STRUCTURAL MONITORING FOR BRIDGE BEHAVIOR AND DAMAGE DETECTION

Marvin W. Halling[1] and Zachary C. Hansen[2]

Abstract

Motion detecting instruments have been used to record seismic motions of the ground, buildings, and bridges for many years. More recently, interest has grown in using this instrumentation for monitoring the state of health of a structure without the need for a “triggering seismic event.” Structural damage will result in permanent changes in structural stiffness. These changes may be detected through structural monitoring. The use of vibrational monitoring is a field of structural analysis that is capable of assisting in both detecting and locating structural damage. This work is aimed at better understanding the variability in dynamic properties so that structural changes can be correctly interpreted if outlying data is recorded.

Introduction

Extensive work in the health monitoring field has been performed in aerospace, automotive, and general mechanical engineering. Many analytical techniques have been developed using data and testing from these disciplines. However, many of the issues regarding monitoring, data collection, processing, and analysis of large civil structures, such as bridges or buildings, are unique and require knowledgeable civil engineering professionals who have an understanding of structural and soil behavior (Aktan *et al.*, 1997). The use of vibrational techniques for structural health monitoring of civil engineering structures is increasing in worldwide usage.

Vibrational techniques focus on the use of small strain linear behavior of elastic structures subjected to low levels of excitation. Damage is assumed to occur at a single point in time, making detection dependent on recognizing a change in the pre-damaged linear system when compared to the post-damaged linear system. Many damage detection and recognition studies discuss data analysis, processing, and evaluation. Central to the problem of identifying and locating damage are the tasks of collection, processing, analyzing, and utilizing the data for the purpose of model calibration and updating.

Dynamic properties of a structure can provide a direct correlation between the physical properties of a structure and its structural integrity. Dynamic properties such as modal response (i.e. Natural Frequencies, Mode Shapes, Damping) can be measured and are dependent upon both the internal and external physical properties of

[1] Associate Professor, Department of Civil and Environmental Engineering, Utah State University, Logan, Utah, USA
[2] Structural Design Engineer, ARW Structural Engineers, Ogden, Utah, USA

a structure. It is known that a structural element's physical properties such as stiffness can change due to surrounding environmental changes. For example, a change in temperature can increase or decrease the stiffness of a structure, and that change in stiffness will result in a change in the dynamic response of the structure. Researchers have also found that heavy rains, diurnal variations, and strong winds can change a structure's natural frequency by up to 3 percent (Bradford *et al.*, 2004).

Dynamic response is unique to each structure and will remain nearly constant unless otherwise affected by changes or damage to the structure. These changes could lead to shifts in the modal parameters such as natural frequency, mode shapes, and damping (Halling *et al.*, 2001). However, localized damage may not always shift or change the modal parameters on a global response spectrum. Localized damage may have a significant effect on higher modes of response (Farrar and Cone, 1995). Using multiple sensors to monitor a structure can provide a more meaningful analysis than what an analysis from just one sensor would yield. Thus, this correlation between the dynamic properties and the actual physical properties of a specific structure can be beneficial in detecting structural changes and damage.

As mentioned above, the dynamic properties of a structure are based upon many factors. Inertia, material stiffness, member length, and mass are all material properties that have an effect on a structures dynamic response. Other factors include boundary conditions, composite behavior, temperature, and loading intensity. All of these factors, as well as establishing a modal response baseline need to be considered when monitoring a structure. Once established, the baseline can be used to compare to current modal responses being calculated, and if there is a difference in response then actions can be taken to investigate if damage or changes have occurred to the bridge. By obtaining these properties and with the aid of computers, structural models can be made which will solve for and extract the dynamic response of a structure, helping to determine the location of damage (Ball, 2005).

This study investigates the natural frequencies of an in-service freeway overpass when subjected to ambient vibration excitation.

Experimental Approaches

The experimental approaches can be separated into ambient vibration, forced vibration, and free vibration monitoring.

Ambient Vibration Excitation

Structural health monitoring is often performed by utilizing ambient vibration sources as structural excitation. Many studies have been conducted using a variety of ambient excitation sources. Some ambient sources are wind, seismic activity, traffic, waves or tidal fluctuations, and ground vibration generated by adjacent industries.

The reasons for the use of ambient excitation are varied, but include low cost, little to no disruption to traffic, long term excitation, and in some cases the frequency content is appropriate for the structure. Factors which make the use of ambient excitation less than ideal include the variability in amplitude, duration, direction, frequency content, and extreme difficulty in accurately measuring the excitation. It is this lack of definitive knowledge of the excitation that usually dictates the analysis techniques that are used.

Forced Vibration Excitation

Structural vibration monitoring using a known excitation source has several significant advantages over utilizing ambient excitation sources. Just as a monitoring scheme is designed to capture certain aspects of structural behavior, designing the excitation can also be of benefit in exposing desired behavioral aspects of the structure (Salawu and Williams, 1995). A researcher can design many excitation parameters, such as the type of wave form, forcing location, amplitude, frequency content, duration, and the time of the day.

By utilizing a known forcing function, many of the uncertainties in the data collection and processing can be avoided. Additionally, although at any given time a structure's response is due to a compilation of all sources of excitation, filtering can be used to determine the response due to the specific forcing. Forced vibration excitation amplitude can also be designed to be significantly higher than the ambient or electronic noise levels, helping to isolate changes in the system. These factors can result in significant advantages over ambient vibration testing, but may come at additional financial expense.

Free Vibration

Free vibrations occur in a flexible system when a body is released from its original or at-rest position. At release the structure begins to vibrate. Energy is dissipated as a result of friction or heat generation, resulting in the free vibration decay. A structure's free vibration characteristics can be analyzed to determine structural properties. Cunha et al. (2001) utilized the sudden release of a suspended mass from the deck of a large cable-stayed bridge to measure the free vibration of the bridge. Eberhard and Marsh (1997) performed a displacement induced free vibration test by applying transverse loads to a bent of a three-span reinforced concrete bridge. Free vibration testing was also performed by Kramer et al. (1999) on the Z-24 bridge, a three span pre-stressed concrete bridge located in Switzerland.

Long Term Monitoring of an In-Service Bridge

A long-term structural health monitoring project funded by the Utah Department of Transportation (UDOT) and the Federal Highway Administration (FHWA) was an overpass bridge at the 21st South Interchange of I-15 in Salt Lake City, Utah. This bridge is referred to as the I-80 Flyover Bridge. The goal of this research project was to install long-term instrumentation that could monitor changes in dynamic behavior of the bridge on a daily basis and to function as a recording station in the event of a triggering earthquake.

The I-80 Flyover serves as a connector from westbound I-80 to westbound SR-201. The bridge includes four individual structures, containing a total of 25 spans, with an overall length of 1.14 kilometers. The superstructure consists of a reinforced concrete deck supported by 3 steel I girders. The selected test structure is a long, multi-span structure with relatively tall columns, and contains several expansion joints. Other characteristics which factored into the selection of this bridge are that it is located only 6 kilometers away from the Wasatch Fault, a large normal fault capable of up to a Magnitude 7.5 event, and that it is founded on very soft deep Lake Bonneville sediments. It is the first bridge instrumented with strong motion instrumentation within the state of Utah. Figure 1 shows an aerial view of the I-80 Flyover.

Figure 1. Aerial Photograph of I-80 SR-201 Bridge.

Twelve modal frequencies were selected for monitoring. These modes are in the set of lower 26 modes identified from modal analysis. Modal frequency data for these twelve modes is tabulated in Table 1.

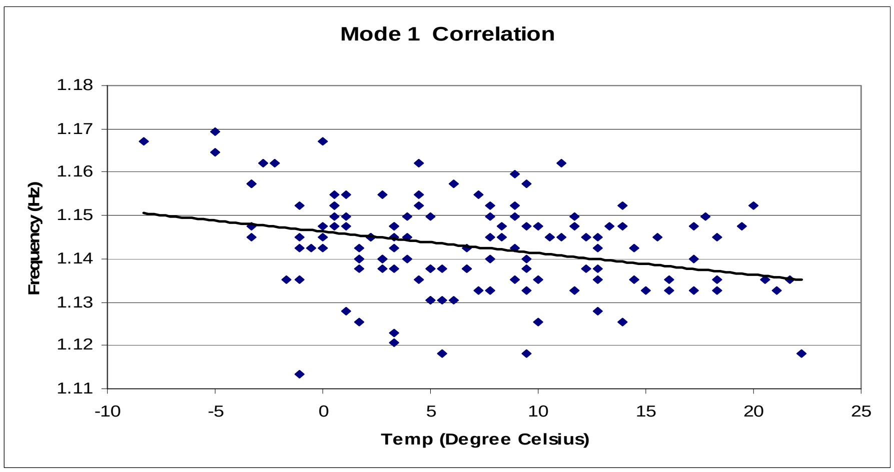
In an effort to show the correlation between frequency and temperature, scatter plots for the 12 selected modes were constructed and a linear regression trend line was fit to each data set. The data gathered during February and March, 2007 were used together to construct the plots. While the correlation coefficients are not large, modes 1, 2, 4, 13, 15, 20, 21, 23, and 25 clearly exhibited a downward trend in frequency as the temperature rises, whereas modes 9, 11, and 22 exhibit an upward trend in frequency with a rise in temperature. A representative sample of these plots is shown in Figures 2 a, b, c, d, and e.

Table 1. Modal Frequencies Chosen For Monitoring

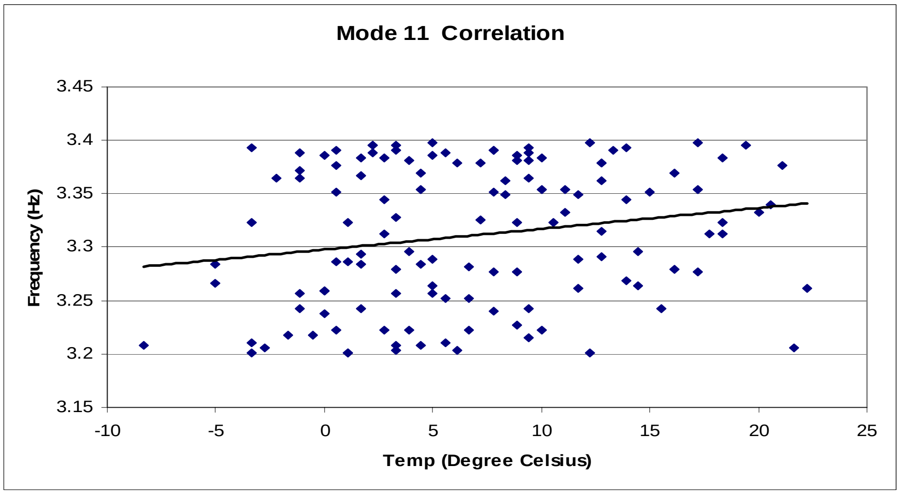
	Mean Natural Frequency (Hz)		
Mode	**Jun '01**	**Feb'07**	**Mar '07**
1	1.109	1.142	1.144
2	1.380	1.370	1.367
4	1.586	1.590	1.588
9	2.670	2.701	2.688
11	3.303	3.296	3.320
13	3.735	3.869	3.826
15	4.777	4.849	4.859
20	9.152	9.593	9.569
21	10.662	10.778	10.747
22	11.835	11.485	11.551
23	12.998	12.887	12.837
25	16.013	15.796	15.585

Although a downward trend in frequency as temperature increases may be anticipated due to decreases in material stiffness at higher temperatures, an upward trend could be the result of the structure expanding due to temperature increases, causing expansion joints and other connections to tighten. Clearly the fact that some modes increase and other decrease is a subject for further investigation.

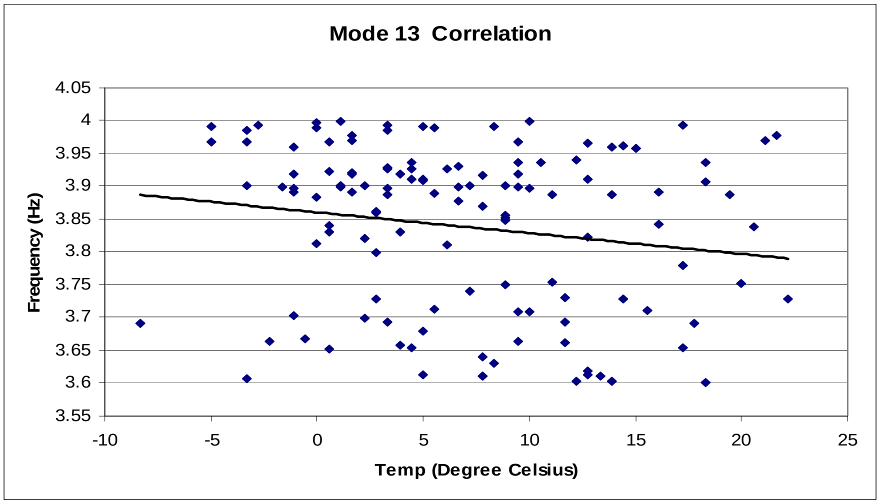
These plots also help to visualize graphically how the data is distributed for these tests. Tight groupings such as those seen for mode 20 (and to a lesser degree, mode 1) reflect that the standard deviations for those modes are smaller than the modes with wider spread data sets such as modes 13 and 22. There are outlying points in each graph, but it is assumed there have been enough data points taken that these points do not have a significant adverse influence on the results.



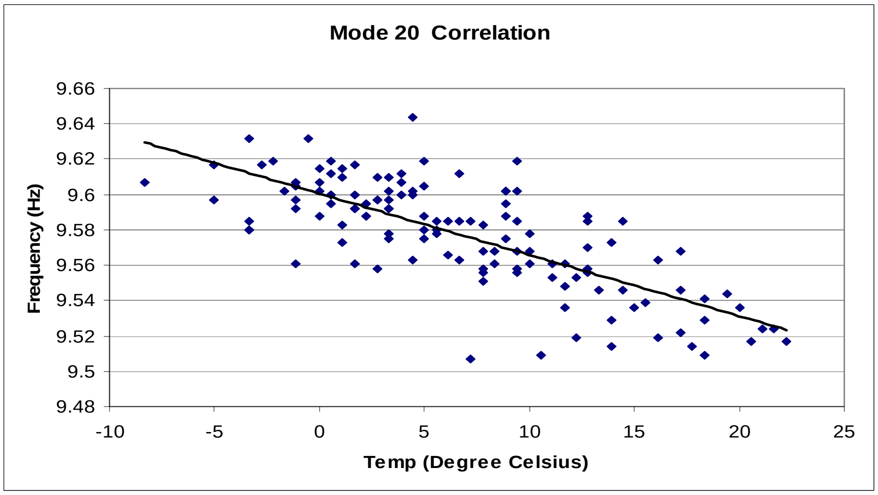
a)


Mode 11 Correlation
3.45
3.4
3.35
3.3
3.25
3.2
3.15
Frequency (Hz)
-10
-5
0
5
10
15
20
25
Temp (Degree Celsius)

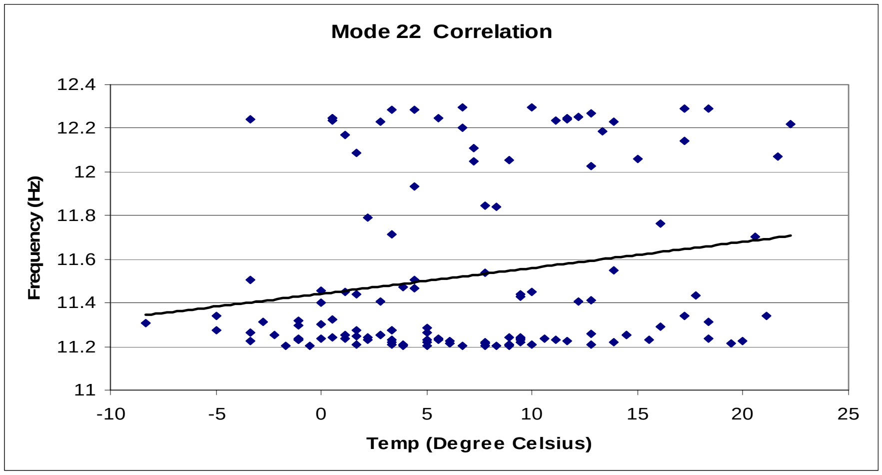
b)


Mode 13 Correlation
4.05
4
3.95
3.9
3.85
3.8
3.75
3.7
3.65
3.6
3.55
Frequency (Hz)
-10
-5
0
5
10
15
20
25
Temp (Degree Celsius)

c)


Mode 20 Correlation
9.66
9.64
9.62
9.6
9.58
9.56
9.54
9.52
9.5
9.48
Frequency (Hz)
-10
-5
0
5
10
15
20
25
Temp (Degree Celsius)

d)



e)

Figure 2. Correlation Plots of Temperature and Natural Frequency.

Studies have shown that extreme temperatures can cause a structure's natural frequencies to shift by up to 3 percent (Bradford *et al.*, 2004). To help reinforce these results, another test was completed in which separate samples were taken from February 13, 2007 to March 26, 2007. The times the samples were taken were solely dependent upon the temperature at the time of the sample. It was decided that data samples would be taken at any time of day if the temperatures were either -1˚ C (30˚ F), 4.5˚ C (40˚ F), or 10˚ C (50˚ F). Not only did the temperature need to be in one of those ranges for the time of sampling, but the temperatures had to be within that same range for at least two hours prior to the time of sampling. It was hoped this would allow the structure more time to become acclimated to the current temperature.

Table 2. Separated Temperature Statistics for Samples Taken Based on Temperature

	Ambient Test (-1˚ C)			Ambient Test (4.5˚ C)			Ambient Test (10˚ C)		
Mode	Mean Natural Frequency	Standard Deviation	Norm. Variation	Mean Natural Frequency	Standard Deviation	Norm. Variation	Mean Natural Frequency	Standard Deviation	Norm. Variation
1	1.146	0.01	0.99%	1.139	0.01	1.26%	1.140	0.01	1.02%
2	1.380	0.01	0.89%	1.370	0.01	0.95%	1.367	0.02	1.17%
3	1.480	0.03	1.72%	1.477	0.01	1.01%	1.477	0.01	0.99%
4	1.588	0.03	1.85%	1.584	0.03	1.70%	1.581	0.02	1.30%
5	1.728	0.03	1.75%	1.735	0.04	2.33%	1.742	0.05	2.61%
6	1.941	0.04	2.08%	1.940	0.04	2.25%	1.949	0.04	2.22%
7	2.289	0.03	1.37%	2.286	0.04	1.54%	2.281	0.03	1.31%
8	2.408	0.05	2.11%	2.403	0.04	1.84%	2.407	0.04	1.53%
9	2.670	0.12	4.42%	2.701	0.10	3.58%	2.688	0.09	3.22%
10	3.034	0.07	2.37%	3.037	0.08	2.76%	3.045	0.08	2.56%
11	3.320	0.06	1.93%	3.329	0.06	1.85%	3.326	0.06	1.77%
12	3.464	0.05	1.32%	3.452	0.05	1.44%	3.466	0.06	1.76%
13	3.829	0.12	3.26%	3.820	0.13	3.48%	3.809	0.14	3.76%
14	4.185	0.08	1.94%	4.190	0.07	1.67%	4.178	0.07	1.76%
15	4.865	0.05	1.06%	4.843	0.06	1.28%	4.847	0.04	0.80%
16	5.127	0.13	2.55%	5.112	0.12	2.27%	5.131	0.12	2.40%
17	5.597	0.11	2.02%	5.587	0.13	2.27%	5.541	0.10	1.81%
18	6.281	0.22	3.57%	6.227	0.18	2.91%	6.243	0.21	3.35%
19	7.619	0.28	3.68%	7.613	0.31	4.05%	7.567	0.27	3.56%
20	9.606	0.02	0.20%	9.590	0.02	0.18%	9.552	0.07	0.75%
21	10.782	0.06	0.52%	10.785	0.16	1.50%	10.744	0.11	1.02%
22	11.499	0.38	3.31%	11.618	0.45	3.85%	11.653	0.46	3.97%
23	12.925	0.05	0.35%	12.872	0.06	0.44%	12.845	0.10	0.75%
24	14.491	0.30	2.07%	14.374	0.24	1.68%	14.257	0.29	2.01%
25	15.588	0.50	3.21%	15.583	0.49	3.14%	15.735	0.47	2.98%
26	17.176	0.36	2.09%	16.948	0.34	2.03%	17.030	0.32	1.87%

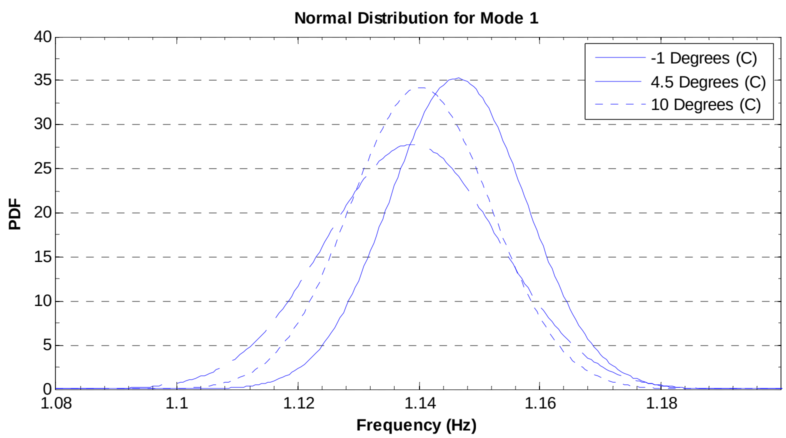
Table 2 shows the statistical information for the data sets of the three described temperature ranges. There exists a slight shift in frequencies from -1˚ C to the higher temperatures, while the standard deviations and normalized variations are relatively unaffected for the three different temperature ranges.

The shifts in natural frequencies for each temperature range with respect to other ranges are calculated and can be seen in Table 3. The shifts in the natural frequency for -1˚ C vs 4.5˚ C and -1˚ C vs 10˚ C ranged from 0.03 – 1.35 percent while the shifts for 4.5˚ C vs 10˚ C had a smaller range of 0.00-0.98 percent. This was an interesting result because the difference in temperature ranges is 5.5˚ C, yet the natural frequencies did not shift proportionally with the change in temperature. One possible reason for this is that the samples taken for a temperature of -1˚ C were below freezing. This extreme temperature could have more adverse affects on the bridge than the more mild temperatures of 4.5˚ C and 10˚ C.

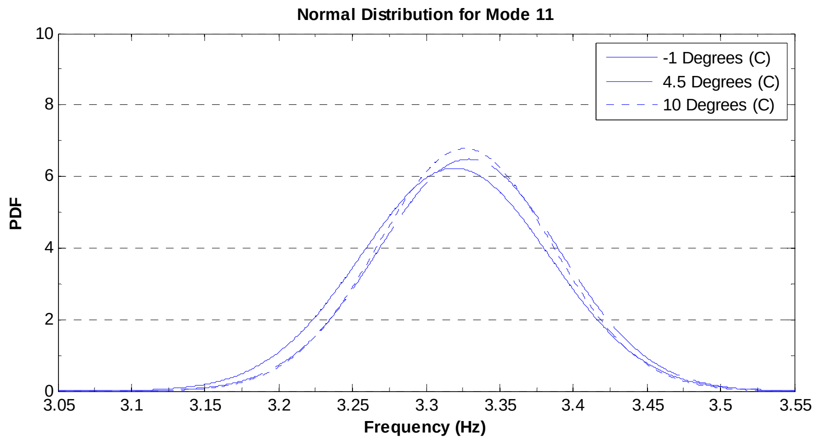
Table 3. Natural Frequency Variations by Temperature

	Mean Natural Frequency (Hz)			Temperature Comparison (% Shift)		
Mode	**-1˚ C**	**4.5˚ C**	**10˚ C**	**-1˚ C vs 4.5˚ C**	**-1˚ C vs 10˚ C**	**4.5˚ C vs 10˚ C**
1	1.146	1.139	1.140	0.65%	0.54%	0.12%
2	1.380	1.370	1.367	0.72%	0.92%	0.21%
4	1.588	1.584	1.581	0.26%	0.45%	0.19%
9	2.670	2.701	2.688	1.15%	0.69%	0.46%
11	3.320	3.329	3.326	0.28%	0.21%	0.08%
13	3.829	3.820	3.809	0.22%	0.53%	0.31%
15	4.865	4.843	4.847	0.46%	0.37%	0.09%
20	9.606	9.590	9.552	0.17%	0.57%	0.40%
21	10.782	10.785	10.744	0.03%	0.35%	0.38%
22	11.499	11.618	11.653	1.04%	1.34%	0.30%
23	12.925	12.872	12.845	0.41%	0.62%	0.21%

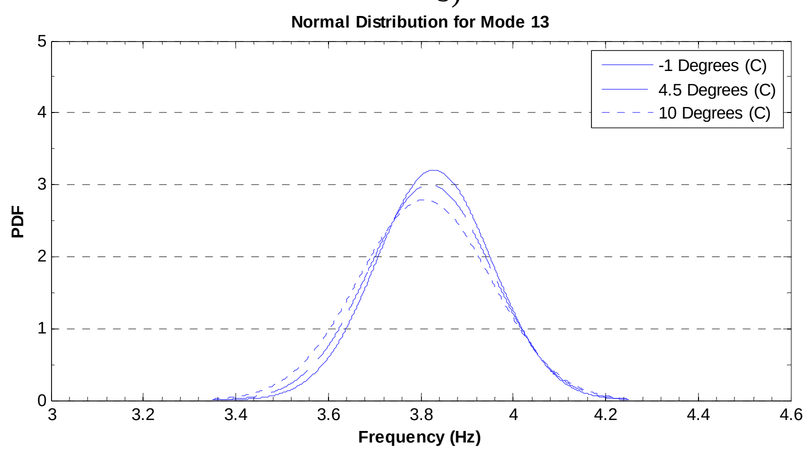
Normal distribution plots based on the standard deviation and mean values from Table 3 were also created. These plots are the corresponding probability density functions for each modal frequency, and they help to visualize how the standard deviations and mean natural frequencies are changing due to changes in temperature. Figures 3 a, b, c, d, and e show the normal distribution plots for a representative sample of the twelve monitored modes.



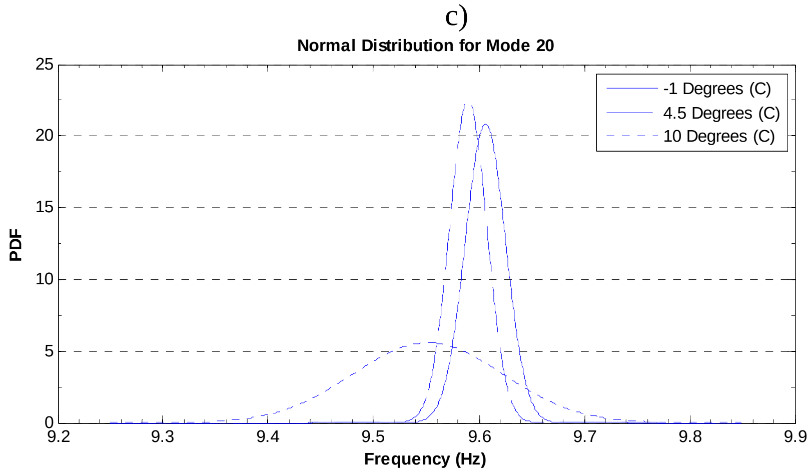
a)


Normal Distribution for Mode 11
PDF
Frequency (Hz)
-1 Degrees (C)
4.5 Degrees (C)
10 Degrees (C)
0
2
4
6
8
10
3.05
3.1
3.15
3.2
3.25
3.3
3.35
3.4
3.45
3.5
3.55

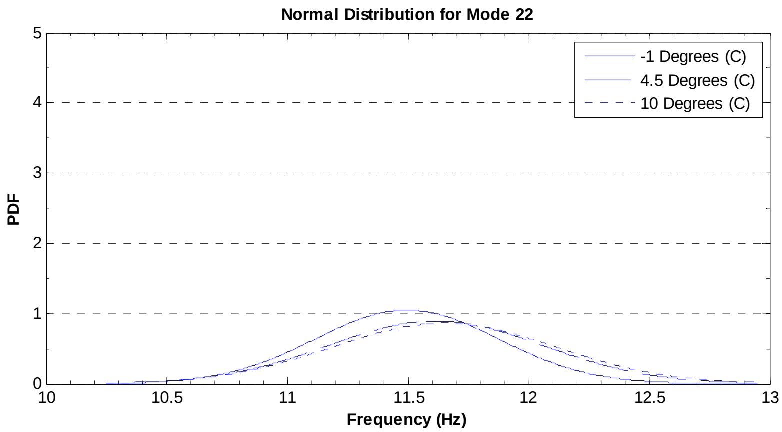
b)


Normal Distribution for Mode 13
PDF
Frequency (Hz)
-1 Degrees (C)
4.5 Degrees (C)
10 Degrees (C)
0
1
2
3
4
5
3
3.2
3.4
3.6
3.8
4
4.2
4.4
4.6

c)


Normal Distribution for Mode 20
PDF
Frequency (Hz)
-1 Degrees (C)
4.5 Degrees (C)
10 Degrees (C)
0
5
10
15
20
25
9.2
9.3
9.4
9.5
9.6
9.7
9.8
9.9

d)



e)

Figure 3. Normal Distribution for Modes 1, 11, 13, 20, and 22 at -1˚ C, 4.5˚ C, and 10˚ C.

These normal distribution plots help to visualize the unique behavior of each mode individually. Modes 11 and 13 and 25 reflect very little change, while modes 1, 2, 4, 21, and 23 clearly show that a shift in frequency has occurred. Although modes 11, 13, and 25 did not experience as significant a change as other modes, a slight shift in both the frequency and standard deviation still occurred.

The change in standard deviation with respect to frequency was also very different for each mode. At higher temperatures the standard deviations for modes 1, 2, 13, 21, 22, 23, and 25 became larger. The opposite happened for modes 4, 9, and 11 where their standard deviations became smaller for higher temperatures.

When the Figure 3 is compared to Figure 2 some interesting correlations can be made. Figure 3 was created using data that was sampled depending entirely on temperature, and Figure 2 was created using data that was sampled every six hours throughout the day regardless of the temperature at that time. Both sets of data showed that modes 1, 2, 4, 13, 15, 20, 21, 23, and 25 showed decreasing frequencies with increasing temperature, while modes 9, 11, and 22 showed increasing frequencies with increasing temperature. Even though the data selection criteria and processing were independent, the twelve monitored modes resulted in consistent results for both analyses.

Conclusions

The use of structural dynamic properties for damage detection is highly dependent on high-quality, long-term data that can be utilized to establish a "baseline" structure. This study includes correlations of modal natural frequencies with temperature over a range of temperatures.

The I-80 Flyover Bridge in Salt Lake City, Utah shows some distinct trends correlating changes in natural frequency with changes in temperature. The ranges of variations in the twelve monitored modes were as high as 3 percent, corresponding to changes in temperature of as much as 11˚C. The amount and direction (up or down) of the shifts were modal dependent. Three of the twelve monitored modes showed an increase in modal frequency as a result of an increase in temperature, while the other nine monitored modes showed a decrease in modal frequency as a result of an increase in temperature.

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References

Aktan, A. E., Farhey, D. N., Helmicki, A. J., Brown, D. L., Hunt, V. J., Lee, K. L., and Levi, A. (1997). "Structural identification for condition assessment: experimental arts." J. Struct. Engrg., 123(12), 1674-1684.

Ball, A.W. 2005. Modal analysis of a multi-span reverse curve steel girder bridge using computer modeling. M.S. thesis. Utah State University, Logan, Utah.

Bradford, S.C., Clinton, J.F., Favela, J., and Heaton, T.H. 2004. Results of Millikan Library forced vibration testing. Technical report: CaltechEERL: EERL-2004-03. California Institute of Technology.

Cunha, A., Caetano, E., and Delgado, R. (2001). "Dynamic tests on large cable-stayed bridge." J. Bridge Engrg., 6(1), 54–62.

Eberhard, M.O. and Marsh, M. L. (1997). "Lateral-load response of two reinforced concrete bents." J. Bridge Engrg., 123(4), 461–468.

Farrar, C.R., and K.M. Cone. 1995. Vibration testing of the I-40 bridge before and after the introduction of damage. Proceedings of the 13th International Modal Analysis Conference (IMAC XIII), February 1995, Nashville, Tennessee. p. 203-209.

Halling, M.W., Muhammad, I., and Womack, K.C. (2001). Dynamic field testing for condition assessment of bridge bents. Journal of Structural Engineering, ASCE. Vol. 127, No. 2, February 2001, p. 161-167.

Kramer, C., De Smet, C. A. M., and Peeters, B. (1999). "Comparison of ambient and forced vibration testing of civil engineering structures." Proc., 17th International Modal Analysis Conference (IMAC XVII), Kissimmee, FL.

Salawu, O. S., and Williams, C. (1995). “Bridge assessment using forced-vibration testing.” J. Struct. Engrg., 121(2), 161-173.