant no. NCET-10-0287), and the Natural Science Foundation of Liaoning Province of China (Grant no. 201102030).

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Hindawi Publishing Corporation
International Journal of Distributed Sensor Networks
Volume 2012, Article ID 459590, 6 pages
doi:10.1155/2012/459590

*Research Article*

# Non-Line-of-Sight Beacon Identification for Sensor Localization

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Received 3 June 2012; Accepted 11 July 2012

Academic Editor: Ting-Hua Yi

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In recent years, sensors are often deployed to sense the target environment. Besides the sensed environmental information, sensor's position information itself is very important in most cases. With the increasing number of sensors, automatic sensor localization becomes necessary. For accurately localizing sensors, line of sight between nodes and beacons is often required. However, in many cases, it is very difficult to ensure that every beacon has the line of sight to a target sensor node. In this paper, a non-line-of-sight beacon identification approach is proposed. With which, the beacons that have no line of sight to a target sensor can be found out and then ruled out, thus the final localization result can be rectified and the accuracy can be improved. Simulations are conducted, and results show that non-line-of-sight beacons can be identified effectively. The proposed approach will be helpful in the real sensor localization in sensor networks, especially for the systems that beacons are sparsely deployed.

## 1. Introduction

Nowadays, wireless sensor networks are often applied to build sensing systems for Structural Health Monitoring (SHM). For instance, Yu et al. developed a wireless MEMS inclination sensor system for monitoring large-scale hook structures [1]. There is no doubt that embedded sensor network systems are always coupled to the physical world. Besides the sensed information, sensor's position information itself is very important for Structural Health Monitoring. At one hand, people try to find the best, and optimal positions for deploying sensors; so that fewer sensors are needed but still can assure sound monitoring result [2–4]. While at the other hand, with the decreasing of sensors' prices nowadays, people tend to deploy more sensors so that they can deploy them more freely [5]. In this case, sensors are often localized after the deployment rather than before the deployment. However, with the increasing number of the deployed sensors, manual localization is not any longer practical. Automatic sensor localization becomes necessary and is needed to be developed.

In localization systems, sensors can be either localized using the ranging-free approach using only the proximity information such as Centroid algorithm [6], DV-Hop propagation method [7], and so on, or using the ranging-based approach such as lateration [8] in which sensors are localized using the distance-based scheme using several fixed beacons. A beacon means a node whose position is already known so that it can be used as the reference point. Triangle rule is applied to determine the sensor position. However, this approach requires precise distances between sensors and beacons. Hence, line of sight between nodes and beacons is necessary to ensure the localization correctness. However, in many cases, it is very difficult to ensure that every beacon has the line of sight to a target sensor node, which will consequently make the distances overestimated for those

beacons that have nonline of sight to the sensor node. Here, nonline of sight means that the receiver will receive the signal after being reflected, refracted, or diffracted instead of receiving the direct path signal. Hence, if a beacon has no line of sight, the distance between the beacon and a receiver will become much larger than its direct path. In this case, the final localization results will definitely been distorted. In this paper, a non-line-of-sight beacon identification approach is proposed. With which, the beacons who have no line of sight to a target sensor can be found out and then ruled out, thus the final result can be rectified and the accuracy can be improved. The proposed method will be helpful in the real sensor localization in sensor networks, especially for the system that beacons are sparsely deployed.

## 2. Distance-Based Sensor Localization

For localization systems, generally speaking, two kinds of schemes can be used, that is, range-based localization scheme which uses absolute point-to-point distance estimates or angle estimates for calculating location and range-free localization scheme which uses only the connectivity and proximity. Usually, lateration is often applied to locate a node with distance estimates from a sensor node to several fixed beacons. Figure 1 shows a two-dimensional lateration with three beacons.

For a three-dimensional localization problem, four beacons not in a same plane are usually required, as shown in Figure 2, and the following equations stand:

$$(x - x_i)^2 + (y - y_i)^2 + (z - z_i)^2 = d_i^2 \quad i = 1, 2, 3, 4, \tag{1}$$

where $(x, y, z)$ are the coordinates of an unknown node; $(x_i, y_i, z_i)$ are the coordinates of the $i$th beacon B$i$; and $d_i$ is the distance between the beacon and a unknown node. Equation (1) can be rewritten to be

$$xx_i + yy_i + zz_i = \frac{r^2 + r_i^2 - d_i^2}{2}, \quad i = 1, 2, 3, 4, \tag{2}$$

where $r$ and $r_i$ represent the distance of the unknown node and the $i$th beacon B$i$ to the origin of coordinates, respectively, which can be expressed as

$$\begin{aligned} r^2 &= x^2 + y^2 + z^2, \\ r_i^2 &= x_i^2 + y_i^2 + z_i^2. \end{aligned} \tag{3}$$

The effect of beacon deployment on the localization was studied, and a projection method was proposed and applied to enable the beacons to be deployed at one plane [5, 9]. The projection method can be shown in Figure 3. With which, only three beacons are required to locate a node in a three-dimensional space. Usually localizing a node in a three-dimensional space requires four beacons which are not in a same plane. However, if the beacons are deployed at the same plane, there will be two possible solutions (positions), which lie in two sides of the beacon plane. Therefore, if we can determine at first which side the node will be at the beacon plane, then the node position can be uniquely

Figure 1: Two-dimensional lateration.

Figure 2: Three-dimensional localization.

and correctly determined [5]. Our simulations also showed that deploying the beacons around the node will improve its localization accuracy. Expanding the beacon span can also decrease the localization error to some extent. The most important advantage is that the projection method can allow the beacons to be deployed at one plane while the localization accuracy is not nearly deteriorated.

## 3. Localization with Non-Line-of-Sight Beacon Included

In sensor localization, sensing directivity of both receiver and transmitter is another important issue. It is always an important effect on the localization accuracy. This is proved by the MIT Cricket experiments that the distance measurement accuracy is good only when the receiver has a very small angle to the transmitter [10]. Otherwise, error increases significantly. This is due to the fact that the gain of both of the receiver and transmitter dramatically drops when the angle increases. For this consideration, it seems that in a beacon grid as shown in Figure 4, it is better to use only the nearest beacon which covers the target node, because they can often guarantee relative small angles to the receiver which in turn provides good distance estimates.

Figure 3: Projection method.

Figure 4: The nearest beacon in a beacon grid to a target node.

With the projection method, three beacons are enough to build up a three-dimensional lateration as shown in Figure 5. If we know the distances from the node to the three beacons, then the possible position will be at the crossing point of the three-spherical surfaces. After we project this three-dimensional lateration into the beacon plane (the plane built by the three beacons), then it can be degraded into a two-lateration problem. In order to identify the non-line-of-sight beacons, the beacon grid should be deployed into a rectangle grid. Thus for the four nearest beacons to a node, say, B1, B2, B3, and B4, they should be on the vertices of a rectangle.

Figure 6 shows the projected figure with beacons B1, B2, and B3. From the figure, we can see that every two circles will generate a line according to their common points. Three beacons will totally generate 3 of such lines, that is, line $l_{12}$, $l_{23}$, and $l_{13}$. These 3 lines will definitely pass the point $c$ which is just the projected point of the node. If beacon B3 has no line of sight resulting in overestimating the distance, the projected radius to beacon B3 will be enlarged. In Figure 6, the dashed line circle is the biased circle indicating the enlarged measured distance. Since circle B1 and circle B2 are not changed, $l_{12}$ remains unchanged, while $l_{23}$ and $l_{13}$ are moved to the $l'_{23}$ and $l'_{13}$, the crossing point $c$ is also moved to $c'$. In fact, as we know, in a two-dimensional plane, two lines are enough to determine a point. To determine the projected point $c$, we only need to examine the lines $l_{12}$ and $l_{23}$ for their convenience. Because line $l_{12}$ is kept unchanged, the possible projected point $c$ can only move along the line $l_{12}$. In this

Figure 5: Three-dimensional lateration.

Figure 6: Projected figure with beacon B3 has nonline of sight.

case, if beacon B3 has no line of sight, then only $y$ value of the node will be changed while there will be no effect on its $x$ value. Vice versa, the nonline of sight of the beacon B1 will only affect the node's $x$ value but will have no effect on its $y$ value. From the mathematical view, since beacons B1 and B2 are at the $x$-axis and the line B2B3 is parallel to $y$-axis, node's position can be found to be

$$x = \frac{r_2^2 - r_1^2 - d_2^2 + d_1^2}{2(x_2 - x_1)},$$
$$y = \frac{r_3^2 - r_2^2 - d_3^2 + d_2^2}{2(y_3 - y_2)}. \tag{4}$$

Suppose the measurement error of $d_3$ is $e$, then the $y$ error of the localization result is

$$\varepsilon_y = -\frac{d_3 \cdot e}{y_3 - y_2} - \frac{e^2}{2(y_3 - y_2)}. \tag{5}$$

Figure 7: Localization with identifying non-line-of-sight beacon.

If the distance measurement error $e$ is much smaller than $d_3$, then (5) can be approximately estimated to be

$$\varepsilon_y \approx c_0 \cdot e, \tag{6}$$

where

$$c_0 = -\frac{d_3}{y_3 - y_2}. \tag{7}$$

## 4. Non-Line-of-Sight Beacon Identification

As shown in Figure 7, four beacons, deployed at the vertices of 4 angles of a rectangle, are used to locate a node. These four beacons can combine four triangles. Localization can be executed by every triangle with the projection method. Without losing generality, if beacon B3 is assumed to have no line of sight, it will not affect the triangle B1B2B4, but will affect all the other triangles. With every triangle, a position estimate can be obtained. So totally 4 position estimates can be gotten. If we examine the position domain, we can determine the non-line-of-sight beacons according to their position distribution.

In Figure 7(b), the grey area indicates the possible right position with line of sight. The size of the grey area is determined by the reasonable localization error. When B3 is blocked, which results in a large positive measurement error, the position result P134 determined by triangle B1B3B4 will have a big $y$ error and P123 will have a big $x$ error, while P124 will fall into the grey area. With this kind of position distribution, we can determine exactly which beacon has non line of-sight. Here, we neglect triangle B2B3B4 because both of its $x$ and $y$ errors are very large. As the nonline of sight of beacon B3 only leads to big error on $y$ error of P134 and $x$ error of P123, we get rid of these two values in order to get a correct answer. Thus, the node's position can be rectified to be as

$$x = \frac{x_{P124} + x_{P134}}{2}, \qquad y = \frac{y_{P124} + y_{P123}}{2}. \tag{8}$$

Figure 8: Simulation model.

On the whole, the whole approach contains next 4 steps.

(i) Determine four beacons which are at the four vertices of a rectangle which covers a target node.

(ii) Separate the four beacons into four groups with each containing three beacons and executing localization procedure.

(iii) Study the position distribution and determine the non-line-of-sight beacon.

(iv) Rule out the wrong effect and rectify the result.

## 5. Simulations

Beacons B1, B2, B3, and B4 are deployed as shown in the Figure 8. Assume that the measurement-distance estimate to beacon B3 suffers an extra positive 50 cm measurement error due to the lack of the line of sight.

Figures 9, 10, and 11 show the localization errors for the $x$-, $y$-, and $z$-direction when the node is in plane $z = 1$ m. It is found that when beacon B3 has a big positive measurement error, in $x$-direction there is no effect, but in $y$- and $z$-direction, this kind of measurement error causes a very big

Figure 9: Localization errors at $x$-direction.

Figure 10: Localization errors at $y$-direction.

Figure 11: Localization errors at $z$-direction.

Figure 12: Contour of $x$ error.

localization error. We also find that error increases when the actual distance from the node to the beacon B3 increases. On the contrary, even when we do not know which beacon has no line of sight, according to the localization results, we can determine that the beacon B3 actually has no line of sight and therefore should be avoided in the sensor localization.

In order to clearly examine the $x$ error, Figure 12 gives its contour figure. In this figure, polygons stand for the error contour at $\pm 3$ cm. Projected beacon positions are also marked in order to consider the effect of the node position on the localization errors. This figure shows that the error is very small if the projected node position is within the beacon span, nearly all localized points have absolute error smaller than 3 cm. However, it also shows that localization error will be slightly larger if the projected node position lies in another area. It shows that for accurate localization, it is better to use the nearest beacon whose project area can cover the project position of a node.

## 6. Conclusions

This paper proposes an approach to identify non-line-of-sight beacon and rectify the localization result. With which, beacons who have no line of sight can be found out effectively by studying localization results distribution from several separated groups, so that the relevant overestimated distances can be ruled out. Thus the final result can be rectified and accuracy can be improved. Simulations show the correctness of the proposed approach.

## Acknowledgments

The authors would like to acknowledge the support from the project funded by the Priority Academic Program Development of Jiangsu Higher Education Institutions (PAPD), National Key Technology Research and Development Program of the Ministry of Science and Technology of the

people's Republic of China (Grant no. 2011BAK02B03), the National Natural Science Foundation of China (Grant no. 50908046), and doctoral fund of Ministry of Education of the people's Republic of China (Grant no. 6205000009).

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