DEVELOPMENT OF FLEXURAL VIBRATION INSPECTION TECHNIQUES TO RAPIDLY ASSESS THE STRUCTURAL HEALTH OF TIMBER BRIDGE SYSTEMS

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ABSTRACT

This paper describes an effort to develop a global dynamic testing technique for evaluating the overall stiffness of timber bridge superstructures. A forced vibration method was used to measure the natural frequency of single-span timber bridges in the laboratory and field. An analytical model based on simple beam theory was proposed to represent the relationship between the first bending mode frequency and bridge stiffness (characterized as *EI* product). The results indicated that the forced vibration method has potential for quickly assessing superstructure stiffness of timber bridges, but improvements must be made in measurement system to correctly identify the first bending mode frequency in bridges in the field. The beam theory model was found to fit the physics of the superstructure of single-span timber bridges and could be used to correlate first bending frequency to global stiffness if appropriate system parameters are Identified

INTRODUCTION

Deterioration, one of the most common damage mechanisms in woad structures, often inflicts damage internally, without visible signs appearing on the surface until load bearing capacity of the affected member is greatly reduced. Determining an appropriate load rating for an existing structure and establishing rational rehabilitation, repair, or replacement decisions can be achieved only after an accurate assessment of existing condition. Knowledge of the condition of the structure can reduce repair and replacement costs by minimizing labor and materials and extending service life.

In general, structural condition assessment requires the monitoring of some indicating parameters that are sensitive to the damage or deterioration mechanism in question. Current inspection methods for wood structures are limited to evaluating each structural member individually, which is a labor-intensive, time-consuming process. For field assessment of wood structures, a more efficient strategy would be to evaluate structural systems or subsystems in terms of their overall performance and serviceability. From this perspective, examining the dynamic response of a structural system might provide an alternative way to gain insight to the ongoing performance of the system. Deterioration caused by any organism or any type of physical damage to the structure reduces the strength and stiffness of the materials and thus could affect the dynamic behavior of the system. For example, if one structural system or section of the system was found to respond to dynamic loads in a manner significantly different from that observed in previous inspections, then a more extensive inspection of that structure would be warranted.

Recent cooperative research efforts of the USDA Forest Products Laboratory, Michigan Technological University, and University of Minnesota Duluth have resulted in significant progress in developing global dynamic testing techniques for nondestructively evaluating the structural integrity of wood structure systems. In particular, a forced vibration response system was developed and used to assess the global stiffness of wood floor systems in buildings [1, 2, and 3]. In these studies, a series of laboratory-

constructed wood floor system and some in-place wood floor structures were examined. An electric motor with an eccentric rotating mass was built and attached to the floor decking to excite the structure. The response of the floor to the forced vibration was measured at the bottom of the joists using a linear variable differential transducer (LVDT). The damped natural frequencies of floor systems were identified by increasing motor speed until the first local maximum deflection response was observed The period of vibration was then estimated from the cycles of this steady-state vibration. This forced vibration approach was investigated in these studies for two reasons. First, the simplicity of this technique requires less experimental skill to perform field vibration testing. This fits the need of field inspectors who usually do not have much advanced training in structural dynamic testing. Second, the cost of testing a structure using the forced vibration method is very low compared with the use of a modal testing method. Furthermore, because this method is a pure time domain method, it eliminates the need for knowledge of modal analysis. Results from previous experimental studies showed that vibration generated through a forcing function could enable a stronger response in wood floor system and give consistent frequency measurement. A decrease in natural frequency seems proportionate to the amount of decay, as simulated by progressively cutting the ends of some joists in laboratory floor settings [1]. It was also found that the analytical model derived from simple beam theory fits the physics of the floor structures and can be used to correlate the natural frequency (first bending mode) to *EI* product of the floor's cross section [3].

Cooperative research to date has provided a reasonable scientific base upon which to build an engineering application of vibration response as part of a wood structure inspection program. The purpose of this study is to extend global dynamic testing methods, specifically the forced vibration testing technique, to timber bridges in the field It is to be used as a first pass method, identifying timber bridges that need more thorough inspection. To simplify the method as much as possible (from field application consideration), we focus only on the first bending mode of the bridge vibration. Specifically, we correlate the frequency of the first bending mode to the stiffness characteristics of single-span girder-type timber bridges.

Analytical Model

The indicator of global structure stiffness that has been chosen is the fundamental natural frequency. For practical inspection purpose, an analytic model is needed for this method to relate the fundamental natural frequency to the global stiffness properties of a bridge. Continuous system theory has been chosen as the means for developing an analytical model that is based on general physical properties of bridges, such as length, mass, and cross-sectional properties.
The superstructures of single-span timber girder bridges are typically constructed of wood beams (stringers), cross bridging, deck boards, and railing systems. It is observed that the stiffness of the stringers predominates over that of the transverse deck sheathing because the thickness of the decking boards is relatively small compared with the height of the stringers. In addition, 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 cross bridging also does not contribute to the bending stiffness of the bridge because it mainly provides lateral bracing to the beams. Thus, we assumed that a single-span wood girder bridge behaves predominately like a beam with resisting moments in the vertical direction. The total mass of the deck and railing system is distributed into the assumed mass of the stringers.

The partial differential equation governing the vertical vibration for a simple flexure beam is

$$\frac{\partial^2 u}{\partial t^2} + \left(\frac{EI}{\rho A}\right)\frac{\partial^4 u}{\partial x^4} = 0 \qquad (1)$$

The solution of this partial differential equation is generally accomplished by means of the

separation of variables and is largely dependent on boundary conditions at each end of the beam. Blevins [4] showed that a general form for the natural frequency for any mode can be derived as

$$f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{\rho A}\right)^{\frac{1}{2}} \qquad (2)$$

where f_i natural frequency (mode i), λ_i a factor dependent on the boundary conditions of the beam, L beam span, ρ mass density of the beam, A cross-sectional area of the beam, and EI stiffness (modulus of elasticity E × moment of inertia I) of the beam.

Consider the vibration of a beam supported at the ends. If vibration is restricted to the first mode, Equation (2) can be rearranged to obtain an expression for the stiffness (EI) as

$$EI = \left(\frac{1}{kg}\right) f_1^2 W L^3 \qquad (3)$$

where f_i is the fundamental natural frequency (first bending mode), k is defined as a system parameter dependent on the boundary conditions of the beam (pin-pin support: k = 2.46; fix-fix support, k = 12.65), W is weight of the beam (uniformly distributed), and g is acceleration due to gravity.

EXPERIMENTAL PROCEDURE

Timber Bridge Structures. Five timber bridges currently in service, all of similar design (timber stringer plus plank deck), on the Kenton Ranger District of the Ottawa National Forest in Michigan's upper Peninsula were examined in October 2002 (Table 1). These bridges were all built in the early 1950s, and their initial designs were based upon American Association of State Highway Transportation Officials (AASHTO) standard truck loading. Bridge length measured on site ranged from 20 to 44 ft (6.1 to 13.4 m) (out-out), and width (out-out) was measured as 15 to 16 ft (4.6 to 4.9 m). The superstructure of each span bridge consists of 10 creasote-treated sawn lumber stringers (6 by 12 in. (15.2 by 30.5 cm) to 6 by 16 in. (15.2 by 40.6 cm)) with 3-in.- (7.6-cm-) thick transverse plank decks nailed perpendicular to the stringers. Running planks nailed in two strips parallel to the direction of the stringers served as a wearing surface for the single-lane bridges.

In addition to field bridges, two bridges were constructed in laboratory settings so that controlled experiments could be conducted. The first laboratory bridge (designated as Lab 1) was actually a field bridge (Onion Creek Bridge) that was removed from service and relocated to the laboratory. The bridge measures 9 ft (2.7 m) wide and 16 ft (4.9 m) long. The superstructure consists of six 6- by 12-in. by 16-ft (15.2- by 30.5-cm by 4.9-m) stringers and 3- by 10-in. (7.6- by 25.4–cm) deck boards and running planking. The second laboratory bridge (designated as Lab 2) was built with six 6–by 12-in. (15.2- by 30.5–cm) Douglas-fir and eastern white pine timbers and 3- by 8-in. (7.6–by 20.3-cm) plank deck with known material properties. Both laboratory bridges were rested on 12- by 12-in. (30.5- by 30.5—cm) sill plates that were anchored to the floor of the lab with angle iron. This approximates a simply supported boundary condition.

Table 1. Summary information for field timber bridges[a]

Bridge name/ crossing	Bridge length out-out (ft)	Bridge width out-out (ft)	No. of simple spans	Live load		Year built
				Initial design	Current posting	
Stony Creek	20	15	1	HS20	15ton	1954
Dead Stream	44	15	2	HS20	20 ton	1954
E.B.O. River	26	15	1	H15	None	1950
Jumbo River	24	16	1	H15	None	1950
Beaver Creek	43	15	2	H15	None	1954

[a] 1 ft = 0.3048 m.

Moisture Content Determination. At the time of bridge testing, the moisture content of wood in each bridge was measured with an electrical-resistance-type moisture meter and 3-in.- (76-mm--long) insulated probe pins in accordance

with ASTM D 4444 [5]. Moisture content data were collected at pin penetrations of 1, 2, and 3 in. (25, 51, and 76 mm) from the underside (tension face) of three different timber beam girders at each bridge. All field data were corrected for temperature adjustments in accordance with [6].

Forced Vibration Testing. A forced vibration technique was used to identify the first bending mode frequency of the bridge structures. This method is a purely time domain method and was proposed because it eliminates the need for modal analysis. Thio method uses an electric motor with a rotating unbalanced wheel to excite the structure (Fig. 1). This creates a rotating force vector proportional to the square of the speed of the motor. Placing the motor at midspan ensured that the simple bending mode of structure vibration was excited A single piezoelectric accelerometer (PCB U353 B51), also at midspan, was used to record the response in the time domain. To locate the first bending mode frequency, the motor's speed was slowly increased from rest until the first local maximum response acceleration was located. The period of vibration was then estimated from 10 cycles of this steady-state motion.

Static Load Testing. Because the primary goal of this work is m relate the vibrational characteristics of these timber bridge structures to a measure of structural integrity, the bridges were also evaluated with the established method of load-deflection analysis. This provided a more direct measure of the structure's *EI* product.

Static load test were conducted at each field bridge using a live load testing method. A test vehicle was placed an each bridge deck and the resulting deflections were measured from calibrated rulers suspended from each timber girder along the midspan cross section using an optical surveying level (Fig. 2). The test vehicle consisted of a fully loaded, tri-axle gravel truck with a gross vehicle weight of 47,740 lb (212.37 kN) (individual axle weights were 13,420, 17,160, and 17,160 lb (59.70, 76.33, and 76.33 kN)) and were spaced at 13.4- and 4.4-ft (4.1- and 1.3-m) intervals, respectively). Deflection readings were recorded prior m testing (unloaded), after placement of the test truck for each load case (loaded), and at the conclusion of testing (unloaded). For each load test, the test vehicle was straddling the bridge centerline with the bridge midspan bisecting the real dual truck axles. Measurement precision was ±0.04 in. (±1.0 mm) with no movements detected at the bridge supports. The static *EI* product of each bridge was then estimated from load-deflection data based upon conventional beam theory.

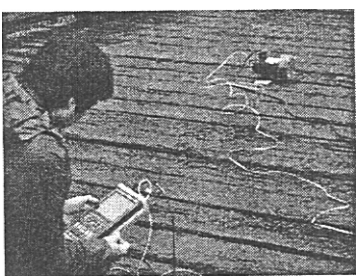

Figure 1. Forced vibration testing of field bridges with a forcing function.

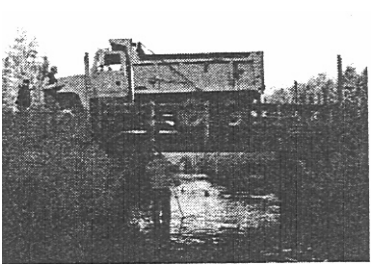

Figure 2. Static load testing of field bridges with a fully loaded gravel truck.

Estimation of Bridge Weight. As known from the theoretical model shown in Equation (3), bridge weight is needed in predicting the structure stiffness using this vibration response method. In this study, bridge weights were

estimated based upon actual dimensions along with an estimated unit weight for the timber components. A conservative unit weight of 50 lb/ft^3 (801 kg/m^3) is required for computing dead loads in the design of timber bridges according to AASHTO Standard Specifications for Highway Bridges. A less conservative unit weigbt of 35 lb/ft^3 (561 kg/m^3), which may more closely represent the actual density of creosote-treated Douglas-fir bridge components, was assumed in computing bridge weights for all five field bridges. Douglas-fir was most likely the wood species because visual evidence of incising ypically associated with Douglas-fir (and other difficult-to-treat species) was observed at all field ridges.

the bridge underside for these two bridges did not permit static load testing on both spans, we did field testing on only one span for each bridge. The tested bridge spans of these two bridges were treated as siogle-span bridges. The span length of tested bridges therefore ranged from 16 to 24.2 ft (4.9 to 7.4 m).

The moisture contents of the timbers in the field bridges ranged from 13 to 25 percent, but most members remained below 20 percent. Average moisture contents also remained below 20 percent, with the exception of the Jumbo River Bridge, wbere some beams had a moisture level of 23 to 25 percent.

Table 2. Physical characteristics and measured natural frequency of timber bridges[a]

Bridge name	Bridge span[b] (ft)	Size of stringers (in.)	Avg. MC of stringers[c] (%)	Measured bending mode frequency (Hz)	
				Forced vibration	Modal analysis
Laboratory bridges					
Lab 1 (Onion Creek)	16	6×12	n/a	19.50	19.70
Lab 2 (Sands)	21	6×12	22	9.63	9.82
Field bridges					
Stony Creek	18.5	5.25×13	18	17.85	23.75
Dead Stream (one span)	21.5	5.38×15.25	15.5	17.70	17.96
E.B.O. River	24.0	6×16	18.9	12.30	14.43
Jumbo Creek	24.2	5.88×15.75	21.8	14.15	13.37
Beaver Creek (one span)	21.5	5.50×15.25	18.2	19.90	17.31

[a] 1 ft = 0.3048 m, 1 in. = 2.54 cm.
[b] Only one span was tested for Dead Stream Bridge and Beaver Creek Bridge. The tested span length of these two bridges is one-half the total span length of the bridges.
[c] Average of the moisture content readings collected at pin penetrations of 3 in. (76 mm) deep from the underside (tension face) of three different timber beam girders.

RESULTS

Physical Characteristics of Timber Bridges. The physical characteristics of two laboratory bridges and five field timber bridges are summarized in Table 2. Of the five field bridges tested, two (Dead Stream and Beaver Creek) actually consist of two spans. Because access to

Measured Natural Frequency. To verify the bending mode frequency measured by the forced vibration method, the modal testing results of these bridges were also obtained. Modal testing and modal analysis were conducted in a parallel study that focused on investigating the use of impact-generated frequency response functions

(FRFs) for bridge evaluation [7,8]. Both forced vibration frequency and first bending mode frequency identified by modal analysis technique are shown in Table 2.

Figure 3 compares measured natural frequency from forced vibration testing and bending mode frequency from standard modal testing. It shows that under laboratory conditions, frequencies measured using the forced vibration method matched quite closely to the bending mode frequencies found from modal analysis techniques, with a difference of less than 2 percent. In the field, however, much more disparity was noticed. The Jumbo Creek and Dead Stream bridges had errors of 5% or less. This corresponded to the bridges where the lowest mode was the bending mode, and it was clearly separated from other modes of the structure [7]. Error for the Stony Creek and E.B.O. River bridges was larger because the forced vibration results corresponded to a mode other than the bending mode. The modal analysis indicated that the 3rd mode frequency for the Stony Creek and the 2nd mode frequency for the E.B.O. River are actually the true bending mode. It seems that closely spaced modes of the bridges made finding the peak amplitude of the bending mode difficult. This could be caused by the boundary conditions and material properties of the bridges not being uniform across the width of the bridge and their much higher modal density compared with other bridges tested.

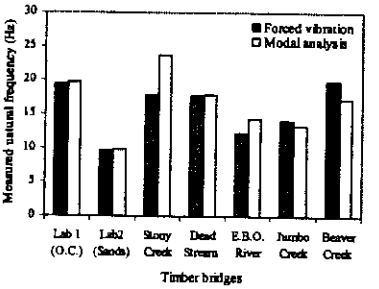

Figure 3. Comparison of measured natural frequency from forced vibration method and the first bending mode frequency from standard modal testing.

Measured Bridge Stiffness. Table 3 shows static live load deflections from field load tests. No permanent deflections were noted on the stringers monitored, and no support movements were detected during truck loading tests. Due to the nature of bridge structure and live truck loading, the deflection of stringers was uneven across the width of each bridge. Maximum deflections typically occurred in the center stringers, ranging from 0.39 in. (9.9 mm) for Beaver Creek Bridge to 0.75 in. (19.0 mm) for Jumbo River Bridge. The average live load deflections ranged from 0.27 in. (6.9 mm) for Stony Creek Bridge to 0.41 in. (10.4 mm) for E.B.O. River Bridge.

Table 3. Summary of static load testing deflections and measured bridge stiffness[a]

Bridge name/ crossing	Maximum live load deflection (in.)	Average live load deflection (in.)	Measured bridge stiffness[a] (*EI* product) ($\times 10^6$ lb-in.^{2})
Stony Creek	0.45	0.27	26268
Dead Stream	0.49	0.28	34801
E.B.O. River	0.55	0.41	42369
Jumbo River	0.75	0.34	38708
Beaver Creek	0.39	0.28	32665

[a] 1 in. = 2.54 cm, 1 lb-in^2 = 2.87×10^3 N-m^2.

To correlate the first bending mode frequency of bridges to load testing results, following assumptions were made as an initial attempt to compute the *EI* product of the bridges: (1) the

superstructure of timber bridges is similar m a beam-like structure with symmetrically placed loads; (2) the bridges are close to being simply supported (for the purpose of static deflection analysis only); (3) the average deflection of each bridge is equivalent to the value that characterized the deflections of all stringers if the bad had been applied evenly across the width of the bridge. For a beam-like structure with these assumptions, the static beam deflection theory provides following relationship:

$$EI = \frac{Pa}{24\delta}(3L^2 - 4a^2) \qquad (4)$$

where P is static load of individual axle, δ average midspan deflection, L the span length of the bridge, and a the distance from bridge support to nearest loading point.

The calculated *EI* products of the field bridges are shown in Table 3 and an hereafter referred to as the measured *EI* because they are derived from measured static data. Based on static load testing results, the structure stiffness (*EI* product) of the field bridges ranged from 26,268 × 10^6 lb-in.^{2} (75.39 × 10^6 N-m^2) (Stony Creek Bridge) to 42,368 ×10^6 lb-in.^{2} (121.64 × 10^6 N-m^2) (for E.B.O. River Bridge).

PREDICTION OF BRIDGE STIFFNESS

Figure 4 shows theoretical prediction for two extreme supporting conditions (free-free and fixed-fixed) and experimental data obtained from field bridges. Here, EI/WL^3 is treated as the independent variable and natural frequency as the dependent variable. The natural frequency is predicted over a range of EI/WL^3 assuming both simply supported and fixed boundary conditions. The measured data are then superimposed on the same set of axes. It is noted that measured results lie between simple support and rigidly fixed boundary conditions, with a bias toward the simply supported prediction. To characterize the boundary condition of each bridge, a system parameter k was determined based on experimental data of the field bridges. The average system parameter that best describes the all field bridges tested was found to be k = 4.20, with a standard deviation of 0.690.

with newly developed system parameter k, the model in Equation (3) could be used to predict the *EI* product of bridges using measured natural frequency. Figure 5 compares the predicted *EI* product from the forced vibration method and measured *EI* product from static load testing. Although the *EI* predictions for Dead Stream and Jumbo Creek bridges are quite close to measured *EI* (with less than 7 percent difference), the overall performance of the prediction model suffers from a significant mor. It appears that prediction of bridge stiffness has a significant variation, from 2 percent minimum to 37 percent maximum difference (in absolute value). Several factors contributed to this prediction error. First, one source is obviously the forced vibration method itself. As we indicated in previous discussion, the estimates of fiat bending mode frequency from forced vibration testing contain significant errors in some cases. The error is a direct result of the bending mode not being the lowest in natural frequency, so that other modes (typically torsion) were misidentified as the bending mode. In the case where first bending mode frequency was properly identified (such as Dead Stream and Jumbo Creek bridges), the predicted *EI* show much less difference from measured *EI*. The second error source in *EI* prediction is most likely the inaccurate estimate of bridge weight. Bridge weight information is essential in calculating *EI* product based on beam theory model. In this study, bridge weights were estimated based upon actual dimensions along with an estimated unit weight for the timber components. The true wood density of each bridge might be significantly different from the assumed unit weight. Other factors could also affect bridge weight, which make estimation difficult (such as species and moisture difference, wood deterioration, dirt or debris collected on the deck). Third, in spite of structure similarities, the boundary condition of each field bridge is unique due to the construction variability, load history, and road and soil

conditions. The overall system parameter k used for EI prediction here is the average value of the system parameter k_i of each bridge, which describes the entire population. The small sample size (five field bridges) is therefore a contributing factor. If more field bridges had been available, a more representative average could have been obtained.

CONCLUSIONS

A forced vibration method was used to measure the natural frequency of single-span timber bridges in the laboratory and in the field An analytical model based on beam theory was proposed to represent the relationship between the first bending mode frequency and bridge stiffness characterized as EI product. From the results of this study, we conclude following:

1. The forced vibration method has the potential to be used in the field to quickly assess timber bridge superstructure stiffness. However, improvements need to be made in testing procedure and measurement system to correctly identify the first bending mode frequency as a forcing function is applied.

2. The weight of timber bridges is essential for predicting bridge stiffness based on beam theory model. Weight estimation based on wood volume and estimated unit weight for the timber components seems inadequate to obtain reliable results.

3. The analytical model generated from simple beam theory fits the physics of single-span girder bridges, but more representative system parameters need to be developed to better correlate measured bending mode frequency to EI product.

Future Research

The experimental data collected from this study are still limited given the structural complexity of timber bridges in the real world. More analytical and experimental work is needed to fully understand the physics and structural conditions in terms of vibration response and load capacity. A new joint timber bridge research project is now underway at University of Minnesota Duluth and the USDA Forest Products Laboratory to further investigate some key issues in vibration modes and boundary conditions and to refine field testing techniques and instrumentation systems. More field timber bridges in northern Minnesota with various structural conditions will be tested with improved forced vibration response methods. To eliminate or reduce error in estimating the bending mode frequency, two accelerometers will be placed on opposite sides of the bridge superstructure. Bending mode frequency will be determined by examining both maximum accelerations and phase information from two simultaneous vibration response signals. Field vibration measurements will also be coupled by condition evaluation using traditional inspection techniques and standard live load tests using a loaded truck. To improve the reliability of vibration response methods, more comprehensive mathematical models will be developed to quantify the sensitivity of bridge response to various environmental, experimental, and architectural factors.

Acknowledgments

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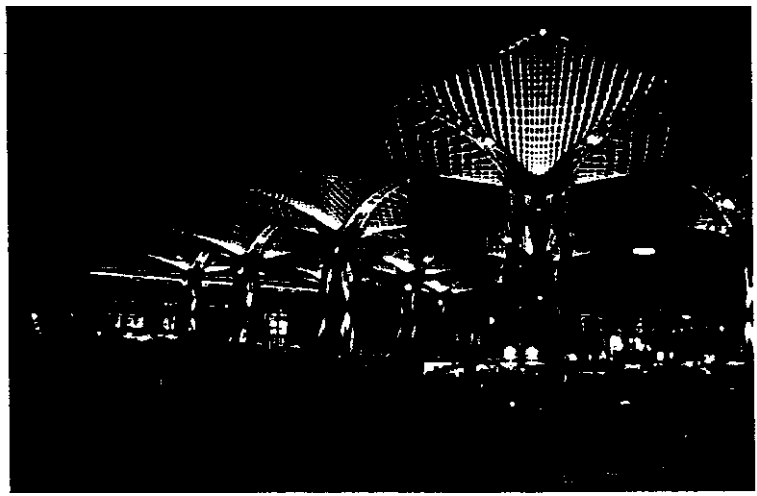

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