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

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16. Abstract (Limit: 250 words)

Current timber bridge inspection procedures used in Minnesota and across the United States are mostly limited to visual inspection of the wood components. Use of advanced techniques like stress wave timing, moisture meters, resistance drills will significantly improve the reliability of the inspections but these inspection techniques are time consuming. The objective of this project was to conduct vibration testing of dowel laminated timber bridge systems to better understand the potential for using vibration testing to assess the structural health and condition of bridges in Minnesota. A second key objective was to improve and automate the vibration testing system that is currently being used. This research showed that the forced vibration system developed is an effective tool for conducting forced vibration tests of timber bridges and that there is a noted increase in frequency during each successive stage of construction. A reliable means for assessing the peak frequencies and an identification of the mode still needs to be developed for this system to use the vibration response to predict the EI product for use in load ratings. Each bridge has a unique set of vibration characteristics that were identified using the automated system. These characteristics showed peaks in amplitude as the frequency of the vibration was increased from 0 - 35 Hz during testing. It is believed that monitoring of the characteristic vibration response for each bridge would be a means of identifying changes in structural health over time due to wood decay, accidents, vandalism, or lack of maintenance.

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

Final Report

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Executive Summary

Inspection and maintenance of more than 4,000 timber bridges in Minnesota and more than 50,000 across the United States creates a unique challenge for public agencies. Nationally, more than 50% of these bridges are structurally deficient or functionally obsolete.

Current timber bridge inspection procedures used in Minnesota and across the United States are mostly limited to visual inspection of the wood components. These techniques have proven adequate for advanced decay detection, but are not adequate when the damage is in the early stage or is located internally in the members. These methods are labor intensive and time consuming, resulting in infrequent inspections that miss structural problems. Maintenance practices for timber structures need more emphasis and priority, since they are often deferred due to budget constraints. 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. A thorough initial analysis using stress wave timing and resistance drilling can take up to 3 person days to complete an inspection of a two-span timber bridge, and up to 1.5 person days for follow-up inspections. These techniques can assess structural changes over time, but do not allow for remote monitoring in between inspections that can lead to identification of immediate deterioration.

The University of Minnesota Duluth (UMD) Natural Resources Research Institute (NRRI) and their cooperators including the USDA Forest Products Laboratory, St. Louis County, Michigan Technological University and others have worked to transfer the use of advanced inspection techniques like stress wave timing, moisture meters, and resistance drilling to inspectors and engineers to significantly improve the reliability of timber bridge inspections, resulting in safer bridges in Minnesota. Vibration testing technology has also been developed and installed on several test bridges to monitor their structural health. Preliminary research and demonstration projects have shown the positive potential for this technology to be a continuous monitoring tool for bridge engineers, leading to early identification of potential problems. Implementation of these combined inspection techniques will enhance the safety of our rural bridge system and be cost-effective. Active health monitoring has the potential to significantly extend the service life of timber bridges and ensure the safety of motorists.

The objective of this project was to conduct vibration testing of dowel laminated timber bridge systems to better understand the potential for using vibration testing to assess the structural health and condition of bridges in Minnesota. Previous research has been limited to short span bridges that behaved like a beam in flexure. In contrast, dowel laminated bridges, as shown in Figure 1.1, have structural integrity in two directions, acting more like a plate. The individual timbers are connected side to side with metal dowels and a spreader beam is attached to the bottom. In addition to testing panelized bridge sections prior to, during and after installation in St. Louis County, testing was conducted on 12 bridge spans in Minnesota.

A second key objective was to improve and automate the vibration testing system that is currently being used. A new system was designed to automatically perform the testing, determine the natural frequency and report the results. A computer laptop was used as the

hardware component with Labview Windows as the software that will control the testing and acquisition of testing data.

The conclusions of this project were:

- The forced vibration system developed is an effective tool for conducting forced vibration tests of timber bridges. The unit automatically controls and performs the testing, by increasing the RPM's of an eccentrically loaded motor attached to the bridge deck. It also collects the vibration response of the bridge, noting the frequency and amplitude of the signal. It is capable of measuring vibrational frequencies in the range of 14 - 35 Hz.
- As dowel laminated bridges are constructed, there is a noted increase in frequency during each successive stage of construction for vibrational peaks 1, 2 and 3. The various construction stages that affected rigidity were the placement of sections, the pinning connections of the shiplap joint sections together using 12-inch carriage bolts, the attachment and tightening of the curbing, deck rails and the spreaderbeam, and finally complete backfilling of the approaches to the bridge. During the 2008 construction of a new 2008 St. Louis County bridge, a frequency increase of over 5 Hz is noted for peak 1, an increase of nearly 8 Hz for peak 2 and an increase of nearly 13 Hz for peak 3.
- An accurate means of determining bridge weight and the true measured stiffness is essential for being able to relate the measured frequency against the measured modulus of elasticity*moment of inertia (EI) product. Weight estimation based on wood volume and estimated unit weight for timber components is inadequate to obtain reliable results. The analytical models for simple beam and simple plate structures used to determine the EI product do not seem to effectively assess the structural performance of the bridge.
- A reliable means for assessing the peak frequencies and an identification of the mode still needs to be developed for this system to use the vibration response to predict the EI product for use in developing future load ratings.
- Each bridge has a unique set of vibrational characteristics that were identified using the automated system. These characteristics showed peaks in amplitude as the frequency of the vibration was increased from 0 - 35 Hz during testing. It is believed that monitoring of the characteristic vibrational response for each bridge would have merit as a means of identifying changes in structural health over time due to wood decay, accidents, vandalism, or lack of maintenance. For the purposes of this testing and potential implementation as a structural health monitoring tool, the results do show that the frequency of various peaks can be repeatedly and accurately determined over time. The challenge is to use this frequency data over time to identify any changes over time, especially decreases, since this would mean that the bridge has decreased in stiffness or rigidity. This may be due to various causes such as loose or failed connections, the presence of decay or other deterioration to wood or metal connectors, structural damage from an external force, or some sort of vandalism.

This technology does have potential application for use in monitoring rural bridge systems. In a separate project funded by the USDA Forest Service, we have instrumented a bridge near Meadowlands, MN, with the vibration equipment used in this project. Monitoring has shown that the vibration response can be monitored long-term and that the signal changes seasonally based on typical winter-summer temperature shifts. The key question that must be determined is how the vibrational response will change due to structural detioration caused by decay or lack of maintenance. Since these problems typically require years to occur, it will take a long time to understand this affect. Further, the current system only assesses the global performance of the superstructure. Near-term implementation of this technology is possible, but further research of experimental bridges is necessary to address these issues. Construction and intentional rapid deterioration of the primary types of Minnesota timber bridges will provide key assessments of the technology, leading to opportunities to use this technology to assess rural bridges in St. Louis and other Minnesota counties. The use of this monitoring technology combined with stress wave and resistance microdrilling will improve rural bridge inspections, leading to increased safety and lengthened service life.

Chapter 1: Introduction

<u>The Challenge</u>

Inspection and maintenance of more than 4,000 timber bridges in Minnesota and more than 50,000 across the United States. Nationally, more than 50% of these bridges are structurally deficient or functionally obsolete.

Current timber bridge inspection procedures used in Minnesota and across the United States are mostly limited to visual inspection, hammer sounding and probing of the wood components. These techniques have proven adequate for advanced decay detection, but are not adequate when the damage is in the early stage or is located internally in the members. These methods are labor intensive and time consuming, resulting in infrequent inspections that miss structural problems. Maintenance practices for timber structures need more emphasis and priority, since they are often deferred due to budget constraints. Use of advanced techniques and equipment like stress wave timing, moisture meters, and resistance drilling will significantly improve the reliability of the inspections. A thorough initial analysis using stress wave timing and resistance drilling can take up to 3 person days to complete an inspection of a two-span timber bridge, and up to 1.5 person days for follow-up inspections. These techniques can assess structural changes over time, but do not allow for remote monitoring in between inspections that can lead to identification of short-term deterioration.

A recent study on the reliability of visual inspections conducted on highway bridges (Phares et al. 2000; Phares et al. 2001) revealed condition ratings to be highly variable and to yield inaccurate results. In response, a large research initiative is underway at the US Department of Transportation - Federal Highway Administration (FHWA) to broaden the use of NDE techniques to help improve the state of the practice for highway bridge inspections (Washer 2000; Friedland and Small 2003). Many NDE techniques are available but few have been widely implemented. For example, one new method of load rating that does not rely on theoretical calculations involves the use of a wireless data acquisition system to rapidly measure strain and static live load deflections with a laser radar system (Washer Fuchs 1999). There are two new NDE techniques that have been recently utilized for condition assessment of timber highway bridges by measuring the structural response of the superstructure under dynamic conditions. The first technique involves measuring the frequency characteristics of the bridge superstructure under induced vibration. The frequency data is analyzed and converted to an equivalent measure of the bridge superstructure system stiffness which can be valuable as an inspection tool (Morison 2003; Peterson et al. 2003; Peterson et al. 2001a; Peterson et al. 2001b). The second technique involves measuring bridge superstructure deflections (at high rate) under dynamic truck loading (Wipf et al. 1999; Horyna et al. 2001). The deflection data is analyzed and compared with static deflections to determine dynamic amplification and impact factors which are compared to existing design values. Highway bridges in USA, Canada, and England constructed of concrete (Salawu 1997; Schwarz et al. 2001; Law et al. 1995; Fafard et al. 1998; Naumoski et al. 2002; Ventura et al. 2000), and steel (Zhao and DeWolf 2002; Halling et al. 2001) have previously been evaluated using these new dynamic techniques.

The University of Minnesota Duluth (UMD) Natural Resources Research Institute (NRRI) and their cooperators including the USDA Forest Products Laboratory, St. Louis County, Michigan

Technological University and others have worked to transfer the use of advanced inspection techniques like stress wave timing, moisture meters, and resistance drilling to inspectors and engineers to significantly improve the reliability of timber bridge inspections, resulting in safer bridges in Minnesota. Vibration testing technology has also been developed and installed on several test bridges to monitor their structural health. Preliminary research and demonstration projects have shown the positive potential for this technology to be a continuous monitoring tool for bridge engineers, leading to early identification of potential problems. Implementation of these combined inspection techniques will enhance the safety of our rural bridge system and be cost-effective. Active health monitoring has the potential to significantly extend the service life of timber bridges and ensure the safety of motorists.

Through UMD's Northland Advanced Transportation Research Laboratories (NATSRL), we have conducted several projects in FY04, FY05 and FY06. In the FY05/06 NATSRL project, Development of Flexural Vibration Inspection Techniques to Rapidly Assess the Structural Health of Rural Bridge Systems, funds were requested to complete forced vibration and load testing on a larger subset of timber bridges in northeastern Minnesota (Brashaw et al 2008). In the course of this project, we have found over 1,200 timber bridges in Minnesota that are of a dowel laminated design that have not been previously evaluated using our vibration techniques. Significant numbers of these bridges are found in Mn/DOT District 1, owned by St. Louis County. These bridges offer new challenges to vibration techniques because they are considered more of a plate structure than a beam structure, with structural members in both the span and cross-span direction. The majority of these are of a dowel laminated design, where individual treated timbers are stacked face to face and connected with long ring shank steel dowels. A spreader beam is installed beneath the deck to increase the lateral distribution of wheel loads to a wider portion of the deck. Thousands of bridges of this design have been built across the United States (Johnson, 2004). In the FY06 project, the research team used a digital scopemeter to collect and manually analyze the vibration response of the bridge, leading to potential error and repeatability when used by multiple inspectors.

The objective of this project was to conduct vibration testing of dowel laminated timber bridge systems to better understand the potential for using vibration testing to assess the structural health and condition of bridges in Minnesota. Previous research has been limited to short span bridges that behaved like a beam in flexure. In contrast, dowel laminated bridges, as shown in Figure 1.1, have structural integrity in two directions, acting more like a plate. The individual timbers are connected side to side with metal dowels and a spreader beam is attached to the bottom. In addition to testing panelized bridge section prior to, during and after installation in St. Louis County, we will conduct testing on 12 bridge spans in Minnesota.

Figure 1.1. Dowel laminated timber bridge near Brimson, Minnesota.

A second key objective will be to improve and automate the vibration testing system that is currently being used. A new system will be designed to automatically perform the testing, determine the natural frequency and report the results. A computer laptop will be used as the hardware component with Labview Windows as the software that will control the testing and acquisition of testing data.

Chapter 2: Development of Automated Testing Equipment

Background

In previous USDA Wood in Transportation projects (Brashaw et al 2006) and a Northland Advanced Transportation Research Laboratories project, Development of Flexural Vibration Inspection Techniques to Rapidly Assess the Structural Health of Rural Bridge Systems (Brashaw et al 2008), 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 expensive and complicated modal analysis. An electric motor with a rotating unbalanced wheel was used to excite the structure (Fig. 2.1), 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. Two 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 manually 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. Figure 2.2 shows the setup of the signal acquisition and the motor control system that was used in that testing.

Figure 2.1. Rotating motor used to excite the timber bridge.

One limiting factor in the previous research has been the manual nature of the testing and data interpretation. One of the key research activities in this project was to support the design and development of an automated testing system. This system was designed to automatically increase the RPM of the vibration forcing motor and capture the resulting signal from the accelerometers, determine the peak amplitude and calculate the frequency of vibration. This system improved the reliability of the testing procedure and reduced variation between inspectors.

Activities

The first step in our process of building a new system was to research the hardware used to control a motor and read the accelerometers. This hardware was then installed within a weather

Figure 2.2. Testing system used to control motor and acquire vibration response.

resistance plastic case for portability and ease. The computer selected was a Toshiba Satellite laptop. This unit had more computing power and speed than other systems evaluated. The proposal had suggested purchase of a "ruggedized" laptop. Several manufacturers of "ruggedized" laptops were evaluated, however, the computer processing speed was very limiting and the costs excessive. As an alternative, a 3 year extended warranty was purchased for the Toshiba, which would cover any damage from weather, impact or other on-site problems.

National Instruments LabVIEW was chosen as the control software package. This automated testing procedure using LabView has improved the testing and data consistency of testing. When the program has finished the test, a graph is displayed identifying the peak frequencies over time from each accelerometer during the test. A subprogram was developed to replay the signal response as desired.

Troubleshooting was conducted on the system on numerous field bridges around the Duluth area and in Alberta, Michigan. These bridges were timber construction or steel and concrete construction. Ongoing testing has been completed on St. Louis County Bridge #242 outside Meadowlands, MN. This troubleshooting identified several data collection improvements and better understanding of the vibration characteristics of the system. Several pictures of the testing system are shown in Figure 2.3. A standard operating procedure for the testing and vibration data playback was developed and is included as Figure 2.4 and 2.5. Table 2.1 shows all of the components of this system.

Figure 2.3. Images from the automated bridge vibration system developed in this project.

Standardized Worksheet	Automated Vibration Testing	Page 1 of 1

Company:	NRRI	Laboratory:	Field Testing	PPE:	
Date:	5/10/2007	Document #:		Tools:	Box, Signal Conditioner, Cables, DAQ
Approved By:	Manager:	Brian Brashaw		Supervisor:	
	Date:	2/20/2007		Date:	

No.	Work Elements	Key Points Safety, Quality, Technique, Cost	Time Elements	Breakdown Auto	Manual	Wait
1	Off Position	All switches should be in off position. If signal conditioner or filter switches are on, check batteries.	10 sec		10	
2	Plug in Cords	Plug in power cord in the power receptor. Plug in motor cord in AC or DC slot depending on the motor type. Plug in BNC cables to the signal conditioner and then to the control box with the respective color codes.	1 minute		1	
3	Turn on the Computer and Insert DAQ Card	With the ribbon attached to the DAQ card plug it in the PCMCIA slot of the computer after booting up.	3 minutes		3	
4	Load Program	Double click on the icon labeled Bridge Vibrator located on the desktop.	10 sec		10	
5	Turn on Control Box Switches	Make sure control box and signal conditioner switches are now on. For the box, the filter switches and the motor switch should be turned on. The motor type determines which switch to turn on. If both get turned on an a red LED is flashing, remove power until all LEDs are off.	5 sec		5	
6	Start Program	Click the white arrow button in the top left hand corner of the window. Follow the prompts until program runs on its own. This save is for playback. Format in Date-Test-Play. Note: when restarting the program after a run be sure to set back to default. Minimize the excel spreadsheet until test is finished.	5 sec		5	
7	Stop Program	Use the stop button located under the graphs to stop the program. The program will also automatically stop.				
8	Save Data	Save the excel spreadsheet. Date-Test-Data	5 sec		5	
9	Disconnect	Turn off all switches. Then unplug power cords. Turn off the computer. Clean surrounding area.	5 minutes		5	
Key	Safety	Poka Yoke	Quality	In-Progress Stock	Time Totals 9m 35s	9:35

Work Area Layout

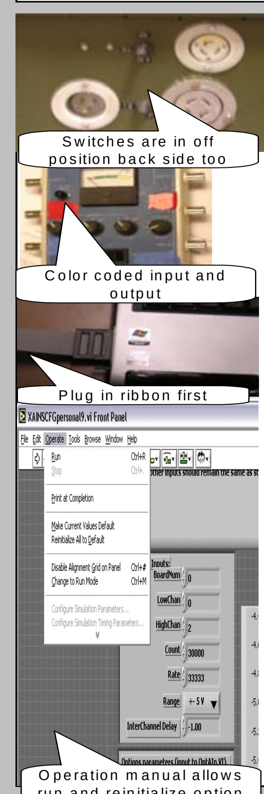


Figure 2.4. Standard operating procedure for automated bridge vibration testing equipment.

Standardized Worksheet	Vibration Data Playback		Page 1 of 1	
Company: NRRI	**Laboratory:**	Field Testing	**PPE:**	
Date: 5/10/2007	**Document #:**		**Tools:**	Computer
Approved By:	**Manager:**	Brian Brashaw	**Supervisor:**	
	Date:	2/20/2007	**Date:**	

No.	Work Elements	Key Points *Safety, Quality, Technique, Cost*	Time Elements	Breakdown: Auto	Manual	Wait
1	Turn on Computer	Turn on the computer and log in to your username.	3 minutes		3	
2	Open Bridge Data Playback	Double click on the icon labeled Bridge Data Playback located on the desktop.	1 minute		1	
3	Start Program	Click the white arrow button in the top left hand corner of the window. Follow the prompts until program runs on its own. Note: when restarting the program after a run be sure to set back to default.	5 sec		5	
8	Shut Down Program	Click the exit button to shutdown the program. DO NOT save changes when prompted to do so.	5 sec		5	

Key	Safety	Poka Yoke	Quality	In-Progress Stock	Time Totals			
	✚	⚡	▲	◆	4m 10s		4:10	

Work Area Layout

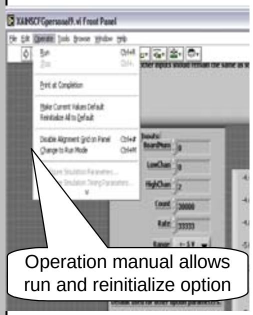


Figure 2.5. Standard operating procedure for playback of vibration data.

Table 2.1. Parts list for automated testing system

Component	Description	Model number	Vendor
Laptop Computer	Satellite P100	PSPADO-1SQ014	Toshiba
Laptop Case	Weatherproof	1495	Pelican
Control Unit Case	Weatherproof	1400	Pelican
Tape Ruler	100 Foot Ruler	1MKT4	Grainger
Power Supply	Control Unit	S82K-00715 (2)	Omron
Signal Conditioner	For Accelerometers	480B21	PCB Electronics
Accelerometers		626B02	IMI sensors
Cables	For Accelerometers	052AE050	IMI sensors
Motor	90 Volt/2500 Rpm	C42D28FK10	Leeson
Power Cable	#12/ 3 Conductor	50 feet	Grainger
Generator	Electric supply	2000i	Honda
Twist lock plugs	Component Connections	Qty. of 8	Grainger
Coax Cables	Connections	6 feet (6)	Grainger
Optical Level	Level and Tripod	55-SLVP-28N	CTS Berger
Folding Rulers	Metric Waterproof	61662 (qty. 30)	Wiha tools
Three low pass filters (0 to 50 Hz)			DigiKey
Nine volt batteries			DigiKey
Fuse			DigiKey
Switch			DigiKey
0 – 90 V DC Linear Motor Control			DigiKey
0 – 180 V AC Linear Motor Control			DigiKey
PCI Mini Card for Data acquisition		PC-CARD-DAS16-16-AO	Measurement Computing
Ribbon Cable	Connect to Laptop		DigiKey
Custom circuits			NRRI
Ruler Brackets	Aluminum Angle	Shop made (qty. 100)	NRRI Machine Shop
Plum Bob	Measurement Transfer	47-973	Grainger
Motor Weight	Mounting plate, Shroud	Shop Made	NRRI Machine Shop
Fasteners	Bolts, Screws,Etc	Misc.	Home Depot
Safety Signs	Folding “men working”	150-LG-EG (2)	Signs Direct
Cordless Drill	For Mounting Items	DS14DVF3	Hitachi

Chapter 3: Vibration Assessment of Dowel-Laminated Bridge Sections during Construction of a Full-Scale Bridge

Background

St. Louis County identified substantial deterioration in county Bridge 53, located on Pioneer Road in Normanna Township. The county bridge engineer closed the outside lanes of the bridge in fall 2007. A new bridge superstructure was ordered from Wheeler Consolidated. This type of bridge is often referred to as a "Wheeler" bridge. This bridge was fabricated in panelized sections at Wheeler's facility located in Whitewood, South Dakota and delivered to St. Louis County in April 2008. These sections were tested before, during and after construction to assess the vibration characteristics using the automated testing equipment developed this project.

Before Bridge Construction

The bridge sections that were to be used for the new Bridge 53 were numbered so the data collected during testing would be clearly identified and to minimize potential testing error through misidentification. There were two different types of cross sections for the bridge members. Sections 1 and 2 had a shiplap type joint on one edge on their cross section and were the outside sections of the bridge deck. Sections 2, 3 and 4 had a shiplap joint on each outer edge and would be the inner sections of the bridge. The numbered sections and their different cross sections can be seen in Figure 3.1. Dimension measurements for the length, width, and depth of each section were collected.

Figure 3.1. Bridge sections to be used as the new superstructure for St. Louis County Bridge 53.

Each bridge section was weighed. As known from the theoretical model shown in Equation (3), bridge weight is needed in predicting the structure stiffness when using the vibration response method, which was used in this research and is outlined later in this chapter. In order for the sections to be weighed, a crane was used to transport the sections from their original position onto two wooden beams. This process can be seen in Figure 3.2. The purpose of the beams was to elevate the sections so they were only in contact with the beams and not the ground. They were spaced apart so that when the sections were set on them there was little to no overhang of the section over the beam. Scales were placed on each corner of both beams for a total of four scales as seen in Figure 3.3. When the section was set onto the beams, the weight measurement for each scale was recorded and the four measurements were then added together to get an estimate of the overall weight of the section.

Figure 3.2. A crane was used to transport sections for weighing and vibration testing.

Figure 3.3. Scales were placed on each corner of two beams. The sections were then set on the scales using a crane to get a weight estimate.

After the weight measurements were recorded, the crane lifted the section and the scales were removed. The section was then set down on two beams for vibration testing. For the first two tests of the first section, the beams were placed on a flatbed trailer seen in Figure 3.4. A review of the vibration data showed substantial damping and poor signal quality. It was concluded that placement of the deck sections on a flatbed trailer would yield inaccurate results because of the springs and tires on the trailer. Because of this, the remainder of the tests occurred with the two beams placed on the ground, which was a gravel surface. This setup can be seen in Figure 3.5.

<u>Vibration test procedure</u>

Vibration testing of the section was then conducted. The general setup can be seen in Figure 3.6. A ½ hp electric motor with a rotating unbalanced disc was attached to the mid-point of the structure. Three piezoelectric accelerometers (PCB 626BO2), also at midspan, were magnetically attached to steel plates attached using lag screws to the outer edges and the midpoint. The motor was controlled and the data collected using a laptop notebook with a customized program developed using Labview Windows. The motor slowly increases in RPM and the resulting vibrational frequency is collected. The amplitude of the frequency response is plotted to identify frequency peaks that occur during testing.

The specific testing steps included:

1. Secure a dc motor (1/2 horsepower) with rotating unbalanced disc to the deck at the center of the span and anchor using steel bolt screws.
2. Secure three metal plates to deck plank, one on each side of the span and one in the center near the motor at midspan using steel lag screws.
3. Attach one magnetic piezoelectric accelerometer to each magnetic metal plate to monitor the bridge vibration signals.

4. Connect motor and accelerometer to laptop computer as noted in Figure 2.7 that details the computer testing process.
5. Record the data and start a second test.

After testing of a section was complete, the member was moved away by the crane and each additional section was tested in vibration.

Figure 3.4. The first two tests of the first section were conducted on a trailer. Due to poor signal quality, this technique was abandoned for the remaining testing.

Figure 3.5. The sections were placed on two beams so that they were elevated above the ground for vibration testing.

Figure 3.6. General test setup used for vibration testing of each section.

Standardized Worksheet	Automated Vibration Testing		Page 1 of 1			
Company:	NRRI	Laboratory:	Field Testing	PPE:		
Date:	5/10/2007	Document #:		Tools:	Box, Signal Conditioner, Cables, DAQ	
Approved By:	Manager:	Brian Brashaw		Supervisor:		
	Date:	2/20/2007		Date:		

No.	Work Elements	Key Points Safety, Quality, Technique, Cost	Time Elements	Breakdown Auto	Manual	Wait
1	Off Position	All switches should be in off position. If signal conditioner or filter switches are on, check batteries.	10 sec		10	
2	Plug in Cords	Plug in power cord in the power receptor. Plug in motor cord in AC or DC slot depending on the motor type. Plug in BNC cables to the signal conditioner and then to the control box with the respective color codes.	1 minute		1	
3	Turn on the Computer and Insert DAQ Card	With the ribbon attached to the DAQ card plug it in the PCMCIA slot of the computer after booting up.	3 minutes		3	
4	Load Program	Double click on the icon labeled Bridge Vibrator located on the desktop.	10 sec		10	
5	Turn on Control Box Switches	Make sure control box and signal conditioner switches are now on. For the box, the filter switches and the motor switch should be turned on. The motor type determines which switch to turn on. If both get turned on an a red LED is flashing, remove power until all LEDs are off.	5 sec		5	
6	Start Program	Click the white arrow button in the top left hand corner of the window. Follow the prompts until program runs on its own. This save is for playback. Format in Date-Test-Play. Note: when restarting the program after a run be sure to set back to default. Minimize the excel spreadsheet until test is finished.	5 sec		5	
7	Stop Program	Use the stop button located under the graphs to stop the program. The program will also automatically stop.				
8	Save Data	Save the excel spreadsheet. Date-Test-Data	5 sec		5	
9	Disconnect	Turn off all switches. Then unplug power cords. Turn off the computer. Clean surrounding area.	5 minutes		5	
Key	Safety	Poka Yoke	Quality	In-Progress Stock	Time Totals 9m 35s		9:35	

Work Area Layout

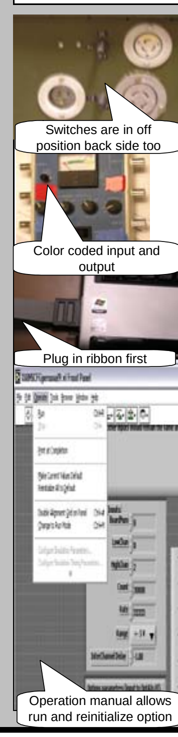


Figure 3.7. Standard operating procedure for automated bridge vibration testing equipment.

During Bridge Construction

Bridge substructure inspection

St. Louis County planned for the replacement of the superstructure of Bridge 53. This included all the pile caps, the bridge deck and the bridge safety railings. The St. Louis County bridge engineer asked the project team to conduct an inspection of the pilings that were to be reused. Stress wave timing was performed on the existing pilings in order to locate any decay in the pilings. This was done to assure that none of the pilings needed to be replaced before the superstructure was put into place. If stress wave timing indicated significant decay in any of the pilings, resistance microdrilling would have been performed on those pilings to locate the decay and determine if the pilings needed to be replaced.

Stress wave timing

An example of the stress wave concept for detecting decay for a circular piling is shown in Figure 3.8. First, a stress wave is induced by striking the specimen with an impact device 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, receives 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. There are several commercially available stress wave timing units. These units will yield comparable results if they are calibrated and operated according to the manufacturer's recommendations. 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 commercial Fakopp Microsecond Timer was used to determine the stress wave time across the pile top and 2 feet below the pile top. 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 timber bridges. The substructure species for bridge 53 was determined to be southern yellow pine.

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 3.8. Fakopp microsecond timer being used on a timber pile.

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 for vibrational testing on a bridge superstructure can be seen in Figure 3.9.

Figure 3.9. General test setup for vibration testing.

Construction

The new superstructure for St. Louis County Bridge 53 consists of the five wood sections previously noted. The placed position of each section can be seen in Figure 3.9. The sections were set into place on the bridge using an excavator. This can be seen in Figure 3.10. All construction was completed by the St. Louis County bridge construction crew.

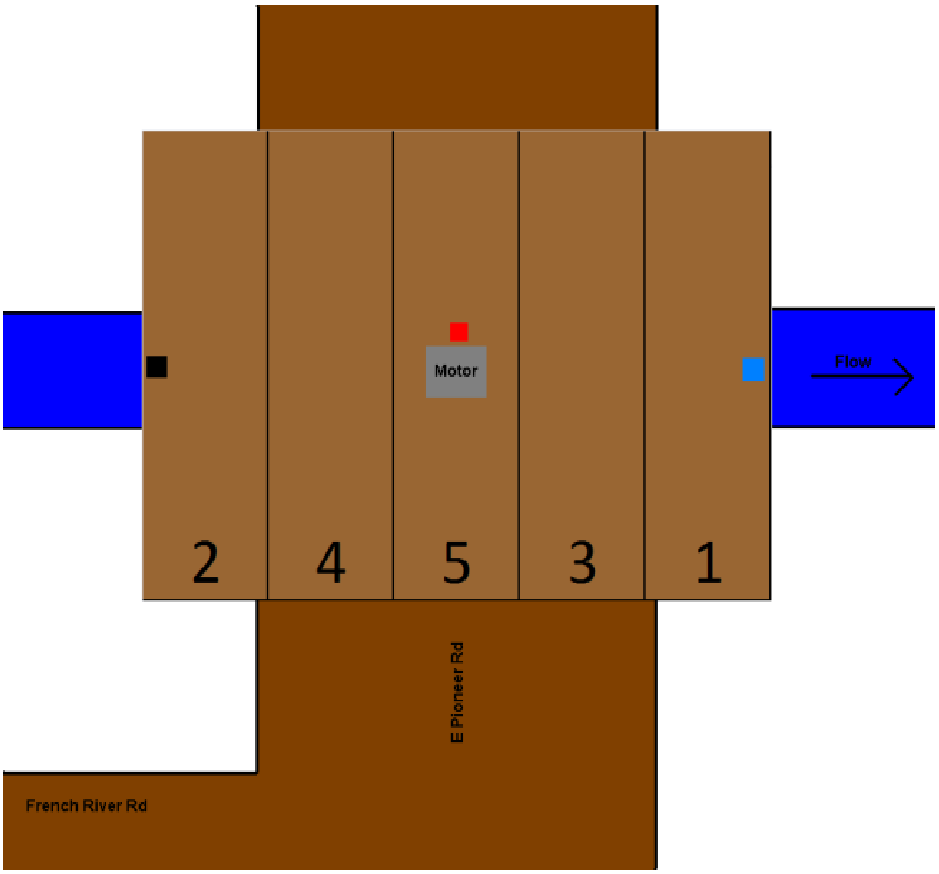


Figure 3.10. Final position of each section in the new Bridge 53 superstructure, as well as the final test setup. Sections 1-5 locations are identified.

Figure 3.11. Wood sections were set into place using an excavator.

Some of the stays that hold the sections in place had to be trimmed with a chainsaw in order for the sections to fit onto the pile cap. Every stay that was cut was treated with copper napthenate to protect the wood from decay. The procedure can be seen in Figures 3.11 and 3.12.

Figure 3.12. Some stays were trimmed with a chainsaw so sections could fit. Copper napthenate was used to treat the newly exposed surfaces.

As soon as the cut stays were treated, the sections could be set into place as seen in Figure 3.13. Once the sections were all in place, the lap joints were pinned with carriage bolts. This can be seen in Figure 3.14. The sections were then bolted to the pile cap and all of the dirt that had been removed was back filled. Figure 3.15 shows the bridge with one side completely back filled and the other side in the process of being back filled.

Figure 3.13. Sections were set into place once stays were trimmed and treated.

Figure 3.14. Lap joints were pinned together with carriage bolts once sections were set into place.

Figure 3.15. Once sections were pinned together and bolted to the pile cap, dirt was back filled.

Once all of the dirt was back filled, the spreader beam was mounted below the deck, but it was not bolted tight. The curbs and guard rails were then installed. Figure 3.16 shows the spreader beam mounted below the deck and Figures 3.17 and 3.18 show the curb and guard rail being installed on one side of the bridge. After the guard rails were installed, the spreader beam was bolted tight, additional dirt work was completed, and the bridge deck was paved with bituminous asphalt. Construction of the bridge was then complete. Vibration testing was conducted during each phase of bridge construction as noted in the following dated sections.

Figure 3.16. Spreader beam mounted below the bridge deck.

Figure 3.17. Curb being installed on bridge deck.

Figure 3.18. Guard rail installed on bridge deck.

Vibration testing notes

An attempt was made by the project team to systematically conduct vibration testing during construction. However, the St. Louis County bridge crew had time and construction constraints and the project team was not always able to clear the bridge deck prior to vibration testing. The team was also not able to test each section independently due to these constraints. This following testing notes are reported as appropriate and will be considered during the data analysis.

6/19/2008 Vibration testing was conducted after the first three sections had been set and were only resting on the pile cap as seen in Figure 3.19. The first series of tests were performed with the motor parallel to the laminations at the center of panel 1. The setup is shown in Figure 3.20. Test 9 was conducted with the motor in the center of panels 1 and 3, but perpendicular to the laminations. The lap joints for the three sections were then pinned and test 10 was run with the motor still in the center of panels 1 and 3, but with the motor shaft rotating parallel to the laminations as shown in Figure 3.21.

Figure 3.19. The first three sections were set into place and then vibration testing was done.

Figure 3.20. The first tests were conducted with the motor in the center of the first section oriented parallel to the laminations.

Figure 3.21. Motor being mounted parallel to the laminations in the center of sections 1 and 3.

6/20/2008 All five of the sections were now in place and their lap joints pinned together. They were not bolted to the pile cap. The motor was placed in the center of the bridge with the shaft parallel to the laminations for the first three tests. For tests 4 through 6, the motor was rotated so the shaft was perpendicular to the laminations. The French River Road side of the bridge was then back filled with dirt and tests 7 through 9 were conducted with the motor perpendicular to the laminations. Tests 10 through 12 were performed with the motor rotated so it was parallel to the laminations. Tests that were run with the motor shaft parallel to the laminations were reading clearer signals so at this point the rest of the tests run on this bridge were run with the motor shaft parallel to the laminations. Once test 12 was completed, the other side of the bridge was half back filled with dirt as shown in Figure 3.22 and test 13 was run.

Figure 3.22. Vibration testing conducted with the French River Rd side of the bridge completely back filled and half of the other side of the bridge back filled.

6/23/2008 For test 1, the spreader beam was mounted, but not bolted tight. Due to time constraints for the bridge crew, it was necessary to conduct testing with some outside influences. The guard rails were lying on the bridge deck near the center and workers were on the deck bolting the curb on. For test 2, the workers were using an impact wrench on the downstream curb while testing was taking place. For test 3, the downstream curb was now tight, as seen in Figure 3.23, and there was impact and pounding during the test. For test 4, there was again drilling and pounding during the test. The excavator was also part way onto the bridge as seen in Figure 3.24 and towards the end of the test the excavator bucket was resting on the bridge. Test 5 was run with both the upstream and downstream curbs tight. There were also two guard rail posts secure on the downstream side. There was again drilling, pounding, and the excavator part way on the bridge during testing. For test 6, three guard rail posts were now secure and there was no outside activity on the bridge.

Figure 3.23. Downstream curb completely installed.

Figure 3.24. Some tests were run with the excavator part way on the bridge deck.

6/26/2008 The upstream rail was now secure as seen in Figure 3.18, but the downstream rail was not. The spreader beam was still not bolted. Three tests were conducted with the motor mounted on top of the bridge. Three additional tests were run with the motor mounted underneath the bridge and the accelerometers mounted on top. Figure 3.25 shows the motor mounted underneath the bridge deck. For the tests with the motor mounted on the bottom, the deck was clear of materials and the downstream railing was being installed.

Figure 3.25. Vibration testing with motor mounted below the bridge deck.

<u>After bridge construction</u>

7/10/2008 The bridge was completed and vibration testing was conducted using the typical procedure. Static load testing of the bridge was completed in September 2008.

Testing Results and Discussion

Bridge 53 substructure inspection

Table 3.2 shows the nondestructive stress wave transmission times collected during testing of the bridge piles prior to installation of the new timber pilecap and bridge superstructure. This testing validated that there was not substantial decay present in the exposed sections of the pile caps. The diameter of the pilings was approximately 12-14 inches. It is commonly reported that stress wave transmission in the range of 220 - 250 microseconds/foot (µs/ft) indicates no or minimal decay in southern yellow pine. Stress wave transit times for the Fakopp timer of greater than 300 µs/ft may indicate moderate decay and over 500 µs/ft severe decay or the presence of splitting. The data for Bridge 53 showed some data over 400 µs/ft. A thorough inspection showed that this was the result of splitting, however. There was no recommendation made to repair or replace any of the pilings on this bridge.

Table 3.2. Stress wave transmission times for St. Louis County Bridge 53 pilings.

Piling Number	Stress Wave Transmission Time (microseconds/ft)	
	Top of piling	2 ft down from top of piling
1	278	277
2	172	258
3	198	222
4	199	350
5	310	679
6	298	343
7	199	586
8	253	427
9	256	322
10	169	186
11	192	285
12	157	324
13	172	184
14	178	201

Note: The measurements highlighted in red were due to splits in the wood and not due to severe decay. Measurements were taken a few inches above and below where the original measurements were taken and the readings indicated sound wood according to Table 3.1.

Dimensions and weights of bridge deck sections

The dimensions and weights for the bridge deck sections were measured prior to installation. The species for the bridge components was southern yellow pine and it was treated with copper napthenate to a retention level of 0.6 pounds of cubic ft.

Table 3.3. Measured values for the dimensions of each section of new Bridge 53.

Section #	Length (ft and in)	Width (in)	Depth (in)	Scale 1 weight (lbs)	Scale 2 weight (lbs)	Scale 3 weight (lbs)	Scale 4 weight (lbs)	Total section weight (lbs)
1	25'11"	67.75"	13.75"	1,200	1,600	1,300	1,700	5,800
2	25'11"	67.00"	13.75"	1,900	750	2,000	1,000	5,650
3	25'11"	75.25"	13.75"	2,100	1,650	1,800	1,400	5,950
4	25'11"	79.75"	13.75"	2,200	2,050	1,650	1,000	6,900
5	25'11"	75.25"	13.75"	1,950	1,800	2,000	1,500	7,250
							Total section weight	31,550

Vibration testing results and discussion

The automated testing system collected the frequency data during vibration testing from each of the three transducers placed along the midpoint of the bridge sections. The data was written to an Excel spreadsheet and then analyzed to determine the frequency of the peak signal responses. Usually, there are several peak responses in the time domain that occur during the testing period. The frequency for each peak is reported in the data analysis. Also, the difference in phase between the transducers is reported. If the transducers are in phase, the phase will typically be close to 0 or 360 degrees. If they are completely out of phase with each other, the phase shift will be near 180 degrees.

The vibration data for the first several tests of individual sections was discarded due to the fact that the sections were supported on a flat bed truck, which resulted in false frequency readings. Table 3.3 and Figures 3.29 shows the frequency data and graph for each section as placed on simple ground supports at the St. Louis County garage facility. The phase data shows that only section 2 was out of phase during the testing. A review of the data did not show any irregularities in the testing data, however, it was an unexpected result since each section should behave similarly to a beam, with the displacement across the section being in the same direction and uniform.

Tables 3.3. - 3.13 show the vibration testing results detailing the frequency of the peaks identified during testing, the phase between the black (upstream), red (center) and blue accelerometers (downstream), and any comments noted during testing as the bridge was being constructed. Shaded section data within each table was also reported in summary Table 3.14. The data was selected based on the project teams confidence in the data, and to represent various stages during the construction process. Figures 3.29. - 3.33. show the graphical frequency peaks and phase results from the testing.

An evalaution of the vibration testing results showed that there was a number of peaks noted during vibration testing for each individual section and at each testing condition during the construction process. As noted in Table 3.14, there were typically three clear peaks noted during testing. In some cases, a 4th peak was detected, but the results for that peak are not considered since we could not detect the peak in all the testing data sets.

There is an increase in frequency noted during each successive stage of the testing for peaks, 1, 2 and 3. This is as expected, since the bridge increases in rigidity and stiffness as the bridge is constructed. The various construction stages that affected regidity were the placement of the individuals into place, so that the ends were restrained, the pinned connections of the shiplap joint sections together using 12-inch carriage bolts, the attachment and tightening of the curbing, deck rails and the spreader beam, and finally complete backfilling of the approaches to the bridge. During construction, a increase of over 5 Hz is noted for peak 1, an increase of nearly 8 Hz for peak 2 and an increase of nearly 13 Hz for peak 3. This is to be expected based on the beam and plate theory formulas that show increased stiffness will increase the expected frequency of vibration.

Analysis of the phase differences between the upstream, midspan and downstream accelerometer results shows that for peak 1, we see that the center accelerometer is approximately 180 degrees out of phase to the transducers located on the outer edges (upstream and downstream as noted in Figure 3.10.). Typically this means that the outer edges of the panels and bridge are moving upward and downward in the same direction at the same time, and opposite of the center of the bridge. For peak 2, the results show that all of the transducers are nearly in phase with each other during the various stages of construction. This would mean that at the peak 2 frequency, all three locations are moving in the same direction, across the width of the bridge. This would typically be reported as a bending mode of vibration, with the midpoint of the bridge at the maximum deflection. An assessment of the peak 3 results show vibration modes similar to peak 1, except for the tests completed after the bridge was completely constructed, which showed characteristics similar to peak 2. Without a comprehensive set of modal testing, it would be difficult to assess the exact vibration patterns and modes, which was outside the scope of this project.

For the purposes of this testing and potential implementation as a structural health monitoring tool, the results do show that the frequency of various peaks can be repeatedly and accurately determined over time. The challenge is to use this frequency data over time to identify any changes over time, especially decreases, since this would mean that the bridge has decreased in stiffness or rigidity. This may be due to various causes such as loose or failed connections, the presence of decay or other deterioration to wood or metal connectors, structural damage from an external force, or some sort of vandalism.

Table 3.4. Vibration data collected for each section of Bridge 53 before it was constructed.

Bridge 53 - Before Construction	
Section #	**Peak 1 Frequency (Hz)**
1	11.1
2	11.7
3	11.8
4	11.1
5	11.1

Table 3.5. Vibration data collected for Bridge 53 on 6/19/2008 showing the frequency/phase relationship for peaks 1 and 2.

6/19/2008			Phase				Phase		
Test	**Test Condition**	**Peak 1 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 2 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
1	First three sections in place, but not bolted tight. Motor perpendicular to the laminations in the center of section 1.	-	-	-	-	-	-	-	-
2		-	-	-	-	-	-	-	-
3		-	-	-	-	-	-	-	-
4		-	-	-	-	-	-	-	-
5		-	-	-	-	-	-	-	-
7		-	-	-	-	-	-	-	-
8		12.73	11.13	11.61	0.48	25.06	274.88	4.45	270.43
9	Motor placed in the center of the first two sections, sections were not pinned together.	12.95	268.98	233.03	35.95	15.42	3.19	171.91	175.10
10	Lap joints pinned, motor parallel to the laminations, and placed in the center of the first two sections.	12.91	12.00	176.27	164.27	15.99	5.85	13.64	7.79

Note:

- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- For tests 1 through 7, the accelerometers weren't reading properly so there was no visible peak in the data.
- Shaded section data is used in summary Table 3.14.

Table 3.6. Vibration data collected for Bridge 53 on 6/19/2008 showing the frequency/phase relationship for peaks 3 and 4.

6/19/2008			**Phase**				**Phase**		
Test	**Test Condition**	**Peak 3 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 4 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
1 2 3 4 5 7 8	First three sections in place, but not bolted tight. Motor perpendicular to the laminations in the center of section 1.	- - - - - - -	- - - - - - -	- - - - - - -	- - - - - - -	- - - - - - -	- - - - - - -	- - - - - - -	- - - - - - -
9	Motor placed in the center of the first two sections, sections were not pinned together.	17.02	24.60	153.13	177.74	-	-	-	-
10	Lap joints pinned, motor parallel to the laminations, and placed in the center of the first two sections.	25.34	167.41	11.64	155.77	-	-	-	-

Note:

- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- Shaded section data is used in summary Table 3.14.

Table 3.7. Vibration data collected for Bridge 53 on 6/20/2008 showing the frequency/phase relationship for peaks 1 and 2.

6/20/2008			Phase				Phase		
Test	**Test Condition**	**Peak 1 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 2 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
1	All five sections in place with lap joints pinned together, but not bolted to pile cap. Motor placed parallel to the laminations.	13.29	6.85	177.09	183.94	16.82	8.09	9.71	1.62
2		13.25	3.07	178.74	175.68	16.77	8.50	10.46	1.96
3		13.30	5.28	185.89	180.61	16.82	8.09	8.71	0.62
		Mean = 13.28				**Mean = 16.80**			
4	Motor perpendicular to the laminations.	13.30	5.28	185.89	180.61	16.87	5.98	10.27	4.29
5		13.18	5.95	181.69	175.74	16.77	10.25	14.04	3.79
6		-	-	-	-	16.80	11.87	14.83	2.96
7	French River Rd side of bridge back filled with dirt.	-	-	-	-	16.95	5.96	11.64	5.68
8		13.25	7.40	182.93	175.53	16.80	10.39	16.17	5.78
9		13.25	1.80	182.84	181.04	16.62	17.40	23.94	6.54
10	Motor parallel to the laminations.	13.38	7.00	183.19	176.18	17.06	8.52	8.64	0.12
11		13.31	2.91	177.74	180.65	17.27	10.88	9.87	1.01
12		13.38	5.20	183.10	177.90	17.17	7.68	10.63	2.96
13	Other side of bridge back filled.	13.37	2.25	180.15	182.40	17.20	8.28	11.96	3.67
14		13.42	1.42	178.42	179.84	17.21	5.95	10.44	4.50

Note:

- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- Shaded section data is used in summary Table 3.14.

Table 3.8. Vibration data collected for Bridge 53 on 6/20/2008 showing the frequency/phase relationship for peaks 3 and 4 if present.

6/20/2008			Phase				Phase		
Test	**Test Condition**	**Peak 3 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 4 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
1	All five sections in place with lap joints pinned together, but not bolted to pile cap. Motor placed parallel to the laminations.	24.36	0.39	167.98	167.59	-	-	-	-
2		24.44	1.16	188.92	190.09	-	-	-	-
3		24.50	0.20	190.92	191.12	**38.13**	20.35	4.06	16.29
		Mean = 24.43							
4	Motor perpendicular to the laminations.	24.57	2.93	183.29	186.22	38.03	21.83	1.20	23.04
5		24.61	3.09	176.19	173.10	-	-	-	-
6		24.66	4.05	183.84	187.89	38.06	18.60	3.12	21.72
7	French River Rd side of bridge back filled with dirt.	24.77	8.19	183.30	191.49	38.10	16.88	3.87	20.74
8		24.79	8.34	183.19	191.53	38.10	17.66	5.67	23.33
9		24.80	8.76	176.84	168.09	38.22	18.27	4.09	22.37
10	Motor parallel to the laminations.	24.84	2.22	175.12	172.90	38.20	15.52	4.14	19.66
11		24.79	3.65	176.66	173.00	38.20	14.34	6.11	20.46
12		24.88	4.02	176.77	172.75	38.42	13.19	6.66	19.84
13	Other side of bridge back filled.	25.08	3.92	182.72	186.64	38.64	10.16	7.27	17.43
14		25.17	4.39	177.72	173.32	-	-	-	-

Note:

- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- Shaded section data is used in summary Table 3.14.

Table 3.9. Vibration data collected for Bridge 53 on 6/23/2008 showing the frequency/phase relationship for peaks 1 and 2.

6/23/2008			Phase				Phase		
Test	**Test Condition**	**Peak 1 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 2 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
1	Spreader beam mounted, but not bolted tight. Guard rails lying on deck near center and workers bolting curb on.	15.39	237.16	37.89	199.27	18.32	7.80	2.71	5.09
2	Workers using impact wrench on downstream curb.	15.33	238.34	44.32	194.03	18.38	12.66	4.36	8.30
3	Downstream curb tight. Still impact and pounding on bridge from workers.	15.10	229.83	44.99	184.83	18.42	10.47	3.51	6.97
4	Again drilling and pounding. Excavator part way on bridge.	15.17	107.60	72.75	180.34	21.67	15.32	67.26	51.94
5	Both upstream and downstream curbs tight. Two guard rail posts secure on downstream side. Again drilling and pounding and excavator part way on bridge.	15.43	98.49	142.95	241.45	18.89	12.89	80.34	67.45
6	Three guard rail posts secure. No outside activity.	15.32	99.97	102.83	202.80	18.38	0.19	2.60	2.80

Note:

- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- Shaded section data is used in summary Table 3.14.

Table 3.10. Vibration data collected for Bridge 53 on 6/23/2008 showing the frequency/phase relationship for peaks 3 and 4.

6/23/2008			Phase				Phase		
Test	**Test Condition**	**Peak 3 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 4 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
1	Spreader beam mounted, but not bolted tight. Guard rails lying on deck near center and workers bolting curb on.	28.38	1.64	142.77	141.13	-	-	-	-
2	Workers using impact wrench on downstream curb.	27.80	0.17	116.99	116.82	-	-	-	-
3	Downstream curb tight. Still impact and pounding on bridge from workers.	28.28	2.57	146.69	144.11	-	-	-	-
4	Again drilling and pounding. Excavator part way on bridge.	-	-	-	-	-	-	-	-
5	Both upstream and downstream curbs tight. Two guard rail posts secure on downstream side. Again drilling and pounding and excavator part way on bridge.	28.97	31.57	141.43	109.86	-	-	-	-
6	Three guard rail posts secure. No outside activity.	29.67	5.79	196.80	202.59	-	-	-	-

Note:

- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- Shaded section data is used in summary Table 3.14.

Table 3.11. Vibration data collected for Bridge 53 on 6/26/2008 showing the frequency/phase relationship for peaks 1 and 2.

6/26/2008			Phase				Phase		
Test	**Test Condition**	**Peak 1 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 2 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
2	Upstream rail secure. Spreader beam still not bolted. Motor mounted in center on top of bridge. Some materials on deck.	14.33	15.91	194.84	178.93	21.77	- 5.99	- 20.86	- 26.85
3		14.27	15.69	170.96	186.65	21.85	0.22	22.68	22.45
Mean		**14.30**				**21.81**			
4	Motor mounted in center on bottom of bridge. Downstream rail being installed.	14.31	29.75	162.30	192.04	19.85	24.40	22.42	1.98
5		14.57	36.03	217.13	181.10	20.04	34.73	24.34	10.39
6		14.24	5.77	190.94	185.17	21.65	65.87	1.53	67.40

Note:
- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- Shaded section data is used in summary Table 3.14.

Table 3.12. Vibration data collected for Bridge 53 on 6/26/2008 showing the frequency/phase relationship for peaks 3 and 4 if present.

6/26/2008			Phase				Phase		
Test	**Test Condition**	**Peak 3 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 4 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
2	Upstream rail secure. Spreader beam still not bolted. Motor mounted in center on top of bridge. Some materials on deck.	29.82	26.09	176.88	150.80	- -	- -	- -	- -
3		29.92	25.97	183.14	209.11	-	-	-	-
Mean		**29.87**				**--**			
4	Motor mounted in center on bottom of bridge. Downstream rail being installed.	27.65	5.08	162.31	157.23	34.92	57.67	132.62	190.29
5		27.20	4.25	201.19	196.94	-	-	-	-
6		29.75	7.69	169.69	177.38	-	-	-	-

Note:
- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- Shaded section data is used in summary Table 3.14.

Table 3.13. Vibration data collected for Bridge 53 on 7/10/2008 showing the frequency/phase relationship for peaks 1 and 2.

7/10/2008			Phase				Phase		
Test	**Test Condition**	**Peak 1 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 2 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
1	Bridge construction completed.	-	-	-	-	-	-	-	-
2		16.60	4.15	182.63	186.78	23.60	0.66	20.09	20.75
3		16.60	0.51	180.82	181.33	23.46	2.91	22.90	19.99
4		16.53	1.08	188.52	189.60	23.54	6.39	69.72	76.11
5		16.58	6.90	186.67	193.57	23.38	5.73	82.02	87.75
Mean		**16.58**				**23.50**			

Note:

- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- Shaded section data is used in summary Table 3.14.

Table 3.14. Vibration data collected for Bridge 53 on 7/10/2008 showing the frequency/phase relationship for peaks 3 and 4 if present.

7/10/2008			Phase				Phase		
Test	**Test Condition**	**Peak 3 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**	**Peak 4 (Hz)**	**Black/Blue**	**Red/Blue**	**Red/Black**
1	Bridge construction completed.	-	-	-	-	-	-	-	-
2		29.97	0.41	12.07	11.66	-	-	-	-
3		29.85	1.73	13.13	11.40	-	-	-	-
4		29.79	2.14	14.26	12.12	-	-	-	-
5		29.75	1.45	15.49	16.94	-	-	-	-
Mean		**29.84**							

Note:

- Areas marked with a dash indicate that there was no visible peak and therefore no phase recorded.
- Shaded section data is used in summary Table 3.14

Table 3.15. Summary of vibration data peaks that were captured during testing as Bridge 53 was constructed in Normanna Township, St. Louis County.

Bridge Construction		**Vibration Peak Measured (Hz)**			
Stage Number	Stage Description	1	2	3	4
1	Individual sections prior to installation	11.36			
2	Motor placed in the center of the first two sections, sections were not pinned together.	12.95	15.42	17.0	-
3	Lap joints pinned, motor parallel to the laminations, and placed in the center of the first two sections.	12.91	15.99	25.34	-
4	All five sections in place with lap joints pinned together, but not bolted to pile cap. Motor placed parallel to the laminations.	13.28	16.80	24.43	38.13
5	Spreader beam mounted, but not bolted tight. Guard rails lying on deck near center and workers bolting curb on.	15.39	18.32	28.38	-
6	Both upstream and downstream curbs tight. Two guard rail posts secure on downstream side. Drilling, pounding were present and excavator was located part way on bridge.	15.43	18.89	28.97	-
7	Three guard rail posts secure. No outside activity.	15.32	18.38	29.67	29.87
8	Upstream rail secure. Spreader beam still not bolted. Motor mounted in center on top of bridge. Some materials on deck.	14.30	21.81	29.87	-
9	Bridge construction completed.	16.58	23.50	29.84	-

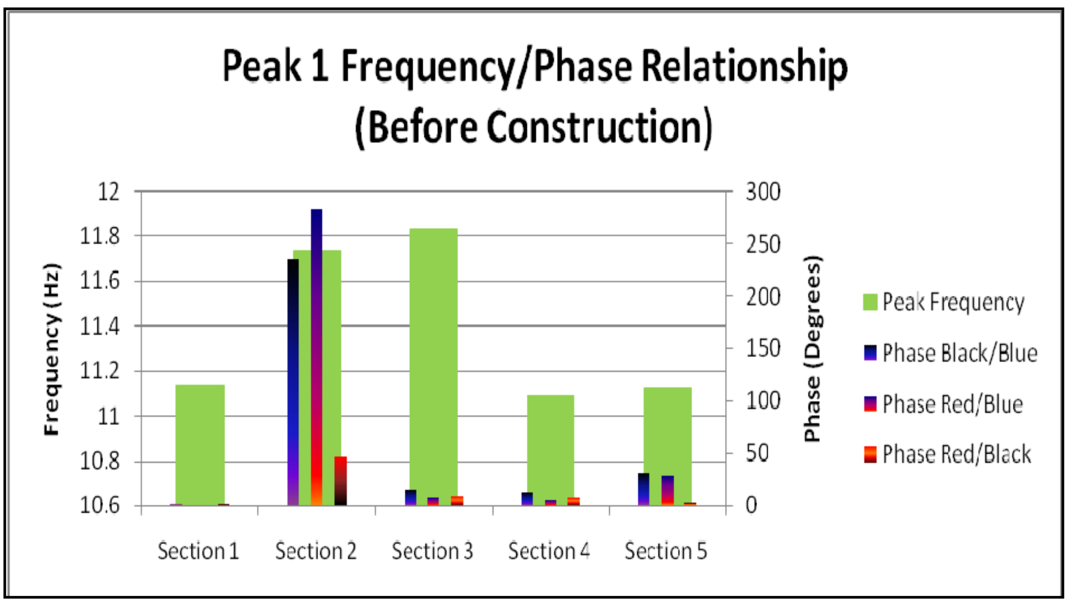


Figure 3.26. Frequency/phase relationship for the first peak of each section of Bridge 53 before construction.

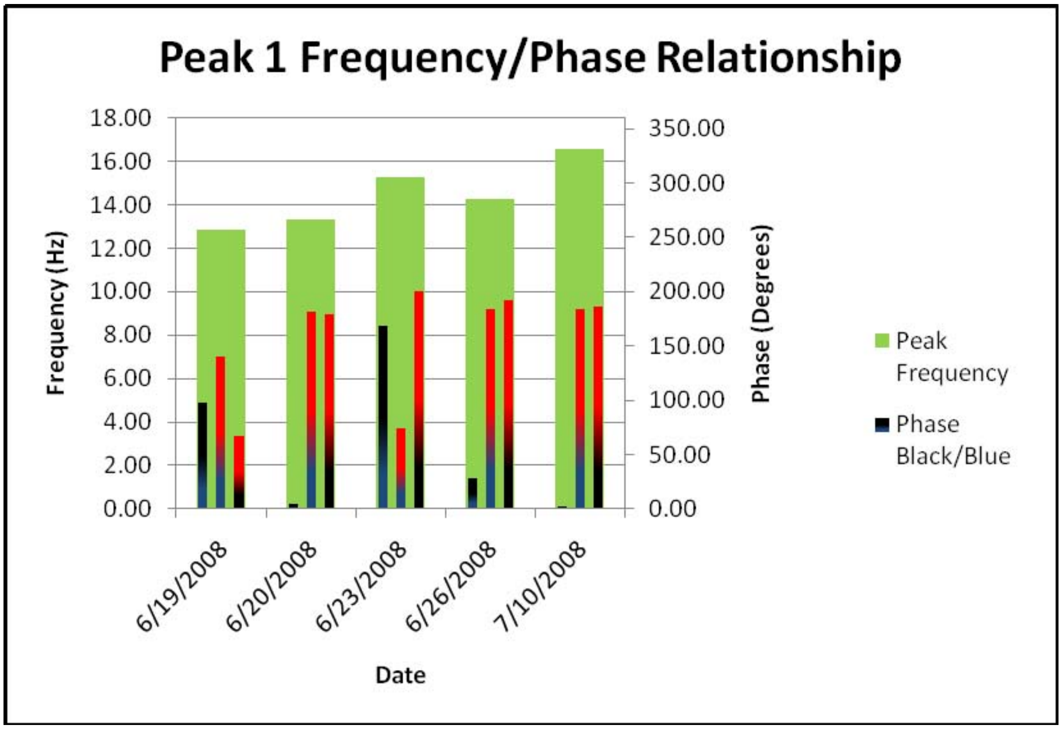


Figure 3.27. Average frequency/phase relationship for peak 1 for each day of construction.

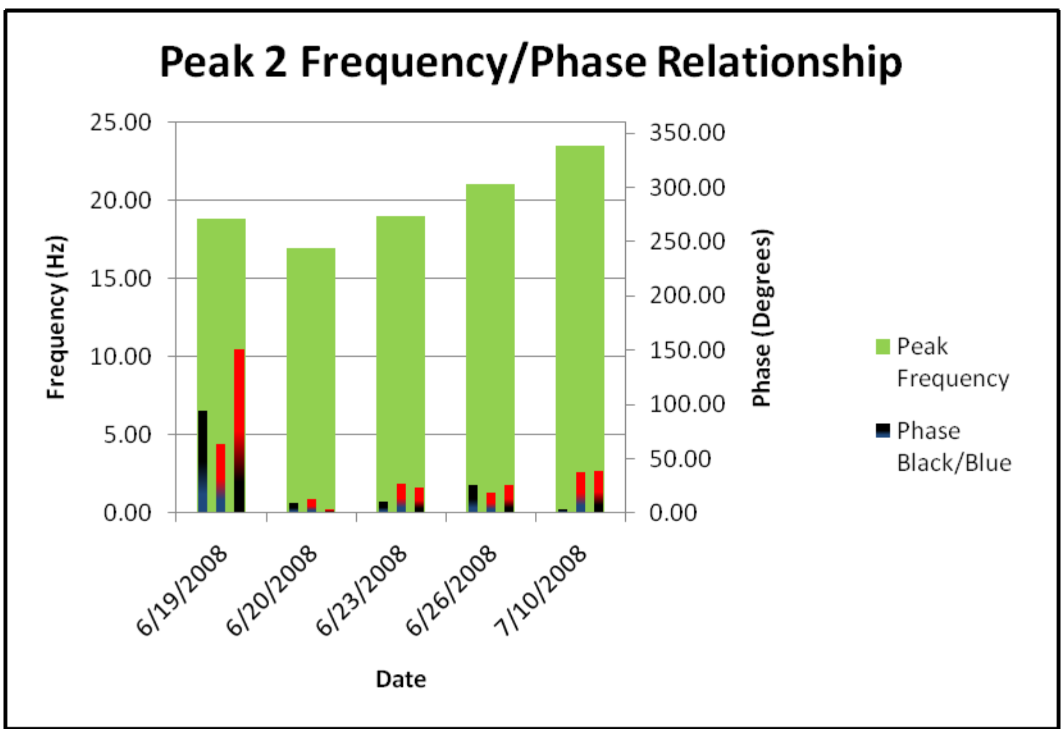


Figure 3.28. Average frequency/phase relationship for peak 2 for each day of construction.

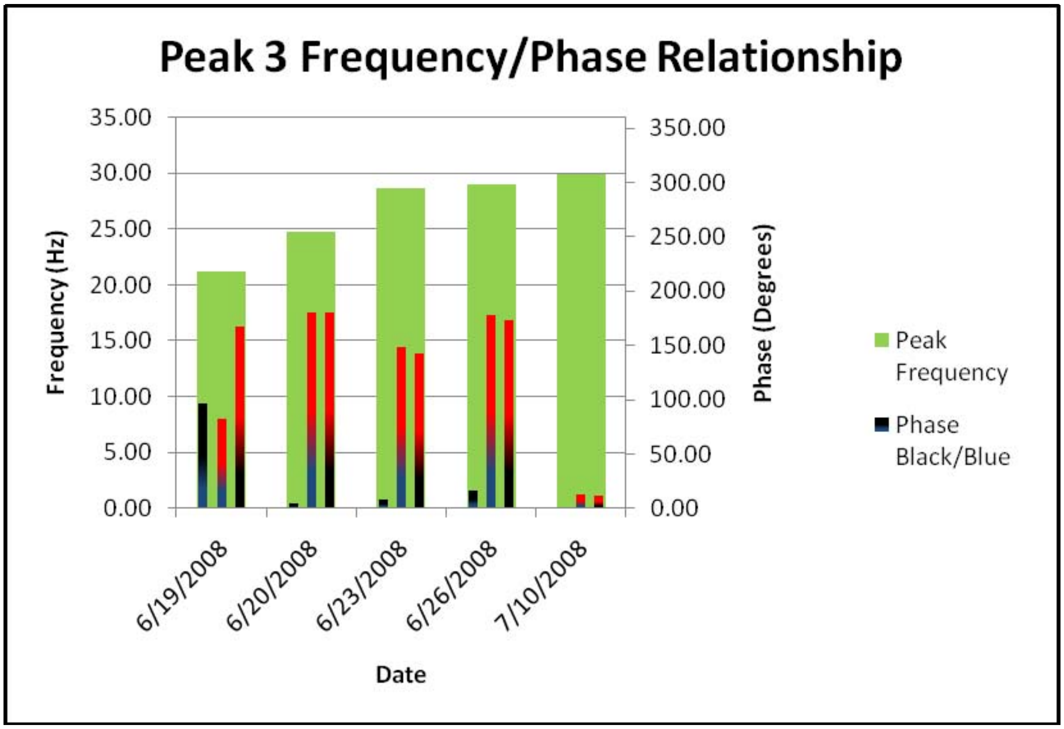


Figure 3.29. Average frequency/phase relationship for peak 3 for each day of construction.

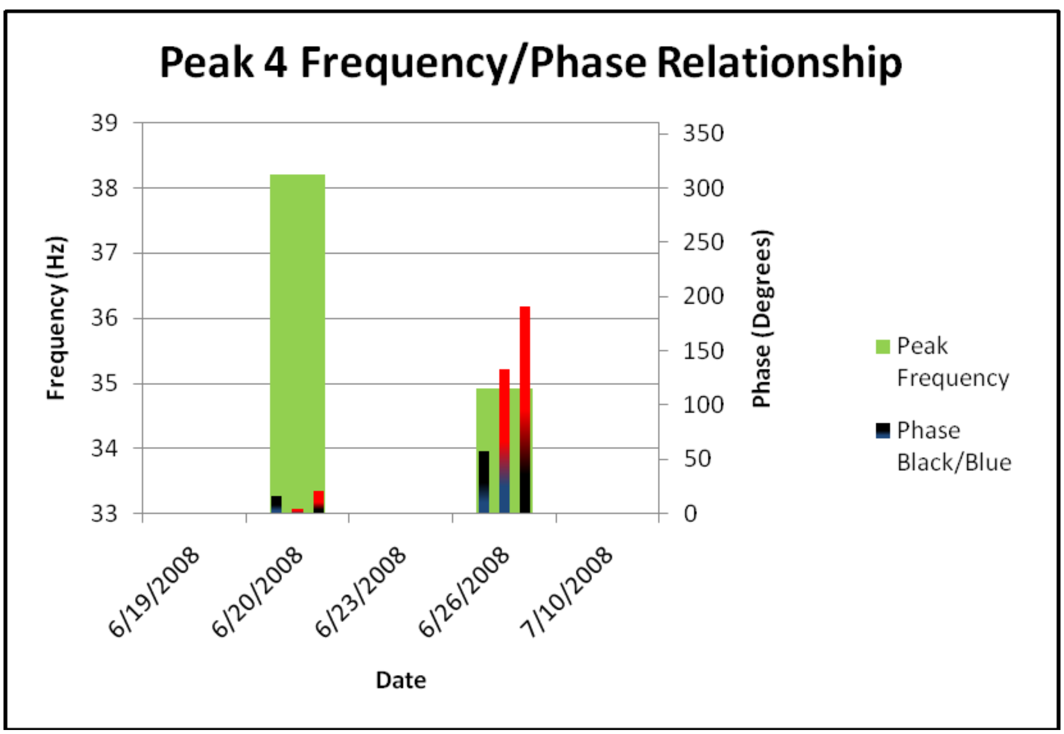


Figure 3.30. Average frequency/phase relationship for peak 4 for each day of construction.

Table 3.16. Static load data collected for Bridge 53.

Data Point	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1 Reading (mm)	Load Case No. 1 Deflection (mm)	Load Case No. 1 Deflection (in)	Load Case No. 2 Reading (mm)	Load Case No. 2 Deflection (mm)	Load Case No. 2 Deflection (in)	Load Case No. 3 Reading (mm)	Load Case No. 3 Deflection (mm)	Load Case No. 3 Deflection (in)
1	451	451	451.0	451	0.0	0.00	451	0.0	0.00	452	1.0	0.04
2	449	449	449.0	447	-2.0	-0.08	449	0.0	0.00	449	0.0	0.00
3	454	454	454.0	449	-5.0	-0.20	454	0.0	0.00	453	-1.0	-0.04
4	448	447	447.5	441	-6.5	-0.26	448	0.5	0.02	447	-0.5	-0.02
5	445	445	445.0	437	-8.0	-0.31	444	-1.0	-0.04	441	-4.0	-0.16
6	444	443	443.5	436	-7.5	-0.30	442	-1.5	-0.06	436	-7.5	-0.30
7	441	441	441.0	432	-9.0	-0.35	438	-3.0	-0.12	432	-9.0	-0.35
8	442	442	442.0	437	-5.0	-0.20	436	-6.0	-0.24	435	-7.0	-0.28
9	443	442	442.5	440	-2.5	-0.10	434	-8.5	-0.33	434	-8.5	-0.33
10	439	439	439.0	438	-1.0	-0.04	432	-7.0	-0.28	431	-8.0	-0.31
11	450	450	450.0	450	0.0	0.00	443	-7.0	-0.28	447	-3.0	-0.12
12	437	437	437.0	437	0.0	0.00	430	-7.0	-0.28	436	-1.0	-0.04
13	436	436	436.0	436	0.0	0.00	432	-4.0	-0.16	435	-1.0	-0.04
14	437	437	437.0	437	0.0	0.00	435	-2.0	-0.08	437	0.0	0.00
15	438	439	438.5	439	0.5	0.02	438	-0.5	-0.02	439	0.5	0.02
A	440	440	440.0	440	0.0	0.00	440	0.0	0.00	440	0.0	0.00
B	467	467	467.0	467	0.0	0.00	467	0.0	0.00	467	0.0	0.00
C	435	436	435.5	435	-0.5	-0.02	435	-0.5	-0.02	436	0.5	0.02
D	437	436	436.5	437	0.5	0.02	436	-0.5	-0.02	437	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 12 (downstream)
- Truck Weights: Gross Vehicle Weight = 51,670 lbs; Rear Axle Weight = 37,310 lbs

Chapter 4: Vibration and Load Testing of In-Service Wheeler 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.

Considering the geometric form of its cross section, a singlespan girder timber bridge may be modeled as a ribbed plate. 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).

$$f_{ij} = \frac{\pi}{2\gamma^{\frac{1}{2}}}\left[\frac{G_1^4 D_x}{a^4} + \frac{G_2^4 D_y}{b^4} + \frac{2H_1H_2D_{xy}}{a^2b^2} + \frac{4D_k(J_1J_2 - H_1H_2)}{a^2b^2}\right]^{\frac{1}{2}} \qquad [1]$$

Where:
i and *j* are mode number indexes,
γ is mass per unit area of deck,
G1, G2, J1, J2, H1, and *H2* are system parameters,
D is a cross-sectional constant,
a is bridge length, and
b is bridge width.

The system parameters in Equation (1) 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 (1):

$$f_i = \frac{\pi}{2\gamma^{\frac{1}{2}}}\left[\frac{G^4 EI_R}{L^4 a_1}\right]^{\frac{1}{2}} \qquad [2]$$

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.

Bridges Tested

Table 4.1 provides an overview of the dowel laminated timber bridges assessed for vibration and load testing during this project. Figure 4.1 shows a representative photo and cross-section of these bridges.

Table 4.1. List of bridges tested with brief summary of each bridge.

St. Louis County Bridge Number	Material Summary	Number of Spans	Year Constructed
53	Heavy timber pilings, Douglas fir girders/stringers, wood/asphalt deck	1	2008
182*	Heavy timber pilings, Douglas fir girders/stringers, wood/asphalt deck	3	1985
263	Heavy timber pilings, Douglas fir girders/stringers, wood/asphalt deck	1	2008
304*	Heavy timber pilings, Douglas fir girders/stringers, wood/asphalt deck	3	1992
313	Heavy timber pilings, Douglas fir girders/stringers, wood/asphalt deck	3	1982
383	Heavy timber pilings, Douglas fir girders/stringers, wood/asphalt deck	1	2003
402	Heavy timber pilings, Douglas fir girders/stringers, wood/asphalt deck	1	1985
568	Heavy timber pilings, Douglas fir girders/stringers, wood/asphalt deck	1	2000

* Bridge 304 and 182 have 3 spans, but the middle span was not accessible for testing.

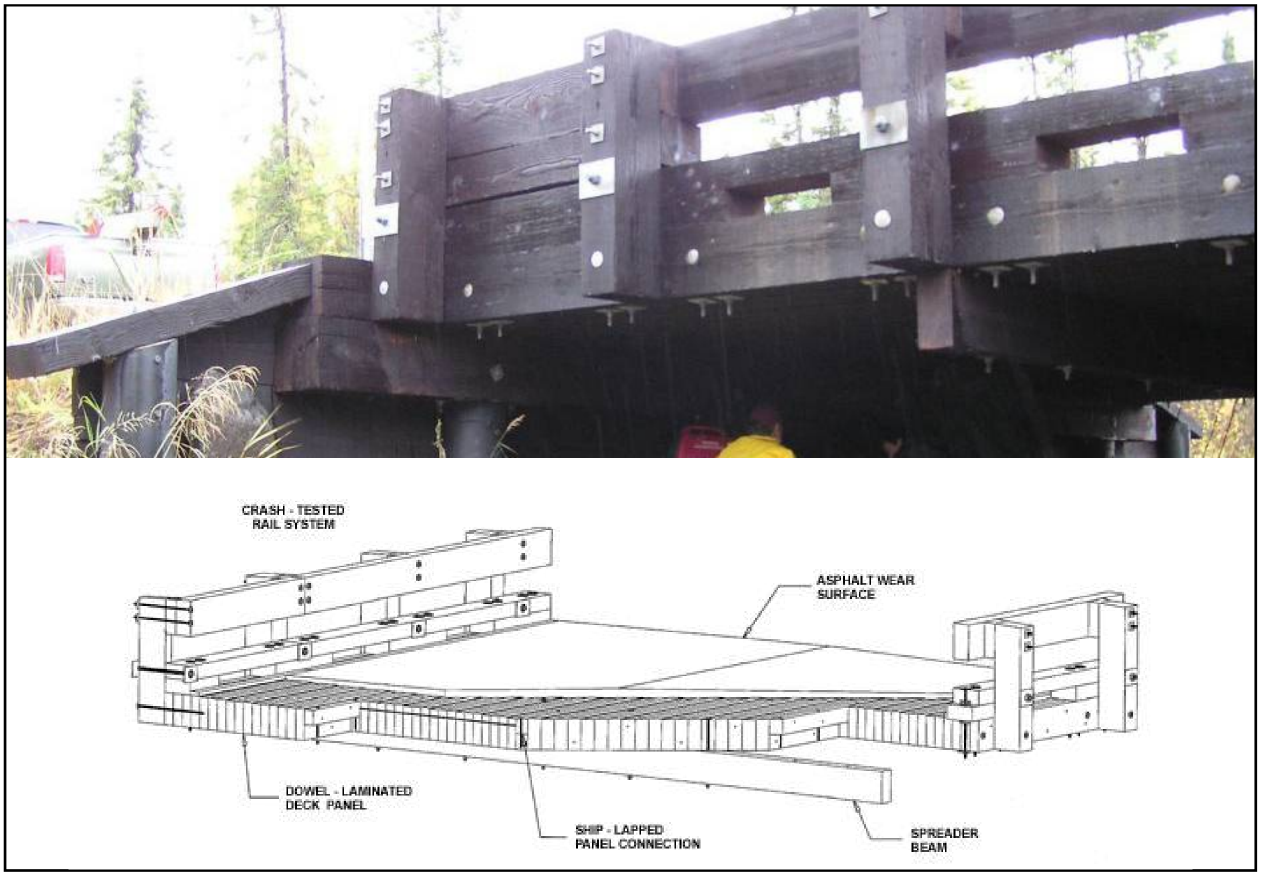


Figure 4.1. Photograph and associated cross-section schematic of a representative dowel laminated timber bridge that was tested during this project.

Procedures

Vibration test procedures for timber bridges

A forced vibration technique was used to identify the first bending mode frequency of the bridge structures. A ½ hp electric motor with a rotating unbalanced disc was attached to the mid-point of the structure. Three piezoelectric accelerometers (PCB 626BO2), also at midspan, were magnetically attached to steel plates attached using lag screws to the outer edges and the midpoint. The motor was controlled and the data collected using a laptop notebook with a customized program developed using Labview Windows. The motor slowly increases in RPM and the resulting vibrational frequency is collected. The amplitude of the frequency response is plotted to identify frequency peaks that occur during testing.

The specific testing steps included:

1. Secure a dc motor (1/2 horsepower) with rotating unbalanced disc to the deck at the center of the span and anchor using steel bolt screws.
2. Secure three metal plates to deck plank, one on each side of the span and one in the center near the motor at midspan using steel lag screws.

3. Attach one magnetic piezoelectric accelerometer to each metal plate to monitor the bridge vibration signals.
4. Connect motor and accelerometer to laptop computer as noted in Figure 2.7 that details the computer testing process.
5. Record the data and start a second test.
6. 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 for the motor attached midspan under the bridge is shown in Figure 4.2. The in testing computer interface that controls the testing and acquires data is shown Figure 4.3 and the data collection accelerometers are shown as attached in Figure 4.4.

Figure 4.2. General motor attachment on underside of bridge deck for conducting vibration testing.

Figure 4.3. Computer controlled interface to control vibration testing and collect vibration characteristics.

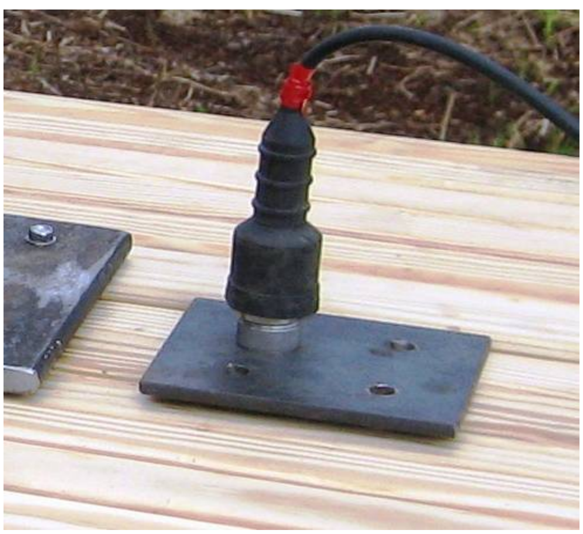


Figure 4.4. Accelerometer attached to steel plate to monitor vibration characteristics of bridge during testing.

Static Load Testing

Because the primary goal of this work was to relate the vibrational characteristics of these timber bridge structures to a measure of structural integrity, the bridge was 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 the truck used prior to testing. The axle spacing was also measured for the 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 plate 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 are resting squarely on piles.
4. Do not conduct bridge test if safety is concern.

Placement of Optical Surveying Level

1. Look for a suitable location on solid ground. Do not place the level 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 bridges, use center loading (with wheel lines straddling roadway centerline) with marks at 3 ft on each side of centerline.
    b. For double lane bridges, 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.

Typical Sequence

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 the bridge location. 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 4.5, 4.6, and 4.7.

Figure 4.5. Truck positioned on bridge.

Figure 4.6. Deflection rulers.

Figure 4.7. 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 40 lb/ft^3 (641 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. A weight unit of 150 lb/ft^3 (2403 kg/m^3) was used for the asphalt wear layer. The estimated weight for each bridge is shown in the following individual bridge testing summary section.

Bridge 53 Testing Summary

Background

Structure: Bridge 53

Location: Normanna Township, St. Louis County, Minnesota

Special Consideration(s): New bridge

Year Built: 2008

Inspection date: August 2008

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

Photos:

Figure 4.8. Bridge 53.

Figure 4.9. Bridge 53.

Figure 4.10. Bridge 53.

Bridge dimensions

Table 4.2. Bridge dimensions for Bridge 53.

Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	46	8	11.5	10	24.5	979.6
Spacer	8	10.5	5	10	2.4	97.2
Spacer	78.5	5	13.25	1	3.0	120.4
Railing	264	6.75	14	2	28.9	1,155.0
Curb	264	5.75	11.5	2	20.2	808.2
Spreader	363.5	12	6	1	15.1	605.8
Deck	264	363.5	13.75	1	902.4	36,097.6
Asphalt	264	363.5	3	1	166.6	24,990.6
					Total	79,201

Note: [1]A factor of 40 lbs/ft^3 was used for wood and 150 lbs/ft^3 for asphalt.

Vibration test data

Table 4.3. Vibration data collected from Bridge 53 for span 1.

Span 1												
		Phase				Phase				Phase		
Test	**Peak 1 Freq. (Hz)**	**Black/ Blue**	**Red/ Blue**	**Red/ Black**	**Peak 2 Freq. (Hz)**	**Black/ Blue**	**Red/ Blue**	**Red/ Black**	**Peak 3 Freq. (Hz)**	**Black/ Blue**	**Red/ Blue**	**Red/ Black**
1	-	-	-	-	-	-	-	-	-	-	-	-
2	16.60	4.15	182.63	186.78	23.60	0.66	20.09	20.75	29.97	0.41	12.07	11.66
3	16.60	0.51	180.82	181.33	23.46	2.91	22.90	19.99	29.85	1.73	13.13	11.40
4	16.53	1.08	188.52	189.60	23.54	6.39	69.72	76.11	29.79	2.14	14.26	12.12
5	16.58	6.90	186.67	193.57	23.38	5.73	82.02	87.75	29.75	1.45	15.49	16.94
Mean	**16.58**	**3.16**	**184.66**	**187.82**	**23.50**	**3.92**	**48.68**	**51.15**	**29.84**	**1.43**	**13.74**	**13.03**

Note: Hz = hertz; black is the downstream, red is the midspan and blue is the upstream accelerometer locations.

Static load test data

Table 4.4. Static load data collected from Bridge 53 for span 1.

Distance from Upstream (in.)	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)
4.5	451	451	451.0	451	0.0	0.00	451	0.0	0.00	452	1.0	0.04
33.875	449	449	449.0	447	-2.0	-0.08	449	0.0	0.00	449	0.0	0.00
63.25	454	454	454.0	449	-5.0	-0.20	454	0.0	0.00	453	-1.0	-0.04
72.25	448	447	447.5	441	-6.5	-0.26	448	0.5	0.02	447	-0.5	-0.02
101.25	445	445	445.0	437	-8.0	-0.31	444	-1.0	-0.04	441	-4.0	-0.16
130.25	444	443	443.5	436	-7.5	-0.29	442	-1.5	-0.06	436	-7.5	-0.29
139.25	441	441	441.0	432	-9.0	-0.35	438	-3.0	-0.12	432	-9.0	-0.35
172.375	442	442	442.0	437	-5.0	-0.20	436	-6.0	-0.24	435	-7.0	-0.28
205.5	443	442	442.5	440	-2.5	-0.10	434	-8.5	-0.33	434	-8.5	-0.33
214.5	439	439	439.0	438	-1.0	-0.04	432	-7.0	-0.28	431	-8.0	-0.31
249.875	450	450	450.0	450	0.0	0.00	443	-7.0	-0.28	447	-3.0	-0.12
285.25	437	437	437.0	437	0.0	0.00	430	-7.0	-0.28	436	-1.0	-0.04
294.25	436	436	436.0	436	0.0	0.00	432	-4.0	-0.16	435	-1.0	-0.04
327.375	437	437	437.0	437	0.0	0.00	435	-2.0	-0.08	437	0.0	0.00
360.5	438	439	438.5	439	0.5	0.02	438	-0.5	-0.02	439	0.5	0.02
AVG.	443.6	443.5	443.5	440.5	-3.1	-0.1	440.4	-3.1	-0.1	440.3	-3.3	-0.1
A	440	440	440	440	0.0	0.00	440	0.0	0.00	440	0.0	0.00
B	467	467	467.0	467	0.0	0.00	467	0.0	0.00	467	0.0	0.00
C	435	436	435.5	435	-0.5	-0.02	435	-0.5	-0.02	436	0.5	0.02
D	437	436	436.5	437	0.5	0.02	436	-0.5	-0.02	437	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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 51,670 lbs; Rear Axle Vehicle Weight = 37,310 lbs

Results

Results obtained from vibration and static load testing for Bridge 53 are shown below.

Vibration frequency

Figure 4.11 shows a representative graph of the frequency data, with the following average peak frequencies; peak 1 = 16.58 Hz, peak 2 = 23.50 Hz, and peak 3 = 29.84 Hz. The midspan accelerometer did not record any frequency data during this test. A review of the black and blue accelerometers showed that they were in phase during all three peaks, indicating that the directional movement was at the same time and in the same direction.

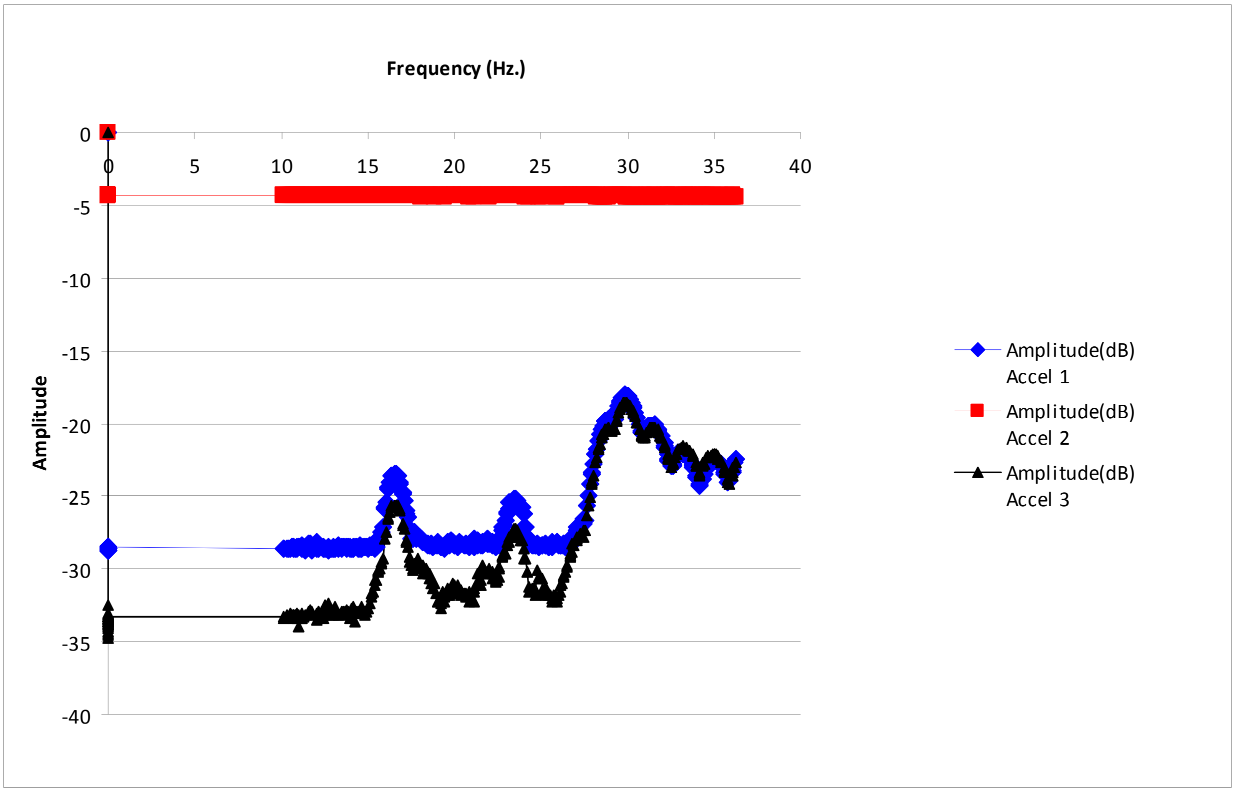


Figure 4.11. A representative frequency pattern for St. Louis County Bridge 53.

Live load

Figure 4.12 shows the deflection of span 1 of Bridge 53 for center loading.

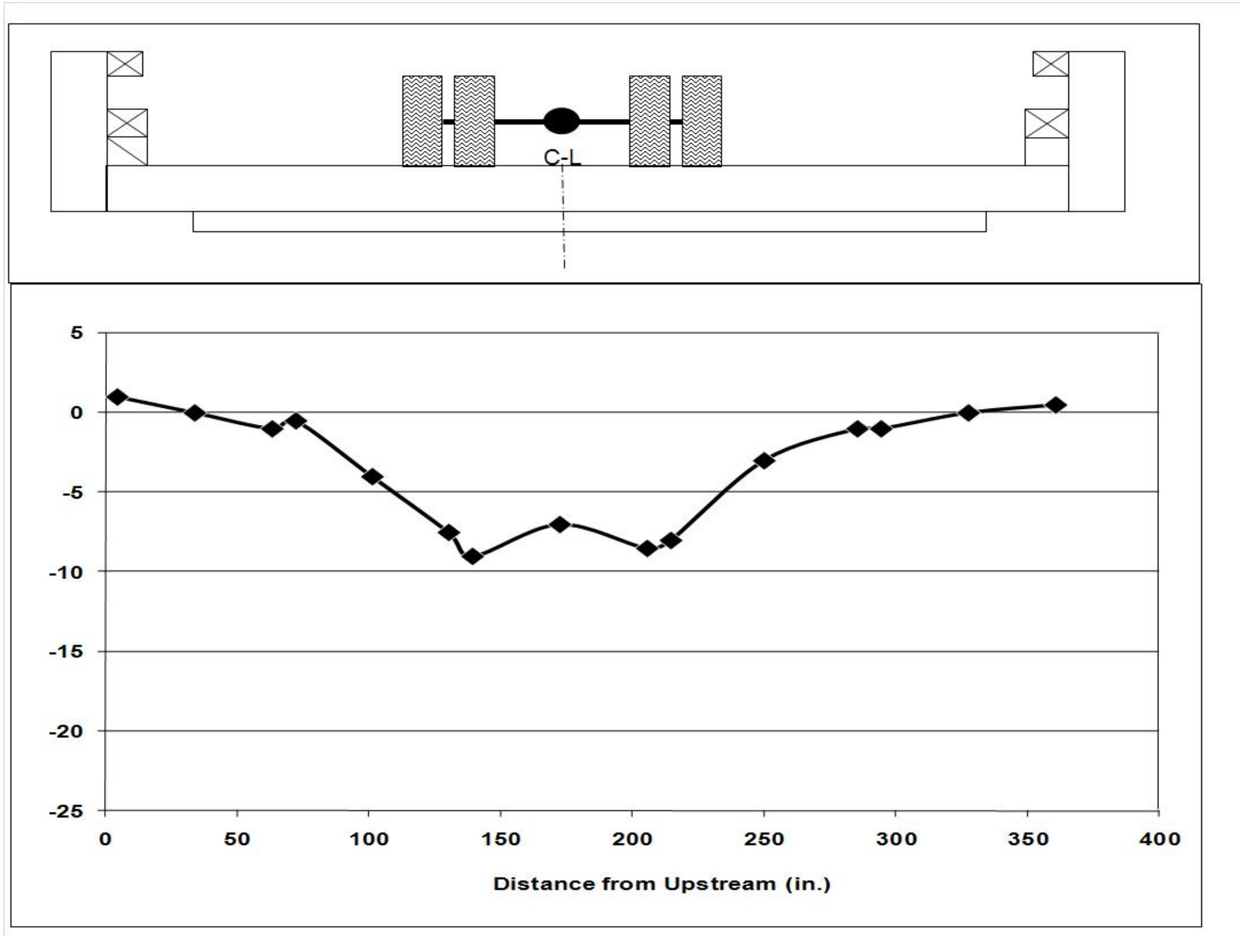


Figure 4.12. Deflection of span 1 of Bridge 53 for center loading.

Based on the load deflection testing, the following bridge stiffness was calculated as shown in Table 4.5.

Table 4.5. Measured deflection and stiffness for Bridge 53.

Bridge No.	Span No.	Span (center to center - ft)	Deflection		Measured Stiffness	
			Average (in.)	Maximum (in.)	Average (kip-in^2 x 10^6)	Maximum (kip-in^2 x 10^6)
53	1	23.00	0.13	0.35	119.44	44.36

Bridge 182 Testing Summary

<u>Background</u>

Structure:	Bridge 182
Location:	Van Buren, Minnesota
Special Consideration(s):	Large Bridge – 3 Spans
Year Built:	1985
Inspection date:	August 2008
Construction details:	Three spans, heavy timber pilings with Douglas fir girders/stringers and an asphalt deck for a running surface

Bridge Photos:

Figure 4.13. Bridge 182.

Figure 4.14. Bridge 182.

Figure 4.15. Bridge 182.

Bridge dimensions

Table 4.6. Bridge dimensions for Bridge 182 span 1 and span 2.

Span 1 Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	46	8	12	12	30.7	1,226.7
Spacer	103.2	6	12	2	8.6	344.0
Spacer	36	6	12	4	6.0	240.0
Railing	361	6	12	2	30.1	1,203.3
Curb	361	6	12	2	30.1	1,203.3
Spreader	394	6	12	1	16.4	656.7
Deck	361	394	3	1	246.9	9,877.4
Asphalt	361	394	3	1	246.9	37,040.1
					Total	**55,479.3**
Span 2 Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	46	8	12	12	30.7	1,226.7
Spacer	103.2	6	12	2	8.6	344.0
Spacer	36	6	12	4	6.0	240.0
Railing	366	6	12	2	30.5	1,220.0
Curb	366	6	12	2	30.5	1,220.0
Spreader	394	6	12	1	16.4	656.7
Deck	366	394	3	1	250.4	10,014.2
Asphalt	366	394	3	1	250.4	37,553.1
					Total	**56,205.0**

Note: [1]A factor of 40 lbs/ft^3 was used for wood and 150 lbs/ft^3 for asphalt.

Vibration test data

Table 4.7. Vibration data collected from Bridge 182 for span 1.

Span 1												
		Phase				Phase				Phase		
Test	**Peak 1 Freq. (Hz)**	**Black/ Blue**	**Red/ Blue**	**Red/ Black**	**Peak 2 Freq. (Hz)**	**Black/ Blue**	**Red/ Blue**	**Red/ Black**	**Peak 3 Freq. (Hz)**	**Black/ Blue**	**Red/ Blue**	**Red/ Black**
1	13.01	-	-	-	23.3	-	-	-	35.2	-	-	-
2												
Mean												

Note: Hz = hertz

Table 4.8. Vibration data collected from Bridge 182 for span 2.

Span 2												
		Phase				Phase				Phase		
Test	**Peak 1 Freq. (Hz)**	**Black/ Blue**	**Red/ Blue**	**Red/ Black**	**Peak 2 Freq. (Hz)**	**Black/ Blue**	**Red/ Blue**	**Red/ Black**	**Peak 3 Freq. (Hz)**	**Black/ Blue**	**Red/ Blue**	**Red/ Black**
1	14.39	-	-	-	30.16	-	-	-	36.98	-	-	-
2												
Mean												

Note: Hz = hertz

Static load test data

Table 4.9. Static load data collected from Bridge 182 for span 1.

Distance from Upstream (in.)	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)
2	315	317	316.0	315	-1.0	-0.04	316	0.0	0.00	316	0.0	0.00
47	317	318	317.5	314	-3.5	-0.14	317	-0.5	-0.02	317	-0.5	-0.02
84	320	321	320.5	310	-10.5	-0.41	320	-0.5	-0.02	319	-1.5	-0.06
100	321	322	321.5	307	-14.5	-0.57	321	-0.5	-0.02	318	-3.5	-0.14
130	328	329	328.5	314	-14.5	-0.57	327	-1.5	-0.06	322	-6.5	-0.26
160	329	329	329.0	315	-14.0	-0.55	327	-2.0	-0.08	316	-13.0	-0.51
176.5	329	329	329.0	312	-17.0	-0.67	325	-4.0	-0.16	312	-17.0	-0.67
208.5	325	325	325.0	314	-11.0	-0.43	315	-10.0	-0.39	312	-13.0	-0.51
237	330	329	329.5	325	-4.5	-0.18	312	-17.5	-0.69	312	-17.5	-0.69
253	328	327	327.5	325	-2.5	-0.10	313	-14.5	-0.57	311	-16.5	-0.65
285.5	326	326	326.0	324	-2.0	-0.08	311	-15.0	-0.59	318	-8.0	-0.31
314	324	323	323.5	323	-0.5	-0.02	306	-17.5	-0.69	318	-5.5	-0.22
330	321	320	320.5	320	-0.5	-0.02	308	-12.5	-0.49	317	-3.5	-0.14
370.5	320	319	319.5	320	0.5	0.02	314	-5.5	-0.22	318	-1.5	-0.06
410	316	316	316.0	317	1.0	0.04	316	0.0	0.00	316	0.0	0.00
AVG.	323.3	323.3	323.3	317.0	-6.3	-0.2	316.5	-6.8	-0.3	316.1	-7.2	-0.3
A	369	368	368.5	369	0.5	0.02	369	0.5	0.02	366	-2.5	-0.10
B	312	316	314.0	316	2.0	0.08	316	2.0	0.08	316	2.0	0.08
C												
D	313	313	313.0	313	0.0	0.00	313	0.0	0.00	313	0.0	0.00

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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 51,670 lbs; Rear Axle Vehicle Weight = 37,310 lbs

Table 4.10. Static load data collected from Bridge 182 for span 2.

Distance from Upstream (in.)	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)
2	280	282	281.0	281	0.0	0.00	282	1.0	0.04	282	1.0	0.04
47	294	294	294.0	289	-5.0	-0.20	295	1.0	0.04	293	-1.0	-0.04
84	297	298	297.5	286	-11.5	-0.45	297	-0.5	-0.02	294	-3.5	-0.14
100	293	294	293.5	278	-15.5	-0.61	292	-1.5	-0.06	288	-5.5	-0.22
130	293	295	294.0	278	-16.0	-0.63	292	-2.0	-0.08	286	-8.0	-0.31
160	299	299	299.0	284	-15.0	-0.59	296	-3.0	-0.12	285	-14.0	-0.55
176.5	307	307	307.0	291	-16.0	-0.63	302	-5.0	-0.20	290	-17.0	-0.67
208.5	310	310	310.0	299	-11.0	-0.43	298	-12.0	-0.47	296	-14.0	-0.55
237	311	311	311.0	306	-5.0	-0.20	295	-16.0	-0.63	295	-16.0	-0.63
253	315	315	315.0	312	-3.0	-0.12	300	-15.0	-0.59	301	-14.0	-0.55
285.5	316	317	316.5	315	-1.5	-0.06	302	-14.5	-0.57	309	-7.5	-0.29
314	320	320	320.0	320	0.0	0.00	306	-14.0	-0.55	316	-4.0	-0.16
330	316	317	316.5	317	0.5	0.02	306	-10.5	-0.41	314	-2.5	-0.10
370.5	323	324	323.5	324	0.5	0.02	322	-1.5	-0.06	323	-0.5	-0.02
410	319	319	319.0	320	1.0	0.04	319	0.0	0.00	320	1.0	0.04
AVG.	308.1	308.6	308.3	301.4	-7.0	-0.3	301.6	-6.8	-0.3	300.7	-7.6	-0.3
A	323	325	324.0	324	0.0	0.00	326	2.0	0.08	324	0.0	0.00
B	363	364	363.5	364	0.5	0.02	364	0.5	0.02	364	0.5	0.02
C	366	367	366.5	366	-0.5	-0.02	366	-0.5	-0.02	366	-0.5	-0.02
D	297	298	297.5	298	0.5	0.02	299	1.5	0.06	298	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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 51,670 lbs; Rear Axle Vehicle Weight = 37,310 lbs

Results

Results obtained from vibration and static load testing for Bridge 182 are shown below.

Vibration frequency

Figure 4.16 shows a representative graph of the frequency data, with the following average peak frequencies for span 1; peak 1 = 13.01 Hz, peak 2 = 23.3 Hz, and peak 3 = 35.2 Hz. Figure 4.17 shows a representative graph of the frequency data, with the following average peak frequencies for span 2; peak 1 = 14.39 Hz, peak 2 = 30.16 Hz, and peak 3 = 36.8 Hz. Both of these graphs have peaks that are less defined than other bridges that were tested. It is believed that this is due to the fact that this bridge was significantly larger (3 spans) and had more mass than other bridges evaluated. The signal may have been damped by the large mass and the signal not clearly captured with the accelerometers used in this project.

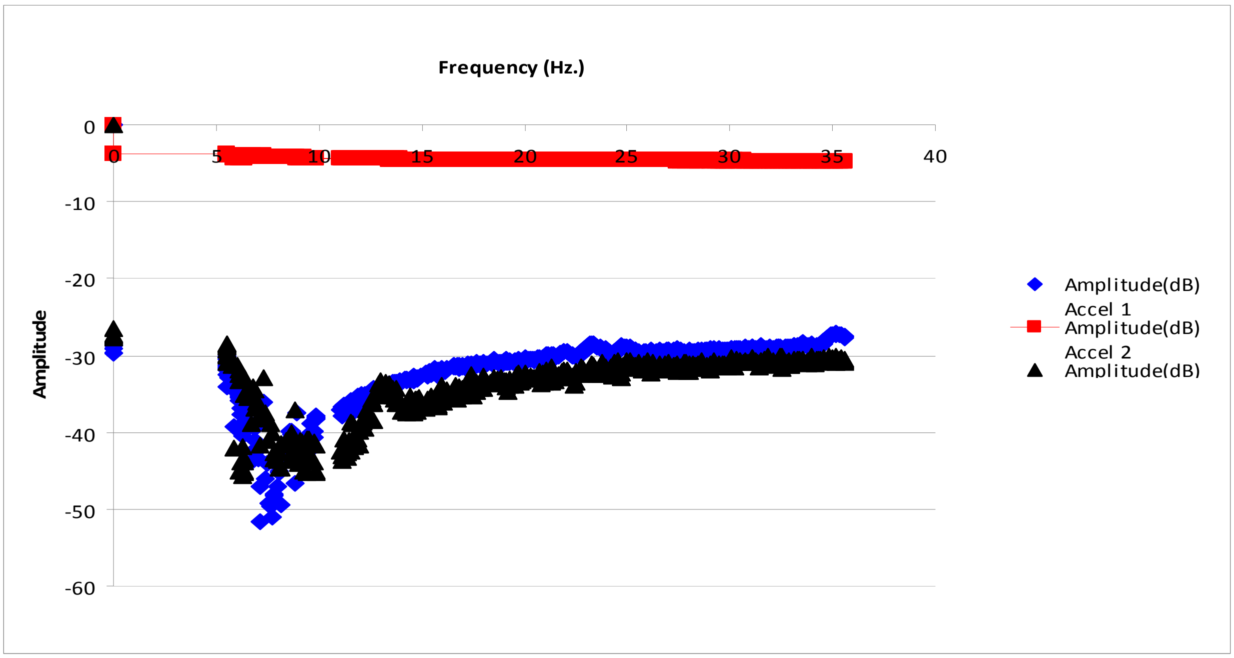


Figure 4.16. A representative frequency pattern for St. Louis County Bridge 182 span 1.

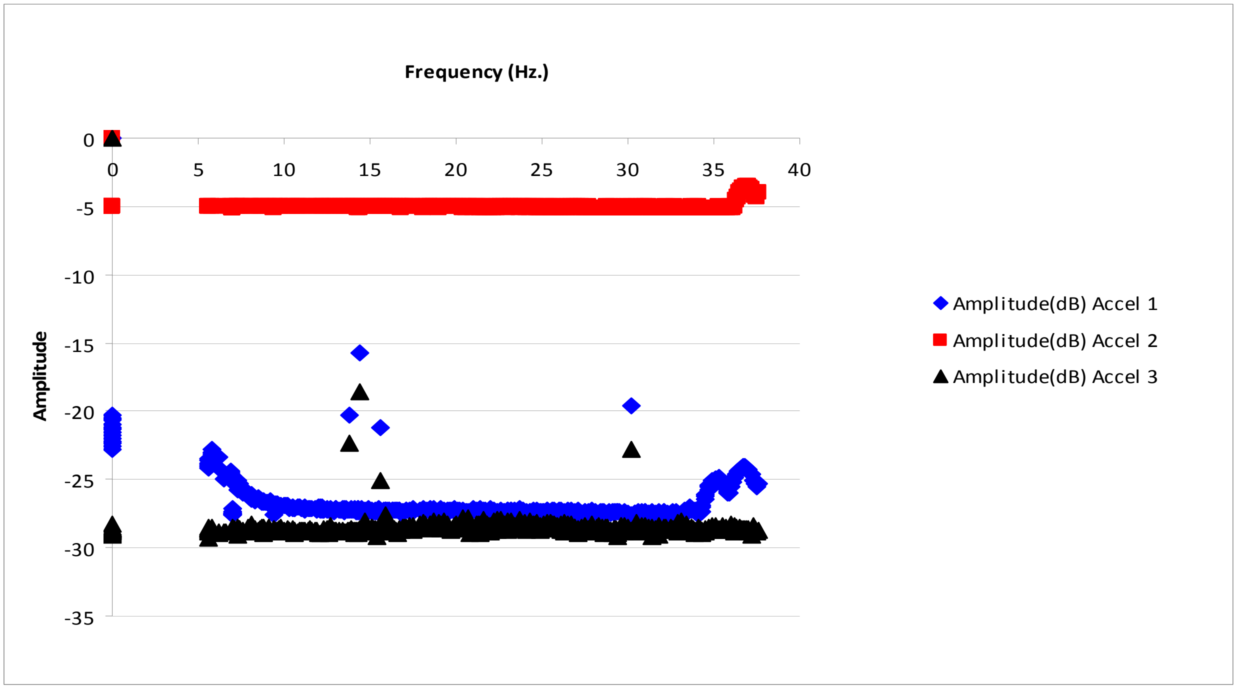


Figure 4.17. A representative frequency pattern for St. Louis County Bridge 182 span 2.

Live load

Figures 4.18 and 4.19 show the deflections of spans 1 and 2 of Bridge 182 for center loading.

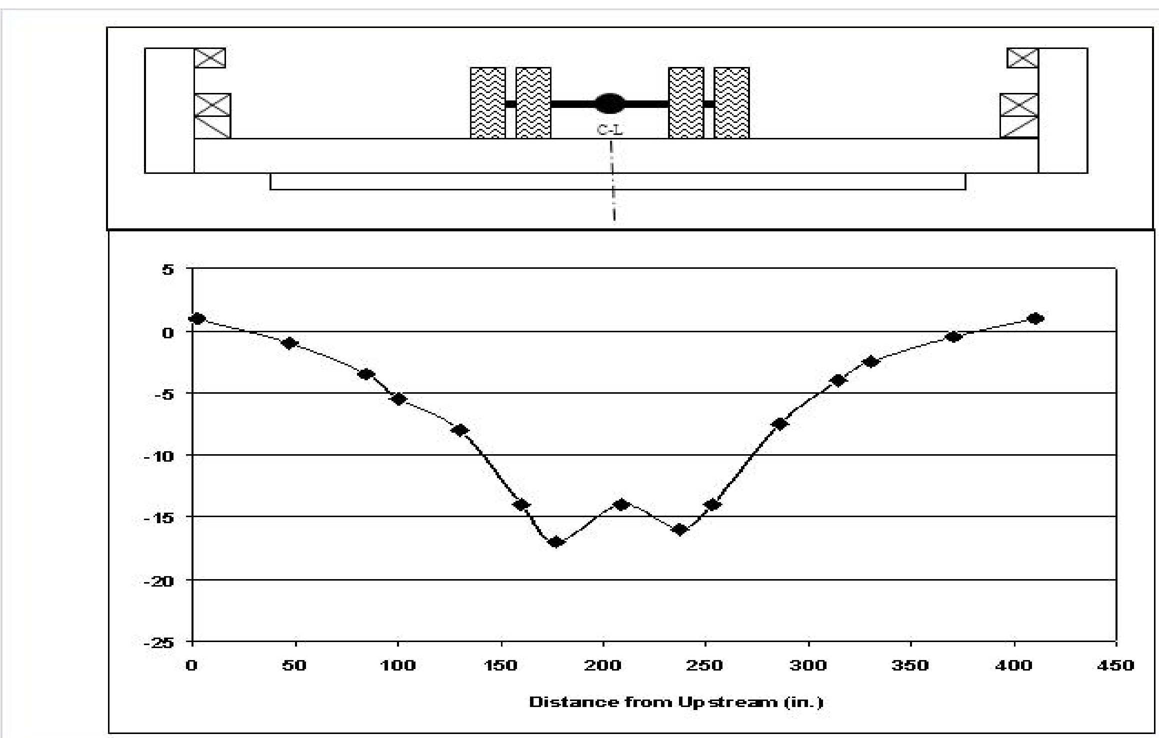


Figure 4.18. Deflection of span 1 of Bridge 182 for center loading.

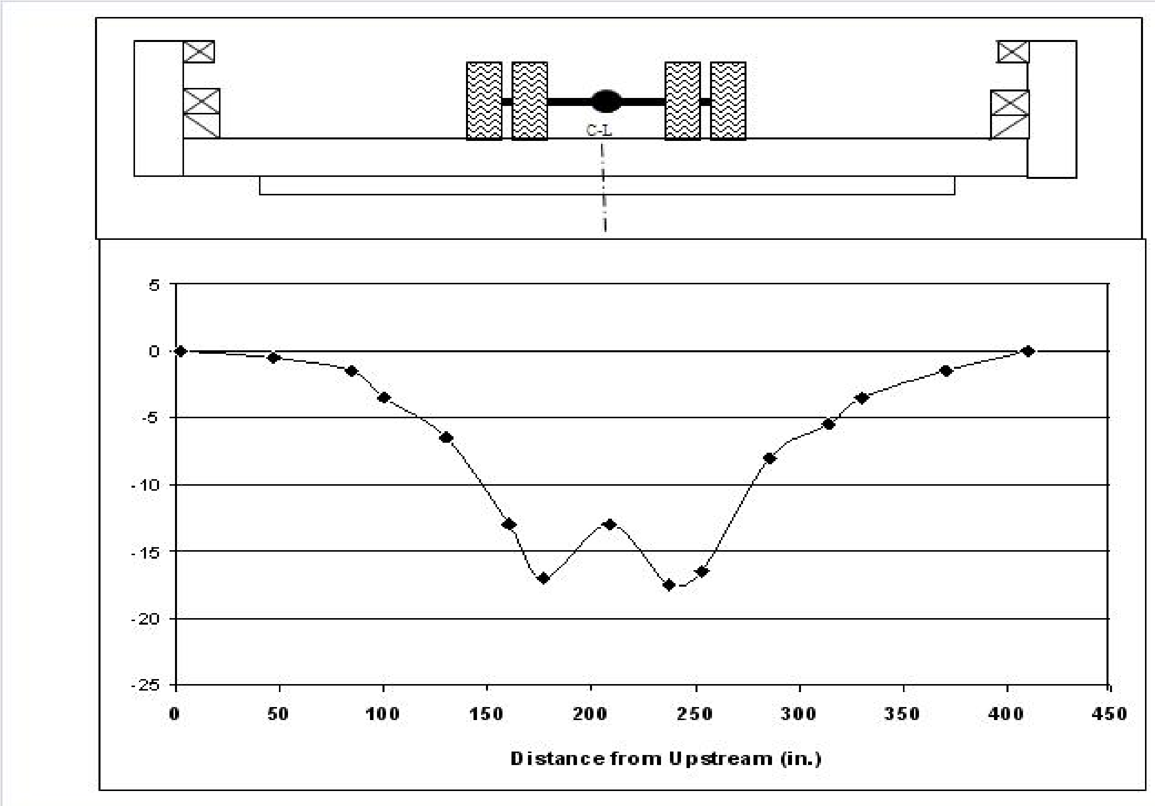


Figure 4.19. Deflection of span 2 of Bridge 182 for center loading.

Based on the load deflection testing, the following bridge stiffness was calculated as shown in Table 4.10.

Table 4.11. Measured deflection and stiffness for Bridge 182.

Bridge No.	Span No.	Span (center to center - ft)	Deflection		Measured Stiffness	
			Average (in.)	Maximum (in.)	Average (kip-in^2 x 10^6)	Maximum (kip-in^2 x 10^6)
182	1	31.08	0.28	0.67	140.02	58.52
182	2	31.50	0.28	0.69	145.88	59.20

Bridge 263 Testing Summary

Background

Structure:	Bridge 263
Location:	Meadowlands, Minnesota
Special Consideration(s):	New bridge
Year Built:	2008
Inspection date:	August 2008
Construction details:	Single span, dowel laminated Wheeler Bridge with an asphalt deck for a running surface

Bridge Photos:

Figure 4.20. Bridge 263.

Figure 4.21. Bridge 263.

Figure 4.22. Bridge 263.

Bridge dimensions

Table 4.12. Bridge dimensions for Bridge 263.

Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	46	8	12	8	20.4	817.8
Spacer	10.5	5.5	8	8	2.1	85.6
Railing	198	6	10.5	2	14.4	577.5
Curb	198	5.5	11.5	2	14.5	579.8
Spreader	315.5	6	12	1	13.1	525.8
Deck	198	315.5	12	1	433.8	17,352.5
Asphalt	198	315.5	3	1	108.5	16,268.0
					Total	**41,191.7**

Note: [1]A factor of 40 lbs/ft^3 was used for wood and 150 lbs/ft^3 for asphalt.

Vibration test data

Table 4.13. Vibration data collected from Bridge 263.

Test	Peak 1 Frequency (Hz)	Phase			Peak 2 Frequency (Hz)	Phase		
		Black/Blue	Red/Blue	Red/Black		Black/Blue	Red/Blue	Red/Black
1	23.24	13.45	176.78	163.33	34.32	5.58	2.72	2.85
2	23.06	13.58	187.71	201.28	34.50	9.03	5.02	4.01
3	23.12	13.72	173.04	159.33	34.47	6.01	4.09	1.93
Average	23.14	13.58	179.18	174.65	34.43	6.87	3.94	2.93

Note: Hz = hertz

Static load test data

Table 4.14. Static load data collected from Bridge 263 for span 1.

Distance from Upstream (in.)	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)
4.5	401	401	401.0	400	-1.0	-0.04	401	0.0	0.00	401	0.0	0.00
41.625												
78.75	406	406	406.0	400	-6.0	-0.24	406	0.0	0.00	405	-1.0	-0.04
87.75	404	404	404.0	400	-4.0	-0.16	404	0.0	0.00	401	-3.0	-0.12
123	406	406	406.0	401	-5.0	-0.20	405	-1.0	-0.04	400	-6.0	-0.24
158.25	414	414	414.0	410	-4.0	-0.16	412	-2.0	-0.08	410	-4.0	-0.16
167.25	407	407	407.0	405	-2.0	-0.08	403	-4.0	-0.16	403	-4.0	-0.16
199.625	407	406	406.5	406	-0.5	-0.02	401	-5.5	-0.22	400	-6.5	-0.26
232	412	412	412.0	412	0.0	0.00	408	-4.0	-0.16	410	-2.0	-0.08
241	413	413	413.0	413	0.0	0.00	408	-5.0	-0.20	411	-2.0	-0.08
274.25	414	414	414.0	414	0.0	0.00	410	-4.0	-0.16	413	-1.0	-0.04
307.5	414	414	414.0	414	0.0	0.00	413	-1.0	-0.04	415	1.0	0.04
AVG.	408.9	409.2	408.9	406.8	-2.0	-0.1	406.5	-2.4	-0.1	406.3	-2.6	-0.1
A	399	399	399	398	-1.0	-0.04	399	0.0	0.00	399	0.0	0.00
B	399	399	399.0	398	-1.0	-0.04	399	0.0	0.00	399	0.0	0.00
C	415	415	415.0	415	0.0	0.00	415	0.0	0.00	416	1.0	0.04
D	421	421	421.0	421	0.0	0.00	420	-1.0	-0.04	421	0.0	0.00

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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
No readings for Data Point 2 because there was no bracket on center of first upstream section.
Truck Weights: Gross Vehicle Weight = 51,670 lbs; Rear Axle Vehicle Weight = 37,310 lbs

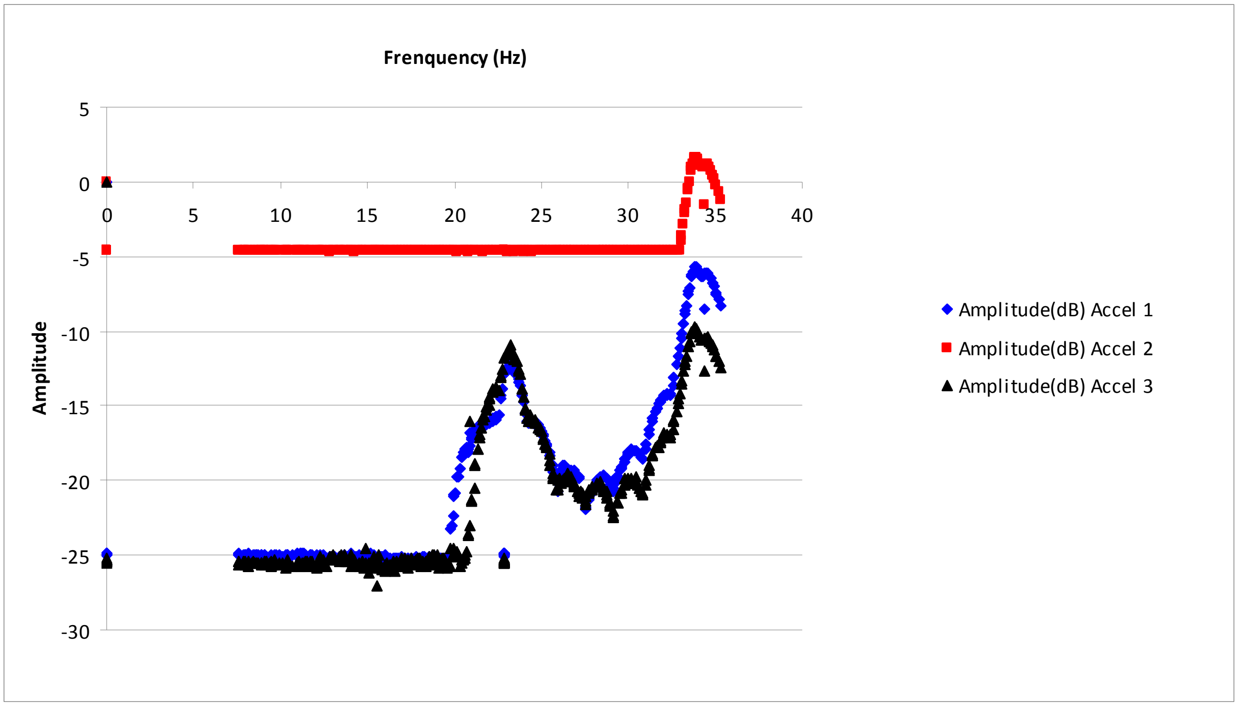
Results

Results obtained from vibration and static load testing for Bridge 263 are shown below.

Vibration frequency

Figure 4.23. shows a representative graph of the frequency data, with the following average peak frequencies; peak 1 = 23.1 Hz and peak 2 = 34.43 Hz. An assessment of the phase data showed the two outer accelerometers were in phase with each other for peak 1, but approximately 180° out of phase with the center accelerometer. However, at the second peak, all three accelerometers were in phase at peak 2, representative of a bending mode of vibration. Only two peaks were noted for this bridge, as compared to 3 peaks on other bridges.

Figure 4.23. A representative frequency pattern for St. Louis County Bridge 263.



Live load

Figure 4.24 shows the deflection of span 1 of Bridge 263 for center loading.

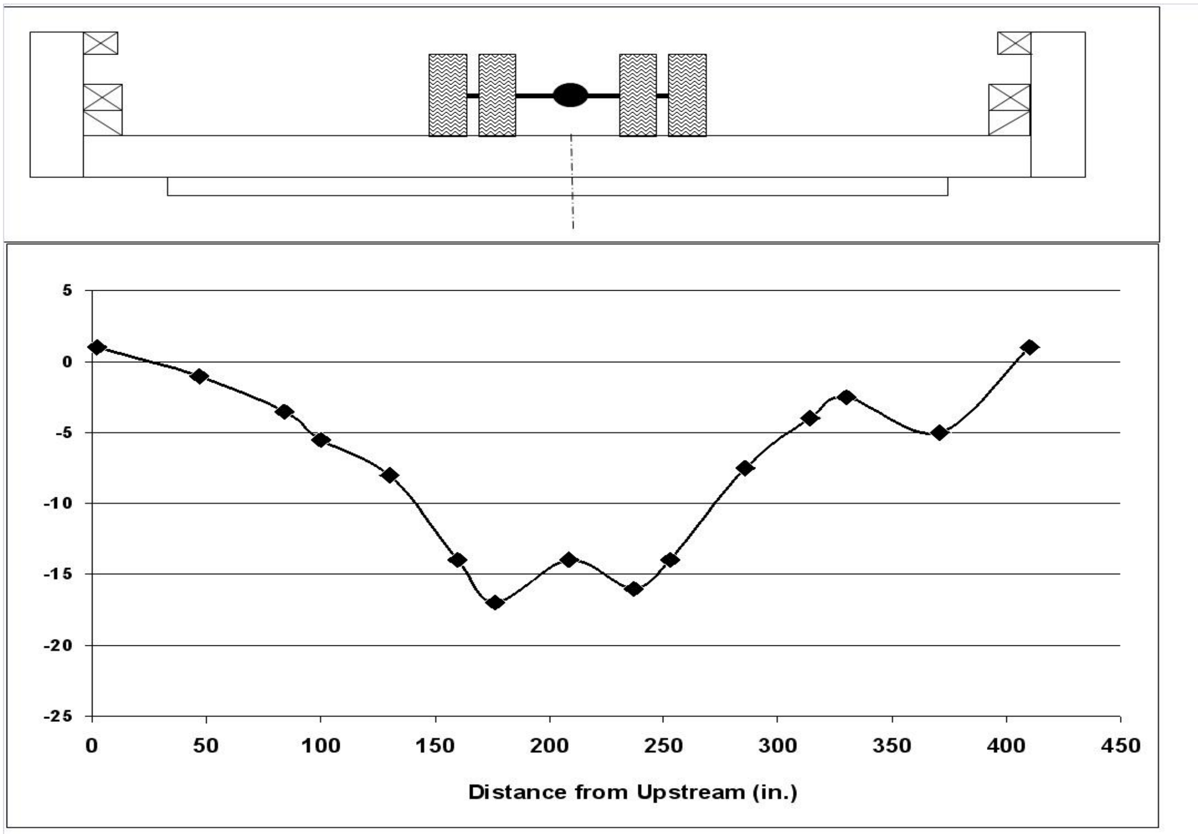


Figure 4.24. Deflection of Bridge 263 for center loading.

Based on the load deflection testing, the following bridge stiffness was calculated as shown in Table 4.14.

Table 4.15. Measured deflection and stiffness for Bridge 263.

Bridge No.	Span No.	Span (center to center - ft)	Deflection		Measured Stiffness	
			Average (in.)	Maximum (in.)	Average (kip-in^2 x 10^6)	Maximum (kip-in^2 x 10^6)
263	1	15.50	0.10	0.26	44.71	17.20

Bridge 304 Testing Summary

Background

Structure:	Bridge 304
Location:	Angora, Minnesota
Special Consideration(s):	Large Bridge – 3 Spans
Year Built:	1992
Inspection date:	August 2008
Construction details:	Three spans, dowel laminated Wheeler Bridge with an asphalt deck for a running surface

Bridge Photos:

Figure 4.25. Bridge 304.

Figure 4.26. Bridge 304.

Figure 4.27. Bridge 304.

Bridge dimensions

Table 4.16. Bridge dimensions for Bridge 304 span 1 and span 2.

Span 1 Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	3.5	7.5	11.5	6	1.0	41.9
Spacer	103.2	6	12	2	8.6	344.0
Spacer	36	6	12	4	6.0	240.0
Railing	17.83	6	12	2	1.5	59.4
Curb	17.83	6	12	2	1.5	59.4
Spreader	411	6	12	1	17.1	685.0
Deck	214	411	10	1	509.0	20,359.7
Asphalt	214	411	3	1	152.7	**22,904.7**
					Total	**50,141.6**
Span 2 Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	46	7.5	11.5	6	13.8	551.0
Spacer	103.2	6	12	2	8.6	344.0
Spacer	36	6	12	4	6.0	240.0
Railing	195.9	6	12	2	16.3	653.0
Curb	195.9	6	12	2	16.3	653.0
Spreader	411	6	12	1	17.1	685.0
Deck	195.9	411	10	1	465.9	18,637.7
Asphalt	195.9	411	3	1	139.8	20,967.4
Total						**48,172.1**

Note: [1]A factor of 40 lbs/ft^3 was used for wood and 150 lbs/ft^3 for asphalt.

Vibration test data

Table 4.17. Vibration data collected from Bridge 304 for span 1.

Span 1								
		Phase				Phase		
Test	Peak 1	Black/Blue	Red/Blue	Red/Black	Peak 2	Black/Blue	Red/Blue	Red/Black
1	17.97	12.82	183.13	170.31	25.30	0.47	10.56	11.03
2	17.29	8.54	193.80	185.25	25.30	1.82	11.99	10.17
Average	17.63	10.68	188.46	177.78	25.30	1.14	11.27	10.60

Note: Hz = hertz

Table 4.18. Vibration data collected from Bridge 304 for span 2.

Span 2								
		Phase				Phase		
Test	Peak 1	Black/Blue	Red/Blue	Red/Black	Peak 2	Black/Blue	Red/Blue	Red/Black
1	17.18	14.91	174.46	159.55	26.45	9.63	2.90	12.53
2	17.13	18.93	175.64	156.71	26.34	9.58	5.43	15.01
Average	17.16	16.92	175.05	158.13	26.40	9.61	4.17	13.77

Note: Hz = hertz

Static load test data

Table 4.19. Static load data collected from Bridge 304 for span 1.

Distance from Upstream (in.)	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)
2	546	545	545.5	545	-0.5	-0.02	546	0.5	0.02	546	0.5	0.02
35	550	550	550.0	548	-2.0	-0.08	550	0.0	0.00	550	0.0	0.00
83	549	547	548.0	543	-5.0	-0.20	548	0.0	0.00	547	-1.0	-0.04
87	551	550	550.5	544	-6.5	-0.26	550	-0.5	-0.02	549	-1.5	-0.06
131	550	550	550.0	543	-7.0	-0.28	549	-1.0	-0.04	547	-3.0	-0.12
160	548	547	547.5	540	-7.5	-0.29	546	-1.5	-0.06	541	-6.5	-0.26
176	551	551	551.0	546	-5.0	-0.20	546	-5.0	-0.20	545	-6.0	-0.24
206	548	547	547.5	545	-2.5	-0.10	541	-6.5	-0.26	540	-7.5	-0.29
236	552	552	552.0	550	-2.0	-0.08	546	-6.0	-0.24	546	-6.0	-0.24
252	550	550	550.0	550	0.0	0.00	543	-7.0	-0.28	547	-3.0	-0.12
280	552	552	552.0	552	0.0	0.00	546	-6.0	-0.24	551	-1.0	-0.04
312	553	553	553.0	553	0.0	0.00	549	-4.0	-0.16	552	-1.0	-0.04
332	551	551	551.0	552	1.0	0.04	550	-1.0	-0.04	551	0.0	0.00
367	551	551	551.0	551	0.0	0.00	551	0.0	0.00	550	-1.0	-0.04
409	564	563	563.5	563	-0.5	-0.02	564	0.5	0.02	564	0.5	0.02
AVG.	551.1	550.6	550.8	548.3	-2.5	-0.1	548.3	-2.5	-0.1	548.4	-2.4	-0.1
A												
B												
C	572	572	572.0	573	1.0	0.04	572	0.0	0.00	572	0.0	0.00
D	536	536	536.0	536	0.0	0.00	536	0.0	0.00	536	0.0	0.00

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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 47,320 lbs; Rear Axle Vehicle Weight = 34,140 lbs

Table 4.20. Static load data collected from Bridge 304 for span 2.

Distance from Upstream (in.)	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)
2	630	630	630.0	630	0.0	0.00	630	0.0	0.00	630	0.0	0.00
35	629	628	628.5	626	-2.5	-0.10	629	0.5	0.02	628	-0.5	-0.02
65	625	625	625.0	620	-5.0	-0.20	626	1.0	0.04	625	0.0	0.00
87	624	624	624.0	617	-7.0	-0.28	624	0.0	0.00	623	-1.0	-0.04
131	622	622	622.0	615	-7.0	-0.28	621	-1.0	-0.04	619	-3.0	-0.12
160	618	618	618.0	612	-6.0	-0.24	617	-1.0	-0.04	612	-6.0	-0.24
176	620	619	619.5	612	-7.5	-0.29	617	-2.5	-0.10	613	-6.5	-0.26
206	618	618	618.0	613	-5.0	-0.20	613	-5.0	-0.20	613	-5.0	-0.20
236	612	612	612.0	610	-2.0	-0.08	604	-8.0	-0.31	604	-8.0	-0.31
252	612	612	612.0	610	-2.0	-0.08	605	-7.0	-0.28	604	-8.0	-0.31
280	604	604	604.0	603	-1.0	-0.04	598	-6.0	-0.24	601	-3.0	-0.12
312	601	601	601.0	601	0.0	0.00	595	-6.0	-0.24	600	-1.0	-0.04
332	603	603	603.0	603	0.0	0.00	599	-4.0	-0.16	602	-1.0	-0.04
367	604	599	601.5	599	-2.5	-0.10	598	-3.5	-0.14	599	-2.5	-0.10
409	596	596	596.0	596	0.0	0.00	596	0.0	0.00	597	1.0	0.04
AVG.	614.5	614.1	614.3	611.1	-3.2	-0.1	611.5	-2.8	-0.1	611.3	-3.0	-0.1
A												
B	620	619	619.5	619	-0.5	-0.02	620	0.5	0.02	620	0.5	0.02
C	610	610	610.0	610	0.0	0.00	610	0.0	0.00	610	0.0	0.00
D	584	584	584.0	584	0.0	0.00	584	0.0	0.00	584	0.0	0.00

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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 47,320 lbs; Rear Axle Vehicle Weight = 34,140 lbs

Results

Results obtained from vibration and static load testing for Bridge 304 are shown below.

Vibration frequency

Figure 4.28. shows a representative graph of the frequency data for span 1; peak 1 = 17.97 Hz and peak 2 = 25.30 Hz. Figure 4.29. shows a representative graph of the frequency data for span 1; peak 1 = 17.16 Hz and peak 2 = 26.40 Hz. A third peak was beginning to appear, but the test was completed before the peak was detected and is therefore not reported. Both spans show that the outer accelerometers are out of phase with the center midspan accelerometer for peak 1, but they are all in phase for peak 2.

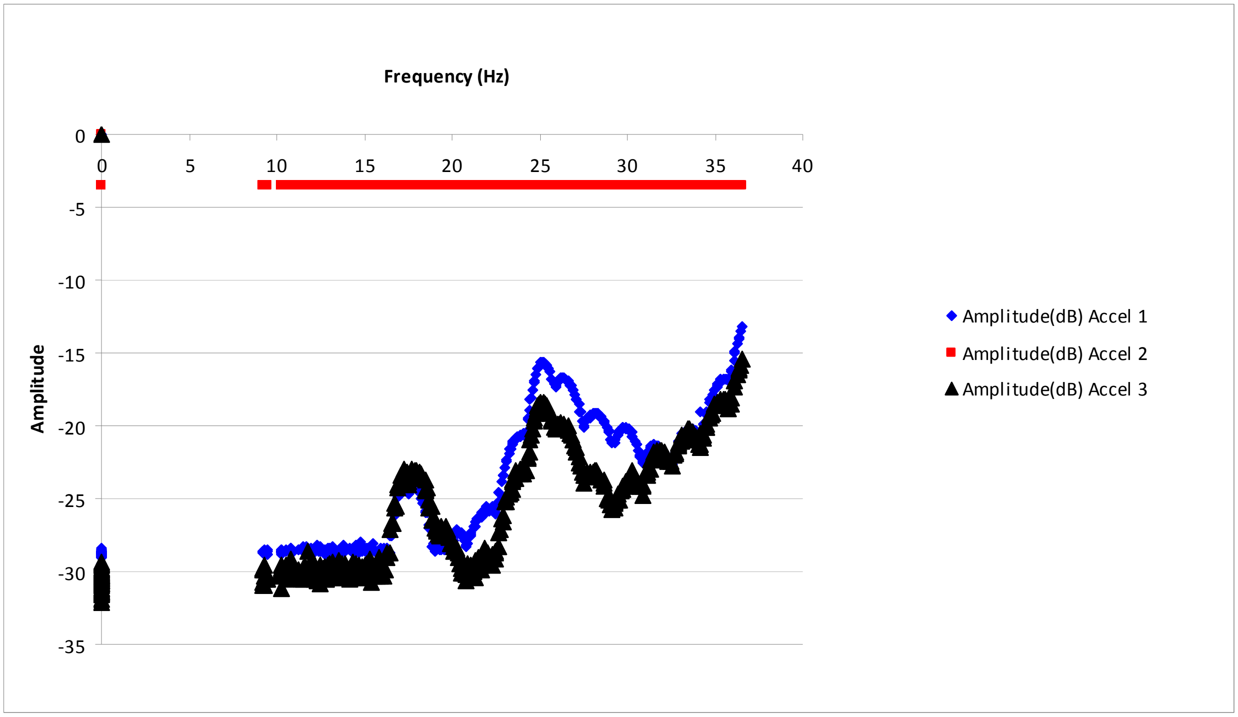


Figure 4.28. A representative frequency pattern for St. Louis County Bridge 304 span 1.

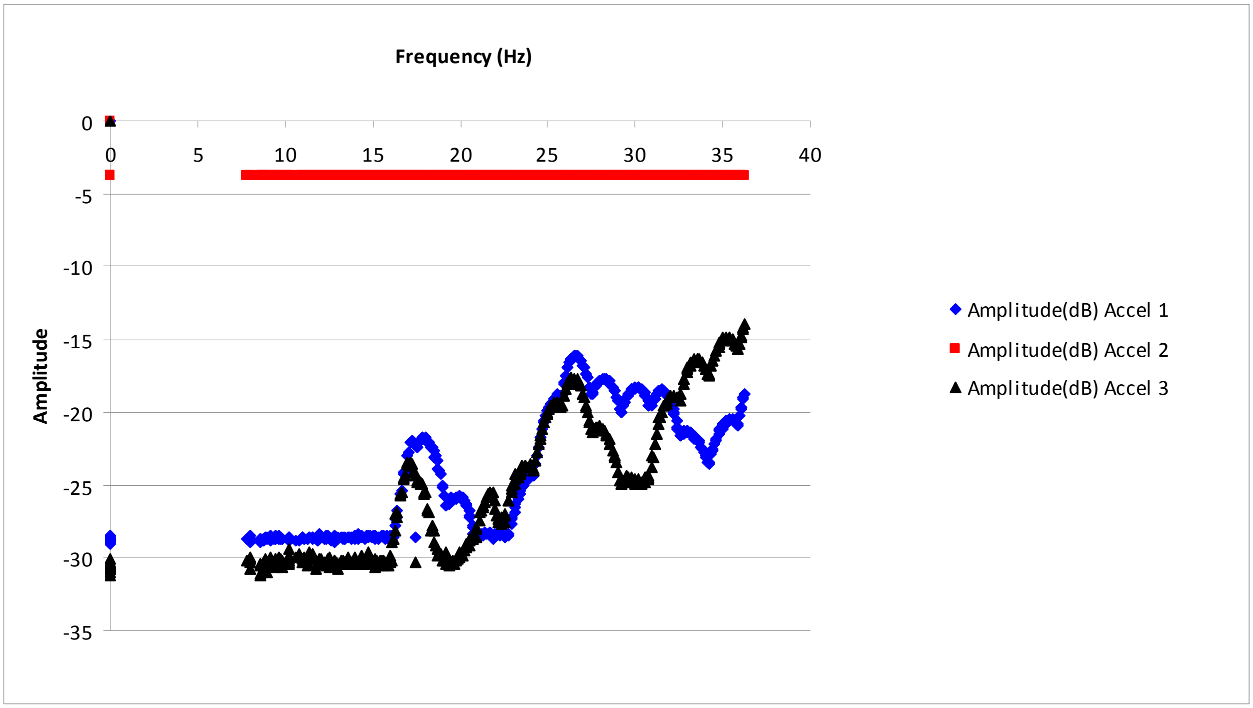


Figure 4.29. A representative frequency pattern for St. Louis County Bridge 304 span 2.

Live load

Figures 4.30 and 4.31 show the deflections of spans 1 and 2 of Bridge 304 for center loading.

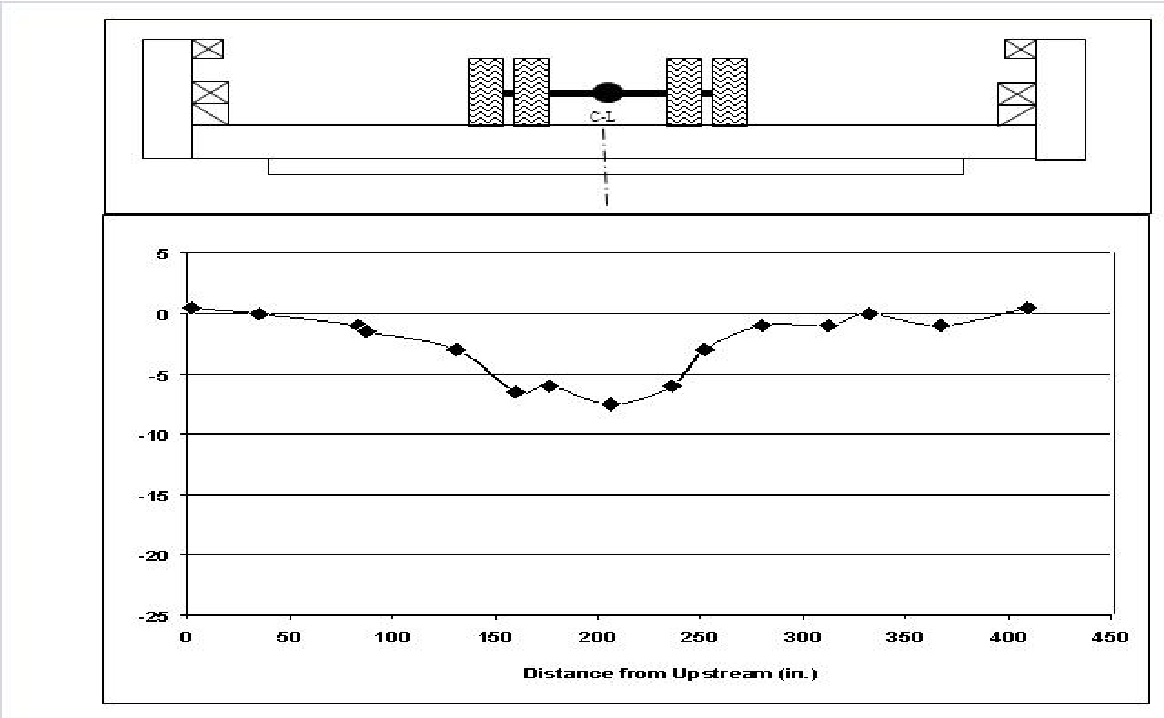


Figure 4.30. Deflection of span 1 of Bridge 304 for center loading.

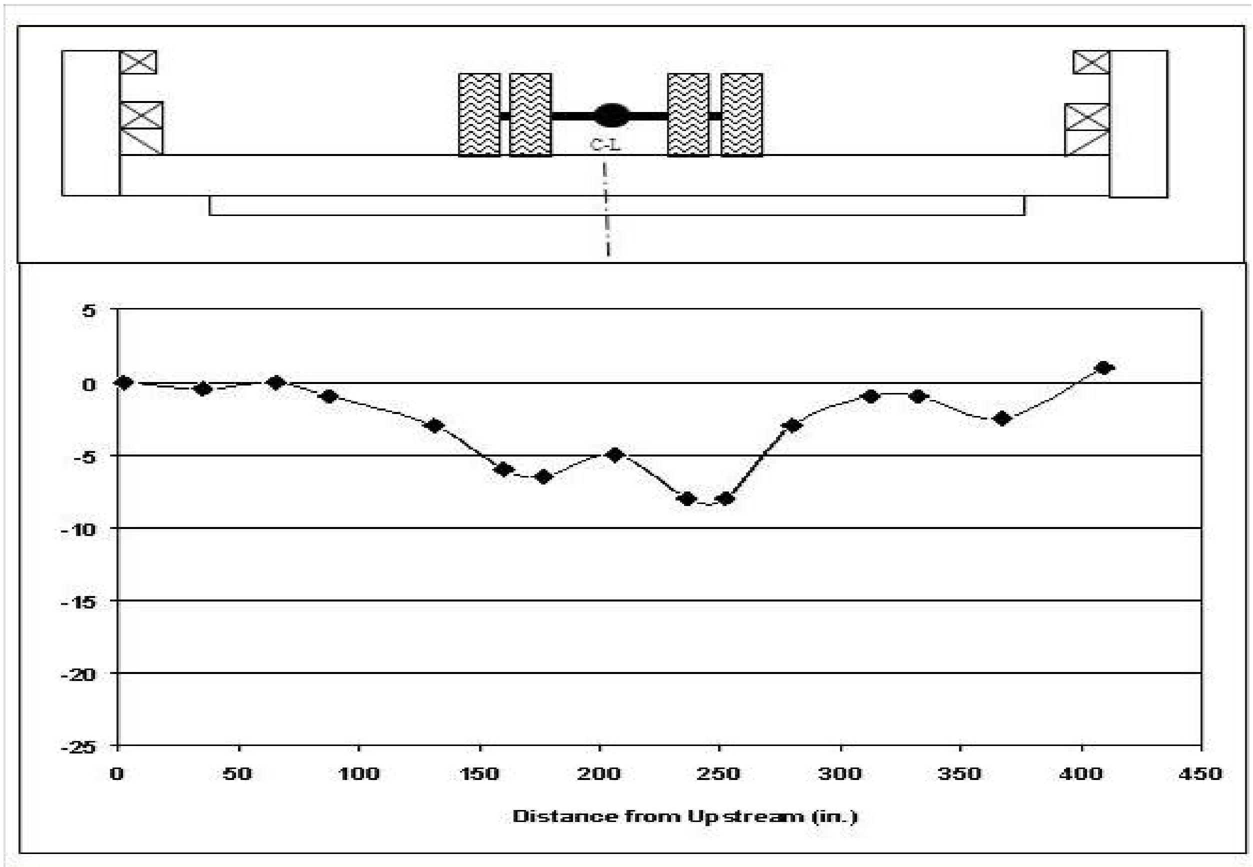


Figure 4.31. Deflection of span 2 of Bridge 304 for center loading.

Based on the load deflection testing, the following bridge stiffness was calculated as shown in Table 4.20.

Table 4.21. Measured deflection and stiffness for Bridge 304 span 1 and span 2.

Bridge No.	Span No.	Span (center to center - ft)	Deflection		Measured Stiffness	
			Average (in.)	Maximum (in.)	Average (kip-in^2 x 10^6)	Maximum (kip-in^2 x 10^6)
304	1	18.83	0.10	0.29	75.59	26.06
304	2	17.33	0.12	0.31	48.38	18.73

Bridge 313 Testing Summary

Background

Structure:	Bridge 313
Location:	Lake Vermilion, Minnesota
Special Consideration(s):	Large Bridge – 3 Spans
Year Built:	1982
Inspection date:	August 2008
Construction details:	Three spans, dowel laminated Wheeler Bridge with an asphalt deck for a running surface. Span 1 is the span closest to St. Louis County Highway (SLH) 115. Span 2 is the center span and Span 3 is the span farthest from SLH 115.

Bridge Photos:

Figure 4.32. Bridge 313.

Figure 4.33. Bridge 313.

Figure 4.34. Bridge 313.

Bridge dimensions

Table 4.22. Bridge dimensions for Bridge 313 span 1, 2, and 3.

Span 1 Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	48	7.5	12	6	15.0	600.0
Spacer	103.2	6	12	2	8.6	344.0
Spacer	36	6	12	4	6.0	240.0
Railing	194	6	12	2	16.2	646.7
Curb	194	6	12	2	16.2	646.7
Spreader	316	6	12	1	13.2	526.7
Deck	194	316	10	1	354.8	14,190.7
Asphalt	194	316	3	1	106.4	15,964.60
					Total	**37458.0**
Span 2 Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	42	7.5	12	12	26.3	1,050.0
Spacer	36	6	12	8	12.0	480.0
Railing	368	6	12	2	30.7	1,226.7
Curb	368	6	12	2	30.7	1,226.7
Spreader	316	6	12	1	13.2	526.7
Deck	368	316	14	1	942.1	37,685.9
Asphalt	368	316	3	1	201.9	30,283.3
					Total	**83,028.2**
Span 3 Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	42	7.5	12	6	13.1	525.0
Spacer	103.2	6	12	2	8.6	344.0
Spacer	36	6	12	4	6.0	240.0
Railing	194	6	12	2	16.2	646.7
Curb	194	6	12	2	16.2	646.7
Spreader	316	6	12	1	13.2	526.7
Deck	195	316	10	1	356.6	1,423.9
Asphalt	195	316	3	1	107.0	16,046.9
					Total	**37,538.0**

Note: [1]A factor of 40 lbs/ft^3 was used for wood and 150 lbs/ft^3 for asphalt.

Vibration test data

Table 4.23. Vibration data collected from Bridge 313 for span 1.

Span 1								
Test	Peak 1 Frequency (Hz)	Phase			Peak 2 Frequency (Hz)	Phase		
		Black/Blue	Red/Blue	Red/Black		Black/Blue	Red/Blue	Red/Black
1	18.32	21.74	161.58	183.32	31.55	30.19	18.24	11.96
2	18.42	24.84	164.82	189.66	31.56	30.33	19.78	10.55
Average	18.37	23.29	163.20	186.49	31.55	30.26	19.01	11.25

Note: Hz = hertz

Table 4.24. Vibration data collected from Bridge 313 for span 2.

Span 2								
Test	Peak 1	Phase			Peak 2	Phase		
		Black/Blue	Red/Blue	Red/Black		Black/Blue	Red/Blue	Red/Black
1	Erratic Data							
2	No Peaks							

Note: Hz = hertz

Table 4.25. Vibration data collected from Bridge 313 for span 3.

Span 3								
Test	Peak 1 Frequency (Hz)	Phase			Peak 2 Frequency (Hz)	Phase		
		Black/Blue	Red/Blue	Red/Black		Black/Blue	Red/Blue	Red/Black
1	18.26	7.28	187.41	180.13	31.50	27.10	10.70	16.40
2	18.30	10.48	187.76	177.28	31.55	28.99	12.76	16.23
3	18.22	6.52	185.18	178.66	31.50	28.36	12.66	15.69
Average	18.26	8.09	186.79	178.69	31.52	28.15	12.04	16.11

Note: Hz = hertz

Static load test data

Table 4.26. Static load data collected from Bridge 313 for span 1.

Distance from Upstream (in.)	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1 Reading (mm)	Load Case No. 1 Deflection (mm) (based on final zero)	Load Case No. 1 Deflection (in)	Load Case No. 2 Reading (mm)	Load Case No. 2 Deflection (mm)	Load Case No. 2 Deflection (in)	Load Case No. 3 Reading (mm)	Load Case No. 3 Deflection (mm)	Load Case No. 3 Deflection (in)
3	1460	1430		1429	-1.0	-0.04	1461	1.0	0.04	1461	1.0	0.04
39	1462	1435		1430	-5.0	-0.20	1462	0.0	0.00	1460	-2.0	-0.08
72	1453	1429		1421	-8.0	-0.31	1452	-1.0	-0.04	1451	-2.0	-0.08
83	1456	1433		1428	-5.0	-0.20	1455	-1.0	-0.04	1453	-3.0	-0.12
116	1446	1427		1422	-5.0	-0.20	1445	-1.0	-0.04	1439	-7.0	-0.28
148	1442	1426		1420	-6.0	-0.24	1439	-3.0	-0.12	1437	-5.0	-0.20
160.5	1440	1425		1421	-4.0	-0.16	1435	-5.0	-0.20	1435	-5.0	-0.20
189	1438	1426		1424	-2.0	-0.08	1429	-9.0	-0.35	1429	-9.0	-0.35
221	1442	1433		1433	0.0	0.00	1437	-5.0	-0.20	1439	-3.0	-0.12
233	1438	1430		1430	0.0	0.00	1431	-7.0	-0.28	1436	-2.0	-0.08
271	1439	1433		1433	0.0	0.00	1434	-5.0	-0.20	1438	-1.0	-0.04
314	1440	1433		1434	1.0	0.04	1438	-2.0	-0.08	1440	0.0	0.00
AVG	1399	1376		1376	0.0	0.00	1410	11.0	0.43	1410	11.0	0.43
A												
B												
C												
D												

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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 47,320 lbs; Rear Axle Vehicle Weight = 34,140 lbs

Table 4.27. Static load data collected from Bridge 313 for span 2.

Distance from Upstream (in.)	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)
3	1378	1379	1378.5	1373	-5.5	-0.22	1379	0.5	0.02	1379	0.5	0.02
39	1377	1377	1377.0	1366	-11.0	-0.43	1377	0.0	0.00	1375	-2.0	-0.08
72	1370	1370	1370.0	1355	-15.0	-0.59	1368	-2.0	-0.08	1365	-5.0	-0.20
83	1358	1359	1358.5	1344	-14.5	-0.57	1355	-3.5	-0.14	1350	-8.5	-0.33
116	1353	1355	1354.0	1340	-14.0	-0.55	1350	-4.0	-0.16	1340	-14.0	-0.55
148	1355	1356	1355.5	1340	-15.5	-0.61	1346	-9.5	-0.37	1341	-14.5	-0.57
160.5	1356	1357	1356.5	1345	-11.5	-0.45	1345	-11.5	-0.45	1343	-13.5	-0.53
189	1348	1348	1348.0	1342	-6.0	-0.24	1333	-15.0	-0.59	1332	-16.0	-0.63
221	1363	1364	1363.5	1360	-3.5	-0.14	1349	-14.5	-0.57	1352	-11.5	-0.45
233	1349	1349	1349.0	1346	-3.0	-0.12	1333	-16.0	-0.63	1340	-9.0	-0.35
271	1354	1354	1354.0	1353	-1.0	-0.04	1341	-13.0	-0.51	1350	-4.0	-0.16
314	1351	1352	1351.5	1352	0.5	0.02	1346	-5.5	-0.22	1351	-0.5	-0.02
AVG.	1359.3	1360.0	1359.7	1351.3	-8.3	-0.3	1351.8	-7.8	-0.3	1351.5	-8.2	-0.3
A												
B												
C												
D												

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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 47,320 lbs; Rear Axle Vehicle Weight = 34,140 lbs

Table 4.28. Static load data collected from Bridge 313 for span 3.

Distance from Upstream (in.)	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1 Reading (mm)	Load Case No. 1 Deflection (mm)	Load Case No. 1 Deflection (in)	Load Case No. 2 Reading (mm)	Load Case No. 2 Deflection (mm)	Load Case No. 2 Deflection (in)	Load Case No. 3 Reading (mm)	Load Case No. 3 Deflection (mm)	Load Case No. 3 Deflection (in)
3	466	466	466.0	466	0.0	0.00	467	1.0	0.04	466	0.0	0.00
39	465	467	466.0	462	-4.0	-0.16	466	0.0	0.00	465	-1.0	-0.04
72	456	457	456.5	450	-6.5	-0.26	456	-0.5	-0.02	454	-2.5	-0.10
83	457	459	458.0	451	-7.0	-0.28	457	-1.0	-0.04	455	-3.0	-0.12
116	457	453	455.0	448	-7.0	-0.28	451	-4.0	-0.16	445	-10.0	-0.39
148	450	452	451.0	444	-7.0	-0.28	448	-3.0	-0.12	446	-5.0	-0.20
160.5	440	442	441.0	436	-5.0	-0.20	436	-5.0	-0.20	436	-5.0	-0.20
189	440	441	440.5	438	-2.5	-0.10	432	-8.5	-0.33	432	-8.5	-0.33
221	439	441	440.0	440	0.0	0.00	435	-5.0	-0.20	436	-4.0	-0.16
233	437	439	438.0	438	0.0	0.00	433	-5.0	-0.20	435	-3.0	-0.12
271	434	435	434.5	435	0.5	0.02	428	-6.5	-0.26	433	-1.5	-0.06
314	434	434	434.0	435	1.0	0.04	433	-1.0	-0.04	435	1.0	0.04
AVG.	447.9	448.8	448.4	445.3	-3.1	-0.1	445.2	-3.2	-0.1	444.8	-3.5	-0.1
A												
B	486	488	487	488	1	0.04	488	1	0.04	487	0.0	0.00
C												
D	450	450	450	450	0	0.00	449	-1	-0.04	450	0.0	0.00

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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner).
Truck Weights: Gross Vehicle Weight = 47,320 lbs; Rear Axle Vehicle Weight = 34,140 lbs

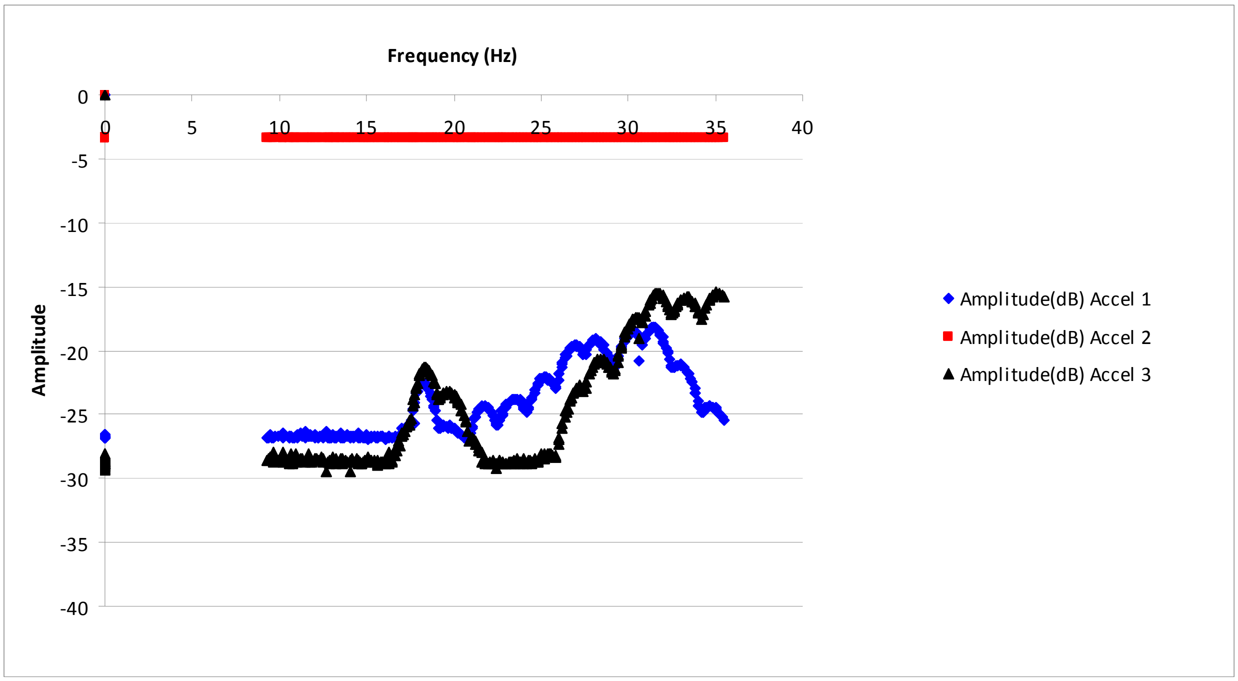
Results

Results obtained from vibration and static load testing for Bridge 313 are shown below.

Vibration frequency

Figure 4.35. shows a representative graph of the frequency data for span 1; peak 1 = 18.37 Hz and peak 2 = 31.55 Hz. Figure 4.36. shows a representative graph of the frequency data for span 2; peak 1 = 18.26 Hz and peak 2 = 31.52 Hz. Figure 4.37. shows a representative graph of the frequency data for span 3. No peaks were detected for Span 3, which was the center span on the bridge. This is likely due to the fact that the motor was installed on the top of the bridge, instead of directly to the wood from the bottom side due to the extreme height and danger. Both span 1 and 2 show that the outer accelerometers are out of phase with the center midspan accelerometer for peak 1, but they are all in phase for peak 2.

Figure 4.35. A representative frequency pattern for St. Louis County Bridge 313 span 1.



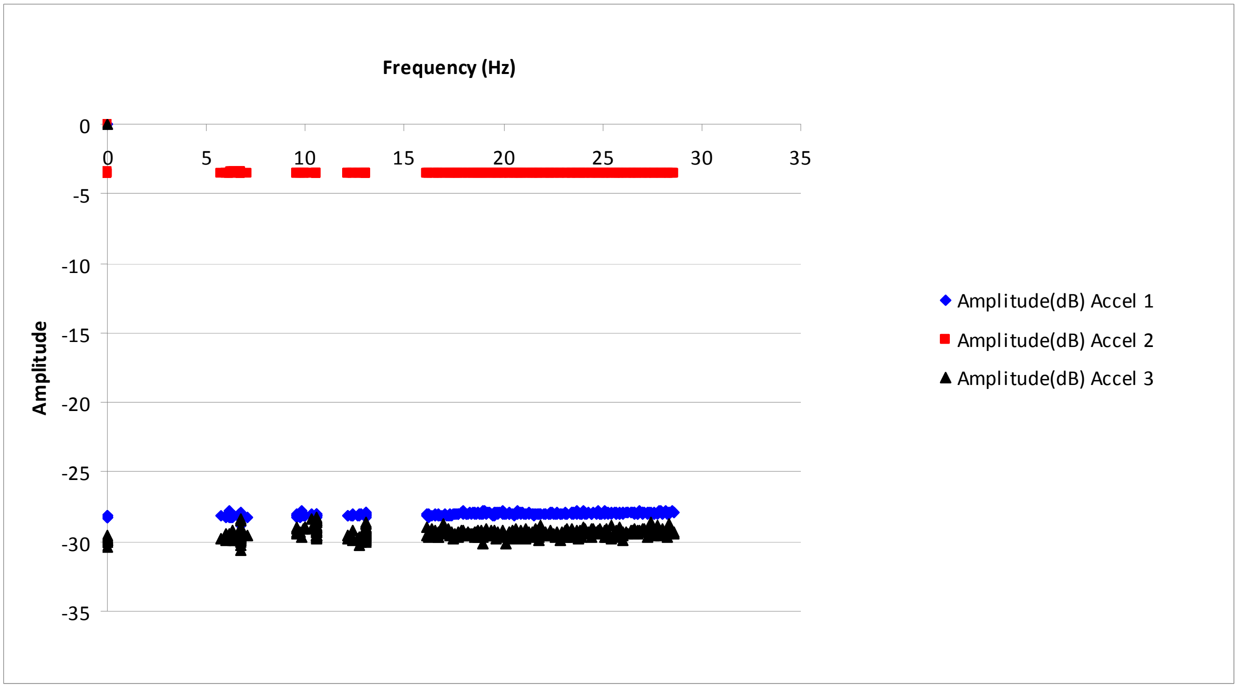


Figure 4.36. A representative frequency pattern for st. Louis County Bridge 313, span 2.

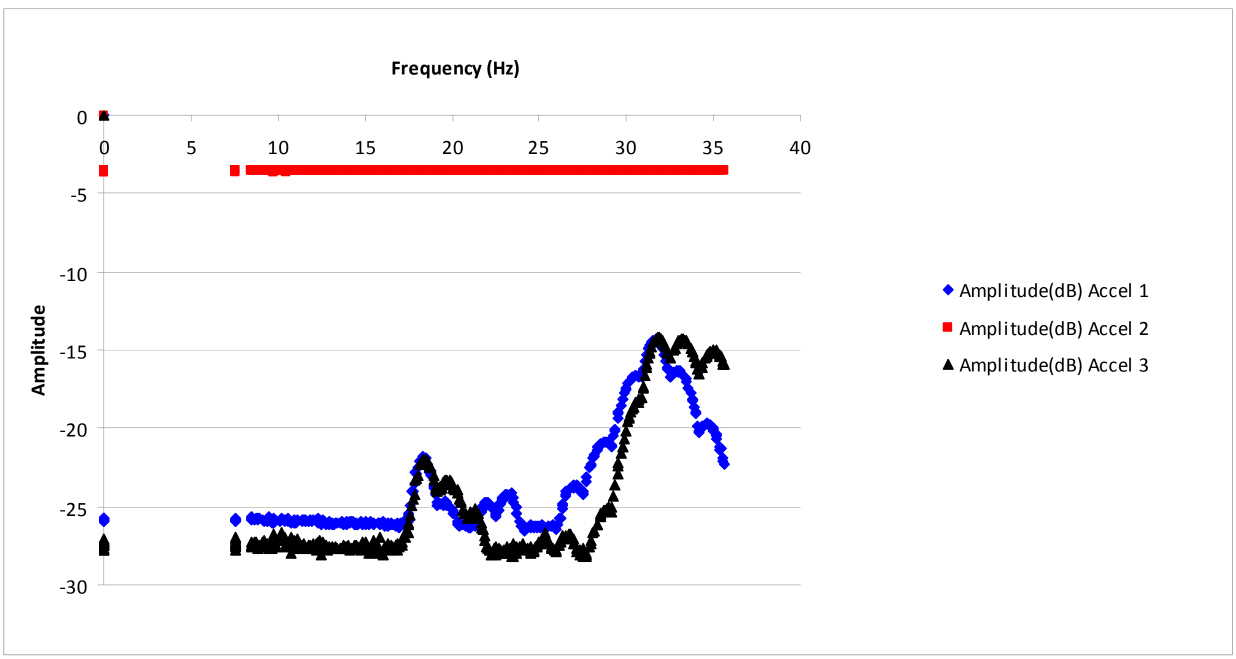


Figure 4.37. A representative frequency pattern for St. Louis County Bridge 313, span 3.

Live load

Figures 4.38, 4.39, and 4.40 show the deflections of spans 1, 2, and 3 of Bridge 313 for center loading.

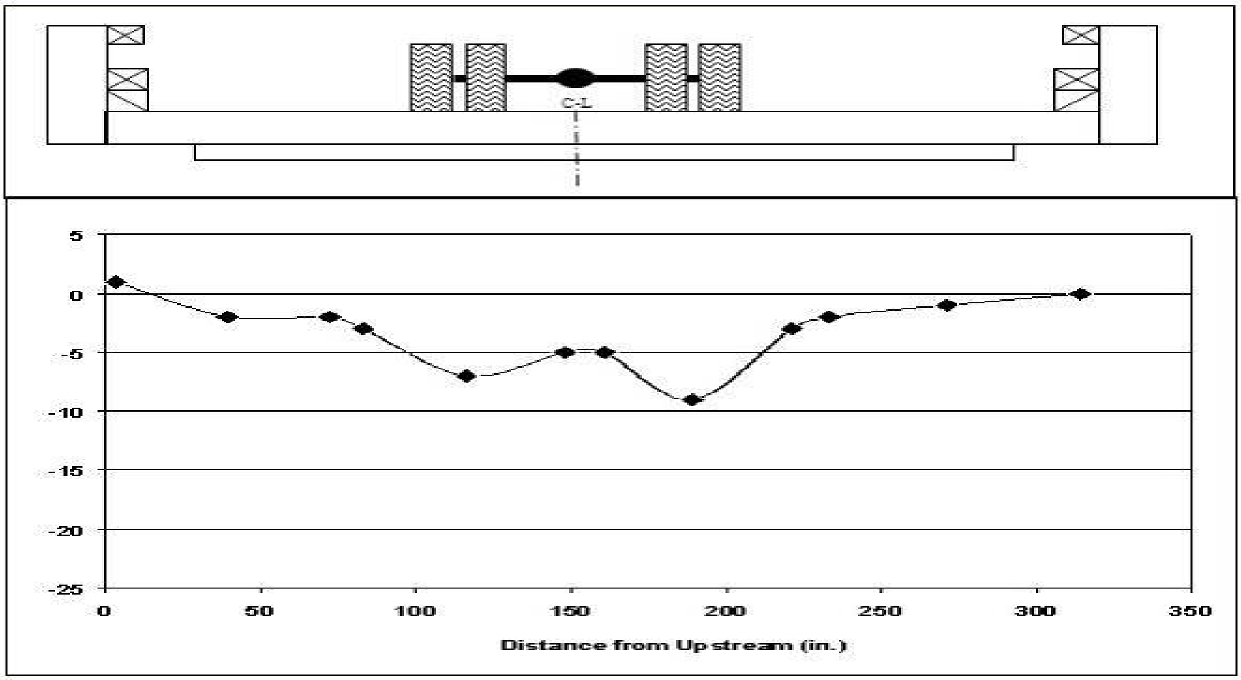


Figure 4.38. Deflection of span 1 of Bridge 313 for center loading.

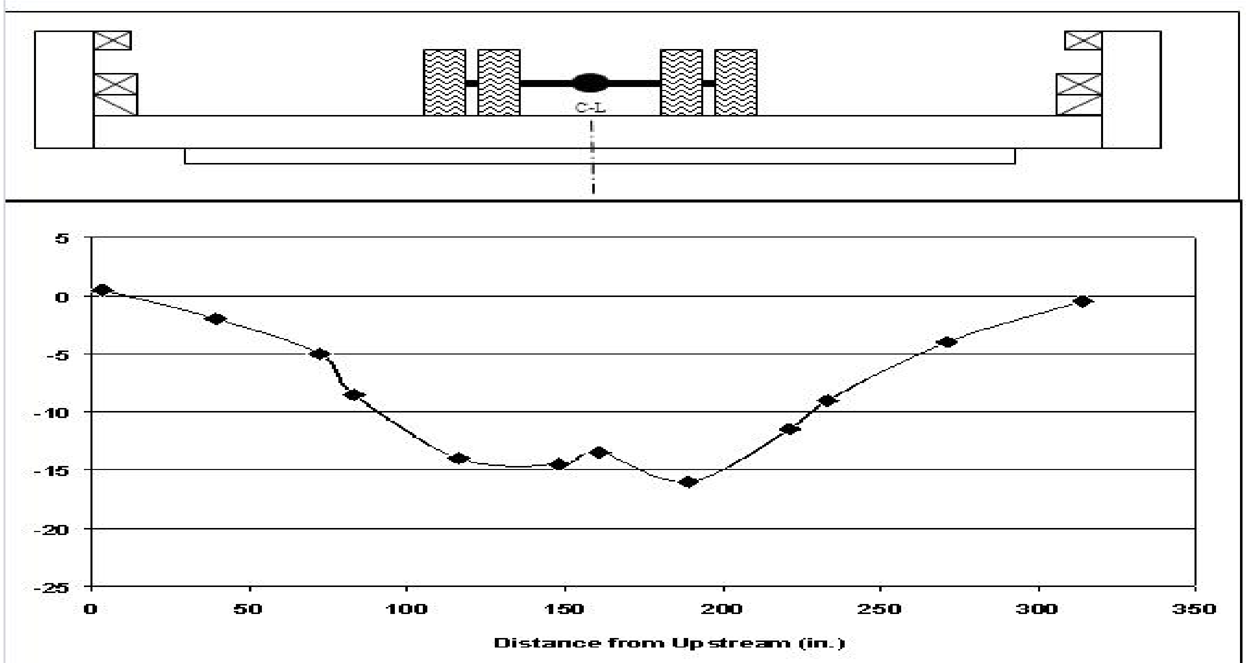


Figure 4.39. Deflection of span 2 of Bridge 313 for center loading.

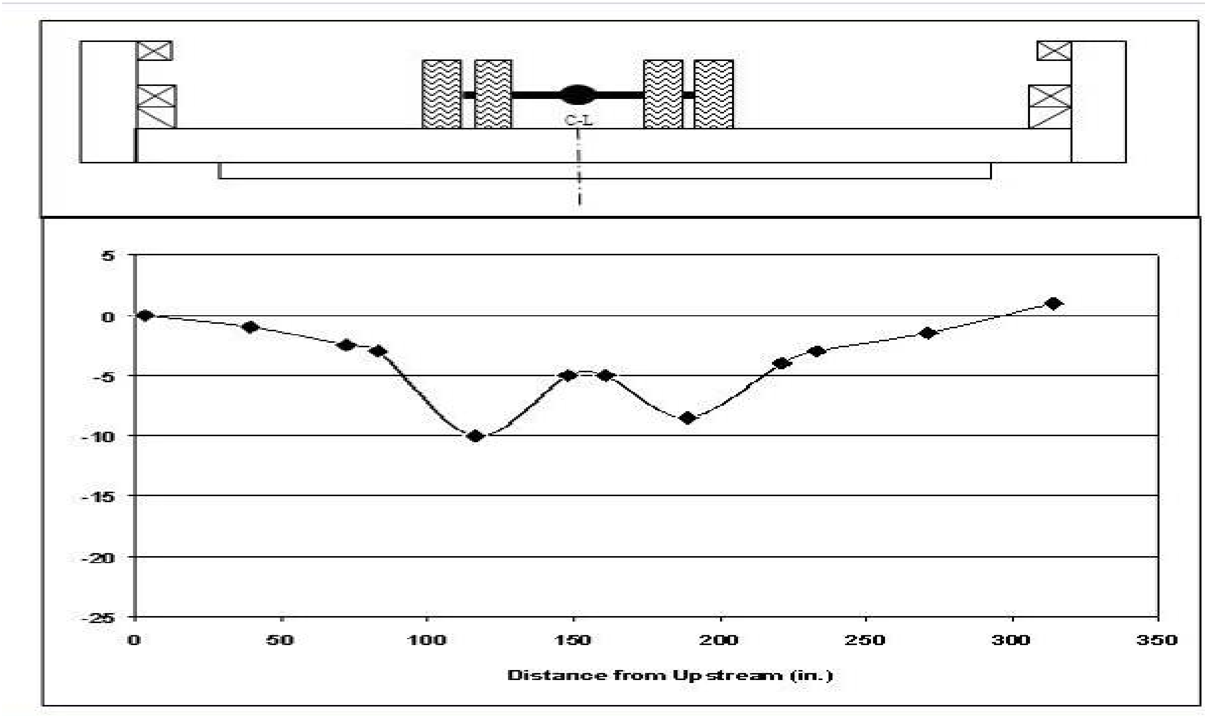


Figure 4.40. Deflection of span 3 of Bridge 313 for center loading.

Based on the load deflection testing, the following bridge stiffness was calculated as shown in Table 4.28.

Table 4.29. Measured deflection and stiffness for Bridge 313 span 1 and span 2.

Bridge No.	Span No.	Span (center to center - ft)	Deflection		Measured Stiffness	
			Average (in.)	Maximum (in.)	Average (kip-in^2 x 10^6)	Maximum (kip-in^2 x 10^6)
313	1	17.17	0.12	0.35	46.97	16.10
313	2 (center)	31.67	0.32	0.63	118.48	60.18
313	3	17.17	0.14	0.39	40.26	14.45

Bridge 383 Testing Summary

Background

Structure: Bridge 383

Location: Ault, Minnesota

Special Consideration(s): None

Year Built: 2003

Inspection date: August 2008

Construction details: Single span, dowel laminated Wheeler Bridge with an asphalt deck for a running surface.

Bridge Photos:

Figure 4.41. Bridge 383.

Figure 4.42. Bridge 383.

Figure 4.43. Bridge 383.

Bridge dimensions

Table 4.30. Bridge dimensions for Bridge 383.

Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	46	8	12	8	20.4	817.8
Spacer	10.5	5.5	8	8	2.1	85.6
Railing	198	6	10.5	2	14.4	577.5
Curb	198	5.5	11.5	2	14.5	579.8
Spreader	315.5	6	12	1	13.1	525.8
Deck	198	315.5	4	1	144.6	5784.2
Asphalt	198	315.5	3	1	108.5	16268.0
					Total	**26731.3**

Note: [1]A factor of 40 lbs/ft^3 was used for wood and 150 lbs/ft^3 for asphalt.

Vibration test data

Table 4.31. Vibration data collected from Bridge 383 for span 1 in September.

Test	Peak 1 Frequency (Hz)	Phase			Peak 2 Frequency (Hz)	Phase		
		Black/Blue	**Red/Blue**	**Red/Black**		**Black/Blue**	**Red/Blue**	**Red/Black**
1	21.87	89.84	210.90	121.06	30.13	100.85	8.97	91.88
2	19.86	25.28	164.80	190.08	30.15	42.88	10.97	31.91
3	19.75	25.22	165.29	190.51	30.04	41.03	8.20	32.84
Average	20.50	46.78	180.33	167.22	30.10	61.59	9.38	52.21

Note: Hz = hertz

Static load test data

Table 4.32. Static load data collected from Bridge 383 for span 1.

Distance from Upstream (in.)	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1 Reading (mm)	Load Case No. 1 Deflection (mm)	Load Case No. 1 Deflection (in)	Load Case No. 2 Reading (mm)	Load Case No. 2 Deflection (mm)	Load Case No. 2 Deflection (in)	Load Case No. 3 Reading (mm)	Load Case No. 3 Deflection (mm)	Load Case No. 3 Deflection (in)
2	569	572	570.5	573	2.5	0.10	572	1.5	0.06	572	1.5	0.06
39	566	568.5	567.3	569	1.8	0.07	569	1.8	0.07	570	2.8	0.11
67.5	561	569	565.0	570	5.0	0.20	565	0.0	0.00	569	4.0	0.16
79.5	564	564	564.0	565	1.0	0.04	557	-7.0	-0.28	564	0.0	0.00
112	561	566	563.5	568	4.5	0.18	561	-2.5	-0.10	566	2.5	0.10
140	558	562	560.0	562	2.0	0.08	555	-5.0	-0.20	556	-4.0	-0.16
151.5	558	560	559.0	559	0.0	0.00	555	-4.0	-0.16	551	-8.0	-0.31
180	558	559	558.5	555	-3.5	-0.14	553	-5.5	-0.22	555	-3.5	-0.14
212	561	561	561.0	553	-8.0	-0.31	560	-1.0	-0.04	554	-7.0	-0.28
225	566	567	566.5	560	-6.5	-0.26	566	-0.5	-0.02	559	-7.5	-0.29
252	563	568	565.5	560	-5.5	-0.22	564	-1.5	-0.06	562	-3.5	-0.14
285	573	576	574.5	566	-8.5	-0.33	574	-0.5	-0.02	574	-0.5	-0.02
297	565	569	567.0	561	-6.0	-0.24	568	1.0	0.04	566	-1.0	-0.04
325	570	573	571.5	571	-0.5	-0.02	572	0.5	0.02	574.5	3.0	0.12
362	566	570	568.0	569	1.0	0.04	569	1.0	0.04	568	0.0	0.00
AVG.	563.9	567.0	565.5	564.1	-1.4	-0.1	564.0	-1.5	-0.1	564.0	-1.4	-0.1
A	570	572	571.0	573	2.0	0.08	573	2.0	0.08	574	3.0	0.12
B	565	566	565.5	568	2.5	0.10	565	-0.5	-0.02	567	1.5	0.06
C	571	572	571.5	572	0.5	0.02	573	1.5	0.06	572	0.5	0.02
D	567	568	567.5	569	1.5	0.06	569	1.5	0.06	568	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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 52,190 lbs; Rear Axle Vehicle Weight = 38,330 lbs

Results

Results obtained from vibration and static load testing for Bridge 383 are shown below.

Vibration frequency

Figure 4.44 shows a representative graph of the frequency data for span 1; peak 1 = 20.50 Hz and peak 2 = 30.13 Hz. Both span 1 and 2 show that the outer accelerometers are out of phase with the center midspan accelerometer for peak 1, but they are all in phase for peak 2.

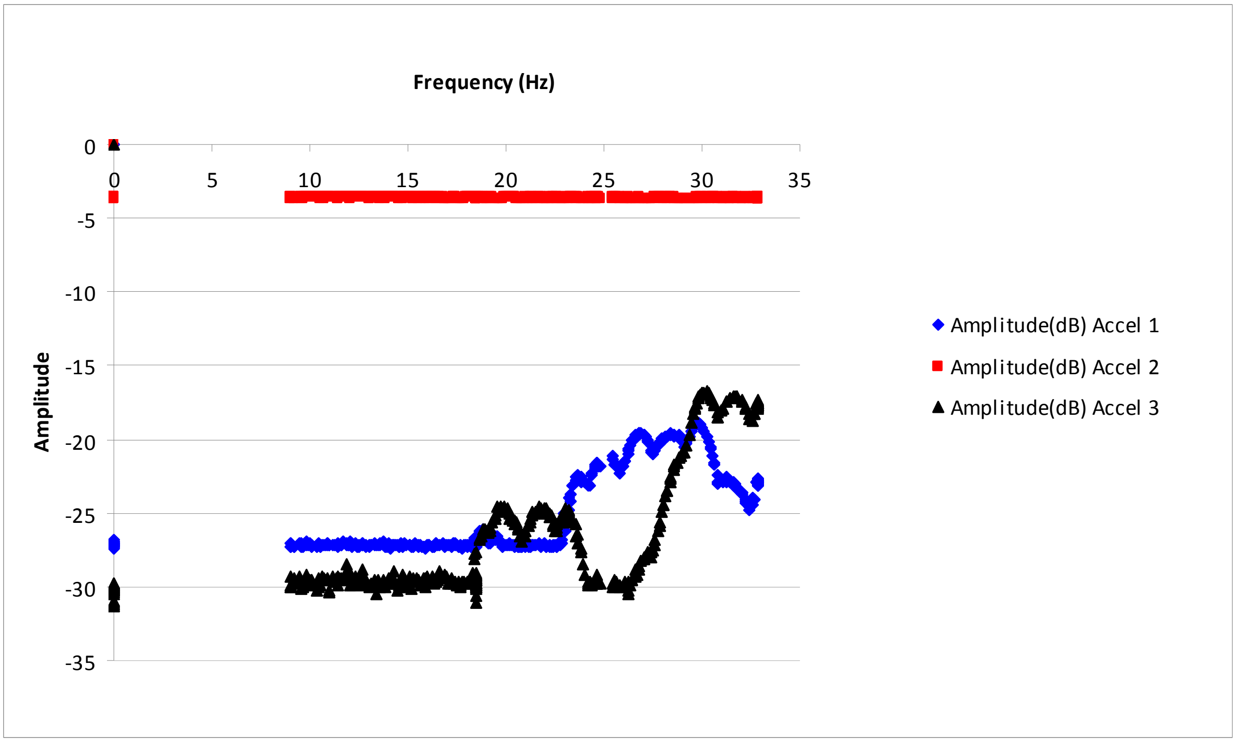


Figure 4.44. A representative frequency pattern for St. Louis County Bridge 383 span 1.

Live load

Figure 4.45 shows the deflection of span 1 of Bridge 383 for center loading.

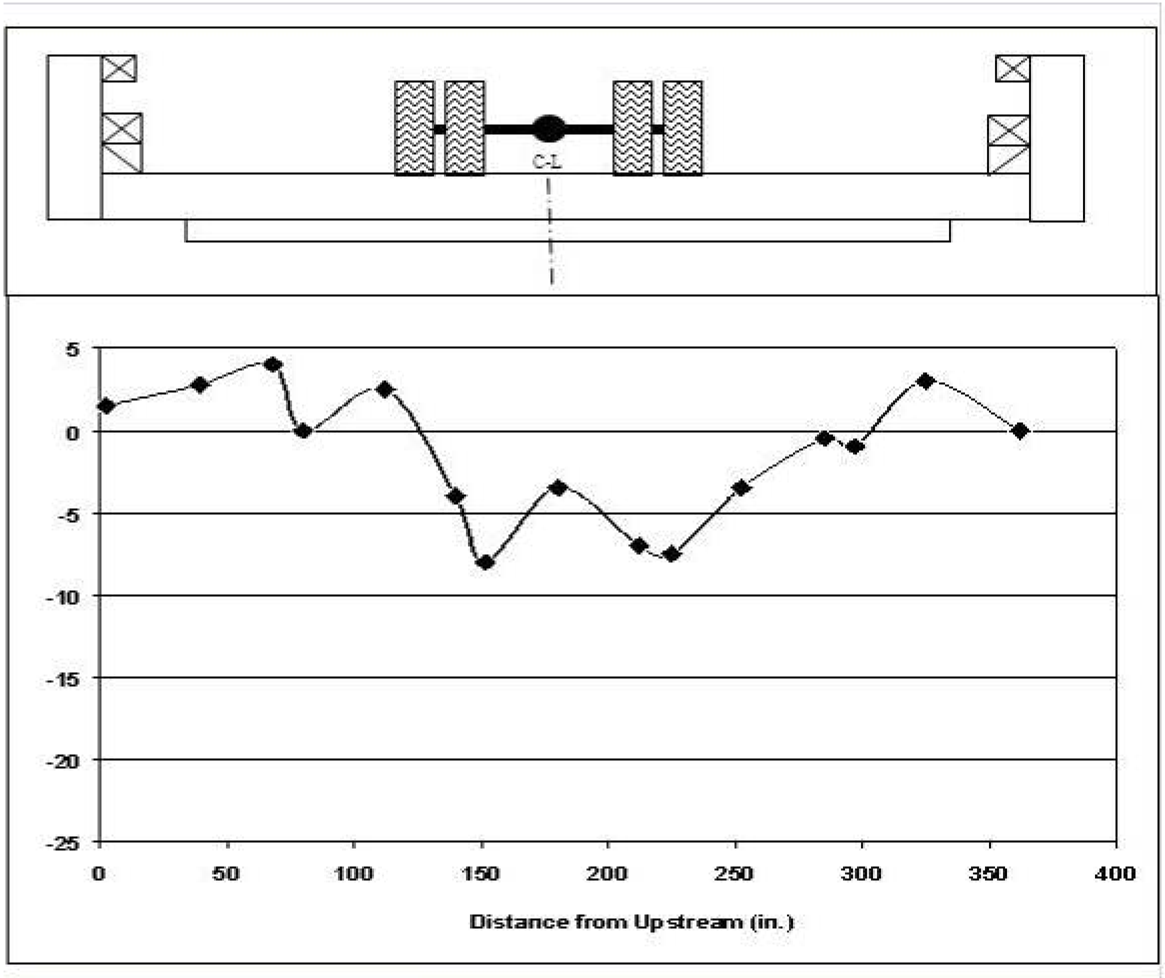


Figure 4.45. Deflection of span 1 of Bridge 313 for center loading.

Based on the load deflection testing, the following bridge stiffness was calculated as shown in Table 4.33.

Table 4.33. Measured deflection and stiffness for Bridge 313 span 1 and span 2.

Bridge No.	Span No.	Span (center to center - ft)	Deflection		Measured Stiffness	
			Average (in.)	Maximum (in.)	Average (kip-in^2 x 10^6)	Maximum (kip-in^2 x 10^6)
383	1	19.88	0.05	0.31	201.92	32.57

Bridge 402 Testing Summary

Background

Structure:	Bridge 402
Location:	McDavitt, Minnesota
Special Consideration(s):	Bridge is skewed across waterway at 30 degree angle
Year Built:	1985
Inspection date:	August 2008
Construction details:	Single span, dowel laminated Wheeler Bridge with an asphalt deck for a running surface.

Bridge Photos:

Figure 4.46. Bridge 402.

Figure 4.47. Bridge 402.

Figure 4.48. Bridge 402.

Bridge dimensions

Table 4.34. Bridge dimensions for Bridge 402.

Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	43.5	7.75	11.5	12	26.9	1,076.9
Spacer	none					0.0
Railing	384	6	10.5	2	28.0	1,120.0
Curb	384	11	5.5	2	26.9	1,075.6
Spreader	362	6	12	1	15.1	603.3
Deck	384	362	14	1	1126.2	45,048.9
Asphalt	384	362	3	1	241.3	36,200
					Total	**97,355.9**

Note: [1]A factor of 40 lbs/ft^3 was used for wood and 150 lbs/ft^3 for asphalt.

Vibration test data

Table 4.35. Vibration data collected from Bridge 402 for span 1.

Span 1				
Test	**Peak 1 Frequency (Hz)**	**Phase**		
		Black/Blue	**Red/Blue**	**Red/Black**
1	18.12	5.14	9.61	4.47
2	18.02	3.95	10.19	6.24
Average	18.07	4.55	9.90	5.35

Note: Hz = hertz

Static load test data

Table 4.36. Static load data collected from Bridge 402 for span 1.

Distance from Upstream (in.)	Initial zero load (mm)	Final zero load (mm)	Average zero load (mm)	Load Case No. 1 Reading (mm)	Load Case No. 1 Deflection (mm)	Load Case No. 1 Deflection (in)	Load Case No. 2 Reading (mm)	Load Case No. 2 Deflection (mm)	Load Case No. 2 Deflection (in)	Load Case No. 3 Reading (mm)	Load Case No. 3 Deflection (mm)	Load Case No. 3 Deflection (in)
2	780	780	780.0	776	-4.0	-0.16	781	1.0	0.04	780	0.0	0.00
33	796	796	796.0	786	-10.0	-0.39	795	-1.0	-0.04	792	-4.0	-0.16
86	783	783	783.0	769	-14.0	-0.55	781	-2.0	-0.08	777	-6.0	-0.24
103.5	781	781	781.0	765	-16.0	-0.63	778	-3.0	-0.12	772	-9.0	-0.35
141	773	773	773.0	758	-15.0	-0.59	767	-6.0	-0.24	760	-13.0	-0.51
174	775	774	774.5	762	-12.5	-0.49	764	-10.5	-0.41	760	-14.5	-0.57
192	766	765	765.5	755	-10.5	-0.41	752	-13.5	-0.53	751	-14.5	-0.57
228	760	759	759.5	755	-4.5	-0.18	744	-15.5	-0.61	746	-13.5	-0.53
261	759	758	758.5	756	-2.5	-0.10	742	-16.5	-0.65	749	-9.5	-0.37
279	754	754	754.0	753	-1.0	-0.04	739	-15.0	-0.59	747	-7.0	-0.28
312	752	752	752.0	751	-1.0	-0.04	742	-10.0	-0.39	748	-4.0	-0.16
363	733	733	733.0	733	0.0	0.00	729	-4.0	-0.16	733	0.0	0.00
AVG.	767.7	767.3	767.5	759.9	-7.6	-0.3	759.5	-8.0	-0.3	759.6	-7.9	-0.3
A	804	804		803			804			804		
B												
C	748	748		744			743			748		
D	783	783		783			783			783		

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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 51,670 lbs; Rear Axle Vehicle Weight = 37,310 lbs

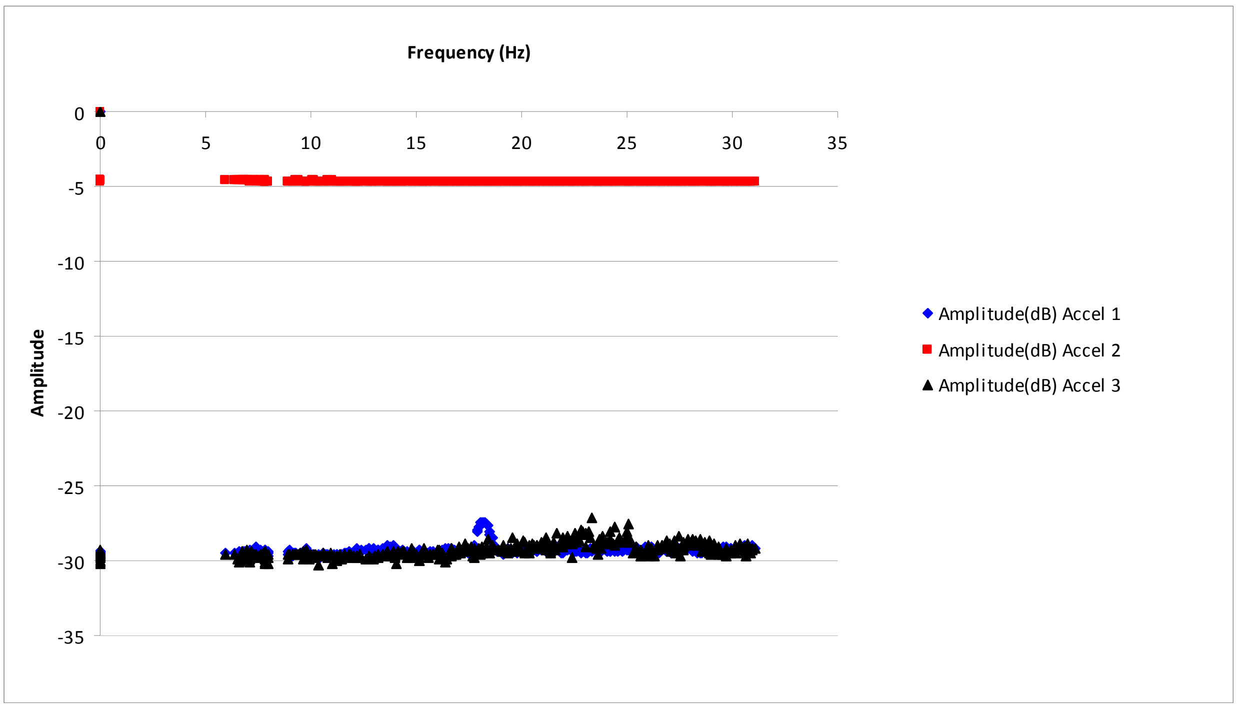
Results

Results obtained from vibration and static load testing for Bridge 402 are shown below.

Vibration frequency

Figure 4.49. shows a representative graph of the frequency data, with the following average peak frequencies for span 1; peak 1 = 18.07. The height of the bridge and water depth resulted in our inability to attach the motor directly to the bottom side, therefore it was attached to the top surface, through the asphalt wear layer. This resulted in poor connection and a loss of forcing energy to the bridge, resulting in poor signal response. There was only one peak detected, and the signal was very weak. A review of the data showed that the accelerometers were in phase for peak 1, but this is likely due to the poor signal, since all other peak 1 responses were out of phase.

Figure 4.49. A representative frequency pattern for St. Louis County Bridge 402.



Live load

Figure 4.50 shows the deflection of span 1 of Bridge 402 for center loading.

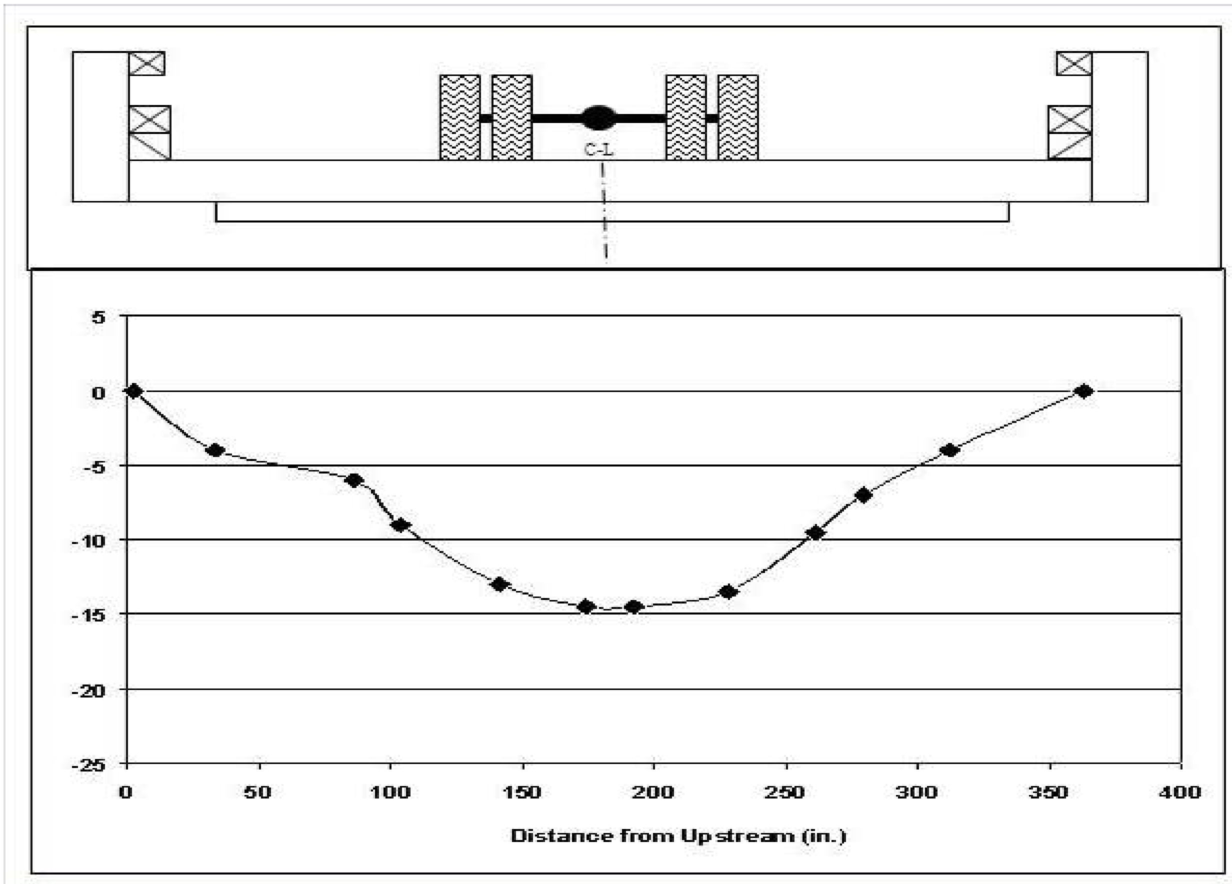


Figure 4.50. Deflection of span 1 of Bridge 402 for center loading.

Based on the load deflection testing, the following bridge stiffness was calculated as shown in Table 4.37.

Table 4.37. Measured deflection and stiffness for bridge 313 span 1 and span 2.

Bridge No.	Span No.	Span (center to center - ft)	Deflection		Measured Stiffness	
			Average (in.)	Maximum (in.)	Average (kip-in^2 x 10^6)	Maximum (kip-in^2 x 10^6)
402	1	29.83	0.31	0.57	111.5484325	60.66669134

Bridge 568 Testing Summary

Background

Structure:	Bridge 568
Location:	Brimson, Minnesota
Special Consideration(s):	None
Year Built:	2000
Inspection date:	August 2008
Construction details:	Single span, dowel laminated Wheeler Bridge with an asphalt deck for a running surface.

Bridge Photos:

Figure 4.51. Bridge 568.

Figure 4.52. Bridge 568.

Figure 4.53. Bridge 568.

Bridge dimensions

Table 4.38. Bridge dimensions for Bridge 568.

Component	Length (in.)	Width (in.)	Thickness (in.)	Quantity	Total Cubic Ft	Estimated Weight (lbs)[1]
Posts	46	8	11.5	10	24.5	979.6
Spacer	8	10.5	5	10	2.4	97.2
Spacer	78.5	5	13.25	1	3.0	120.4
Railing	264	6.75	14	2	28.9	1,155.0
Curb	264	5.75	11.5	2	20.2	808.2
Spreader	363.5	12	6	1	15.1	605.8
Deck	264	363.5	4	1	222.1	8,885.6
Asphalt	264	363.5	3	1	166.3	24,990.6
					Total	**37,642.4**

Note: [1]A factor of 40 lbs/ft^3 was used for wood and 150 lbs/ft^3 for asphalt.

Vibration test data

Table 4.39. Vibration data collected from Bridge 568 for span 1.

July												
Test	**Peak 1 Frequency (Hz)**	**Phase**			**Peak 2 Frequency (Hz)**	**Phase**			**Peak 3 Frequency (Hz)**	**Phase**		
		Black/ Blue	**Red/ Blue**	**Red/ Black**		**Black/ Blue**	**Red/ Blue**	**Red/ Black**		**Black/ Blue**	**Red/ Blue**	**Red/ Black**
1	16.43	3.77	179.61	183.38	21.41	7.89	47.92	40.04	26.69	12.42	27.06	14.64
2	16.52	6.83	178.44	171.60	21.46	3.95	35.69	31.74	26.50	3.07	15.98	19.05
3	16.43	0.97	178.85	177.88	21.46	14.40	36.34	21.94	26.29	9.79	28.81	19.02
Average	16.46	3.86	178.97	177.62	21.44	8.75	39.99	31.24	26.49	8.43	23.95	17.57

Note: Hz = hertz

Static load test data

Table 4.40. Static load data collected from Bridge 568 for span 1.

Distance from Upstream (in.)	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)
2	762	762	762.0	762	0.0	0.00	762	0.0	0.00	762	0.0	0.00
38.5	765	765	765.0	763	-2.0	-0.08	765	0.0	0.00	765	0.0	0.00
62.5	770	770	770.0	765	-5.0	-0.20	770	0.0	0.00	769	-1.0	-0.04
74.5	768	768	768.0	761	-7.0	-0.28	767	-1.0	-0.04	766	-2.0	-0.08
110	765	766	765.5	759	-6.5	-0.26	764	-1.5	-0.06	761	-4.5	-0.18
141.5	758	758	758.0	751	-7.0	-0.28	756	-2.0	-0.08	749	-9.0	-0.35
153.5	757	757	757.0	748	-9.0	-0.35	754	-3.0	-0.12	748	-9.0	-0.35
189	754	755	754.5	750	-4.5	-0.18	748	-6.5	-0.26	746	-8.5	-0.33
217	743	743	743.0	741	-2.0	-0.08	734	-9.0	-0.35	733	-10.0	-0.39
233	749	750	749.5	749	-0.5	-0.02	742	-7.5	-0.29	740	-9.5	-0.37
261	730	731	730.5	730	-0.5	-0.02	721	-9.5	-0.37	725	-5.5	-0.22
292.5	720	721	720.5	721	0.5	0.02	712	-8.5	-0.33	718	-2.5	-0.10
301	721	722	721.5	722	0.5	0.02	715	-6.5	-0.26	719	-2.5	-0.10
328.5	716	716	716.0	717	1.0	0.04	713	-3.0	-0.12	715	-1.0	-0.04
360	700	699	699.5	701	1.5	0.06	699	-0.5	-0.02	700	0.5	0.02
AVG.	745.2	745.5	745.4	742.7	-2.7	-0.1	741.5	-3.9	-0.2	741.1	-4.3	-0.2
A	748	748	748.0	748	0.0	0.00	748	0.0	0.00	748	0.0	0.00
B	768	770	769.0	769	0.0	0.00	769	0.0	0.00	768	-1.0	-0.04
C	708	705	706.5	707	0.5	0.02	706.5	0.0	0.00	705	-1.5	-0.06
D	697	696	696.5	699	2.5	0.10	696	-0.5	-0.02	696	-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 A (upstream left corner), Data Point B (upstream right corner), Data Point C (downstream left corner), Data Point D (downstream right corner)
Truck Weights: Gross Vehicle Weight = 52,190 lbs; Rear Axle Vehicle Weight = 38,330 lbs

<u>Results</u>

Results obtained from vibration and static load testing for Bridge 568 are shown below.

Vibration frequency

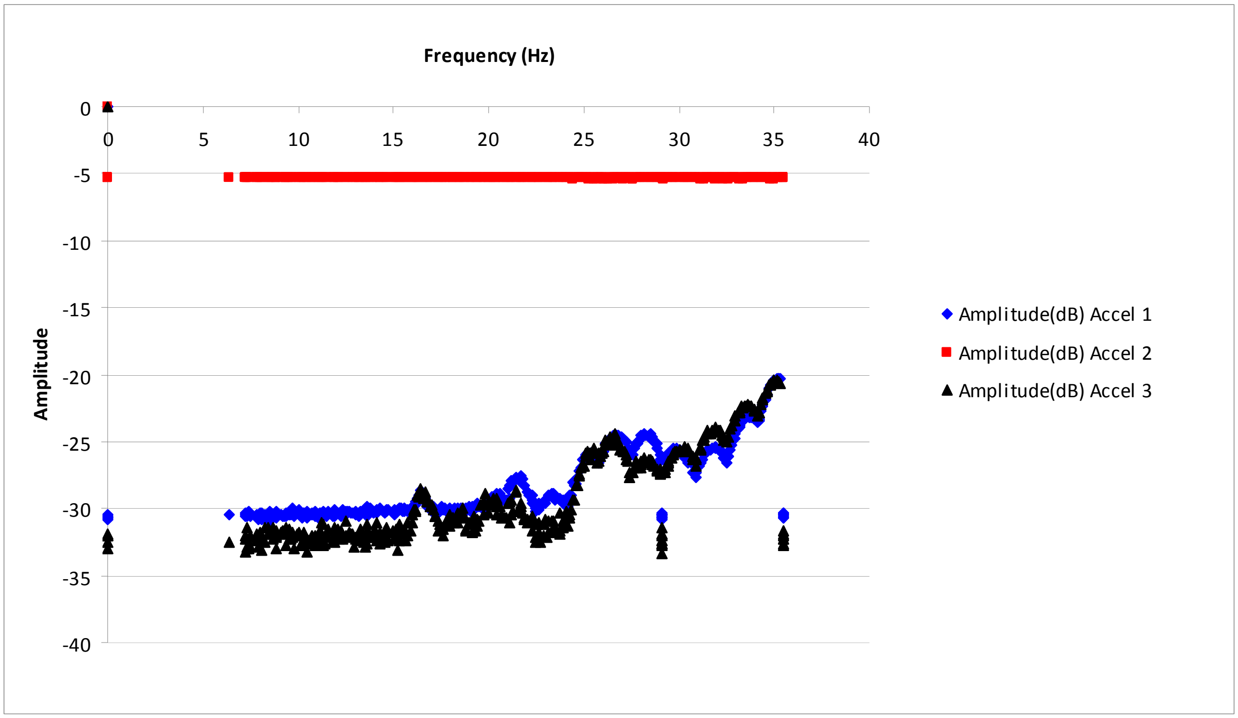


Figure 4.54. A representative frequency pattern for St. Louis County Bridge 568.

Live load

Figure 4.55 shows the deflection of span 1 of Bridge 568 for center loading.

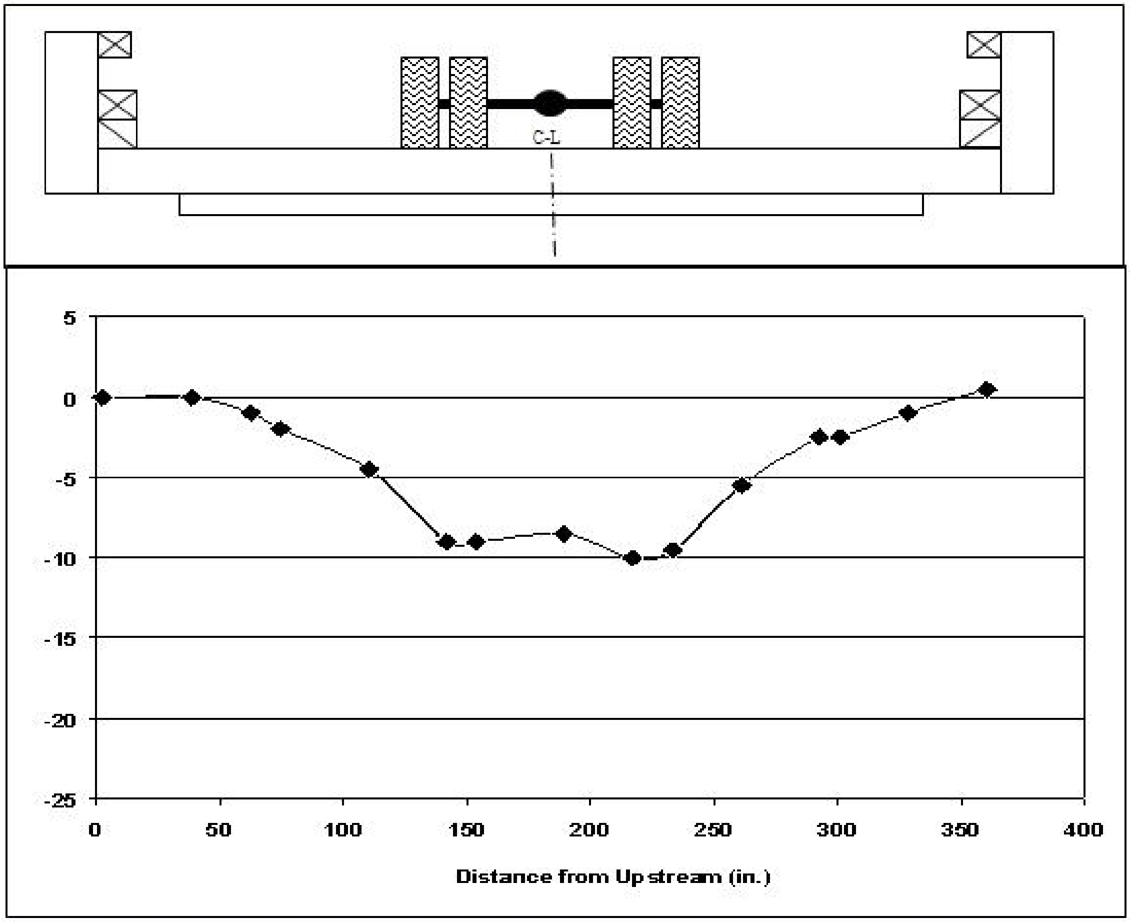


Figure 4.55. Deflection of span 1 of Bridge 568 for center loading.

Summary of Testing Results

Table 4.41 shows a summary of the results of the vibration and load testing for each bridge, to include the span, the measured EI product, and the frequency of vibration for each peak detected during testing. If a bridge had two spans, each span is reported independently. Static midspan deflections ranged from approximately 0.10 in. to 0.32 in. and were all less than the recommended L/360 span-to-deflection criteria. The vibration data showed a range of frequency peaks, with the first peaks ranging from 13 to 23 Hz. The second peak for each bridge ranged from 21.4 to 31.6 Hz. In some cases, a 3rd peak was evident, ranging from 29.8 to 37.0 Hz.

Table 4.41. Summary of static and dynamic bridge stiffness values.

Bridge No.	Static loading			Dynamic vibration		
	Span Length (ft)	Midspan deflection (in.)	Measured stiffness, EI* (x 10^6 lb-in.^{2})	Frequency (Hz)		
				Peak 1	Peak 2	Peak 3
53	23.00	0.13	119.45	16.6	23.5	29.8
182 – span 1	31.08	0.28	140.02	13.0	23.3	35.2
182 – span 2	31.50	0.28	145.88	14.4	30.2	37.0
263	15.50	0.10	44.71	23.1	34.4	-
304-span 1	18.83	0.10	75.59	17.6	25.3	-
304-span 2	17.33	0.12	48.38	17.2	26.4	-
313-span 1	17.17	0.12	46.97	18.4	31.6	-
313-span 2	31.67	0.32	118.48	-	-	-
313-span 3	17.17	0.14	40.26	18.3	31.5	-
383	19.88	0.11	201.92	20.5	30.1	-
402	29.83	0.31	111.55	18.1	-	-
568	20.10	0.17	61.48	16.5	21.4	26.5

* based upon beam theory equation [2]

Since full-scale modal testing was not in the scope of this project, we were not able to determine which frequency peak corresponded to which mode of vibration. The signals will continue to be analyzed in future projects in an attempt to determine the exact mode. It is believed that both peak 1 and peak 2 are bending modes, with peak one being a transverse mode and peak 2 being a bending mode. This assumption is based on a review of the phase readings from the accelerometers. The outer accelerometers, along the outer edges of the bridge were typically in phase with each other and 180 degrees out of phase with the center, mid-span sensor for peak 1 and all three sensors were in phase with each other for peak 2.

The first and second peak frequencies decreased slightly with increasing span length. This was unexpected since the EI stiffness increased as the span increased. Previous testing data for girder style bridges has shown that the frequency of vibration increases with EI. The graphical relationships for the frequency, span length and EI are shown in figures 4.56. and 4.57. The increase in measured stiffness as determined through load testing is due to the increased strength and stiffness requirements for the longer span bridges. This is primarily accomplished through increased cross-section of the bridge members for the longer spans, which we feel skews the relationship with frequency, which is more aligned with the modulus of elasticity of the material. A separate analysis focused solely on deflection tends to support this interpretation. Figures 4.58. and 4.59. show the relationship between frequency of vibration and deflection and frequency and deflection normalized by span length. To normalize the data, the deflection was divided by the span length, reported as inches of deflection per inch of span length. These graphs show that as the deflection increases, the frequency decreases. This is as expected.

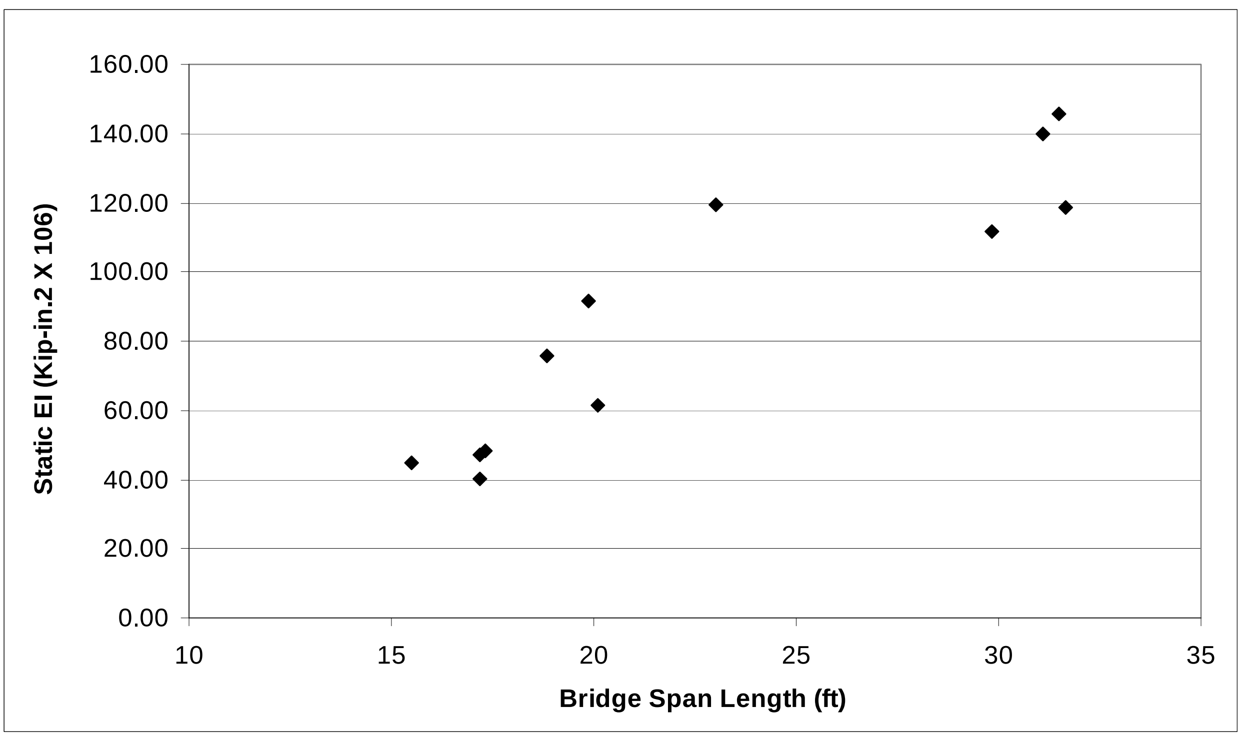


Figure 4.56. Relationship between the static EI measurement and the bridge span length for the bridges tested during this project.

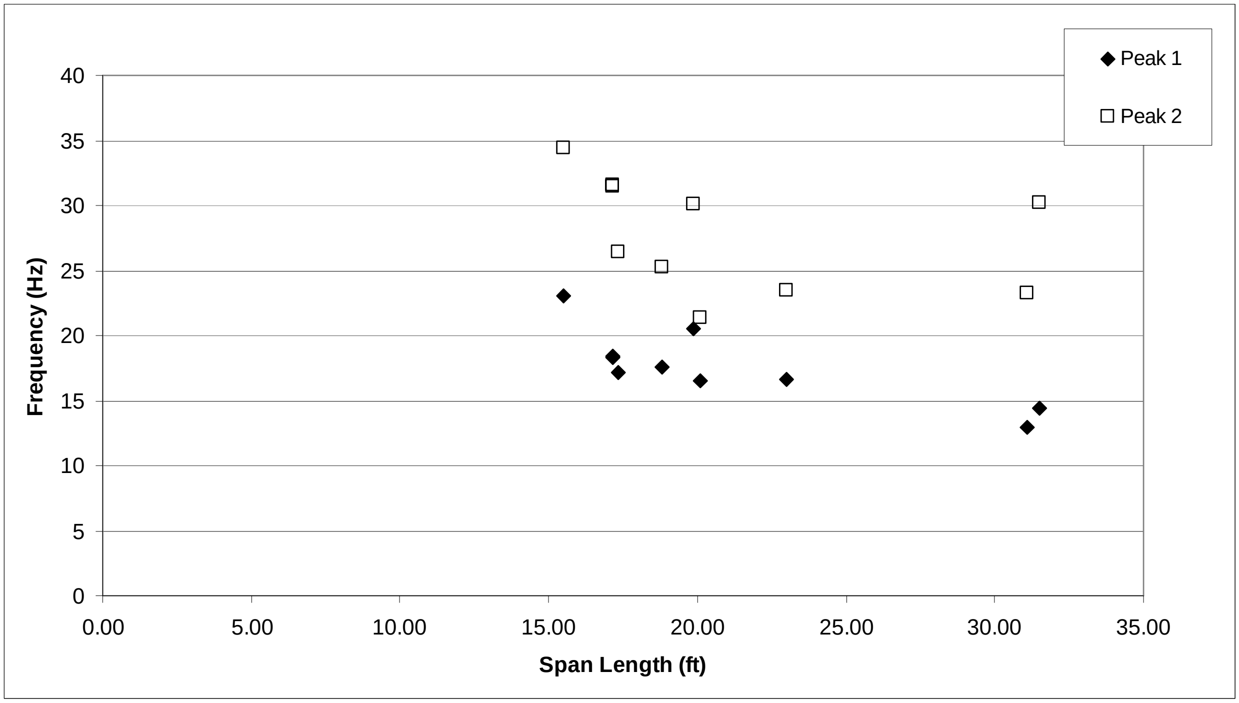


Figure 4.57. Relationship between the frequency of peak 1 and peak 2 to the span length for the dowel laminated bridges in this study.

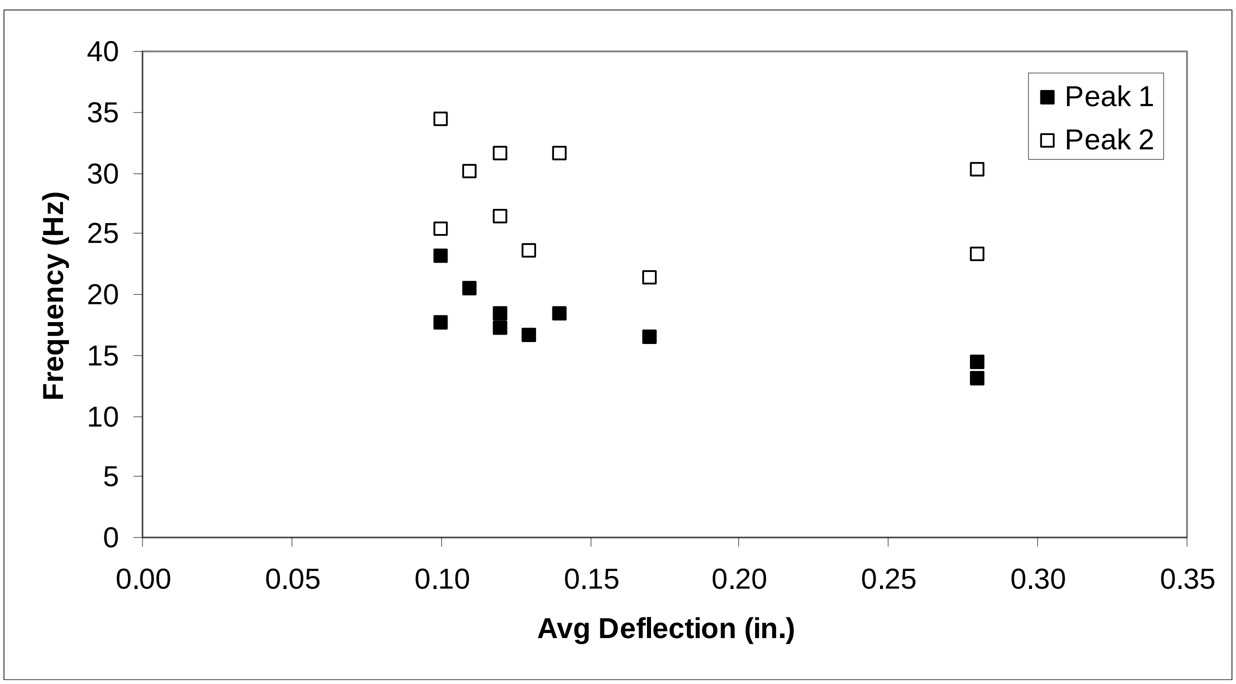


Figure 4.58. Relationship between the frequency of peak 1 and peak 2 average deflection for the dowel laminated bridges in this study.

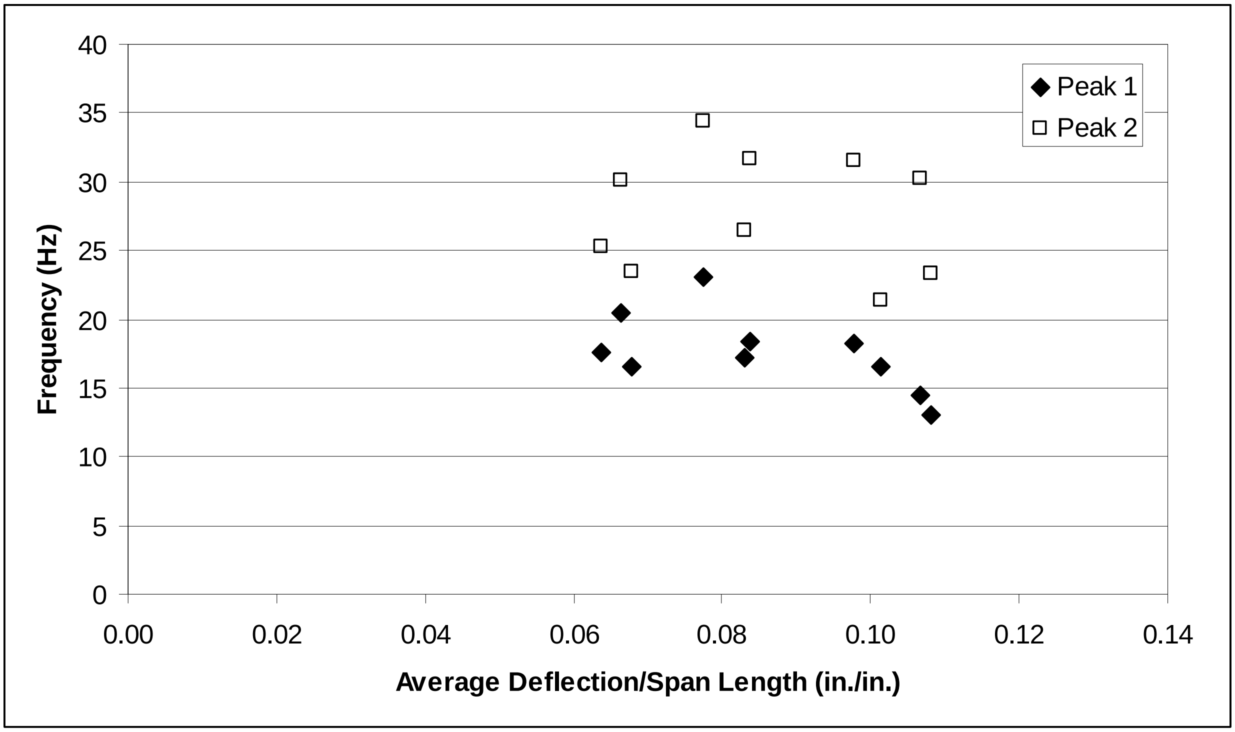


Figure 4.59. Relationship between the frequency of peak 1 and peak 2 and the average deflection/span length for the dowel laminated bridges in this study.

In order to better understand the relationship and to correlate the first and second 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:

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

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 $40{,}260 \times 10^6$ lb-in. to $154{,}189 \times 10^6$ lb-in. Figure 4.60. shows the relationship between the frequency for peaks 1 and 2 and the measured EI. Due to the limited number of data points, a regression was not completed on the data, but a visual review shows that the relationship was not overly useful, since the results are not as expected. This chart shows that as the frequency increases, the EI product decreases, which is the opposite of our hypothesis. We believe that this is because the EI computed using the beam theory is inappropriate.

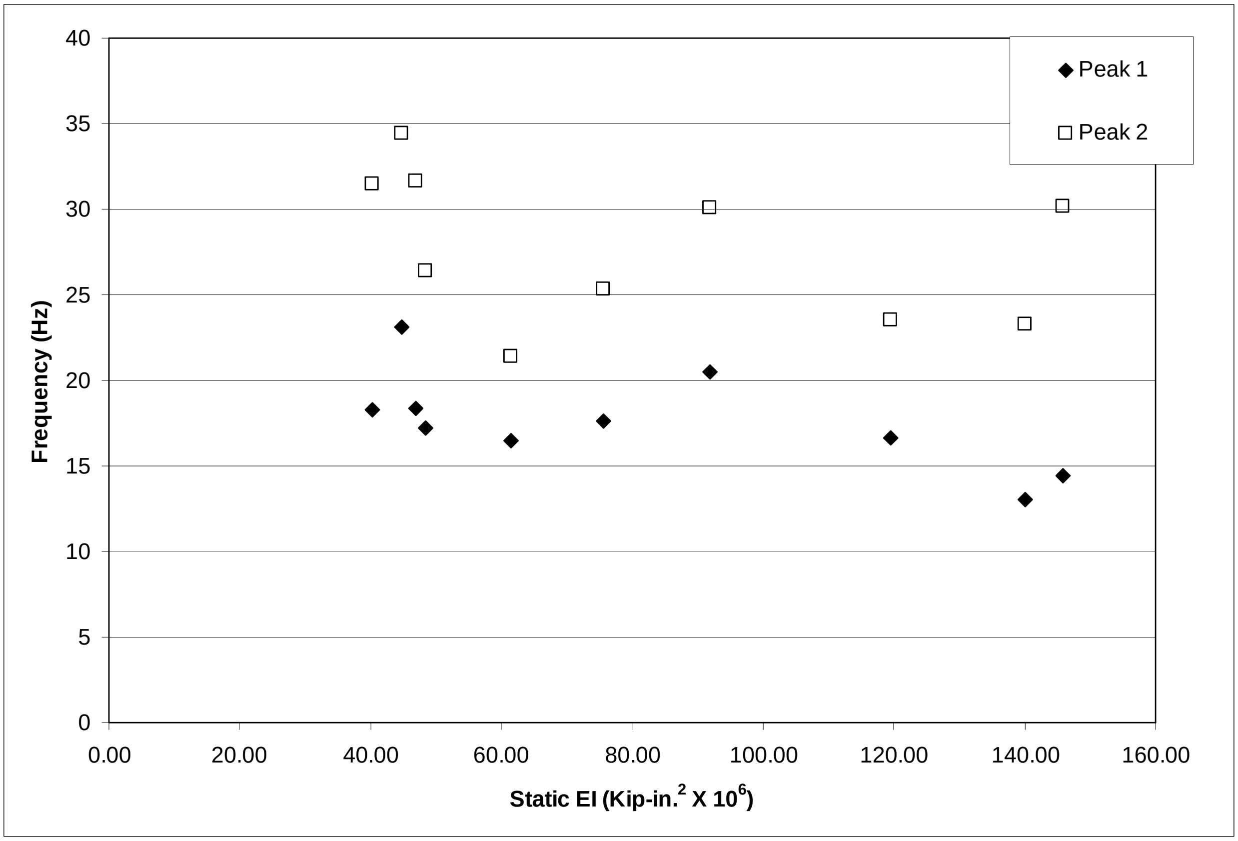


Figure 4.60. The relationship between the measured static EI and the peak 1 and 2 frequencies for the bridges tested in this study.

The estimates of frequency from forced vibration testing may contain significant errors in some cases. The error may be 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 with an estimated unit weight for the timber components. The true wood density of each bridge might be significantly different from the assumed unit weight. Other factors could also affect bridge weight, which make estimation difficult (such as species and moisture difference, wood deterioration, dirt or debris collected on the deck). Third, in spite of structure similarities, the boundary condition of each field bridge is unique due to the construction variability, load history, and road and soil conditions. The overall system parameter k used for EI prediction here is the average value of the system parameter k_i of each bridge, which describes the entire population.

A final consideration is that the assumptions that were made for calculating the EI of dowel laminated bridges using the beam theory are incorrect, resulting in errors in estimating the true EI. Future assessments should be made using a line load and appropriate EI equations will be selected in order to improve our predictive data. Instead, the signal response for each bridge is unique, and subject to change of time as the bridge deteriorates from various causes including wood decay, failed or broken connections, lack of maintenance among others.

Chapter 5: Conclusions

An automated testing system was developed and used to conduct forced vibration testing of dowel laminated timber bridges in northeastern Minnesota. The system controlled an eccentrically loaded motor attached to the bridge deck and collected data from several accelerometers mounted to the surface of the bridge deck. The conclusions of this project were:

- The forced vibration system developed is an effective tool for conducting forced vibration tests of timber bridges. The unit automatically controls and performs the testing, by increasing the RPM's of an eccentrically loaded motor attached to the bridge deck. It also collects the vibration response of the bridge, noting the frequency and amplitude of the signal. It is capable of measuring vibrational frequencies in the range of 14 - 35 Hz.
- As dowel laminated bridges are constructed, there is a noted increase in frequency during each successive stage of construction for vibrational peaks 1, 2 and 3. The various construction stages that affected rigidity were the placement of the individuals into place, so that the ends were restrained, the pinning connections of the shiplap joint sections together using 12-inch carriage bolts, the attachment and tightening of the curbing, deck rails and the spreaderbeam, and finally complete backfilling of the approaches to the bridge. During the 2008 construction of a new 2008 St. Louis County bridge, an frequency increase of over 5 Hz is noted for peak 1, an increase of nearly 8 Hz for peak 2 and an increase of nearly 13 Hz for peak 3.
- An accurate means of determining bridge weight and the true measured stiffness is essential for being able to relate the measured frequency against the measured EI product. Weight estimation based on wood volume and estimated unit weight for timber components is inadequate to obtain reliable results. The analytical models for simple beam and simple plate structures used to determine the EI product do not seem to effectively assess the structural performance of the bridge.
- A reliable means for assessing the peak frequencies and an identification of the mode still needs to be developed for this system to use the vibration response to predict the EI product for use in developing future load ratings.
- Each bridge has a unique set of vibrational characteristics that were identified using the automated system. These characteristics showed peaks in amplitude as the frequency of the vibration was increased from 0 - 35 Hz during testing. It is believed that monitoring of the characteristic vibrational response for each bridge would have merit as a means of identifying changes in structural health over time due to wood decay, accidents, vandalism, or lack of maintenance. For the purposes of this testing and potential implementation as a structural health monitoring tool, the results do show that the frequency of various peaks can be repeatedly and accurately determined over time. The challenge is to use this frequency data over time to identify any changes over time, especially decreases, since this would mean that the bridge has decreased in stiffness or rigidity. This may be due to various causes such as loose or failed connections, the presence of decay or other deterioration to wood or metal connectors, structural damage from an external force, or some sort of vandalism.

This technology does have potential application for use in monitoring rural bridge systems. In a separate project funded by the USDA Forest Service, we have instrumented a bridge near Meadowlands, MN with the vibration equipment used in this project. Monitoring has shown that the vibration response can be monitored long-term and that the signal changes seasonally based on typical winter-summer temperature shifts. The key question that must be determined is how the vibrational response will change due to structural detioration caused by decay or lack of maintenance. Since these problems typically require years to occur, it will take a long time to understand this affect. Further, the current system only assesses the global performance of the superstructure. Near-term implementation of this technology is possible, but further research of experimental bridges is necessary to address these issues. Construction and intentional rapid deterioration of the primary types of Minnesota timber bridges will provide key assessments of the technology, leading to opportunities to use this technology to assess rural bridges in St. Louis and other Minnesota counties. The use of this monitoring technology combined with stress wave and resistance microdrilling will improve rural bridge inspections, leading to increased safety and lengthened service life.

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