BRIDGE SIGNATURES FOR CONTINUOUS BRIDGES

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16. Abstract

The prototype monitoring system, previously used on a relatively new Connecticut bridge, has been mounted on an older bridge. The system was modified to improve the resolution of the frequency data. This has provided an opportunity to further automate the software and to learn how aging influences the vibrational signature. One of the findings has been that frozen bearings alter this signature. This study has been part of a continuing research program to establish that vibrational monitoring can be used to evaluate the structural integrity of a bridge.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	Litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

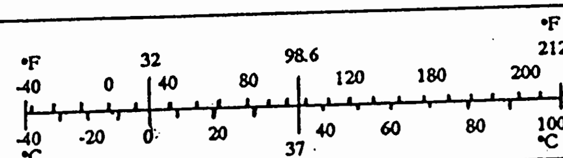
NOTE: Volumes greator than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
	MASS			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

*SI is the symbol for the International System of Measurement

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
	MASS			
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
	TEMPERATURE (exact)			
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



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PREFACE

This report has been abstracted from the master's thesis (Reference 1) of Patricia E. Conn, graduate research assistant. It describes the use of the vibrational bridge monitoring system on an older, continuous bridge. The previous use of the monitoring system on a new, simply supported bridge was reported in References 2 and 3.

The prototype bridge monitoring system used in this study was developed with a Connecticut Department of Higher Education Goodyear Cooperative High Technology Research and Development Grant (the Connecticut Department of Economic Development has now taken over this program). Vibra•Metrics, Hamden, Connecticut, originally built the system. Their modification of the system before placement on the bridge reported in this study is gratefully acknowledged.

The Joint Highway Research Advisory Council of the University of Connecticut and the Connecticut Department of Transportation has assisted in the installation and operation of the system and has provided funding for this project.

Chapter 1

INTRODUCTION

The development and initial installation of a remote, continuous monitoring system was reported by O'Leary, Accorsi and DeWolf (Reference 2). Monitoring is based on traffic-induced vibrational data which defines a signature for the bridge. The system was developed by researchers at the University of Connecticut and Vibra•Metrics, Hamden, Connecticut with funding from the State of Connecticut Department of Economic Development.

Bagdasarian and DeWolf (Reference 3) reported on the use of the system on the initial bridge, a new single span, determinate bridge which was not expected to experience any signature changes due to aging. The objective was to show how the vibrational signature is defined. This is based on the use of spectra, which are a graphical presentation showing acceleration information at different frequency levels. The natural frequencies and mode shapes were developed from the spectra. Included in this study were suggestions for modifications to the system and to the software for automization of the signature.

The work reported in this report is based on installation of the monitoring system, following modification as suggested by Bagdasarian and DeWolf, on an older, two-span continuous, indeterminate bridge. The goals were to continue the process of automating the data collection and development of the signature, as suggested by Bagdasarian and DeWolf, and to evaluate the signature development process on an older, continuous bridge. This included the influence of temperature changes on the signature. The study shows that the bearings are partially frozen, thus creating axial forces in the bridge girders. This results in corresponding changes to the natural frequencies.

Chapter 2

MONITORING SYSTEM INSTALLATION

2.1 Monitoring System Description

The vibrational monitoring system was built by Vibra·Metrics of Hamden, Connecticut. It consists of 16 accelerometers, two cluster boxes and a sentry unit. The accelerometers are magnetically attached to the bridge. They are placed on the spans to best acquire the vibrational mode shapes. A finite element analysis is useful in identifying the appropriate mode shapes, which then are used in determining the accelerometer locations. The two cluster boxes provide for interaction with 8 accelerometers each. They are linked to the sentry unit. This unit contains a computer with software for processing the data from the accelerometers. It also provides for communication with a remote monitoring site.

Improvements to the monitoring system were made before installation on the bridge, as suggested by O'Leary and Bagdasarian (References 2 and 3). The frequency resolution was increased from $\pm.05$ Hz to $\pm.039$ Hz. This would provide for better natural frequency stability. Additionally, high speed modems (9600 baud) were added to facilitate faster data transfer.

2.2 Bridge Description

The bridge selected was the Route 195 overpass crossing Interstate 84 in Tolland, Connecticut. It has two equal length spans, with continuity at the center support. Three lanes are provided on the 196-foot bridge. The superstructure consists of a concrete slab on seven non-prismatic welded steel plate girders. The cross-section is shown in Figure 1. The steel is ASTM A373. The composite 7" reinforced concrete slab is overlaid by a 2-1/2" bituminous wearing surface. The bridge was built in 1954 and the bituminous deck was replaced in 1976.

The bridge was chosen because it is an older, two-span continuous bridge. Previously, the monitoring system was used on a simple span bridge. The successful use of the monitoring system on a continuous bridge would provide added validity to the monitoring approach. It was also felt desirable to test the system on a bridge which has aged. Presently, visual inspections have confirmed that the bridge is structurally sound.

The plan of the bridge, with locations for the 16 accelerometers and equipment, is shown in Figure 2.

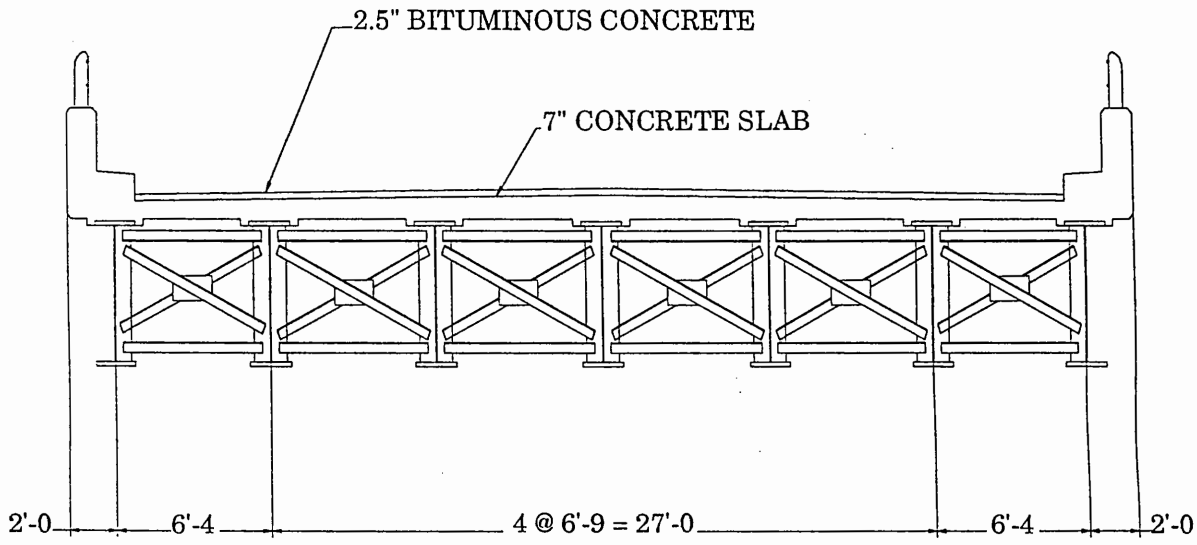

Figure 1. Bridge Cross Section

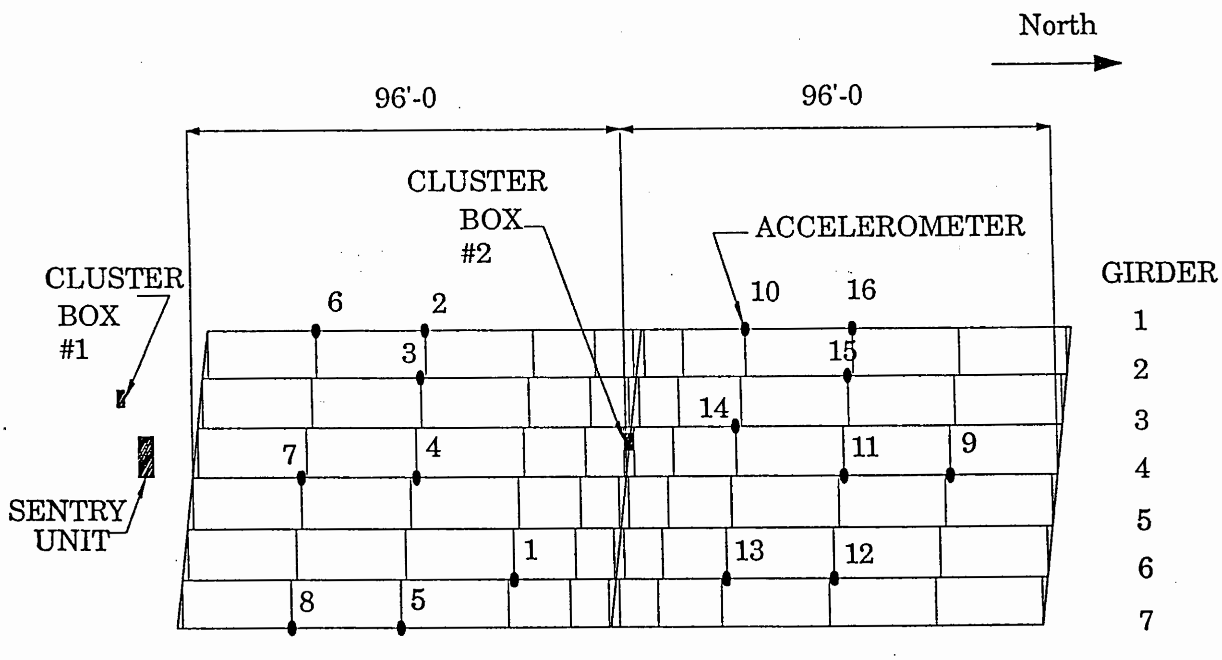

Figure 2. Plan with Equipment Locations

CHAPTER 3

EXPERIMENTAL RESULTS

3.1 General

Previous work at the University of Connecticut has defined the baseline signature of a bridge in terms of its natural frequencies and mode shapes. The acceleration patterns form a part of this signature. The monitoring system along with a portable system, referred to as the Smart Meter, were used to collect frequency spectra and process them into mode shapes.

3.2 Natural Frequencies

The natural frequencies were determined from frequency spectra, which plot volts (proportional to acceleration) versus frequency. An example is shown in Fig. 3. Natural frequencies are associated with peaks in this diagram. However, not all peaks correspond to natural frequencies. It is necessary to compare spectra obtained at different locations on the bridge, compare spectra from different data sets and to use phase comparisons to determine which peaks are at natural frequencies.

The natural frequencies for the first bending, first torsional, and second bending modes were identified using the permanent monitoring system and the Smart Meter. In November, these values were 3.6, 4.15, and 5.3 Hz respectively, as determined from the permanent system, and 3.6, 4.2, and 5.3 Hz, as determined from the Smart Meter.

Small peaks were found at frequency values near 7.5 and 8.5 Hz on a regular basis, as shown in Figure 3. However, when an attempt was made to identify the mode shapes for these frequencies, the phase information was unrecognizable.

In the range of 12.0 to 16.0 Hz, there are many peaks which are consistently excited. These peaks are shown in Figure 4. Two peaks usually appear at 14.1 and 15.0 Hz, but again, the phase information was unrecognizable. While analytical and finite element beam models indicate that there is another bending mode in this range, the large number of peaks within this range do not allow determination of the frequency from the test data.

3.3 Mode Shapes

The bridge's mode shapes are directly related to acceleration levels. To plot a mode shape, the phase angle of each channel is determined with respect to channel one. A phase angle close to π is out of phase (negative) and a phase angle close to zero is in phase with channel one (positive). These positive and negative signs are then applied to the acceleration level at that specific frequency. The analytically determined three dimensional modes shapes for the first and second bending modes and the first torsional mode shape are shown in Figures 5, 6, and 7 respectively.

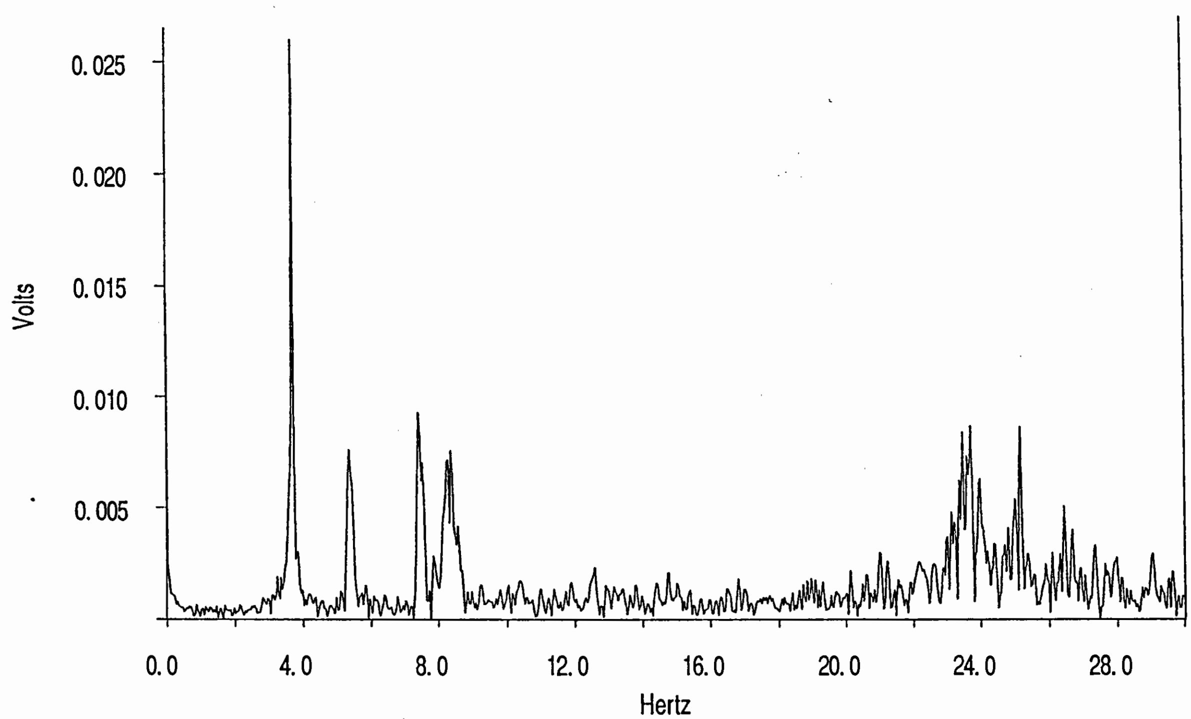

Figure 3. Typical Frequency Spectrum

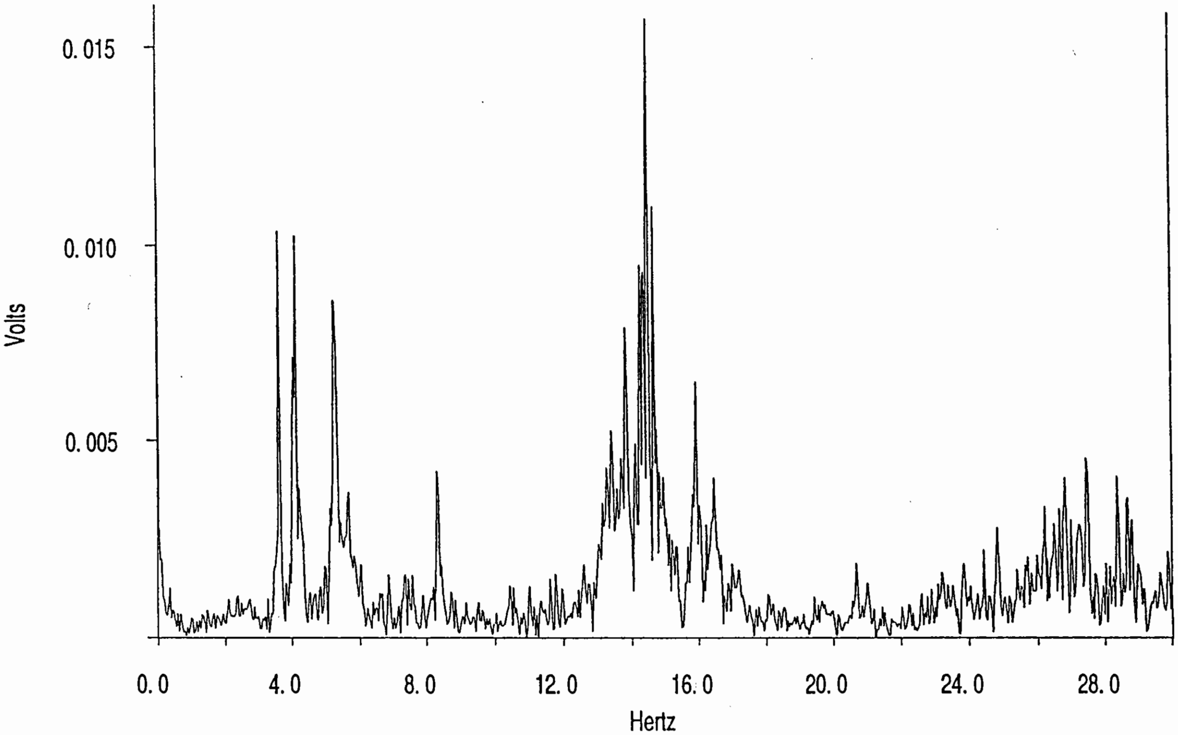

Figure 4. Typical Frequency Spectrum Showing Multiple Peaks Between 12 and 16 Hz

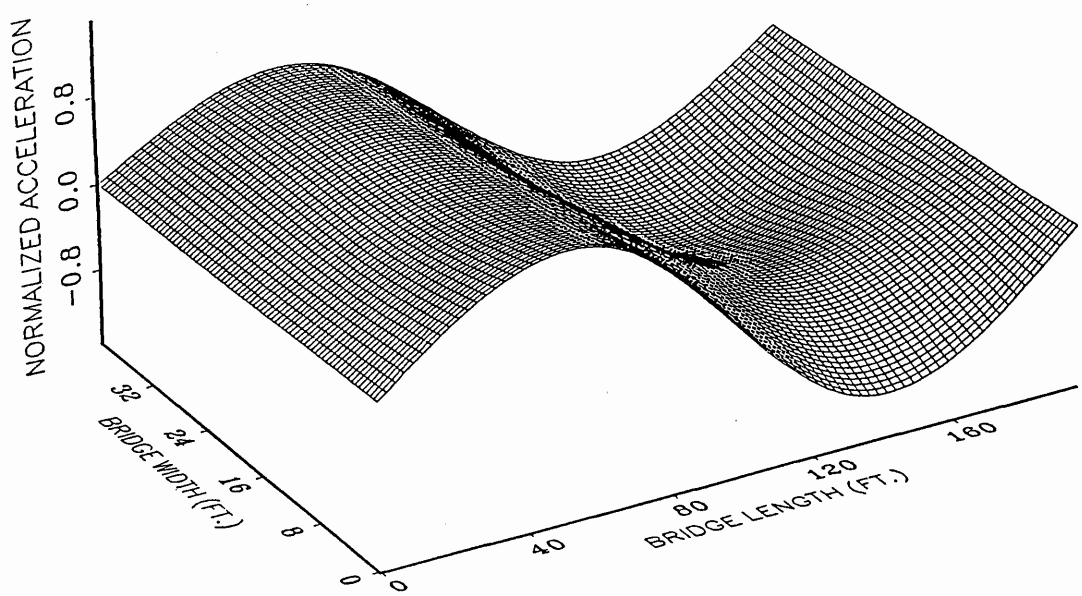

Figure 5. First Bending Mode Shape

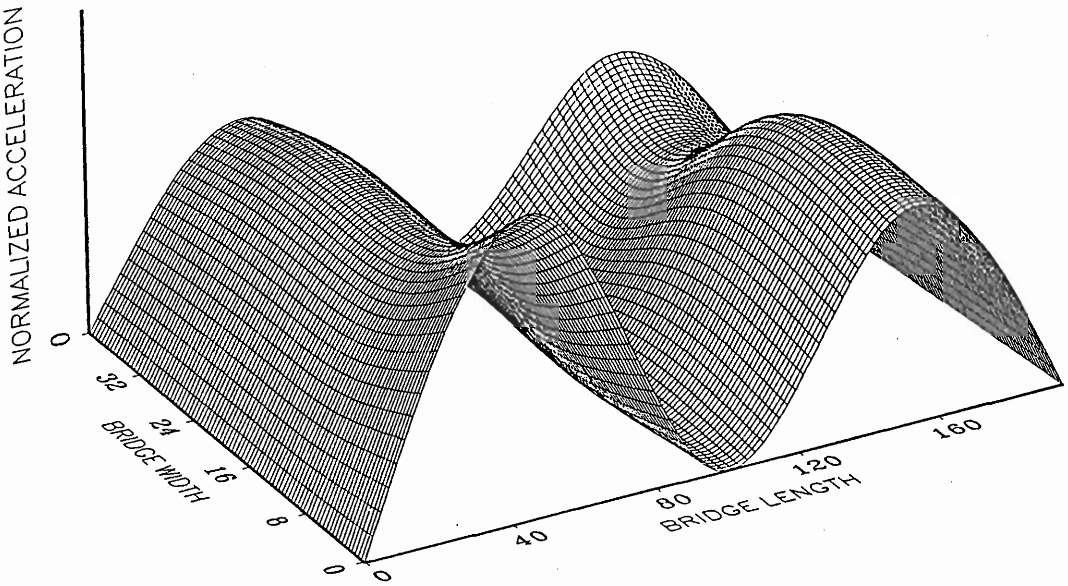

Figure 6. Second Bending Mode Shape

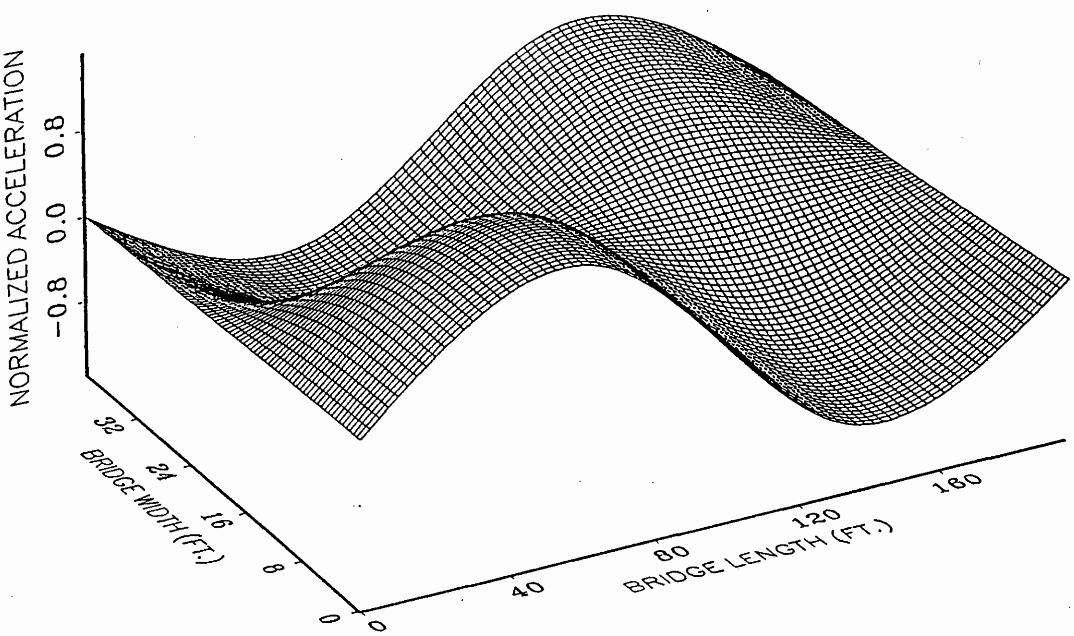

Figure 7. First Torsional Mode Shape

The stability of the mode shapes was determined by comparing the experimentally obtained mode shapes from different data sets. These mode shapes were determined from globally normalized values of acceleration levels. Five randomly selected sets of data were compared in two dimensions to assess the stability. Figure 8 shows the first flexural mode shape of an exterior girder. There is a 29% difference in maximum displacement between the five shapes. In Figure 9, the first torsional mode was compared along a cross-section of the quarter points in the first span. The difference in maximum displacement was 27% for this mode. The second bending mode along an exterior girder is shown in Figure 10. There is a change in the maximum displacement of 43% for this mode. These results indicate that the first bending and first torsional mode shapes are more stable and that the second bending mode, which exhibits a higher change, is less stable. This could be explained by the fact that it takes more energy to excite higher modes and therefore, the higher modes may not always be as stable for monitoring purposes.

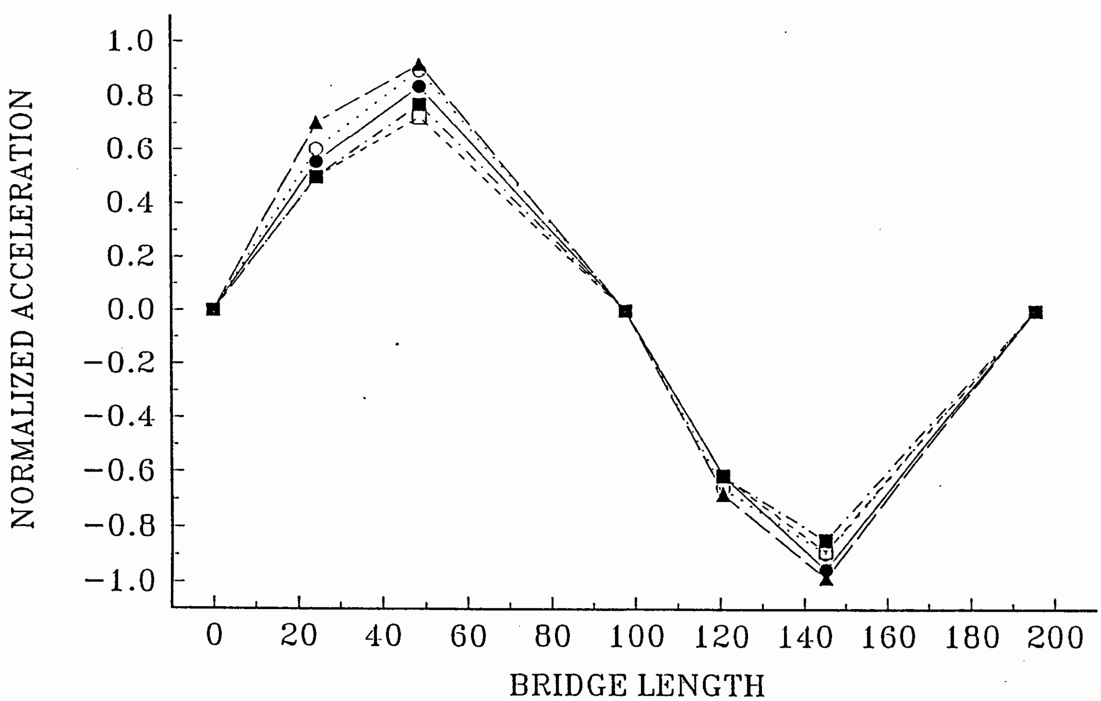

Figure 8. First Bending Mode Shape, Taken Along Exterior Girder

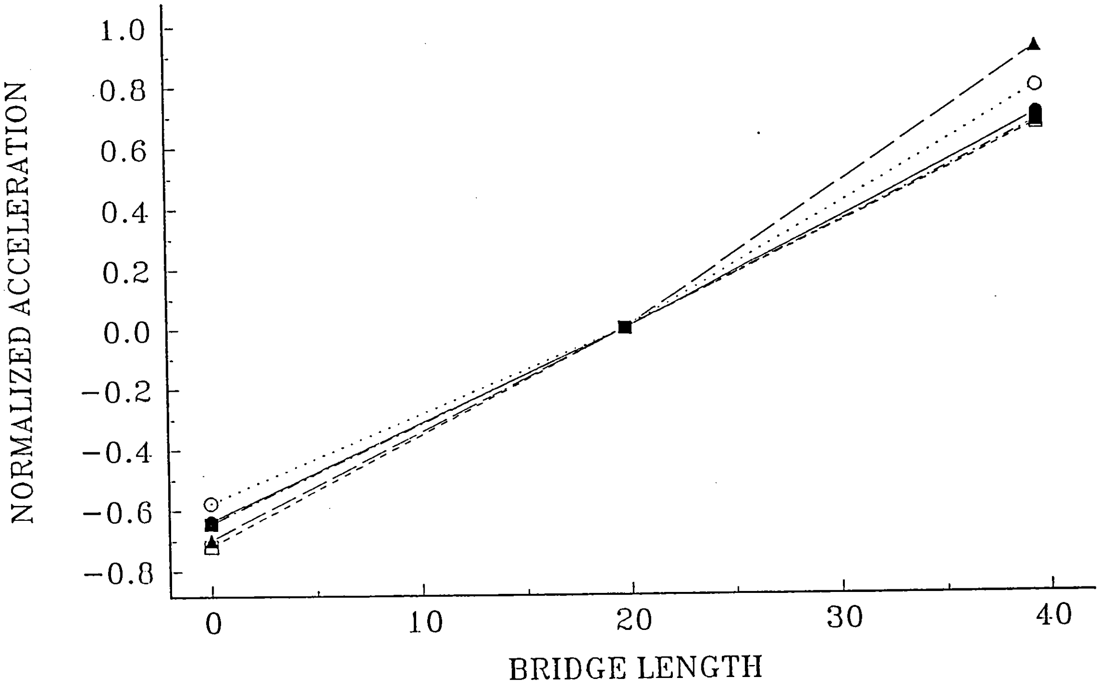

Figure 9. First Torsional Mode Shape, Taken At Interior Cross-Section of One Span

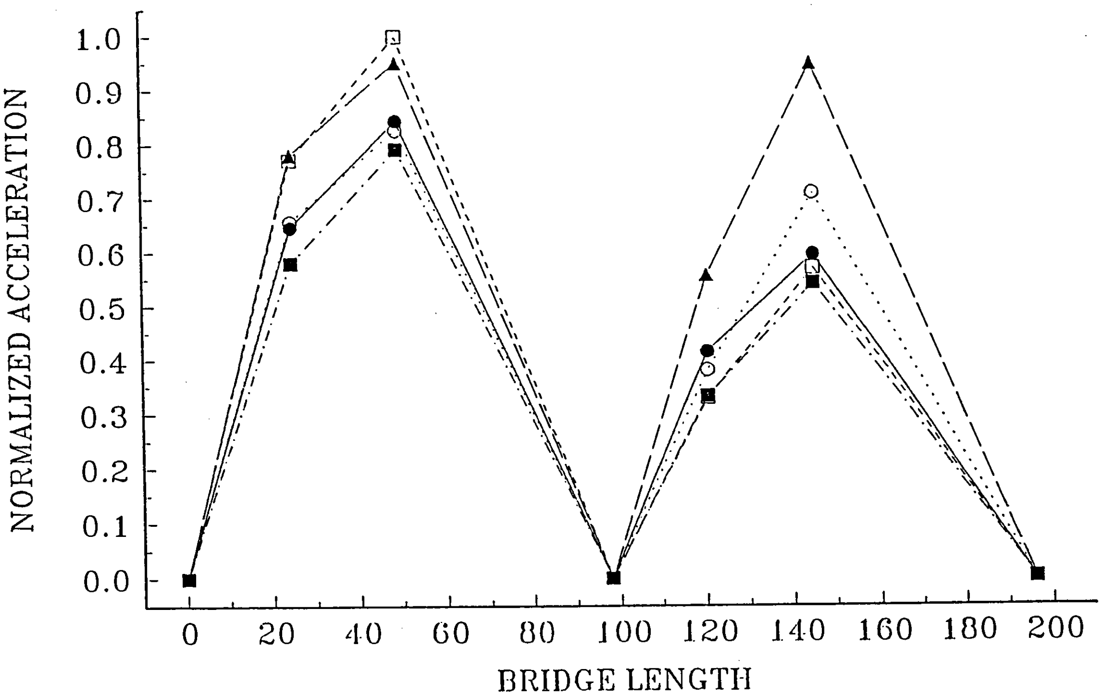

Figure 10. Second Bending Mode Shape, Taken Along Exterior Girder

3.4 Acceleration Trends

Acceleration levels are directly related to the weight, speed, and lane position of the vehicle travelling over a bridge. For this reason, a comparison of actual acceleration levels is not a good indication of bridge behavior. However, it has been indicated by Lauzon and DeWolf (Reference 4) that structural damage affects relative acceleration levels and therefore the mode shapes. This damage is seen in the relationship between the acceleration levels of different channels in one dataset. Acceleration levels were normalized with respect to the channel with the maximum acceleration for each of twenty datasets. A statistical analysis has been performed on these datasets in an attempt to identify baseline acceleration trends. Tables 1, 2, and 3 show the results of this analysis for the first and second bending and first torsional mode shapes.

Chan	Min	Max	Mean	σ	Chan	Min	Max	Mean	σ
1	.625	.801	.752	.035	9	.645	.769	.714	.034
2	.770	.949	.866	.052	10	.555	.681	.625	.038
3	.818	1.000	.931	.045	12	.873	1.000	.967	.038
4	.916	1.000	.985	.023	13	.605	.768	.712	.042
6	.498	.703	.600	.057	14	.556	.699	.640	.037
7	.686	.797	.739	.027	15	.691	.982	.885	.059
8	.582	.761	.679	.047	16	.800	.984	.875	.048

Table 1: Acceleration Trends for First Bending Mode

Chan	Min	Max	Mean	σ	Chan	Min	Max	Mean	σ
1	.471	.866	.587	.093	9	-------	-------	-------	------
2	.825	1.000	.948	.062	10	.653	.797	.704	.037
3	.608	.835	.715	.056	12	.500	1.000	.691	.104
4	-------	-------	-------	-------	13	.358	.712	.505	.072
6	.577	.761	.680	.054	14	.000	.480	.164	.161
7	-------	-------	-------	-------	15	.278	.808	.672	.106
8	.614	1.000	.757	.108	16	.864	1.000	.973	.040

Table 2: Acceleration Trends for First Torsional Mode

Chan	Min	Max	Mean	σ	Chan	Min	Max	Mean	σ
1	.363	.690	.558	.082	9	.451	.902	.808	.099
2	.506	1.000	.815	.115	10	.280	.971	.425	.086
3	.657	1.000	.890	.081	12	.618	.571	.919	.107
4	.565	1.000	.963	.099	13	.376	1.000	.547	.077
6	.444	.825	.649	.097	14	.223	.695	.442	.069
7	.508	.884	.822	.081	15	.465	.958	.781	.120
8	.525	.902	.735	.113	16	.447	.948	.755	.144

Table 3: Acceleration Trends for Second Bending Mode

3.5 Temperature Effects

At the onset of colder winter weather in December, the natural frequencies began to shift from 3.6 to 4.0 Hz, from 4.15 to 4.8 Hz, and from 5.2 to 6.0 Hz, while maintaining the same mode shapes.

Review of possible reasons for this change indicated that temperature was the most likely cause. It has been stated that temperature changes affect natural frequencies, though information is not available for bridges. Plots of temperature vs. frequency were drawn to determine a relationship between these values for each of the three mode shapes. These plots are shown in Figures 11, 12, and 13. It was determined that the temperature only changed the frequencies for values between 0°F to 60°F. After 60°F, although not shown in the graphs, the line levels off and remains constant.

Since the accelerometers are designed to endure temperatures as low as -65°F, it was concluded that the frequency shifts were due to an imperfection in the bridge structure itself. This change is most likely due to rocker bearings which no longer rotate as expected in colder weather. The change in natural frequencies due to temperature changes is discussed in the next chapter.

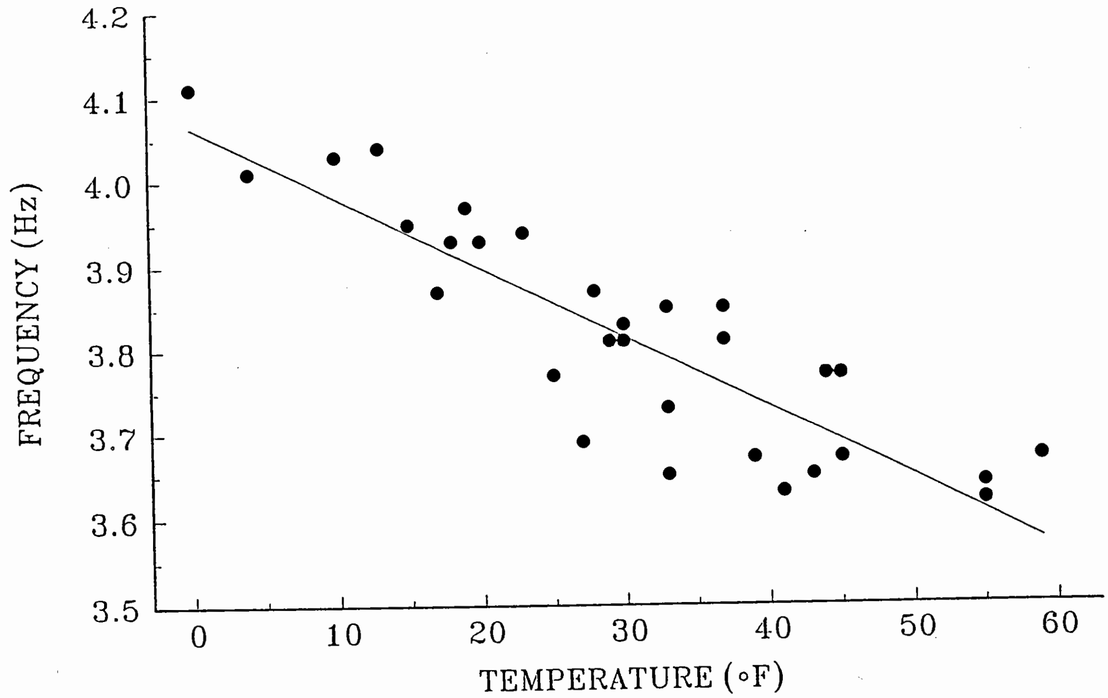

Figure 11. Frequency vs. Temperature for First Bending Mode

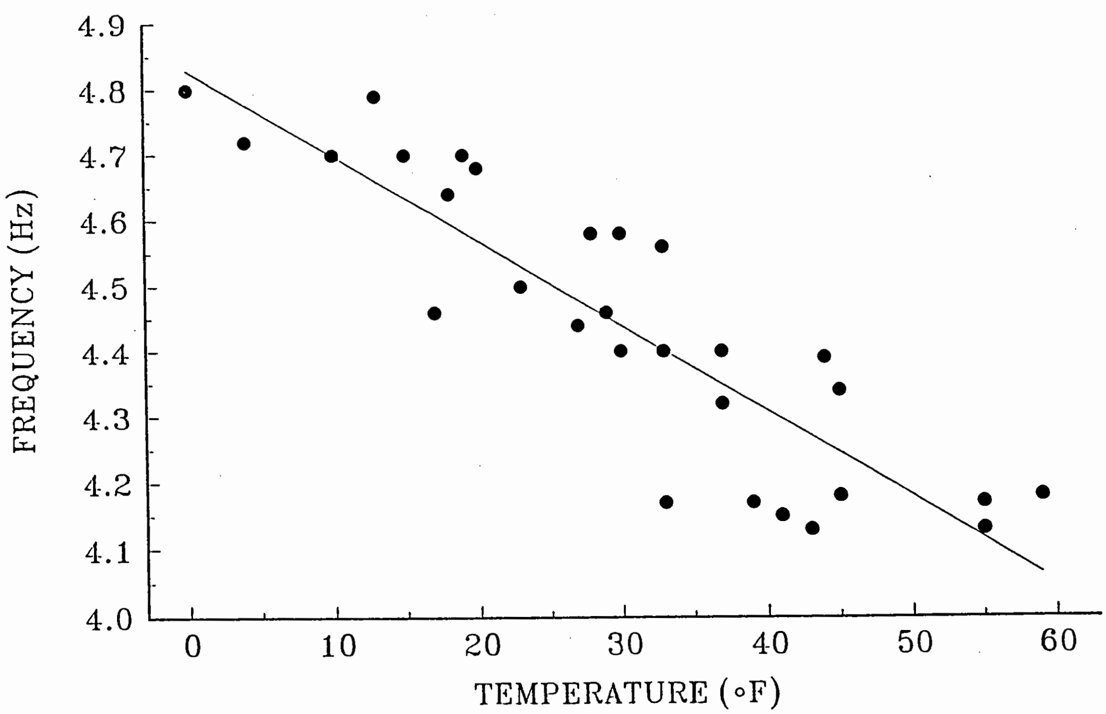

Figure 12. Frequency vs. Temperature for First Torsional Mode

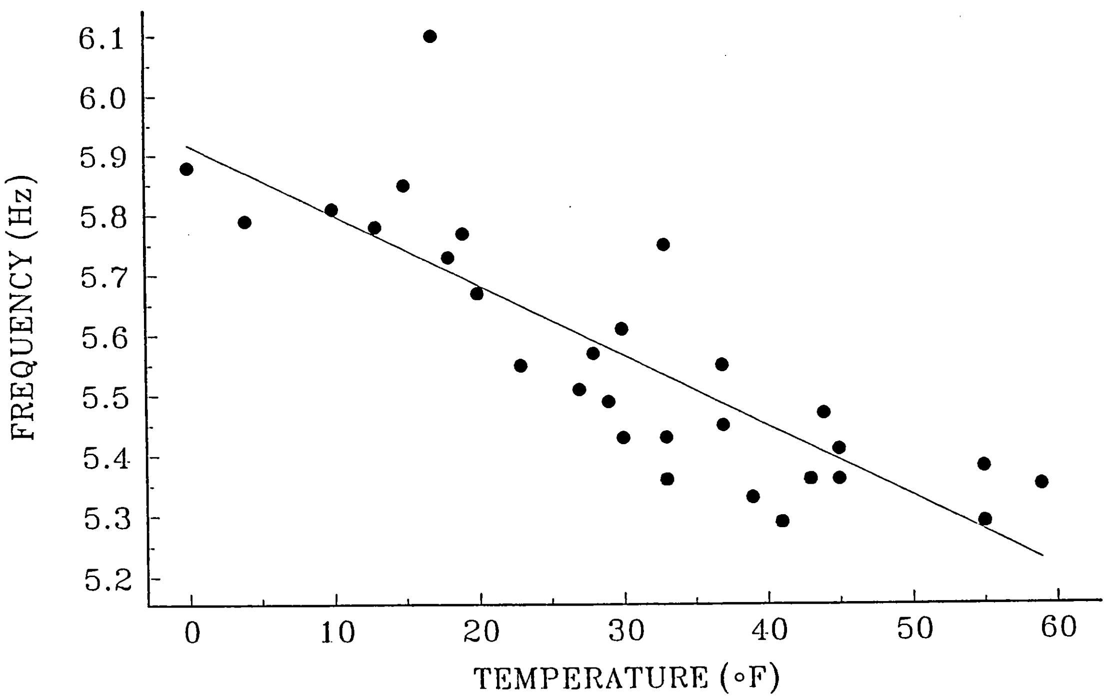

Figure 13. Frequency vs. Temperature for Second Bending Mode

CHAPTER 4

ANALYTICAL RESULTS

4.1 Introduction

As shown in the last chapter, reduction in temperature has lead to increases in the natural frequencies. This chapter provides an analytical treatment to show how temperature alters the natural frequencies.

4.2 Vibration of a Two-Span Continuous Beam

The flexural natural frequencies of a prismatic, simply-supported oscillating beam are given by:

$$\omega_n = \frac{n^2 \pi^2}{l^2} \sqrt{\frac{EI}{m}} \tag{1}$$

where EI = stiffness, l = length, m = mass/unit length, n = 1,2,3...

For the case of an oscillating prismatic, single-span beam, the displacement can be given as:

$$\begin{aligned} X &= C_1(\cos kx + \cosh kx) + C_2(\cos kx - \cosh kx) \\ &+ C_3(\sin kx + \sinh kx) + C_4(\sin kx - \sinh kx) \end{aligned} \tag{2}$$

The basis for this equation can be used to approximate the fundamental flexural frequency of a two-span continuous beam, founded on the same assumptions. A two-span continuous beam can be treated as two simply-supported prismatic members with separate displacements, as follows:

$$\begin{aligned} X_1 &= C_1(\cos kx + \cosh kx) + C_2(\cos kx - \cosh kx) \\ &+ C_3(\sin kx + \sinh kx) + C_4(\sin kx - \sinh kx) \end{aligned} \tag{3}$$

$$X_2 = C_1(\cos kx + \cosh kx) + C_2(\cos kx - \cosh kx) + C_3(\sin kx + \sinh kx) + C_4(\sin kx - \sinh kx) \tag{4}$$

Applying the boundary conditions (simply supported at ends and continuous over the center support), and solving for the constants produces the following characteristic equation:

$$(\sin^2 kl)(\sinh 2kl) - (\sin kl)(\cos kl)(\cosh 2kl) + (\sin kl)(\cos kl) = 0 \tag{5}$$

The first four roots of equation (5) are: $k_1 l = \pi$, $k_2 l = 3.94$, $k_3 l = 2\pi$ and $k_4 l = 7.1$. These are based on prismatic members.

The natural frequencies of the two-span continuous, prismatic bridge are a function of these k_n values, and are given by:

$$\omega_n = \frac{k_n^2}{2\pi}\sqrt{\frac{EI}{m}} \tag{6}$$

The actual bridge consists of seven non-prismatic girders. Each plate girder is made up of three different cross sections. The size of the web plate is consistent throughout the bridge, but the flange plates change size along the length of each girder. The values of I (ranging from 54026 to 113850 in^4) and $\bar{m}$ (ranging from 29.3 to 34.7 slugs) were substituted into Equation 6. The frequency ranges are then:

$$\begin{aligned} \omega_1&: 3.2 - 4.2\ Hz \\ \omega_2&: 5.0 - 6.6\ Hz \\ \omega_3&: 12.6 - 16.8\ Hz \\ \omega_4&: 16.1 - 21.5\ Hz \end{aligned} \tag{7}$$

These are the first four flexural frequencies, based on prismatic members. These were also determined with a finite element analysis, including the variable moment of inertia. The values are:

$$\begin{aligned} \omega_1 &: \ 3.8\ Hz \\ \omega_2 &: \ 6.2\ Hz \\ \omega_3 &: \ 14.9\ Hz \\ \omega_4 &: \ 18.8\ Hz \end{aligned} \tag{8}$$

As expected, these values fall between those in Equation 7.

4.3 Vibration of a Two-Span Continuous Beam Including a Tensile Force

Equation 6 can be modified to include the influence of axial forces on the natural frequencies (Reference 5). The natural frequencies are then:

$$\omega_n = \frac{k_n^2}{2\pi}\sqrt{\frac{k_n^2 EI + N}{m}} \tag{9}$$

where N = the axial force (negative for compression, positive for tension).

The performance of the bridge results in an increase in the natural frequencies during the colder weather. This is consistent with bearings which are not rotating at lower temperatures. Thus, the bridge is not able to contract as the temperature drops. This induces a tensile force in each girder. This axial force **N** is then:

$$N = E \bullet A \bullet \alpha \bullet (\Delta T) \tag{10}$$

where α is the coefficient of expansion of steel and ΔT is the change in temperature.

The value of **N** due to a frozen bearing is approximated as follows. Experimentally, frequency changes were noticed for temperatures under 60°F. The minimum temperature in which data was acquired was approximately 0°F. Based on these experimental results, a temperature change of 60°F was used for ΔT. Using this ΔT, the smallest and largest values of **A**, and material properties of steel, minimum and maximum values of **N** are 1360 and 1785 kips.

The minimum and maximum values of **N** were used in Equation 9 to obtain the first four flexural frequencies. The results are tabulated in Table 4.

	ω_1	ω_2	ω_3	ω_4
no N	3.2-4.2	5.0-6.6	12.6-16.8	16.1-21.5
with N	3.3-4.4	5.1-6.8	12.8-17.0	16.3-21.7
% increase	+3-5%	+2-3%	+1-2%	+1%

Table 4: Comparison of Frequencies With and Without Axial Force

Table 4 shows smaller frequency changes than indicated by the experimental data. The actual shift in the first bending natural frequency was from 3.6 to 4.0 Hz, a 10 percent increase. This is much larger than the approximated increase in the range from 3 to 5 percent. For the second bending mode, the experimental values increased from 5.2 Hz to 6.0 Hz, a 13 percent increase. The analytical model showed an increase of 2 to 3 percent for the second bending mode. One reason for the discrepancy could be that the analytical model is only for a single prismatic beam. It also neglects the influence of the concrete deck. The 60°F temperature change is approximate. Additionally, it is likely that part of the cause for the large experimental increase is attributable to restriction of rotational displacement at the bearings in addition to the prevention of the horizontal translation.

A more complete analysis would need to look at the deck as a two-dimensional plane, including torsional deformations. It would also be necessary to evaluate both transational and rotational displacements of the bearings at the two ends to fully determine how much tensile force is induced in the structure. Before that can be done, extensive field measurements at different temperature levels would be needed to define the bearing's displacements.

CHAPTER 5

CONCLUSIONS

As shown in previous work with the vibrational monitoring system on a newer bridge, natural frequencies and mode shapes were relatively stable under varied traffic conditions. These form the bridge's baseline signature. The goal was to use the vibrational behavior in different forms to monitor the structural integrity of the bridge. As an example, statistical comparisons of the acceleration trends have been shown to produce indicators of changes.

The monitoring system was placed on a bridge with two equal length spans and continuity at the center support. The composite, non prismatic bridge was built in 1954, with replacement of the bituminous deck in 1976. Thus, this study provided the opportunity to review a continuous bridge which is older than the bridge previously monitored. As a consequence, changes have occurred to the vibrational information during the year, due to weather conditions. This was evaluated in this study.

Three natural frequencies at 3.6, 4.15 and 5.2 Hz were identified as the first bending, first torsional and second bending modes, respectively. Additional peaks were noted at 7.5 Hz and 8.5 Hz, but could not be identified because of unrecognizable mode shapes. A cluster of peaks exists in the area between 12 and 16 Hz, but they can not be identified due to the number of peaks occurring in that area and unrecognizable phase information. Previous studies have shown that for bridges only the lowest natural frequencies are excited sufficiently to produce recognizable vibrational information.

The data collected was relatively consistent during the warmer weather. In the winter, the substantially lower temperatures caused the natural frequencies to increase. It was concluded that these are due to lack of full displacement at the bearings at the end of the bridge, which should be free to rotate and provide displacement in the horizonal direction. The reduction in ability to displace horizontally creates tensile forces in the bridge. The reduction in rotation changes the end bearings from pinned to partially fixed. Both result in increased natural frequencies. An analytical treatment has been used to account for part of this change.

The use of the vibrational monitoring system on the bridge demonstrates that the relatively small changes in the overall performance, due to temperature changes combined with frozen bearings, are discernable from comparisons of the natural frequencies. It is important to note that while the frequency changes are not that large, and likely within the range of variations due to variations in vehicles, a statistical comparison with multiple sets of data will produce an indication that there has been a change in the bridge. Thus, the range of frequencies for a given mode will statistically shift upward as the temperature decreases.

This study has been a part of a continuing effort to establish that vibrational information can be used to evaluate the structural integrity of a bridge. A substantial portion of the effort, not reported in this report, involved the continued automization of the data, needed for continuous monitoring.

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