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TIMBER BRIDGE EVALUATION: A GLOBAL NONDESTRUCTIVE APPROACH USING IMPACT GENERATED FRFs

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ABSTRACT

Bridges require periodic inspections to ensure the safety of those using the structure. A visual inspection has historically been the most common form of investigation for timber bridges. This poses many problems when inspecting bridge timbers since often the damage is internal, leaving no visible signs of decay on the surface. Localized nondestructive evaluation (NDE) techniques have been developed to allow assessment of the internal parts of timber structural members. Testing an entire structure in this manner can be tedious and time consuming. Steel structures have long been tested from a global perspective, basing the assessment of health of the structure on modal parameters. This paper presents an effort to extend global testing methods, specifically impact testing, to timber bridges.

NOMENCLATURE

E	Modulus of Elasticity
G	Shear Modulus
I	Area moment of inertia of bridge cross section
M	mass per unit length
L	span of bridge deck
A	area of bridge cross section
K	shear factor
u(x,t)	bridge deck response
I	mode number
l	factor that varies by mode number and boundary condition
PDE	partial differential equation
NDE	non-destructive evaluation
FE	finite element
FRF(s)	frequency response function(s)
f_n	natural frequency in Hertz
z	damping ratio

1. INTRODUCTION

Many methods of damage detection have been developed I for civil engineering structures. Methods can be grouped in four categories, depending on the complexity of the method[1]. The immediate goal of this study, determining if damage in the form of decay is present in timber bridge structures, falls into the most basic category. This team hopes to lead the way to future development of more complex damage detection schemes.

The test method developed in this study must meet several requirements. First, the test must be easy to perform. Field staff without much advanced training in experimental structural dynamics testing will, in the end, be performing this test. Second, the test must be cost effective. To justify the development of this method, it must provide a cost and time savings over existing methods of inspecting timber bridges. To simplify the method as much as possible, the only mode of interest is the first bending mode of the bridge deck. Specifically, we will correlate the frequency of the first bending mode to the stiffness characteristics of the bridge stringers. A continuous system model is employed to develop an analytical relationship between first bending mode's frequency and specific properties of each bridge structure. A similar approach has been developed by the Forest Products Laboratory of the U.S. Forest Service for detection of decay in wood flooring systems[2]. Developing a Finite Element (FE) model to relate the natural frequency to the bridge's properties is not practical since this method will be used on a wide variety of bridges.

Standard modal analysis techniques are used to identify the first bending mode's natural frequency of vibration. Furthermore, accurate modal data will provide the basis for expanding into more complex damage detection methods.

Reliable, detailed data about a structure's mode shapes are needed for the damage detection algorithms discussed in the literature. Furthermore, most methods require a detailed FE model of the undamaged structure.

Obtaining modal data on timber structures presents a wide array of challenges. One of the biggest challenges is that the material properties vary widely throughout the structure. Natural checks, voids, and knots can dramatically change local stiffness and density properties. Furthermore, the mechanical properties of each timber specimen are dependent on the species of wood and how the specimen was cut with respect to the grain of the timber. Finally, timber structures are generally not built with as much dimension and tolerance control as other civil engineering structures. This complicates the structure's dynamics and adds difficulty to modeling a timber structure.

2. EXPERIMENTAL METHOD

2.1 Description of the Structures

Six timber bridges of similar design have been examined in this preliminary study. Most of the bridges are Comprised of a single span with an average overall length of about 20 feet. Ten stringers with an average size of 6 inches wide by 16 inches deep provide the substructure of the bridge, The stringers are covered by 3 inch high by 8-inch wide deck boards nailed perpendicular to the stringers. Finally, to provide a durable surface for traffic, 3 inch tall by 10 inch wide planks are nailed in two strips parallel to the direction of the stringers to create the driving surface. The figure below shows typical structural details of the bridges

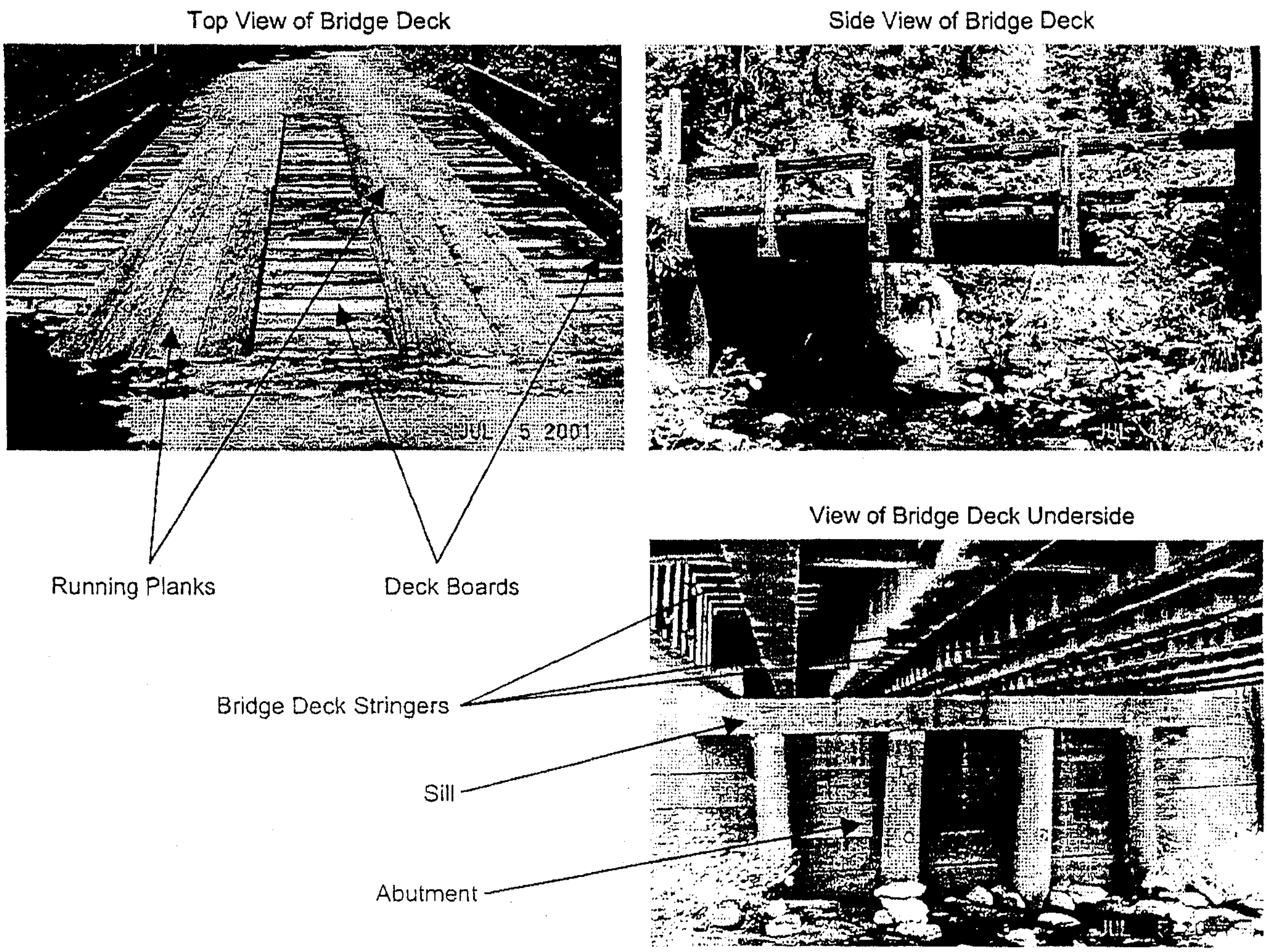
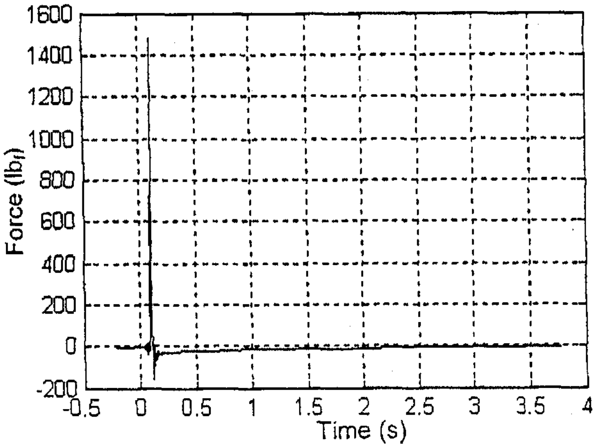


Figure 1 - Structural Details of a Typical Bridge Under Test

2.2 Excitation Method

The bridges under study are almost exclusively confined to Forest Service roads inside of U.S. National Forests. As such, traffic is very sparse on these roads, making measured input excitation the obvious choice. Since electrical power is not readily available at the bridge locations, impact excitation eliminates the need for a shaker and its associated amplifier/power supply, Two types of modal hammers were used in this study. It was found that a heavy (12 Ib) PCB modal sledgehammer and a medium (3 Ib) PCB modal sledgehammer were both able to excite the bridge structures under test. The medium hammer is very similar in design to smaller hammers made by PCB Corporation. 'The large hammer is a modified Lixie dead-blow sledgehammer. The large hammer was able to provide more energy over a more narrow frequency bandwidth, providing better excitation to the structure. Typical time and frequency domain representations of the impacts are illustrated in Figure 2. Obtaining valid results with impact excitation depends on many factors, The quality of timber bridge FRFs has been shown to be extremely sensitive to two of these factors in particular. First, the large modal sledge can be difficult to use. Ensuring that the hammer face strikes normal to the bridge surface is difficult. Second, nonlinear characteristics of the structure are seen if the strength of the blow becomes too severe. Both shifting peaks and peak amplitude changes have been documented. It is felt that local deformation of the structure as well as changes in the characteristics of joints primarily secured by friction force may be the cause of this. It is not, however, fully understood at this time. Figure 3 displays FRFs that illustrate the nonlinear behavior of the bridge structures.

Figure 2 - Typical Input Time Record and Associated Frequency Content



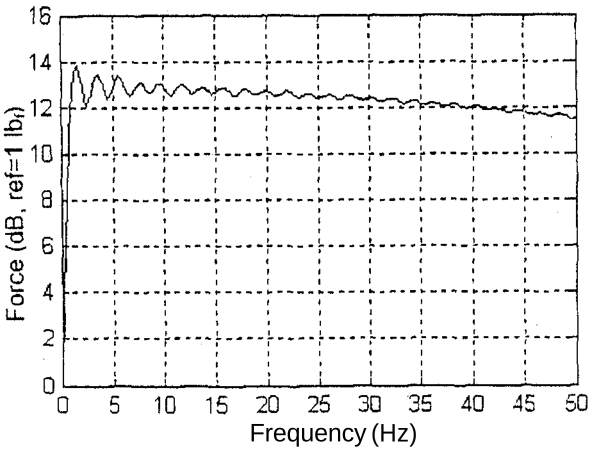
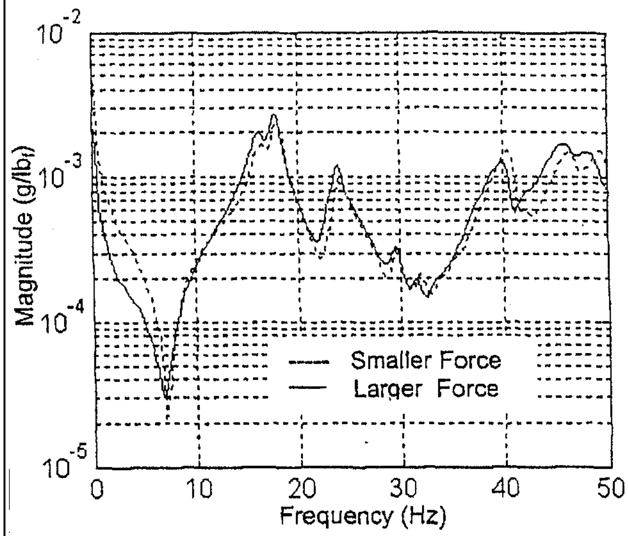


Figure 3 - Illustration of Nonlinear Behavior



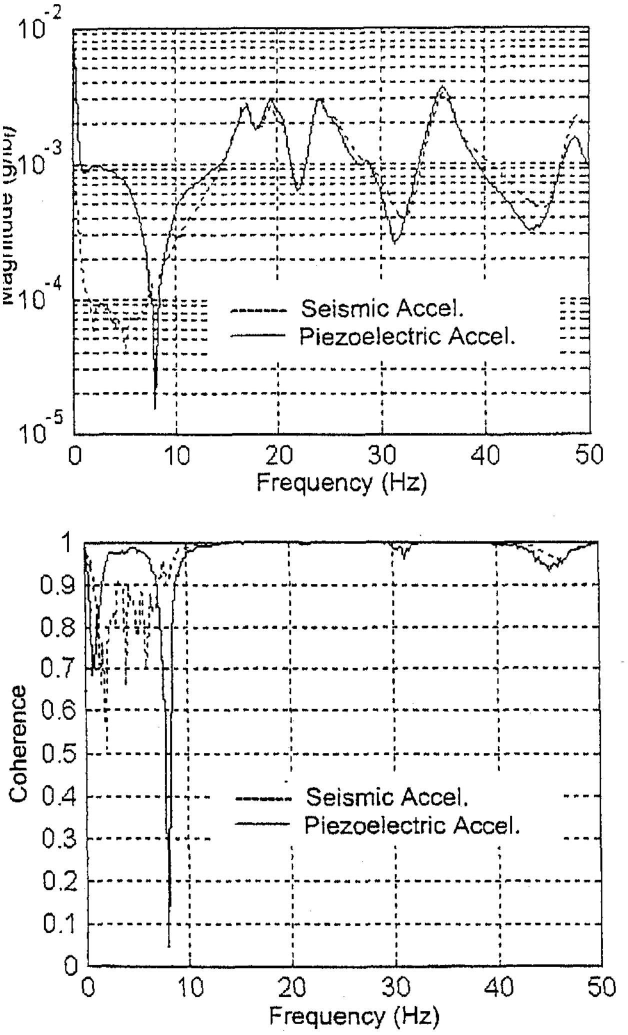
2.3 Response Measurements

The frequency range of interest for this test is very low (0-50 Hz). It is essential that the selected accelerometers be capable of acquiring coherent measurements at low frequency, Many commonly available accelerometers use a piezoelectric crystal to generate the acceleration measurements. Since this method will not produce a response at zero Hz., FRF measurements taken with piezoelectric accelerometers were compared with those taken with seismic accelerometers capable of measuring acceleration in the very low frequency and DC regime. It was found that the selected piezoelectric accelerometers were able to make sufficiently accurate measurements in the frequency range of interest. This determination was based on the coherence function measured on a variety of bridges and at a variety of locations. Figure 4 shows an example.

2.4 Response and Excitation Locations

To better understand the structural dynamics of the type of timber bridges included in our study, a 45-point modal analysis of the East Branch Ontonagon River bridge was completed. The accelerometers were laid out in three rows

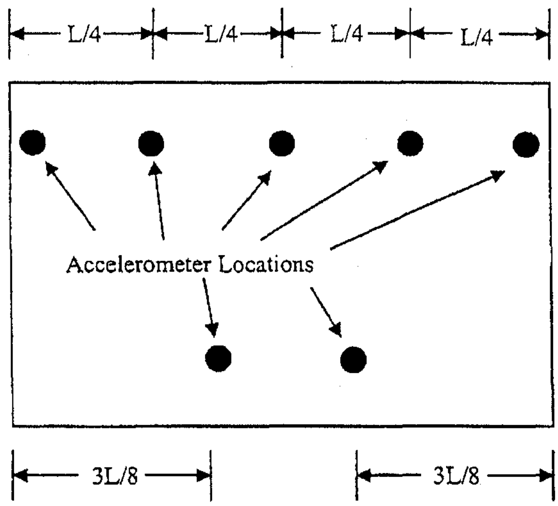
Figure 4 - Comparison of Accelerometers



of 15 lengthwise down the bridge. One row was down the center while the other two rows were at the edges of the deck. The driving point was near mid-span, along one edge of the deck. It was hoped that this driving point would excite as many modes as possible. This test required multiple moves of transducers and also required closing the bridge to traffic. From this data intelligent choices about transducer locations could be made so that, for the rest of the survey, the frequency of the first bending mode could be identified as quickly and easily as possible.

Portable data acquisition was available in multiples of 4 channels with a maximum number of 16 channels. The modal results from the detailed investigation showed that eight channels of acquisition were sufficient to positively identify the first bending mode: five accelerometers along one side of the structure provide a good means for identifying the first bending mode shape in the longitudinal direction; two accelerometers on the opposite side of the bridge deck enable detection of torsional motion, The transducer lay-outs are illustrated in Figure 5. The accelerometers were mounted directly above the second stringer away from each side of the bridge.

Figure 5 - Top View of Bridge Deck Showing 7 Acclerometer Array



2.5 Signal Processing Parameters

Ten averages were used to gather the FRF measurements. This decision was based on the fact that the measured FRFs became stable after this number, so that adding to the number of averages did not significantly improve the quality of the measurements. The sample rate and record length were adjusted so that a span of 50 Hz with 0.25 Hz resolution was obtained. A force window was used to zero the force input channel after 20% of the time record length. An exponential window as also applied. Since the response measurements were totally observed transients, the exponential window was not needed for this purpose but was used to help remove the response to uncorrelated inputs that, after the correlated signal decayed, were strong enough to affect the coherence measurement[3]. This would have the effect of adding some external damping to the measurements that is not actually related to the dynamics of the structure. The polynomial cure-fitting method was employed to calculate the poles of the system and the associated mode shapes. This local method of curve fitting was used because the poles of the system did not remain stable throughout the measurement process.

2.6 Modal Results

Six modes were identified in the range from 0-50 Hz for the East Branch of the Ontonagon River Bridge. Figure 6 provides graphical representations of each of the modes. FRF synthesis and the Modal Assurance Criterion (MAC)

Figure 6 - Experimental Modal Results from the East Branch of the Ontonagon River Bridge

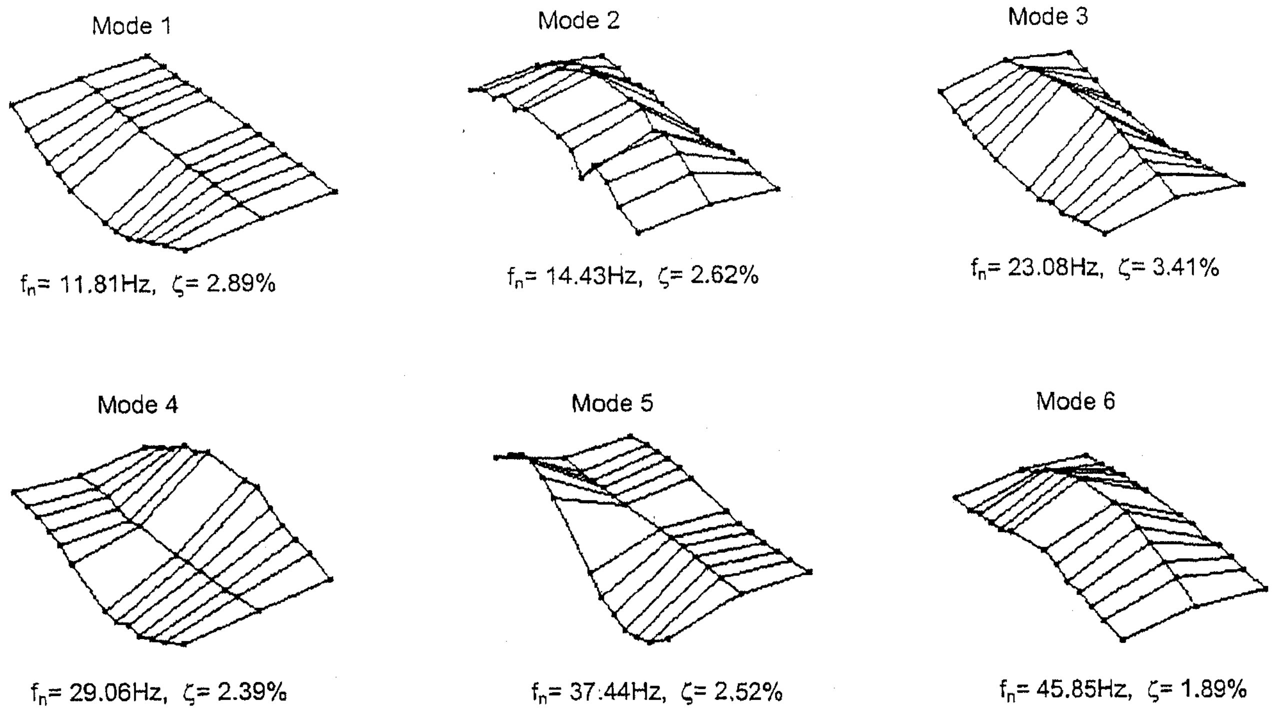
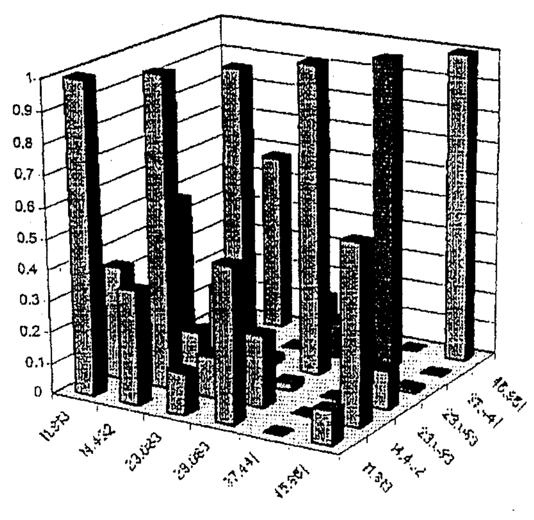


Figure 7 - MAC Matrix



matrix were used to appraise the validity of the modal results. The MAC matrix (Figure 7) shows that some of the modes at different frequencies show a substantial degree of similarity. One explanation for this is that the response of important points on the structure that differentiate between the modes were not measured[4]. Secondly, it is possible that the calculated mode shapes do not accurately describe the motion of the structure at all measurement degrees of freedom for all modes. Synthesized FRFs support this idea. As the distance between the excitation and response locations increases, synthesized FRFs do not closely follow the measured FRFs at frequencies near the 14.43 Hz mode and the 29.06 Hz mode. Inaccuracies in the estimated residues for these modes at certain measurement degrees of freedom will lead to error in the calculated mode shapes. Mode shapes that are not entirely accurate for all measurement locations could lead to the appearance of similarities in the mode shapes that do not actually exist in the true mode shapes of the structure.

Results from the other bridges yielded similar results. The frequency and damping information for the first bending mode of all the bridges tested are tabulated in Table 1.

Bridge Name	Span (ft)	Natural Frequency (Hz)	Damping Ratio (%)
East Branch	24	14.43	2.62
Beaver Creek	21.5	17.31	3.44
Dead Stream	21.5	17.96	4.61
Kidney Creek	24	7.02	4.07
Jumbo Creek	24.17	13.37	2.77
Stony Creek	21.42	23.752	2.69

2.7 Future Experimental Work

Of foremost importance is discovering why it has been difficult to observe pure first bending modes in the bridge structures. It may be a function of bow the bridge is being excited. The amount of energy needed to excite the entire structure may be great enough to exacerbate the nonlinear characteristics of the structure. Another possibility is that the boundary conditions or material properties are not uniform across the width of the bridge. The presence of rails along each side of the bridge is a possible source of added stiffness in a localized area. The experimental technique will be investigated in depth by working on a bridge structure constructed in a laboratory setting. A bridge deck with constant properties across the width of the bridge will be constructed and placed under controlled boundary conditions. This will allow the dynamic response of the bridge deck to be separated from the influence of inconsistent boundary conditions. More information about the linearity of timber structures must also be gathered since peak shifting has been noted in field measurements. Swept sine excitation will be used to investigate this because a constant known force level can be applied at any chosen frequency.

3. ANALYTIC MODEL

3.1 Derivation of the Model

An analytic model is used to relate the stiffness properties of the bridge to its measured natural frequency. Continuous system theory has been chosen as the means for developing the model. The stringers are assumed to provide all of the stiffness. This assumption is made because, compared to the height of the stringers, the thickness of the deck material is very small. Also, the deck is not continuous and the deck boards are nailed perpendicular to the stringers, reducing the stiffness that would be provided in the case of simple bridge bending. The total mass of the deck is distributed into the assumed mass of the stringers. The partial differential equation (PDE) governing the transverse vibration for a

$$\frac{\partial^2 u}{\partial t^2}+\left(\frac{E\cdot I}{m}\right)\frac{\partial^4 u}{\partial x^4}=0 \qquad (1)$$

accomplished by means of the separation of variables method and is largely dependant on the boundary Conditions at each end of the beam. Blevins has shown that a general form for the natural frequency for any mode (i) can be derived, and is given in equation 2, where l_i is a factor dependant on the boundary conditions of the beam[5]. Determining the actual boundary conditions of a bridge in the

$$f_i=\frac{\lambda_i^2}{2\pi L^2}\left(\frac{E\cdot I}{m}\right)^{\frac{1}{2}} \qquad (2)$$

field is probably the largest single obstacle remaining in this study. The effects of rotary inertia and shear in the beam's cross section can further influence the natural frequencies of a beam. The more complicated PDE including these factors is shown in equation 3. Closed form solutions for this partial differential equation are generally not available. It is, however, possible to relate the frequency solutions of equation 3 to those of equation 1. The method for relating those frequencies will not be discussed in depth here but is expanded upon in reference [5].

$$E\cdot I\cdot\left(\frac{\partial^4 u}{\partial x^4}\right)+m\cdot\left(\frac{\partial^2 u}{\partial t^2}\right)-\left(J+\frac{E\cdot I\cdot m}{k\cdot A\cdot G}\right)\cdot\left(\frac{\partial^4 u}{\partial x^2\partial t^2}\right)+\frac{J\cdot m}{k\cdot A\cdot G}\cdot\left(\frac{\partial^4 u}{\partial t^4}\right)=0 \qquad (3)$$

3.2 Calibration of the Model

Following is the proposed method for calibrating the model. Controlled laboratory experiments on a bridge similar to those found in the field will be performed. First, material properties for each piece of timber used to build the structure will be recorded. This data, along with a static midpoint deflection due to a midpoint load will be used to estimate the section modulus (El product) of the bridge structure. Modal Analysis will be performed on the bridge structure with simply supported end conditions. Since the boundary condition factor, l, is known for simply supported end conditions, it will be possible to determine if the this type of bridge deck better fits the model generated from simple flexure beam theory or if rotational inertia and shear effects must be included.

Once the relationship between bending mode frequency and section modulus is understood for simply supported boundary conditions it is possible to determine the boundary condition factor which best describes the bridges currently in service. To do this, an experimental set-up that matches the design implemented in the field will be constructed. Again, modal analysis will be used to identify the new frequency at which the deck is oscillating. Finally, a new value for l corresponding to the boundary conditions of the bridges as found in the field can be determined. The analytical relationship between the mechanical properties of the bridge and the frequency of the first bending mode will then be complete. Validation will then commence.

3.3 Validation of the Model

If this model is to be useful, it must be validated with field data. This will be accomplished with a survey of bridges currently in service. Standard static deflection tests will be used to determine the stiffness properties of each bridge. Modal analyses (some already completed) will be used to find the frequency of the first bending mode. This data set will be used to prove the validity of the model.

4. CONCLUSIONS

The modal data obtained so far has raised more questions than it has answered. Several conclusions, however, can be drawn from the preliminary field data. First, the bridge structures exhibit nonlinear behavior. Changes in both resonant frequencies and magnitude of FRFs due to a change in the intensity of the force input into the structures have been observed. Second, mode shapes of pure bending along the length of the bridge structures have not been observed. The reason for this behavior is unclear, although three probable causes have been identified. It is possible that the excitation method is failing to properly excite the mode. Second, Non-uniform boundary conditions may prevent a pure bending mode from being present. Finally, the structure itself may not have uniform material properties, making a pure bending mode response impossible.

5. ACKNOWLEDGEMENTS

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