Experimental Vibration Analysis for Structural Identification of a Long-Span Suspension Bridge

J. Zhang, Ph.D.[1]; J. Prader[2]; K. A. Grimmelsman, A.M.ASCE[3]; F. Moon[4]; A. E. Aktan, M.ASCE[5]; and A. Shama, Ph.D., P.E., M.ASCE[6]

Abstract: Structural identification (St-Id) of long-span bridges by ambient vibration testing provides a starting point for quantitatively characterizing the actual in-service mechanical characteristics and behaviors of these complex constructed systems. However, various uncertainties involved in the experimental and identification processes impact the reliability of St-Id, especially if vibration testing is the sole experiment. Such uncertainties represent perhaps the most fundamental barrier to more widespread applications of measurements in civil engineering practice. The goal of this paper is to leverage a vibration test of a long-span suspension bridge to illustrate a number of possible strategies for coping with the uncertainties confronted when identifying the dynamic characteristics of large-scale constructed systems. The design and implementation of a field test in the context of St-Id are first presented to illustrate how uncertainties can be mitigated by following a disciplined approach to designing the experiment. Next, data preprocessing strategies including data inspection, time window selection, band-pass filtering, averaging, and windowing are proposed to reduce data errors. Three separate postprocessing methods, including Peak Picking, PolyMAX, and Complex Mode Indicator Function, are executed independently to verify the reliability of the data processing results. Experimental results for both the towers and suspended spans are correlated with simulations from three-dimensional finite-element analysis of the long-span bridge for St-Id. **DOI: 10.1061/(ASCE)EM.1943-7889.0000416.** *© 2013 American Society of Civil Engineers.*

CE Database subject headings: Suspension bridges; Span bridges; Vibration; Signal processing; Uncertainty principles; Finite element method; Experimentation.

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Introduction

Structural identification (St-Id), which refers to any systematic approach for identifying structural parameters through the use of input and/or output test data, is a key enabler for structural health monitoring applications. Following 30 years of development after the pioneering work of Abdel-Ghaffar and Housner (1977), numerous applications in engineering practice have further advanced the state of the art and developed St-Id into an effective tool for improving the reliability of bridge operational and maintenance management decisions. The characterization of an in-service structure by full-scale field-testing is one of the most critical steps in the overall St-Id process as it is the primary source of the quantitative and objective data needed to link analysis with actual in situ characteristics of a bridge.

In the case of St-Id applications to long-span bridges, the in-service characterization of the structure is most often accomplished by measuring vibrations of the structure in a global sense. The dynamic or modal characteristics (natural frequencies, mode shapes, and damping ratios) of the structure, which are directly related to the structural properties (mass, stiffness, connections, boundary conditions, etc.) and can also be determined from analytical models of the structure, are extracted from the measured vibration data using any number of available modal parameter identification techniques. Based on practical considerations, the vibration measurements of the structure are usually recorded while the structure is subject to unmeasured and uncontrolled dynamic excitation provided by service and ambient environmental loads. There is a relatively long history and numerous examples in the literature of long-span bridges that have been characterized using ambient vibration testing, which is also referred to as operational modal analysis (Abdel-Ghaffar and Housner 1977; Peeters and De Roeck 2001; Shama et al. 2001; Ko et al. 2002; Grimmelsman et al. 2007; Conte et al. 2008; Siringoringo and Fujino 2008; Pakzad and Fenves 2009; Brownjohn et al. 2010; Hong et al. 2011). Perhaps the best-known example is the comprehensive characterization of the Golden Gate Bridge performed by Abdel-Ghaffar and Scanlan (1985a). Although this application was performed more than 20 years ago, many aspects of the approach employed are still used

[1]Professor, Key Laboratory of C&PC Structures of the Ministry of Education, Southeast Univ., Nanjing 210096, China; and International Institute for Urban Systems Engineering, Southeast Univ., Nanjing 210096, China; formerly, Research Assistant Professor, Dept. of Civil, Architectural, & Environmental Engineering, Drexel Univ., 3141 Chestnut Street, Philadelphia, PA 19104 (corresponding author). E-mail: jianzhang.civil@gmail.com

[2]Ph.D. Candidate, Dept. of Civil, Architectural and Environmental Engineering, Drexel Univ., Philadelphia, PA 19104.

[3]Assistant Professor, Dept. of Civil Engineering, Univ. of Arkansas, Fayetteville, AR 72701.

[4]Associate Professor, Dept. of Civil, Architectural and Environmental Engineering, Drexel Univ., Philadelphia, PA 19104.

[5]John Roebling Professor, Dept. of Civil, Architectural and Environmental Engineering, Drexel Univ., Philadelphia, PA 19104.

[6]Director of Seismic Engineering, Hardesty and Hanover, 1501 Broadway New York, NY 10036; formerly, Senior Supervisory Structural Engineer, Parsons Corporation, 100 Broadway, New York, NY 10005. E-mail: AShama@hardesty-hanover.com

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today for the St-Id of large-scale constructed structures; however, there have also been some notable advances made possible by the evolution of computer hardware and data-acquisition software.

In the past, ambient vibration testing of long-span bridges was primarily accomplished using a limited number of stationary reference accelerometers and a similarly small number of roving accelerometers that were marched across the structure in numerous discrete test setups. Some obvious drawbacks of this approach were that the locations of the reference sensors had to be carefully selected, and any changes in the structure's response (e.g., nonstationarity) between setups would render the integration of such data unreliable. Many recent ambient vibration tests of long-span bridges (Pakzad and Fenves 2009; Conte et al. 2008; Grimmelsman et al. 2007) have utilized tethered or wireless sensor networks that included large numbers of accelerometers that were distributed across the structure and that measured vibrations from the spatially distributed locations simultaneously. The capability to deploy such large arrays of sensors has allowed multiple-reference modal parameter identification methods to be employed in the analysis of the measurement data (He et al. 2009).

Another significant change that has occurred is that it is now possible to measure, process, and analyze very long acceleration-time records with much greater resolution using modern data acquisition hardware. The amount and quality of the measurement data routinely collected and analyzed in recent ambient vibration testing applications was not practical with the analog-to-digital tape recorder systems that were employed in the past. With this advance, it is now more practical to evaluate the statistical characteristics of the identified dynamic properties, to identify any variations in them, and to establish their reliability in a quantitative sense. Further, there are recent examples where such analyses have been done in real-time at the sensor before the data were transmitted wirelessly to a remote station for subsequent analysis and archival (Lynch 2007; Pakzad and Fenves 2009).

Presently, there are many more modal parameter identification methods that can be used to analyze ambient vibration measurement data than were available 30 years ago. In the past, ambient vibration data analysis was primarily accomplished using the Peak Picking approach on the measurement data in the frequency domain. Many mathematically more rigorous modal parameter identification methods originally employed for classical experimental modal analysis (controlled dynamic testing) have since been extended to output-only modal analysis (ambient vibration testing), and others were developed and refined specifically for output-only modal analysis applications. Some of the more commonly used of these identification methods include Ibrahim Time Domain (Ibrahim 1977), Eigensystem Realization Algorithm, Stochastic Subspace Identification (SSI), and Frequency Domain Decomposition. Although the basic Peak Picking approach has been shown to be reasonably reliable for modal parameter identification of long-span bridges, the more rigorous identification methods have been shown to simplify the identification process and to increase the reliability of the identified results, particularly for cases where the modes of the structure are closely spaced.

Despite these improvements in testing and analysis capabilities, many uncertainties remain associated with ambient vibration testing, data processing, interpretation, and structural modeling that still pose major challenges to reliable St-Id, and the resulting errors may hinder a more routine adoption of St-Id approaches in support of bridge operational and maintenance management decisions. The writers believe that the most critical of these remaining uncertainties include:

1. Prevailing excitations (wind and traffic) and environmental conditions (humidity, wind, and most important, temperature) impart uncertainties into ambient or forced-vibration experiments of long-span bridges;
2. Frequently unavoidable measurement noise and bias errors that arise from the deployed experimental hardware components (sensors, cabling, and data acquisition system) and the experiment design (array density and distribution, data acquisition parameters, on-site quality control, etc.) and especially the electromagnetic interference in the field;
3. Real structures are not accurately and completely modeled in the St-Id process, especially in relation to complex structural components like suspension cables, bearings, soil-pile-foundation interaction, and tower-superstructure interfaces; and
4. Various data preprocessing methods with predetermined parameters for data filtering, resampling, windowing, and averaging produce different forms of cleaned data, and various post-processing methods may produce different identification results, depending on their capabilities to capture the signal characteristics in a noisy environment.

Many of these sources of uncertainty are related to knowledge gaps and are unlikely to be mitigated by further advances in measurement, signal processing, or data processing approaches. In addition, there are frequently nontechnical constraints such as access and logistical considerations associated with full-scale testing of in-service structures that can limit the reliability of the structural identification results to a similar degree as the sources of uncertainty associated with experimental technologies and knowledge gaps.

A number of researchers have recognized the importance of investigating and scientifically characterizing these uncertainties for more reliable St-Id results. Moon and Aktan (2006) conducted a detailed review of the impact of uncertainty on the structural identification of constructed systems. In their review, the unique attributes of constructed systems observed during St-Id studies were classified as either aleatory uncertainty (including heterogeneity, sensor/cable noise, etc.) or epistemic uncertainty (unknown or less understood structural behaviors). Peeters and De Roeck (2001) studied frequency variations as a function of temperature by using one year of data from the Z24-Bridge measurement. Nagayama et al. (2005) identified structural modal characteristics from ambient vibration data for a long-span bridge, and studied their dependencies on the wind velocity and response amplitude level. Reynders et al. (2008) developed a method to bound modal parameter uncertainty by using the reference-based covariance-driven stochastic subspace identification (SSI) algorithm. Omenzetter et al. (2004) proposed a multivariate outlier detection method to automatically filter out the outliers from unusual events (like sudden settlement of foundation, ground movement, excessive traffic load, or failure of post-tensioning cables). A Bayesian framework was presented by Beck and Katafygiotis (1998) to explicitly incorporate the uncertainties for structural parameter identification. A state-of-the-art report detailing available approaches, methods, and technologies for effective practice of St-Id of constructed systems has been prepared by the ASCE SEI Committee on Structural Identification of Constructed Systems (2011), in which the uncertainties involved in each stage of the St-Id process were discussed.

Objective and Scope

The objective of this paper is to investigate a methodology for the experimental characterization of long-span bridges by ambient vibration testing, given the state-of-technology and with a particular focus on data quality evaluation, uncertainty reduction, and reliable

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data interpretation. The experimental characterization of a long-span suspension bridge, including field-testing, signal processing, and finite-element (FE) analysis, is presented as an example to demonstrate the proposed methodology. A multiple-reference measurement scheme was first designed and implemented for this bridge to simultaneously measure vibrations in each of the principal response directions (vertical, lateral, and longitudinal). It included 30 stationary accelerometers that were distributed throughout the bridge spans and towers and were monitored during the entire field-testing program. In addition to the stationary accelerometers, 15 roving accelerometers were temporarily installed in specific regions of the bridge during four different instrumentation setups. Significant care was taken in calibrating the sensors and for on-site quality control to reduce data uncertainty during the experiment stage. The data preprocessing efforts included a number of stages (data inspection, time window selection, digital filtering, exponential windowing, and time domain or frequency domain averaging) and examined various approaches to establish their influence on uncertainty reduction. Following this, three different data postprocessing methods including Peak Picking, PolyMAX, and Complex Mode Indicator Function were executed independently to identify the structural dynamic characteristics. A three-dimensional (3D) FE analysis of the suspension bridge was performed, and the modal parameters of the FEM were correlated with those identified from the ambient vibration measurements. The dynamic characteristics of the towers were included in this investigation because these represent important structural components in the global seismic response of the structure (which was the motivation behind performing the test) and because the dynamics of the coupled superstructure-tower system can influence the reliability of the structural identification results.

Bridge Description (Step 1)

The initial step in the St-Id framework involves familiarization with the issue that is driving the application and conceptualization of the structure itself. The structure that was investigated is a long-span suspension bridge (Fig. 1) with a 549-m main span, and an anchorage-to-anchorage total length of 886 m. The bridge was commissioned in January 1961 and currently carries approximately 105,000 vehicles each day. The suspension bridge was the first such bridge designed by Othmar Ammann following the 1940 Tacoma Narrows Bridge failure, and he used an 8.5-m-deep stiffening truss under the deck. Six traffic lanes are supported by a series of transverse floor trusses, and the floor trusses are framed into two longitudinal stiffening trusses within the same planes of the main cables. A system of lateral stiffening trusses coupled with the longitudinal and floor trusses forms a rigid frame to resist wind and other loads. Two main cables, each 976 m in length, support the main span and two side spans of the bridge. Structural safety inspections and rehabilitation work have been performed on several occasions since 1980, which included replacing the roadway deck, repairing the structural steel, modifying expansion joints, replacing existing rocker bearings, improving the drainage system, rewrapping the main cables, and rehabilitating the electrical systems. The structural identification of the suspension bridge through ambient vibration monitoring, data analysis, and FE analysis that is described in this article was performed as part of a seismic vulnerability assessment program for the bridge that was led by the Parsons Corporation (New York, New York).

Finite-Element Model (Step 2)

FEM analysis results are useful for selecting the appropriate experimental characterization approaches and designing the instrumentation layout (Step 3) for structural identification.

Modeling Main Structural Components

In this study, a 3D FEM of the suspension bridge was constructed using the commercial FE software *ADINA 8.7*. The main cables of the bridge were modeled using elastic cable elements, which only transmit longitudinal forces and can be used with large displacements. All members of the stiffening and floor trusses, lower lateral bracing, and upper lateral bracing were modeled using 3D elastic beam elements with six degrees of freedom at each node. The deck system is a 127 mm concrete-filled steel grid, which is supported by W18 $\times$ 55 stringers. The concrete-filled steel grid was modeled with shell elements that had an elastic modulus corresponding to the concrete, and the stringers were modeled by 3D elastic beam elements. The suspended spans are supported by two multicell steel towers, in which vertical steel plates are arranged in rectangular patterns, connected with plates and angles. Three-dimensional beam elements were used to model different portions of the towers. Because the section properties of the towers vary with the tower height from changes in the thicknesses of the plates and the size of the rectangular cells, a relatively large number of beam elements were used. Vertical links pinned at both ends support the main and side-span trusses at the towers, and restrain the movement of the main suspended span in the longitudinal direction. Gap elements were provided between the centerline of tower legs and the centerline of the vertical links in the longitudinal direction to evaluate the potential for contact between the suspended span and the towers during a simulated seismic event. The structural mass was determined from the bridge's original construction plans and was distributed to each node. The structural model (Fig. 2) consisted of: 15,175 nodes; 294 cable elements for the main cables and the suspenders; 9,386 beam elements for the stiffening and floor trusses, bottom and upper laterals, and stringers; and 6,048 shell elements for the concrete-filled steel grid.



Fig. 1. Photograph of the suspension bridge

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Connection Modeling

Careful modeling of the structural connections among the complex subsystems of major long-span bridges is an effective way to reduce uncertainty associated with analytical aspects of the structural identification. The wind tongues are an example of a complex structural subsystem for this bridge, and their analytical modeling is somewhat challenging. The wind tongues transfer lateral loads from the superstructure to the tower struts and anchorages at the main and side spans [Fig. 3(a)]. The wind tongues at the anchorage are vertical members at the center of the floor truss that bear in the transverse direction against a steel bearing, which is rigidly attached to the anchorage. The wind tongues were represented by 3D beam elements, rigid links, and gap elements as illustrated in Fig. 3(b). Each gap element has two nodes, with one slaved to the wind tongue and the other slaved to the anchorage. The two gap elements at both sides of the tongue transmit all transverse force from the wind tongue directly to the anchorage. Special care was also taken in modeling the relief joint connection of the deck-stringer system and the cable-tower connections; however, for brevity, these are not presented here.

Modeling Foundations and Soil-Structure Interaction

The soil-structure interaction (SSI) simulation is important in the FE modeling especially for seismic evaluation of the studied structures. The towers and anchorages of the suspension bridge are founded on deep reinforced concrete caissons. Unlike the pile foundations which support the approach structures of this bridge, the caissons have significant weight, and their seismic response will be driven mainly by their inertial interaction with the surrounding soil. Gapping and separation between the caisson and the surrounding soil is a major parameter in the evaluation of the caisson's behavior under seismic loads. For this reason, the most practical Winkler approach was used to represent the caisson-soil interaction (SSI), and a Winkler spine model was developed to simulate the caisson-soil system.

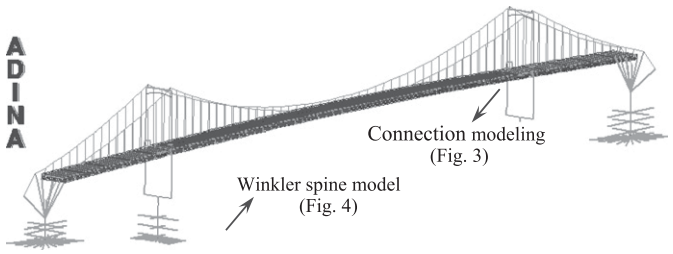


Fig. 2. FEM of the suspension bridge

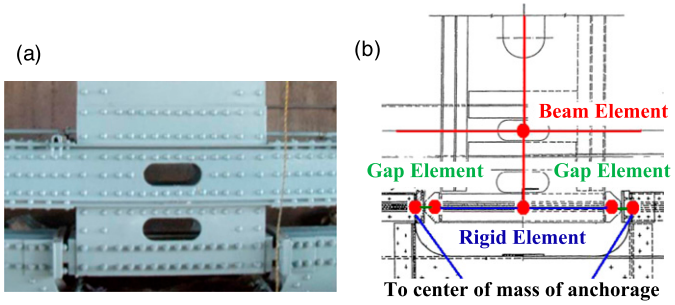


Fig. 3. Connection modeling of the wind tongue: (a) photo; and (b) FE modeling

In applying the Winkler approach, a discretization scheme was first established for the caisson-soil system of the global model. Fig. 4(a) illustrates the North Tower caisson and its soil profile and parameters that were established from subsurface exploration. Fig. 4(c) shows the discretization schemes for the North Tower caisson-soil system. The caisson in the global model was represented by a combination of 3D elastic beam elements representing the spine of the caisson, rigid links, and SSI elements to represent the soil plastic properties and gapping effects. This particular representation of the caissons in the global model is henceforth referred to in this article as the Winkler spine model. The nonlinear load-deformation characteristics of the near-field SSI elements were calibrated by comparing nonlinear static pushover analysis results from the Winkler spine model with those from a 3D continuum FEM of the soil-caisson system [Fig. 4(b)] constructed in *ADINA 8.7*. The continuum FEM can simulate both the plastic behavior of the soil and the caisson-soil gapping behavior (geometric nonlinearity). The soil modulus of elasticity was calculated from the distribution of the shear wave velocities for the soil profile [Fig. 4(a)], and other parameters were established from soil exploration. Once the Winkler spine model was calibrated from the 3D FEM, it was able to simulate the actual caisson-soil interaction behavior. A similar discretization scheme was also established for the South Tower, South Anchorage, and North Anchorage.

Ambient Vibration Testing (Step 3)

Sensors, Cables, and Data Acquisition

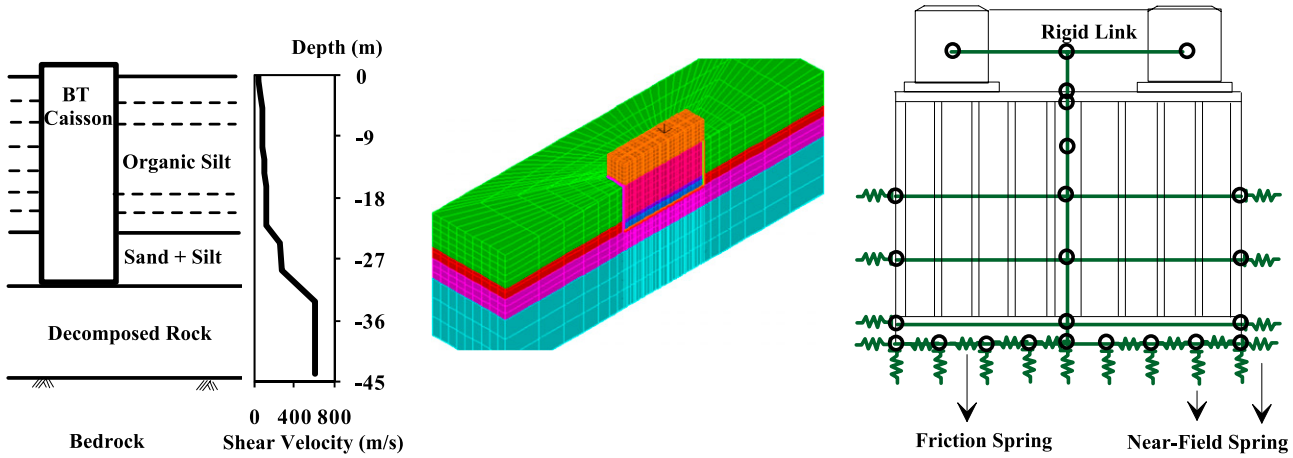
A vibration-based St-Id program was conducted by the writers to evaluate the dynamic characteristics and related earthquake vulnerability of the bridge. The vibrations of the bridge from operating and ambient environmental loads were measured using Model 393C seismic accelerometers from PCB Piezotronics. These accelerometers have a frequency range of 0.025–800 Hz. The frequency band of interest for the dynamic characterization of this suspension bridge was determined from preliminary FEM analysis to be in the range of 0.1–∼10 Hz, and the performance specifications of the seismic accelerometers were deemed more than adequate for this frequency band. The monitoring program employed a cabled instrumentation setup. The cabling plan devised for the vibration testing incorporated several long runs of multiple pair cables that were installed between a data acquisition cabinet located near the middle of the main span and breakout boxes placed near the accelerometer locations. Data provided by the sensor manufacturer indicated that the performance of the accelerometers in the relatively low-frequency band relevant to the dynamic characterization of the bridge is not affected by the use of long cables. The data acquisition system used for the ambient vibration testing consisted of an HBM MGCPlus 24-bit, synchronous data acquisition chassis with ML801B voltage modules and a laptop computer with data acquisition software. The vibration signals were recorded with a sampling rate of 200 Hz during the testing, and the analog signals were subject to a 20-Hz antialiasing filter before they were digitized.

Instrumentation Scheme

The multiple-reference instrumentation scheme devised for the test (Fig. 5) included both roving and stationary accelerometers. A number of accelerometers were roved to different locations on the bridge in a series of test setups while the positions of a slightly larger number of accelerometers were kept stationary throughout the entire testing program.

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(a) Caisson and soil profiles (b) FE model (c) Winkler spine model

Fig. 4. Modeling of the soil-caisson interaction

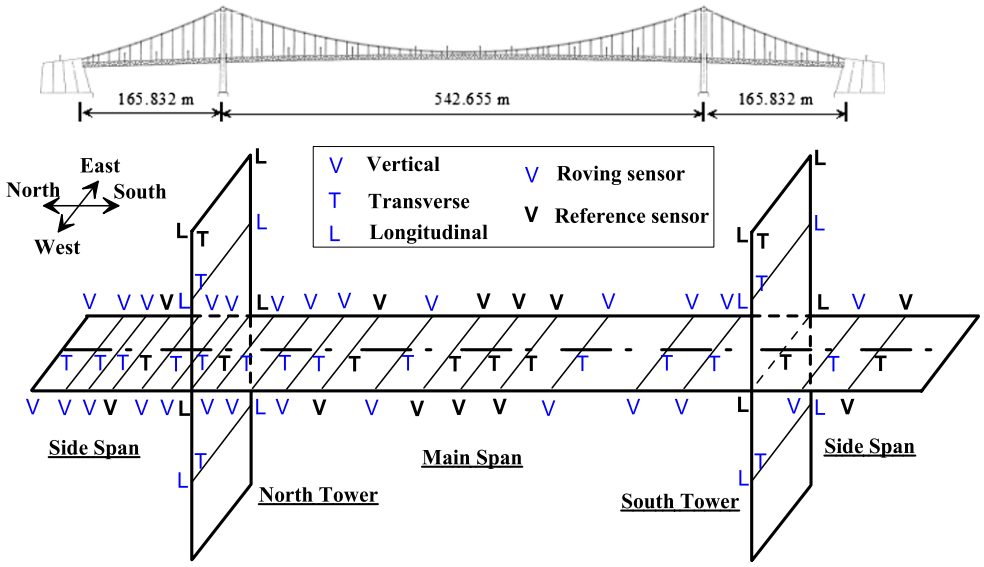


Fig. 5. Accelerometer layout for stiffening trusses and tower

As illustrated in Fig. 5, a total of 45 uniaxial accelerometers were arranged on the structure such that the vertical, longitudinal, and lateral (transverse) components of the vibration response at each measurement station could be recorded. Of this total, 30 accelerometers remained at stationary positions throughout the testing and served as reference sensors. The reference accelerometers were installed at 10 measurement stations that were located on the stiffening trusses in the main span and both side spans, and in both towers. The typical layout for the three accelerometers that were used at each reference measurement station on the main and side-span stiffening trusses is illustrated in Fig. 6(a). Local vibrations of the floor truss vertical member were determined to be well beyond the frequency band associated with global lateral vibrations of the stiffening truss system. The remaining 15 accelerometers were positioned in the various subregions of the bridge (side spans, main spans, and towers) using a total of four roving setups to provide additional measurements and added spatial resolution in these regions. The multiple-reference, roving instrumentation scheme provided multiple and spatially well-distributed reference sensors that were used with in-depth vibration data from the main subregions of the bridge. It was expected that having multiple and spatially distributed reference locations available for the data analysis and interpretation stages would permit a significantly more stable and reliable identification of the dynamic characteristics of the structure.

Field Test and Data Collection

The on-site portion of the vibration-testing program included both the installation and verification of the operation of the accelerometers, cabling, and data acquisition components. Each accelerometer used in the vibration-testing program was calibrated in the laboratory using a suspended mass calibration procedure. After the proper operation of the installed accelerometers and cables were verified in the field, the data acquisition system was set to record 1-h-long data sets throughout the duration of the ambient vibration test program. The initial setup took two weeks with a team of three personnel, and the roving generally took around 8 h to complete.

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Data Processing (Step 4)

Step 4 of the St-Id process involves the processing and interpretation of data collected during Step 3. Errors in the measured structural response can lead to inaccurate and unreliable identification results and if not mitigated, this can render any decisions made using the results of the identification incorrect and invalid. For modal identification of the spans, ambient vibration data from 38 sensors mounted vertically on the spans (Fig. 5) were processed.

Two- and Three-Dimensional Data Processing

In vibration testing, the sensors are generally deployed in the longitudinal, transverse, and vertical directions on the bridge to capture the structural responses in all planes simultaneously. Because of the number of sensors and the very different signal-to-noise ratios typically observed for different response directions, it is common to postprocess the data sets for each direction separately to obtain two-dimensional (2D) mode shapes (termed 2D processing). Although the 2D approach to designing and postprocessing vibration data may be common and convenient, in the case of structures with torsional coupling of critical lower modes, applying only a 2D processing approach may render the test-analysis correlation incomplete and uncertain (Pan et al. 2010).

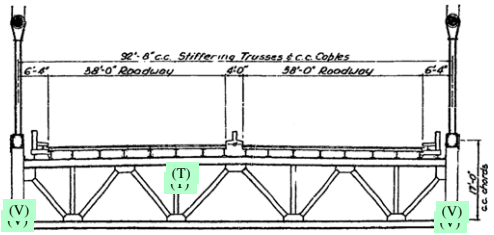
In the case of the test bridge, the critical modes were not coupled by torsion and the data could be reliably processed using the 2D approach. The automated data processing procedure illustrated in Fig. 7 was developed and used to mitigate the various kinds of uncertainty often encountered in the raw measurement data from constructed systems and to ensure a reliable structural identification.

Data Preprocessing for Uncertainty Reduction

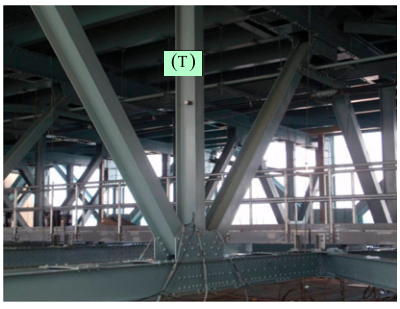
Data preprocessing procedures were implemented to clean the raw data before it was postprocessed to identify the structure's modal parameters. The measured acceleration records were first visually inspected by plotting their time histories to identify any malfunctioning sensors or data segments containing pronounced errors such as bias or large spikes. A time window selection algorithm that computed the mean and SD of the acceleration amplitude for each time window was then developed to automatically remove any data segments that contained pronounced noise. The data were subsequently passed through a digital Butterworth band-pass filter with cut-off frequencies at 0.1 and 10 Hz to remove any linear DC offset and frequency components outside of the frequency band of interest.

After completing the initial data quality checks and filtering, two different data averaging methods, including time domain averaging and frequency domain averaging, were used in the data preprocessing stage to minimize the effects of random errors on the identified modal properties. In the time domain averaging method, a complete 1-h acceleration time record was divided into an equal number of segments that were directly averaged together using a 50% overlap of adjacent data blocks in the time domain. Time domain averaging of the measured responses can indeed attenuate the random components of the measured signals that are associated with traffic and other random effects and will enhance the deterministic components (the structural modes) contained in the measured response signals. Cross-correlation analyses between each averaged time-domain data record and a selected reference averaged time-domain record were performed to transform the ambient vibration data into free-decay structural responses (James et al. 1995). In the frequency domain averaging method, each time segment was first transformed to the frequency domain by fast Fourier transform (FFT), and the resulting spectra were averaged in the frequency domain.

The cross-power spectral density functions (CPSD), computed from a single data set that was processed with the two different averaging methods, are shown in Fig. 8(a). Singular value (SV) decomposition of the spectral density matrices computed from the two averaging methods was also performed and the SV magnitudes are plotted in Fig. 8(b). It should be noted that peak values of the SV magnitudes correspond to the natural frequencies, as used in the complex mode indicator function (CMIF) postprocessing method. It can be seen that both averaging methods yield very similar results for



(a) Vertical and transverse (lateral) accelerometers on stiffening truss



(b) Transverse (lateral) accelerometer as-installed during field test

Fig. 6. Typical accelerometer layout on stiffening trusses at span measurement stations

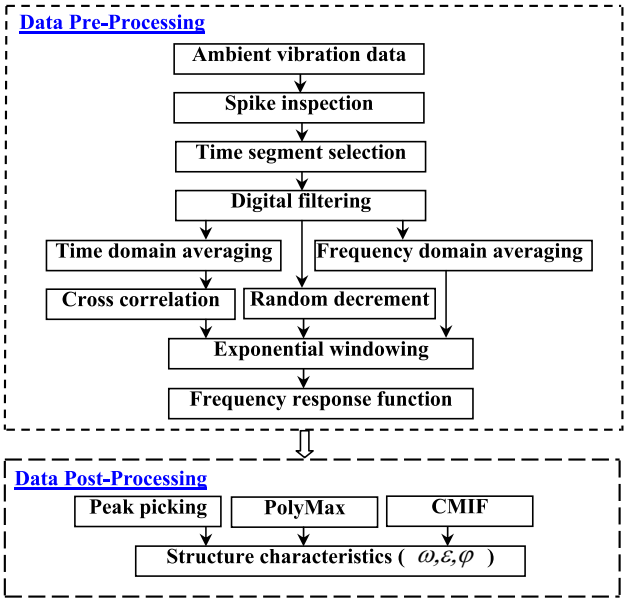


Fig. 7. Flowchart of the data processing procedure

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the identification of natural frequencies. Only the data processed using the time-domain averaging approach was used for the subsequent data postprocessing steps described in next section. The reliability of this data was also scrutinized by evaluating the similarity of CPSD plots computed from independent data sets that were processed in the same manner.

After the data averaging and cross-correlation analysis was completed, an exponential window was applied to the resulting free-decay time responses to help minimize the effects of leakage. In this study, the exponential window was designed to reduce the amplitude at the end of the signal to 0.001% of the initial amplitude of the signal. Fig. 9(a) shows windowed and unwindowed free-decay responses obtained from cross-correlation functions applied to a measured random vibration signal. The CPSDs of the vibration data with and without the exponential window applied are illustrated in Fig. 9(b). It is observed that both curves show peaks at the same frequencies; however, the curve created with the exponential window is smoother and the peaks are easier to identify. The use of an exponential window adds artificial damping to the signals, which must be removed in later stages of the analysis. The FFT was used to transform the time domain test data to the frequency domain by using a blocksize of 16,384 points. Because the sampling rate for the raw measurement data was 200 Hz, the selected blocksize yielded a frequency resolution of 0.0122 Hz. This was deemed adequate for identifying any closely spaced modes of the bridge.

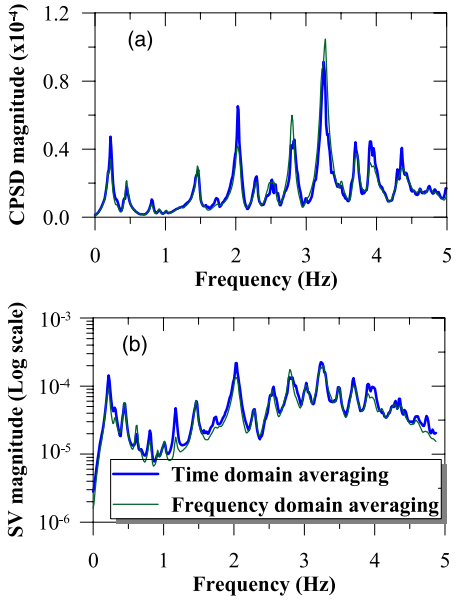


Fig. 8. Time domain and frequency domain averaging

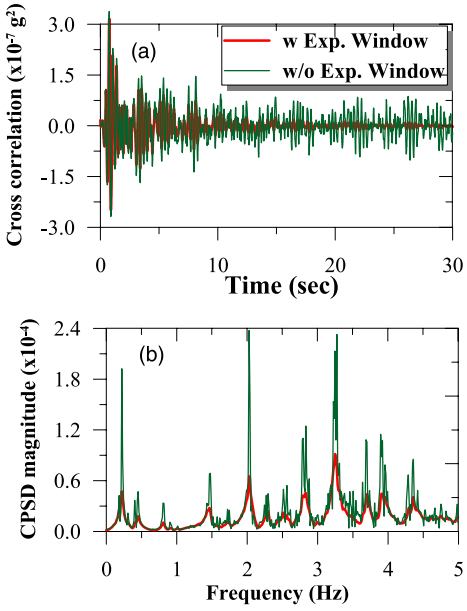


Fig. 9. Cross-correlation function and the effect of exponential windowing

Data Postprocessing for Modal Parameter Identification

The specific modal identification methods employed for the measured vibration data are a more important consideration when a structure has closely spaced modes, when the measurements have very low signal-to-noise ratios, and for the damping ratio estimates. To evaluate the reliability of the modal identification results, three separate methods (Peak Picking, PolyMAX, and CMIF methods) were independently applied to the measurement data (Fig. 10). Peak Picking is one of the most basic approaches that can be used to identify structural modal properties from the output-only measurements recorded in an ambient vibration test. The basic premise of this approach is that when a lightly damped structure is subject to stationary and uniform, broadband random excitation, the frequency spectra for the measurement channels will contain peaks at the locations of the structure's resonant frequencies. A drawback of this approach is that the frequency spectra can also contain peaks that are associated with measurement noise and errors, or harmonics that may be present in the unmeasured excitation spectrum. The PolyMAX method is a noniterative frequency-domain parameter estimation method (Peeters et al. 2004). The CMIF method, which is also referred to as the frequency domain decomposition method when applied to output-only vibration measurement data, estimates the eigenvalues and eigenvectors through SV decomposition of the spectral density matrix computed from the measured data and can be considered as an extension of the Peak Picking method (Catbas et al. 2004).

Vibration measurements from the roving accelerometers that were concentrated in the various subregions of the bridge (side spans, main spans, and the towers) were combined with the measurements from the large number of stationary reference channels distributed throughout the structure to perform the structural characterization. For instance, the decoupled vertical responses from the roving and stationary accelerometers at measurement stations located on the side and main spans were utilized to identify the frequencies and mode shapes for the complete bridge in the vertical direction.

In applying the Peak Picking method, a mode indicator function (MIF) was first developed by using the normalized sum of the autopower spectral densities computed from the measurements on a particular component of the bridge in one direction (e.g., the vertical response direction for the stiffening trusses or the lateral response direction of a tower, etc.). The peaks in the MIF plot [Fig. 10(a)] were identified using an automated peak identification routine, and normalized mode shape vectors corresponding to each identified peak location were extracted from the data. Unreliable identification results can be associated with peaks in the unmeasured excitation spectra, noise and errors in the data, or by interactions between the oscillations of different bridge components. Other characteristics of the data such as the phase angles and the

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coherence functions can be examined to help identify and exclude any noise-related or otherwise unreliable natural frequencies from the final identification results.

The multiple-reference nature of the test was also utilized to further evaluate and exclude unreliable results from the identified modal parameters. To accomplish this, the preliminary mode shapes were inspected and the frequencies associated with mode shapes that were essentially the same for all possible reference sensors considered were deemed more likely to be associated with the largest signal-to-noise ratios in the measurements and thus were identified as natural frequencies of the structure.

The PolyMAX method and the CMIF method were also utilized to independently identify dynamic characteristics of the suspension bridge. The major benefit of the PolyMAX modal parameter estimation technique is in its ability to produce very clear stabilization diagrams. The basic idea of the stabilization diagram is that several runs of the complete pole identification process are made, by using models of increasing order. The pole values of the true eigenmodes always appear at a nearly identical frequency, while false poles tend to scatter around the frequency range. Fig. 10(b) shows the stabilization diagram obtained with the PolyMAX method, from which all physical poles are clearly found and frequency and damping ratios are calculated. Fig. 10(c) shows a typical CMIF plot for modal parameter identification of the bridge span, from which structural frequencies are clearly identified at the peaks in the CMIF. Using three independent postprocessing techniques allowed the writers to identify the modal characteristics of the bridge with greater confidence and reliability than would have been possible using only one approach.

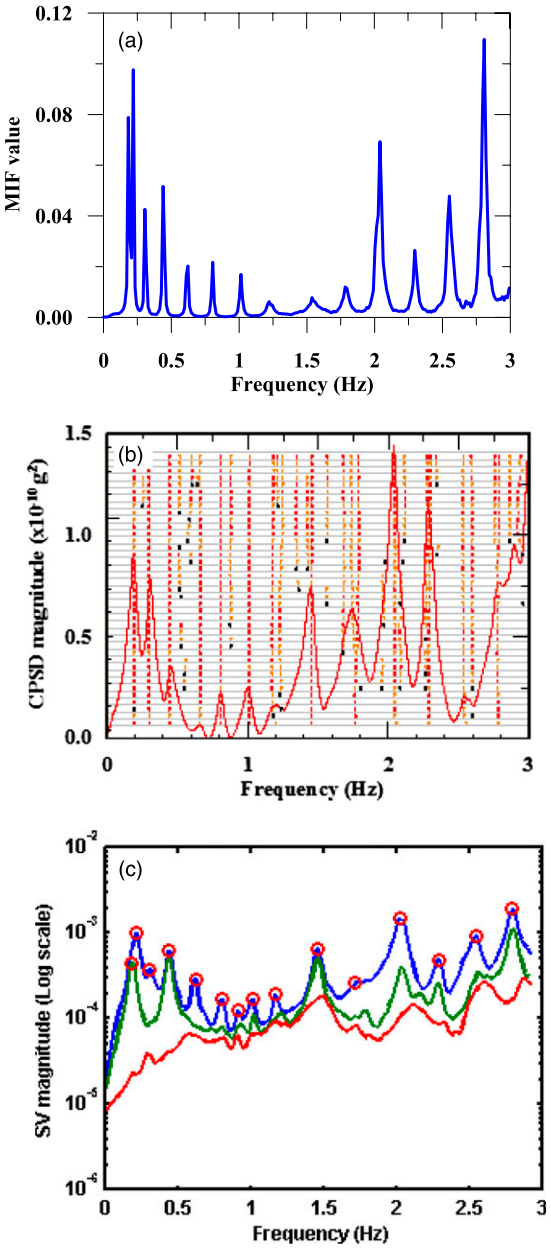


Fig. 10. Data postprocessing: (a) mode indicator function (MIF) from the Peak Picking method; (b) stabilization diagram from the PolyMAX method; and (c) complex mode indicator function (CMIF) plot

Correlation Analysis of Experimental and Analytical Results (Step 5)

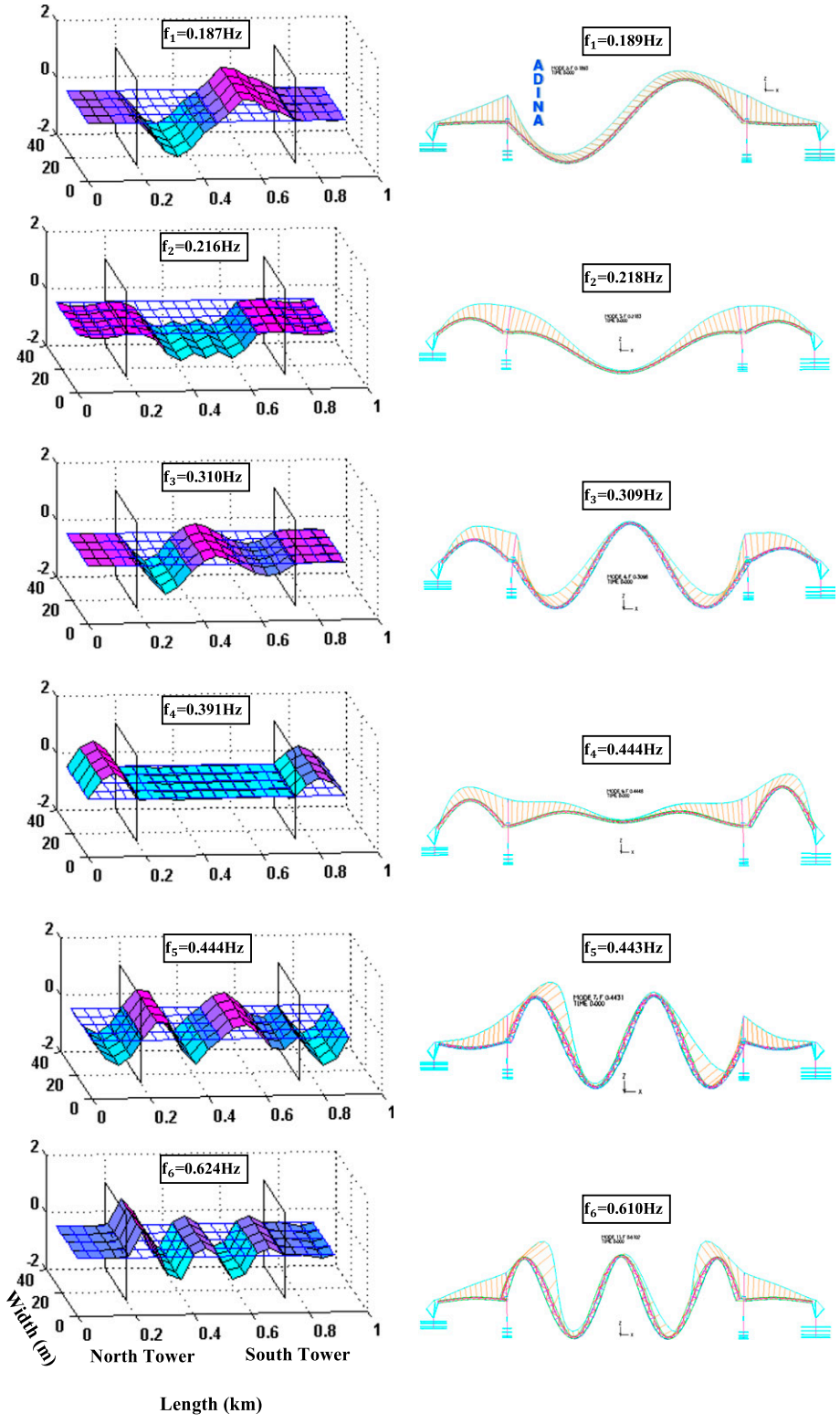
Step 5 of the St-Id process involves the correlation of the analytical model with the experimental results, which may or may not require additional calibration or updating. The natural frequencies determined from an eigenvalue analysis of the 3D FEM described previously are summarized in Table 1. These results were compared with those identified from the ambient vibration data using the three independent postprocessing methods. The first six vertical mode shapes of the entire stiffening truss obtained from the FE analysis and the ambient vibration test data are compared in Fig. 11 and show very good agreement with each other. The natural frequencies and mode shapes obtained in other directions (e.g., lateral and torsional) from the FE analysis and ambient vibration test also showed good correlation, though, for brevity, they are not provided here. The reasonable agreement observed between the experimental and analytical results indicated that the St-Id results from the a priori FEM did not reveal any blatant errors in modeling and test results. The seismic design criteria that were established for the project required that the first three vertical modes and first three lateral modes shall be compared with those obtained from ambient vibration measurements of the bridge for verification of the structural model. The constructed FEM satisfied this design criteria, and was deemed acceptable for use as a tool for supporting bridge maintenance and retrofit-related decisions. Given the objective of St-Id, a rigorous FEM updating effort was not required.

Table 1. Lateral and Vertical Mode Frequencies

	Experimental (Hz)			
Mode	PolyMAX	Peak picking	Complex mode indicator function	Finite element analysis (Hz)
1st vertical	0.187	0.183	0.195	0.189
2nd vertical	0.216	0.219	0.219	0.218
3rd vertical	0.310	0.305	0.305	0.310
4th vertical	0.391	0.378	0.378	0.444
5th vertical	0.444	0.440	0.439	0.443
6th vertical	0.624	0.623	0.610	0.610
1st transverse	0.150	0.159	0.146	0.152
2nd transverse	0.467	0.476	0.512	0.483
3rd transverse	—	0.794	0.793	0.756
4th transverse	1.009	1.025	1.025	1.054
5th transverse	1.451	1.538	1.550	1.630

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(a) Experiment (b) FE analysis

Fig. 11. Mode shape correlation

Tower Modal Parameter Identification

There are a number of important challenges to consider for reliable dynamic characterization of cable-supported bridge towers by ambient vibration testing. The first is that the ambient vibrations measured from these tower structures generally have smaller amplitudes than those measured from the suspended spans because of their larger stiffness, and the weak dynamic coupling between towers and spans. As a result, the tower response measurements may have lower signal-to-noise ratios and greater uncertainty than those

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obtained from the suspended spans. The second challenge is that the ambient dynamic excitation acting on the towers of a cable-supported bridge can be expected to have a fundamentally different character than the ambient dynamic excitation acting on the flexible spans. In ambient vibration testing, dynamic excitation mainly from traffic is transmitted from the spans to the towers. However, the spans of cable-supported bridges generally have relatively low frequencies (less than 5 Hz) and the dynamic excitation provided by traffic and wind acting on the spans will be filtered by the dynamic behavior of the spans and cables. Furthermore, the towers generally have a different bandwidth than the spans and thus the tower modes are typically poorly excited. As a result, significant uncertainty can exist in manipulating the measured ambient vibration responses of the towers to extract reliable modal parameters.

The roving accelerometers that were used in conjunction with the stationary reference accelerometers during the second and fourth test setups were primarily concentrated on the North and South Towers to characterize their ambient vibration responses. The experimental vibration analysis of the South Tower is presented as a typical example for the proposed tower identification methodology. The four measurement stations located in each tower during these two test setups were at different elevations along the height of the tower. Each measurement station in the towers included three accelerometers arranged in the longitudinal and transverse response directions as shown in Fig. 12. The variations of the measured tower vibration amplitudes for a single 12-h time period are illustrated in Fig. 13. It should be noted that 90% percentiles of the absolute values of the data in each segment (15-min data in each segment) are shown instead of maximum values to exclude the effects of any spikes in the measurements. Fig. 13 clearly illustrates that the vibration of the tower varies significantly with time, in which each curve denotes the measured acceleration response from a different elevation on the tower. This variation is most likely from changes in the volume of the traffic on the structure. As shown in Fig. 14, the intensities of the tower are different during periods of high traffic flow (morning commute) on the structure than they are during periods where the traffic flow is lighter (late at night). Given the uncertainty associated with identifying the modal parameters of the towers, a rigorous processing regime is needed.

To rapidly identify stable tower modes, an automated method for tower modal parameter identification was developed. To illustrate the method, three different tower vibration data sets are selected for processing. These data sets include: (1) an average of the measured acceleration time histories recorded during a 12-h period; (2) tower vibration data measured during 15-min low traffic flow on the structure [Fig. 14(a)]; and (3) tower vibration data measured during 15-min high traffic flow on the structure [Fig. 14(b)]. The same data preprocessing procedure that was used for the span vibration measurements was applied to condition and clean the raw measurement data, and the CMIF method was applied to identify the modal parameters from each data set. The following criteria were used to identify stable tower modes: (1) the variation in the frequencies identified from the three data sets is less than 5%; and (2) the modal assurance criterion values computed for the mode shapes identified from the three data sets are larger than 0.95. The automated data processing approach permitted stable poles to be more easily isolated from a large number of polynomial (numerical) poles. Table 2 summarizes the frequencies identified from the three data sets and the corresponding modal assurance criterion values computed for their mode shapes. The identified mode shapes are plotted from the averaged data set in Fig. 15. It is notable that the second bending and torsional mode shapes are repeated at different frequencies, which are likely the result of coupling between the motions of tower and span components. The phenomenon of repeated tower mode shapes has also been observed in the dynamic characterizations of other suspension-bridge towers (Abdel-Ghaffar and Scanlan 1985b; Grimmelsman 2006; Brownjohn et al. 2010) and is

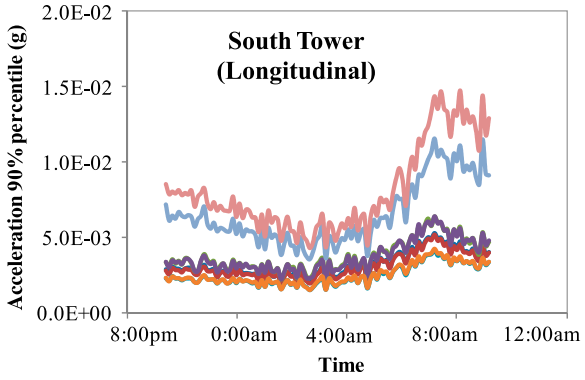


Fig. 13. Time variation of tower vibration amplitudes

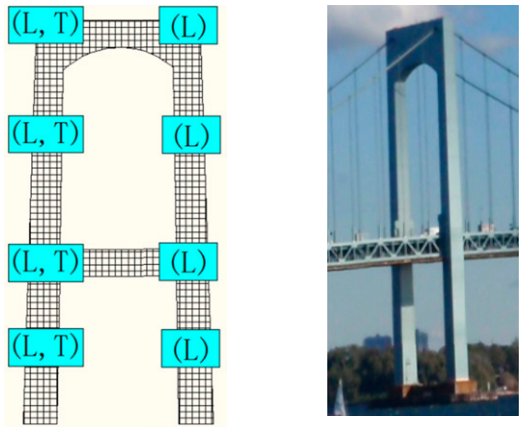


Fig. 12. Typical layout of longitudinal and transverse accelerometers installed on the towers

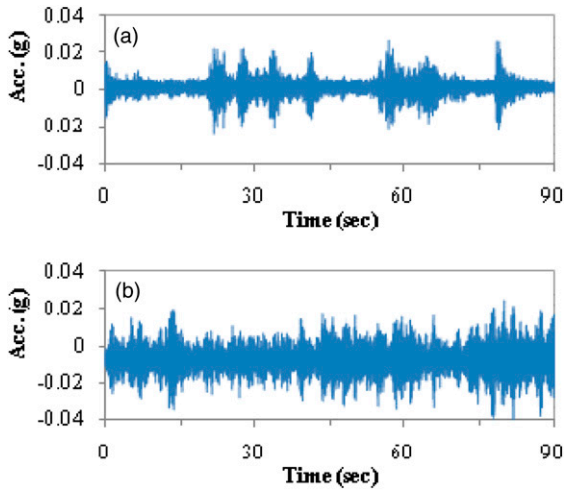


Fig. 14. Tower acceleration time histories: (a) weak excitation; (b) strong excitation

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Table 2. Identified Tower Frequencies in the Longitudinal Direction

Data set	Measures used	Mode 1	Mode 2	Mode 3						Mode 4			
Averaged data	f (Hz)	1.88	4.60	5.16	5.60	5.90	6.09	6.19	6.43	6.79	7.04	7.39	7.69
Weak excitation	f (Hz)	1.84	4.61	5.16	5.64	5.86	6.07	5.97	6.41	6.51	6.91	7.17	7.94
	Modal assurance criterion	0.97	0.92	0.98	0.99	0.99	0.97	0.98	0.98	0.98	1.00	0.99	0.99
Strong excitation	f (Hz)	1.88	4.43	5.14	5.69	5.90	6.06	5.69	6.42	6.76	7.14	7.36	7.69
	Modal assurance criterion	0.91	0.92	1.00	1.00	1.00	0.99	0.99	0.97	0.99	1.00	0.99	0.99

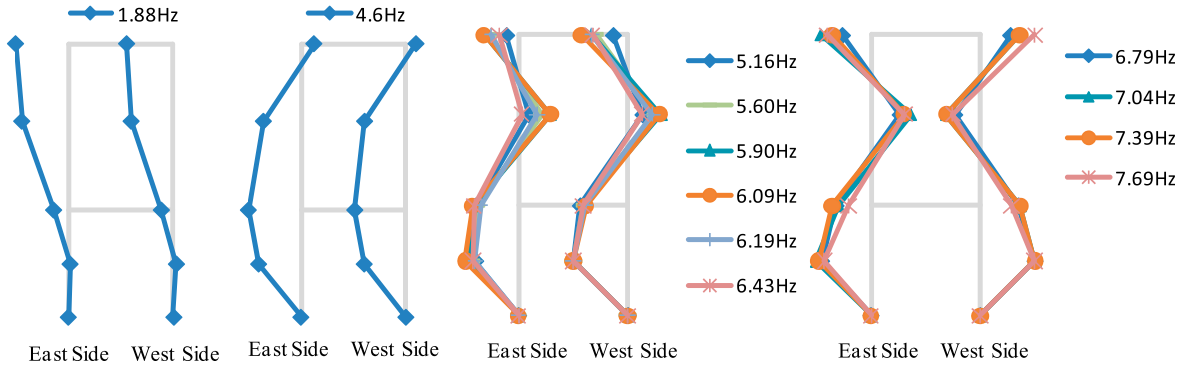


Fig. 15. Identified tower modal shapes at the longitudinal direction

most likely from the filtering of the ambient excitation by the suspended span and cables which introduces harmonics into the tower response measurements. FEM analysis results can help to distinguish between pure tower modes and tower vibrations incidental to the span vibrations (Abdel-Ghaffar and Scanlan 1985a, b), assuming that the FEM accurately predicts the pure tower modes. The use of multiple reference data analysis approaches may also help to identify the pure tower modes from the global response measurements (Grimmelsman 2006). The tower mode shapes generated using the different reference locations will tend to be consistent for pure tower modes. The signal-to-noise ratio will be maximum along the full height of the tower at the pure tower modes, whereas for span modes, the signal-to-noise ratio in the tower measurements will be largest only where the suspended spans and main cables connect to the towers (tower-top cables, roadway-level spans), which are analogous to discrete shakers exciting the tower. The writers believe that forced-vibration testing and analysis of the towers would provide additional estimates of the tower dynamic characteristics and the best means to further validate or refute the identification results obtained from ambient vibration testing. Forced-vibration testing might be especially necessary when the experiment purpose is seismic retrofit where the pure tower modes may be critical for designing and evaluating the efficacy of different retrofit scenarios. However, such testing obviously poses additional experimental and logistical challenges.

Conclusions and Discussion

An ambient vibration test, data interpretation, and FE analysis of a long-span suspension bridge has been presented, with a focus on mitigating various uncertainties in the experimental and data processing stages for reliable structural identification. The following conclusions are drawn based on the investigation thus far:

1. Operational modal analysis of long-span bridges may serve as a feasible tool for their long-term operational and health monitoring, although recognizing and mitigating sources of uncertainty is a critical step for ensuring reliable structural identification and health monitoring. A more effective means for mitigating these sources of uncertainty may be to better integrate analytical model simulation results and heuristics with the modal parameter identification process, or by employing controlled dynamic excitation in conjunction with the experimental characterization of some structural components such as the towers.
2. A multiple-reference testing scheme was developed and implemented to obtain high-resolution vibration response measurements. Including multiple reference locations spatially distributed across the structure improved the effectiveness and reliability of the modal identification procedure. Multiple-reference measurements also provided an effective means to identify the most likely candidates for the natural frequencies and mode shapes by visually comparing the consistency of the identified shapes constructed from different reference locations.
3. Various data preprocessing techniques have been developed to reduce data errors that are embedded in raw measurement data. Visual inspection of the signals, time window selection, digital filtering, cross-correlation construction, exponential windowing, and data averaging in time or frequency domain significantly reduce bias and random errors affecting the data quality and improve the reliability of the identified results. Three data postprocessing methods (Peak Picking, PolyMAX, and CMIF) were applied independently to the measurement data for modal parameter identification. The good agreement observed among the St-Id results from different methods verified the quality of the data from ambient test and the effectiveness of the entire data processing procedure.
4. Three-dimensional FE analysis was performed for the long-span suspension bridge. Careful modeling of complex structural components including connections and the soil-structure interaction aided in mitigating uncertainty from the analytical point of view, and produced good correlation between the results obtained from St-Id and experiment. Given the cloud of epistemic uncertainty that governs the behavior of large

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constructed systems, FE analysis results are helpful in judging the reliability of a field experiment, and integrating analysis and experiment together with heuristics is helpful for understanding how complex constructed systems actually behave.

5. The tower identification results included repeated mode shapes from the dynamic interactions between the superstructure components and the towers. This observation is quite important with respect to the assumed nature of ambient dynamic excitation for output-only modal analysis, and the writers believe this can easily lead to incorrect modal identification results for these important components of the overall structure, particularly if the measurement data are analyzed and evaluated independently of any analytical results. FEM analyses and multiple-reference measurements can provide a means to distinguish between pure tower modes and tower vibrations incidental to the span vibrations, but forced-excitation testing may provide the most reliable means for characterizing the dynamic characteristics of the towers.

Acknowledgments

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