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Title: Lessons Learned From Applications Of Vibraton-Based
Damage Identification Methods To A Large Bridge Structure

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Lessons Learned from Applications of Vibration-Based Damage Identification Methods to a Large Bridge Structure

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ABSTRACT

Over the past 30 years detecting damage in a structure from changes in dynamic parameters has received considerable attention from the aerospace, civil, and mechanical engineering communities. The general idea is that changes in the structure's physical properties (i.e., stiffness, mass, and/or damping) will, in turn, alter the dynamic characteristics (i.e., resonant frequencies, modal damping, and mode shapes) of the structure. Properties such as the flexibility matrix, stiffness matrix, and mode shape curvature, which are obtained from modal parameters, have shown promise for locating structural damage. However, the application of these techniques to large civil engineering structures is limited because of the inability to find structures that the owners will allow to be damaged. Also, the cost associated with testing these structures can be prohibitive. In this paper, the authors' experiences with performing modal tests on a large highway bridge, in its undamaged and damaged state, for the purpose of damage identification will be summarized. Particular emphasis will be made on the lessons learned form this experience and the lessons learned from recent tests on another bridge structure.

INTRODUCTION

The interest in the ability to monitor a structure and detect damage at the earliest possible stage is pervasive throughout the civil, mechanical, and aerospace engineering communities. Current damage detection methods are either visual or localized experimental methods such as acoustic or ultrasonic methods, magnetic field methods, radiography, eddy-current methods and thermal field methods (Doherty, 1987) All these experimental methods require that the vicinity of the

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damage is known *a priori* and that the portion of the structure being inspected is readily accessible. Subjected to these limitations, these experimental methods can detect damage on or near the surface of the structure. The need for more global damage detection methods that can be applied to complex structures has led to the development of methods that examine changes in the vibration characteristics of the structure.

Global damage or fault detection, as determined by changes in the dynamic properties or response of structures, is a subject that has received considerable attention in the literature beginning approximately 30 years ago. Based on the amount of information provided regarding the damage state, these methods can be classified as providing four levels of damage detection. The four levels are (Rytter, 1993):

1. Identify that damage has occurred;
2. Identify that damage has occurred and determine the location of damage;
3. Identify that damage has occurred, locate the damage, and estimate its severity; and
4. Identify that damage has occurred, locate the damage, estimate its severity, and determine the remaining useful life of the structure.

The basic premise of the global damage detection methods that examine changes in the dynamic properties is that modal parameters, notably resonant frequencies, mode shapes, and modal damping, are a function of the physical properties of the structure (mass, damping, stiffness, and boundary conditions). Therefore, changes in physical properties of the structure, such as its stiffness or flexibility, will cause changes in the modal properties. A detailed summary of vibration-based damage detection methods can be found in (Doebling et al., 1996) where the general limitations and successes of these methods are discussed. Also discussed in this reference are specific examples of the application of vibration-based damage identification algorithms to bridge structures.

It is the intent of this paper to provide a brief summary of one such study that was made of the I-40 Bridge over the Rio Grande in Albuquerque, NM. This project was performed jointly with researchers from Sandia National Laboratory and New Mexico State University. A detailed discussion of the bridge geometry, testing procedures, data acquisition, and data reduction can be found in (Farrar et al., 1994) In particular, the lessons learned from this study will be emphasized. The authors hope that this discussion will help other investigators avoid some of the pitfalls that were experienced on this test. Additional lessons learned have come from a subsequent test on Alamosa Canyon Bridge in southern New Mexico. Finally, it should be pointed out that many of the lessons have been learned through interactions with other engineers including those at Los Alamos and Sandia National Laboratories and faculty and staff at the University of Cincinnati and Drexel University.

TEST STRUCTURE GEOMETRY

The I-40 Bridges over the Rio Grande in Albuquerque, NM, razed in 1993, formerly consisted of twin spans (there are separate bridges for each traffic direction) made up of a concrete deck supported by two welded-steel plate girders and three steel stringers. Loads from the stringers were transferred to the plate girders by floor beams located at 6.1 m (20 ft) intervals. Cross-bracing was provided between the floor beams. Fig. 1 shows an elevation view of the portion of the bridge that was tested. The cross-section geometry of each bridge is shown in Fig. 2. Each bridge was made up of three identical sections. Each section had three spans; the end spans were of equal length, approximately 39.9 m (131 ft), and the center span was approximately 49.4 m (163 ft) long. All subsequent discussions of the I-40 bridge will refer to the bridge carrying eastbound traffic, particularly the three eastern spans, which were the only ones tested.

DAMAGE SCENARIOS

The damage that was introduced was intended to simulate fatigue cracking that has been observed in plate-girder bridges. This type of cracking results from out-of-plane bending of the plate girder web and usually begins at welded attachments to the web such as the seats supporting the floor beams. Four levels of damage were introduced to the middle span of the north plate girder close to the seat supporting the floor beam at mid-span. Damage was introduced by making various torch cuts in the web and flange of the girder.

The first level of damage, designated E-1, consisted of a two-foot-long, 10-mm-wide (3/8-in-wide) cut through the web centered at mid-height of the web. Next, this cut was continued to the bottom of the web to produce a second level of damage designated E-2. For the third level of damage, E-3, the flange was then cut halfway in from either side directly below the cut in the web. Finally, the flange was cut completely through for damage case E-4 leaving the top 4 ft of the web and the top flange to carry the load at this location. The various levels of damage are shown in Fig. 3. Photographs of the damage can be found in (Farrar et al., 1994).
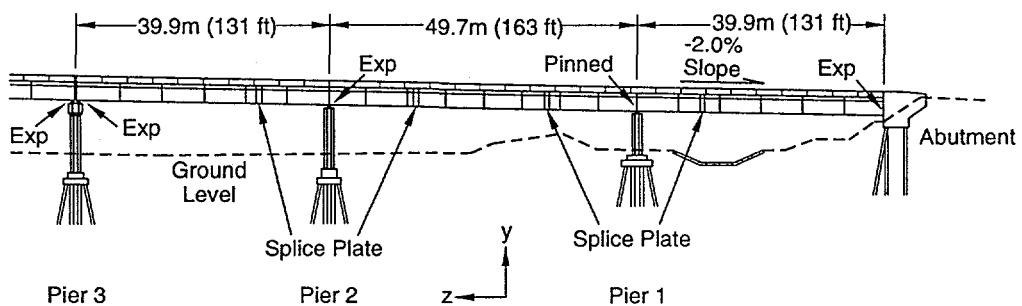
Fig. 1. Elevation view of the portion of the I-40 Bridge that was tested.
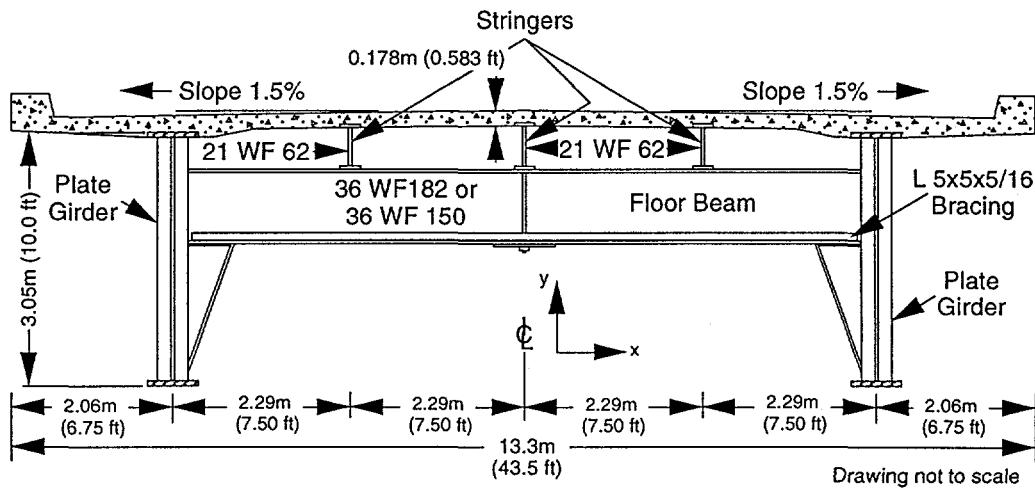
Fig. 2. Corss-section geometry of the I-40 Bridge.

E-1   E-2   E-3   E-4
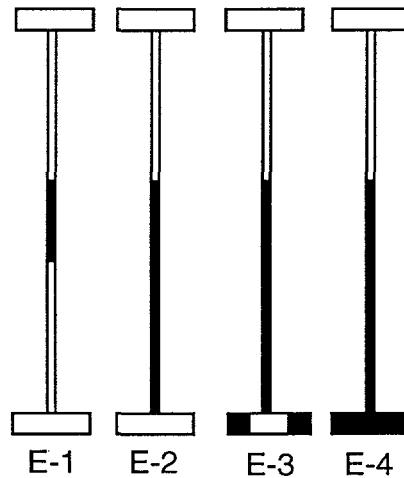
Fig. 3 Damage Scenarios

MEASUREMENT OF MODAL PROPERTIES

The damage identification methods used in this study analyze mode shape data and, in some cases, the corresponding resonant frequencies. The experimental procedures used to obtain these data are described in this section. To obtain these data, a forced vibration test was conducted on the undamaged bridge. Eastbound traffic had been transferred to a new bridge just south of the one being tested. The

westbound traffic continued on the original westbound bridge. Excitation from traffic on the adjacent bridges could be felt on the bridge being tested. Next, the four different levels of damage were introduced into the middle span of the north plate girder. Forced vibration tests similar to those done on the undamaged structure were repeated after each level of damage had been introduced. A detailed summary of the experimental procedures can be found in (Farrar et al., 1994).

Excitation

Engineers from Sandia National Laboratory (SNL) provided a hydraulic shaker that generated the measured force input. The SNL shaker consists of a 96.5 kN (21,700-lb) reaction mass supported by three air springs resting on top of drums filled with sand. A 9.79 kN (2200-lb) hydraulic actuator bolted under the center of the mass and anchored to the top of the bridge deck provided the input force to the bridge. A random-signal generator was used to produce a 2000-lb peak-force uniform random signal over the frequency range of 2 to 12 Hz. An accelerometer mounted on the reaction mass was used to measure the force input to the bridge. The shaker was located on the eastern-most span directly above the south plate girder and midway between the abutment and first pier. A more detailed summary of the Sandia shaker can be found in (Mayes and Nusser, 1994).

Data Acquisition

The data acquisition system used in these tests consisted of a computer workstation that controlled 29 input modules and a signal processing module. The workstation was also the platform for a commercial data-acquisition/signal-analysis/modal-analysis software package. The input modules provided power to the accelerometers and performed analog-to-digital conversion of the accelerometer voltage-time histories. The signal-processing module performed the needed fast Fourier transform calculations. A 3500-watt AC generator was used to power this system in the field.

Two sets of integrated-circuit, piezoelectric accelerometers were used for the vibration measurements. A coarse set of measurements (SET1) was first made. These accelerometers were mounted in the vertical direction, on the inside web of the plate girder, at mid-height and at the axial locations shown in Fig. 4. A more refined set of measurements (SET2) was made near the damage location. Eleven accelerometers were placed in the global Y direction at a nominal spacing of 4.88 m (16 ft) along the mid-span of the north plated girder. All accelerometers were located at mid-height of the girder. The spacing of these accelerometers relative to the damage is shown in Fig. 5.
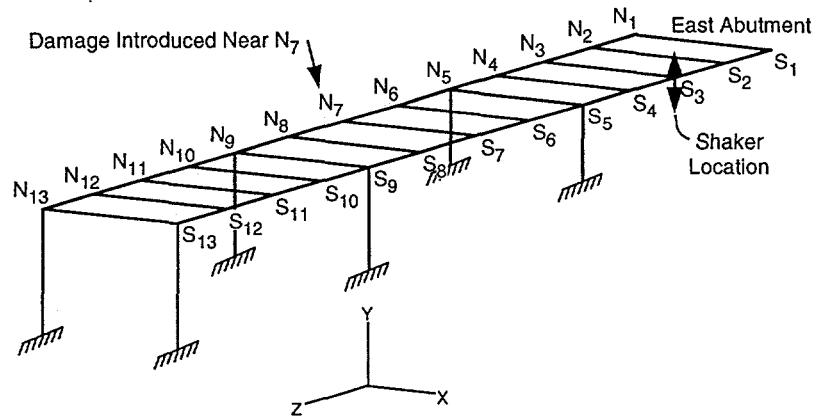
Fig. 4. Set1 (coarse) acceleromater locations.
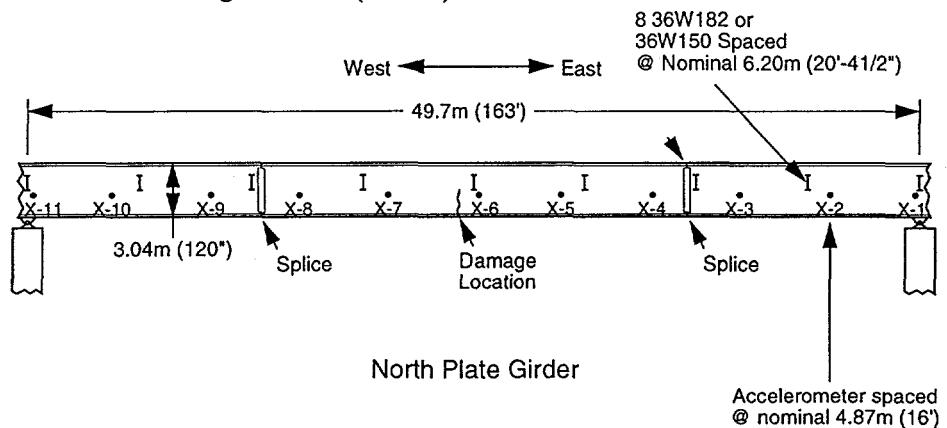
Fig. 5. Set2 (refined) accelerometer locations.

Modal Parameter Identification

Standard experimental modal analysis procedures were applied to data obtained from the SET1 accelerometers during the forced vibration tests to identify the modal parameters of the bridge in its damaged and undamaged condition. In this context experimental modal analysis refers to the procedure whereby a measured excitation (random, sine, or impact force) is applied to a structure, and the structure's response (acceleration, velocity, or displacement) is measured at discrete locations that are representative of the structure's motion. Both the excitation and the response time histories are transformed into the frequency domain in the form of frequency response functions (FRFs). Modal parameters (resonant frequencies, mode shapes, modal damping) can be determined by curve-fitting a Laplace domain representation of the equations of motion to the measured frequency domain data (Ewins, 1985). A rational-fraction polynomial, global, curve-fitting algorithm in a commercial modal analysis software package was used for these fits. Figure 6 shows the first three modes of the undamaged bridge identified from these data. By
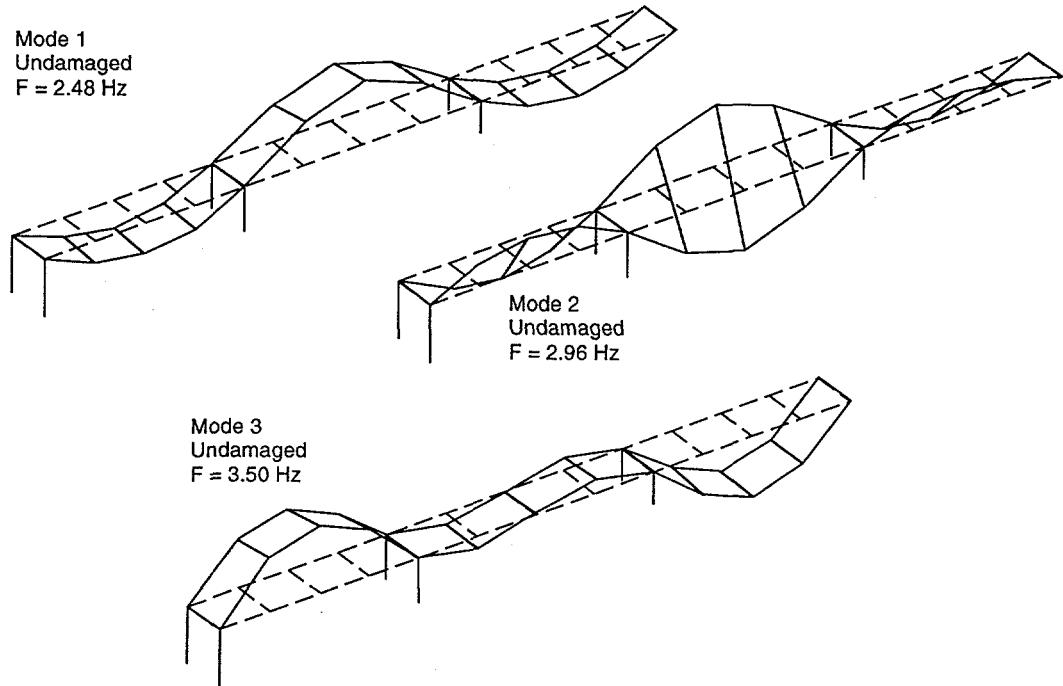
Fig. 6. First three modes measured on the undamaged structure.

measuring the input force and the corresponding driving point acceleration, these mode shapes can be unit-mass normalized.

Immediately after the forced vibration tests with the SET1 accelerometers were complete, the random excitation tests were repeated using the refined SET2 accelerometers. For these tests the input was not monitored. Operating shapes were determined from amplitude and phase information contained in the cross-power spectra (CPS) of the various accelerometer readings relative to the accelerometer X-3 shown in Fig. 5. Determining operating shapes in this manner, as discussed by (Bendat and Piersol, 1980), simulates the methods that would have to be employed when the responses to ambient excitations are measured. For modes that are well spaced in frequency these operating shapes will closely approximate the mode shapes of the structure. However, without a measure of the input force these modes cannot be mass normalized.

INFLUENCE OF DAMAGE ON CONVENTIONAL MODAL PROPERTIES

Changes in Resonant Frequencies and Modal Damping

Table I summarizes the resonant frequency and modal damping data obtained during each modal test of the undamaged and damaged bridge. No significant changes in the dynamic properties can be observed until the final level of damage is introduced. At the final level of damage the resonant frequencies for the first two modes have dropped to values 7.6 and 4.4 percent less, respectively, than

those measured during the undamaged tests. For modes where the damage was introduced near a node for that mode (modes 3 and 5) no significant changes in resonant frequencies can be observed.

Changes in Mode Shapes

A modal assurance criterion (MAC), sometimes referred to as a modal correlation coefficient (Ewins, 1985), was calculated to quantify the correlation between mode shapes measured during different tests.

Table II shows the MAC values that are calculated when mode shapes from tests t17tr (damage level E-1), t18tr (damage level E-2), t19tr (damage level E-3), and t22tr (damage level E-4) are compared to the modes measured on the undamaged forced vibration test, t16tr. The MAC values show no change in the mode shapes for the first three stages of damage. When the final level of damage is introduced, significant drops in the MAC values for modes 1 and 2 are noticed. These two modes are shown in Fig. 7 and can be compared to similar modes identified for the undamaged bridge in Fig 6. When the modes have a node near the damage location (modes 3 and 5), no significant reductions in the MAC values are observed, even for the final stage of damage. This result corresponds to the observed similarity in mode 3 shown in Figs. 6 and 7.

From the observed changes in modal parameters it is clear that damage can only be definitively identified after the final cut was made in the bridge. Prior to the final cut, one could not say that the changes observed were caused by damage or were within the repeatability of the tests. In two tests at increasing levels of damage (t17tr and t18tr) the resonant frequencies were actually found to increase slightly from that of the undamaged case. These slight increases in frequency were measured independently by other researchers studying this bridge at the same time (Farrar, et al. (1994)) and are assumed to be caused by changing test conditions. The examination of changes in the basic modal properties (resonant frequencies and mode shapes) demonstrates the need for more sophisticated methods to examine modal data for indications of damage. Also, the need for statistical analysis of the data and quantification of the environmental effects on the measured modal properties is evident when one considers the small changes that are being examined. The topic of variability in modal properties and statistical analysis is discussed in more detail in (Farrar et al., 1997) and (Doebling et al., 1997).

DAMAGE IDENTIFICATION STUDIES

For comparative purposes, five linear modal-based damage identification algorithms were applied to data obtained from the I-40 bridge in its undamaged and damaged

Mode 1
After Final Stage of Damage
F = 2.30 Hz

Mode 2
After Final Stage of Damage
F = 2.84 Hz

Mode 3
After Final Stage of Damage
F = 3.49 Hz
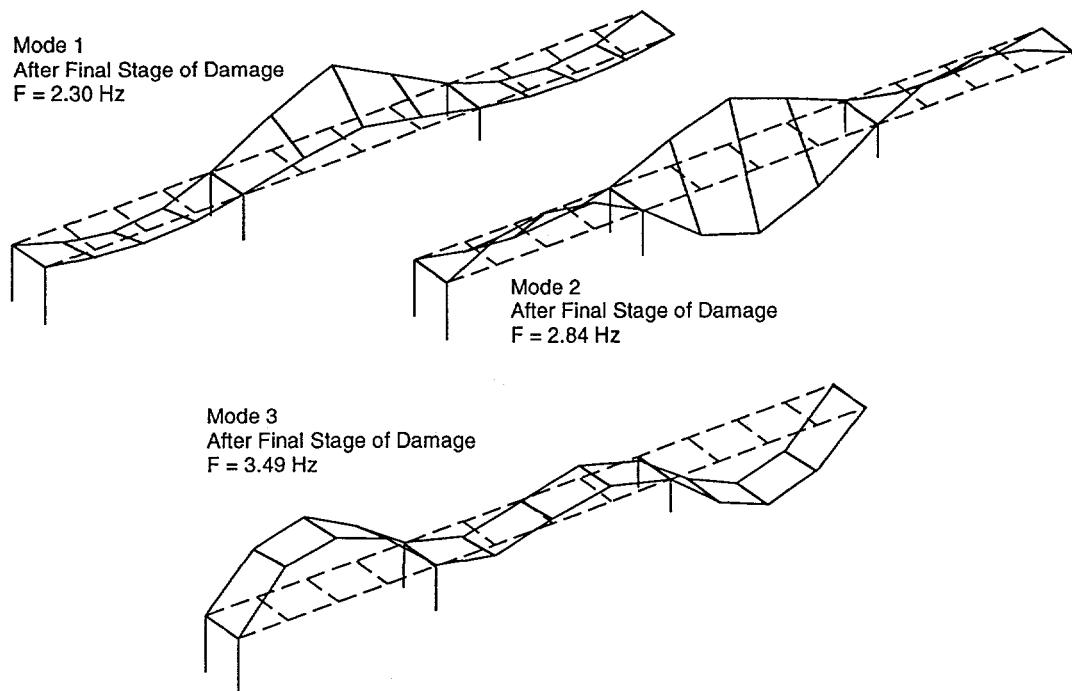
Fig. 7. First three modes measured after the final level of damage.

TABLE I

Resonant Frequencies and Modal Damping Values Identified from Undamaged and Damaged
Forced Vibration Tests Using SET1 Accelerometers

		Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6
Test Designation	Damage Case	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)
t16tr	Undamaged	2.48/ 1.06	2.96/ 1.29	3.50/ 1.52	4.08/ 1.10	4.17/ 0.86	4.63/ 0.92
t17tr	E-1 cut at center of web	2.52/ 1.20	3.00/ 0.80	3.57/ 0.87	4.12/ 1.00	4.21/ 1.04	4.69/ 0.90
t18tr	E-2 cut extended to bottom flange	2.52/ 1.33	2.99/ 0.82	3.52/ 0.95	4.09/ 0.85	4.19/ 0.65	4.66/ 0.84
t19tr	E-3 bottom flange cut half way	2.46/ 0.82	2.95/ 0.89	3.48/ 0.92	4.04/ 0.81	4.14/ 0.62	4.58/ 1.06
t22tr	E-4 bottom flange cut completely	2.30/ 1.60	2.84/ 0.66	3.49/ 0.80	3.99/ 0.80	4.15/ 0.71	4.52/ 1.06

TABLE II
Modal Assurance Criteria:
Undamaged and Damaged Forced Vibration Tests

Modal Assurance Criteria	Undamaged (test t16tr) X First level of damage, E-1 (test t17tr)					
Mode	1	2	3	4	5	6
1	0.996	0.006	0.000	0.003	0.001	0.003
2	0.000	0.997	0.000	0.005	0.004	0.003
3	0.000	0.000	0.997	0.003	0.008	0.001
4	0.004	0.003	0.006	0.984	0.026	0.011
5	0.001	0.008	0.003	0.048	0.991	0.001
6	0.001	0.006	0.000	0.005	0.005	0.996
Modal Assurance Criteria	Undamaged (test t16tr) X Second level of damage, E-2, (test t18tr)					
Mode	1	2	3	4	5	6
1	0.995	0.004	0.000	0.004	0.001	0.002
2	0.000	0.996	0.000	0.003	0.002	0.002
3	0.000	0.000	0.999	0.006	0.004	0.000
4	0.003	0.006	0.005	0.992	0.032	0.011
5	0.001	0.006	0.008	0.061	0.997	0.004
6	0.002	0.004	0.000	0.005	0.005	0.997
Modal Assurance Criteria	Undamaged (test t16tr) X Third level of damage, E-3 (test t19tr)					
Mode	1	2	3	4	5	6
1	0.997	0.002	0.000	0.005	0.001	0.001
2	0.000	0.996	0.001	0.003	0.002	0.002
3	0.000	0.000	0.999	0.006	0.006	0.000
4	0.003	0.005	0.004	0.981	0.032	0.011
5	0.001	0.006	0.004	0.064	0.995	0.003
6	0.002	0.002	0.000	0.004	0.009	0.995
Modal Assurance Criteria	Undamaged (test t16tr) X Fourth level of damage, E-4 (test t22tr)					
Mode	1	2	3	4	5	6
1	0.821	0.168	0.002	0.001	0.000	0.001
2	0.083	0.884	0.001	0.004	0.001	0.002
3	0.000	0.000	0.997	0.005	0.007	0.001
4	0.011	0.022	0.006	0.917	0.010	0.048
5	0.001	0.006	0.003	0.046	0.988	0.002
6	0.005	0.005	0.000	0.004	0.009	0.965

condition. These algorithms require mode shape data (in some cases unit-mass normalized mode shape data) and resonant frequencies. A summary of these algorithms and their implementation for the study reported herein can be found in (Farrar and Jauregui, 1996). All five methods are based on the observation that in the vicinity of damage there will be a local increase in the structure's flexibility. This increase will alter the mode shapes of the structure in the damage vicinity, hence a comparison of mode shape data processed by these five different methods, before and after damage, should reveal the location of the damage. These methods provide Level 2 damage indication as discussed in the introduction.

Tables III and IV summarize the results from applying the five damage detection algorithms to the experimental modal data from the SET1 and SET2

TABLE III
Summary of Damage Detection Results using Experimental Modal Data from Coarse Set of Accelerometers (SET1)

Damage Case	E-1	E-2	E-3	E-4
Damage Index Method	**	**	**	*
Mode Shape Curvature Method	**	*	**	*
Change in Flexibility Method	O	O	**	*
Change in Uniform Load Surface Curvature Method	O	O	O	*
Change in Stiffness Method	**	**	**	*
* Damage located, ** Damage located using only 2 modes; O Damage not located				

TABLE IV
Summary of Damage Detection Results using Experimental Modal Data from Refined Set of Accelerometers (SET2)

Damage Case	E-1	E-2	E-3	E-4
Damage Index Method	●	●	●	●
Mode Shape Curvature Method	● ● ●	● ●	●	●
Change in Flexibility Method	O	O	O	●
Change in Uniform Load Surface Curvature Method	O	● ● ●	●	●
Change in Stiffness Method	O	O	O	●
● Damage located, ● ● Damage narrowed down to two locations, ● ● ● Damage narrowed down to three locations, O Damage not located				

instruments, respectively. In this study, the Damage Index Method was found to have performed the best. All methods were able to definitively locate the damage for the final case, E-4. For the intermediate damage cases mixed results were obtained.

IN HIND SIGHT, THINGS WE SHOULD HAVE DONE

Based on subsequent analysis and observations related to the I-40 bridge tests (Doebling and Farrar, 1997), subsequent tests on another bridge (Doebling et al., 1997 and Farrar et al., 1997), interactions with other researchers in the field (particularly those at the Univ. of Cincinnati and Drexel Univ.), and review of the technical literature related to bridge testing, there are several things that should have been done during these tests (and have been done on subsequent tests) to improve the confidence in the damage ID results. These improvements are listed below:

Perform A More Thorough Pre-Test Visual Inspection.

Visual inspection is used to ascertain the initial condition of the structure. Pay particular attention to boundary conditions and changes to the neighboring vicinity of the test structure. During the tests visual inspection revealed that the east end of the top portion of the south girder was no longer in contact with the concrete at the top of the abutment. During earlier tests observations of the end condition were not made.

Perform Linearity Check.

The system identification portion of the experimental modal analysis procedure typically relies on the assumption that the structure is linear. Linearity can be checked, to some degree, by exciting the structure at different levels and overlaying the measured FRFs for a particular point. Ideally, with the thought of an on-line monitoring system in mind, these different excitation levels would span the range of loading observed during ambient traffic vibration measurements. On subsequent bridge tests it has been observed linearity was only observable over a portion of the spectrum as shown in Fig. 9. Also, a change in the linearity properties can in itself be an indication of damage.
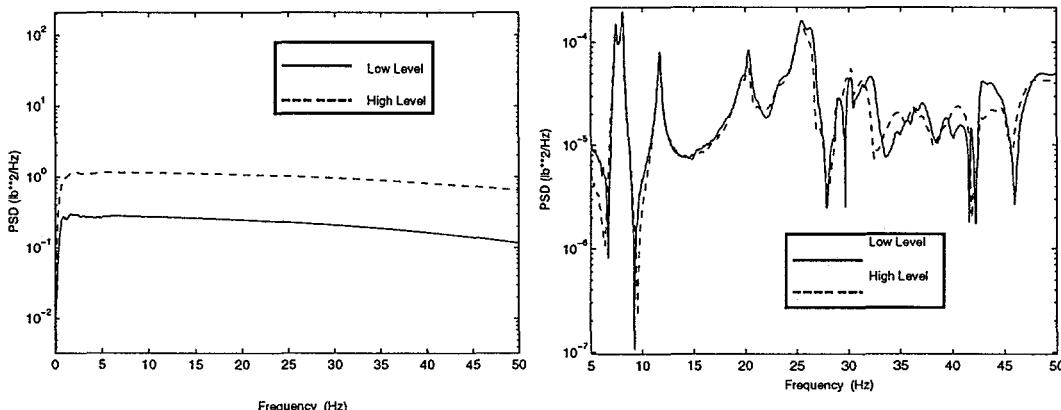

Fig. 9. PSDs of impact excitations used in the linearity check. Corresponding FRF magnitudes are shown on the right.

Perform Reciprocity Checks.

In addition to the assumption of linearity, the system identification portion of the experimental modal analysis typically relies on the assumption that the structure will exhibit reciprocity. Performing a reciprocity check is much more involved when a large shaker is being used because of the setup time involved in relocating the shaker. Also, to check the reciprocity of the structure alone, one must relocate the accelerometers and cables as well as the shaker. Without moving the instrumentation, the reciprocity check will involve reciprocity of the electronics as well as that of the structure. The difference in reciprocity of the structure itself and reciprocity of the structure and the electronics can be seen in Fig. 10 where the check was performed on the Alamosa Canyon Bridge (Farrar, et al., 1997). Because, in general, the electronics will not be moved once the test has started, the latter test is more representative of the reciprocity of the system. Figure 10 shows that reciprocity was only observed over a portion of the spectrum.

Perform As Many Environmental and Testing Procedure Sensitivity Studies As Possible.

Sensitivity of modal test results to environmental conditions and test procedures such as changes in temperature, traffic loading, wind, excitation method, etc. should be quantified to the extent possible. Subsequent tests (after the potential damage has occurred) should be performed under similar environmental conditions using similar test procedures, if possible. Also, a baseline noise measurement should be made for the data acquisition system.
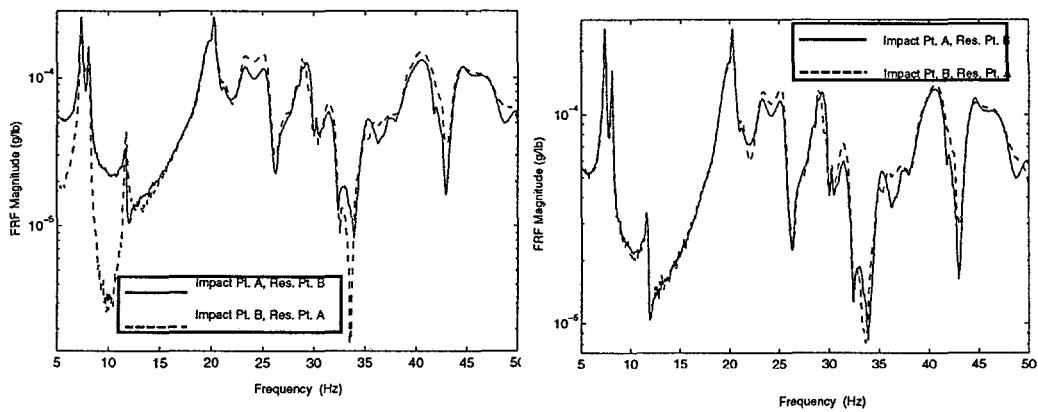

Fig. 10. FRF reciprocity check of the structure and electronics. FRF reciprocity check of the structure only is shown on the right.

Perform False-Positive Studies

As a means of quantifying the effects observed in the sensitivity studies, sets of data from the undamaged structure should be analyzed with the damage id algorithm to demonstrate that the algorithm will not falsely predict damage when in fact none has occurred.

Perform Statistical Analysis

Closely coupled to the false-positive study is the need to perform statistical analysis of the measured modal properties. Performing statistical analysis of the identified modal parameters as discussed in (Doebling, et al., 1997) is imperative if one is to establish that changes brought about by damage are greater than the test-to-test repeatability.

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. A thorough study of the variability of the modal parameters must be conducted before modal-based damage identification algorithms can be applied with any confidence.

Figure 11 shows the first mode frequencies for the Alamosa Canyon Bridge in southern New Mexico along with their 95% confidence limits plotted as a function of the measurement completion time. Also plotted on Fig. 12 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. Figure 11 clearly shows the need for performing a statistical analysis of the identified modal properties before a damage ID algorithm is applied to these quantities.

(Doebling, et al., 1997) shows the results of similar analyses applied to the estimation of modal damping, mode shapes and mode shape curvature. Figure 12 shows the first mode of the I-40 bridge in its undamaged state and after the first level of damage. In this figure points 1-13 correspond to accelerometer locations S-1 - S-13 in Fig. 4. Points 14 - 26 correspond to accelerometer locations N-1 - N-13 with the damage at point 20. Error bounds for the modal amplitudes have been calculated by Monte Carlo statistical procedures. From this figure it is clear that there is a statistically significant change in the mode shape at this first level of damage. However, the results shown in this plot can not be used to definitively state that the change in the mode shape resulted from damage as opposed to changing environmental conditions.
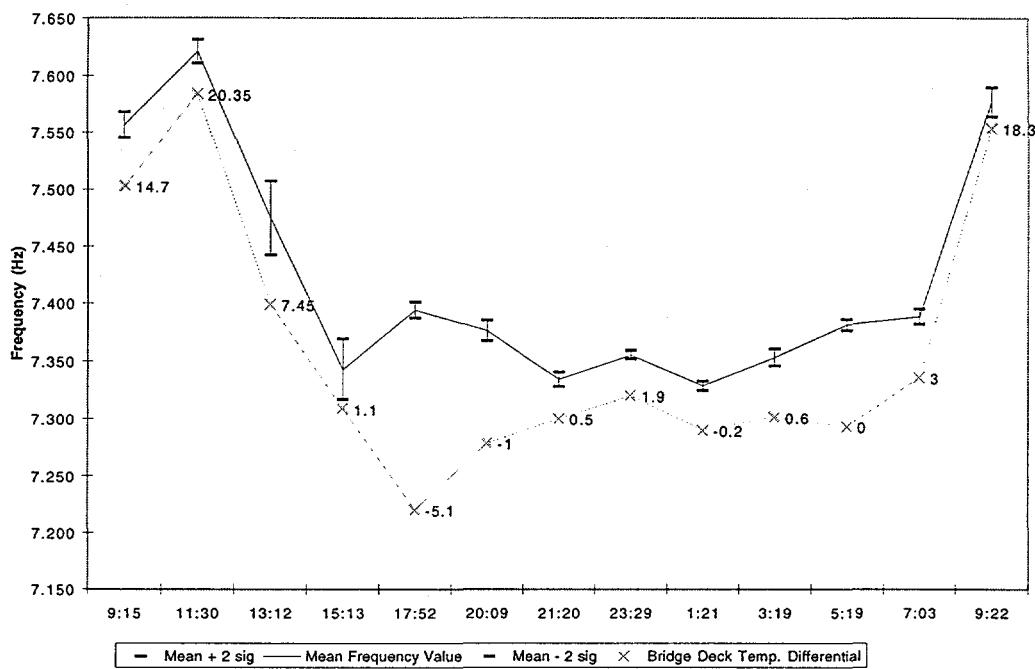
Fig. 11. Change in the first mode frequency during a 24 hr time period.
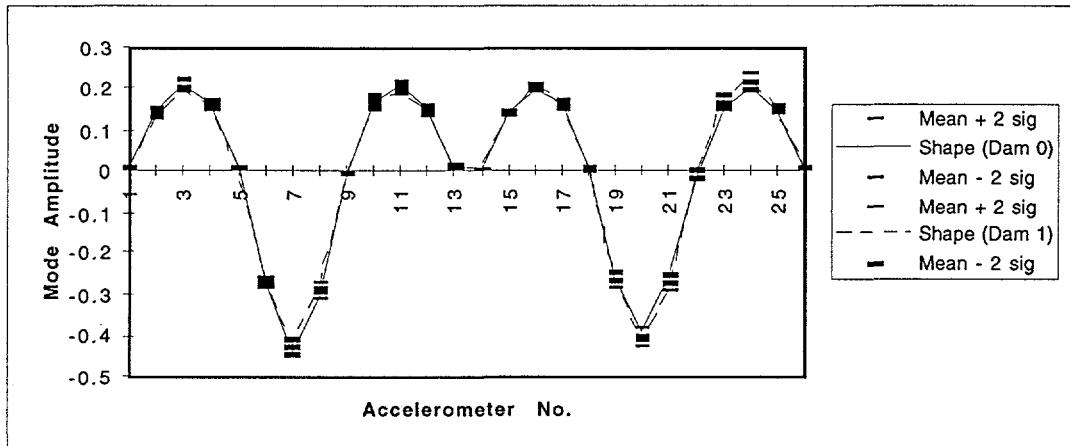
Fig. 12. First mode shape amplitudes and their corresponding 95% confidence limits for the undamaged structure compared to similar quantities measured after the first damage case.

THINGS THAT WERE BEYOND OUR CONTROL

For the I-40 bridge test several conditions occurred that were beyond our control and that could have significantly influenced the experimental modal analyses

have unavoidable conditions arise that are beyond the control of the experimentalist and that can potentially influence the outcome the study. The only thing that can be done is to note the condition and perform additional tests in an attempt to quantify the influence of the changing condition. Examples of some of these unplanned changing conditions on the I-40 bridge are listed below.

1. The load cell located between the Sandia actuator and reaction mass showed that the vibration from traffic on the adjacent bridges, transferred through the ground to the piers and abutment of the bridge being tested, caused the bridge deck to put a peak force of 150 lb. into the reaction mass.

Coherence functions can be used to determine if sources of excitation other than the Sandia shaker are significantly contributing to the measured response. For an ideal linear system the coherence function will yield a value of one. If the response is completely unrelated to the input, this function will yield a value of zero. Values between zero and one result when there is extraneous noise in the measurements, the structure is responding in a nonlinear manner, or sources of input other than the one being monitored are causing the response. For lightly damped structures, low coherence can also occur around resonances when the system response is calculated from a series of time windows as was done in these tests. The response in a particular window is strongly dependent on energy input
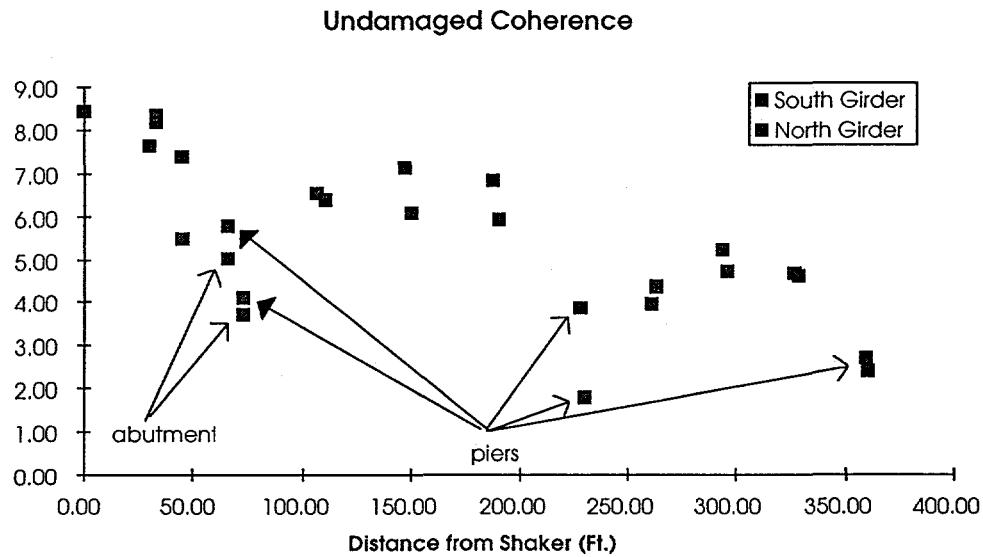

Fig. 11. Area under the coherence function over the frequency range of 2 - 11 Hz plotted as a function of the sensors distance from the shaker.

during the previous window, particularly at resonance, and this response will be uncorrelated with input measured during the current window. A plot of the area under the coherence function for the various measurement locations as a function of their distance from the shaker is shown in Fig. 11. The reduction in coherence that can be observed in this plot is caused by the inputs that result from extraneous sources of noise (traffic on the adjacent spans) causing a greater portion of the measured response at locations further from the excitation source. With the exception of measurement points directly above the support locations where the signal-to-noise ratio is inherently low, there is a distinct trend of poorer coherence as a function of distance from the shaker. The effects of the extraneous inputs are minimized by the averaging process used to calculate the FRFs.

2. Demolition of the concrete deck at the west end of the bridge was started before the forced vibration tests and continued while they were underway proceeding to the third span in from the west end. Portions of the foundation around the north side of the east abutment were removed to build an access ramp for construction work. Both the demolition and the construction of the access ramp can be viewed as changing the boundary conditions of the test structure. Forced vibration measurements taken before and after the access ramp was constructed showed no changes in the resonant frequencies of the structure. Because forced vibration measurements were not made before the demolition of the west end began, the extent of this change on the measured modal properties could not be easily quantified.

WHAT CAN BE DONE PRIOR TO DAMAGE USING INITIAL MEASUREMENTS AND FEM?

Once a finite element analysis has been benchmarked or correlated against the measure modal properties simulated damage scenarios can be introduced into the model and either an eignevalue analysis can be performed or, to better simulate an actual modal test, a time-history analysis can be performed. Mode shape data can then be obtained from either type of analysis and the various damage ID methods can be applied to the observed changes in the modal properties. If a statistical analysis has been applied to the measured modal parameters of the baseline or undamaged structure, then it can be established that the changes in the monitored modal properties such as mode shape curvature resulting from the simulated damage are greater than the variations that can be attributed to experimental repeatability. In addition, the statistical variations calculated for the measured modal properties on the undamaged structure can be assumed to apply to the numerical results from the damaged structure. The use of statistical variations measured on the undamaged structure and assumed for the numerical simulation of the damaged structure can then be used to establish the threshold damaged level that can be reliably detected.

IN RETROSPECT, WHAT WOULD WE DO DIFFERENT

Given infinite resources we would have liked to perform the damage ID process using finite element model updating techniques to obtain a direct comparison of these methods relative to methods that only examine measured modal properties. (Simmermacher et al., 1995) have recently performed a damage assessment of the I-40 bridge using model updating techniques. Ideally, we would have liked to investigate the ability of the various damage ID methods to detect the damage using ambient traffic excitation, but this type of test poses many safety concerns. Also, we would have liked to introduce multiple damage scenarios into the structure.

SUMMARY AND CONCLUSIONS

The application of five linear damage identification methods using experimental modal data gathered from the I-40 Bridge over the Rio Grande in Albuquerque, NM has been summarized. In this study linear damage identification implies that linear dynamic models were used to model the structure both before and after damage. The nature of the damage applied to the I-40 bridge is such that the linear damage models are applicable to these damage scenarios. This study provides a direct comparison of various damage identification algorithms when applied to a standard problem. The authors are not aware of other such comparisons that have been reported in the technical literature.

Examination of results from the experimental modal analyses verify other investigators findings that standard modal properties such as resonant frequencies and mode shapes are poor indicators of damage. The more sophisticated damage detection methods investigated herein showed improved abilities to detect and locate the damage. In general, all methods investigated in this study identified the damage location correctly for the most severe damage case; a cut through more than half the web and completely through the bottom flange. However, for several of these methods, if they had been applied blindly, it would be difficult to tell if damage had not also occurred at locations other than the actual one. The methods were inconsistent and did not clearly identify the damage location when they were applied to the less severe damage cases. Results of this study show that the Damage Index Method performed the best when the entire set of tests are considered. This performance is attributed to the methods of normalizing changes in the damage parameters relative to the undamaged case. Also, the Damage Index Method works with mode shape data that do not have to be unit-mass normalized. This feature is desirable when an on-line monitoring system that uses ambient traffic excitation is being considered.

Another observation from this study, which the authors feel is important, is that the Damage Index Method is the only method tested that has a specific criterion for determining if damage has occurred at a particular location. The other methods only look for the largest change in a particular parameter and it is ambiguous at

times to determine if these changes indicate damage at more than one location.

Based on further analysis and observations related to the I-40 bridge tests, subsequent tests on another bridge, interactions with other researchers performing similar tests, and review of the technical literature related to bridge testing, there are several things that should have been done during these tests to improve the confidence in the damage ID results. These improvements include detailed visual inspection of the bridge, performing linearity checks, performing reciprocity checks, performing false-positive studies, performing test condition sensitivity studies, and performing statistical analyses of the measured modal properties.

The lack of application of vibration-based damage detection methods to large civil engineering structures is due, in part, to the limited application and refinement of this technology to *in situ* structures. The authors hope that future researchers in this field will benefit from the lessons learned through trial and error as discussed in this paper.

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