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Nondestructive Assessment of Single-Span Timber Bridges Using a Vibration-Based Method

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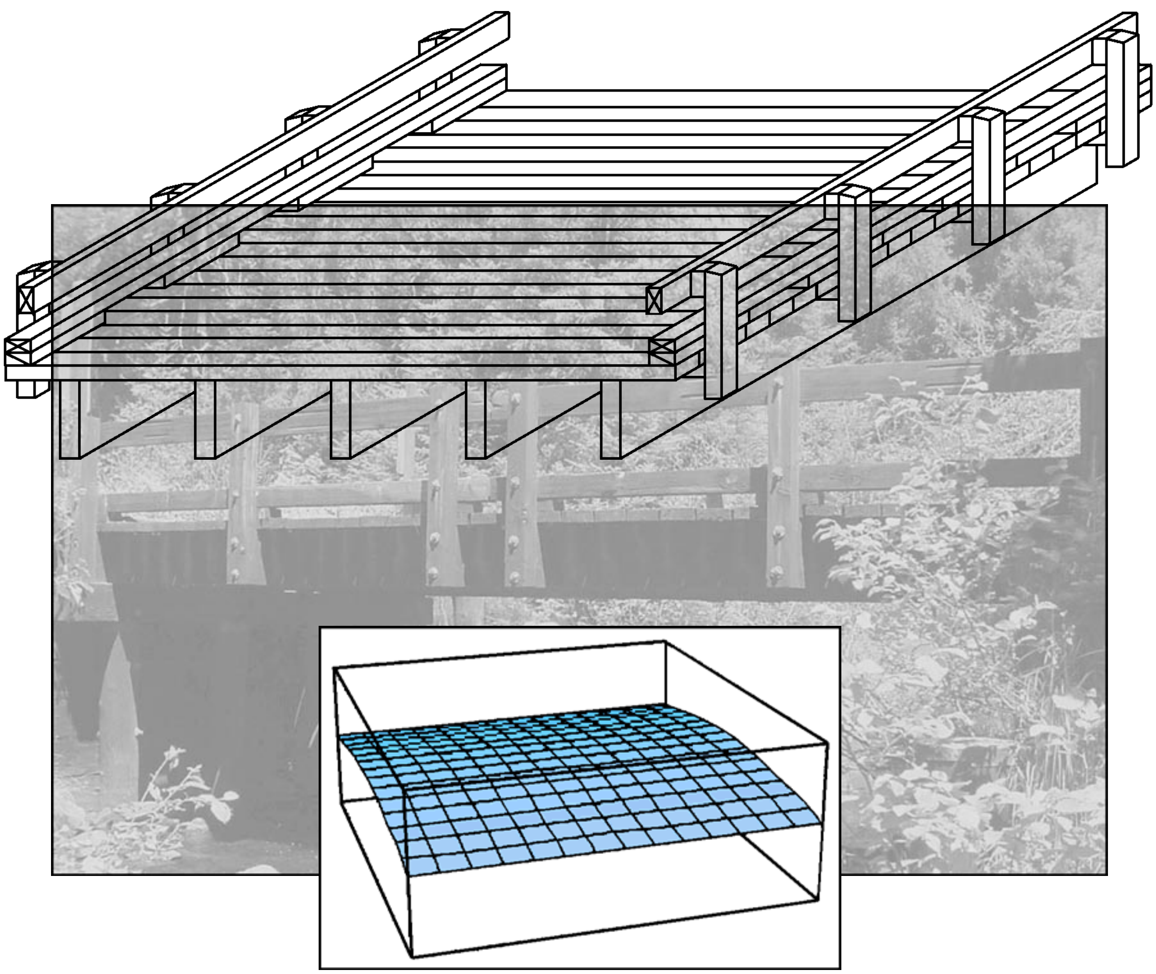

Abstract

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 the stiffness of the timber bridge superstructure. However, improvements must be made in the 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.

Keywords: timber bridges, NDE, vibration

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Introduction

The deterioration of wood structures often occurs internally before any external signs of damage appear. The load-bearing capacity of the affected member is greatly reduced before surface deterioration is visible. Determination of an appropriate load rating for an existing structure and rational decisions about rehabilitation, repair, or replacement can be made only after an accurate assessment of the existing condition. Knowledge of the condition of a 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 into the ongoing performance of the system. Deterioration caused by an organism or 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 a structural system or section of the system were found to respond to dynamic loads in a manner significantly different from that observed in previous inspections, this would warrant a more extensive inspection of the structure.

Recent cooperative research efforts of the USDA Forest Service 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 (Soltis and others 2002, Ross and others 2002, Wang and others 2005). In these studies, a series of laboratory-constructed wood floor systems and several 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 the 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.

The forced vibration approach was investigated for two reasons. First, compared with other techniques, the simplicity of this technique requires less experimental skill to perform field vibration testing. This fits the need of field inspectors who usually have little 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 forced vibration 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 systems 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 joists in laboratory floor settings (Soltis and others 2002). In addition, the analytical model derived from simple beam theory was found to fit the physics of floor structures and can be used to correlate the natural frequency (first bending mode) to *EI* product of the floor cross section (Wang and others 2005).

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 (in regard to field application), we focus on only 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.

Figure 1—Typical single-span girder timber bridge: (a) side view, (b) structure, (c) deck.

Analytical Model

The fundamental natural frequency was chosen as the indicator of global structure stiffness. For the purpose of practical inspection, an analytic model is needed for relating the fundamental natural frequency to the global stiffness properties of a bridge. Continuous system theory was 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. Specifically, continuous beam and plate theories were investigated as bases for the model, providing two initial relationships between bending mode frequency and *EI* product.

Beam Theory

The superstructure of a single-span timber girder bridge is typically constructed of wood beams (stringers), cross bridging, deck boards, and a railing system (Fig.1). The beam theory model is based on the modal response of a single beam of rectangular cross section. To relate a single-span wood girder bridge to the beam theory, we assume that the stiffness of the stringers predominates over that of the transverse deck sheathing and provides all the stiffness. The total mass of the deck and railing system is distributed into the assumed mass of the stringers. This assumption is made 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. Under this assumption, a single-span wood girder bridge behaves predominately like a beam with resisting moments in the vertical direction.

The partial differential equation (PDE) governing vertical vibration for a simple flexure beam is

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

where

u is vertical displacement variable,

t time,

x variable along beam length,

E modulus of elasticity,

I area moment of inertia of bridge cross section, and

m mass per unit length.

The solution of this PDE is generally accomplished by means of the separation of variables and is largely dependent on boundary conditions at each end of the beam. Blevins (1993) showed that a general form for the natural frequency for any mode can be derived as

$$f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2} \tag{2}$$

where

f_i is natural frequency (mode i),

λ_i a factor dependent on beam boundary conditions,

L beam span,

I moment of inertia, and

EI stiffness of beam.

Equation (2) ignores the contributions of rotary inertia and shear in the beam cross section. The more complicated PDE including these factors is

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

where

J is mass moment of inertia of bridge cross section,

k shear coefficient,

A cross-sectional area, and

G modulus of rigidity.

Close form solutions for this PDE are not generally available. In analyzing the relative importance of added factors to natural frequencies, Morison (2003) found that for typical timber bridge properties, the effect on frequency is only about 1%.

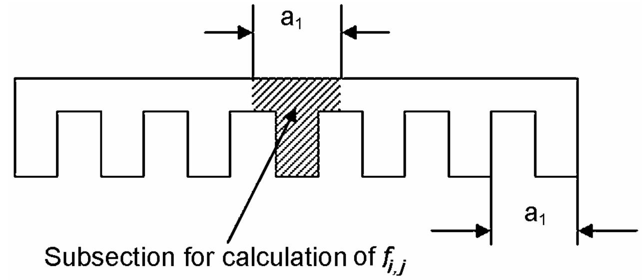

Figure 2—Ribbed plate cross-section.

Plate Theory

Considering the geometric form of its cross section, a single-span girder timber bridge may be modeled as a ribbed plate (Fig. 2). Here, the assumption is that the deck is continuously and perfectly attached to the stiffening ribs. The general expression for the natural frequencies of an orthotropic plate is (Blevins 1993)

$$f_{ij} = \frac{\pi}{2\gamma^{1/2}}\left[\frac{G_1^4 D_x}{a^4} + \frac{G_2^4 D_y}{b^4} + \frac{2H_1 H_2 D_{xy}}{a^2 b^2} + \frac{4D_k(J_1 J_2 - H_1 H_2)}{a^2 b^2}\right]^{1/2} \tag{4}$$

where

i and j are mode number indexes,

γ is mass per unit area of deck,

G_1, G_2, J_1, J_2, H_1, and H_2 are system parameters,

D is a cross-sectional constant,

a is bridge length, and

b is bridge width.

The system parameters in Equation (4) with a subscript of 1 refer to the width dimension of the bridge (constrained edges), and a subscript of 2 relates to the length dimension (free edges). For a simple bending mode with elementary boundary conditions, many system parameters become zero, allowing for great simplification of Equation (4):

$$f_i = \frac{\pi}{2\gamma^{1/2}}\left[\frac{G^4 EI_r}{L^4 a_1}\right]^{1/2} \tag{5}$$

where

I_r is area moment of inertia of reduced cross-section,

L length of bridge, and

a_1 stringer spacing.

In this model, the total mass of the system is distributed over the surface area of the bridge deck and the area moment of inertia is calculated for a subsection of the overall bridge cross-section, as illustrated in Figure 2.

Experimental Methods

Timber Bridge Structures

Two experimental 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 measured 9 ft (2.7 m) wide by 16 ft (4.9 m) long, and the superstructure consisted 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) measured 9 ft (2.7 m) wide by 21 ft (6.4 m) long. It 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 anchored to the floor with angle iron, which approximated a simply supported boundary condition.

In addition to laboratory bridges, five field timber bridges currently in service on the Kenton Ranger District of the Ottawa National Forest in the Upper Peninsula of Michigan, all of similar design (timber stringer plus plank deck), were examined in October 2002 (Table 1). All these bridges were 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 from 15 to 16 ft (4.6 to 4.9 m) (out–out). The superstructure of each bridge span consisted of 10 creosote-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. Details for each field bridge are summarized in Appendix A.

Moisture Content Determination

At the time of bridge testing, wood moisture content of 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 (ASTM 2000). 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 for each bridge. All field data were corrected for temperature adjustments in accordance with Pfaff and Garrahan (1984).

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. An electric motor with a rotating unbalanced wheel is used to excite the structure (Fig. 3), which 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 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 was to relate the vibrational characteristics of the timber bridge structures to a measure of structural integrity, the bridges were also evaluated with the established method of static load–deflection field testing. Stiffness of bridge superstructure (*EI* product) could then be estimated from the field test results.

Static load tests were conducted on each field bridge using a live load testing method. A test vehicle was placed on the bridge deck, and the resulting deflection was measured from calibrated rulers suspended from each timber girder along the midspan cross-section using an optical surveying level

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

Bridge	Bridge length, out–out (ft)	Bridge width, out–out (ft)	Simple spans (no.)	Live load: Initial design	Live load: Current posting	Year built
Stony Creek	20	15	1	HS20	15 ton	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. [b]East Branch Ontonagan River

Figure 3—Forced vibration testing of field bridges with a forcing function.

Figure 4—Static load testing of field bridges with fully loaded gravel truck.

(Fig. 4). 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 of 13,420, 17,160, and 17,160 lb (59.70, 76.33, and 76.33 kN)). The rear 17,160-lb (76.33-kN) axles were spaced 4.4 ft (1.3 m) apart, and the 13,420-lb (59.70-kN) axle was 13.4 ft (4.1 m) from the rear axles (Fig. 5). Deflection readings were recorded prior to 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 straddled the bridge centerline; the bridge midspan bisected the rear dual truck axles. Figure 5 details the loading condition for the static test. Measurement precision was ±0.04 in. (±1.0 mm) with no vertical movements detected at the bridge supports. The static *EI* product of each bridge was estimated from load–deflection data on the basis of conventional beam theory.

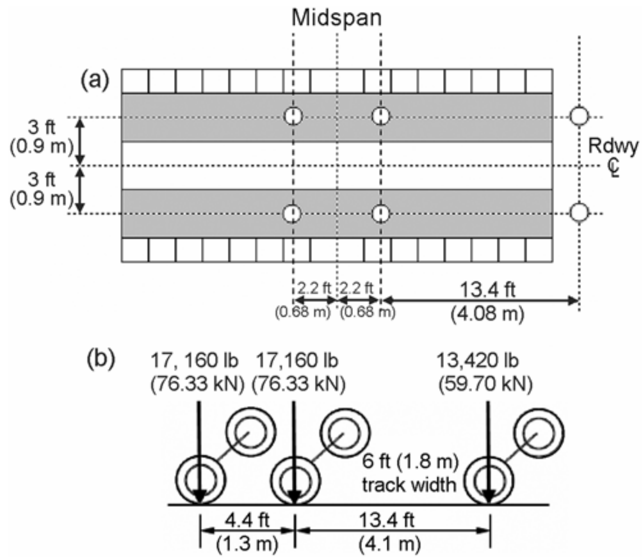

Figure 5—Schematic of static loading pattern: (a) top view, (b) side view.

Estimation of Bridge Weight

As known from the theoretical model shown in Equations (2) and (5), bridge weight is needed in predicting structural stiffness using the vibration response method. In this study, bridge weights were estimated based on actual dimensions along with an estimated unit weight for 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 weight 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 typically associated with Douglas-fir (and other difficult-to-treat species) was observed at all field bridges.

Results and Discussion

Physical Characteristics of Timber Bridges

The physical characteristics of the 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 consisted of two spans. Because access to the bridge underside of 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 single-span bridges. The span length of tested bridges therefore ranged from 16 to 24.2 ft (4.9 to 7.4 m).

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

Bridge	Bridge span[b] (ft)	Size of stringers (in.)	Avg. moisture content of stringers[c] (%)	Measured bending mode frequency (Hz)	
				Forced vibration	Modal analysis
Laboratory bridges					
Lab 1 (Onion Creek)	16	6 by 12	n/a	19.50	19.70
Lab 2 (Sands)	21	6 by 12	22	9.63	9.82
Field bridges					
Stony Creek	18.5	5.25 by 13	18	17.85	23.75
Dead Stream (1 span)	21.5	5.38 by 15.25	15.5	17.70	17.96
E.B.O. River	24.0	6 by 16	18.9	12.30	14.43
Jumbo River	24.2	5.88 by 15.75	21.8	14.15	13.37
Beaver Creek (1 span)	21.5	5.50 by 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 was half the total span length.
[c]Average of moisture content readings collected at pin penetration 3-in. (76-mm) deep from underside (tension face) of three timber beam girders.

The moisture content of the field bridge timbers ranged from 13% to 25%, but that of most timbers was below 20%. Average moisture content was below 20%, with the exception of the Jumbo River Bridge, where some beams had a moisture level of 23% to 25%.

Model Validation

To select the most appropriate model, a validation process was performed through testing a laboratory-constructed timber bridge. The experimental bridge setup designated as Lab 2 was selected because a known value of the boundary condition factor exists for this setup. This allowed comparison of the model's prediction with the actual measured performance of the structure. Figure 6 shows the results of the validation process. Each candidate model was used to predict the relationship between *EI* product and bending mode frequency for the bridge, using the known values of mass and the physical dimensions of the system. The data points represent the measured performance of the bridge. It is evident from this figure that beam theory is the obvious choice to model bridge behavior.

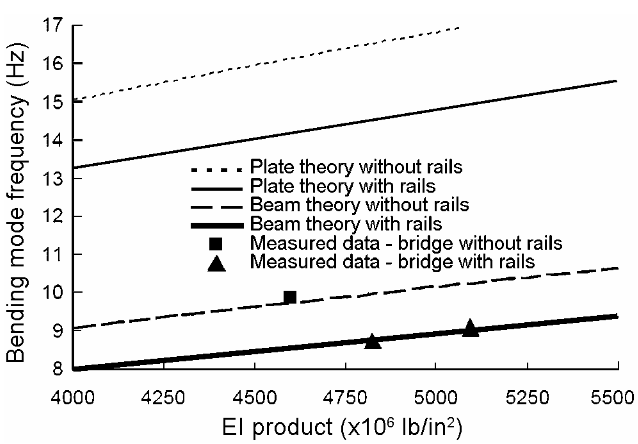

Figure 6—Model predictions and measured data for laboratory bridge.

The error between the beam theory predictions and measured frequencies was less than 3%. The huge overprediction of frequency with the plate model points to one of the largest discrepancies between the plate model and the physical structure. The deck is a segmented structure in reality, whereas the plate model assumes that the deck is continuous. Because the deck is segmented, it does not carry much of the bending moment and therefore contributes little to the bending stiffness of the bridge structure. The beam model, on the other hand, matches this assumption and best captures the physics of this type of timber bridge. Therefore, the simple beam model was proposed to represent the relationship between the first bending mode frequency and bridge stiffness.

Measured Natural Frequency

To verify the bending mode frequency measured by the forced vibration method, modal testing results of the bridges were obtained. Modal testing and analysis were conducted in a parallel study focused on investigating the use of impact-generated frequency response functions (FRFs) for bridge

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

Bridge	Maximum live load deflection (in.)	Average live load deflection (in.)	Measured bridge stiffness (*EI* product) ($\times 10^6$ lb-in^2)
Stony Creek	0.413	0.272	26,268
Dead Stream	0.394	0.280	34,801
E.B.O. River	0.551	0.413	42,369
Jumbo River	0.433	0.346	38,708
Beaver Creek	0.386	0.276	32,665

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

evaluation (Morison 2003, Morison and others 2002). The results of both forced vibration frequency and first bending mode frequency identified by modal analysis technique are shown in Table 2.

Figure 7 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 (less than 2% difference) the bending mode frequencies found from modal analysis techniques. In the field, however, much more disparity was noticed. The difference in frequencies for the Jumbo River and Dead Stream bridges was 5% or less. These results correspond to the bridges for which the bending mode was the lowest mode, and this mode was clearly separated from other modes of the structure (Morison 2003). 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 bridge and the 2nd mode frequency for the E.B.O. River bridge were actually the true bending mode. It seems that the closely spaced modes of the bridges made finding the peak amplitude of the bending mode difficult. This could have been caused by the lack of uniformity for the boundary condition and material properties across the width of the bridges and the much higher modal density of these bridges compared with that of the other bridges tested.

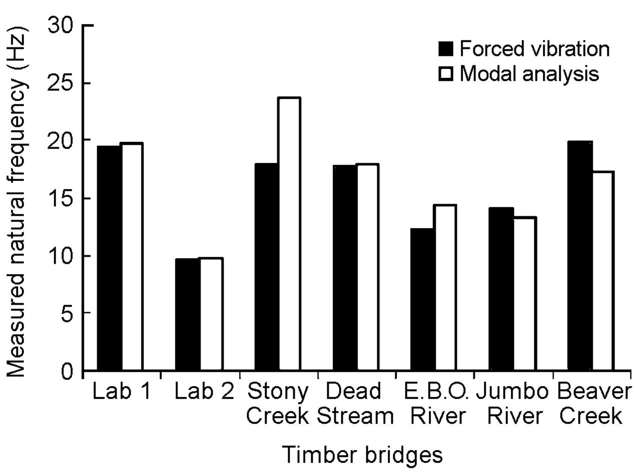

Figure 7—Comparison of measured natural frequency from forced vibration method and first bending mode frequency from standard modal testing.

Measured Bridge Stiffness

Table 3 shows static live load deflections from field load tests. No permanent deflection was noted on the stringers monitored, and no support movements were detected during truck loading tests. Because of the nature of the bridge structure and live truck loading, the deflection of stringers was uneven across the width of each bridge. Maximum deflection typically occurred in the center stringers, ranging from 0.386 in. (9.8 mm) for Beaver Creek Bridge to 0.551 in. (14.0 mm) for E.B.O. River Bridge. The average live load deflection ranged from 0.272 in. (6.9 mm) for Stony Creek Bridge to 0.413 in. (10.5 mm) for E.B.O. River Bridge. The load–deflection curves of the filed bridges are shown in Appendix B.

To correlate the first bending mode frequency to load testing results, the following assumptions were made to compute the *EI* product of the bridges:

1. The superstructure of timber bridges is similar to 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 characterizes the deflections of all stringers if the load is applied evenly across the width of the bridge.

Given these assumptions, the static beam deflection theory provides the following relationship for a beam-like structure:

$$EI = \frac{Pa}{24\delta}(3L^2 - 4a^2) \qquad (6)$$

where

P is static load of individual axle,

δ average midspan deflection,

L span length of bridge, and

a distance from bridge support to nearest loading point.

The calculated *EI* products of the field bridges are shown in Table 3 and are hereafter referred to as the measured *EI* because they were derived from measured static data. Based on static load testing results, the structural stiffness (*EI* product) of the field bridges ranged from $26{,}268 \times 10^6$ lb-in^2 (75.39×10^6 N-m^2) (Stony Creek Bridge) to $42{,}368 \times 10^6$ lb-in^2 (121.60×10^6 N-m^2) (E.B.O. River Bridge).

Prediction of Bridge Stiffness

Equation (2) is a general form for the natural frequency of any mode for a simply supported beam. If vibration is restricted to the first mode, Equation (2) can be rearranged to obtain an expression for stiffness (*EI*) as

$$EI = \left(\frac{1}{Kg}\right) f_1^{\,2} W L^3 \qquad (7)$$

where

- f_1 is fundamental natural frequency (first bending mode),
- K system parameter dependent on beam boundary conditions (pin–pin support, $K = 2.46$; fix–fix support, $K = 12.65$),
- W weight of beam (uniformly distributed), and
- g acceleration due to gravity.

Figure 8 shows the 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 described all field bridges tested was found to be 4.20, with a standard deviation of 0.690.

With the newly developed system parameter K, the model in Equation (7) could be used to predict the *EI* product of bridges using measured natural frequency. Figure 9 compares the predicted *EI* product from the forced vibration method and measured *EI* product from static load testing. Although the *EI* predictions for the Dead Stream and Jumbo River Bridges were quite close to measured *EI* (less than 7% difference), the overall performance of the prediction model suffers from a significant error. It appears that prediction of bridge stiffness has a significant variation, from 2% minimum to 37% maximum difference (in absolute value).

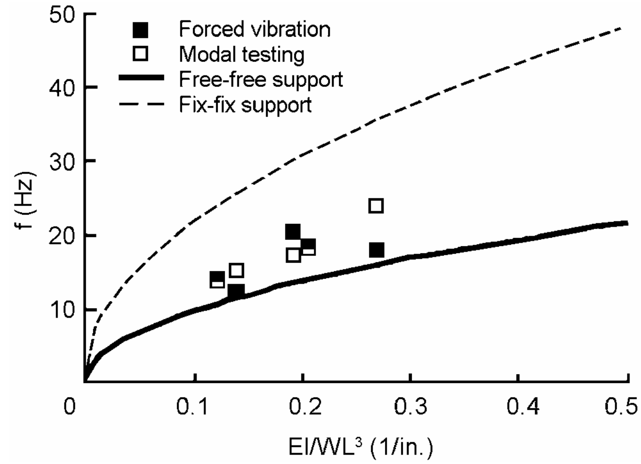

Figure 8—Theoretical predictions and experimental data.

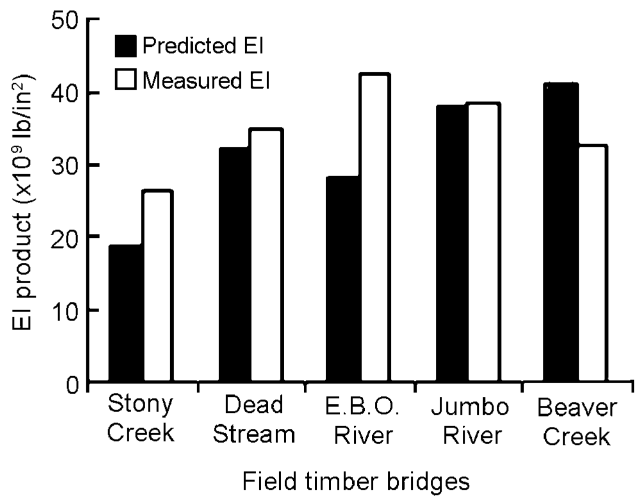

Figure 9—Comparison of predicted *EI* and measured *EI* of field brides.

Several factors contributed to this prediction error. First, one source is obviously the forced vibration method itself. As indicated in the previous discussion, the estimates of first bending mode frequency from forced vibration testing contained significant errors in some cases. The error was 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 for the Dead Stream and Jumbo River Bridges), the predicted *EI* showed 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 on actual dimensions along with an estimated unit weight for 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 structural 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 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 the following:

- 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.
- Timber bridge weight is essential for predicting bridge stiffness based on the beam theory model. Weight estimation based on wood volume and estimated unit weight for timber components seems inadequate to obtain reliable results.
- The analytical model generated from simple beam theory fits the physics of single-span girder bridges better than the model from pate theory, 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 the University of Minnesota–Duluth and the USDA Forest Service, Forest Products Laboratory to further investigate 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 acceleration 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.

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Appendix A—Bridge Information Summaries

Stony Creek Bridge

Location
- State: Michigan
- County: Houghton County
- Highway: Forest Road 3660 or County Route D162
- Waterway crossing: Stony Creek
- Section, Township, Range: S33, T47N, R36W

Owner: Houghton County
Year built: 1954

Design configuration: Simply supported
- Length: 20 ft
- Width: 15 ft
- No. spans: 1
- No. traffic lanes: 1
- Loading: AASHTO HS20–44

Superstructure type: Sawn lumber stringer with transverse plank deck
- No. stringers: 10
- Stringer size: 5.25 by 13 in.
- Spacing (c–c): 18 in.
- Decking size: 3 by 8 in.
- Species: Douglas-fir
- Preservative: Creosote
- Skew: 0°

Substructure type: Timber pile with timber cap
Wearing surface: Timber running planks
Recent condition rating: 2001 inspection rating of 7 for all components

Dead Stream Bridge

Location

State: Michigan
County: Houghton
Highway: Forest Road 2400 or County Route D163C
Waterway crossing: Dead Stream
Section, Township, Range: S33, T47N, R35W

Owner: Houghton County
Year built: 1954

Design configuration: Simple span

Length: 44 ft
Width: 15 ft
No. spans: 2
No. traffic lanes: 1
Loading: AASHTO HS20–44

Superstructure type: Sawn lumber stringer with transverse plank deck

No. stringers: 10
Stringer size: 15.25 by 5.38 in.
Spacing (c–c): 18 in.
Decking size: 3 by 8 in.
Species: Douglas-fir
Preservative: Creosote
Skew: 0°

Substructure type: Timber pile with timber cap
Wearing surface: Timber running planks
Recent condition rating: 2001 inspection rating of 7 for all components

East Branch Ontonagon River Bridge

Location

State: Michigan
County: Houghton
Highway: Forest Road 3500
Waterway crossing: East Branch Ontonagon River
Section, Township, Range: S26, T47N, R36W

Owner: Ottawa National Forest
Year built: 1950

Design configuration: Simple span

Length: 26 ft
Width: 15 ft
No. spans: 1
No. traffic lanes: 1
Loading: AASHTO H15–44

Superstructure type: Sawn lumber stringer with transverse plank deck

No. stringers: 10
Stringer size: 6 by 16 in.
Spacing (c–c): 18 in.
Decking size: 3 by 8 in.
Species: Douglas-fir
Preservative: Creosote
Skew: 0°

Substructure type: Timber pile with timber cap
Wearing surface: Timber running planks (size)
Recent condition rating: 2002 ratings—deck (5), superstructure (6), substructure (5)

Jumbo River Bridge

Location
State: Michigan
County: Iron County
Highway: Forest Road 3610
Waterway crossing: East Branch Jumbo River
Section, Township, Range: S10, T46N, R37W

Owner: Ottawa National Forest
Year built: 1950

Design configuration: Simple span
Length: 24 ft
Width: 16 ft
No. spans: 1
No. traffic lanes: 1
Loading: AASHTO H15–44

Superstructure type: Sawn lumber stringer with transverse plank deck
No. stringers: 10
Stringer size: 5.88 by 15.75 in.
Spacing (c–c): 18 in.
Decking size: 4 by 6 in.
Species: Douglas-fir
Preservative: creosote
Skew: 0°
Substructure type: Timber pile with timber cap
Wearing surface: Timber running planks
Recent condition rating: 2002 ratings—deck (7), superstructure (6), substructure (6)

Beaver Creek Bridge

Location
State: Michigan
County: Houghton
Highway: Forest Road 1100
Waterway crossing: Beaver Creek
Section, Township, Range: S29, T48N, R37W

Owner: Ottawa National Forest
Year built: 1954

Design configuration: Simple span
Length: 43 ft
Width: 15 ft
No. spans: 2
No. traffic lanes: 1
Loading: AASHTO H15–44

Superstructure type: Sawn lumber stringer with transverse plank deck
No. stringers: 10
Stringer size: 5.50 by 15.25 in.
Spacing (c–c): 18 in.
Decking size: 3 by 8 in.
Species: Douglas-fir
Preservative: Creosote
Skew: 0°

Substructure type: Timber pile with timber cap
Wearing surface: Timber running planks
Recent condition rating: 2000 ratings—deck (6), superstructure (6), substructure (5)

Appendix B—Summary of Static Load Test Data