TEMPERATURE EFFECTS ON STRUCTURAL HEALTH MONITORING OF AN INTEGRAL ABUTMENT BRIDGE

Tuan Anh Pham, March Consulting, Canada
Bruce Sparling, University of Saskatchewan, Canada
Leon Wegner, University of Saskatchewan, Canada
bruce.sparling@usask.ca

Abstract
A numerical study was undertaken to investigate the influence of ambient temperature changes on vibration-based detection of small-scale damage within a bridge deck. The structure under consideration was a two-span traffic overpass featuring integral abutment construction in which thermal expansion is accommodated through bending, rather than by expansion joints. Since vibration-based damage detection relies on the identification of small changes in dynamic properties to infer the presence and location of damage, damage-induced changes must be distinguishable from those originating from other sources, including thermal variations. It was found that temperature variations over the range expected at the bridge site produced significant changes in both the fundamental natural frequency and mode shape. Although patterns of mode shape changes due to thermal effects differed from those caused by damage, particularly in the vicinity of the damage, the temperature effects were found to be larger for the conditions studied and, thus, had the potential to mask critical damage indicators.

1 Introduction
Structural health monitoring (SHM) encompasses a wide variety of condition assessment strategies used in modern infrastructure management programs. Such strategies vary from simple visual inspections, to local non-destructive evaluation (including chain drags, half-cell potential, ultrasonics, acoustic emissions, etc.), to global SHM methods that measure changes to the overall response of a structure as indicators of damage. Vibration-based damage detection (VBDD) methods are a set of global SHM techniques whose common underlying premise is that deterioration of a structure’s physical properties (i.e. damage) can be inferred from changes in measured dynamic characteristics [1]. As a result, VBDD methods rely on accurate measurements of modal vibration parameters (notably natural frequencies and mode shapes) taken periodically over the lifetime of a structure, as well as at some initial baseline (preferably pristine) stage.

A potential complication in the practical implementation of VBDD techniques is the need to distinguish damage-induced changes to dynamic response characteristics from those brought about as a result of other factors. For structures located in northern climates, for example, temperature related effects are a major concern in this respect. In the present study, a field testing and numerical investigation was undertaken to evaluate the application of VBDD assessments to a structurally indeterminate traffic bridge subjected to a wide range of ambient temperature variations.
2 Description of Structure and Field Testing Program

Located in Saskatoon, Canada, the Attridge Drive overpass structure was constructed in 2001 featuring two spans (38.7 m and 30.6 m), six lanes of traffic with an overall deck width of 27 m, and a skewed alignment of approximately 16° (see Figures 1 and 2). The composite superstructure consists of nine 1400 mm deep steel girders and a 225 mm conventional reinforced concrete deck thickened locally to 300 mm above the girders.

To eliminate potential corrosion problems associated with expansion joints, the overpass features integral abutment construction in which the bridge deck and girders have been made continuous with the abutment walls. Thermal expansion is then accommodated by flexural action within the resulting frame, achieved primarily through bending in the slender abutment walls and rotation at the pile supports. As a result, the overpass structure is highly indeterminate and susceptible to temperature effects.

The instrumentation for the field study consisted of seven accelerometers (model Episensor FBA ES-U, Kinemetrics Inc., California), each configured for a maximum range of ±0.5 g and precision of 0.00025 g. As shown on Figure 2, one reference accelerometer remained at a fixed location in the south-west quadrant of the overpass, while the remaining six were moved between the four quadrants. In each of the four quadrants, the six mobile accelerometers were mounted at 5 m intervals longitudinally. Because the bridge was in service during testing, the accelerometers were installed only along the two outer edges of the barrier walls.

Dynamic excitation was provided exclusively by ambient traffic loading, with measurements timed to coincide with the passage of a heavy vehicle. For each accelerometer setup, the responses associated with at least ten significant loading events were recorded, thus enabling some response averaging to attenuate the effects of random measurement noise and excitation characteristics. Time series response data for each event were acquired at a sampling rate of 300 Hz for a period of 10 seconds.

Modal properties of the overpass were inferred from Fast Fourier Transform (FFT) analyses of the accelerometer data. Natural frequencies were identified using a peak-picking procedure based on the averaged, normalized response spectrum for all loading events. Corresponding mode shapes were then extracted from the original acceleration spectra; in order to assemble the data from the various loading events and instrumentation set-ups, modal amplitudes for a specific event obtained from each of the mobile accelerometers were normalized by the corresponding value from the stationary reference instrument, thus providing a common basis for combining all the modal results.

Figure 1 The Attridge Drive overpass viewed from the north-east.
Ambient temperatures at both the top and bottom of the deck were measured intermittently throughout the period of testing, the duration of which averaged six hours (approximately 9:00 a.m. to 3:00 p.m.) over the course of one day.

3 Description of Numerical Study

A linear elastic finite element (FE) model of the overpass (see Figure 3) was generated using the commercial software package ADINA [2]. The bridge deck was modeled using 8-node, 3-dimensional isoparametric solid elements. The concrete deck was discretised into three vertical layers, each of which featured 60 elements longitudinally and 31 elements transversely; the top layer was made 50 mm deep to represent the concrete cover over the top layer of reinforcing steel, while the middle and lowest layers were 75 mm and 100 mm deep, respectively. A further 90 mm thick layer of asphalt was placed above the concrete. Solid elements were also used for abutments, median and barrier walls. The girders were modelled using 4-node shell elements, while the diaphragms were modelled using a combination of truss, beam and shell elements. Linear translational springs were used to represent the soil backfill behind the abutment walls, whereas the pile foundations at the base of the abutments were modeled using both translational and rotational springs. The complete model comprised 12166 elements and 15369 nodes.

The FE model was manually calibrated to match the lowest three natural frequencies and mode shapes obtained from the measured field data taken at the reference ambient temperature of approximately 20°C. This was done primarily by adjusting the “effective” elastic modulus of the

Figure 2 Plan view of overpass showing instrumentation locations (dimensions in mm).

Figure 3 Schematic of finite element model showing locations of simulated damage.
concrete bridge deck to account for the presence of flexural cracking, with adjustments made separately in the longitudinal and lateral directions to reflect the orthotropic behaviour of the deck slab. Also considered in the calibration process was the assumed stiffness of the backfill and pile supports for the abutment walls.

Once the numerical model had been calibrated for the reference temperature range, the dynamic properties of the system were studied at varying temperatures, simulating temperature effects in two ways. First, a temperature change was imposed uniformly on the structure, with representative coefficients of thermal expansion defined for each material. Secondly, temperature-dependent adjustments were made to the concrete and asphalt material properties, based on information reported in the literature [3, 4, 5, 6]. Since a preliminary study indicated that the system dynamic properties were relatively insensitive to temperature-related changes in soil properties, the backfill and pile stiffness values were assumed to remain constant over all temperatures considered.

Comparisons of the numerical and measured natural frequencies for the lowest three modes of the system are provided in Table 1 for three of the five temperature ranges encountered in the field testing program. The temperature ranges listed in Table 1 refer to the variation in temperatures recorded during field measurements on a given day; corresponding FE model results were calculated at the middle of each temperature range. Good agreement was achieved between measured natural frequencies and FE model results, with relative differences of less than 1% observed for the fundamental mode for all five temperature ranges. Good correlation was also found between measured and numerical mode shapes, with calculated Modal Assurance Criteria (MAC) values exceeding 0.944 for the first three modes in each temperature range; a MAC value of unity indicates perfect correlation between the measured and numerical mode shapes [7].

Simulated damage was inflicted on the numerical model by removing selected elements at various locations on the bridge deck. As illustrated on Figure 3, nine damage scenarios were investigated, each of which involved the removal of two longitudinally contiguous elements at a given location from both the asphalt (90 mm thick) and top concrete (50 mm thick) layers of the bridge deck. Such damage may arise, for example, as a result of mechanical wear or due to corrosion-induced spalling. Lateral dimensions of individual damage states ranged from 0.3 m to 3 m, depending on the size of the elements removed. Results presented in this paper focus on damage case 6, located adjacent to the median near the middle of the west span of the overpass.

While several VBDD methods were used in this study to evaluate changes in vibration mode shapes due to various imposed condition states (defined by changes in ambient temperature and/or inflicted damage), the results presented herein focus on a direct comparison between mode shapes

<table>
<thead>
<tr>
<th>Temperature Range (°C)</th>
<th>Natural Frequency (Hz)</th>
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<tbody>
<tr>
<td></td>
<td>1&lt;sup&gt;st&lt;/sup&gt; Mode</td>
</tr>
<tr>
<td>-12 to -5</td>
<td>Measured 3.353</td>
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<tr>
<td></td>
<td>FE Model 3.344</td>
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<tr>
<td>20 to 26</td>
<td>Measured 3.076</td>
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<tr>
<td></td>
<td>FE Model 3.083</td>
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<tr>
<td>30 to 40</td>
<td>Measured 3.076</td>
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<tr>
<td></td>
<td>FE Model 3.048</td>
</tr>
</tbody>
</table>
corresponding to a given condition state and the baseline condition state (undamaged at an ambient temperature of 20° C). For the $i$th vibration mode, if the mode shape vectors for a given condition state and the baseline condition state are denoted by $\phi_i^*$ and $\phi_i$, respectively, the change in mode shape $\Delta \phi_i$ caused by the change in condition states is simply defined as

$$\Delta \phi_i = \phi_i^* - \phi_i$$

(1)

If damage is present, it will cause a localized decrease in stiffness. As a result, the greatest change in mode shape, identified as the largest absolute value in the vector $\Delta \phi_i$, is expected to occur at the location of the damage. Mode shape changes due to temperature variations, on the other hand, will reflect the widespread influence of both the temperature-dependent material properties and axial member forces generated as a result of thermal expansion or contraction.

In the current study, mode shapes for the various condition states were generated based on stressed eigenvalue analyses of the calibrated numerical model. To reflect field measurement limitations, only the vertical eigenvector components at nodes on the surface of the bridge deck were extracted to define the mode shapes.

4 Presentation and Discussion of Results

4.1 Temperature Effects on Dynamic Properties

The influence of ambient temperature on measured natural frequencies of the overpass is shown in Figure 4; the horizontal error bars included on this graph represent the temperature variation that occurred over the test period. A distinctly bilinear relationship between natural frequency and temperature is apparent for all three modes, a result that is broadly consistent with findings of others [8]. The natural frequency is seen to increase at a faster rate with decreasing temperatures below approximately 5°-10° C, although the location of the break point on the curves is somewhat difficult to determine due to the relatively limited data in this range.

As demonstrated in Table 1, the observed trends in decreasing natural frequencies with increasing ambient temperatures were accurately reproduced by the numerical model. Based on a limited parametric study, it appeared that the temperature-induced changes were largely due to increases in the elastic modulus values for the concrete and asphalt at lower temperatures.

The influence of temperature change on the fundamental mode shape is illustrated in Figure 5. Here, the changes in mode shape $\Delta \phi_1$ for ambient temperature changes from 20° C to -25°C, and from 20° C to 35°C, are plotted both over the surface of the bridge deck and along three
longitudinal lines where instrumentation could be readily installed during normal operation of the bridge: along the north and south edges of the deck, and along the median. It is evident that these large scale temperature variations induce significant changes in the mode shape, manifest in fairly smooth undulations with wavelengths approximately corresponding to the deck spans. With decreasing temperatures, mode shape changes are seen to vary in both the longitudinal and lateral directions, while for increasing temperatures the changes are fairly consistent across the width of the deck, varying primarily in the longitudinal direction.

4.2 Mode Shape Changes due to Damage

Mode shape changes due to damage case 6, located near the middle of the west span and adjacent to the median (see Figure 3), are shown in Figure 6 for a constant ambient temperature of 20°C. In the immediate vicinity of the simulated damage, the mode shape change plots feature the distinct, narrow positive peak centred on the damage location, along with adjacent negative depressions, that are characteristic of damage scenarios situated away from abutments and piers [9]. Further away from the damage, on the other hand, mode shape changes have a lower magnitude and exhibit smoothly varying patterns that do not provide any apparent indication of the damage location. This trend is clearly evident in the longitudinal line plots of mode shape changes along the north and south edges of the deck.

For the practical application of VBDD, this suggests that sensors must be placed in close proximity to damage site in order to obtain a clear, unambiguous indication of the damage location. Similar conclusions were reached in previous studies [9]. Furthermore, the damage-induced mode shape changes are seen to be an order of magnitude smaller than corresponding changes due to large temperature variations.

The potential for temperature effects to mask damage-induced changes in VBDD investigations is clearly demonstrated in Figure 7, which shows the change in mode shape that would be observed between damage state 6 measured at -25°C and the reference, undamaged condition (at 20°C). Due to the predominance of the temperature-induced changes, the much smaller damage-related changes can only be detected in the form of a small, localized peak in the longitudinal line plot generated along the median. Such a peak might be difficult to distinguish from variations associated with experimental uncertainty [9].

At this stage, however, comparisons have been carried out solely on a visual basis. More sophisticated pattern recognition algorithms may be able to identify underlying patterns more efficiently, permitting a more definitive evaluation.

5 Summary

A numerical study was undertaken to investigate the changes in dynamic properties of an integral abutment bridge due to large-scale temperature variations. It was found that ambient temperature variations produced significant changes in both the natural frequencies and mode shapes of the structure. Although the patterns of mode shape changes due to temperature effects differed to some degree from those caused by low levels of damage to the bridge deck, the considerably larger temperature-induced changes were shown to be capable of effectively masking the effects of such damage. These results, while not precluding the successful application of vibration-based damage detection to such indeterminate structures, do demonstrate that effective measures must be implemented to attenuate thermal influences.
Figure 5 Surface and longitudinal line plots of the change in fundamental mode shape due to a change in temperature from the reference condition (20° C): (a) Final temperature of -25° C; and (b) Final temperature of +35° C.

Figure 6 Mode shape changes due to damage case 6 at 20° C.
Figure 7  Mode shape changes due to damage case 6 at -25° C, compared to the baseline undamaged case at 20° C.

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References


