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Title: VIBRATION-BASED HEALTH MONITORING AND MODEL REFINEMENT OF CIVIL ENGINEERING STRUCTURES

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Vibration-Based Health Monitoring and Model Refinement of Civil Engineering Structures

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ABSTRACT

Damage or fault detection, as determined by changes in the dynamic properties of structures, is a subject that has received considerable attention in the technical literature beginning approximately 30 years ago. The basic idea is that changes in the structure's properties, primarily stiffness, will alter the dynamic properties of the structure such as resonant frequencies and mode shapes, and properties derived from these quantities such as modal-based flexibility. Recently, this technology has been investigated for applications to health monitoring of large civil engineering structures. This presentation will discuss such a study undertaken by engineers from New Mexico State University, Sandia National Laboratory and Los Alamos National Laboratory. Experimental modal analyses were performed on an undamaged interstate highway bridge and immediately after four successively more severe damage cases were inflicted in the main girder of the structure. Results of these tests provide insight into the abilities of modal-based damage id methods to identify damage and the current limitations of this technology. Closely related topics that will be discussed are the use of modal properties to validate computer models of the structure, the use of these computer models in the damage detection process, and the general lack of experimental investigation of large civil engineering structures.

Keywords: Damage detection, vibration, bridge, model refinement, testing, dynamic, health monitoring.

1. 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, magnet field methods, radiographs, eddy-current methods and thermal field methods[1]. All of these experimental techniques require that the vicinity of the 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 additional 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.

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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[2]. The basic idea is that modal parameters (notably frequencies, mode shapes, and modal damping) are functions of the physical properties of the structure (mass, damping, and stiffness). Therefore, changes in the physical properties, such as a reduction in local stiffness caused by a crack or loose connection, will cause changes in the modal properties.

Ideally, an end goal for a robust vibration-based damage detection scheme will be that, using current measures of vibration response, the method will be able to identify that damage has occurred at a very early stage, locate the damage within the sensor resolution being used, provide some estimate of the severity of the damage, and predict the remaining useful life of the structure[3]. In light of the recent advance in remotely monitored vibration sensing systems, the method should also be well-suited to automation. To the greatest extent possible, the method should not rely on the engineering judgment or the use of an analytical model of the structure. It is the authors' opinion that this level of sophistication in vibration-based damage detection does not currently exist.

A less ambitious, but more attainable, goal would be to develop a method that has the features listed above, but that uses an initial measurement of an undamaged structure as the baseline for future comparisons of measured response. Also, the methods should be able to take into account operational constraints. For example, a common assumption with most damage-identification methods reported in the technical literature to date is that the mass of the structure does not change appreciably as a result of the damage. However, there are certain types of structures such as offshore oil platforms where this assumption is not valid. Another important feature of damage-identification methods, and specifically those methods which use prior models, is their ability to discriminate between the model/data discrepancies caused by modeling errors and the discrepancies that are a result of damage.

The challenge to vibration-based damage detection stems from the fact that damage typically is a local phenomena while lower-frequency modal parameters are global properties. Higher frequency modal parameters capture local effects, but for large structures it is difficult to excite the higher frequency response. Also, the increased density of modes in the higher frequency range makes it difficult to accurately identify the associated modal properties. The current state-of-the-art for vibration-based damage detection methods that can be applied to a general class of structures is, given severe enough damage, the methods can identify that damage has occurred and locate the damage within the resolution of the sensors. If an accurate structural model is assumed, these methods can, in some cases, estimate the extent of the damage.

The effects of damage on a structure can be classified as linear or nonlinear. A linear damage situation is defined as the case when the initially linear-elastic structure remains linear-elastic after damage. The changes in modal properties are a result of changes in the geometry and/or the material properties of the structure, but the structural response can still be modeled using linear equations of motion. The complete severing of an element in a truss structure such that the severed element does not rattle is an example of a linear damage scenario. Nonlinear damage is defined as the case when the initially linear-elastic structure behaves in a nonlinear manner after the damage has been introduced. One example of nonlinear damage is the formation of a fatigue crack that subsequently opens and closes under the normal operating vibration environment. Other examples include loose connections that rattle and nonlinear material behavior. A robust damage-detection method will be applicable to both of these general types of damage. To date, the majority of the papers published in the literature only address linear damage detection.

Vibration-based damage detection methods can be classified as model-based and non-model-based methods. Model-based methods require some assumed model of the structure, typically a finite element model, that is correlated with experimental data (experimentally determined resonant frequency and mode shape data) before and after a damaging event. Changes in model-based parameters such as stiffness matrix indices provide a measure of the location and extent of damage in the structure. Non-model-based damage detection methods usually compare changes in experimentally measured modal properties such as resonant frequencies, mode shapes, or modal-based flexibility matrices to determine the location of damage in a structure. A subset of these methods can also analyze the measured response (typically acceleration) time histories directly for indications of damage. Non-model-based methods have a greater difficulty in determining the extent of damage, but they are more amenable to automation as they are less dependent on some assumed model of the structure.

To date, field verification of damage detection algorithms applied to large civil engineering structures are scarce as few full size structures are made available for the necessary destructive testing. This paper will discuss the results of one of the few systematic studies of vibration-based damage identification technologies that has been performed on a full scale *in situ* structure . Because the Interstate 40 (I-40) bridges over the Rio Grande in Albuquerque, New

Mexico were to be razed, investigators were able to introduce simulated cracks into the structure, perform vibration tests on the structure in it undamaged and damaged condition, and then study various vibration-based damage identification methods. Staff from Los Alamos and Sandia National Laboratories (LANL and SNL) performed experimental modal analyses on the bridge in its undamaged and damaged conditions[4,5] as part of a large project directed by New Mexico State University. Data from these tests were analyzed with various damage identification algorithms by the LANL and SNL engineers[6,7]. Subsequently, these data were transferred to several universities (Univ. of Colorado, Texas A&M Univ., Stanford Univ., Univ. of Houston, Univ. of New Mexico) where other researchers applied additional damage detection methods to these data. Results from the LANL investigation are reported along with general discussions regarding testing of large civil engineering structures and correlation of computer models of these structures with measured vibration response.

2. I-40 BRIDGE GEOMETRY AND DAMAGE SCENARIOS

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 are transferred to the plate girders by floor beams located at 6.1 m (20 ft) intervals. Cross-bracing is 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 is made up of three identical sections. Except for the common pier located at the end of each section, the sections are structurally independent. A section has three spans; the end spans are of equal length, approximately 39.9 m (131 ft), and the center span is approximately 49.4 m (163 ft) long. Five plate girders are connected with four bolted splices to form a continuous beam over the three spans. The portions of the plate girders over the piers have increased flange dimensions, compared with the mid-span portions, to resist the higher bending stresses at these locations. Connections that allow for thermal expansion as well as connections that prevent longitudinal translation are located at the base of each plate girder, where the girder is supported by a concrete pier or abutment. These connections are labeled "exp" and "pinned" in Fig. 1. 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.

The damage that was introduced was intended to simulate fatigue cracking. This cracking has been attributed to out-of-plane bending of the plate girder web at locations where cross beams are supported by seats welded to the web. Four levels of damage were introduced to the middle span of the north plate girder close to the seat supporting the floor beam at midspan. The first level of damage, designated E-1 in Fig. 3, consisted of a two-foot-long (61.0 cm) cut through the web approximately 3/8-in-wide (0.95-cm-wide) centered at mid-height of the web. Next, this cut was continued to the bottom of the web, E-2 . The flange was then cut halfway in from either side directly below cut in the web, E-3. Finally, the flange was cut completely through leaving the top 4 ft (122 cm) of the web and the top flange to carry the load at this location, E-4.

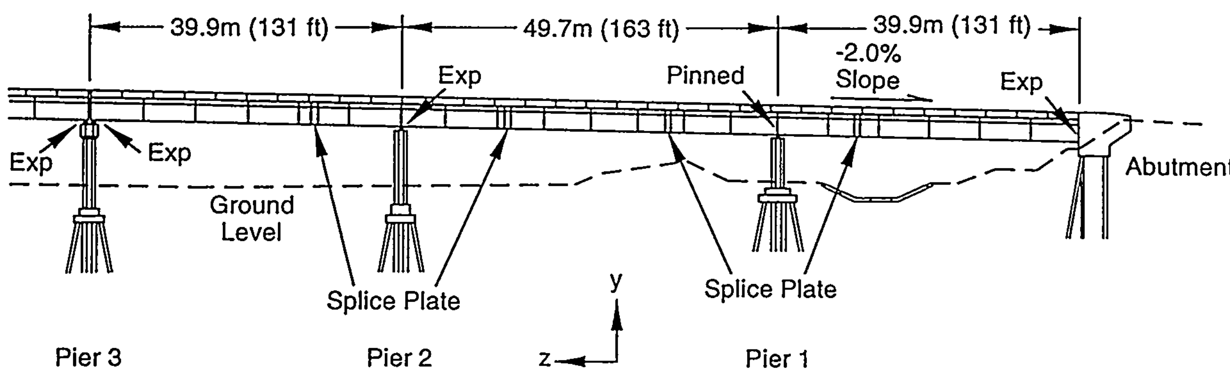


Figure 1. Elevation view of the portion of the eastbound bridge that was tested.

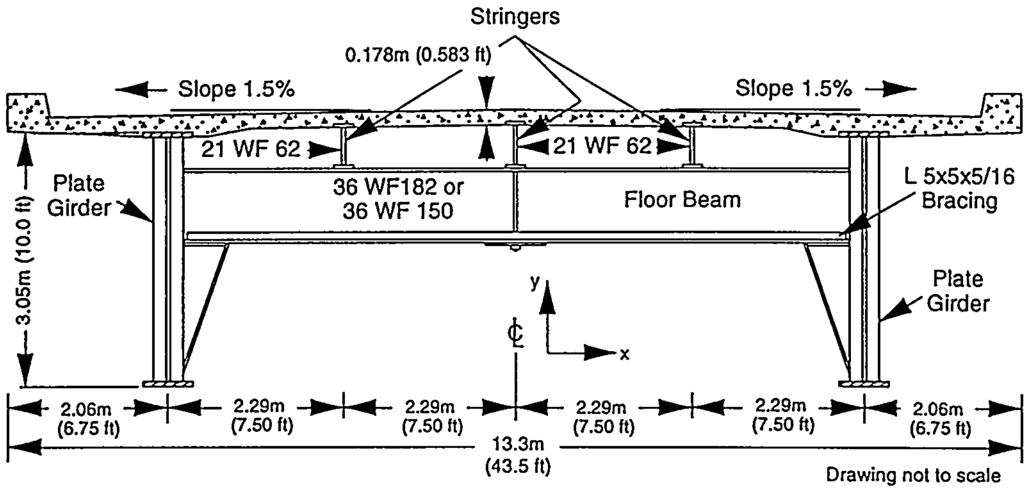


Figure 2. Typical cross-section geometry of the bridge.

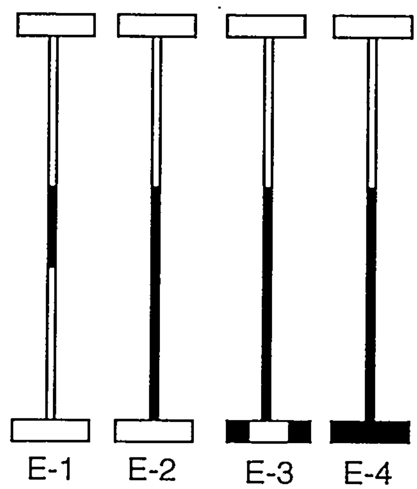


Fig. 3. Portions of the north plate girder that were cut during the various damage scenarios.

3. 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. 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 Ref. 4.

3.1 Excitation

Engineers from SNL provided a hydraulic shaker that generated the measured force input[5]. 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. The shaker was located on the eastern-most span directly above the south plate girder and midway between the abutment and first pier. Figure 4 shows the shaker location relative to the damage location. 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.

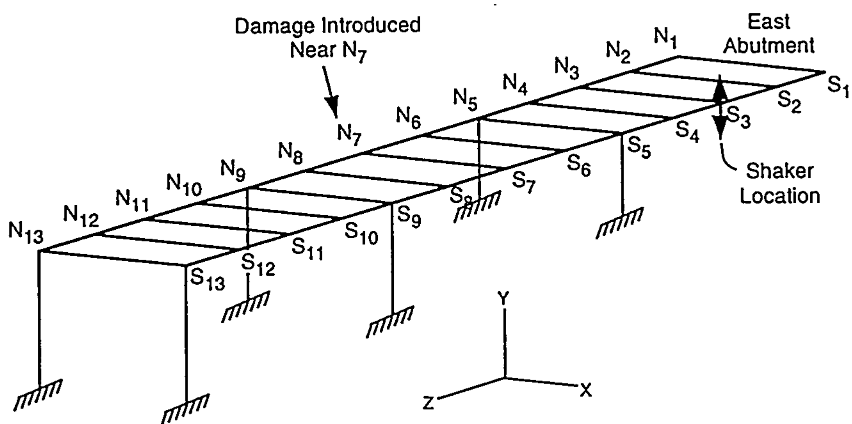


Fig. 4. Location of shaker and accelerometers.

3.2 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 with twenty-six accelerometers 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. Cables ranging from 21.3 m to 88.9 m (70 ft to 291 ft) connected the accelerometers to the input modules. A more refined set of measurements (SET2) was made near the damage location. Eleven accelerometers were placed 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.

Sampling parameters were specified that calculated the frequency response functions (FRFs, the frequency domain measure of the response normalized by the input) and cross-power spectra (CPS) from 30 averages of 32-s time windows discretized with 1024 samples yielding a frequency resolution of 0.03125 Hz over a frequency range of 0 - 12.5 Hz. Hanning windows were applied to the time signals to minimize leakage, and AC coupling was specified to minimize DC offsets.

3.3 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 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[8]. A rational-fraction polynomial, global, curve-fitting algorithm in a commercial modal analysis software package was used to fit the analytical models to the measured FRF data and to extract resonant frequencies, mode shapes, and modal damping values. Figure 6 shows the first modes of the bridge, both in its undamaged state and after the final level of damage had been introduced, as identified by these procedures. By 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 CPS of the various accelerometer readings relative to the accelerometer N-3 shown in Fig. 5. Determining operating shapes in this manner simulates

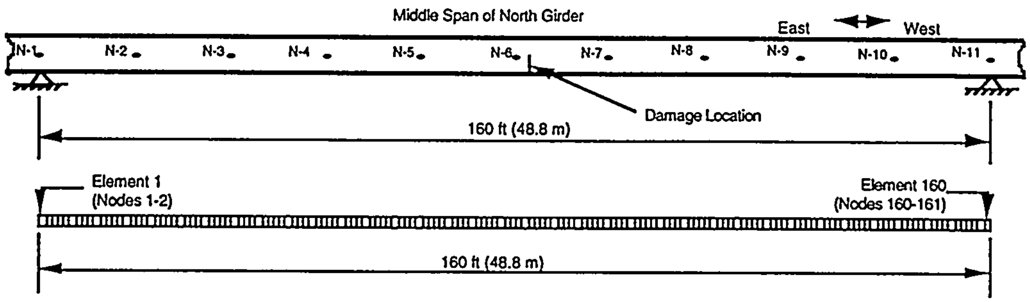


Fig. 5. Refined accelerometer locations.

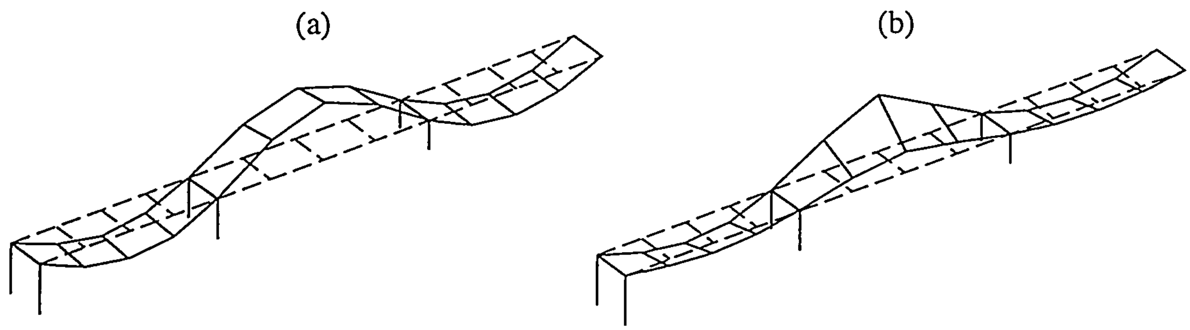


Fig. 6. First mode of the bridge (a) before damage, (b) after the final level of damage.

the methods that would have to be employed when the responses to ambient excitations are measured[9]. 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.

4. INFLUENCE OF DAMAGE ON CONVENTIONAL MODAL PROPERTIES

4.1 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 change 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.

4.2 Changes in Mode Shapes

A modal assurance criterion (MAC), sometimes referred to as a modal correlation coefficient, was calculated to quantify the correlation between mode shapes measured during different tests. The MAC makes use of the orthogonality properties of the mode shapes to compare two modes. 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. 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.

TABLE I Resonant Frequencies and Modal Damping Values Identified from Forced Vibration Tests							
		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	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	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	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	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

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 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 bridge modal properties and statistical analysis of these properties is discussed in more detail in other studies[10].

5. DAMAGE IDENTIFICATION METHODS

Five linear damage identification methods were applied to the I-40 bridge data[6]. Length limitations preclude a thorough development and discussion of all these methods. Instead, this paper will briefly summarize the results from one of these methods.

5.1 Damage Index Method

The Damage Index Method[11] can be used to locate damage in structures given their characteristic mode shapes before and after damage. This method locates damage in a structure by examining the change in strain energy stored in the structure when it deforms in a particular mode shape. To apply this method, a model for the structure must be assumed. In the case of the I-40 bridge a beam model has been used. For a structure that can be represented as a beam, a damage index, β, is developed based on the change in strain energy stored in the structure when it deforms in its particular mode shape. For location j on the beam this change in the ith mode strain energy is related to the changes in curvature of the mode at location j. The damage index for this location and this mode, β_{ij}, is defined as

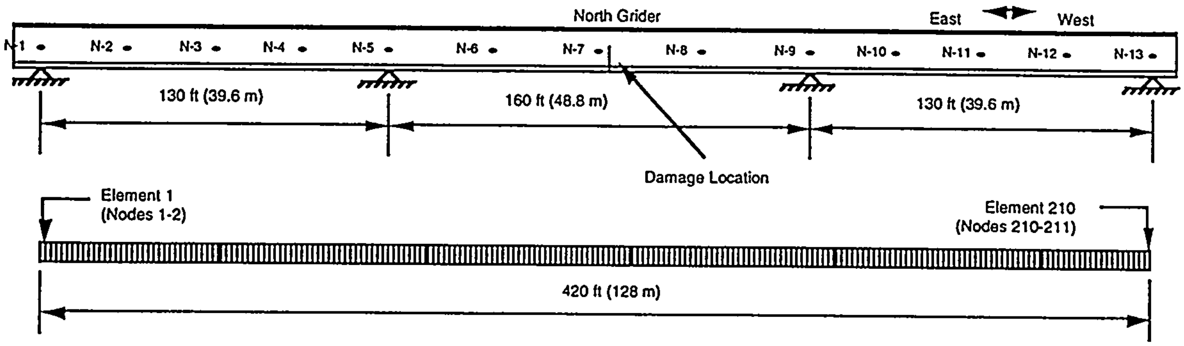
$$\beta_{ij} = \frac{\left(\int_a^b [\psi_i^{*''}(x)]^2 dx + \int_0^L [\psi_i^{*''}(x)]^2 dx\right)}{\left(\int_a^b [\psi_i''(x)]^2 dx + \int_0^L [\psi_i''(x)]^2 dx\right)} \cdot \frac{\int_0^L [\psi_i''(x)]^2 dx}{\int_0^L [\psi_i^{*''}(x)]^2 dx}, \tag{1}$$

where ψ_i'' and $\psi_i^{*''}$ are the second derivatives of the ith mode shape corresponding to the undamaged and damaged structures, respectively. L is the length of the beam. a and b are the limits of a segment of the beam where damage is being evaluated. When more than one mode is used, these authors define the damage index as the sum of damage indices from each mode. For mode shapes obtained from ambient data, the modes are normalized such that

$$\{\psi_n\}^T [M] \{\psi_n\} = 1, \tag{2}$$

where [M] is assumed to be the identity matrix .

To determine mode shape amplitudes at locations between sensors, the mode shapes are fit with a cubic polynomial. As shown in Fig. 5 for the refined set of accelerometers (SET2), the middle span of the north girder is divided into 160 0.305-m (1-ft) segments. Modal amplitudes are interpolated for each of the 161 nodes forming these segments. Similarly, for the coarse set of accelerometers shown in Fig. 7 the entire length of the north girder (all three spans) is divided into 210 0.610 m (2-ft) segments with mode shape interpolation yielding amplitudes at 211 node locations as shown in Fig. 9. Statistical methods are then used to examine changes in the damage index and associate these changes with possible damage locations. A normal distribution is fit to the damage indices, and values falling two or more standard deviations from the mean are assumed to be the most likely location of damage.



Coarse accelerometer locations

Fig. 7. Discretization of the north plate girder used to interpolate mode shape amplitudes.

6. APPLICATION OF DAMAGE IDENTIFICATION METHODS TO EXPERIMENTAL DATA

Figures 8 through 9 show the results from the damage index methods when applied to data from damage cases E-1 and E-4 obtained with the SET2 and SET1 instruments. For the SET2 instruments the actual damage location was between nodes 82 and 83. For the SET1 instruments the actual damage location was between nodes 106 and 107. Also shown for the data from the SET1 instruments is the influence of the number of modes on the damage detection process. These plots show the improved ability to locate damage when more sophisticated damage detection methods are applied to the modal data, which in its unprocessed form did not give indications of damage until the final level was inflicted.

7. MODEL REFINEMENT

The I-40 bridge project provided an unique opportunity to destructively test a large civil engineering structure and study its dynamic response when subjected to various damage conditions. For most *in situ* civil engineering structures, engineers are not given the luxury of performing such destructive tests. However, non-damaging modal tests can be performed on the *in situ* structures to determine how well the analytical models that were used in the design process represent the actual structure. Discrepancies between the analytical models and the as-built structure typically arise from simplifications employed in the modeling process to represent boundary and support conditions, connectivity between various structural elements, unknown material properties (particularly those associated with soil and concrete), and energy dissipation (damping) mechanisms. Currently, it is a rare exception when tests are performed on civil engineering structures for the purpose of correlating their as-built condition with the assumed analytical model used in the design process. In contrast, the aerospace and automotive industries spend considerable time performing such correlation studies, model refinement and re-analysis. The model refinement technology has shared the same rapid growth that the vibration-based damage detection field has over the last twenty years[12]. To illustrate the civil engineering community's lack of use of this technology, the I-40 bridge replacement project will be compared and contrasted to the FORTE satellite project at Los Alamos National Laboratory.

The FORTE satellite is a space vehicle that will carry instruments designed for weapons nonproliferation studies. It is a one-of-a-kind satellite that cost approximately 20 million dollars to design, test, build and launch. Most of this cost is associated with the development of the instruments that will be carried by the satellite. The I-40 bridge replacement project widened the bridges over the Rio Grande from three lanes each direction to five lanes each

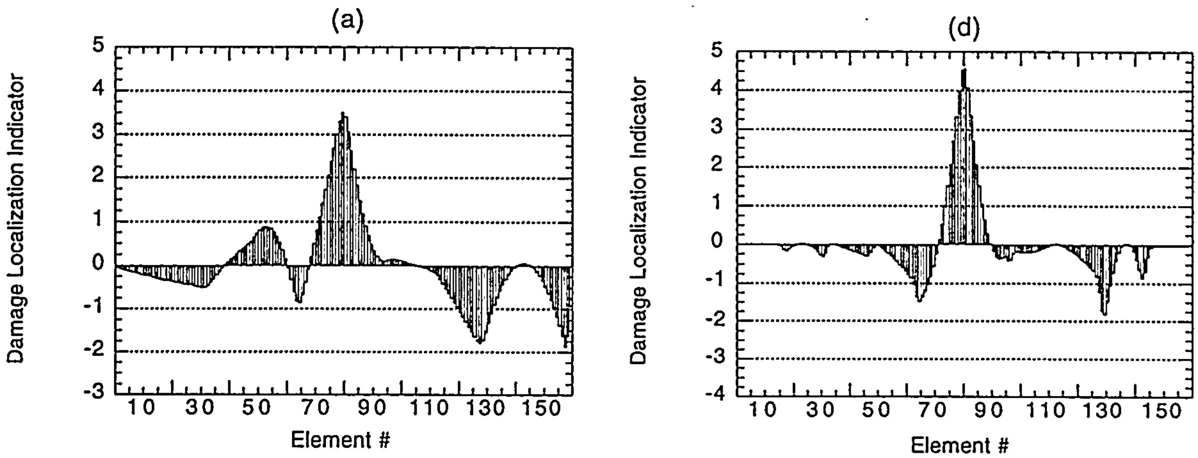


Fig. 8. Results of damage index method applied to the SET2 modal data from damage scenario E-1 (a) and E-4 (d).

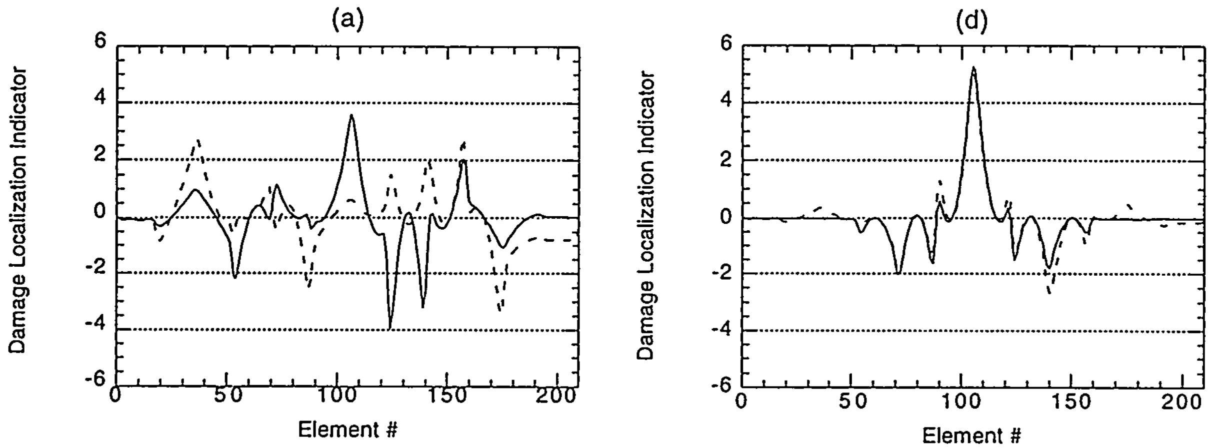


Fig. 9. Results of damage index method applied to the SET1 modal data from damage scenario E-1 (a) and E-4 (d): ———, two modes; - - - - - , six modes.

direction. The cost associated with demolition of the existing bridges, design and construction of the new bridges, was approximately 18 million dollars.

Similarities between the two projects are:

1. The cost associated with each project is similar.

2. Both structures are one-of-a-kind, although it could be argued that there are other structures very similar in design to the I-40 replacement bridges.

3. The finite element method is used for structural analysis of both structures. Modeling uncertainties associated with both structures include boundary conditions, connectivity of structural elements and damping that the structure will exhibit (if dynamic analyses are, in fact, used). Factors of safety on ultimate load are similar.

4. Both structures are designed to accommodate fluctuations in thermal environments, although the satellite must function under much more extreme variations in thermal environments.

5. Contractors bid for the construction of each project, low bidder usually wins the contract, and the owner has ultimate responsibility to monitor the quality of the contractor's work.

Now, some of the differences between the FORTE structure and the I-40 bridge project, both general and specific will be listed

1. The satellite has one-time, short-duration mechanical loading that is of primary concern along with extreme short term thermal cycles between -20 degrees C and 200 degrees C. Depending on the orientation of the orbit, thermal cycles can occur over 1-2 hr time periods or over several day time periods. The mechanical loading occurs during launch. The bridge has continuously varying traffic loads, could potentially experience short-term extreme loads such as those arising from earthquakes. Changing environmental conditions (extreme seasonal temperature swings typically are in the range of -20 degrees C to 45 degrees C) can produce thermal loads on the bridge.

2. After launch, the satellite structure is no longer monitored. The I-40 bridges are require to be inspected at 2 year intervals.

3. There are well established design codes for bridges, but there are no design codes for satellites.

3. The bridge must perform a life-safety function while the FORTE satellite has no life-safety function.

4. Although difficult to quantify, the failure of the bridge will have a tremendous economic impact on the local community. Failure of the satellite will have relatively little community economic impact.

5. Because of launch vehicle failure and failure of the satellite to separate from the launch vehicle, there is a 25% chance that the satellite will never be put into service. The bridge has a 100% chance of being put into service.

6. Bridges are perceived by the general public as "low-tech" where as satellites are perceived as "high-tech" and the FORTE satellite is considered high-tech relative to most other satellites because of its all-composite construction.

7. Quality assurance procedures for the FORTE satellite are not as formalized, at least at Los Alamos, as a state highway department would typically have for bridge construction. Therefore, there is more reliance on testing for the satellite as discussed under item 9 below.

8. The I-40 bridge was analyzed for static loads only. Dynamic loads such as those arising from earthquake or wind are analyzed with equivalent static loads. The FORTE satellite has static analysis performed on it as well as dynamic analysis. Dynamic analyses examine transient loading that results from the drop of the Pegasus launch vehicle off of the airplane, rocket ignition and stage separation shocks, random loads caused by air turbulence, and acoustic loads caused by the rocket engine.

9. Other than materials tests associated with soil and concrete, no testing of the I-40 bridge is done, either before or during its service life. The FORTE satellite has static and dynamic tests performed on it. The static tests are performed to verify the workmanship of the FORTE contractor. Dynamic tests include modal testing (low level random excitation geared at identifying the resonant frequencies, mode shapes, and modal damping of the satellite system, transient dynamic tests, random dynamic tests and acoustic tests. Because of its relatively small size, the FORTE satellite can be tested in a laboratory while the I-40 bridge would have to be tested *in situ*. The primary difference in cost associated with testing the two structures will be associated with the excitation system needed for the bridge if traffic is not used as an ambient excitation source.

In addition, a full-scale engineering model of the FORTE satellite was built and tested prior to the tests of the actual satellite. It is a rare case where even a scale model of a civil engineering structure is built and tested as part of the engineering procedure.

10. Obviously, because no tests were performed on the I-40 bridge, there can be no refinement of analytical models based on the measured response. The results of all testing performed on the FORTE satellite were used to improve the analytical models of the satellite and the structure was subsequently re-analyzed with these refined models.

This comparison points out an area of technology where the civil engineering community has lagged behind the aerospace and mechanical engineering communities. In general, regardless of the fact that there are know uncertainties in the assumed analytical models of the structural system, civil engineers do not test their complete structural systems and they make no effort to verify that the as-built structures respond in a manner that was assumed in the analysis of the structure. Two questions then arise:

1.) Why have civil engineers not incorporated system testing into their structural engineering profession, and
2.) Has this lack of model verification proved detrimental to public?

There is no single compelling reason why civil engineers have not incorporated system vibration testing into their structural engineering profession. If a survey of structural engineers was taken, it is the authors' guess that the reason identified for the lack of vibration testing and model verification would be based on some form of a benefit-cost argument associated with doing the tests. In addition, this survey would, most likely, reveal that the majority of structural engineers have never performed a test, do not know what the current state of the art is in dynamic testing with regard to instrumentation, data acquisition systems, excitation methods, and system identification procedures (methods to identify dynamic properties such as resonant frequencies and mode shapes given the measured acceleration-time histories form a dynamic test), or know how the results from a test could be used to systematically refine their analytical model. However, it is the authors' opinion that the problem is more deep-rooted than that. The lack of interest in this technology starts in our university education system where civil engineers specializing in structural engineering will, in general, not get a course in structural dynamics at the undergraduate level. If they do get a structural dynamics course, it most likely will have no experimental component to it. Even at the graduate level, a civil engineering student will not learn experimental techniques related to vibration testing unless their advisor's research project specifically involves dynamic testing. When testing is done, the civil engineering community tends to take an element approach as opposed to a system approach because it is difficult to test large structural systems in a laboratory. This element approach is reflected in major national structural design codes such as the AISC steel design code[13] and the ACI concrete design code[14].

Because extreme dynamic loads on civil engineering structures do not occur on a regular basis, many structures go through their entire useful life without ever being subjected to their design loads. However, when structures do experience extreme load, such as those during earthquakes, inevitably there are failures. In many cases, nondestructive vibration testing would not have provided any insight into the potential for failure. However, there are instances such as failures from short-column effects (failures where architectural walls, not considered in the structural analysis, cause columns to fail in shear rather than the assumed bending failure mode) where vibration testing could have potentially revealed the discrepancies between the assumed model and the actual *in situ* response. In another instance, a bridge that the authors recently tested in southern New Mexico responded with complex modes (modes where the phase relationship between different degrees of freedom vary) even though it would be considered lightly damped. To the author' knowledge, no structural engineer would consider complex modes in the dynamic analysis of a structure. Clearly, there is a discrepancy between the assumed model for this structure and its actual behavior. As long as extreme loads are not experienced, this discrepancy would not cause concern. If this bridge were located in a high seismic zone and experienced a design-level earthquake, the response could potentially be significantly different from that predicted by the assumed analytical model. Studies are currently being done to see if using real modes, only, would produce a more conservative analysis than one that considers the complex modes.

The observations discussed above should not be construed to indicate that the authors are proponents of dynamic testing for all civil engineering structures. Most structures have a considerable amount of conservatism incorporated in their design and there is an extensive experience base associated with many classes of structures. However, it does appear that the civil engineering community has possibly erred on the side of too little testing of complete structural systems, particularly when new systems are put into place. As an example, one of the authors recently talked with representatives of companies marketing passive damping devices for structures and base isolation systems, both of which are relatively new technologies for large structures. When asked if they had performed tests on the completed structures where these devices had been installed, the response was that they had not. It is the authors' opinion that for new and unique types of construction, and for certain structures performing important lifeline functions, such as hospitals, that the process of testing the as-built structural system, analytical model refinement based on testing results, re-analysis, and, if necessary, retrofit should become part of standard civil engineering practice.

SUMMARY AND CONCLUSIONS

The application of a linear damage identification method using experimental modal data gathered from the I-40 Bridge over the Rio Grande in Albuquerque, NM has been reported. 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. 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. Results of this study show that the Damage Index Method performed well 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 found in the literature that has a specific criterion for determining if damage has occurred at a particular location. The probabilistic method employed by the Damage Index Method to determine if changes in the monitored parameter were indicative of damage could be implemented with the other methods as well.

Two areas of research currently being pursued by the authors were suggested by the results reported in this study. First, there is a need to develop statistical analysis procedures based on Monte Carlo methods or bootstrap methods that can establish confidence limits on identified modal parameters. Then the changes in these parameters, or quantities derived from these parameters such as the damage index or the modal-based flexibility, can be attributed to damage rather than the repeatability of the experiment. Second, it appears that only using the modes that are most influenced by damage can enhance the damage detection process. Therefore, the authors are currently studying methods to screen and select the modes to be used in the damage detection process.

The authors acknowledge that the application of these damage detection methods was simplified because of the beam-like behavior of the damaged member and because the location of the damage was known prior to the application of these methods. Clearly, more tests on actual structures such as the one reported in this study are needed before these methods can be applied with sufficient confidence to warrant the field deployment of remote monitoring systems. Finally, it should be noted that this comparative study has been extended to include finite element model updating methods of damage detection as reported by Simmermacher[15].

Finally, by means of comparison between structural analyses and testing typically performed on a highway bridge relative to that typically performed on a satellite, the authors have tried to demonstrate that the civil engineering community is lagging behind other engineering disciplines with regards to testing of structural systems and updating of analytical models based on these testing results. If we are to have architectural surety, testing and model refinement will have to become a much more standard part of structural engineering than they are currently practiced today.

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