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MONITORING AND ANALYSIS OF A CABLE-STAYED BRIDGE MODEL USING WIRELESS SENSOR NETWORK

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ABSTRACT

Structurla health monitoring (SHM) is a revolutionary method of determing the safty situation of structures. In the conventional SHM system, the expensive cost for purchase and installation of the SHM system components, such as sensors, data loggers, computers, and connecting cables, is a big obstruction. With the quick development of seonsor technology, Micro-Electro-Mechanical Systems (MEMS), modern network technique, and wireless transmission technique, a new SHM method which is based on wireless sensor network is applicable for obtaining structural health and performance information. This paper studies the dynamic characteristics of a laboratory cable-stayed bridge. At first, the state-of-art in the smart wireless sensor technology is reviewed. The subsystems of a smart wireless sensor are discussed, and then available wireless sensor boards developed by the present authors are applied to SHM of a cable-stayed bridge model in this study. A modified Ibrahim time-domain (ITD) method is adopted for the extraction of dynamic characteristics of the bridge model from experimental measurement. At last, the relationships between the undamped natural frequencies and mode shapes of the cable-stayed bridge and those of the single pylon and the deck beam are analyzed. Out results which are validated by a wired data acquisition system, demonstrate the bridge model is characterized by the presence of many closely spaced, coupled modes, and indicate the feasibility of wireless sensing technology in civil engineering structure health monitoring application.

KEYWORDS

Structural health monitoring, wireless sensor network, cable-stayed bridge, Ibrahim time-domain method.

INTRODUCTION

Long-span cable-stayed bridges have been extensively employed in the last four decades due to their inherent mechanical effectiveness, economical design, and esthetic appearance. The structure of the cable-stayed bridge is complex, and the cables are flexible, lightweight, low damping, etc. All these features make the mode of the cable-stayed bridge prone to wind-, rain/wind-, earthquake-, and traffic-induced vibrations. Determination of the dynamic behavior of these structures under time-varying loads has become a practical engineering issue of worthy attention.

Conducting dynamic testing is regarded as one of the most reliable experimental methods available for assessing actual dynamic properties of these complex bridge structures (Liu *et al.* 2008). Such tests serve to complement and enhance the development of analytical techniques and models that are integrated into analysis of the structure over its operational life. During the past two decades, many researchers have conducted full-scale on-site dynamic tests on cable-stayed bridges. Typical examples of full-scale dynamic tests on bridges are provided in the References (e.g. Ko *et al.* 2002; Ozkan *et al.* 2001; He *et al.* 2005; Wenzel and Pichler 2005). These dynamical tests use wired data acquisition systems to collect structural vibration data from various locations in the structure (induced by ambient sources e.g. moving vehicles, winds) for analysis. However, due to available technology, wired data acquisition systems have been centralized in terms of collecting and analyzing information. Thus drawbacks includes expensive, time-consuming sensor installation, signal degradation, and data flooding.

Emerging wireless technologies have the promise of overcoming these deficiencies. This technology allows the engineering community to measure and evaluate ambient structural vibrations quickly and accurately (e.g. Spencer *et al.* 2004 and Rice *et al.* 2001). A wireless sensor network based on data acquisition system promises enormous benefits such as ease and flexibility of deployment in addition to low maintenance and deployment costs. Recent years have witnessed the development of a number of wireless smart sensors based on the MEMS technology, including Berkeley motes manufactured by Intel and Crossbow. The Berkeley Mote smart sensor, with its embedded microprocessor and wireless communication capability, has emerged as an important new open hardware/software platform for structural dynamical testing and SHM. The use of wireless communications in lieu of wires within a structural monitoring system was initially proposed by Straser and Kiremidjian (1998), as a means of reducing installation costs in large-scale civil structures. In addition, their work illustrated the freedom of a wireless system infrastructure provides rapid and reconfigurable installations. Recently, Lynch *et al.* (2001) has extended their work to include computational microcontrollers in the hardware design of wireless sensors so that various system identification and damage detection algorithms can be embedded for local execution within the sensor. To date, a handful of bridges and buildings have been instrumented with wireless monitoring systems including the Alamosa Canyon Bridge (New Mexico), Geumdang Bridge (Korea), WuYuan Bridge (China), Voigt Bridge (California) and a historic theater in Detroit, Michigan (Cho *et al.* 2008). These extensive field studies attest to the accuracy and reliability of wireless sensors in traditional structural monitoring applications.

Modal analysis is essential to study the dynamic behavior of cable-stayed bridge. In modal analysis of a cable-stayed bridge, the aspects of three dimensionality, multi-modal contribution, and modal coupling are of great importance (Abdel, 1991). Challenges in the modal parameter identification technique applied to a cable-stayed bridge come from its nature as a large and continuous structure, which makes it difficult to be represented with a lumped-mass model. Accordingly, efficient system identification techniques should be developed to provide accurate information that accommodates observation of such aspects.

The Ibrahim time domain method, known as the ITD method (Ibrahim and Mikulcik 1997), uses Impulse Response Function (IRF) data to identify modal parameters. It constructs an eigenvalue problem from the IRF data and solves the problem to derive the natural frequencies, damping loss factors and modal constants. This method is useful when the input data is unknown, in particular in structural dynamical testing under ambient loads, and thus chosen for dynamical analysis in this study. However, this method, when originally proposed, constructs the eigenvalue problem using displacement, velocity IRF data. As a result, numerical integration is needed to obtain displacement and velocity time histories from acceleration measurement at each response point. This paper improves the ITD method to rely on sampling measured data in acceleration form only.

The focus of the work described in this paper is to assess twofold: (1) the capabilities of a low-cost wireless structural testing system to rapidly collect the dynamic responses of civil infrastructures, and (2) experimental determination of the dynamic properties of a laboratory cable-stayed bridge model based on modal analysis using a modified ITD approach. The wireless prototype sensor adopted in this paper consists of a self-customized sensor board and a commercial IRIS mote node in direct communication with a high-performance data repository where data is stored and analyzed. To extract the bridge modal characteristics, an improved ITD method is developed to directly use the real time acceleration signals from wireless sensors and autonomously identify the dynamic properties of the bridge. The paper concludes with a discussion on the results obtained from the wireless testing system compared with those from a wired counterpart.

WIRELESS SENSORS PLATFORM

In this section, we propose the hardware and software design of the SHM system based on WSN. The system architecture is presented first. Then we discuss the off-the shelf sensor mote and the extension sensor board design. At last, software design is mentioned.

System Architecture

The wireless SHM system is composed of a number of sensor nodes and a base station, as shown in Figure 1,which is responsible for network coordination and data collection. These nodes are usually small in size with limited processing power, limited memory and limited energy source. Each sensor node consists of a basic “mote” node and a sensor board, which acquires vibration data of the structure through an accelerometer, and then transmits wirelessly the acquired values. Figure 2 presents an overview of the sensor node hardware as a block diagram. The coordinator node is connected to the MIB-520 USB programming board, thus acting as a gateway between the WSN and the central server.

Figure 1. Wireless Sensor Network

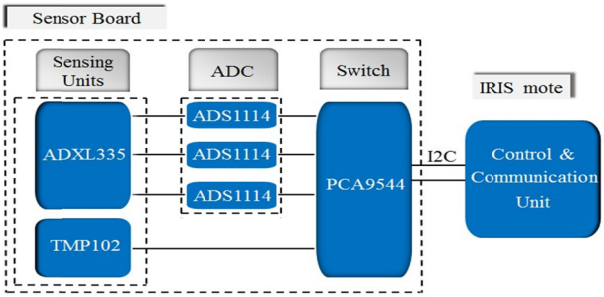


Figure 2. Hardware Block diagram

The Sensor Mote

For each node, a mote with a microcontroller provides local processing and storage capability and a low-power radio communication. An off-the-shelf sensor mote, the IRIS wireless module, is chosen because it has a good tradeoff between processing and communication capability. The IRIS mote and the properties of its different components are shown in Figure 3. Compared with previous generations of MICA Motes (Crossbow), IRIS provides users with high-level functional integration up to three times improved radio range and twice the program memory. The IRIS mote powered by two AA batteries, is used for enabling low-power wireless sensor networks. It has a 51-pin extension for the connection of analog inputs and I/O options. These I/O options include I2C, SPI and UARTs.

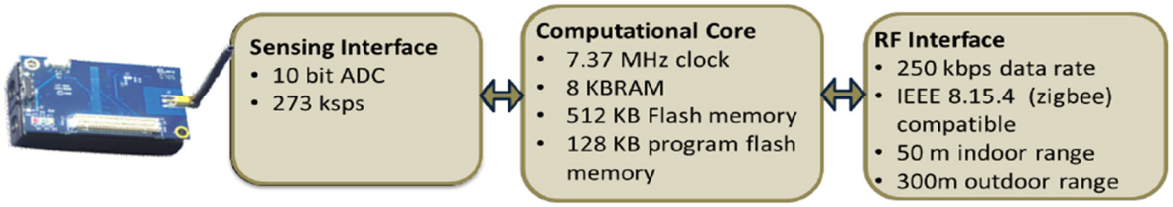


Figure 3. A picture of the IRIS mote and it's properties

Extension Sensor Board Design

An extension sensor board, as shown in Figure 4, is designed for SHM applications, which could be connected with the IRIS mote using the 51-pin expansion connector. The sensor board integrates accelerometer, strain interface, and temperature. For the measurement of accelerations, a commercially available MEMS type accelerometer sensor ADXL335 is used to monitor the accelerations in three directions. The ADXL335 is a small, thin, low power, complete 3-axis accelerometer with signal conditioned voltage outputs. The measurements can be both static acceleration (e.g. gravity) and dynamic (e.g. shock or vibration). It measures acceleration with a minimum full-scale range of ±3g. Capacitors of 0.1µF are added at three pins to implement low-pass filtering with a cutoff frequency (or analog filter bandwidth) of 50 Hz for anti-aliasing and noise reduction. The analog signal is fed to a 16-bit analog-to-digital converter (ADC), called ADS1114, for each of the three axes (channels). TMP102 is a Low Power Digital Temperature Sensor. A 4-channel switch, PCA9544, is used to control the three ADCs and TMP102 by the I2C-buses. Also, the switch is controlled by I2C interface from the mote.

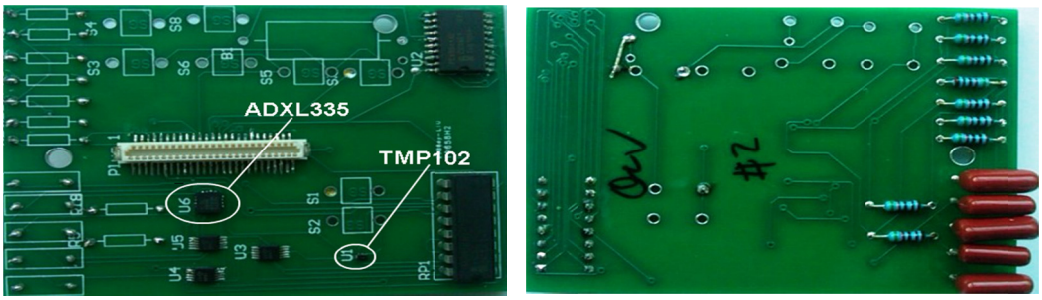


Figure 4. Front and back of the extension sensor board

Software Architecture

A WSN based structural monitoring system is a distributed sensing system. The core component of the software system is the one running on the sensor node. In order to support the node hardware we developed a software driver using NesC language for TinyOS. TinyOS is an operating system developed by UC Berkeley, which provides the software infrastructure for the sensor networks. NesC is an extension to the C programming language, for cooperating applications on TinyOS. The driving software could be divided into acquisition module, wireless communication module, synchronization module. The Accelsample application (TinyOS version 2.1) installed to the IRIS mote is responsible for time synchronization, sampling the data during structure health monitoring, and transmitting the data to the base station. The software architecture of the sensor node uses new components integrated into the TinyOS infrastructure to satisfy the requirements discussed above.

Time Synchronization

The time synchronization in the system makes use of the TinyOS implementation of Flooding Time Synchronization Protocol (FTSP). A detailed overview of the communication protocol used for time synchronization and reliable data transfer can be found in (Elson 2003). Time synchronization can operate in one of 2 motes. To achieve better Synchronization performance, the FTSP is chosen to realize time synchronization in sampling software. Figure 5 shows the time error testing for 800 data transmitting packages. The largest error is 57.78 μs, and the average error is 33.82 μs. This time precision satisfies most structural health monitoring requirements. This communication protocol has also been shown immune to data loss.

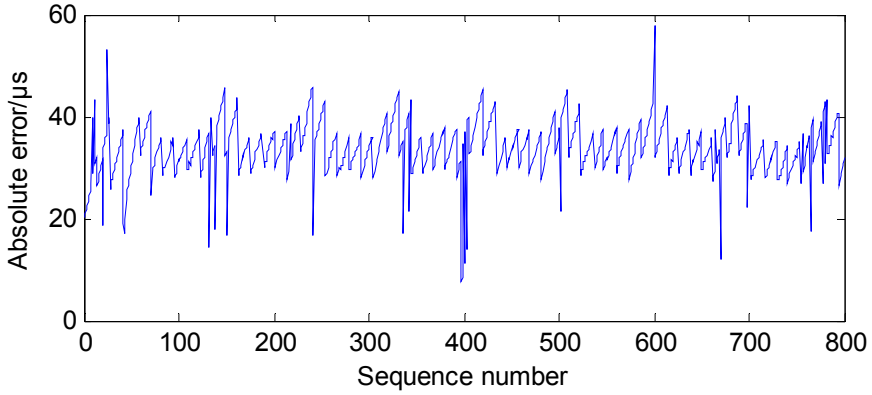


Figure 5. Time Synchronization error

MODIFIED ITD METHOD

In this paper, the ITD approach is improved by directly using the real time acceleration signals. The deficiency of the amount of data needed is remedied by sampling the displacement data with a selected time delay. The free response of a structure means either the free decays measured from random execution of the structure or the impulse response obtained from the inverse of a frequency response function (FRF). In the following, we discuss this modified time domain method which is efficient for modal analysis.

Constructing Mathematical Model

The free displacement response of a *N-DOF* viscous damping system can be represented with the following formula according to the principle of mode superposition.

$$\left[X\left(t_i\right)\right]=\sum_{i=1}^{n}\left(\phi_i e^{\lambda_i t_i}+\phi_i^{*} e^{\lambda_i^{*} t_i}\right) \tag{1}$$

Eq. 1 is a conjugate relationship and can be simplified into a new form:

$$\left[X\left(t_i\right)\right]=\sum_{i=1}^{2n}\Phi_i e^{\lambda_i t_i} \tag{2}$$

Eq. 2 is based on the displacement $X(t_i)$. The acceleration $\ddot{X}\left(t_i\right)$ can be derived from twice derivative of displacement data in the following form:

$$\left[\ddot{X}(t_i)\right]=\frac{d^2[X(t_i)]}{dt_i^{\,2}}=\sum_{i=1}^{2n}\Phi_i\lambda_i^2 e^{\lambda_i t_i} \tag{3}$$

When the measuring point is N, the sampled discrete points is M, Eq. 3 can be expressed as:

$$\begin{bmatrix} \ddot{X}_{1,1} & \ddot{X}_{1,2} & \cdots & \ddot{X}_{1,M} \\ \ddot{X}_{2,1} & \ddot{X}_{2,2} & \cdots & \ddot{X}_{2,M} \\ \vdots & \vdots & \ddots & \vdots \\ \ddot{X}_{N,1} & \ddot{X}_{N,2} & \cdots & \ddot{X}_{N,M} \end{bmatrix} = \begin{bmatrix} \phi_{1,1} & \phi_{1,1}^* & \cdots & \phi_{1,N}^* \\ \phi_{2,1} & \phi_{2,1}^* & \cdots & \phi_{2,N}^* \\ \vdots & \vdots & \ddots & \vdots \\ \phi_{N,1} & \phi_{N,1}^* & \cdots & \phi_{N,N}^* \end{bmatrix} * \begin{bmatrix} \lambda_1^2 & 0 & \cdots & 0 \\ 0 & (\lambda_1^*)^2 & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & (\lambda_N^*)^2 \end{bmatrix} \begin{bmatrix} e^{\lambda_1 t_1} & e^{\lambda_1^* t_2} & \cdots & e^{\lambda_1 t_M} \\ e^{\lambda_1^* t_1} & e^{\lambda_1^* t_2} & \cdots & e^{\lambda_1^* t_M} \\ \vdots & \vdots & \ddots & \vdots \\ e^{\lambda_N^* t_1} & e^{\lambda_N^* t_2} & \cdots & e^{\lambda_N^* t_M} \end{bmatrix} \tag{4}$$

It can be simplified as following formula

$$\left[\ddot{X}\right]_{N,M}=\hat{\Phi}_{N,2N}\Gamma_{2N,M}=\Phi_{N,2N}\nabla_{2N,2N}\Gamma_{2N,M} \tag{5}$$

This equation establishes direct links between the complex natural frequencies, mode shapes and the time response of the system. It alone, however, is insufficient to solve for modal data. The approach is to sample the same response data but with a duration shift Δt.

For all the response points, through using a delayed sampling for Δt_1, which is the integer multiple of Δt, a vector form of Eq. 5 can be written as:

$$\left[\ddot{Y}\right]_{N,M}=\left[\ddot{X}(t_i+\Delta t_1)\right]=\sum_{i=1}^{2N}\hat{\Phi}_i e^{\lambda_i(t_i+\Delta t_1)} \tag{6}$$

Where

$$\left[\ddot{Y}\right]_{N,M}=\hat{\Phi}_{N,2N}\begin{bmatrix} e^{\lambda_1\Delta t_1} & 0 & \cdots & 0 \\ 0 & e^{\lambda_1^*\Delta t_1} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & e^{\lambda_N^*\Delta t_1} \end{bmatrix}\Gamma_{2N,M} \tag{7}$$

Using a delayed sampling matrix $\left[\ddot{Y}\right]_{N,M}$ and normal sampling matrix $\left[\ddot{X}\right]_{N,M}$ to construct augmented matrix

$$\left[Q_{\ddot{X}\ddot{Y}}\right]_{2N,M}=\begin{bmatrix}\ddot{X}\\ \ddot{Y}\end{bmatrix}=\begin{bmatrix}\hat{\Phi}_{N,2N}\Gamma_{2N,M}\\ \hat{\Phi}_{N,2N}\Delta_{2N,2N}\Gamma_{2N,M}\end{bmatrix}=\left[\Upsilon_{2N,2N}\right]\left[\Gamma_{2N,M}\right] \tag{8}$$

Likewise, we can increase the time shift to Δt_2, which is the integer multiple of Δt and may be equal or not be equal to Δt_1, and sample the response data again. For all the response points, a vector can be written as:

$$\left[\tilde{Q}_{\ddot{X}\ddot{Y}}\right]_{2N,M}=\begin{bmatrix}\ddot{X}(t_i+\Delta t_2)\\ \ddot{Y}(t_i+\Delta t_2)\end{bmatrix} \tag{9}$$

and

$$\left[\tilde{Q}_{\ddot{X}\ddot{Y}}\right]_{2N,M}=\begin{bmatrix}\hat{\Phi}_{N,2N}\Omega_{2N,2N}\Gamma_{2N,M}\\ \hat{\Phi}_{N,2N}\Delta_{2N,2N}\Omega_{2N,2N}\Gamma_{2N,M}\end{bmatrix} \tag{10}$$

Where

$$\left[\tilde{Q}_{\ddot{X}\ddot{Y}}\right]_{2N,M}=\left[\tilde{\Upsilon}_{2N,2N}\right]\left[\Gamma_{2N,M}\right] \tag{11}$$

and

$$\left[\tilde{\Upsilon}_{2N,2N}\right]=\left[\Upsilon_{2N,2N}\right]\left[\Omega_{2N,2N}\right] \tag{12}$$

$\left[\Omega_{2N,2N}\right]$ is

$$\left[\Omega_{2N,2N}\right]=\begin{bmatrix} e^{\lambda_1\Delta t_2} & 0 & \cdots & 0 \\ 0 & e^{\lambda_1^*\Delta t_2} & \cdots & 0 \\ \vdots & \vdots & \ddots & \vdots \\ 0 & 0 & \cdots & e^{\lambda_N^*\Delta t_2} \end{bmatrix} \tag{13}$$

According to the relationship between augmented matrix $\left[Q_{\ddot{X}\ddot{Y}}\right]$ with $\left[\tilde{Q}_{\ddot{X}\ddot{Y}}\right]$, we have

$$\tilde{Q}_{\ddot{X}\ddot{Y}}Q_{\ddot{X}\ddot{Y}}^{-1}\Upsilon_{2N,2N}=\Upsilon_{2N,2N}\Omega_{2N,2N} \tag{14}$$

Defining that

$$A=\tilde{Q}_{\ddot{X}\ddot{Y}}Q_{\ddot{X}\ddot{Y}}^{-1} \tag{15}$$

A is the structural system matrix.

In actual application, the sampled discrete points M is larger than 2 times of the measuring points N ($M>2N$), so the structural system matrix A can be achieved by the least squares method to reduce error. Eq.14 can be simplified as

$$A\Upsilon_{2N,2N} = \Upsilon_{2N,2N}\Omega_{2N,2N} \tag{16}$$

Because $\Omega_{2N,2N}$ is a diagonal matrix, the Eq. 16 is equivalent to

$$A\Upsilon_{2N,2N} = \Omega_{2N,2N}\Upsilon_{2N,2N} \tag{17}$$

This equation constitutes a complex eigenvalue problem ($\Omega_{2N,2N}$ contains structural modal frequency and modal damping ratios; $\Upsilon_{2N,2N}$ contains structural mode shapes). These eigenvalues can be converted to the natural frequencies and damping ratio.

The ith diagonal element in Eq. 13 is the eigenvalue in the form:

$$\Omega_i = e^{\lambda_i \Delta t_2} \tag{18}$$

And

$$\lambda_i = ln(\Omega_i) \Big/ \Delta t_2 \tag{19}$$

From these solutions, we can derive the undamped modal frequency ω_{di} and modal damping ratio ξ_{di}:

$$\omega_{di} = |\lambda_i| \Big/ 2\pi \tag{20}$$

$$\varepsilon_{di} = \sqrt{\frac{1}{1+\frac{Im(\lambda_i)}{Re(\lambda_i)}}} \tag{21}$$

And the structural mode shapes Υ_i is

$$\Upsilon_i = \frac{1}{\lambda_i^2}\begin{bmatrix}\hat{\phi}_i \\ \varphi_i\end{bmatrix} \tag{22}$$

Complex mode shapes vector is the first N elements array based on the acceleration signal (i=1, 2, ... , 2N).

Using the augmented matrix $[Q]$ to replace $[\ddot{X}]$ in the analysis described from Eqs 3and 9 will lead to the identification of many numerical vibration modes as well as genuine modes. To distinguish the latter from the former, a mode shape coherence and confidence factor (MSCCF) can be defined. For a selected measurement point, the MSCCF defined as:

$$\text{MSCCF} = \frac{(\Upsilon_k^u)^T(\Upsilon_k^l)^* e^{\lambda_k \Delta t_1}}{(\Upsilon_k^l)^T(\Upsilon_k^l)^*} \tag{23}$$

The (Υ_k^u) is the upper half part of the k order mode, and (Υ_k^l) is the lower half part of the k order mode. If the mode is a genuine one, MSCCF value will be close to unity. Otherwise, it is likely the mode is a numerical one. In practical engineering applications, the measured data always contains noise. In this case, if the value of MSCCF is more than 0.95, it can be regard as a genuine one.

VALIDATION TEST

Description of test structure

As shown in Figure 6a, the bridge model represents a reduced-scale model of Shandong Binzhou Yellow River Cable-stayed Bridge (see Figure 6b). This single-tower double-cable-planes bridge model is mainly made of aluminum. Effective stiffness criterion is utilized in the reduced-scale dynamic-elastic bridge model design. Additional masses are added so the bridge model would give similar dynamic characteristics within the range of the prototype bridge. The bridge model is eventually built in a ratio of 1/40 to the prototype bridge considering lab conditions and economic. Table1 lists the specific parameters of the bridge model. Several experiments have been previously conducted with various configurations of this structure (Zhou *et al.* 2007). Their analysis results based on the similarity theory have showed that the reduced-scale bridge model with added masses has a good degree of similarity with the prototype bridge.

(a) Reduced-scale model (b) Original

Figure 6. Photo of Binzhou Yellow River Cable-stayed Bridge

Table 1. Experimental Cable-Stayed Bridge Model Mechanical Characteristics

element	Cross section	material	density (kg/m^3)	modulus (GP)	Poisson ratio	dimension (m)
middle tower	Box □	Aluminum alloy	2700	68.6	0.34	3.1×0.1375×0.095
side tower	Box □	Aluminum alloy	2700	68.6	0.34	1.9×0.12×0.075
deck	Separate Box	Aluminum alloy	2700	68.6	0.34	15.2×0.82×0.07
cable	Circle ○	High-Strength Steel wire	7850	200	0.30	varies

Description of Modal Test

A set of nearly identical forced vibration tests are scheduled to record acceleration time responses using white-noise ambient loads generated by two electronic-dynamic vibrators shaking the bridge deck up-down. For wireless tests, two wireless sensors (one for measuring, and the other for reference), which run on two AA batteries, are used to transmit the acceleration signal gathered by the customized sensor boards. FTSP method described in previous section is programmed and embedded into the sensors for time synchronization, taking advantage of the embedded processing capabilities residing on the distributed network of wireless sensor nodes. All wireless sensor nodes communicated in a single-network star topology with a central coordinator node connected to the serial port of a desktop. All measurement data is wirelessly streamed to the desktop in time history format. Wireless records are sampled using the standard sampling frequency of 100 Hz and baseline-corrected before processing. The duration of each record is 20 sec.

To validate the wireless sensor network performance and to provide a basis for comparison, 14 piezoelectric accelerometers are distributed along the center line of the bridge model in a pattern established to derive the mode shapes. The sensor layout is shown in Figure 7. All records are sampled using the standard sampling frequency of 1000 Hz and baseline-corrected before processing. A user-friendly LabVIEW graphical user interface is developed to control the vibration generator and collect the structural dynamical responses.

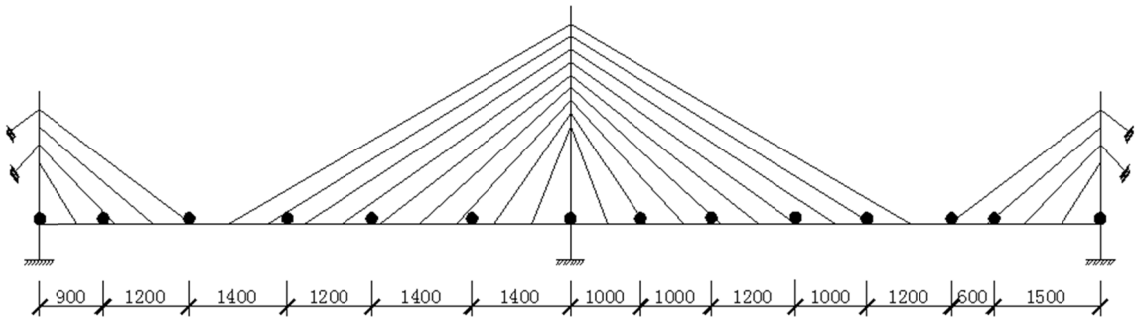


Figure 7. Sensor layout physical location

Modal Analysis Using Proposed Method

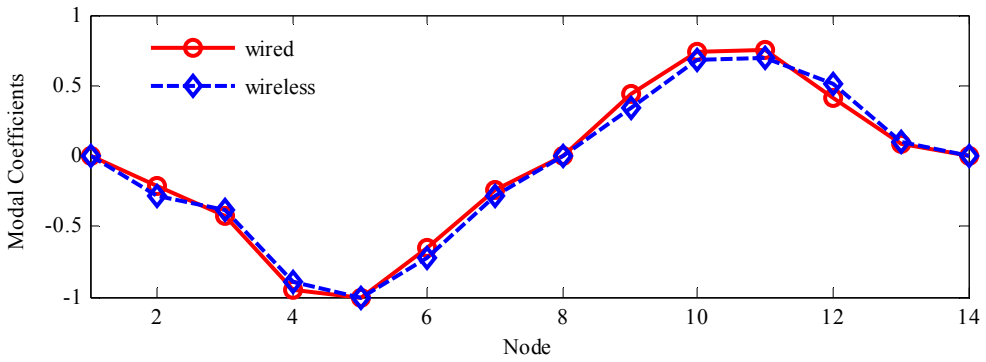
While the identification method can simultaneously generate the global modes, the local modes of structural

members can also be identified by employing the sensors associated only with those of the members. In this section, the deck modes and pylon modes are examined. Considering the rigid shape of the deck, especially the fixed connection between the pylon and deck, and the longitudinal restraints, domination of vertical vibration is expected. For this purpose, sensors measuring accelerations of deck in vertical direction and pylon in lateral direction are utilized as outputs.

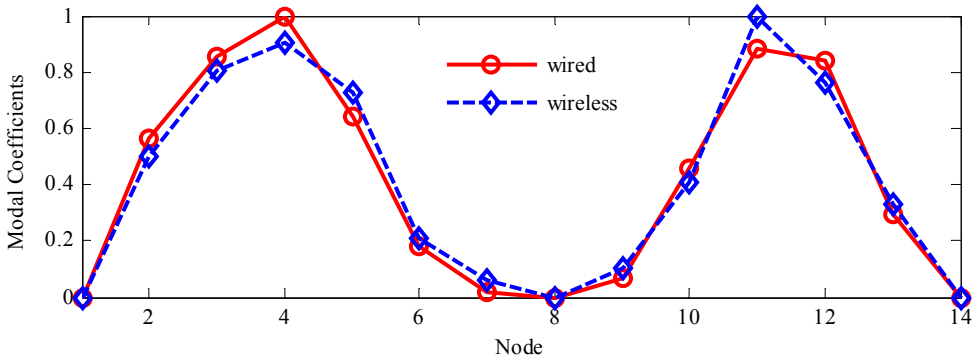
Using only output acceleration time histories processed by the modified ITD technique, natural frequencies and associated damping ratios of bridge deck and pylon are extracted and listed in Table 2. For the frequency zone, the estimated natural frequencies for the first six vertical bending modes of deck and the first two sidesway modes of pylon, are 3.81, 8.62, 10.52, 9.96, and 10.08 Hz, respectively. The corresponding modal damping ratios are 4.27%, 0.51%, 0.66%, 0.53%, and 0.42%, respectively. In order to distinguish the genuine mode from the numerical vibration modes due to the augmented measurement data in the proposed algorithm, the MSCCF, is defined and computed before the identification of mode shapes from the test data. For the five extracted mode shapes, the MSCCF values are 0.9992, 0.9997, 0.9996, 0.9861, and 0.9997, respectively. Note that the MSCCF values identified are nearly equal to unity in all cases. Therefore, the new method proposed here is able to provide reliable modal parameters in those cases. While there are no repeated modes, there are very close natural frequencies of the bridge model.

Table 2. Modal parameters of bridge model identified using modified ITD

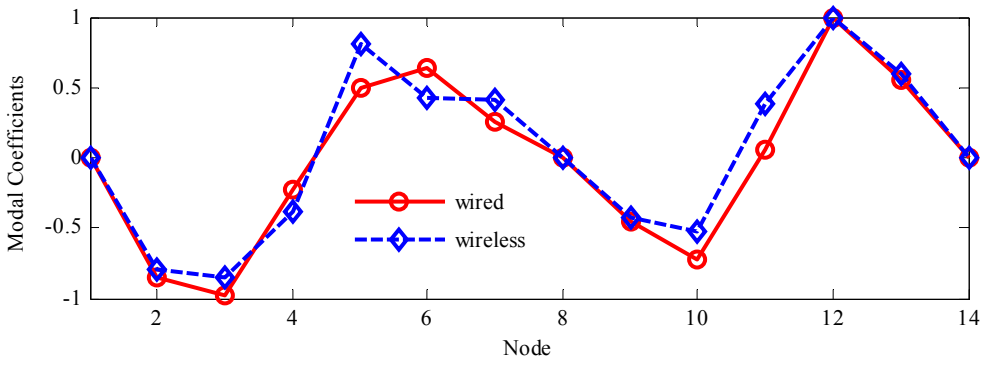
order	Frequency（Hz）	Damping Ratio	MSCCF
1	3.81 (deck)	4.27%	0.9992
2	8.62 (deck)	0.51%	0.9997
3	10.52 (deck)	0.66%	0.9996
4	9.96(pylon)	0.53%	0.9861
5	10.08(pylon)	0.42%	0.9997



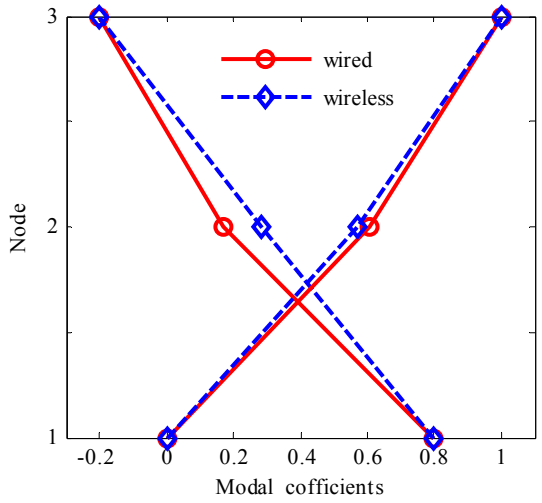
(a) 1st vertical mode



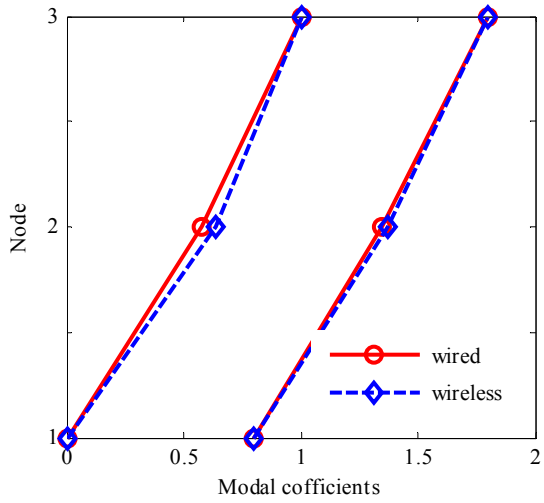
(b) 2nd vertical mode



(c) 3rd vertical mode



(d) pylon first symmetric sideway



(e) pylon first anti-symmetric sideway

Figure 8. Modal shapes of the bridge deck and pylon identified by modified ITD method

The estimated three vertical bending mode shapes of deck and two sideway mode shapes of pylon are shown in Figure 8. Their mode shapes fit those of wired testing counterparts very well. The results are completely symmetrical or anti-symmetric, which present the structural characteristic of the cable-stayed bridge. The fundamental mode of the bridge is the first vertical bending with a natural frequency of 3.81 Hz. This mode exhibits a zero node symmetrical pattern of girder. The results of modal analysis of the pylon indicate that the first and second modes are dominated by lateral motion, with the difference in the phase. The first mode of pylon exhibits an in-phase motion, while the second one shows the out-of-phase motion. Both pylon modes at the frequency range of 9.96-10.08 Hz are associated with the deck motion. The interactions between the deck and pylon motion exist in such a way that the pylon modes are resulted from their girder's predominant counterpart modes. Damping ratios of pylon modes are found lower than the deck modes.

CONCLUSIONS

In this paper, a prototype wireless sensor board has been developed and successfully integrated with the IRIS mote to conduct modal tests for a laboratory cable-stayed bridge model. In particular, a modified ITD method has been proposed in the modal parameter identification procedure to account explicitly for real time acceleration signals that does not need numerical integration to obtain displacement and velocity time histories from acceleration measurement at each response point. Out results provide conclusive evidence of the complex dynamic behavior of the bridge. The dynamic response of the cable-stayed bridge is characterized by the presence of many closely spaced, coupled modes. For most modes, the experimental modal frequencies and mode shapes from both wireless tests and wired tests compare quite well. The test results demonstrate the feasibility of wireless sensing technology in civil engineering structure health monitoring application.

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