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Author(s):

Charles R. Farrar, ESA-EA, LANL
Scott W. Doebling, ESA-EA, LANL
Erik G. Straser, ESA-EA, LANL
Phillip Cornwell, Rose-Holman Institute of Technology*

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VARIABILITY OF MODAL PARAMETERS MEASURED ON THE ALAMOSA CANYON BRIDGE

Charles R. Farrar[1], Scott W. Doebling[1], Phillip J. Cornwell[3], Erik G. Straser[3]

[1]Engineering Analysis Group (ESA-EA)
MS P946
Los Alamos National Laboratory
Los Alamos, NM 87545

[2]Dept. of Mechanical Eng.
Rose-Holman Inst. of Tech.
Terre Haute, IN, 47803

[3]J. A. Blume Earthquake Eng. Center
Stanford University
Stanford, CA 94305-4020

ABSTRACT. *A significant amount of work has been reported in technical literature regarding the use of changes in modal parameters to identify the location and extent of damage in structures. Curiously absent, and critically important to the practical implementation of this work, is an accurate characterization of the natural variability of these modal parameters caused by effects other than damage. To examine this issue, a two-lane, seven-span, composite slab-on-girder bridge near the town of Truth or Consequences in southern New Mexico was tested several times over a period of nine months. Environmental effects common to this location that could potentially produce changes in the measured modal properties include changes in temperature, high winds, and changes to the supporting soil medium. In addition to environmental effects, variabilities in modal testing procedures and data reduction can also cause changes in the identified dynamic properties of the structure.*

In this paper the natural variability of the frequencies and mode shapes of the Alamosa Canyon bridge that result from changes in time of day when the test was performed, amount of traffic, and environmental conditions will be discussed. Because this bridge has not been in active use throughout the testing period, it is assumed that any change in the observed modal properties are the result of the factors listed above rather than deterioration of the structure itself.

1. INTRODUCTION.

Recent advances in wireless, remotely monitored data acquisition systems coupled with the development of modal-based damage detection algorithms make the possibility of a self-monitoring bridge appear to be within the capabilities of current technology. However, before such a system can be relied upon to perform this monitoring, the variability of the modal properties that are the basis for the damage detection algorithm must be understood and quantified. This understanding is necessary so that the artificial intelligence/expert system that is employed to discriminate when changes in modal properties are indicative of damage will not yield false indications of damage.

Although the number of papers reporting experimental modal analyses results from bridge structures has greatly increased in recent years, very few of the articles examine the variability in the modal properties that can arise from changes in environmental conditions or from random and systematic errors inherent in the data acquisition/data reduction process. Tumer and Pretlove [1] state that there is "some evidence" to show that the natural frequencies of the bridge structures they tested did not change more than 0.5% as a result of environmental effects. Rytter [2] summarizes a paper by Askegard and Mossing [3] where changes in the resonant frequency and damping are plotted as a function of the time of year and the ambient temperature. Readings were taken over a three year period and it was found that the resonant frequency would vary as much as 10% during the year with the lower frequencies occurring when it was hottest. This cycle was seen to repeat itself for the three years during which data were obtained. Rucker, et al. [4] and Rohrmann and Rucker [5] report results of tests performed on bridges over a six-month time period. This study shows that the natural frequencies increase as the mean temperature decreases. On a time scale of days these tests show the first mode frequency varying from 2.3 to 2.8 Hz.

These results imply that a thorough study of the variability in modal parameters must be conducted before modal-based damage identification algorithms can be applied with any confidence. This paper reports results from tests specifically designed to examine the variability in modal parameters of a bridge caused by environmental effects, service conditions and data reduction methods.

2. TEST STRUCTURE.

The Alamosa Canyon Bridge has seven independent spans with a common pier between successive spans. An elevation

Figure 1. Elevation View of the Alamosa Canyon Bridge

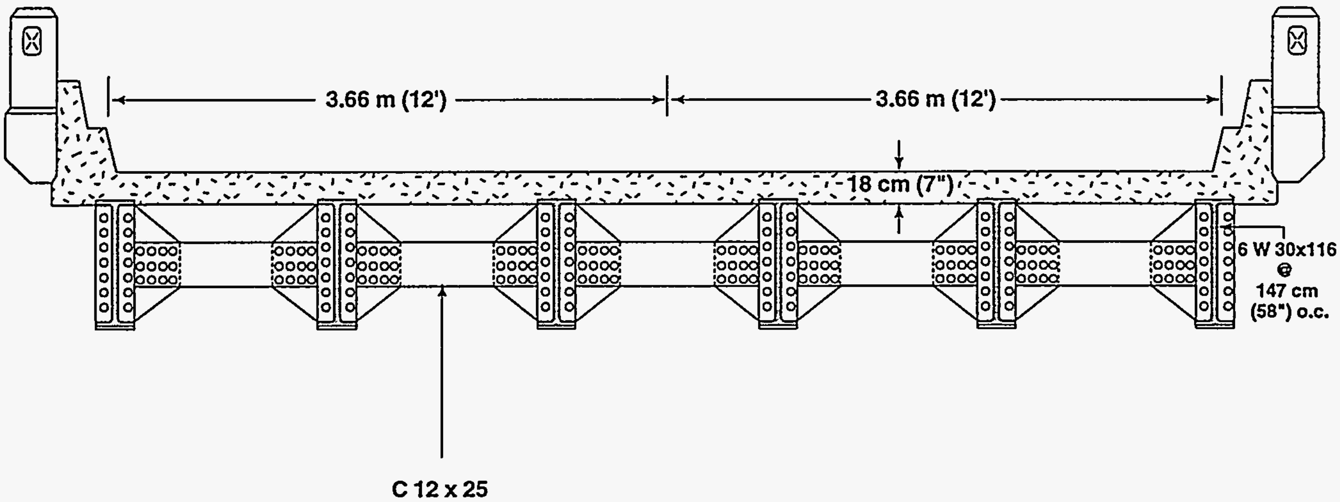

Figure 2. Cross-section view of the Alamosa Canyon Bridge.

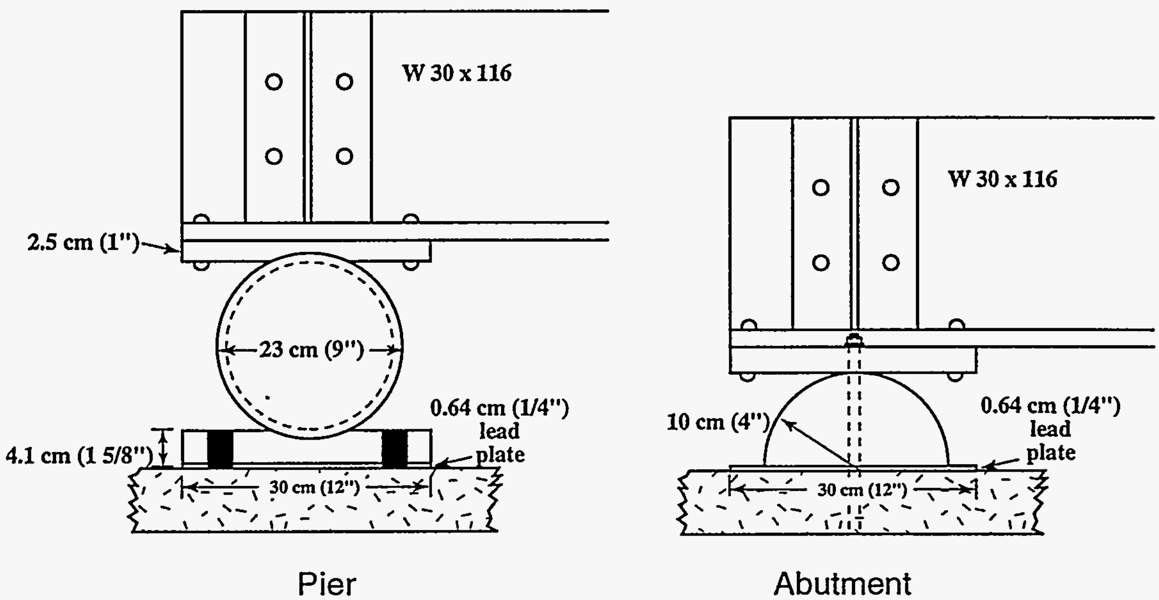

Figure 3. Support details at the pier and abutment.

view of the bridge is shown in Fig.1. Each span consists of a concrete deck supported by six W30x116 steel girders. The roadway in each span is approximately 7.3 m (24 ft) wide and 15.2 (50 ft) long. A concrete curb and guard rail are integrally attached to the deck. Plans for the bridge do not show shear studs on the top flanges of the girders. Inspection of the bridge showed that the upper flanges of the girders are imbedded in the concrete. Four sets of cross braces are equally spaced along the length of the span between adjacent girders. The cross braces are channel sections (C12x25). A cross section of the span at a location showing the interior cross braces is shown in Fig. 2. At the pier the girders rest on rollers as shown in Fig. 3. Also shown in Fig. 3 is the connection detail at the abutment where the beams are bolted to a half-roller to simulate a pinned connection. The bridge is aligned primarily in a north-south direction.

3. DATA ACQUISITION

The data acquisition system used in the vibration tests consisted of a Toshiba TECRA 700 laptop computer, four HP 35652A input modules that provide power to the accelerometers and perform analog to digital conversion of the accelerometer signals, an HP 35651A signal processing module that performs the needed fast Fourier transform calculations, and a commercial data acquisition/signal analysis software package from Hewlett Packard. A 3500 watt GENERAC Model R-3500 XL AC generator was used to power this system.

The data acquisition system was set up to measure acceleration and force time histories and to calculate frequency response functions (FRFs), power spectra and coherence functions. Sampling parameters were specified that calculated the FRFs from a 16-s time window discretized with 2048 samples. The FRFs were calculated for a frequency range of 0 to 50 Hz at a frequency resolution of 0.0625 Hz. A Force window was applied to the signal from the hammer's force transducer and an exponential windows were applied to the signals from the accelerometer. AC coupling was specified to minimize DC offsets.

A PCB model 086B50 impact sledge hammer was used to provide the excitation source. The hammer weighed approximately 53.4 N (12 lbs) and had a 7.6-cm-dia. (3-in-dia) steel head. The sensor in the hammer had a nominal sensitivity of 0.73 mV/lb and a peak amplitude range of 5000 lbs. The hammer tip designated by the manufacturer as "super-soft" was used to broaden the time duration of the impact and, hence, better excite the low frequency response of the bridge.

A Wilcoxon Research model 736T accelerometer was used to make the driving point acceleration response measurement adjacent to the hammer impact point. This accelerometer has a nominal sensitivity of 100 mV/g, a specified frequency range of 5 - 15,000 Hz, and a peak amplitude range of 50 g. Two 2.54-cm-sq. (1-in-sq.) aluminum blocks were epoxied to the top surface of the bridge in order to mount the driving point accelerometers.

PCB model 336c piezoelectric accelerometers were used for the vibration measurements. These accelerometers have a nominal sensitivity of 1 V/g, a specified frequency range of 1 - 2000 Hz, and an amplitude range of ± 4 g. All accelerometers were mounted to the bottom flange of the steel girders using PCB model 080A05 magnetic mounts.

A total of 31 acceleration measurements were made on the concrete deck and on the girders below the bridge as shown in Fig. 4. Five accelerometers were spaced along the length of each girder. Because of the limited number of data channels measurements were not made on the girders at the abutment or at the pier. Two excitations points were located on the top of the concrete deck. Point A was used as the primary excitation location. Point B was used to perform a reciprocity check. The force-input and acceleration-response time histories obtained from each impact were subsequently transformed into the frequency domain so that estimates of the PSDs, FRFs, and coherence functions could be calculated. Thirty averages were typically used for these estimates. With the sampling parameters listed above and the overload reject specified, data acquisition for a specific test usually occurred over a time period of approximately 30 - 45 minutes.

Five indoor-outdoor thermometers were located across the center of the span. Two thermometers were positioned such that their outdoor sensor was taped to the outside web surface at midheight of the exterior girders. The indoor readings from these two thermometers were made on the inside, bottom flange of the exterior girders. A third thermometer was taped to the underside of the concrete deck at the middle of the span. The outside sensor for this thermometer was located adjacent to the indoor sensor yielding almost identical temperature readings. The two remaining thermometers were located on the top side of the bridge. Their outside sensors were taped to the bridge deck immediately adjacent to the concrete curbs. The indoor sensor was located on the top of the guard rail. All sensors were shaded from direct sun light either by the bridge itself or by shades made from duct tape and cups. All temperature reading were made by visual inspection of the thermometers.

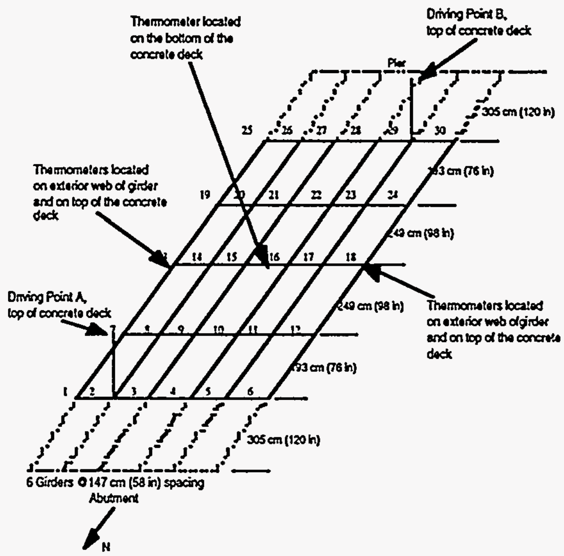

Figure 4. Accelerometer, impact, and thermometer locations.

4. RECIPROCITY AND LINEARITY CHECKS

Almost all modal analysis algorithms are developed based on the assumption that the structure will exhibit linearity and reciprocity. Therefore, before any tests were performed to investigate the variability of modal parameters, tests were first conducted to check the validity of these assumptions.

First, measurements were made using two impact levels, whose PSD amplitudes are approximately a factor of 5 different, to test the linearity of the response over this range of loading. Figure 5 shows an overlay of the input PSDs and Fig. 6 shows the corresponding overlay of FRF magnitudes when these inputs were applied at Pt. A and response measurements were made at location 6 (See Fig. 4). These tests were performed sequentially between 4:00 and 6:00 AM when temperature differences across the bridge were negligible. Coherence functions for these measurements yielded values of 0.9 or greater across the entire spectrum. Figure 6 shows that the structure was exhibiting linear response in the range of 5 to 25 Hz. Above 30 Hz there is a noticeable difference in the two measurements suggesting the possibility that nonlinearities were be excited in this frequency range or that signal-to noise-ratios were poor thus providing the appearance of nonlinear response. This frequency range also corresponds to the lowest coherence in the measurements.

Figure 7 shows the FRF magnitudes for an impact applied at Pt. A (See Fig. 4) and a response measured at Point B. Also shown in this figure is the FRF magnitude for an impact applied at Pt. B and a response measured at Point A. A similar plot is shown in Fig. 8, but here the accelerometers at Pts. A and B have been switched. By switching the accelerometers the reciprocity being measured is that of the structure alone. From Figures 7 and 8 it is evident that the structure itself is exhibiting reciprocity in the 5 to 25 Hz region. Above thirty Hz one could not make this claim. Also, when Fig. 7 is compared to Fig. 8 it is evident that the electronics are contributing to the loss of reciprocity, particularly at the third natural frequency near 11.5 Hz.

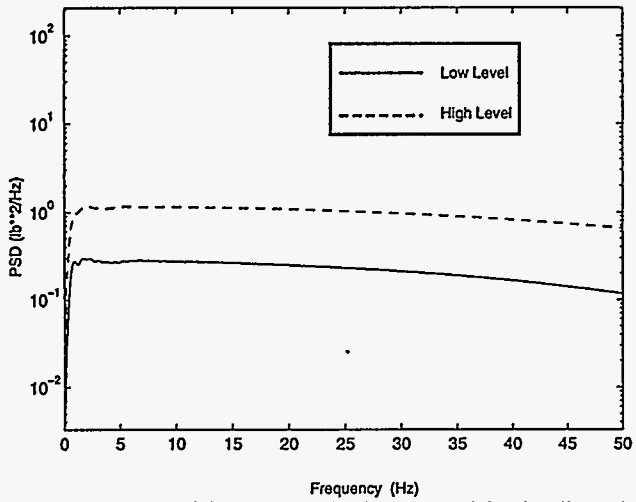

Figure 5. PSDs of impact excitations used in the linearity check at driving Pt. A.

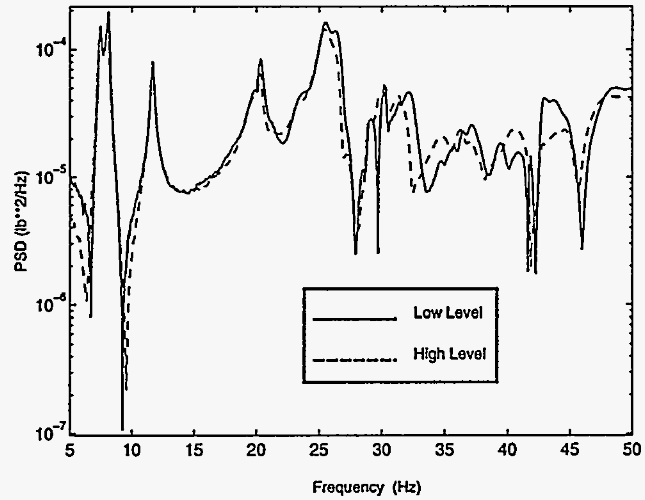

Figure 6. FRF magnitudes measured at location 6 (impact applied at point B)

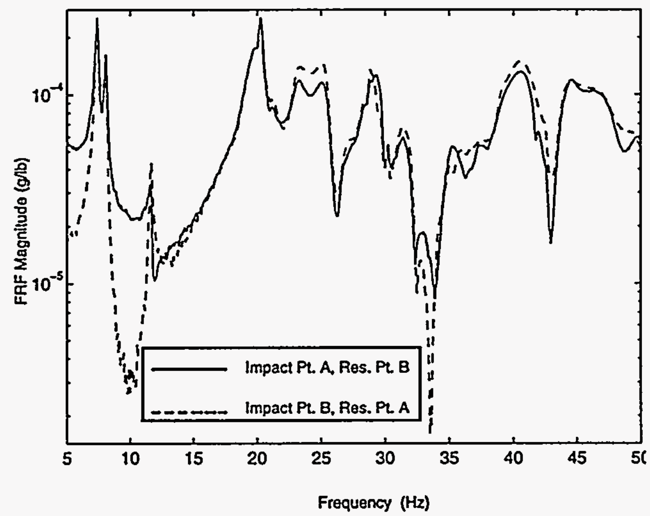

Figure 7. FRF magnitudes used to check reciprocity of the structure and the electronics.

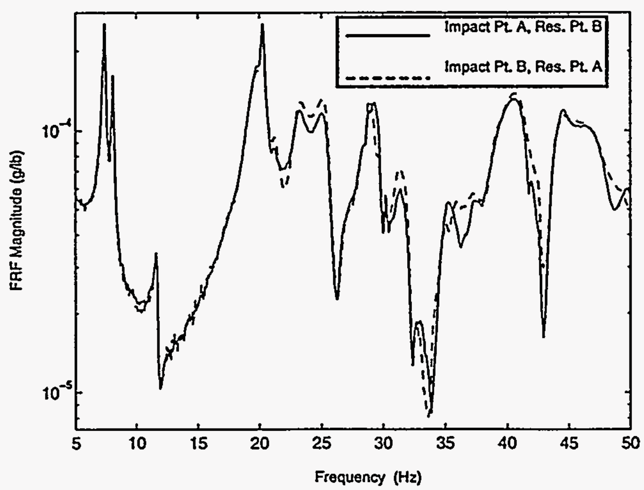

Figure 8. FRF magnitudes used to check reciprocity of the structure only.

5. TEMPORAL VARIABILITY

The experimental modal analyses were performed on data measured at two hour increments over a 24-hour time-period to investigate the change in the modal properties as a function of time. The experimental modal analysis method was identical to that described in [6].

The first step in the analysis of the data was the determination of the approximate number of modes to be fit. This number is determined using the Multivariate Mode Indicator Function (MIF) [7] and the Complex Mode Indicator Function (CMIF) [8]. In this analysis, the CMIF and MIF were computed, and then zoomed to frequency bands of 10 Hz at a time. Approximately 9 modes of significant strength were located between 0 Hz and 30 Hz by inspection of the CMIF and MIF, as discussed in [6].

The next step in the analysis was the application of ERA [9]. The ERA procedure is based upon the formation of a Hankel matrix containing the measured discrete-time impulse response data, computed using the inverse fast Fourier transform of the measured FRFs. The model resulting from the ERA analysis had 80 modes, but it was known from examination of the MIF and CMIF that the data contains only about 9 modes in the band of interest. Thus it was necessary to apply some discrimination procedures to select the modes that were physically meaningful. There are three indicators developed specifically for use with ERA [10]: Extended Modal Amplitude Coherence (EMAC), Modal Phase Collinearity (MPC), and Consistent Mode Indicator (CMI), which is the product of EMAC and MPC. Typically, values of EMAC = 0.7, MPC = 0.7, and CMI = 0.5, and then see if all of the modes of interest (as determined by MIF and CMIF inspection) are preserved. In the current study, all of the 9 modes of interest passed this criteria.

Statistical uncertainty bounds on the measured frequency response function magnitude and phase were computed from the measured coherence functions, assuming that the errors were distributed in a Gaussian manner, according to the method developed by Bendat and Piersol [11]:

Monte Carlo analyses were then performed, using the previously determined uncertainty bounds on the FRFs, to establish statistical uncertainty bounds on the identified modal parameters (frequencies, damping ratios, and mode shapes)[11]. The basic idea of a Monte Carlo analysis is the repeated simulation of random input data, in this case the FRF with estimated mean and standard deviation values, and compilation of statistics on the output data, in this case the ERA results.

Figure 9 shows the first mode frequencies along with their 95% confidence limits plotted as a function of the measurement completion time. Also plotted on Fig. 9 is the change in temperature between the two thermometer readings made on the concrete deck (east - west). This figure clearly shows that the change in modal frequencies are related to the temperature differentials across the deck. The first mode frequency varies approximately 5% during this 24 Hr time period. Similar variations and correlation with deck temperature differentials were observed for the other modes of the structure.

6. VARIABILITIES CAUSED BY EXCITATION SOURCE

Only two excitations sources were used in these tests: hammer impact and ambient. A comparison of these test procedures and the statistics associated with the results obtained from these tests is given in [13]. Comparisons of the ambient test results were made to impact test results from data measured at the same time of day to minimize the differences that can be attributed to thermal effects. These results show difference in the the frequencies. Mode shapes calculated from the data sets corresponding to the different excitation methods were very similar with no observable trends that could be related to the excitation method. The damping values obtained, however, did show significant differences. Lower damping was found during the ambient test. This difference can be attributed to the significantly lower levels of excitation in the ambient tests. Other excitation methods that should be investigated to complete this study include random and swept sine excitations using an electrodynamic or hydraulic shaker, repeatable controlled impact from a drop hammer and step relaxation methods.

7. VARIABILITY CAUSED BY VEHICLE WEIGHT

Impact modal test were performed with four cars on the bridge and compared to impact tests without cars. For one span the concrete deck and reinforcing steel weighs approximately 525 kN (118 kips) and the steel girders, cross bracing and gusset plates weigh 178 kN (40 kips) yielding a total span weight of 703 kN (158 kips). The four cars that were placed on the bridge weighed approximately 99 kN (22 kips). Assuming the parked cars have no other effects on the dynamics of the structure other than the addition of mass, they should lower the frequencies by a value proportional to the square root of the mass ratios, in this case approximately 6.4%. This result was not observed in the measured modal frequencies from test performed at similar time of the day (again, to minimize thermal effects) as shown in Table I.

TABLE I

Change in Modal Frequencies Resulting From Added Mass of Cars

Mode	Measured Modal Frequency, No Cars (Hz)	Predicted Drop in Modal Frequency, Resulting from Car Mass (Hz)	Measured Modal Frequency Change Resulting From Car Mass (Hz)
1	7.38	6.91	7.43
2	8.04	7.53	8.03
3	11.5	18.3	11.5
4	19.5	21.9	19.8
5	23.4	21.9	23.4
6	25.2	23.6	25.6

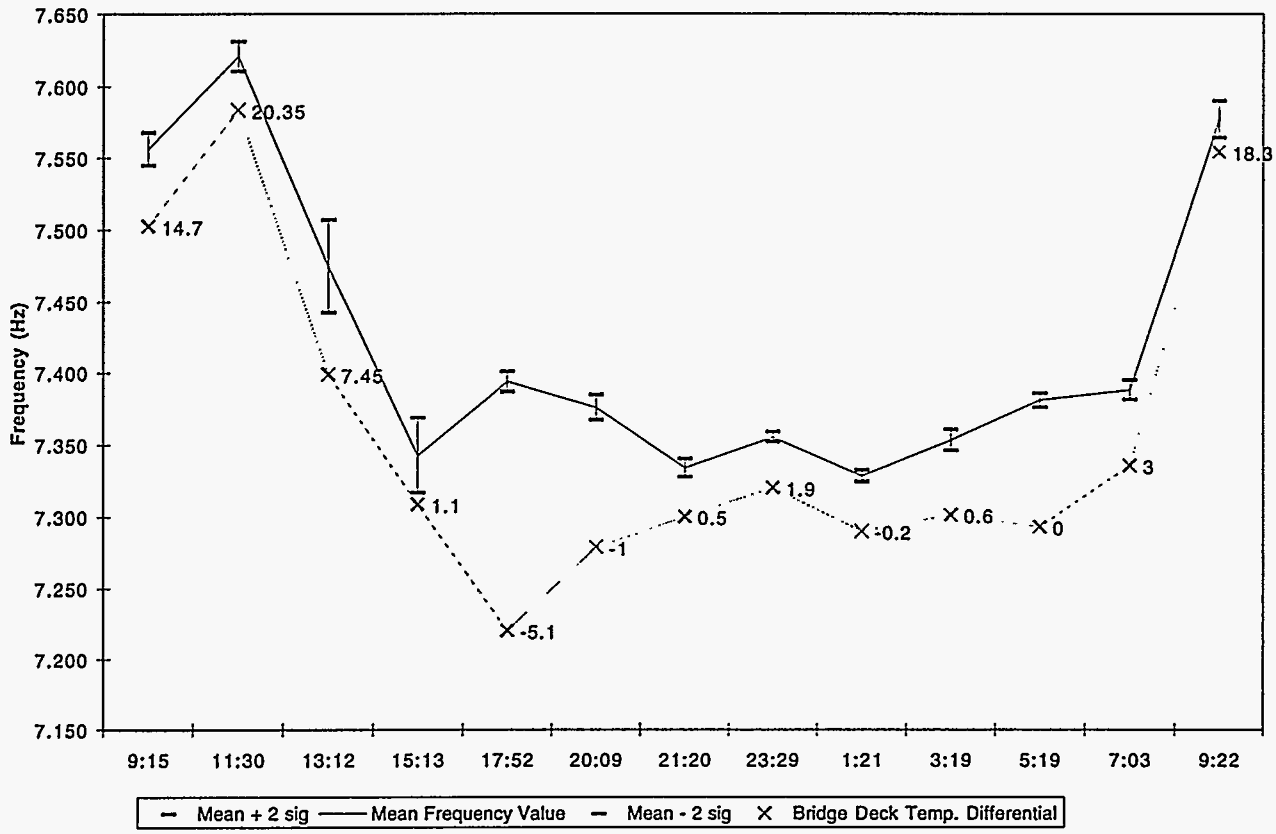

Figure 9. Change in the first mode frequency during a 24 hr time period.

The Alamosa Canyon Bridge was rated for 133 kN (30 kip). In theory two vehicles, one from each direction, could be on the span at one time adding 366 kN (60 kips) to the weight of the structure. Based on the added mass alone, these vehicles could reduce the measured resonant frequencies 19% from those measured when no vehicles are on the bridge.

8. VARIABILITIES INTRODUCED DURING DATA REDUCTIONS

Variabilities can be introduced in the data reduction process based on the parameter identification algorithms employed and the analyst that is applying them. Specific analyses were not performed to investigate the variabilities resulting from such effects. It is the authors' opinion, however, that these effects will be significantly smaller than the environmental effects, particularly for the forced-vibration tests. Reduction of ambient vibration data is not as well documented as that for forced vibration data, hence, it is assumed that more variability will be introduced in the associated data reduction process. The statistical analysis methods summarized in [6] can be used quantify the variabilities introduced by different data reduction algorithms and the variabilities introduced by a particular analyst.

9. SUMMARY AND CONCLUSIONS

The tests reported in this paper show that significant variability can be introduced into the experimental modal analysis results obtained from an *in situ* bridge. The variability arises from environemental effects such as thermal gradients, service conditions such as traffic loads, and from variabilities associated with the measurement and data reduction process.

Statistical analysis can be used to quantify the random errors introduced during the measurement process. Variability introudced during the data reduction process can be quantified by having the modal analysis performed by different analysts using different parameter identification routines. However, it is the authors experience that if the analysts reducing the data are experienced the variability resulting from this source is considerably less than the variabilities caused by environmental effects and service conditions.

Before modal-based damage identification procedures can be routinely applied to a bridge, particularly in a remote monitoring mode, the effects of these variability sources on the modal-based parameters monitored by the damage identification algorithm must be quantified. Such quantification may require measurements to be made at different times of the year, during different weather conditions, and when the bridge is experiencing different service conditions. Based on the results of such tests, it is conceivable that bounds can be developed for the modal-based paramters that are monitored by the damage identification system. Damage must cause changes in these parameters that are outside these bounds for a difinitive statement to be made regarding the onset of damage in the bridge.

10. ACKNOWLEDGMENTS

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