Effects of Changing Ambient Temperature on Finite Element Model Updating of the Dowling Hall Footbridge

Babak Moaveni[1] and Iman Behmanesh[2]

ABSTRACT

In this paper, effects of changing ambient temperatures on finite element (FE) model updating of the Dowling Hall Footbridge are investigated. The Dowling Hall Footbridge is located on the Tufts University campus in Medford, Massachusetts. The footbridge is equipped with a continuous monitoring system that records vibration and temperature of the bridge once an hour or when triggered by large vibrations. Natural frequencies, mode shapes, and modal damping ratios of the structure are extracted from measured ambient vibration data using an automated data-driven stochastic subspace identification algorithm. The identified natural frequencies and mode shapes are then used for calibration/updating of an initial FE model of the bridge. However, the identified natural frequencies show significant variability with changing ambient temperature. This variability propagates through the FE model updating process and therefore yields uncertainty in the FE model updating results. A static polynomial model is estimated to represent the relationship between identified natural frequencies and measured temperatures. This model is then used to "remove" the temperature effects from the identified natural frequencies. Two sets of FE models are updated in this study based on 17 weeks of hourly-identified modal parameters, before and after removing the temperature effects. The proposed approach is successful in minimizing the effects of changing ambient temperature on FE model updating of the Dowling Hall Footbridge. Accounting for the temperature effects in the FE model updating process reduces the variability of temperature-sensitive updating parameters and therefore decreases the probability of missed identification of damage.

KEYWORDS: Temperature Effects on Structural Identification; Finite Element Model Updating; Continuous Structural Health Monitoring; System Identification

1 INTRODUCTION

Major structural failures in recent years have focused public attention on the need for improved infrastructure monitoring and maintenance [1]. In January 2009, the American Society of Civil Engineers (ASCE) issued its latest Report Card for America's Infrastructure [2], the fourth since 1998. This report asserts that our current infrastructure is poorly maintained, is unable to meet current and future demands,

[1] Corresponding author, Assistant Professor, Dept. of Civil and Environmental Engineering, Tufts University, Medford, Massachusetts 02155; E-mail: babak.moaveni@tufts.edu

[2] Graduate student, Dept. of Civil and Environmental Engineering, Tufts University, Medford, Massachusetts 02155.

and is in some cases, unsafe. Deteriorating conditions and inflation have added hundreds of billions to the total cost of repairs, needed upgrades and replacements. In this report, bridges receive a grade of **C**. More than 26% of the nation's bridges are either structurally deficient or functionally obsolete. An estimated $17 billion annual investment is needed to substantially improve current bridge conditions. As part of the solution, ASCE proposes that owners of the infrastructure should be required to perform ongoing evaluations and maintenance to keep them functioning at a safe and satisfactory level. To manage the nation's infrastructure system, it is essential to understand the true state of structural health and rate of degradation of each significant bridge structure. This often cannot be determined from visual inspections alone. Vibration-based structural health monitoring (SHM) provides information that is complementary to visual inspections.

The basis for vibration-based SHM is that the dynamic parameters of a structure are functions of its physical properties (mass, damping, and stiffness). Therefore, changes in these physical properties due to "structural damage" will be reflected by changes in dynamic parameters such as natural frequencies, damping ratios and mode shapes. Numerous methods for vibration-based damage assessment of structures have been proposed in the literature. Extensive reviews on vibration-based damage identification have been provided in [3-5]. Sensitivity-based FE model updating is among these methods [6-7]. In this method, the physical parameters of a FE model of the structure are updated to match the measured modal properties of the structure as damage evolves, and the updated modeling parameters are used to detect, locate, and quantify damage. In some recent studies, FE model updating methods have been successfully applied for damage identification of real-world, large-scale structures [8-11]. However, the accuracy and spatial resolution of the damage identification results depend significantly on the accuracy and completeness of the identified modal parameters [12]. The estimation variability/uncertainty of the modal parameters can be influenced by several factors. One of the most important factors (and one of the few that can be measured) is changing environmental conditions, such as ambient air temperature [13-17]. Therefore, separation methods are needed to remove the effects of changing ambient temperatures from system identification (e.g., natural frequencies) and damage identification (e.g., model calibration factors) results. Even though researchers have underlined the importance of environmental effects in structural identification, little work has been done to quantify these effects on damage identification results.

The focus of this study is (1) to quantify the variation of FE model updating results for the Dowling Hall Footbridge induced by the measured ambient temperatures, and (2) to reduce this variation through removing the temperature effects from identified natural frequencies. The paper is organized in the following order. In Section 2, the Dowling Hall Footbridge and its continuous monitoring system are introduced. A brief review of the automated system identification process and modeling of the identified natural frequencies versus measured temperatures are provided in Section 3. In Section 4, the initial and reference FE models of the footbridge as well as the sensitivity based FE model updating process used in this study are reviewed. Two series of FE model updating are performed using the hourly-identified natural frequencies before and after removing the temperature effects. Variation in the FE model updating results before and after removing the temperature effects and a discussion of the observations are presented in Section 5. Finally, some concluding remarks are offered in Section 6.

2 DOWLING HALL FOOTBRIDGE

2.1 Footbridge Structure

The Dowling Hall Footbridge is located on the Medford campus of Tufts University. Figure 1 shows the south view of the footbridge. The bridge consists of two 22 m spans and it is 3.9 m wide. It connects Dowling Hall on its eastern end to Tufts main campus on its western end. The footbridge is supported by an abutment on the west side and by two piers, one in the mid-span and one on the east side near Dowling Hall. The pier heights are 3.8 m and 11.4 m in the mid-span and eastern side, respectively. The footbridge is composed of a steel frame with a reinforced concrete deck. The bottom and top chords of the frame are made from A992 steel type TS10×6×5/16 and TS10×6×3/8, respectively. The stringers are TS10×4×5/16 except at the piers, which are W10×60. All diagonal and vertical members are TS8×6×5/16 except at the supports, where the vertical members are TS8×6×1/2. The footbridge deck is equipped with a pipe-glycol heating system to prevent snow and ice buildup during winter time. More details about the Dowling Hall Footbridge can be found in [18].

Figure 1. South view of Dowling Hall Footbridge

2.2 Continuous Monitoring System

A continuous monitoring system was designed and deployed on the Dowling Hall Footbridge in the fall of 2009 and has been providing continuous data since January 2010. The monitoring system consists of eight accelerometers and ten thermocouples, which are connected to a data acquisition device and a communication system that transfers the measured data wirelessly to a host computer in the Department of Civil and Environmental Engineering at Tufts University. The monitoring program continuously samples the acceleration channels at a 2,048 Hz sampling rate and the temperature channels at 1 Hz. A five-minute data sample is recorded once each hour, beginning at the top of the hour or when the one-second root-mean square (RMS) value of an acceleration channel exceeds 0.03 g. Sample recording can also be triggered manually. Before the permanent continuous monitoring system was installed on the bridge, a set of dynamic tests was conducted in April 2009 for system design purposes. Twelve temporarily installed accelerometers were used for this preliminary test. The objective of the test was to determine the level of bridge response amplitude due to ambient excitation and to estimate the natural frequencies and mode shapes of the footbridge. Knowledge of the mode shapes allowed sensor location planning to avoid placement of sensors at modal nodes. Figure 2 shows the identified modal parameters of the first six most excited vibration modes based on the preliminary test data. In this plot, mode shapes are interpolated between the sensor locations (indicated by empty circles) using a cubic spline.

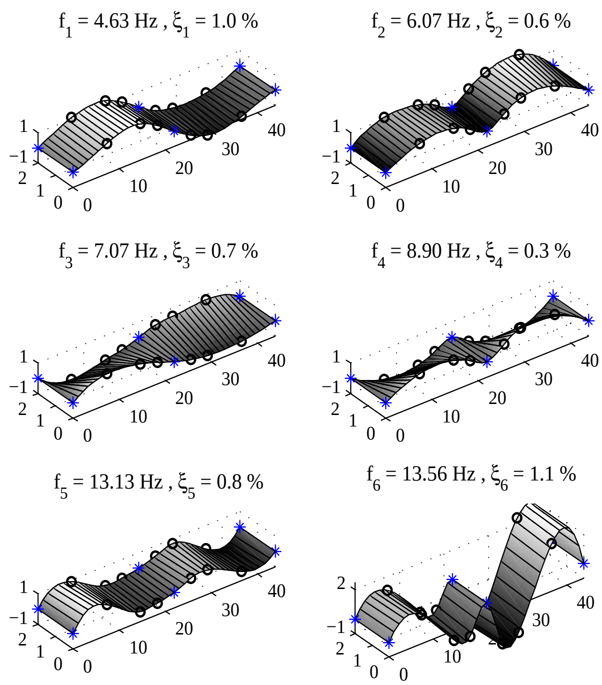
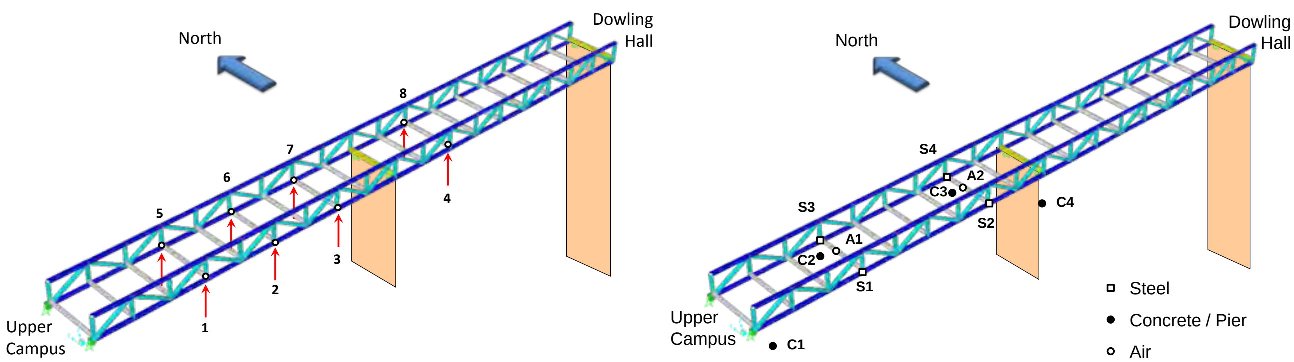


Figure 2. Identified modal parameters from preliminary test data

PCB Piezoelectronic model 393B04 accelerometers were selected as the vibration sensors. The eight accelerometers were mounted to aluminum L-brackets that were fixed to the underside of the footbridge using epoxy. The layout of the accelerometers is shown in Figure 3a. It is worth noting that installation of instrumentation on the eastern side of the bridge, nearest to Dowling Hall, was outside the scope of the project due to the height above ground. The temperature sensors in this system are type T thermocouples manufactured by Omega Engineering. Layout of the ten thermocouples in the monitoring system is shown in Figure 3b. The system monitors air temperature at two locations ("A1" and "A2"), steel temperature at four locations ("S1" to "S4"), pier temperature at two locations ("C1" and "C4"), and bridge deck temperature at two locations ("C2" and "C3"). The National Instruments cRIO-9074 integrated chassis/controller is the core component of the data acquisition system. Two National Instruments NI-9234 four-channel dynamic signal acquisition modules measure the acceleration response of the footbridge. One National Instruments NI-9213 sixteen-channel thermocouple input module monitors the temperature sensors. The cRIO-9074 and other equipment were installed in a weatherproof enclosure located under the bridge. Figure 4 shows the enclosure and equipment layout. More information about design and deployment of this continuous monitoring system can be found in [19].



(a) (b)

Figure 3. Layout of (a) accelerometers, and (b) thermocouples on the bridge

Figure 4. View inside the enclosure

3 SYSTEM IDENTIFICATION

3.1 Automated Operational Modal Analysis

The data-driven stochastic subspace identification (SSI-Data) method is applied to the cleaned ambient vibration data for modal identification of the footbridge [20]. The data cleansing process consists of: (1) down-sampling from 2,048 Hz to 128 Hz for computational efficiency, (2) filtering between 2 and 55 Hz using a Finite Impulse Response (FIR) filter, (3) removing voltage spikes in the time domain, and (4) re-filtering to remove any high frequency components introduced by cleaning the voltage spikes. Multiple reference channels are used in the application of SSI-Data [21]. Channels 1, 2, 3, 5, 6, and 7 (see Figure 3a) are used as references; Channels 4 and 8 are not considered references due to their larger noise levels (these channels are the farthest from the enclosure).

The system identification process was automated using stabilization diagrams, considering system orders of 2-96 (corresponding to 1-48 modes). At each step, modes identified at the current system order are compared with modes identified at the previous system order. If the frequency matches within 1%, the damping ratio matches within 30% (relative), and the mode shapes match within 95% using the Modal Assurance Criterion (MAC) metric [22], the mode is judged to be “stable” between the two system orders. A mode that remains stable for seven successive system orders is considered a physical mode of the system. In addition, modes with identified damping ratios less than zero or higher than 2% (damping was found to be significantly lower than 2%) are also excluded. The best system order is then determined by finding the order that returns a maximum number of physical modes of interest [19]. Figure 5 shows the natural frequencies of the first six identified modes of the footbridge identified during the 17-week monitoring period considered in this study (January 4 to May 1, 2010), while Table 1 reports the statistics (mean and coefficient-of-variation) of modal parameters identified during this period. The MAC values are computed between each identified mode shape and the “reference” mode shape. The modal parameters extracted from data recorded at 7:00pm on April 19, 2010 are considered reference modal parameters. This choice of date and time for the reference modal parameters and temperatures is due to the following facts: (1) the corresponding measured temperatures are close to the average temperatures during the warm season when the identified natural frequencies are less sensitive to the temperature effects, (2) all the vibration modes considered in this study are well excited at this time and therefore are identified accurately, and (3) the corresponding natural frequencies are close to the average natural frequencies of the footbridge in warm weather. The temperature of the steel at sensor S3 for the reference data set was recorded as 16°C.

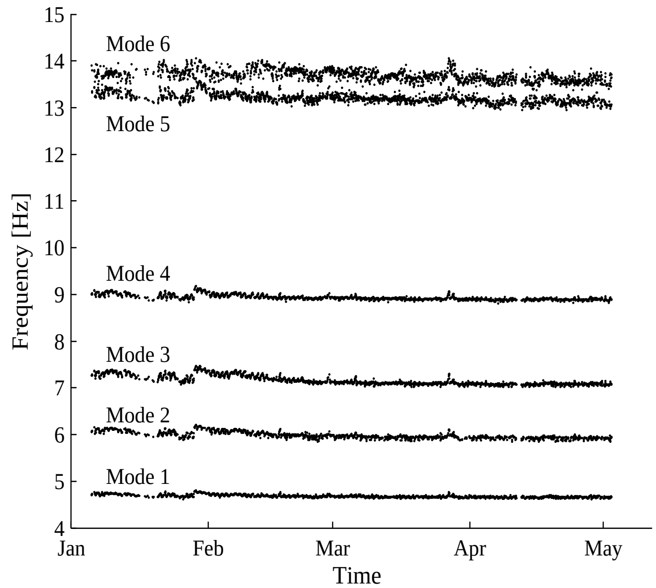


Figure 5. Variation of identified natural frequencies versus time during the 17-week monitoring period

Table 1. Statistics of modal parameters before and after removing temperature effects

		Mode					
		1	2	3	4	5	6
Natural frequencies [Hz]	Mean	4.68	5.98	7.16	8.93	13.20	13.69
	COV [%]	0.67	1.16	1.39	0.63	0.74	0.87
Damping ratio [%]	Mean	0.3	0.4	0.3	0.2	0.5	0.7
	COV	0.92	0.72	0.60	0.78	0.57	0.48
MAC	Mean	0.99	1.00	1.00	1.00	0.94	0.97
	COV [%]	0.015	0.012	0.015	0.031	0.124	0.044
Temperature-removed frequencies [Hz]	Mean	4.66	5.93	7.08	8.89	13.14	13.63
	COV [%]	0.31	0.47	0.40	0.21	0.43	0.61
Reduction in frequency COV		53.8%	59.3%	71.3%	66.2%	42.3%	29.7%

3.2 Removing Temperature Effects from Identified Natural Frequencies

From Figure 5 and Table 1, it can be seen that the identified natural frequencies show significant variability during the monitoring period. This variability could be due to several factors such as measurement noise, estimation error, amplitude of excitation, additional mass due to live loads, and ambient temperature. Among these, ambient temperature is the most influential factor that its effects can be accounted for. Figure 6 shows the identified natural frequencies of the six considered modes plotted versus the temperature measurement of sensor S3. In general, the natural frequencies increase as the temperatures decrease. However, this increase is much more significant when temperatures go below the freezing point.

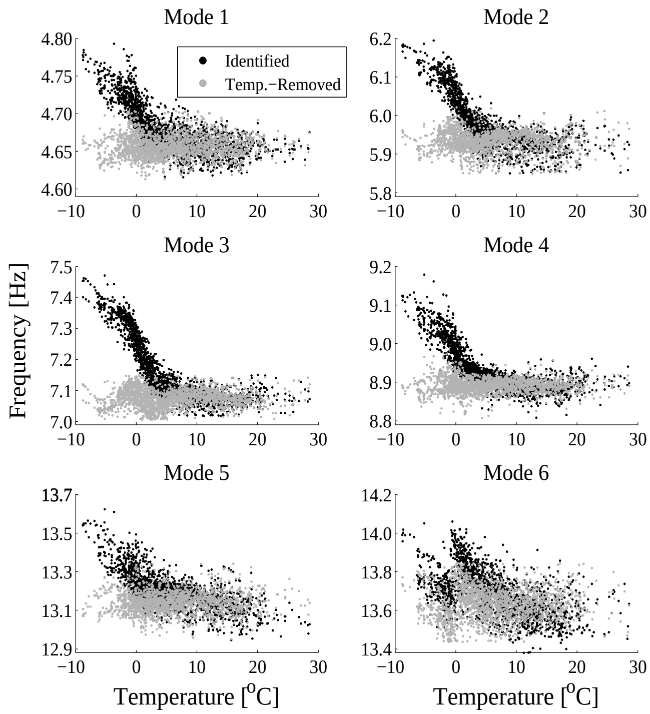


Figure 6. Variation of identified natural frequencies versus temperature at sensor S3 before and after removing temperature effects

In a recent study [16], the relationship between identified natural frequencies of the Dowling Hall Footbridge was modeled as a function of measured temperatures using different classes of models such as static linear, bi-linear, quadratic, third order, and fourth order polynomials as well as an auto-regressive with exogenous input (ARX) dynamic model. A fourth-order regression model, as shown in Eq. (1), was found to best fit this relationship. It is worth noting that the regression model used in this study is slightly different from the one presented in [16].

$$f_i^{\,j} = \beta_o^j + \sum_{k=1}^{n_{var}=3} (T_{ik}\beta_k^j + T_{ik}^2\beta_{n_{var}+k}^j + T_{ik}^3\beta_{2n_{var}+k}^j + T_{ik}^4\beta_{3n_{var}+k}^j) + e_i^j \tag{1}$$

In this equation, T_k are measurements from sensors S3, C1, and C2, corresponding to steel, pier and concrete deck temperatures, respectively; e denotes the estimation error; β factors are the coefficients of the model; i denotes the time index; and j represents the mode number. The coefficients of Eq. (1) are recalculated for the 17 weeks of data used in this study. The temperature effects are then removed from the identified natural frequencies as shown in Eq. (2).

$$\tilde{f}_i^{\,j} = f_i^{\,j} - \sum_{k=1}^{n_{var}=3} (T_{ik}\beta_k^j + T_{ik}^2\beta_{n_{var}+k}^j + T_{ik}^3\beta_{2n_{var}+k}^j + T_{ik}^4\beta_{3n_{var}+k}^j) + \sum_{k=1}^{n_{var}=3} (\overline{T}_{ik}\beta_k^j + \overline{T}_{ik}^2\beta_{n_{var}+k}^j + \overline{T}_{ik}^3\beta_{2n_{var}+k}^j + \overline{T}_{ik}^4\beta_{3n_{var}+k}^j) \qquad (2)$$

In this equation, the temperature-removed natural frequencies $\tilde{f}_i^{\,j}$ are computed by subtracting the measured temperature terms, T_k of Eq. (1), from the identified natural frequencies, $f_i^{\,j}$, and then adding the reference temperature terms, $\overline{T}_k$. Reference temperatures correspond to the measured data used for calibration of the reference FE model as described in Section 4.3. Data recorded at 7pm on April 19, 2010 is selected as the reference data set in this study. This reference data set is selected such that the reference FE model has limited/reasonable modeling errors. The statistics (mean and coefficient-of-variation) of natural frequencies after removing the temperature effects are also reported in Table 1. The variability in identified natural frequencies of all six modes has been significantly reduced by removing the temperature effects. The third mode has the largest reduction in its coefficient-of-variation (COV) with 71.3%, while the sixth mode has the smallest reduction with 29.7%. Also in Figure 6, the temperature-removed natural frequencies are plotted versus the temperature measurement of sensor S3. From this figure, it can be observed that the temperature-removed frequencies have significantly less sensitivity to the variation of temperature than the identified natural frequencies before removing temperature effects, especially for modes 1 to 5. Natural frequencies of mode 6 show large variability in both cases due to larger estimation errors of this mode. In the following sections, two sets of natural frequencies, before and after removing temperature effects, will be used for FE model updating of the Dowling Hall Footbridge and the results will be compared.

4 INITIAL AND REFERENCE FE MODELS, AND THE FE MODEL UPDATING PROCESS

This section briefly reviews modeling of an initial FE model of the Dowling Hall Footbridge, the sensitivity-based FE model updating process used, and calibration of a reference FE model for the footbridge. FE model updating is a nonlinear, least-squares optimization problem in which selected parameters of the FE model (e.g., element stiffness values) will be updated/calibrated to minimize the discrepancies between experimentally measured and FE computed response features such as modal parameters. The first step in the updating process consists of calibrating an initial FE model of the structure, created based on design information, to a reference FE model that corresponds to “as built” properties of the structure in its undamaged/baseline state. Consequently, the reference FE model can be updated to match the modal parameters identified at different states of a structure’s health for damage identification.

4.1 Initial Finite Element Model

An initial FE model of the footbridge is created based on the design drawings and visual inspection of the footbridge, using the MATLAB-based structural analysis software FEDEASLab [23]. Careful attention is paid to model geometry since the dimensions in the design drawings do not exactly match the actual dimensions for a few components of the structure. The FE model consists of 197 nodes, 272 frame elements, and 80 shell elements. All the steel members are modeled using frame elements with an elastic modulus of 2.0×10^{8} kN/m^2 and a density of 7,849 Kg/m^3. The concrete deck is modeled using 10 cm-thick shell elements with an elastic modulus of 1.4×10^{7} kN/m^2 and a density of 2,403 Kg/m^3. The total mass of the footbridge is estimated as 64.0 metric tons. The mass of railings and curb on each side of the bridge deck are added to the nodes connecting the bottom chord elements. For the support at the campus side, all rotational degrees of freedom (DOFs) as well as translational DOFs in longitudinal and transversal directions are restrained, but a spring is used to model the vertical flexibility. Translational flexibility of support at two piers is modeled by three springs in longitudinal, transversal and vertical directions for each pier, with initial values of each spring obtained from separate FE models of piers. The rotational DOFs at the connection of footbridge to piers are free.

It is worth noting that creating a detailed and accurate initial FE model is key to successful FE model updating for damage identification because the model updating cannot account for modeling errors [24]. For example, an initial model was made without considering the offset between the concrete shell elements and the centerline of stringers (there is a 17.8 cm offset between deck and stringers). Large modeling errors, especially in estimation of higher (fifth and sixth) modal frequencies, were observed that could not be resolved by model updating. This is due to the fact that only a small number of model parameters can be updated from the data obtained from the limited number of sensors, and not all modeling parameters are observable from the measurements (i.e., some are not sensitive to measurements).

4.2 FE Model Updating Process

In the FE model updating process, a limited number of physical parameters of the FE model will be updated so the modal parameters (natural frequencies and mode shapes) of the FE model will match their experimentally identified counterparts. The updating parameters used in this study are the substructure updating factors, a_i , which are defined as the relative changes in the effective moduli of elasticity of elements in considered substructures (i.e., groups of elements):

$$a_i = \frac{E_i^0 - E_i^{updated}}{E_i^0} \tag{3}$$

with E_i^0 and $E_i^{updated}$ corresponding to the reference and updated effective moduli of elasticity of the elements in substructure i. In this study, the footbridge is divided into five substructures as shown in Figure 7. The substructures are defined based on the locations of accelerometers.

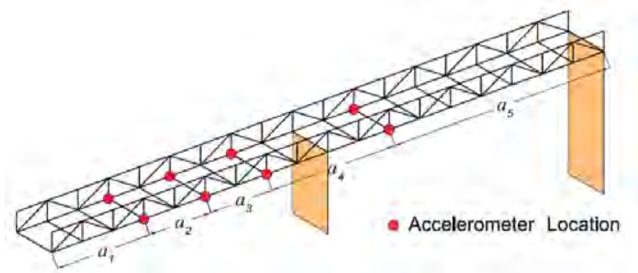


Figure 7. Considered substructures along the footbridge

The objective function to be minimized in this study is defined as:

$$f(\mathbf{a}) = \frac{1}{2}\mathbf{r}(\mathbf{a})^{\mathrm{T}}\mathbf{W}\mathbf{r}(\mathbf{a}) + \frac{1}{2}\mathbf{a}^{\mathrm{T}}\mathbf{W}_{\mathrm{a}}\mathbf{a} \tag{4}$$

In Eq. (4), $\mathbf{a}$ is the vector of updating parameters (substructure updating factors), $\mathbf{r}(\mathbf{a})$ is the residual vector containing the differences between the experimentally identified modal parameters and those computed from the FE model, $\mathbf{W}$ is a diagonal weight matrix, and $\mathbf{W_a}$ is a diagonal regularization weight matrix. The regularization weights are zeros in the case of calibrating the reference model. Otherwise, the diagonal components of $\mathbf{W_a}$ are 0.001 for all the substructures. The residual vector, $\mathbf{r}(\mathbf{a})$, in the objective function defined in Eq. (4) contains:

$$\mathbf{r}(\mathbf{a}) = \begin{bmatrix} \mathbf{r}_f(\mathbf{a}) \\ \mathbf{r}_s(\mathbf{a}) \end{bmatrix} \tag{5}$$

in which $\mathbf{r}_f(\mathbf{a})$ and $\mathbf{r}_s(\mathbf{a})$ represent the eigenfrequency and mode shape residuals, respectively, and their components for mode i are defined as:

$$r_f^i(\mathbf{a}) = \frac{\lambda_i(\mathbf{a}) - \tilde{\lambda}_i}{\tilde{\lambda}_i} \tag{6}$$

$$\mathbf{r}_s^i(\mathbf{a}) = \frac{\mathbf{\Phi}_i(\mathbf{a})}{\left\|\mathbf{\Phi}_i(\mathbf{a})\right\|} - \frac{\tilde{\mathbf{\Phi}}_i}{\left\|\tilde{\mathbf{\Phi}}_i\right\|} \tag{7}$$

In Eq. (6), $\lambda_i(\mathbf{a}) = \left(2\pi f_i(\mathbf{a})\right)^2$ and $\tilde{\lambda}_i = \left(2\pi \tilde{f}_i\right)^2$ where $f_i(\mathbf{a})$ and $\tilde{f}_i$ denote the FE computed and experimentally identified natural frequencies, respectively; while $\mathbf{\Phi}_i\left(\mathbf{a}\right)$ and $\tilde{\mathbf{\Phi}}_i$ refer to the FE computed and experimentally identified mode shapes. Natural frequencies and mode shapes of the first six modes are used in the residual vector and therefore, the residual vector consists of 6 eigenfrequency residuals and 48 mode shapes residuals (6 modes x 8 sensors).

In this study, diagonal components of the weight matrix **W** are defined based on the square inverse coefficient-of-variation (COV) of the temperature-removed frequencies over the 17 weeks of data. The relative weights of eigenfrequency residuals are 1.0, 0.2, 1.0, 1.0, 0.2, and 0.1 for modes one to six, respectively. Each component of mode shape residual has the weight of its corresponding eigenfrequency divided by the number of mode shape components (eight). Sensitivities of the eigenfrequencies and mode shapes to updating parameters are computed as proposed in [25]. Before performing the current model updating study on the Dowling Hall Footbridge, a numerical investigation was performed to check the observability of the updating parameters with respect to identified modal parameters. The updating parameters were found to be observable from the identified modal parameters with realistic levels of uncertainty which in turn justified the FE model updating of this footbridge based on experimental data.

A standard Trust Region Newton Method [26] is used to minimize the objective function of Eq. (4). The method is available in the MATLAB optimization toolbox [27]. The substructure updating factors were constrained in the range of -2 to 0.90 during the updating process. The upper-bound and lower-bound constrains are not very strict as they allow a 90% loss or a 200% increase in the stiffness of each substructure, which are much larger than the expected changes in the updating stiffness values. The optimization process was performed using the “fmincon” function in MATLAB, with Jacobian and first-order estimates of the Hessian matrices calculated analytically, based on the sensitivities of the modal parameters. The maximum number of iterations for each optimization is limited to 30. In general, the objective function is not convex. However, this function can be considered convex in the vicinity of its global minimum. When calibrating the reference model, several initial points were used in the optimization process to verify the global optimum is reached. During calibration of the reference model based on the hourly measured data, only one initial point was used in each optimization assuming that the initial point of zero is close enough to the global minimum so the objective function is convex in the considered region. Given the fact that the updating factors are reasonably small (i.e., zero initial points are not far from the optimum values), global optimization was reached in most cases. An updated FE model is not accepted (flagged) if the minimum objective function is larger than 0.01. This indicates that optimization needs more iteration steps or needs another initial point to reach the global minimum.

4.3 Reference Finite Element Model

Reference or baseline FE model of the Dowling Hall Footbridge is obtained in a two-step process: (1) the initial FE model, described in Section 4.1, is updated with two updating factors corresponding to the mass of deck and bottom chords, and three updating factors corresponding to the vertical stiffness of supports, and (2) the model from step 1 is updated again using the five substructures shown in Figure 7. The first step was performed due to the large uncertainties in the estimated mass of deck and nonstructural components (e.g. railing, curb), as well as support stiffness. It is worth noting that a reference model obtained from performing the second step alone resulted in significantly large updating factors that cannot be justified physically. This is due to the fact that, in this case, the updating factors compensate for the modeling errors in structural components that are not updated. Table 2 compares the reference modal parameters with those of the initial and reference FE models. From this table, it can be seen that the difference between identified and FE model computed natural frequencies of modes 1, 3, and 4, which have the most weight in the updating process, are significantly reduced after step 1. Also, excellent MAC values, at least 0.98, are obtained after conducting step 1. The reference FE model obtained after the two-step process has low modeling errors, which is a key factor in successful model updating.

Table 2. Modal parameters of the initial and reference FE models and those identified on April 19, 2010 at 7:00pm

	Mode					
	1	2	3	4	5	6
Initial model natural freq. [Hz]	4.51	5.98	6.34	8.18	13.02	13.77
Ref. model natural freq. [Hz], Step 1	4.65	6.09	7.03	8.91	13.18	13.47
Ref. model natural freq. [Hz], Step 2	4.66	6.05	7.09	8.87	13.30	13.53
Identified natural freq. [Hz]	4.68	5.98	7.07	8.89	13.14	13.65
MAC (initial model and identified)	1.00	0.99	0.98	0.99	0.93	0.92
MAC (ref. model and identified), Step 1	1.00	0.99	0.98	1.00	0.99	0.99
MAC (ref. model and identified), Step 2	1.00	0.99	0.99	1.00	0.99	0.99

In step 1, the nodal masses of the deck and nonstructural components (applied at the bottom chords) are reduced by 12% and 83%, respectively, i.e., the density of the deck and masses of railings and curb in the initial model were overestimated. The vertical stiffness of the supports are calibrated as 1.1×10^5 kN/m, 4.4×10^5 kN/m, and 5.4×10^6 kN/m for the campus plaza abutment, middle pier, and the pier nearest to Dowling Hall, respectively. The stiffness of the campus plaza abutment proved to be significantly less than its initial value (7.1×10^5 kN/m). Stiffness of the middle support is close to its initial estimate

(6.1×10^5 kN/m) based on 3-D FE modeling, while the vertical stiffness of the pier at the Dowling Hall side is updated to a much larger value than its initial estimate (2.1×10^5 kN/m) based on the assumption that there is an expansion join at the connection between the bridge deck and the Dowling Hall building. However, visual inspection revealed that connections of the bridge deck to the Dowling Hall restrict the vertical motion between the bridge and the Dowling Hall at this point. This can be verified by the observed deformation of the top chord of the bridge near the Dowling Hall end due to settlement of the pier (Figure 8). In the second step, the same substructures that are used in each updating during the 17-week monitoring period are considered, resulting in substructure updating factors of -12%, 31%, -45%, 1%, and -3%, for substructures one to five, respectively. Note that the large updating factors at substructures 2 and 3 can be attributed to the modeling errors. In both steps, FE model calibration is performed using the reference modal parameters.

Figure 8. Deformation of top chord at its connections to Dowling Hall building

5 FE MODEL UPDATING RESULTS

5.1 Before Removing the Temperature Effects

Vibration response of the footbridge is recorded once every hour, which should provide $24\times7=168$ sets of modal parameters per week or $168\times17=2{,}856$ sets of modal parameters over the 17-week monitoring period considered in this study. However, the number of model updating runs during this period is only 2,088. The missed updating runs can be mostly attributed to technical problems with the monitoring system and system identification errors. The technical problems include network connection failure, electrical outage of the main computer, and other similar issues. Low signal-to-noise ratio of measured data and estimation uncertainty of the system identification method used are the main sources of identification errors. Detailed information about the rate of weekly data loss due to different sources is provided in Table 3. The system identification errors result in missed or poor identification of one or several of the six vibration modes considered in the model updating process. Modes 1, 3, and 4 are the most reliably identified modes, as they are identified in most cases and their natural frequencies show smaller variations. Therefore, the modal parameters of these three modes are always used in the model updating process, while modal parameters of modes 2, 5, and 6 are only used in a subset of the updating runs. It is noteworthy that each model updating run takes approximately 30 minutes of CPU time on a PC with dual core Intel Xeon 2GHz processor resulting a total of 1,044 hours of computation for each set of model updating results (before and after removing temperature effects). The computations for this study were performed using Tufts high-performance computing research cluster and were completed in almost one week.

Table 3. Data loss breakdown during the 17-week monitoring period

Week	CD[1]	TP[2]	IE[3]	UP[4]	TL[5]	TL (%)	AD[6]
Jan 4 – Jan 10	128	40	13	14	67	40	101
Jan 11- Jan 17	92	76	20	5	101	60	67
Jan 18 – Jan 24	110	58	13	4	75	45	93
Jan 25 – Jan 31	162	6	23	19	48	29	120
Feb 1 – Feb 7	166	2	8	4	14	8	154
Feb 8 – Feb 14	167	1	8	15	24	14	144
Feb 15 – Feb 21	165	3	10	20	33	20	135
Feb 22 – Feb 28	168	0	20	9	37	17	139
Mar 1 – Mar 7	168	0	15	13	28	17	140
Mar 8 – Mar 14	167	1	21	8	30	18	138
Mar 15 – Mar 21	165	3	22	4	29	17	139
Mar 22 – Mar 29	168	0	23	17	40	24	128
Mar 30 – Apr 4	167	1	33	7	41	24	127
Apr 5 – Apr 11	157	11	34	7	52	31	116
Apr 12 – Apr 18	168	0	45	15	60	36	108
Apr 19 – Apr 25	168	0	34	8	42	25	126
Apr 26 – May 1	167	1	41	13	55	33	113

[1]: number of hourly collected datasets
[2]: number of missed datasets due to technical problems (168 - CD)
[3]: number of missed modal parameters due to identification errors
[4]: number of missed updating cases due to not reaching global minimum
[5]: total number of lost datasets (CD - IE)
[6]: number of available datasets for model updating (168 - TL)

A measured data set is not used for FE model updating if one of the following is true: (a) the natural frequency of mode 1, 3, or 4 is not identified; (b) the MAC value of mode 1, 3, or 4 with respect to the mode shape of the reference FE model is less than 0.90; or (c) error between the identified natural frequency of mode 1, 3, or 4 and their simulated counterparts from Eq. (1) is more than 1%. In the 2,088 updating cases, modes 1, 3, and 4 are always used. Mode 2 is included in 1,745 cases, mode 5 in 1,770 cases, and mode 6 in 1,798 cases. Identified modal parameters of modes 2, 5, or 6 are not used in the updating process if one of the following is true: (a) the natural frequency is not identified; (b) the MAC value with corresponding mode shape from the reference FE model is less than 0.80; or (c) the error between the identified natural frequency and the corresponding simulated frequency from Eq. (1) is more than 1.5%.

Table 4 reports the statistics of the 2,088 substructure updating factors for the 5 considered substructures obtained during the monitoring period. Variability of the updating parameters is found to be much larger than the variability of identified natural frequencies. This is due to the fact that the sensitivities of updating parameters to the natural frequencies used in the updating process are very large, i.e., small changes in natural frequencies result in significant changes in the stiffness of some substructures. It can also be observed that the updating factors of substructures 1-3 have larger variability than those of substructures 4-5. Note that the standard deviations of the updating factors are close to the COV of the updating parameters (moduli of elasticity). These two quantities are identical when the updating factors are zero-mean. The histograms of the substructure updating factors are plotted in Figure 9a. Figure 10 shows the variation of updating factors versus the measured steel temperature by sensor S3 (black dots). It can also be observed that the updating factors of substructures 1, 3, and 5 have higher correlations with temperature, especially below freezing point. The updating factors decrease (i.e., stiffness increase) as temperatures go below freezing.

Table 4. Statistics of the 2088 estimated substructure updating factors

	Substructure				
	1	2	3	4	5
	Before removing temperature effects				
Mean	-0.078	0.065	-0.065	-0.014	-0.018
Maximum	0.174	0.272	0.338	0.075	0.034
Minimum	-0.329	-0.347	-0.655	-0.211	-0.155
STD	**0.073**	**0.059**	**0.143**	**0.021**	**0.025**
	After removing temperature effects				
Mean	-0.043	0.057	-0.017	-0.009	0.002
Maximum	0.181	0.277	0.358	0.106	0.037
Minimum	-0.226	-0.399	-0.520	-0.214	-0.0106
STD	**0.048**	**0.061**	**0.113**	**0.021**	**0.013**
Reduction in STD	**34.8%**	**-3.2%**	**20.8%**	**0.3%**	**50.4%**

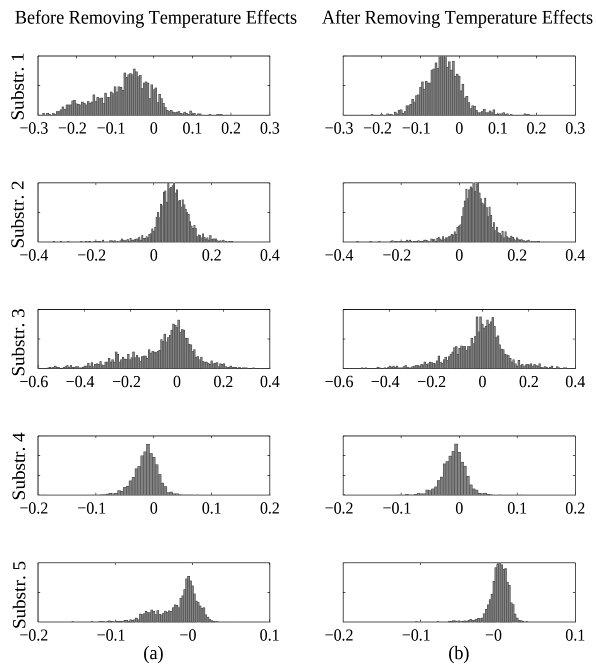


Figure 9. Histograms of substructure updating factors

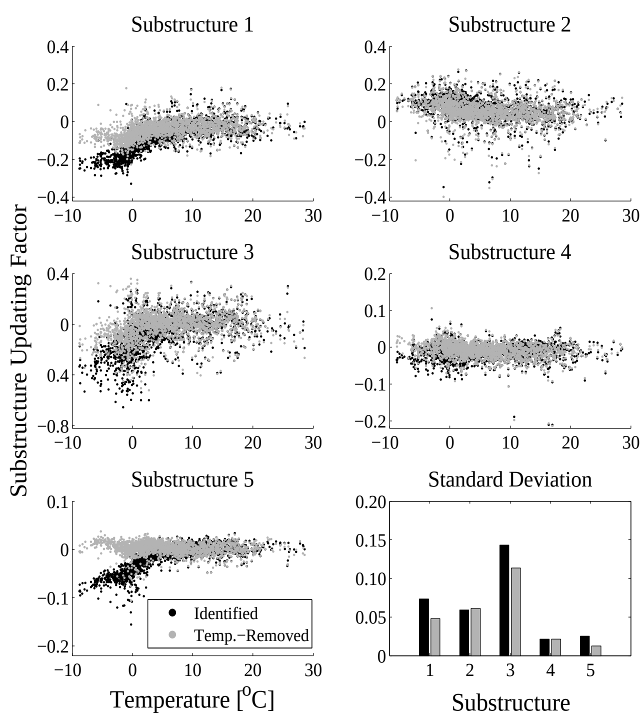


Figure 10. Variation of substructure updating factors versus temperature of sensor S3 (black dots refer to updating results using identified modal parameters, while grey dots correspond to results using temperature-removed natural frequencies)

In order to evaluate the quality of updated FE models, the residuals between the FE model-computed and experimentally identified natural frequencies of all 2,088 updated models are plotted in Figure 11 versus temperatures at sensor S3 (black dots). In general, the natural frequency residuals are small, indicating the accuracy of updated FE models. Modes 1, 3, and 4 have the smallest residuals, while residuals of mode 2 are the largest. These observations are consistent with the estimation uncertainty of the modes and therefore the assigned weights to each mode residual in the optimization process. Frequency residuals of modes 1, 2, 3, and 5 show larger correlations with measured temperatures. These residuals are higher for temperatures below freezing for modes 1, 3, and 5.

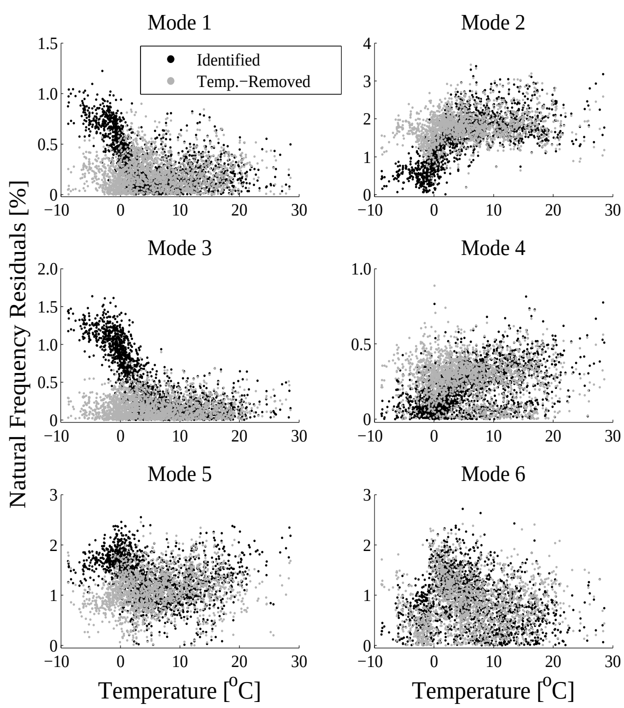


Figure 11. Variation of frequency residuals versus temperature at sensor S3 (black dots correspond to residuals before removing temperature effects and grey dots are residuals after removing temperature effects)

5.2. After Removing the Temperature Effects

In this section, the FE model updating process is repeated using temperature-detrended natural frequencies and identified mode shapes of the six vibration modes. Effects of changing ambient temperatures are removed from the 2,088 sets of identified natural frequencies using the fourth order polynomial model presented in Section 3.2. Table 4 reports the statistics of the substructure updating factors identified based on the temperature-detrended natural frequencies. From the comparison of statistics of the two sets of updating factors, corresponding to identified and temperature-removed frequencies, it can be seen that: (a) the average updating factors of all substructures become closer to zero after the temperature effects are removed; (b) the standard deviations of updating factors of substructures 1, 3, and 5 are decreased by 38.1, 20.0, and 50.6 percent, respectively; and (c) standard deviations of substructure updating factors 2 and 4 did not decrease because these two substructures did not show any correlation with temperature (Figure 10). Removing temperature effects brings the updating factors closer to zero and reduces their variations, which will result in more accurate damage identification results especially at temperatures below freezing. Since the footbridge was not physically damaged during the 17-week monitoring period, the calculated non-zero updating factors after removal of temperature effects are due to other sources of uncertainty/variability such as FE modeling errors, estimation errors of modal parameters, frequency-temperature modeling errors, and pedestrian traffic on the bridge. The histograms of substructure updating factors after removing the temperature effects are shown in Figure 9(b). The histograms of substructures 1, 3, and 5 are narrower than their counterparts in Figure 9(a), in which the temperature effects are not removed from the natural frequencies. The histograms of substructures 2 and 4 are shifted closer to zero without notable reduction in the variation of the updating factors.

Variations of the new set of updating factors are shown by gray dots in Figure 10, along with those obtained from identified natural frequencies before removing temperature effects, which are shown by black dots. From this figure, it can be observed that correlation of the updating factors with temperature measurements is significantly reduced after temperature effects are removed. By comparing the natural frequency residuals (between FE model and experimental data) after removing temperature effects with the frequency residuals before removing temperature effects in Figure 11, it can be observed that the correlation of the residuals with temperature is removed. In addition, the residuals are generally reduced except for modes 2 and 4, i.e., the calibrated FE models are more accurate after removing the temperature effects. It is also worth noting that the average of natural frequency residuals for any mode is less than 1.8%, indicating a good fit between the updated FE models and measured data.

6 SUMMARY AND CONLUSIONS

A prototype continuous monitoring system was installed on the Dowling Hall Footbridge in November 2009. The monitoring system consists of eight accelerometers to monitor vibrations and ten thermocouples to measure temperatures. A set of data is recorded once an hour or when triggered by large vibrations. The monitoring system has been running continuously since January of 2010 and is still providing data. In this study, the measured data during the first 17 weeks of monitoring (January 5 to May 1) are used to investigate the effects of changing ambient temperatures on the FE model updating of this footbridge. Modal parameters are extracted from measured vibration data using an automated, data-driven stochastic substructure identification method. A polynomial model is estimated to represent the relationship between identified natural frequencies and measured temperatures. The model is then used to remove the temperature effects from the identified natural frequencies.

An initial FE model of the footbridge is calibrated to represent the reference/baseline FE model and this model is then updated to match each set of the hourly identified modal parameters. The reference model is created in a two-step process. First, the mass of deck and bottom chords, and the vertical stiffness of the three supports are updated. In this step the natural frequency residuals are reduced from more than 10% in the initial FE model to less than 2%. In the second step, the bridge is partitioned into five substructures and the equivalent stiffnesses of these substructures are updated. The same substructures are then updated based on the data measured hourly during the 17-week monitoring period.

Effects of temperature on the FE model updating are investigated by comparing the results based on two sets of identified natural frequencies: before and after removing temperature effects. The temperature effects are removed from the identified natural frequencies using a fourth-order polynomial regression model. A total number of 2×2,088 = 4,176 model updating runs are performed and the statistics of the substructure updating factors are studied. The variations of updating parameters are reduced after the temperature effects are removed. The standard deviations of three out of five updating factors are reduced up to 50.4%, but the standard deviation of the other two factors remained almost unchanged. It was observed that the reduction in variation of substructure updating factors after removing the temperature effects is significantly less than the reduction in identified natural frequencies (29.7% to 71.3%). Natural frequency residuals of mode 1, 3, and 5 are also significantly reduced after removing the temperature effects. This indicates better fit between the updated FE models and experimental data. Removing temperature effects results in (a) smaller variability in the updated stiffness parameters of FE models, (b) lower natural frequency residuals, and (c) updating factors closer to zero. This yields more accurate results when FE model updating is used for localization and quantification of damage as loss of stiffness. Examples of damage for this type of structure include steel cross-section reduction, cracks in concrete deck, and changes in boundary conditions. It is worth noting that the non-zero updating factors after removing temperature effects are due to other sources of uncertainty/variability such as FE modeling errors, estimation errors of modal parameters, frequency-temperature modeling errors, and pedestrian traffic on the bridge. Effects of estimation and modeling errors on the FE model updating results can be accounted for in probabilistic model updating procedures such as Bayesian FE model updating.

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