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Development of Flexural Vibration Inspection Techniques to Rapidly Assess the Structural Health of Rural Bridge Systems

Final Report

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16. Abstract (Limit: 200 words)

Approximately 4,000 vehicle bridges in the State of Minnesota contain structural timber members. Recent research at the University of Minnesota Duluth Natural Resources Research Institute (UMD NRRI) has been conducted on vibration testing of timber bridges as a means of developing rapid in-place testing techniques for assessing the structural health of bridges. The technique involves measuring the frequency characteristics of the bridge superstructure under forced flexural vibration. The peak frequency of vibration was measured and compared to a set of load testing data for each of 9 bridges. Each bridge was also inspected using commercially available advanced inspection equipment to identify any major structural problems with individual bridge components such as timber pilings, pile caps, and girders. Two bridges were identified that needed immediate maintenance attention. The relationship between the load deflection data and the vibration characteristics showed a useful relationship and the results indicate that forced-vibration methods have potential for quickly assessing timber bridge superstructure stiffness. However, improvements must be made to the measurement system to correctly identify the 1st bending mode frequency of the field bridges. This global vibration technique has potential benefits for routine inspections and long-term health monitoring of timber bridge superstructures.

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Executive Summary

Approximately 4,000 vehicle bridges in the State of Minnesota contain structural timber members. Current inspection techniques are limited to visual, sounding and coring inspections to assess the quality and performance of individual bridge members. The majority of these bridges are found in rural environments. These techniques are suitable for identifying advanced decay, but have limited effectiveness for early-stage decay that causes substantial structural degrade and decreased safety if not detected. Wood is a naturally occurring engineering material that is prone to deterioration caused by decay fungi and insect attack. For this reason, it is important to conduct frequent inspections of timber bridges with modern inspection equipment.

Recent collaborative research between the University of Minnesota Duluth Natural Resources Research Institute, Michigan Technological University and the USDA Forest Products Laboratory has developed vibration testing techniques for short-span, simply supported timber bridges. In contrast to typical bridge inspections where individual components, such as pilings, girders, etc., have been evaluated, the entire bridge is tested as a system by using free and/or forced vibration. Specifically, the technique involves measuring the frequency characteristics of the bridge superstructure under free or induced flexural vibration. This research showed that both forced and free vibration could be used as rapid inspection techniques to determine the stiffness of the bridge and the corresponding overall condition.

The focus of this research project was to use forced vibration testing techniques and load testing on an additional 12-plus timber bridge spans (from 9 bridges) of varying ages and designs to develop a data set for use in future commercialization and technology transfer activities. At the same time, a comprehensive inspection of each timber bridge was conducted using best practices as a means of understanding the physical health of each bridge tested. These inspections used a combination of visual examinations, physical and mechanical testing, stress wave timing techniques and resistance microdrilling techniques.

Inspection reports were completed for each individual timber bridge tested in this project. The combination of testing methods identified several bridges that required repair to the timber pilings, pile caps and girder beams, including St. Louis County bridges 242 and 53. These repairs were made by the bridge maintenance crew from St. Louis County. The completed repairs significantly increased the service life of these bridges.

The vibration and load testing showed a useful relationship between the peak frequency of vibration and the calculated stiffness of the bridge as determined through load testing. A correlation coefficient squared (R^2) of 0.84.

The conclusions of the project were:

- The use of commercially available inspection equipment allowed the research team to identify critical areas of structural deterioration, resulting in completed repairs by St. Louis County. This deterioration typically took place in the bridge substructure, including pilings and pile caps. The use of stress wave timing and resistance microdrilling equipment allowed the inspection team to identify and quantify the decay in these bridges.

- The use of vibration testing allowed inspectors to conduct rapid inspections on bridge sections to identify a peak frequency of vibration. When compared to the bridge stiffness as measured by load testing, a useful relationship occurred. The frequency of vibration increases with bridge stiffness.

- The vibration approach and equipment developed for this project show potential for assessing rural steel and concrete bridges; however, new techniques and appropriate equipment need to be developed to adequately measure the vibration. The frequency range for the concrete bridges evaluated exceeded the available vibration capacity of the forcing motor used in this testing.

Additional studies should utilize field instrumentation that can clearly identify first bending mode frequencies with real-time data processing tools and include automated control and data acquisition. Testing should also be conducted on dowel laminated bridge structures, which represent over 1,200 bridges in Minnesota.

Chapter 1
Introduction

The use of wood in timber bridges has many benefits including the fact that wood is a renewable and sustainable resource, timber bridges are often more economical than steel and concrete bridges, and they can be installed easily in rural environments. There are currently over 41,000 bridges in service with a span of over 20 ft with an average age of 40 years old (FHWA 2002). This represents 7 percent of the bridges reported in the National Bridge Inventory. Recent programs like the USDA Wood In Transportation Program have funded research to develop a new class of timber bridges and associated inspection techniques. Recently the Federal Highway Administration (FHWA 2002) expanded the usage of federal “preventative maintenance” funds to include state and local bridges.

Wood is a natural occurring engineering material that is prone to deterioration caused by decay fungi and insect attack. For this reason, it is important to conduct frequent inspections of timber bridges with modern inspection equipment. As noted in the USDA Timber Bridge Manual, “Bridge members infected with decay fungi experience progressive strength loss as the fungi develop and degrade the wood structure. The degree of strength reduction depends on the area of the infection and the stage of decay development, whether advanced, intermediate, or incipient. In the advanced or intermediate stages, wood deterioration has progressed to the point where no strength remains in infected areas. At this stage, suitable detection methods can be used by the inspector to accurately define the affected areas with some degree of certainty. At the incipient or early stages of development, detection is much more difficult and the effect of strength loss varies among types of fungi.” It is important to identify early stage decay to ensure the safety of the structure and allow for treatment in service.

Background discussions with Mn/DOT bridge inspection program managers and the St. Louis County bridge engineer revealed that current timber inspection procedures in Minnesota are limited to visual inspection of the wood components, sounding with a hammer, and coring to confirm suspected damage areas. These techniques have proved adequate for advanced decay detection, but are not adequate when the damage is in the early stage or is located internally in the members. All inspections are completed by evaluating individual components of these bridges, including pilings, pile caps, girders, decking, and railings. Use of advanced techniques like stress wave timing, moisture meters, and resistance drills will significantly improve the reliability of the inspections but these inspection techniques are time consuming.

Deterioration, one of the most common damage mechanisms in wood 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 (Morison et al 2002, Morison 2003, Peterson et al 2003, Wang et al 2005) 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 et al 2002, Ross et al 2002). In these studies, a series of laboratory-constructed wood floor systems 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 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 some joists in laboratory floor settings (Soltis et al 2002). 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 (Wang et al in press).

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

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

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 1993) showed that a general form for the natural frequency for any mode can be derived as

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

where f_i is 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

$$EI = \left(\frac{1}{kg}\right) f_i^{\,2} W L^3 \tag{3}$$

where f_1 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.

Research Objectives

The objective of this project will be conduct vibration testing of timber, steel and concrete bridges in northeastern Minnesota to determine flexural frequency characteristics. Recent collaborative research between the UMD NRRI, Michigan Technological University, and the USDA Forest Products Laboratory has developed vibration testing techniques for short span, simply supported timber bridges. This research showed that both forced and free vibration could be used as rapid inspection techniques to determine the stiffness of the bridge and the corresponding overall condition.

The focus of this research was to use these vibration testing techniques and load testing on an additional 9+ timber bridges of varying ages and designs to develop a data set for use in future commercialization and technology transfer activities. We plan to critically assess the testing techniques used in Michigan and adapt them for vibration testing in Minnesota. Further, we want to investigate the feasibility of the testing equipment and techniques for use on short span steel and concrete bridges.

Chapter 2
Bridges Tested

Table 2.1. List of bridges tested with brief summary of each bridge.

St. Louis County Bridge Number	Material Summary	Number of Spans	Year Constructed
85	Heavy timber pilings, Douglas fir girders/stringers, wood/asphalt deck	2	1946
153	Heavy timber pilings, Douglas fir girders/stringers, wood deck	1	1943
242	Heavy timber southern yellow pine pilings, Douglas fir girders/stringers, wood deck	2	1944
305	Heavy timber southern yellow pine pilings, Douglas fir girders/stringers, wood deck	2	1940
357	Heavy timber southern yellow pine pilings, Douglas fir girders/stringers, wood deck	2	1940
619	Heavy pine timber pilings, Douglas fir girders/stringers, wood deck	1	Unknown
726	Heavy timber southern yellow pine pilings, Douglas fir girders/stringers, wood deck	2	1944
CR-1	Heavy pine timber pilings, Douglas fir girders/stringers, wood deck	1	1960
CR-2	Heavy pine timber pilings, Douglas fir girders, wood deck	1	1960

Chapter 3
Procedures

Visual Inspection

The simplest method for locating deterioration is visual inspection. An inspector observes the structure for signs of actual or potential deterioration, noting areas that require further investigation. When assessing the condition of a structure, visual inspection should never be the sole method used. Visual inspection requires strong light and is useful for detecting intermediate or advanced surface decay, water damage, mechanical damage, or failed members. Visual inspection cannot detect early stage decay, when remedial treatment is most effective. During an inspection the following signs of deterioration were investigated:

- Fruiting bodies
- Sunken faces and localized collapse
- Staining or discoloration
- Insect activity
- Plant and moss growth
- Missing members
- Checks and splits
- Alterations
- Loose or missing connections

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 (ASTM 2000). Moisture content data were collected at pin penetrations of 2 in. (51 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 (Pfaff and Garrahan 1984).

Stress Wave Timing

An example of the stress wave concept for detecting decay within a rectangular wood member is shown in Figure 3.1. First, a stress wave is induced by striking the specimen with an impact device that is instrumented with an accelerometer that emits a start signal to a timer. A second accelerometer, which is held in contact with the other side of the specimen, serves to the leading edge of the propagating stress wave and sends a stop signal to the timer. The elapsed time for the stress wave to propagate between the accelerometers is displayed on the timer. All commercially available timing units, if calibrated and operated according to the manufacturer's recommendations, yield comparable results. The use of stress wave velocity to detect wood decay in timber bridges and other structures is limited only by access to the structural members under consideration. It is especially useful on thick timbers 89 mm (3.5 in.) where hammer sounding is not effective. A detailed explanation of the use of stress wave timing and interpretation of the testing is detailed in publications prepared by Brashaw et al (2005).

A Fakopp Microsecond Timer was used to determine the stress wave time across the piling 6 inches above the water line, at 6 inches below the pile top and at a point midway between these two measurements. The Fakopp was also used to test the pile caps at several locations, starting on one end continuing along its length at locations between the stringers. It was also used to test the girders 12 inches away from the end above the abutment.

The Fakopp is very accurate at determining the presence of decay at the testing location and is useful in mapping the decay locations. Table 3.1 shows the stress wave transmission times perpendicular to the grain for several species at various degradation levels for the species present in the timber bridges.

Table 3.1. Stress wave transmission times perpendicular to the grain with various levels of degradation using the Fakopp Microsecond Timer.

	Stress Wave Transmission Times (microseconds/ft)			
Species	Sound Wood	Moderate Decay	Severe Decay	Splits
Douglas fir	130-260	300-400	500+	300-700
Southern yellow pine	220-250	300-400	500+	300-700

Figure 3.1. Fakopp microsecond timer being used on a timber pile.

Resistance Microdrilling

Resistance microdrilling was used to identify and quantify decay, voids, and insect galleries in wood beams, columns, poles, and piles. The resistance drill system measures the resistance of wood members to a 1.5-mm drill bit with a 3.0-mm head that passes through them. The drill bit is fed at a fixed movement rate allowing the inspector to determine the exact location and extent of the damaged area. This system produces a chart showing the relative resistance over its travel path. This chart can be produced either as a direct printout or can be downloaded to a computer. Areas of sound wood have varying levels of resistance depending on the density of the species and voids show no resistance. The inspector can determine areas of low, mild, and high levels of decay. A detailed explanation of the use of resistance microdrilling and interpretation of the testing is detailed in publications prepared by Brashaw et al (2005).

In areas of concern noted during visual and stress wave inspections, the IML F-300 resistance drill was used to test the cross-section and determine the resistance of the wood to a small diameter drill bit. An example of the Fakopp testing is shown in Figure 3.1 and the resistance drilling testing in Figure 3.2.

The resistance drilling unit was very accurate at determining the presence of decay at the drilling location. It measures the resistance on a 0-100% amplitude scale. Typical measures of resistance for sound softwoods are > 25%, 10-20% for moderate decay and 0-10% for advanced or severe decay.

Figure 3.2. Example of resistance drill testing.

Vibration Test Procedures for Timber Bridges

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 used because it eliminates the need for modal analysis. An electric motor with a rotating unbalanced wheel was used to excite the structure, 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. Three piezoelectric accelerometers (PCB 626BO2), also at midspan, were 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 as captured using a Fluke digital scopemeter. The specific testing steps included:

1. Secure a dc motor (1/2 horsepower) with rotating unbalanced wheel to the deck plank at the center of the bridge span and anchor using steel bolt screws.
2. Secure two magnetic metal plates to deck plank, one on each side of the bridge at midspan using steel bolt screws.
3. Attach one piezoelectric accelerometer to each magnetic metal plate to monitor the bridge vibration signals.
4. Start up motor. Slowly increase motor speed to put the bridge into low frequency transverse vibration.
5. Find the first bending mode frequency by locating the first local maximum response acceleration using a digital scopemeter.
6. Record the data and start a second test.

Once all of the tests have been completed, the data is reviewed briefly and photographs are taken before leaving the test site. The typical test setup can be seen in Figure 3.3.

Figure 3.3. General test setup for vibration testing.

Static Load Testing

Because the primary goal of this work is to 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 tests were conducted at each field bridge using a live load testing method. A test vehicle was placed on 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. The test vehicle consisted of a fully loaded, tri-axle gravel truck. The gross vehicle weight and individual axle weights were measured for each truck used prior to testing. The axle spacing was also measured for each truck. 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 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. The specific static load test steps include:

Initial Assessment of Bridge Condition

1. Look for any signs of major distress in any of the support girders and load carrying beams.
2. Inspect top and bottom of road using visual and (hammer) sounding methods.
3. Look closely at abutments and pier supports to ensure that cap beams should be resting squarely on piles).
4. Do not conduct bridge test if safety is concern -- Use traffic control as appropriate!

Placement of Optical Surveying Level

1. Look for a suitable location on solid ground, not on soft soils or muck.
2. Ensure that the technician has a good view of all centerspan rulers, including the closest and farthest sight distances.
3. The height of instrument should be as high as possible, but not higher than 1 foot from top of deflection rulers.
4. Ensure that the operator has sight of four corners of each span tested to measure for possible vertical support movements.

Load Test Setup

1. Start with *UNDERSIDE* measurement of the bridge.
2. Measure (face-face) support span lengths along both edges of bridge, then place mark or nail at midspan location.
3. Measure beam, plank dimensions, and note any unusual repairs.
4. Measure all support bearing lengths for the abutment cap and pier cap beams.
5. Measure (center-center) spacing of all bridge beams at centerspan x-section.
6. Attach brackets and rulers near the center of each beam along the centerspan x-section.
7. Attach brackets and rulers near the support corners (and near centerline for wide bridges) of the span and make sure level instrument can read them ok.
8. Using a plumb bob, transfer the midspan x-section to the deck or curbs.
9. Continue with *TOPSIDE* measurements of the bridge.

10. Measure bridge (out-out) width over planks, and note any overhang at edges.
11. Mark the bridge centerline by using ½ of the bridge width (out-out).
12. Measure the bridge length (out-out) at topside, including all support bearings.
13. Mark truck locations (this point will bisect the rear axles and will be directly between the rear dual wheels) with crayon and paint.
    a. For single lane bridge, use center loading (with wheel lines straddling roadway centerline) with marks at 3 ft on each side of centerline.
    b. For double lane bridge, position truck in each lane in addition to above center loading. Place additional marks 2 ft on each side of centerline.
14. Measure & record truck axle spacing and weights.
15. Commence load test.

<u>Typical Sequence</u>

1. Position truck.
2. Take photograph.
3. Take deflection readings.
4. Check survey level bubble to ensure level.
5. Repeat sequence until testing is complete.

Once all of the data has been collected, it is reviewed briefly before the truck leaves for home. Photographs of the bridge, both end and side views, are then taken with no people, vehicles, or anything else on the bridge. The main components of the setup for load testing can be seen in Figures 3.4, 3.5, and 3.6.

Figure 3.4. Truck positioned on bridge.

Figure 3.5. Deflection rulers.

Figure 3.6. Optical surveying level.

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 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 the 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.

Chapter 4
Individual Bridge Testing Summary

Bridge 85 Testing Summary

Background

Structure:	Bridge 85
Location:	County Road 258, Duluth, Minnesota
Special Consideration(s):	None
Estimated age:	1946
Inspection date:	July 2005
Construction details:	Two span; heavy timber pilings with Douglas fir girders/stringers and a wood/asphalt deck for a running surface

Bridge Photos:

Figure 4.1. Bridge 85.

Figure 4.2. Bridge 85.

Figure 4.3. Bridge 85.

Vibration Test Data

The vibration testing data for Bridge 85, spans 1 and 2 are shown listed in Tables 4.1 and 4.2.

Table 4.1. Vibration data collected from Bridge 85 for span 1.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
39.4	24.35	18.36	0	1519	25.3	40	25
			0	1571	26.2	38.8	25.7

Note:
- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Table 4.2. Vibration data collected from Bridge 85 for span 2.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
18.52	24.3	17.52	1694	28.2	36	27.7	0
			1659	27.7	36.8	27.2	1000
			1706	28.4	35.6	28.1	2000
			1767	29.5	34.8	28.7	2000

Note:
- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Static Load Test Data

The static load testing data for Bridge 85, spans 1 and 2 are shown listed in Tables 4.3 and 4.4.

Table 4.3. Static load data collected from Bridge 85 for span 1.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1			Load Case No. 2			Load Case No. 3		
				Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)
1	1467	1468	1467.5	1465	-2.5	-0.1	1468	0.5	0.02	1468	0.5	0.02
2	1460	1460	1460	1457	-3	-0.12	1460	0	0	1460	0	0
3	1454	1454	1454	1444	-10	-0.39	1453	-1	-0.04	1451	-3	-0.12
4	1450	1451	1450.5	1442	-8.5	-0.33	1450	-0.5	-0.02	1443	-7.5	-0.3
5	1447	1447	1447	1438	-9	-0.35	1444	-3	-0.12	1438	-9	-0.35
6	1452	1453	1452.5	1448	-4.5	-0.18	1443	-9.5	-0.37	1443	-9.5	-0.37
7	1456	1456	1456	1455	-1	-0.04	1447	-9	-0.35	1447	-9	-0.35
8	1457	1453	1455	1453	-2	-0.08	1443	-12	-0.47	1449	-6	-0.24
9	1457	1457	1457	1457	0	0	1448	-9	-0.35	1456	-1	-0.04
10	1449	1449	1449	1449	0	0	1445	-4	-0.16	1449	0	0
A	1450	1450	1450	1449	-1	-0.04	1450	0	0	1450	0	0
B	1461	1460	1460.5	1462	1.5	0.06	1460	-0.5	-0.02	1462	1.5	0.06
C	1485	1486	1485.5	1485	-0.5	-0.02	1485	-0.5	-0.02	1485	-0.5	-0.02
D	1435	1436	1435.5	1437	1.5	0.06	1435	-0.5	-0.02	1436	0.5	0.02

Note:

- mm = millimeters
- in = inches
- Load Case No. 1 (upstream) and Load Case No. 2 (downstream) was 2 ft centerline, rear axles at midspan, and front axle off span.
- Load Case No. 3 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 10 (downstream)
- Truck Weights:
  - Gross Vehicle Weight = 64,560 lbs, Rear Axle Vehicle Weight = 45,700 lbs

Table 4.4. Static load data collected from Bridge 85 for span 2.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1			Load Case No. 2			Load Case No. 3		
				Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)
1	1475	1475	1475	1472	-3	-0.12	1476	1	0.04	1476	1	0.04
2	1462	1462	1462	1451	-11	-0.43	1462	0	0	1461	-1	-0.04
3	1454	1454	1454	1444	-10	-0.39	1454	0	0	1451	-3	-0.12
4	1449	1449	1449	1438	-11	-0.43	1448	-1	-0.04	1438	-11	-0.43
5	1448	1448	1448	1436	-12	-0.47	1444	-4	-0.16	1437	-11	-0.43
6	1447	1446	1446.5	1443	-3.5	-0.14	1443	-3.5	-0.14	1436	-10.5	-0.41
7	1443	1443	1443	1443	0	0	1434	-9	-0.35	1432	-11	-0.43
8	1448	1447	1447.5	1448	0.5	0.02	1437	-10.5	-0.41	1443	-4.5	-0.18
9	1448	1447	1447.5	1448	0.5	0.02	1438	-9.5	-0.37	1446	-1.5	-0.06
10	1437	1448	1442.5	1438	-4.5	-0.18	1433	-9.5	-0.37	1438	-4.5	-0.18
A	1487	1487	1487	1486	-1	-0.04	1488	1	0.04	1487	0	0
B	1439	1440	1439.5	1439	-0.5	-0.02	1438	-1.5	-0.06	1440	0.5	0.02
C	1470	1469	1469.5	1468	-1.5	-0.06	1469	-0.5	-0.02	1470	0.5	0.02
D	1444	1443	1443.5	1444	0.5	0.02	1441	-2.5	-0.1	1444	0.5	0.02

Note:

- mm = millimeters
- in = inches
- Load Case No. 1 (upstream) and Load Case No. 2 (downstream) was 2 ft centerline, rear axles at midspan, and front axle off span.
- Load Case No. 3 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 10 (downstream)
- Truck Weights:
  - Gross Vehicle Weight = 64,560 lbs, rear Axle Vehicle Weight = 45,700 lbs

Bridge 85 Results

Results obtained from the inspection are shown for each component type of the bridge in the following sections.

Decking

The decking was inspected from the underside since the running surface was asphalt paved. The deck was noted to be in good condition.

Railings, Rail Posts and Curbs

Visual inspection noted these components in good condition. No stress wave timing or resistance drilling was conducted.

Pilings

The pilings were in fair condition with numerous splits that have been strapped for added support; the timing data shows no major concerns with the exception of the bottom of piles 3, 7 and 9.

Pile Caps

The pile caps were not solid timber but laminated lumber. Therefore, timing data was not performed due to the timbers not being solid timber. Visual inspection noted that pile cap A and C are out of plum due to the abutments pushing inward.

Girders

Deterioration was not noted in any of the girders ends.

Vibration Frequency

There was an average frequency of vibration of 27.9 Hertz (Hz) for span 1 and 25.3 Hz for span 2.

Live Load

Figures 4.4 and 4.5 show the deflections of spans 1 and 2 of Bridge 85 for center loading.

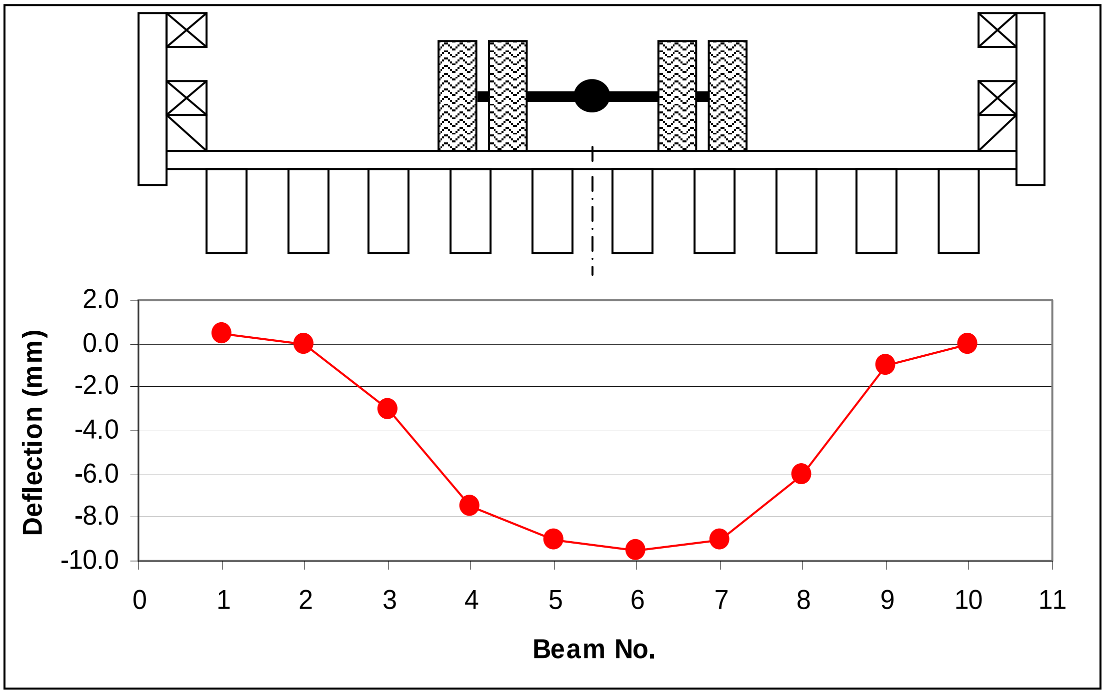

Figure 4.4. Deflection of span 1 of Bridge 85 for center loading.

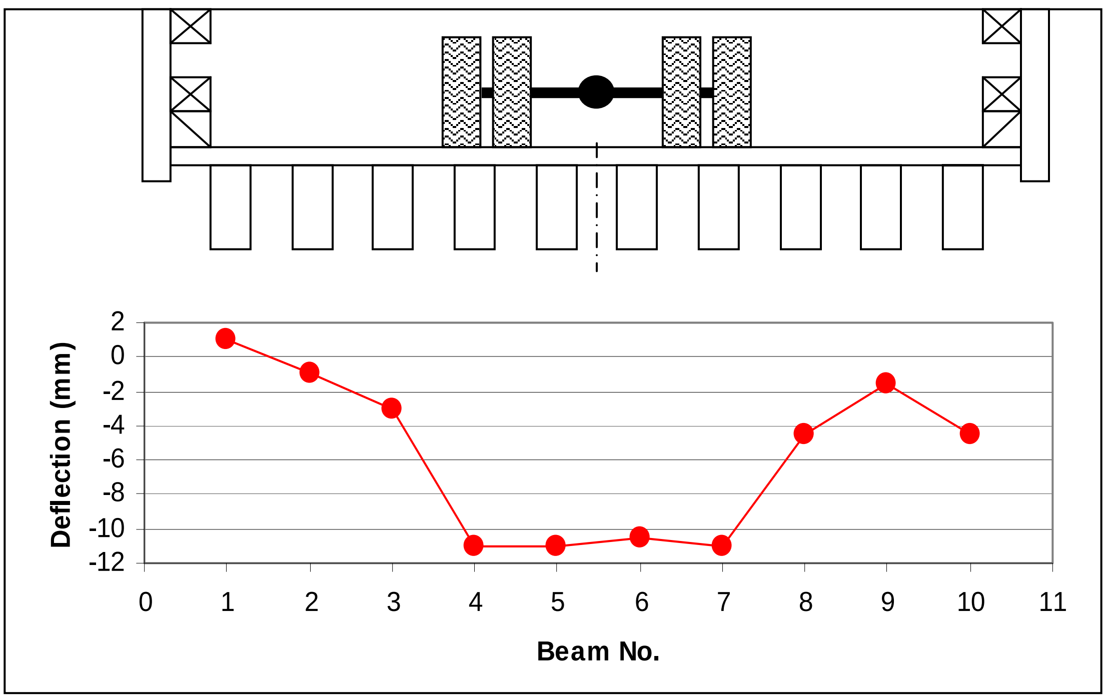

Figure 4.5. Deflection of span 2 of Bridge 85 for center loading.

Bridge 153 Testing Summary

Background

Structure:	Bridge 153
Location:	County Road 840, Floodwood, Minnesota
Special Consideration(s):	None
Estimated age:	1943
Inspection date:	July 2005
Construction details:	Single span; heavy timber pilings with Douglas fir girders/stringers and a wood deck for a running surface

Bridge Photos:

Figure 4.6. Bridge 153.

Figure 4.7. Bridge 153.

Figure 4.8. Bridge 153.

Vibration Test Data

The vibration testing data for Bridge 153, span 1 is shown listed in Table 4.5.

Table 4.5. Vibration data collected from Bridge 153 for span 1.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
19.83	24.083		0	1710	28.5	35.4	28.2
			0	1678	28	36.2	27.6
			0	1618	27	37.4	26.7

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Static Load Test Data

The static load testing data for Bridge 153, span 1 is shown listed in Table 4.6.

Table 4.6. Static load data collected from Bridge 153 for span 1.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1			Load Case No. 2			Load Case No. 3		
				Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)
1	824	823	823.5	821	-2.5	-0.1	824	0.5	0.02	824	0.5	0.02
2	813	813	813	808	-5	-0.2	813	0	0	812	-1	-0.04
3	804	803	803.5	796	-7.5	-0.3	803	-0.5	-0.02	800	-3.5	-0.14
4	804	804	804	798	-6	-0.24	803	-1	-0.04	799	-5	-0.2
5	794	793	793.5	787	-6.5	-0.26	791	-2.5	-0.1	786	-7.5	-0.3
6	774	773	773.5	767	-6.5	-0.26	769	-4.5	-0.18	765	-8.5	-0.33
7	775	774	774.5	771	-3.5	-0.14	767	-7.5	-0.3	766	-8.5	-0.33
8	765	764	764.5	763	-1.5	-0.06	756	-8.5	-0.33	756	-8.5	-0.33
9	765	764	764.5	764	-0.5	-0.02	756	-8.5	-0.33	759	-5.5	-0.22
10	765	764	764.5	764	-0.5	-0.02	757	-7.5	-0.3	761	-3.5	-0.14
11	766	765	765.5	766	0.5	0.02	759	-6.5	-0.26	765	-0.5	-0.02
12	759	758	758.5	759	0.5	0.02	756	-2.5	-0.1	759	0.5	0.02
A	780	780	780	779	-1	-0.04	780	0	0	780	0	0
B	769	768	768.5	769	0.5	0.02	767	-1.5	-0.06	769	0.5	0.02
C	877	876	876.5	874	-2.5	-0.1	877	0.5	0.02	877	0.5	0.02
D	762	762	762	762	0	0	761	-1	-0.04	762	0	0

Note:

- mm = millimeters
- in = inches
- Load Case No. 1 (upstream) and Load Case No. 2 (downstream) was 2 ft centerline, rear axles at midspan, and front axle off span.
- Load Case No. 3 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 12 (downstream)
- Data Point A upstream left corner, B downstream left corner, C upstream right corner, D downstream right corner
- Truck Weights: Gross Vehicle Weight = 44,340 lbs, Rear Axle Weight = 31, 620 lbs

Bridge 153 Results

Results obtained from the inspection are shown for each component type of the bridge in the following sections.

Decking

The decking appeared to be in good condition. Some of the decking fasteners have become loose and should be replaced. Decay was not evident. No stress wave timing or resistance drilling was conducted.

Railings, Rail Posts, and Curbs

Visual inspection noted these components in good condition. No stress wave timing or resistance drilling was conducted.

Pilings

Pile numbers 2 and 3 were not measured. The piles all showed high microsecond time data with pile 9 having the highest reading of 1640 microseconds (μs). Piles 9 and 10 were not in contact with the pile cap.

Pile Caps

The pile caps both showed acceptable stress time data and moisture content.

Girders

Deterioration was not noted in any of the ends of the girders. Girder number 3 showed higher deflection during the live load test.

Vibration Frequency

The span had average frequency of vibration of 27.5 Hz with no dead load.

Live Load

Figure 4.9 shows the deflection of span 1 of Bridge 153 for center loading.

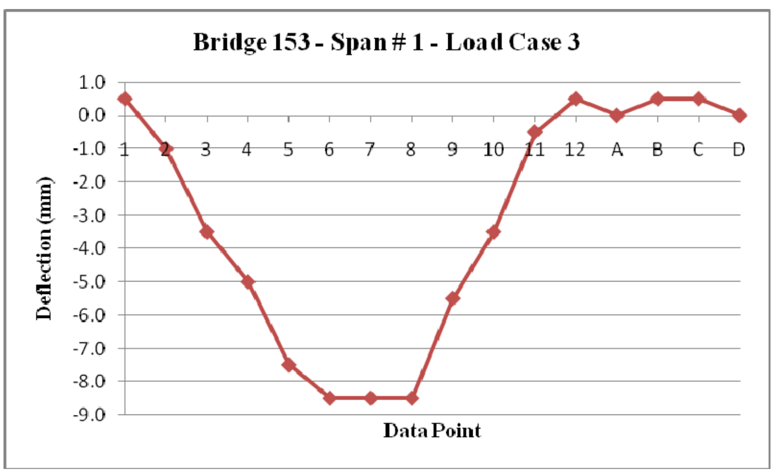

Figure 4.9. Deflection of span 1 of Bridge 153 for center loading.

Bridge 242 Testing Summary

Background

Structure:	Bridge 242
Location:	County Road 211, Meadowlands, Minnesota
Special Consideration(s):	The bridge was rebuilt in January, 2006
Estimated age:	1944
Inspection date:	July 2005
Construction details:	Two span; heavy timber southern yellow pine pilings with Douglas fir girders/stringers and a wood deck for a running surface

Bridge Photos:

Figure 4.10. Bridge 242.

Figure 4.11. Bridge 242.

Figure 4.12. Bridge 242.

Vibration Test Data

The vibration testing data for Bridge 242, spans 1 and 2 are shown listed in Tables 4.7 and 4.8.

Table 4.7. Vibration data collected from Bridge 242 for span 1.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
37.95	24.2	17.6	0	1363	22.7	44.8	22.3
			0	1323	22.1	45.6	21.9
			0	1469	24.5	46	21.7

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Table 4.8. Vibration data collected from Bridge 242 for span 2.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
18.52	24.3	17.6	0	1329	22.2	45.6	21.9
			0	1298	21.6	46.4	21.6
			0	1326	22.1	46	21.7

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Static Load Test Data

The static load testing data for Bridge 242, spans 1 and 2 are shown listed in Tables 4.9 and 4.10.

Table 4.9. Static load data collected from Bridge 242 for span 1.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1			Load Case No. 2			Load Case No. 3		
				Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)
1	1871	1871	1871	1865	-6	-0.24	1871	0	0	1871	0	0
2	1867	1868	1867.5	1859	-8.5	-0.33	1867	-0.5	-0.02	1866	-1.5	-0.06
3	1868	1868	1868	1860	-8	-0.31	1867	-1	-0.04	1864	-4	-0.16
4	1879	1879	1879	1871	-8	-0.31	1878	-1	-0.04	1873	-6	-0.24
5	1872	1872	1872	1865	-7	-0.28	1870	-2	-0.08	1865	-7	-0.28
6	1881	1881	1881	1876	-5	-0.2	1877	-4	-0.16	1874	-7	-0.28
7	1872	1871	1871.5	1869	-2.5	-0.1	1865	-6.5	-0.26	1864	-7.5	-0.3
8	1877	1877	1877	1876	-1	-0.04	1869	-8	-0.31	1870	-7	-0.28
9	1877	1876	1876.5	1876	-0.5	-0.02	1869	-7.5	-0.3	1872	-4.5	-0.18
10	1874	1874	1874	1874	0	0	1866	-8	-0.31	1872	-2	-0.08
11	1864	1864	1864	1864	0	0	1857	-7	-0.28	1863	-1	-0.04
12	1869	1869	1869	1869	0	0	1866	-3	-0.12	1869	0	0
A	1877	1877	1877	1874	-3	-0.12	1877	0	0	1877	0	0
B	1870	1870	1870	1870	0	0	1867	-3	-0.12	1870	0	0
C	1869	1868	1868.5	1865	-3.5	-0.14	1868	-0.5	-0.02	1868	-0.5	-0.02
D	1873	1873	1873	1873	0	0	1872	-1	-0.04	1873	0	0

Note:

- mm = millimeters, in = inches
- Readings are in mm; Deflections are listed in both mm and in.
- Load Case No. 1 (upstream) and Load Case No. 2 (downstream) was 2 ft centerline, rear axles at midspan, and front axle off span.
- Load Case No. 3 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 12 (downstream)
- Data Point A upstream left corner, B downstream left corner, C upstream right corner, D downstream right corner
- Truck Weight: Gross Vehicle Weight = 44,340 lbs, Rear Axle Weight = 31,620 l

Table 4.10. Static load data collected from Bridge 242 for span 2.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1			Load Case No. 2			Load Case No. 3		
				Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)
1	1886	1885	1885.5	1883	-2.5	-0.1	1886	0.5	0.02	1886	0.5	0.02
2	1893	1893	1893	1887	-6	-0.24	1984	91	3.58	1893	0	0
3	1892	1892	1892	1885	-7	-0.28	1892	0	0	1890	-2	-0.08
4	1896	1895	1895.5	1889	-6.5	-0.26	1895	-0.5	-0.02	1890	-5.5	-0.22
5	1899	1899	1899	1892	-7	-0.28	1897	-2	-0.08	1891	-8	-0.31
6	1901	1900	1900.5	1894	-6.5	-0.26	1897	-3.5	-0.14	1893	-7.5	-0.3
7	1899	1898	1898.5	1895	-3.5	-0.14	1892	-6.5	-0.26	1891	-7.5	-0.3
8	1895	1894	1894.5	1894	-0.5	-0.02	1887	-7.5	-0.3	1887	-7.5	-0.3
9	1899	1898	1898.5	1898	-0.5	-0.02	1890	-8.5	-0.33	1894	-4.5	-0.18
10	1900	1899	1899.5	1900	0.5	0.02	1891	-8.5	-0.33	1897	-2.5	-0.1
11	1894	1893	1893.5	1894	0.5	0.02	1887	-6.5	-0.26	1893	-0.5	-0.02
12	1892	1891	1891.5	1892	0.5	0.02	1888	-3.5	-0.14	1892	0.5	0.02
A	1875	1874	1874.5	1872	-2.5	-0.1	1874	-0.5	-0.02	1875	0.5	0.02
B	1873	1873	1873	1873	0	0	1872	-1	-0.04	1873	0	0
C	1915	1915	1915	1913	-2	-0.08	1914	-1	-0.04	1912	-3	-0.12
D	1912	1911	1911.5	1912	0.5	0.02	1910	-1.5	-0.06	1912	0.5	0.02

Note:

- mm = millimeters
- in = inches
- Readings are in mm; Deflections are listed in both mm and in.
- Load Case No. 1 (upstream) and Load Case No. 2 (downstream) was 2 ft centerline, rear axles at midspan, and front axle off span.
- Load Case No. 3 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 12 (downstream)
- Data Point A upstream left corner, B downstream left corner, C upstream right corner, D downstream right corner
- Truck Weight: Gross Vehicle Weight = 44,340 lbs, Rear Axle Weight = 31,620 lbs

Bridge 242 Results

Results obtained from the inspection are shown for each component type of the bridge in the following sections.

Decking

The decking appeared to be in good condition. No major fastener damage was noted. Decay was not evident. No stress wave timing or resistance drilling was conducted.

Railings, Rail Posts and Curbs

Visual inspection noted these components in good condition. No stress wave timing or resistance drilling was conducted.

Pilings

Pile numbers 5 and 6 showed presence of substantial decay as noted by stress wave timing data. Piling number 6 was visually broken. Pile numbers 7, 8, 10, and 11 appear to have significant decay present at the top of the pile in contact with the pile cap. They should be further investigated on site if the pile cap is removed or repaired.

Pile Caps

Pile cap A shows the presence of decay at a distance between 8 and 18 ft from the upstream end. Pile cap B (at bridge center pile support) showed significant decay along the entire length and should be considered for replacement. Resistance drilling data shows that there is severe internal decay present in the pile cap. At the upstream end, there is an internal void present of 4 inches, increasing to 11 inches at 17 ft. At the downstream end, the pile cap is in good condition.

Girders

Deterioration was not noted in any of the girders ends. We did not have access to the girders above the center piles and pile cap.

Vibration Frequency

There was a frequency of vibration of 21.9 Hertz (Hz) for Span 1 and 21.5 Hz for Span 2.

Live Load

Figures 4.13 and 4.14 show the deflections of spans 1 and 2 of Bridge 242 for center loading.

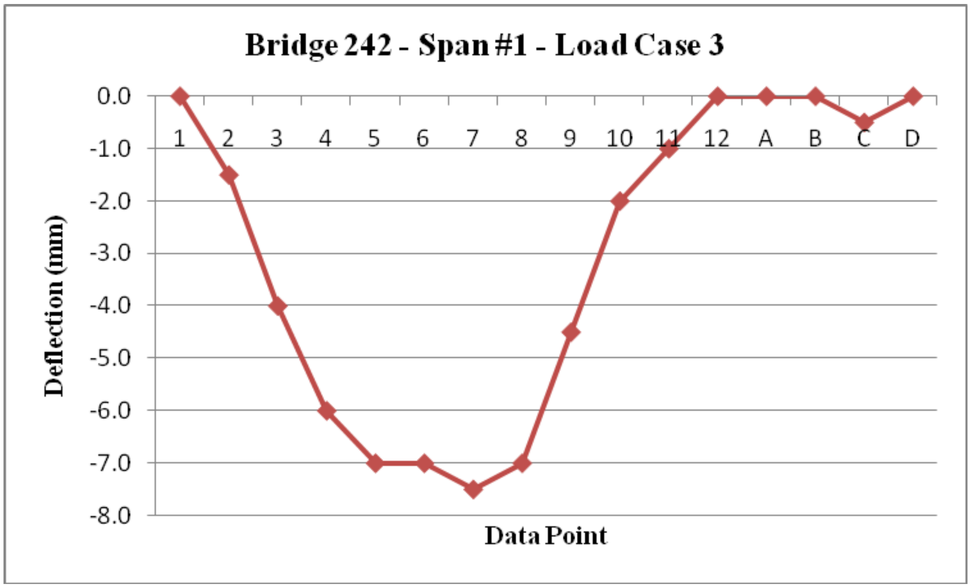

Figure 4.13. Deflection of span 1 of Bridge 242 for center loading.

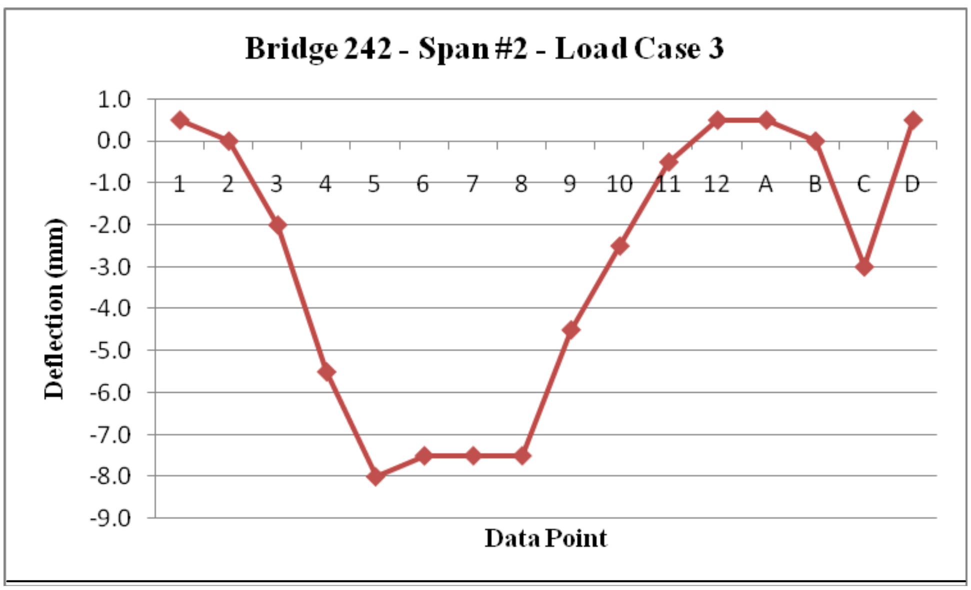

Figure 4.14. Deflection of span 2 of bridge 242 for center loading.

Bridge 305 Testing Summary

Background

Structure:	Bridge 305
Location:	County Road 936, North of Virginia, Minnesota
Special Consideration(s):	Bridge is skewed across waterway.
Estimated age:	1940
Inspection date:	July 2005
Construction details:	Two span; heavy timber southern yellow pine pilings with Douglas fir girders/stringers and a wood deck for a running surface

Bridge Photos:

Figure 4.15. Bridge 305.

Figure 4.16. Bridge 305.

Vibration Test Data

The vibration testing data for Bridge 305, spans 1 and 2 are shown listed in Tables 4.11 and 4.12.

Table 4.11. Vibration data collected from Bridge 305 for span 1.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
60	24	20.83	0	1888	31.5	318	31.4
			0	1990	30.4	304	32.9
			0	1899	31.7	320	31.3

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Table 4.12. Vibration data collected from Bridge 305 for span 2.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
60	24	21.33	0	2096	34.9	292	34.2
			0	2109	35.2	286	35

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Static Load Test Data

The static load testing data for Bridge 305, spans 1 and 2 are shown listed in Tables 4.13 and 4.14.

Table 4.13. Static load data collected from Bridge 305 for span 1.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1			Load Case No. 2			Load Case No. 3		
				Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)
1	1161	1160	1160.5	1159	-1.5	-0.06	1160	-0.5	-0.02	1161	0.5	0.02
2	1172	1171	1171.5	1168	-3.5	-0.14	1172	0.5	0.02	1172	0.5	0.02
3	1176	1175	1175.5	1170	-5.5	-0.22	1176	0.5	0.02	1175	-0.5	-0.02
4	1182	1181	1181.5	1175	-6.5	-0.26	1182	0.5	0.02	1180	-1.5	-0.06
5	1192	1192	1192	1186	-6	-0.24	1192	0	0	1188	-4	-0.16
6	1197	1197	1197	1191	-6	-0.24	1196	-1	-0.04	1191	-6	-0.24
7	1212	1211	1211.5	1206	-5.5	-0.22	1210	-1.5	-0.06	1206	-5.5	-0.22
8	1218	1218	1218	1214	-4	-0.16	1215	-3	-0.12	1212	-6	-0.24
9	1226	1226	1226	1224	-2	-0.08	1221	-5	-0.2	1220	-6	-0.24
10	1234	1233	1233.5	1232	-1.5	-0.06	1228	-5.5	-0.22	1228	-5.5	-0.22
11	1246	1246	1246	1246	0	0	1240	-6	-0.24	1242	-4	-0.16
12	1255	1255	1255	1255	0	0	1249	-6	-0.24	1252	-3	-0.12
13	1263	1263	1263	1263	0	0	1256	-7	-0.28	1261	-2	-0.08
14	1275	1274	1274.5	1274	-0.5	-0.02	1269	-5.5	-0.22	1273	-1.5	-0.06
15	1277	1276	1276.5	1276	-0.5	-0.02	1273	-3.5	-0.14	1276	-0.5	-0.02
A	1188	1186	1187	1185	-2	-0.08	1188	1	0.04	1188	1	0.04
B	1340	1340	1340	1340	0	0	1338	-2	-0.08	1340	0	0
C	1142	1142	1142	1141	-1	-0.04	1142	0	0	1142	0	0
D	1217	1216	1216.5	1216	-0.5	-0.02	1214	-2.5	-0.1	1216	-0.5	-0.02
E	1317	1317	1317	1316	-1	-0.04	1313	-4	-0.16	1315	-2	-0.08
F	1282	1282	1282	1280	-2	-0.08	1279	-3	-0.12	1278	-4	-0.16
G	1246	1246	1246	1242	-4	-0.16	1247	1	0.04	1243	-3	-0.12

Table 4.14. Static load data collected from Bridge 305 for span 2.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1			Load Case No. 2			Load Case No. 3		
				Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)
1	1139	1138	1138.5	1137	-1.5	-0.06	1139	0.5	0.02	1139	0.5	0.02
2	1132	1133	1132.5	1130	-2.5	-0.1	1133	0.5	0.02	1133	0.5	0.02
3	1133	1133	1133	1129	-4	-0.16	1134	1	0.04	1133	0	0
4	1135	1134	1134.5	1131	-3.5	-0.14	1134	-0.5	-0.02	1133	-1.5	-0.06
5	1136	1136	1136	1133	-3	-0.12	1136	0	0	1134	-2	-0.08
6	1133	1132	1132.5	1128	-4.5	-0.18	1132	-0.5	-0.02	1128	-4.5	-0.18
7	1136	1136	1136	1133	-3	-0.12	1135	-1	-0.04	1132	-4	-0.16
8	1137	1137	1137	1135	-2	-0.08	1135	-2	-0.08	1133	-4	-0.16
9	1139	1139	1139	1137	-2	-0.08	1134	-5	-0.2	1134	-5	-0.2
10	1141	1141	1141	1140	-1	-0.04	1136	-5	-0.2	1137	-4	-0.16
11	1147	1147	1147	1146	-1	-0.04	1141	-6	-0.24	1143	-4	-0.16
12	1152	1152	1152	1152	0	0	1147	-5	-0.2	1150	-2	-0.08
13	1155	1155	1155	1155	0	0	1149	-6	-0.24	1154	-1	-0.04
14	1163	1163	1163	1163	0	0	1159	-4	-0.16	1163	0	0
15	1169	1169	1169	1169	0	0	1167	-2	-0.08	1169	0	0
A	1127	1127	1127	1126	-1	-0.04	1128	1	0.04	1128	1	0.04
B	1196	1195	1195.5	1195	-0.5	-0.02	1193	-2.5	-0.1	1195	-0.5	-0.02
C	1132	1133	1132.5	1131	-1.5	-0.06	1132	-0.5	-0.02	1133	0.5	0.02
D	1141	1141	1141	1141	0	0	1140	-1	-0.04	1141	0	0

Note:

- mm = millimeters, in = inches
- Load Case No. 1 (upstream) and Load Case No. 2 (downstream) was 2 ft centerline, rear axles at midspan, and front axle off span.
- Load Case No. 3 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 15 (downstream)
- Data Point A upstream left corner, B downstream left corner, C upstream right corner, D downstream right corner
- Truck Weight: Gross Vehicle Weight = 50,280 lbs, Rear Axle Weight = 36,860 lbs

Bridge 305 Results

Results obtained from the inspection are shown for each component type of the bridge in the following sections.

Decking

The decking appeared to be in good condition. Some fasteners showed damage from snowplowing. Decay was not evident. No stress wave timing or resistance drilling was conducted.

Railings, Rail Posts and Curbs

Visual inspection noted these components in good condition. No stress wave timing or resistance drilling was conducted.

Pilings

The pilings all showed high microsecond time data with pile numbers 1 and 2 being severely decayed; cross bracing of the pilings have deteriorated to the point where it has no function. Several of the pilings should be further investigated to determine replacement.

Pile Caps

Pile cap A showed good microsecond timing data. Pile cap B showed very high timing data beginning at the 4-foot upstream end to the 24-foot mark on the downstream end. Pile cap C showed high timing data on the upstream end for 10 feet and for 6 feet on the downstream end.

Girders

Many of the girder ends were not accessible for close inspection, but appeared to be in good condition. The ends above the center support should be inspected while the river is frozen.

Vibration Frequency

There was a no load frequency of vibration of 31.8 Hertz (Hz) for span 1 and 34.6 Hz for span 2

Live Load

Figures 4.17 and 4.18 show the deflections of spans 1 and 2 of Bridge 305 for center loading.

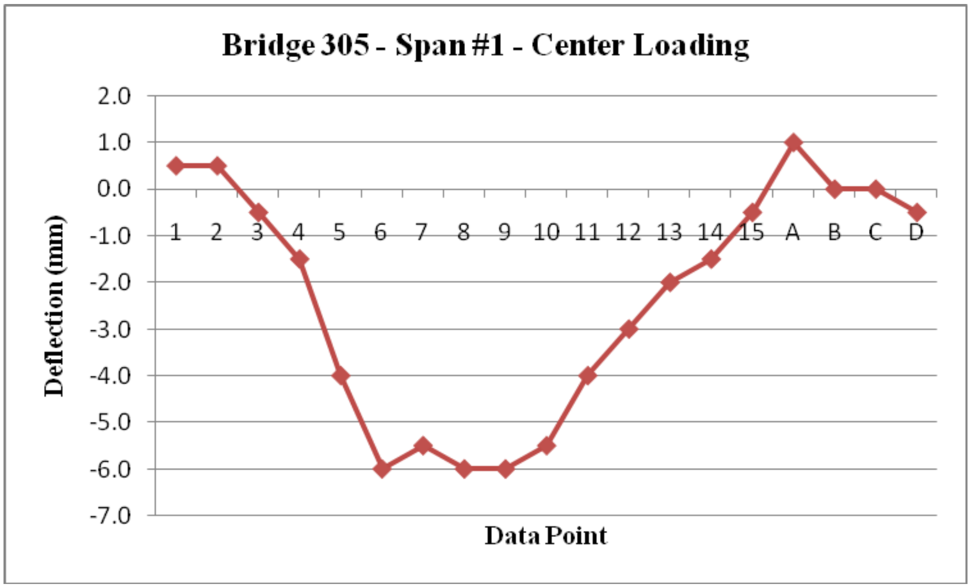

Figure 4.17. Deflection of span 1 of Bridge 305 for center loading.

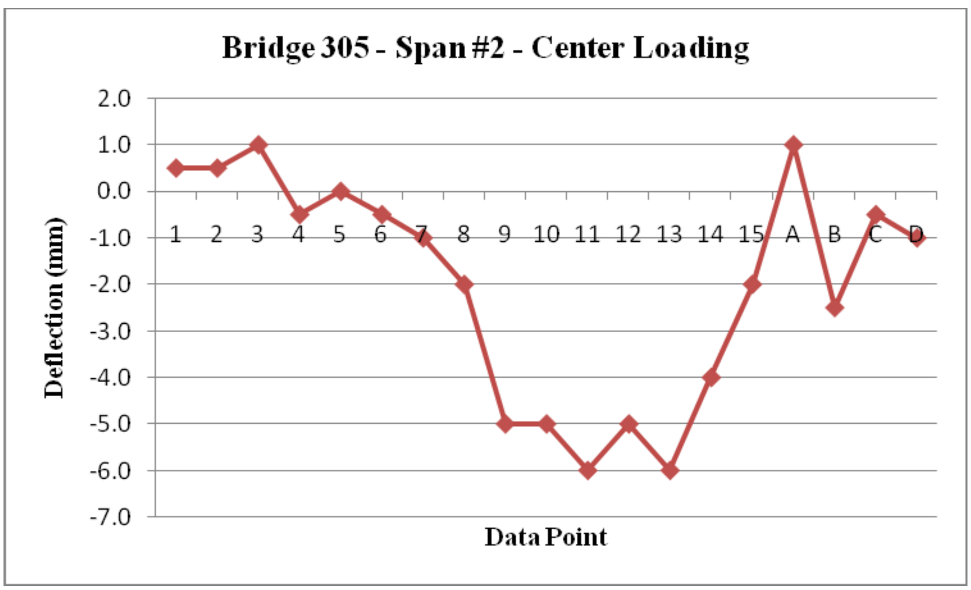

Figure 4.18. Deflection of span 2 of Bridge 305 for center loading.

Bridge 357 Testing Summary

Background

Structure:	Bridge 357
Location:	County Road 652, North of Virginia, Minnesota
Special Consideration(s):	None
Estimated age:	1940
Inspection date:	July 2005
Construction details:	Two span; heavy timber southern yellow pine pilings with Douglas fir girders/stringers and a wood deck for a running surface

Bridge Photos:

Figure 4.19. Bridge 357.

Figure 4.20. Bridge 357.

Figure 4.21. Bridge 357.

Vibration Test Data

The vibration testing data for Bridge 357, spans 1 and 2 are shown listed in Tables 4.15 and 4.16.

Table 4.15. Vibration data collected from Bridge 357 for span 1.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
27	24	12.17	0	1120	18.7	540	18.5
			0	1125	18.8	540	18.5

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Table 4.16. Vibration data collected from Bridge 357 for span 2.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
27	24	12.17	0	1069	17.8	568	17.6
			0	1096	18.3	552	18.1
			0	1042	17.36	584	17.1

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Static Load Test Data

The static load testing data for Bridge 357, spans 1 and 2 are shown in Tables 4.17 and 4.18.

Table 4.17. Static load data collected from Bridge 357 for span 1.

Data Point Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Center Loading Reading (mm)	Center Loading Deflection (mm)	Center Loading Deflection (in.)
1	849	849	849.0	849	0.0	0.00
2	840	842	841.0	841	0.0	0.00
3	838	837	837.5	838	0.5	0.02
4	836	836	836.0	834	-2.0	-0.08
5	832	832	832.0	828	-4.0	-0.16
6	833	833	833.0	827	-6.0	-0.24
7	834	834	834.0	827	-7.0	-0.28
8	833	833	833.0	827	-6.0	-0.24
9	834	833	833.5	827	-6.5	-0.26
10	834	834	834.0	827	-7.0	-0.28
11	831	831	831.0	825	-6.0	-0.24
12	832	832	832.0	830	-2.0	-0.08
13	833	834	833.5	833	-0.5	-0.02
14	833	833	833.0	832	-1.0	-0.04
15	838	838	838.0	839	1.0	0.04
A	847	847	847.0	847	0.0	0.00
B	831	831	831.0	832	1.0	0.04
C	845	845	845.0	845	0.0	0.00
D	844	845	844.5	845	0.5	0.02
E	829	829	829.0	828	-1.0	-0.04
F	845	845	845.0	849	4.0	0.16

Table 4.18. Static load data collected from Bridge 357 for span 2.

Data Point Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Center Loading Reading (mm)	Center Loading Deflection (mm)	Center Loading Deflection (in.)
1	831	831	831.0	832	1.0	0.04
2	838	838	838.0	838	0.0	0.00
3	837	836	836.5	835	-1.5	-0.06
4	838	838	838.0	835	-3.0	-0.12
5	834	834	834.0	830	-4.0	-0.16
6	831	831	831.0	824	-7.0	-0.28
7	836	836	836.0	830	-6.0	-0.24
8	837	837	837.0	830	-7.0	-0.28
9	835	834	834.5	827	-7.5	-0.30
10	832	832	832.0	825	-7.0	-0.28
11	829	829	829.0	824	-5.0	-0.20
12	831	831	831.0	828	-3.0	-0.12
13	835	835	835.0	834	-1.0	-0.04
14	837	837	837.0	838	1.0	0.04
15	830	830	830.0	830	0.0	0.00
A	832	832	832.0	833	1.0	0.04
B	829	829	829.0	829	0.0	0.00
C	846	845	845.5	846	0.5	0.02
D	841	841	841.0	841	0.0	0.00
E	840	837	838.5	839	0.5	0.02
F	837	840	838.5	836	-2.5	-0.10

Note:

- mm = millimeters
- in = inches
- Truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 15 (downstream)
- Truck Weight: Gross Vehicle Weight = 50,280 lbs, Rear Axle Weight = 36,860 lbs

Bridge 357 Results

Results obtained from the inspection are shown for each component type of the bridge in the following sections.

Decking

The decking appeared to be in good condition. Some fasteners showed damage from snowplowing. Decay was not evident. No stress wave timing or resistance drilling was conducted.

Railings, Rail Posts and Curbs

Visual inspection noted these components in good condition. No stress wave timing or resistance drilling was conducted.

Pilings

Piles number 6, 7, 8, and 10 showed the presence of decay as noted by stress wave timing data. Piling number 7 shows very high timing data and should be further investigated.

Pile Caps

Pile cap A showed the presence of decay at a distance between 4 and 20 ft from the upstream end. Pile cap B (at bridge center support) showed decay and insect damage in the center section of the pile cap. Pile cap C timing data showed the timber in good condition.

Girders

Deterioration was not noted in any of the girders ends and they were noted to be in good condition.

Vibration Frequency

Span 1 had a no load frequency of vibration of 18.5 Hertz (Hz) for span 1 and 17.6 Hz for span 2.

Live Load

Figures 4.22 and 4.23 show the deflections of spans 1 and 2 of Bridge 357 for center loading.

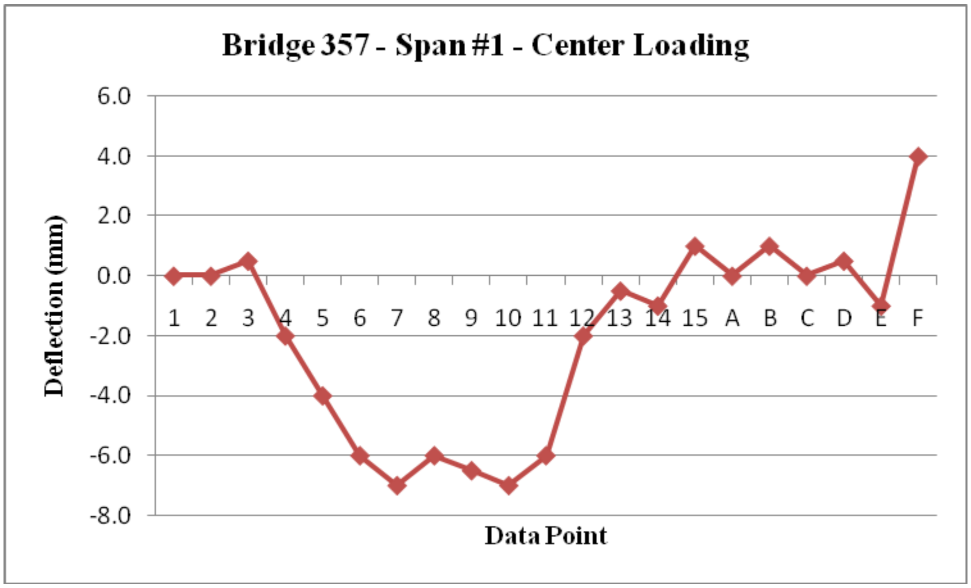

Figure 4.22. Deflection of span 1 of Bridge 357 for center loading.

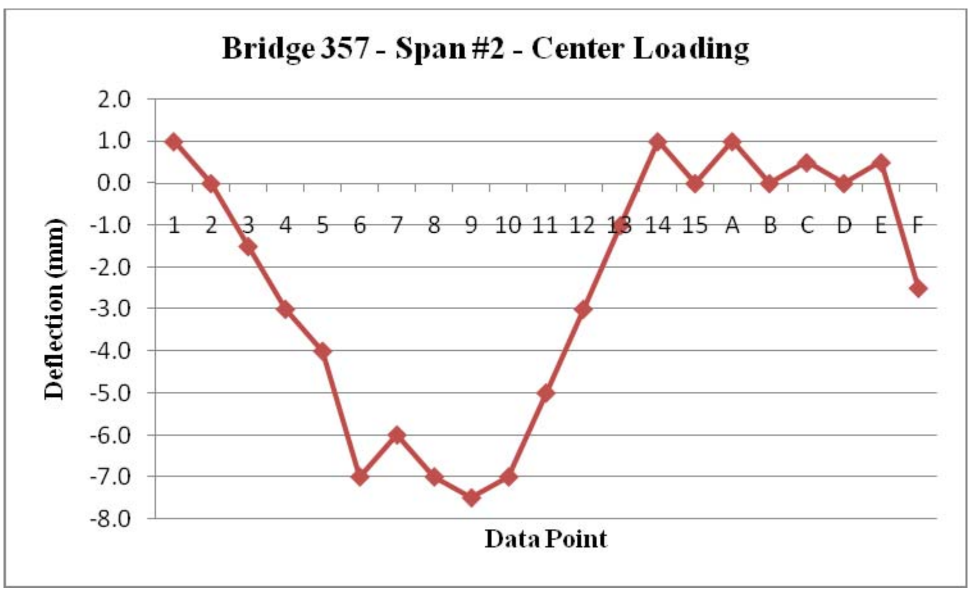

Figure 4.23. Deflection of span 2 of Bridge 357 for center loading.

Bridge 619 Testing Summary

Background

Structure: Bridge 619

Location: County Road 66, North of Kinney, Minnesota

Special Consideration(s): None

Estimated age: Unknown

Inspection date: June 2005

Construction details: Single span, heavy timber pine pilings with Douglas fir girders/stringers and a wood deck for a running surface

Bridge Photos:

Figure 4.24. Bridge 619.

Figure 4.25. Bridge 619.

Figure 4.26. Bridge 619.

Vibration Test Data

The vibration testing data for Bridge 619, span 1 is shown listed in Table 4.19.

Table 4.19. Vibration data collected from Bridge 619 for span 1.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
17.9	18.1	16	0	1353	22.55	44.8	22.32
			0	1348	22.46	45.2	22.12

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Static Load Test Data

The static load testing data for Bridge 619, span 1 is shown listed in Table 4.20.

Table 4.20. Static load data collected from Bridge 619 for span 1.

Data Point Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Center Loading Reading (mm)	Center Loading Deflection (mm)	Center Loading Deflection (in.)
1	606	606	606.0	601	-5.0	-0.20
2	607	607	607.0	599	-8.0	-0.31
3	608	603	608.0	598	-10.0	-0.39
4	593	592	592.5	583	-9.5	-0.37
5	597	596	596.5	584	-12.5	-0.49
6	588	587	587.5	579	-8.5	-0.33
7	591	591	591.0	583	-8.0	-0.31
8	588	587	587.5	579	-8.5	-0.33
9	376	375	375.5	367	-8.5	-0.33
10	387	387	387.0	383	-4.0	-0.16
11	591	590	590.5	588	-2.5	-0.10
12	592	592	592.0	592	0.0	0.00
13	597	597	597.0	598	1.0	0.04

Note:

- mm = millimeters
- in = inches
- Load Case No. 1 was with truck straddling centerline, rear axles at midspan, and front axle off bridge.
- Readings are in mm; Deflections are in both mm and in.
- Data Point 1 (downstream) and Data Point 13 (upstream)
- Truck Weight: Gross Vehicle Weight = 46,540 lbs, Rear Axle Weight = 33,680 lbs

Bridge 619 Results

Results obtained from the inspection are shown for each component type of the bridge in the following sections.

Decking

The decking appeared to be in good condition. Some fasteners showed damaged from snowplowing. Decay was not evident. No stress wave timing or resistance drilling was conducted.

Railings, Rail Posts and Curbs

The railings are constructed with 2 ½ inch steel angle.

Pilings

Due to the high water at the time of testing, pilings # 1, 2, 3 and 4 were not tested with the microsecond timer but were visually inspected. There is decay and shell damage visible, and pilings 4 through 8 are considered partially decayed with microsecond time readings in the 500 to 600 μs range.

Pile Caps

Pile caps A and B could not be measured with the microsecond timer due to the fact that only one side was accessible. They were visually inspected and appeared to be in good condition. Hammer sounding was satisfactory.

Girders

Girders 4 and 5 are 24 inches in depth and are notched for proper top side alignment, and are in good condition. Girder # 1 (upstream) has a split along its entire length and decay is present on both ends. Girders 7, 8, and 12 showed high timing data on one end

Vibration Frequency

Span 1 had a no load frequency of vibration of 22.2 Hertz (Hz.)

Live Load

Figure 4.27 shows the deflection of span 1 of Bridge 619 for center loading.

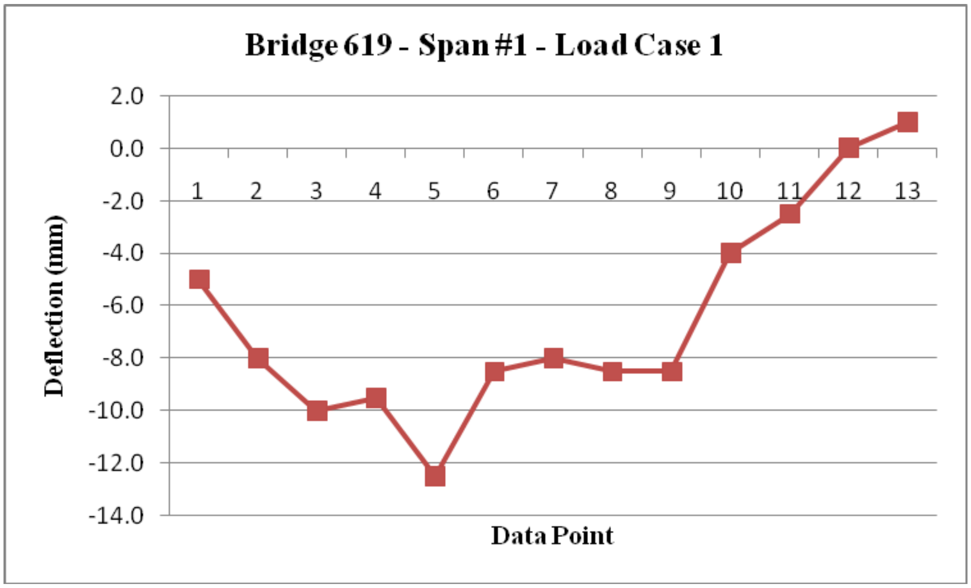

Figure 4.27. Deflection of span 1 of Bridge 619 for center loading.

Bridge 726 Testing Summary

Background

Structure:	Bridge 726
Location:	County Road 469, NW of Chisholm, MN.
Special Consideration(s):	Bridge was repaired after our testing was completed
Estimated age:	1944
Inspection date:	July 2005
Construction details:	Two span; heavy timber southern yellow pine pilings with Douglas fir girders/stringers and a wood deck for a running surface

Bridge Photos:

Figure 4.28. Bridge 726.

Figure 4.29. Bridge 726.

Figure 4.30. Bridge 726.

Vibration Test Data

The vibration testing data for Bridge 726, spans 1 and 2 are shown listed in Tables 4.21 and 4.22.

Table 4.21. Vibration data collected from Bridge 726 for span 1.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
19.36	24.35	18.36	0	1884	31.4	32.4	30.9
			0	1804	30.1	33.8	29.6
			0	1898	31.6	33	30.3
			1000	1786	29.8	34	29.4
			1000	1844	30.7	33	30.3
			1000	1818	30.3	33.8	29.6

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Table 4.22. Vibration data collected from Bridge 726 for span 2.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
18.52	24.3	17.52	0	1935	32.3	31.6	31.6
			0	1935	32.3	31.2	32.1
			0			31	32.3

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Static Load Test Data

The static load testing data for Bridge 726, spans 1 and 2 are shown listed in Tables 4.23 and 4.24.

Table 4.23. Static load data collected from Bridge 726 for span 1.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1			Load Case No. 2			Load Case No. 3		
				Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)
1	605	605	605	602	-3	-0.12	606	1	0.04	606	1	0.04
2	606	606	606	600	-6	-0.24	606	0	0	605	-1	-0.04
3	609	608	608.5	600	-8.5	-0.33	608	-0.5	-0.02	605	-3.5	-0.14
4	605	605	605	596	-9	-0.35	604	-1	-0.04	599	-6	-0.24
5	603	603	603	593	-10	-0.39	601	-2	-0.08	594	-9	-0.35
6	610	609	609.5	600	-9.5	-0.37	606	-3.5	-0.14	600	-9.5	-0.37
7	599	598	598.5	592	-6.5	-0.26	593	-5.5	-0.22	590	-8.5	-0.33
8	607	606	606.5	602	-4.5	-0.18	598	-8.5	-0.33	597	-9.5	-0.37
9	615	615	615	613	-2	-0.08	607	-8	-0.31	607	-8	-0.31
10	614	613	613.5	613	-0.5	-0.02	605	-8.5	-0.33	607	-6.5	-0.26
11	620	619	619.5	620	0.5	0.02	611	-8.5	-0.33	616	-3.5	-0.14
12	625	625	625	625	0	0	617	-8	-0.31	622	-3	-0.12
13	624	624	624	624	0	0	619	-5	-0.2	623	-1	-0.04
14	638	637	637.5	638	0.5	0.02	635	-2.5	-0.1	638	0.5	0.02

Note:

- mm = millimeters
- in = inches
- Load Case No. 1 (upstream) and Load Case No. 2 (downstream) was 2 ft centerline, rear axles at midspan, and front axle off span.
- Load Case No. 3 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 14 (downstream)
- Truck Weight: Gross Vehicle Weight = 44,860 lbs, Rear Axle Weight = 31,720 lbs

Table 4.24. Static load data collected from Bridge 726 for span 2.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1			Load Case No. 2			Load Case No. 3		
				Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)	Reading (mm)	Deflection (mm)	Deflection (in)
1	704	703	703.5	700	-3.5	-0.14	704	0.5	0.02	704	0.5	0.02
2	712	712	712	706	-6	-0.24	712	0	0	711	-1	-0.04
3	710	710	710	703	-7	-0.28	710	0	0	709	-1	-0.04
4	709	709	709	702	-7	-0.28	709	0	0	705	-4	-0.16
5	708	708	708	702	-6	-0.24	708	0	0	703	-5	-0.2
6	704	703	703.5	698	-5.5	-0.22	702	-1.5	-0.06	697	-6.5	-0.26
7	711	710	710.5	705	-5.5	-0.22	707	-3.5	-0.14	704	-6.5	-0.26
8	712	712	712	709	-3	-0.12	707	-5	-0.2	706	-6	-0.24
9	716	715	715.5	714	-1.5	-0.06	709	-6.5	-0.26	709	-6.5	-0.26
10	715	715	715	714	-1	-0.04	709	-6	-0.24	710	-5	-0.2
11	725	724	724.5	724	-0.5	-0.02	718	-6.5	-0.26	721	-3.5	-0.14
12	717	717	717	717	0	0	710	-7	-0.28	715	-2	-0.08
13	721	720	720.5	721	0.5	0.02	715	-5.5	-0.22	720	-0.5	-0.02
14	709	709	709	709	0	0	706	-3	-0.12	710	1	0.04

Note:

- mm = millimeters
- in = inches
- Load Case No. 1 (upstream) and Load Case No. 2 (downstream) was 2 ft centerline, rear axles at midspan, and front axle off span.
- Load Case No. 3 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 14 (downstream)
- Truck Weight: Gross Vehicle Weight = 44,860 lbs, Rear Axle Weight = 31,720 lbs

Bridge 726 Results

Results obtained from the inspection are shown for each component type of the bridge in the following sections.

Decking

The decking appeared to be in good condition. No major fastener damage was noted. Decay was not evident. No stress wave timing or resistance drilling was conducted.

Railings, Rail Posts and Curbs

Visual inspection noted these components in good condition. No stress wave timing or resistance drilling was conducted.

Pilings

Piles number 5 and 6 showed presence of substantial decay as noted by stress wave timing data. Piling number 6 is visually broken. Piles number 7, 8, 10, and 11 appear to have significant decay present at the top of the pile in contact with the pile cap. They should be further investigated on site if the pile cap is removed or repaired.

Pile Caps

Pile cap A shows the presence of decay at a distance between 8 and 18 ft from the upstream end. Pile cap B (at bridge center pile support) showed significant decay along the entire length. Resistance drilling data shows that there was severe internal decay present in the pile cap. At the upstream end, there is an internal void present of 4 inches, increasing to 11 inches at 17 ft. At the downstream end, the pile cap is in good condition.

Girders

Deterioration was not noted in any of the girders ends. We did not have access to the girders above the center piles and pile cap.

Vibration Frequency

There was a frequency of vibration of 21.9 Hertz (Hz) for span 1 and 21.5 Hz for span 2.

Live Load

Figures 4.31 and 4.32 show the deflections of spans 1 and 2 of Bridge 726 for center loading.

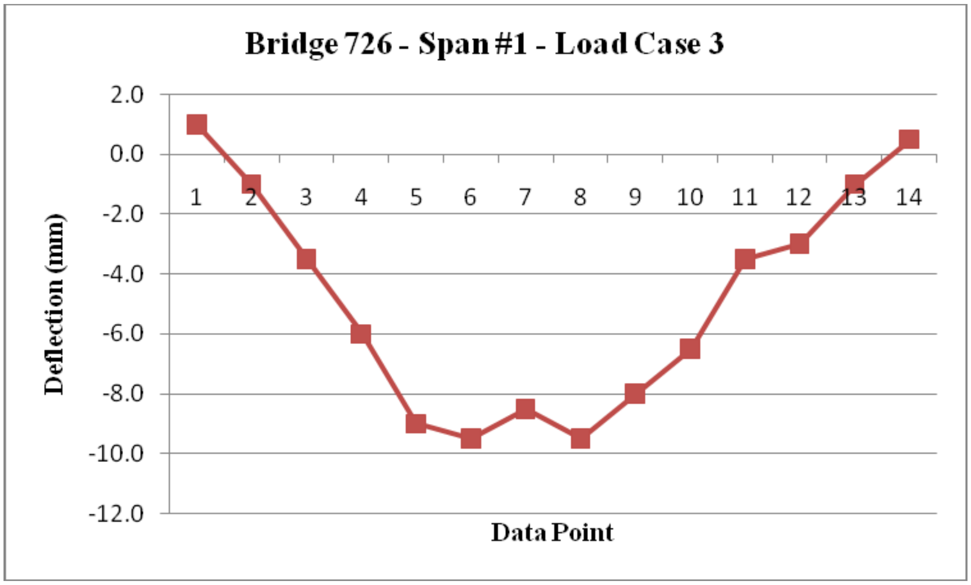

Figure 4.31. Deflection of span 1 of Bridge 726 for center loading.

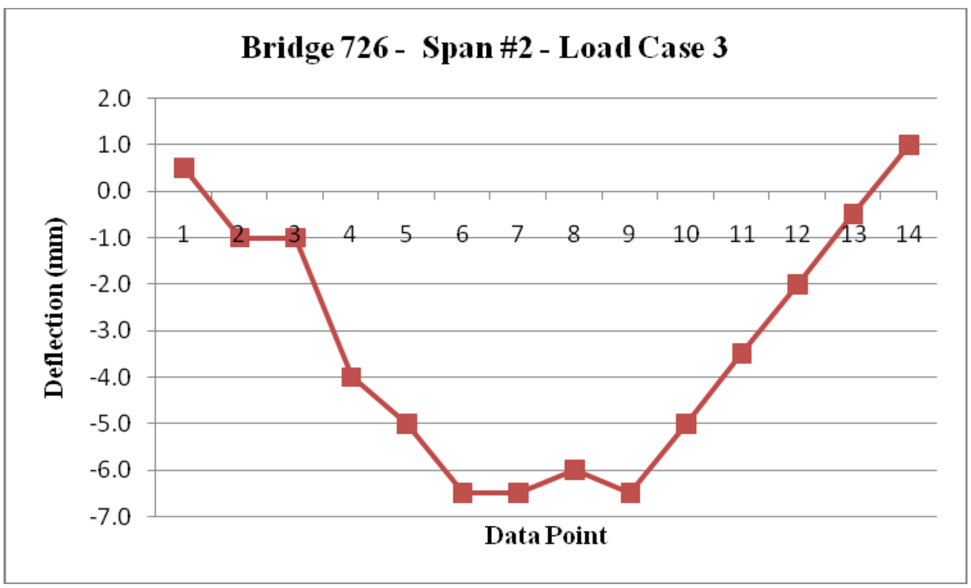

Figure 4.32. Deflection of span 2 of Bridge 726 for center loading.

Bridge CR-1 Testing Summary

Background

Structure:	Cloquet River Bridge (structure # L1950)
Location:	USFS Road # 102 13.8 Miles SW of Jct. TH 1
Special Consideration(s):	None
Estimated age:	1960
Inspection date:	September 2005
Construction details:	Single span, heavy pine timber pilings with Douglas fir girders/stringers and a wood deck for a running surface

Bridge Photos:

Figure 4.33. Bridge CR-1.

Figure 4.34. Bridge CR-1.

Vibration Test Data

The vibration testing data for Bridge CR-1, span 1 is shown listed in Table 4.25.

Table 4.25. Vibration data collected from Bridge CR-1 for span 1.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
20	15	17.17	0	1289	21.48	472	21.18
		17.33	0	1273	21.21	476	21
			0	1309	21.81	464	21.55

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Static Load Test Data

The static load testing data for Bridge CR-1, span 1 is shown listed in Table 4.26.

Table 4.26. Static load data collected from Bridge CR-1 for span 1.

Data Point Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Center Loading		
				Reading (mm)	Deflection (mm)	Deflection (in.)
1	503	503	503.0	500	-3.0	-0.12
2	500	500	500.0	495	-5.0	-0.20
3	507	506	506.5	499	-7.5	-0.30
4	495	495	495.0	485	-10.0	-0.39
5	499	499	499.0	491	-8.0	-0.31
6	503	503	503.0	496	-7.0	-0.28
7	505	504	504.5	497	-7.5	-0.30
8	507	507	507.0	500	-7.0	-0.28
9	511	511	511.0	506	-5.0	-0.20
10	514	514	514.0	512	-2.0	-0.08
A	440	440	440.0	438	-2.0	-0.08
B	442	442	442.0	441	-1.0	-0.04
C	567	567	567.0	566	-1.0	-0.04
D	585	585	585.0	584	-1.0	-0.04

Note:

- mm = millimeters
- in = inches
- Load Case No. 1 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 10 (downstream)
- Truck Weight: Gross Vehicle Weight = 45,900 lbs, Rear Axle Weight = 29,500 lbs

Bridge CR-1 Results

Results obtained from the inspection are shown for each component type of the bridge in the following sections.

Decking

The decking appeared to be in good condition. Some fastener damage was noted. Decay was not evident. No stress wave timing or resistance drilling was conducted.

Railings, Rail Posts and Curbs

Visual inspection noted the rail posts are in good condition and the railings boards need replacement.

Pilings

The piles were in good condition with the exception of some high microsecond timing data at the centers of pilings # 3, 10, 13 and 14. They should be further investigated on site if the pile cap is removed or repaired.

Pile Caps

Pile cap A showed the presence of significant decay due to old bolt holes in the timber, which allowed water to enter. Decay was present throughout its entire length and it should be considered for replacement. Resistance drilling data shows that there is severe internal decay present in the pile cap. Pile cap B is in good condition with no evidence of decay.

Girders

Deterioration was not noted in any of the girders.

Vibration Frequency

Span 1 had a no load frequency of vibration 21.2 Hertz (Hz.)

Live Load

Figure 4.35 shows the deflection of span 1 of Bridge CR-1 for center loading.

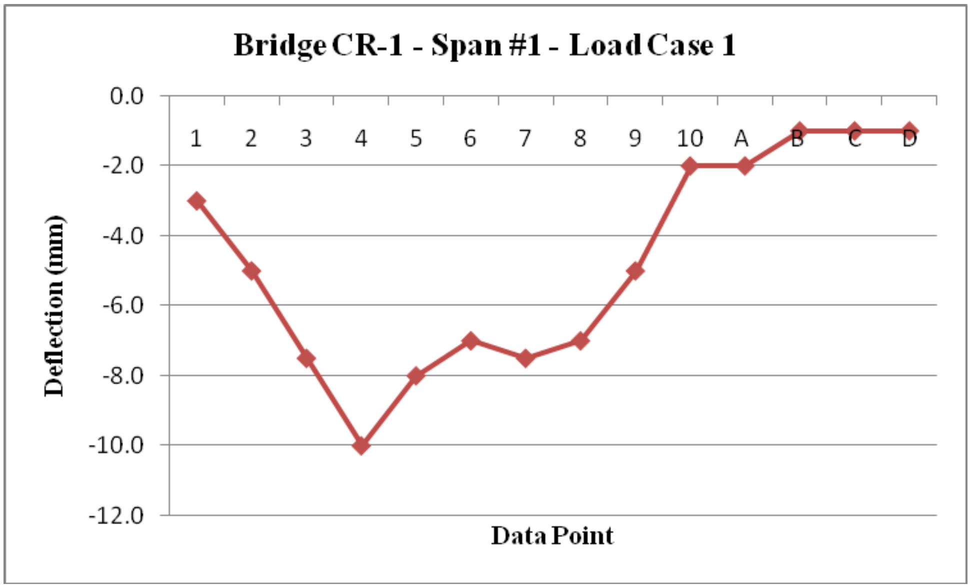

Figure 4.35. Deflection of span 1 of Bridge CR-1 for center loading.

Bridge CR-2 Testing Summary

Background

Structure:	Cloquet River Bridge
Location:	14.2 Miles SW. of JCT. 1 (Cloquet River Road)
	Forest Service Road # 102
Special Consideration(s):	None
Estimated age:	1960
Inspection date:	September 2005
Construction details:	Single span, heavy pine timber pilings with Douglas fir girders and a wood deck for a running surface

Bridge Photos:

Figure 4.36. Bridge CR-2.

Figure 4.37. Bridge CR-2.

Vibration Test Data

The vibration testing data for Bridge CR-2, span 1 is shown listed in Table 4.27.

Table 4.27. Vibration data collected from Bridge CR-2 for span 1.

				Motor		Vibration Test	
Length (ft)	**Width (ft)**	**C-C Span (ft)**	**Dead load (lb)**	**Speed (rpm)**	**Frequency (Hz)**	**Time/cycle (ms)**	**Frequency (Hz)**
20.42	15	16.3	0	1163	19.38	524	19.08
		17.0	0	1145	19.08	532	18.79
			0	1148	19.13	528	18.93

Note:

- ft = feet, lb = pounds, rpm = revolutions per minute, Hz = hertz, ms = milliseconds

Static Load Test Data

The static load testing data for Bridge CR-2, span 1 is shown listed in Table 4.28.

Table 4.28. Static load data collected from Bridge CR-2 for span 1.

Data Point Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Center Loading		
				Reading (mm)	Deflection (mm)	Deflection (in.)
1	830	830	830.0	828	-2.0	-0.08
2	829	825	827.0	821	-6.0	-0.24
3	800	800	800.0	792	-8.0	-0.31
4	825	824	824.5	815	-9.5	-0.37
5	824	823	823.5	815	-8.5	-0.33
6	818	816	817.0	809	-8.0	-0.31
7	818	821	819.5	811	-8.5	-0.33
8	820	820	820.0	813	-7.0	-0.28
9	818	815	816.5	810	-6.5	-0.26
10	821	821	821.0	821	0.0	0.00
A	830	829	829.5	828	-1.5	-0.06
B	822	825	823.5	822	-1.5	-0.06
C	839	835	837.0	833	-4.0	-0.16
D	820	820	820.0	821	1.0	0.04

Note:

- mm = millimeters
- in = inches
- Load Case No. 1 was truck straddling centerline, rear axles at midspan, and front axle off span.
- Data Point 1 (upstream) and Data Point 10 (downstream)
- Truck Weight: Gross Vehicle Weight = 45,900 lbs, Rear Axle Weight = 29,500 lbs

Bridge CR-2 Results

Results obtained from the inspection are shown for each component type of the bridge in the following sections.

Decking

The decking appeared to be in good condition. Some fasteners need to be replaced. Decay was not evident. No stress wave timing or resistance drilling was conducted.

Railings, Rail Posts and Curbs

Visual inspection noted these components need repair, as the 2 ½" x 7" railing is damaged and should be replaced on both sides of the bridge.

Pilings

The pilings are in good condition with the exception of the top section of pile # 1, which had a high microsecond time reading, and the center sections of piles # 4 and 14.

Pile Caps

Pile cap A showed the presence of substantial decay near the center of its length due to a split along its length. Resistance drilling could not be done at the time due to the high water level. Pile cap B microsecond timing data showed the timber to be in good condition.

Girders

Deterioration was not noted in any of the girders.

Vibration Frequency

Span 1 had a no load frequency of vibration of 18.9 Hertz (Hz.)

Live Load

Figure 4.38 shows the deflection of span 1 of Bridge CR-2 for center loading.

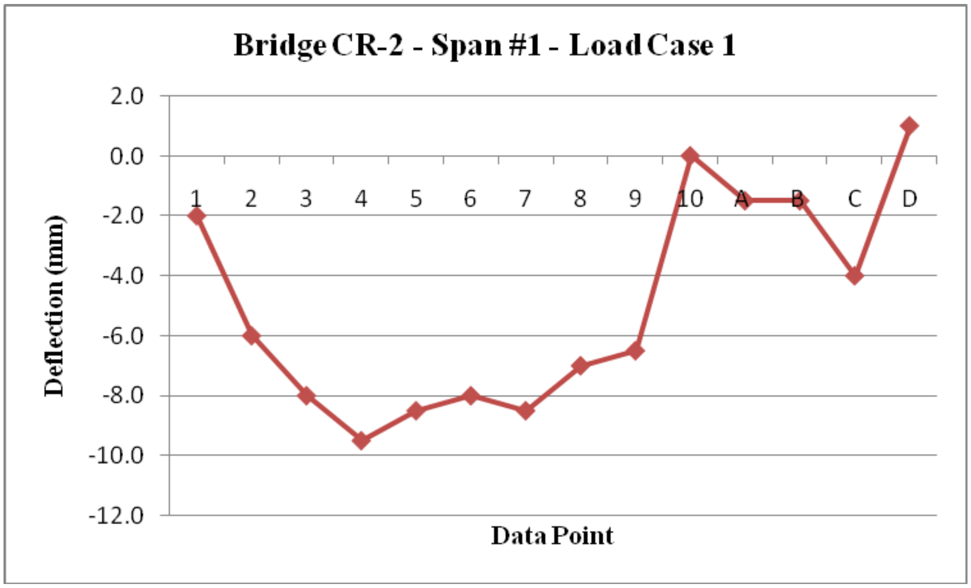

Figure 4.38. Deflection of span 1 of Bridge CR-2 for center loading.

Chapter 5
Testing Results

The visual, stress wave timing and resistance drill inspections identified a number of problems with several bridges in St. Louis County. This information was communicated to the County bridge engineer and their bridge crew conducted repairs to bridges 242 and 726. Problems were also noted on other bridges and they are being monitored by the County.

Table 5.1 shows the results of the vibration and load testing for each bridge and the measured and predicted EI product. If a bridge had two spans, each span is reported independently. Static midspan deflections ranged from approximately 0.10 in. to 0.25 in. and were all less than the recommended L/360 span –to-deflection criteria. First mode frequencies ranged from 17 to 35 Hz and generally decreased with increasing span length. 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 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 characterized the deflections of all stringers if the load 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. Based on static load testing results, the measured structure stiffness (*EI* product) of the field bridges ranged from 11,899 × 10^6 lb-in. (Bridge 357 span 1) to 154,189 × 10^6 lb-in. (Bridge 305 span 2).

A regression analysis was conducted on the measured stiffness versus the measured frequency of vibration. A correlation coefficient square (R^2) of 0.84 resulted. Figure 5.1 shows the relationship between frequency and static load stiffness. Bridge 242 spans 1 and 2 showed the greatest variability from the regression line. It is projected that this is due to the high level of decay found in the substructure pilings and pile cap on 242. It is believed that this decay affected the vibration characteristics of the superstructure. The inspection results for bridge 619 also showed some decay in the bridge pilings that may affect the vibration stiffness relationship. The estimates of first bending mode frequency from forced vibration testing may 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. 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 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.

Table 5.1. Summary of static and dynamic bridge stiffness values.

Bridge No.	Static loading		Dynamic vibration	
	Midspan deflection (in.)	Measured stiffness, EI (x 10^6 lb-in.^{4})	1^{st} mode frequency (Hz)	Predicted stiffness, EI* (x 10^6 lb-in.^{4})
85-span 1	0.18	56,051	25.4	77,348
85-span 2	0.12	89,107	27.7	97,236
153	0.12	50,768	27.5	62,454
242-span 1	0.12	48,851	22.0	34,222
242-span 2	0.11	56,488	21.7	35,186
305-span 1	0.12	107,745	31.9	164,941
305-span 2	0.09	154,189	34.6	207,704
357-span 1	0.12	15,141	18.5	7,047
357-span 2	0.12	11,899	17.6	5,176
619	0.25	17,906	22.2	14,049
CR1	0.24	24,899	21.2	27,644
CR2	0.24	22,397	18.9	20,265

* based upon beam theory

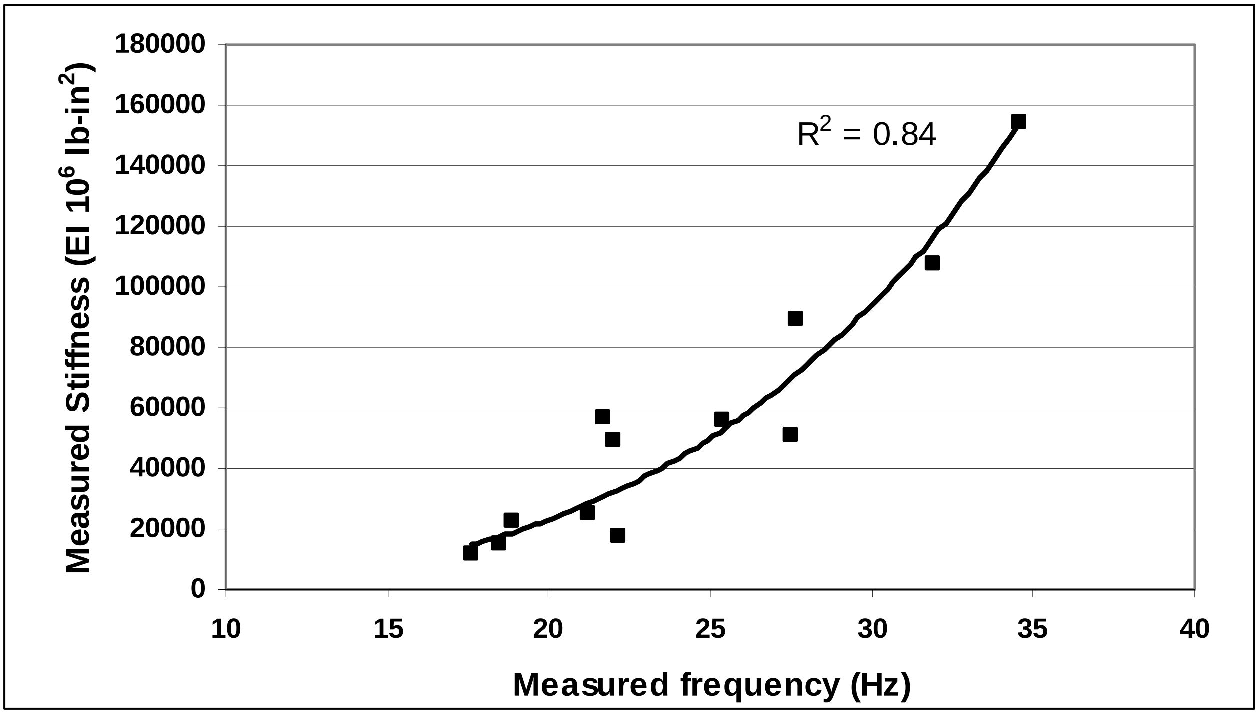

Figure 5.1. Relationship (y = 0.7007x3.4714) between static-load measured stiffness and measured frequency.

Chapter 6
Conclusions

The results indicate that forced-vibration methods have potential for quickly assessing timber bridge superstructure stiffness. However, further improvements need to be made in identifying 1st bending mode and estimation of bridge weight. The beam theory model matched the physics of the single-span, timber beam superstructures better than plate theory. This global vibration technique has potential benefits for routine inspections and long-term health monitoring of timber bridge superstructures.

Specific conclusions for this project include:

- The use of commercially available inspection equipment allowed the research team to identify critical areas of structural deterioration, resulting in completed repairs by St. Louis County. This deterioration typically took place in the bridge substructure, including pilings and pile caps. The use of stress wave timing and resistance microdrilling equipment allowed the inspection team to identify and quantify the decay in St. Louis County bridges 242 and 726. This information was communicated to the County and appropriate repairs were completed, extending the service life of the bridges.
- The use of vibration testing allowed inspectors to conduct rapid inspections on bridge sections to identify a peak frequency of vibration. When compared to the bridge stiffness as measured by load testing, a useful relationship occurred. The frequency of vibration increases with bridge stiffness.
- The vibration approach and equipment developed for this project show potential for assessing rural steel and concrete bridges, however new techniques and appropriate equipment need to be developed to adequately measure the vibration. The frequency range for the concrete bridges evaluated exceeded the available vibration capacity of the forcing motor used in this testing.

Additional studies should utilize field instrumentation that can clearly identify 1st bending mode frequencies with real time data processing tools and include automated control and data acquisition. Testing should also be conducted on dowel laminated bridge structures, which represent over 1,200 bridges in Minnesota.

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