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Evaluation of Fleet Management Techniques for Timber Highway Bridges

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Abstract

The general condition of the nation's bridges presents a complex management issue when considering cost, safety, and time. Consequently, the management of those bridges can become an overwhelming task. The need for a management system that is specific to rural systems may help to improve the management of this significant number of bridges. Although individual bridges are often different when compared at the "detail" level, bridges with similar construction, material, or behavior may be able to be managed as a group rather than on an individual basis. The work described in this report seeks to study if such a "fleet management" approach is viable.

Keywords: timber, wood, bridges, management, maintenance

SI conversion factors

Inch-pound unit	Conversion factor	SI unit
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
pound (lb)	0.4536	kilogram (kg)
pounds/square inch (lb/in ²)	6.894	kilopascal (kPa)

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1 Introduction

1.1 General Introduction

Roughly 600,000 bridges are described in the U.S. National Bridge Inventory (NBI), a compilation of bridge data supplied to the Federal Highway Administration (FHWA) by individual states. Of the roughly 600,000 bridges, 28% of them (FHWA 2005) are considered functionally obsolete or structurally deficient. The generally poor condition of the nation's bridges presents a complex management issue when considering cost, safety, and time. Because of such a large number, the management of these bridges can become an overwhelming task.

1.2 Current Bridge Management Systems

Due to a mandate by the Intermodal Transportation Efficiency Act (ISTEA) of 1991, bridge management systems have been employed throughout the nation, and their use has been received with mixed opinions by bridge owners. A number of bridge management systems exist, including Pontis, one of the more commonly used systems (Small and others 2000).

Pontis was first developed through the FHWA, six state departments of transportation (DOTs), and a private consultant during the late 1980s. Individual bridge elements of the same material are assigned a quantitative measure of deterioration that represents the current state of that element. These measures are then used to determine the overall state of the bridge, and when combined with multiple bridges, a network-level condition can be formulated. Deterioration models based on the Markov process are developed and used to predict the condition state of individual bridges. A number of state and local DOTs currently use Pontis or a similar system, yet a need for a bridge management system that is specific to rural systems may help to improve the management of these bridges.

1.3 Fleet Management and Timber Bridges

The use of bridge management systems has vastly improved the state of bridge management, though conceivably—by taking advantage of the nature of bridge behavior and

construction and material similarities—great improvements can still be made. Even when individual bridges are compared and proven to be different in many facets, similarities exist in construction or behavior, and tendencies can be identified that promote the implementation of a group-based management system. In short, bridges with similar construction, material, or behavior may be able to be managed as a group rather than on an individual basis. This concept is derived from fleet management techniques found in other industries including, but not limited to, trucking, airline, and busing.

Fleet management techniques are necessary to be competitive in other industries (Clark and others 1994). Cost efficiency, time savings, and safety are only a couple of the associated benefits. Though being “competitive” in bridge management is not as critical as in profit-driven industries, the benefits are desirable and can offer many advantages.

Timber was often overlooked as a bridge building material throughout the 20th century as advancements of steel and concrete nearly eliminated the use of timber in bridge projects. The steel and concrete industries were quite successful in advancing their products through a vast amount of research and relatively inexpensive material costs. Timber, however, offers a number of benefits for bridge construction, including its strength and light weight. Sensitivities to weather conditions and de-icing agents are minimal, and constructability and life cycle costs rival those of concrete and steel.

In an attempt to revitalize the use of timber in highway bridge construction, the U.S. Congress passed legislation known as the Timber Bridge Initiative in 1988, and the USDA Forest Service was assigned the task of administering the timber bridge program. Part of the USDA Forest Service, the Forest Products Laboratory, was assigned the research portion of the Timber Bridge Initiative. In 1992, as part of the Intermodal Surface Transportation Efficiency Act, the Forest Products Laboratory joined with the FHWA Turner–Fairbanks Highway Research Center to implement the FHWA timber bridge research program. As part of this program, university researchers have been employed to conduct research advancing timber bridge construction.

Although timber was widely overlooked as a primary bridge building material, a relatively large number of timber bridges still exist throughout the United States. Many of these bridges are nearing the end of their life, and much like the overall bridge population, these bridges are in need of maintenance and rehabilitation. Unlike steel and concrete bridges, a majority of timber bridges are constructed on rural low-volume roadways. Local jurisdictions often control these bridges, with very little funding for inspection, maintenance, or rehabilitation, and an effective management strategy that minimizes unnecessary actions would prove to be beneficial. A research study intended to develop a fleet maintenance strategy for rural timber bridges was developed at Iowa State University in cooperation with the Forest Products Laboratory and is the subject of this report.

This report details the process and development of the rural timber bridge fleet management strategy. Appendixes C to Q are 15 individual reports that summarize visual inspection and static load testing performed during summer 2006 as part of the development of the fleet management strategy. Those 15 reports document in detail the bridge details, types and locations of instrumentation, modeling parameters, and other information.

1.4 Objective and Scope

The objectives associated with this work are as follows:

1. Optimize system preservation activities by enhancing management approaches by taking advantage of structural similarities and better performance indicators.
2. Change management decisions from being based upon code-evaluated individual bridges to behavior-based evaluations of a bridge fleet.
3. Determine the concepts of and information needed for the adoption and implementation of fleet management strategies.
4. Assess the viability of fleet management strategies.

To satisfy these objectives, the scope of the project included five tasks: a literature review, fleet identification, field inspection and testing, development of management tool, and development of final conclusions.

The literature review was intended to identify if other research has been conducted that specifically attempts to implement a fleet management strategy for bridges and also to identify the process of fleet management practiced in other industries. A literature review allowed adapting conclusions drawn from previous research, avoiding duplication, and gaining insight to current practices.

Fleet identification basically consists of properly identifying bridges of similar characteristics. A fleet may consist of bridges with similar geometry, behavior, or performance. This research study limits the fleet to timber bridges, and other properties discussed later narrow the fleet even more.

Field inspection and testing are required to properly assess the condition and behavior under live load for a statistically representative sample of the fleet. The intention is to identify similar deterioration patterns, behavior, or performance.

The previous tasks were used to develop a management tool to use for fleets of bridges. This tool gives guidelines for the maintenance schedule in order to preserve a certain level of performance within the bridge fleet. From the previous four tasks, recommendations and conclusions of the research study were developed. Explanations and limitations of the management concept are discussed as well as points needing further exploration.

1.5 Report Content

This report discusses the process of developing a rural timber bridge management system using a fleet management strategy. Section 2 provides a literature review of fleet management strategies utilized in other industries, such as trucking or busing. An emphasis is placed on preventive maintenance programs and their benefits. A summary of the fleet strategy and statistical sampling methods of two bridge research studies is included. Also, the California Bridge Health Index is explained, which is partially adapted to this research. Section 3 provides the fleet management of timber bridges concept; the evaluation methodology used outlines methods for obtaining information necessary to develop the bridge management concept. Next, the development of the bridge management concept by creating a four-level performance metric is explained. Section 4 discusses some applications and limitations of the management concept, and conclusions and recommendations from the work are presented. Appendix A provides a summary of individual contributions to the performance parameters, Appendix B introduces the individual bridge performance reports (Apps. C to Q).

2 Literature Review

2.1 Fleet Management Concept

Numerous industries continue to develop and practice fleet management. Most individuals associate fleet management with the transportation industry; however, the term fleet can also describe machinery, computers, or even bridges. In fact, “a fleet refers to multiple units of an equipment type” (Cassady and others 1998). Stated simply, fleet management is the overseeing of a fleet. According to Wyrick and Storhaug (2003), “Fleet management comprises all actions needed to maintain and operate pieces of equipment throughout its life from the beginning stages of equipment acquisition to the final stages of asset disposal. Such areas include maintenance and repair, inventory control, training, and safety issues.”

Proper fleet management can improve operational efficiency and effectiveness. For-profit industries require proper fleet management in order to successfully operate a competitive enterprise. However, only in recent history has fleet management become an elaborate matrix of techniques

and procedures. Fleet managers are responsible for the seemingly countless aspects of fleet operation including, but not limited to, budgeting, equipment maintenance, and tracking expenses. Without an appropriate strategy or methodology, fleet management can quickly become a daunting task.

2.2 Fleet Management Methodology

Five significant topics of fleet management methodology are discussed in this section: (1) centralization of fleet management, (2) fleet determination and standardization, (3) maintenance management systems, (4) benchmarking, and (5) data, knowledge, and information sharing.

2.2.1 Centralization of Fleet Management

If the creation of centralized fleet management is done correctly, it may provide efficiency and productivity, consequently avoiding unnecessary costs. A fleet manager will more easily encourage and have a clearer view of standard procedures and cost awareness (Clark and others 1994). Establishing standard policies and procedures is important to the development of uniformity. Another benefit of centralization is the timely manner in which fleet management tasks, such as data collection and performance assessment, can occur. Time and money are more easily lost when one entity of fleet management is not working in cooperation with others. Additionally, comparison between multiple fleets is more simply achieved, and duplication of common tasks can be avoided.

2.2.2 Fleet Determination and Standardization

A further point of general fleet management methodology involves determining the appropriate fleet size and establishing the starting point for fleet operations. Fleet management is affected by fleet size, so fleet-size-dependent policies should be created. Fleet sizing is dependent upon numerous factors including, but not limited to, similar make, age, and condition. The principal factors affecting fleet maintenance costs are fleet age and condition. Advanced age for a fleet drives up both direct maintenance and repair costs. Establishing fleet condition is essential to fleet operations. This task may be started while determining fleet size. If the initial condition is not properly determined, the starting point for fleet operations will be incorrect. Because all factors are not objective, a fleet manager may be required to use subjective judgment when determining fleet size (Abrams and others [no date]).

Standardizing the fleets also aids in efficiency and productivity. Similar components can be monitored, and a greater probability exists that unacceptable trends will be found. By standardizing fleets, potential problems among similar components can be identified and corrected before the issue becomes unmanageable. Financial comparisons are more easily made. Short-term costs are tracked more effectively, and forecasting long-term costs becomes more accurate.

2.2.3 Maintenance Management Systems

Perhaps the most important aspect of a fleet management methodology is the maintenance management systems. The greatest potential for cost savings rests in maintenance programs. Currently, most maintenance practices rely on failure-first methods, where maintenance procedures are performed only on components that malfunction and are necessary to the overall performance. With respect to motor vehicle fleets and machinery, component failure is a costly ordeal. Repeated breakdowns occur while in service, thereby reducing productivity and efficiency. The believed remedy to this problem is the implementation of a preventive maintenance program. Preventive maintenance programs are further explored within this section.

2.2.4 Benchmarking

Benchmarking is simply a method of comparing costs and performance of a fleet with other fleets that are deemed to be the best in a respective industry. More specifically, benchmarking is “a continuous systematic process for evaluating the products, services, and work processes of organizations that are recognized as representing best practices for the purpose of organizational improvement” (Galletti and Lee 2002). Organizations that do not practice benchmarking are able to compare a fleet only to the past performance of that same fleet. Utilizing cost and performance data from a singular fleet is good practice; however, there is no way to indicate existing deficiencies that are normally identified when compared to other fleets. Benchmarking sets the standard, and in the case of the transportation or manufacturing industries, standards create a competitive industry. Regardless of the industry or organization, when benchmarking, the potential for significant cost savings exists.

Comparing particular attributes of costs and performance, such as for maintenance or inspection, allows one to identify specific areas of excellence or needed improvement. When addressing bus maintenance, Maze (1987) recommends the use of performance measurement in fleet management. Maze divided fleet performance measures into six fundamental areas: fleet reliability indicators, fleet maintainability indicators, fleet availability indicators, maintenance work quality indicators, maintenance work productivity indicators, and maintenance control indicators.

Table 1 specifies each performance measurement area and what each addresses. Though each area may not be applicable to all organizations and industries, it is important to acknowledge the benefits of performance measures to benchmarking.

A six-step benchmarking process is described by Galletti and Lee (2002):

1. Determine the focus of the benchmarking study.
2. Understand the organization in order to understand the process to be benchmarked.

Table 1—Performance measurement areas^a

Performance measurement area	Description
Fleet reliability indicator	Addresses the likelihood of a fleet member operating properly at any given time
Fleet maintainability indicator	Addresses the costs needed to operate the fleet and perform maintenance
Fleet availability indicator	Addresses the likelihood of a certain number of fleet members being in operation at any given time
Work quality indicator	Addresses the quality of work performed on any fleet member and whether the work properly corrected an existing problem
Maintenance work productivity indicator	Compares the amounts of time used to complete similar tasks
Maintenance control indicators	Generally measures the maintenance activities of an organization in whole, providing information that describes how well the maintenance crew met the objectives of the organization

^aMason and others 2003.

3. Determine what to measure and determine the measures of performance.
4. Determine what to benchmark against.
5. Benchmark.
6. Improve performance.

This six-step process is a general process that can be used, but certainly more specific criteria can and should be developed relative to the particulars of the organization or industry. Fleet managers can adapt this process to specific objectives established by the organization.

2.2.5 Data, Knowledge, and Information Sharing

To further the efficiency and performance of fleets, a forum of data, knowledge, and information sharing should be created. Creating a forum effectively advances an organization beyond its capability as a single organization, and the gain of invaluable experience from other fleet managers will prove useful. The idea of a forum is especially important for fleet managers that do not control a large number of fleets or have large fleets that do not produce large amounts of usable information. By having access to other current information, fleet managers have, in effect, numerous fleets under their control.

Working in a cooperative manner enables each fleet manager to identify the concerns common to many or all fleet managers. Problems are discussable and better solutions are attainable when drawing from the experience and expertise of all cooperating fleet managers. Innovative methods that improve fleet management will result from a cooperative

forum. Following are descriptions of two examples of established information-sharing programs.

As part of a fleet maintenance program, the Transit Cooperative Research Program (TCRP) was developed and reauthorized as part of the Transportation Equity Act for the 21st Century (Abrams and others [no date]). TCRP was developed to “quickly spread information about best practices on various topics of interest to transit fleet managers and others.” TCRP has proven to be an effective way for fleet managers in the trucking industry to discuss maintenance issues. Similar programs, if not already in existence, should be created in the same fashion for other organizations and industries.

A web-based reporting program entitled Federal Automotive Statistical Tool (FAST) was developed for the U.S. General Services Administration and the U.S. Department of Energy. FAST was developed partly to automate the Federal fleet data reporting. Fleet managers with access to FAST have access to reports that have been generated through multiple fleets.

The internet is powerful tool that can be utilized to organize a forum. Information and experiences can be posted on a bulletin-board-type format, and specific data are easily downloadable. The internet allows fleet managers from all over the world to participate in a forum specific to his or her industry.

2.3 Preventive Maintenance

At the core of any fleet maintenance program is a preventive maintenance program. Preventive maintenance is defined as a “systematic servicing and inspection on a predetermined interval” (Dolce 1984). That preventive maintenance is beneficial to the life expectancy of bridges is well known. The problem is when and to what extent preventive maintenance should take place to maximize cost efficiency. A vast number of bridges are under the control of small jurisdictions, and bridge managers do not have unlimited funds. Even necessary maintenance is often restricted by limited financial resources. Because of the necessity for maintaining bridges, a manager is required to make the best decisions possible while accounting for limited fund and personnel allocations.

2.3.1 Objective of Preventive Maintenance

The objective of a preventive maintenance program is to minimize failure by consistently knowing the current fleet condition and correcting deficiencies before serious problems arise. A preventive maintenance program minimizes unscheduled repairs by ensuring that the developed scheduled maintenance policy is strictly followed, extends fleet life, and minimizes life cycle costs. A more detailed list of objectives adapted from the *Federal Fleet Management Guide* (Runzheimer 2005) follows:

- To maximize the useful life and reliability of the fleet while minimizing total life cycle costs.

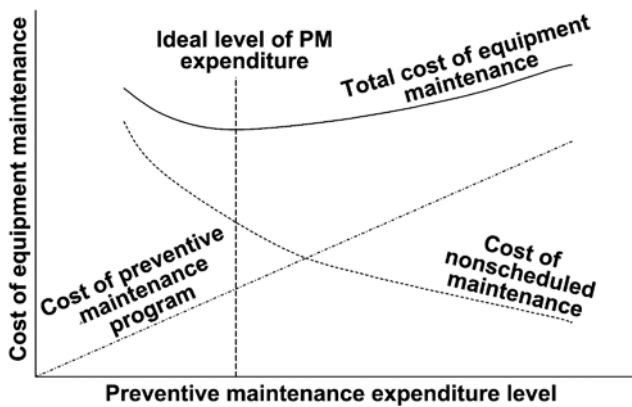


Figure 1. Effect of preventive maintenance levels on total equipment maintenance costs (adapted from Dolce 1984).

- To ensure that preventive maintenance work is scheduled at appropriate intervals.
- To ensure that preventive maintenance work is performed in compliance with schedules.
- To perform appropriate preventive maintenance tasks according to fleet requirements.
- To complete preventive maintenance in a timely manner.
- To minimize the cost of preventive maintenance to the organization.

2.3.2 Effects of Preventive Maintenance

Preventive maintenance involves identifying suspect areas and sources of premature malfunction based on previous experience. Once identified, one can service the suspect areas. Contradictory to what many believe, most often the total expenses associated with unscheduled repairs exceed those of scheduled repairs, provided that a proper preventive maintenance program exists (Dolce 1984). One needs to be aware of excessive preventive maintenance because this will create expenses that are not recovered through uninterrupted service. Properly minimizing the involvement of a preventive maintenance program will minimize overall expenses. Figure 1 shows the effect of preventive maintenance levels on total equipment maintenance costs.

2.3.3 Preventive Maintenance Development

To develop effective fleet-specific preventive maintenance programs, fleet managers should consult the manufacturer's recommended maintenance standards where applicable, fully understand the conditions by which the fleet operates, and keep and study reports of the fleet history. Individual experience and self-study are invaluable assets to the development of preventive maintenance programs.

Fleet history reports should be both comprehensive and individualized. Each fleet and preventive maintenance procedure should be identifiable by a code. These codes should

specifically detail the fleet, the maintenance procedure to take place, and when that maintenance procedure is scheduled. An example of this scheduled maintenance is seen in vehicle maintenance requirements located in a vehicle owner's manuals (for example, "change oil every 3,000 miles").

The preventive maintenance schedule should be designed to address fleet needs at various intervals. Preventive maintenance categories are often used to identify different needs and different times. Servicing is frequently divided into several different categories because all servicing does not require the same level of work. Dolce (1984) notes four different categories of planned maintenance: A, B, C and D. A involves frequent but minor service work; B involves less frequent but more complex work than that of A; C involves both minor and major testing and overhaul and replacement of certain components; D often involves planned replacement or rebuilding in entirety.

For a preventive maintenance plan to be effective, one should develop a simple and user-friendly schedule. Though one may find in the scope of fleet needs that maintenance procedures at irregular intervals is most effective, effectiveness is lost by the impracticality of the schedule. An effective preventive maintenance schedule should be mathematically consistent. For example, if category A maintenance is scheduled for every 6 months, then category B maintenance should be every 12 months, and category C every 24 months.

If unscheduled or emergency repairs are required, the preventive maintenance schedule should be checked so that any preventive maintenance can be concurrently administered if it is soon due. This will help maximize the effective use of limited resources.

To apply proper inspection techniques, inspections must be performed in a timely manner. Problems that pose minor concerns could develop into much more severe and costly problems if not for frequent inspections. Inspections should include review of reports, visual inspection of components, and any necessary testing that is deemed appropriate (Dolce 1984).

Preventive maintenance compliance reports should be generated. These reports will show if preventive maintenance procedures are not being strictly followed. Adjustments to the preventive maintenance schedule may be necessary. Highest priority should be given to those tasks that were not completed on schedule.

As part of servicing and inspection, detection of any problem areas should occur. These problem areas could be severe in nature and could cause failure. These problem areas should be properly noted. If it is found that the same components are failing at regular intervals, the fleet manager should identify those trouble-prone components and direct maintenance over them before failure.

Correction should take place as needed. Further damage could occur if not properly corrected. Further damage requires further expense, and further expense negates the original intent of preventive maintenance.

2.3.4 Planning Preventive Maintenance

Properly planning a preventive maintenance program is essential to controlling total maintenance expenses. The combined costs of preventive maintenance and unscheduled maintenance should be minimized. Spending large amounts of money on a preventive maintenance program will greatly decrease the expenditures related to component failure; however, when large amounts of money are spent, the amount of money saved in failure expenses may not cover the expenses associated with preventive maintenance. Prior experience is a major advantage in planning a program of this nature. One should avoid costly replacements or repairs unless prior experiences warrant those actions. Maintenance history and records also aid in planning (Dolce 1984).

2.3.5 Effects of Technological Advances on Preventive Maintenance

Traditional management coupled with technological advances has created a new array of management issues. Formerly, fleet managers relied mostly on experience and general knowledge. Today, technological advances have enabled fleet managers to better predict and optimize management activities and, more specifically, maintenance activities. Successful adaptation of technological advances greatly determines an organization's ability to reduce fleet costs through improved maintenance. Creating thorough maintenance records always is an issue of utmost importance, and this process continually becomes more economical as the cost of the technology continues to decrease. One has the ability to retrieve extensive databases very quickly, enabling fleet managers to identify trends and support management decisions. Predictability is quite possibly the greatest asset of electronic databases. A decision formerly dependent on past experience is now bolstered by statistical data. Predictability alone provides a significant platform for the reduction of cost.

Advantages of a fleet management information system are stated below (adapted from Clark and others (1994)):

- Large volumes of information can be input and statistically analyzed.
- Dispersed fleet operations work with standardized data definitions, data input fields, and data reports.
- Statistical history enables comparisons over time and across organizational divisions.
- Managers can more speedily identify problems and unearth answers to management questions.

- Query programs enable flexibility for selecting and extracting data and reporting in different formats and from different statistical perspectives.
- Computer-generated numbers carry an aura of truth and can thereby lend credence to strong policy enforcement or recommendations for changes in policies or programs.

2.4 Examples of Fleet Management of Bridges

Currently, limited literature exists that specifically addresses the application of fleet management techniques to bridge management. Two projects in particular have at least partially applied fleet strategies: (1) Re-Qualification of Aged Reinforced Concrete T-Beam Bridges in Pennsylvania and (2) Performance Assessment of Prestressed Concrete I-Girder Bridges in Michigan.

2.4.1 T-Beam Bridges in Pennsylvania

In May 1998, the Pennsylvania Department of Transportation (PennDOT) presented researchers with a need for determining the capacity of reinforced T-beam bridges in Pennsylvania (Catbas and others 2003). A large number of these bridges exist within the state, and an increasing concern for the overall condition developed. Given that the number of bridges was so large, inspection of each bridge individually was not feasible. Many similarities between the bridges existed, thereby originating the idea of treating the bridge population as a fleet. Although the bridges varied in material properties, geometry, structural details, and visual appearances, the idea formed that a few independent parameters controlled the load-resisting mechanisms and critical failure modes. In some ways this idea is controversial in that each bridge will not be treated as an individual bridge but rather as part of a group. Though the project did not revolve entirely around the idea of fleet management, as part of the overall project the researchers attempted to demonstrate the viability of fleet management of bridges.

According to the National Bridge Inventory of 2001, Pennsylvania had 2,440 T-beam bridges, third behind California and Kentucky. Of those bridges, 1,899 were single span and 60% were older than 60 years. The project was primarily directed to finding a more accurate load capacity than that derived from NBI condition ratings. Due to the vast number and high percentage of obsolete T-beam bridges, a revised, and expectedly increased, load capacity rating would improve the overall condition of the population of bridges and delay seemingly imminent maintenance, rehabilitation, or replacement.

Due to the large number of bridges within the population and virtual impossibility of a thorough inspection and load test of all, a plan to extract a statistically representative number of bridges was developed. A statistically representative sample of the entire population was intended to be a depiction of the independent parameters most likely to affect

Table 2—Critical parameters affecting load capacity ratings^a

Nominal structural parameters	Condition parameters	Information from PennDOT districts
Materials	Age	PennDOT documentation
Geometry	Climate	List of most critical bridges
Detailing	Location	District engineers feedback
Substructure	Maintenance	
Boundary conditions	Deterioration	
	Damage	
	Condition rating	

^aCatbas and others 2003.

the actual load capacity rating of each bridge. One should note that a majority of the Pennsylvania T-beam bridges were constructed using a standard set of drawings for which the structural details and element dimensions are dependent on the span length and width of the bridge; thus, radical geometrical variation in the bridges was most likely eliminated. The most critical parameters that were assumed are presented in Table 2. A statistical analysis of bridge characteristics was performed, and a sample of 60 bridges was selected.

It was assumed by the authors of the Pennsylvania T-beam project that by studying a small sample of the entire population the load ratings for individual bridges in the sample may be determined. Though the term “representative statistical sample” was frequently used throughout the publication describing the study, statistical justification and tests were never identified. Even so, it is assumed by the authors of this report that statistical justification was in fact completed for the sample of 60 bridges. One should take note that of the total 1,899 single span T-beam bridges, complete information of only 1,651 was documented, from which the sample of 60 was taken.

2.4.2 I-Girder Bridges in Michigan

A new management procedure was desired for Michigan’s prestressed concrete (PC) I-girder bridges (Birgul and others 2003). The objective of this project was to develop specific remedies for protection and repair corresponding to developed states of distress. A large percentage of the total PC I-girder bridges were constructed during the 1960s, and several construction methods that were practiced during this decade are no longer practiced; a number of deterioration patterns have been observed in these bridges. The result is a bridge population with various rates and degrees of deterioration. A method to describe the condition state of the entire population was therefore deemed necessary.

At the time of this study, a total of 699 PC-I girder bridges in Michigan existed, of which 345 were constructed from 1960 to 1970. Aside from these older bridges being

designed for significantly lighter loads than current bridges, the most major difference in the bridge construction was at the girder ends. Currently, the most common practice for girder end construction involves casting the girder ends into diaphragms over the respective abutment or pier. Previously, these I-girder bridges were constructed using simple span techniques where a discontinuity existed at the abutments and pier locations. In combination with relatively severe environmental conditions, de-icing agents, and increased truck traffic, it has been found that this detailing promotes accelerated deterioration at the girder ends.

A more thorough inspection and documentation of the total population of I-girder bridges in Michigan were necessary to better evaluate the process of protection and repair methods. However, as with any large population, a thorough field investigation of each bridge was not feasible. Instead, these bridges were classified and identified according to their characteristics, and a statistical representative group of 20 was selected for a detailed field inspection. Much like the Pennsylvania T-beam research, specific statistical methods for determining the sample size were not identified within the report. Even so, it is again assumed that statistical methods were used in determining the total sample size.

2.5 California Bridge Health Index

The California Bridge Health Index (Shepard and Johnson 2001) is a diagnostic tool developed by the California Department of Transportation (Caltrans) with intentions of maximizing duration of service while minimizing life-cycle costs of California bridges. The Bridge Health Index is a single-number assessment of a bridge’s condition based on the bridge’s economic worth, determined from an element-level inspection guided by the CoRE AASHTO Guide for Commonly Recognized Structural Elements (Thompson and Shepard 2000).

Two primary differences exist between the California Bridge Health Index and the Federal Highway Administration’s Sufficiency Rating (FHWA 2005). First, the Bridge Health Index is based on an element-level inspection, whereas the Sufficiency Rating takes into account only the single number assessment of each of three components (deck, superstructure, and substructure). Multiple components of the deck, superstructure, and substructure exist, and therefore a problem arises when a single number assessment is intended to describe the condition of the entire element. The Bridge Health Index can describe elements of the bridge individually, thereby providing a more accurate assessment of the bridge condition. Second, the Bridge Health Index accounts only for the structure condition, whereas the Sufficiency Rating includes a bridge’s function (for example, traffic carried by the bridge and capacity in relation to traffic demand). Thus, a bridge that may be considered in good condition by the Bridge Health Index could be considered obsolete on the Sufficiency Rating scale.

The California Bridge Health Index has a number of economic applications and benefits. This study, however, focuses on the bridge condition description. At the element level, a total quantity of each respective element that is present throughout the bridge is determined; from that quantity, the percentage that lies in each of five condition states is apportioned. Unlike the Sufficiency Rating, the condition of individual elements can be described by multiple condition states at the element level.

3 Fleet Management of Timber Bridges Concept and Application

As previously stated, objectives of this project were to illustrate the viability of fleet management strategies and to develop a bridge management system for rural bridges using a fleet management technique. To reach the objectives it was necessary to first complete four major tasks to collect the information that would be needed for this and any fleet management program: fleet identification, information requests, visual inspection, and static load tests. These four tasks are described in the following sections. Subsequently, the development of the management approach is given.

3.1 Information Collection

To identify a particular fleet to examine using fleet management approaches, the National Bridge Inventory (NBI) database must be investigated. Only the bridges categorized as a timber structure were identified and considered for use in this work. Of the roughly 600,000 bridges in the NBI, approximately 30,000 are considered timber structures. The Bridge Engineering Center (BEC) of Iowa State University has previous experience with the inspection and testing of timber structures throughout the United States. Of those bridges, a number have been timber girder bridges with a bituminous wearing surface. To create continuity among past research projects and to apply past investigative experience, a similar sample of timber bridges was identified. This subsample of timber bridges with a bituminous wearing surface was further reduced by applying structural similarity criteria. Specifically, only multi-beam or girder, single-span bridges were considered. In the 2005 NBI database, 2,703 bridges were categorized as single-span, timber girder bridges with a bituminous wearing surface.

The procedure for random sampling is described in the following paragraphs. The process of random sampling was performed using a random number generator, and individual bridges were selected if the random number corresponded to the assigned number. Once sampled, available bridge information was evaluated for completeness, and if found to be incomplete, another random number was generated and the corresponding bridge replaced the previous.

To determine the sample, the central limit theorem (Mason and others 2003) and X^2 goodness-of-fit test (Crow and others 1960) were employed using what the authors considered

“valid” numerical bridge parameters. These parameters included such characteristics as age, length, and width. By removing all other parameters and varying characteristics of a bridge and focusing solely on a single descriptor (such as length), a sample representative of the population with respect to that descriptor could be identified. It is known that a sample representative of the population with respect to length, for example, is not necessarily representative of the population in every aspect.

The central limit theorem states that regardless of the true probability distribution of individual observations, the standard normal distribution can be used to approximate the distribution of the sample mean. Simply stated, in spite of a population distribution being decidedly non-normal, the mean of random samples with adequate size will be normally distributed about the population mean, only if the sample is independent and is from a single probability distribution having mean μ and standard deviation σ (Mason and others 2003). It is suggested that a sample size of 30 or larger is typically acceptable and the normal distribution will adequately approximate the sampling distribution. A smaller sample size of 5 to 10 is adequate to apply the central limit theorem as long as the distribution is normal or slightly skewed.

Maisel (1996) provides mathematical justification for samples of certain sizes. Assuming the sample is independent and from a single probability distribution having mean μ and standard deviation σ , determining the sample size is completely dependent on the shape of the population distribution. If the distribution of the population is normal, then theoretically the mean of a sample consisting of one observation would be normally distributed. If the population is not normal but is symmetrical, then a small sample would still have an approximately normal distribution. If the distribution of the population is largely skewed, then a large sample would be necessary to achieve a normal distribution for the sampling distribution. One can see how the population distribution shape is important to the sample size. To find the correct sample size, the population distribution shape must be accounted for.

Maisel incorporated a skew index (SKEW) into sample size determination. The sampling distribution of the mean for simple random samples will approximate a normal distribution if

- the population has a skew index in the range of +1 to –1 and the sample size is 25 or more, or
- the population distribution has a skew index outside the range of +1 to –1 and the sample size equals $25 \times \text{SKEW}^2$.

The skew index is evaluated by

$$\text{SKEW} = \frac{(A - B + C)}{(\text{Standard deviation})^3} \quad (1)$$

where

$$A = \sum_1^N \frac{(n_i)^3}{N}$$

$$B = 3 \times \mu \times \sum_1^N \frac{(n_i)^2}{N}$$

$$C = 2 \times \mu^2$$

and

N = total number in population

n = individual population values

μ = mean of the distribution

Using these equations and applying them to the categories of age, width, and length of the timber bridge population previously described, sample sizes of 25, 28, and 33 were obtained, respectively.

The chi-squared (X^2) goodness-of-fit test (Crow and others 1960) can be used to determine if a sample distribution of a single parameter (such as length) represents the population distribution of that same parameter. In this test the null hypothesis (H_0) states that the proportions in the sample are similar to the population proportions or, more simply, the sample is a good fit. Conversely, the alternative hypothesis (H_a) states that at least one of the proportions is different or is not a good fit. The population distribution is divided into n bins or categories, with a bin being any length provided that the expected frequency in that bin is five or greater. The observed frequency, O_i , in the sample distribution is compared with the expected frequency, E_i , by means of

$$X^2 = \sum_{i=1}^n \frac{(O_i - E_i)^2}{E_i} \quad (2)$$

The observed frequency is simply the number of bridges within the overall population observed to have values within a certain bin. The expected frequency is the number of bridges expected to fall within a certain bin for a particular sample size and is found by multiplying the sample size by the observed frequency and dividing by the total number in the population.

If the calculated X^2 statistic is greater than $X_{\alpha, n-1}^2$ for $f = n - 1$ degrees of freedom, at the significance level α , then the null hypothesis that states the sample distribution is similar in shape to the population distribution is rejected. Conversely, if the opposite is true, then the null hypothesis is not rejected and the sample distribution is considered similar in shape to the population distribution.

Several sources state that the sample size is required to be sufficiently large to apply the X^2 goodness-of-fit test. The

term “sufficiently large” is subjective and therefore a specific sample size is not indicated nor is a minimum sample size specified. One source suggests that a sample size as few as 20 is adequate for accurately representing the population with respect to a single parameter (Garson 2009). One downfall of a smaller sample size is that one is at risk for Type II errors. Type II errors are not rejecting the null hypothesis when in fact the null hypothesis is false, effectively stating that a given sample is an accurate representation of the distribution of a population when in fact the sample is not. Alternatively, Type I errors are rejecting the null hypothesis when in reality the null hypothesis is true.

The choice of a significance level α is largely subjective and is often determined by the necessity of not committing Type I or Type II errors. The probability of committing these errors decreases with a decreased significance level; a more conservative approach is put into practice by decreasing the significance level. In other words, a sample distribution must adhere more closely to the population distribution to avoid being rejected when the significance level is lowered, or the probability of a test statistic being significant is increased when the significance level is lowered. Typically, a significance level of 0.05 or 0.01 is used (Crow and others 1960).

Based on the information gathered addressing sample size, the minimum suggested sample size of 20 was used and the X^2 statistic was calculated. A significance level of 0.05 was selected.

Table 3 displays the calculated X^2 statistics and the X^2 critical statistics for the six National Bridge Inventory (NBI) categories studied. It shows that for each category, the sample provides a good-fit distribution and is representative of the overall population at a significance level of 0.05.

It was concluded that a random sample of at least 20 bridges would sufficiently represent the population with respect to parameters investigated. Thus, the research team decided to randomly select 100 bridges for which information would be requested. This was done to ensure that “complete” information would be obtained for the minimum number of 20 bridges.

3.1.1 Information Requests

An attempt was made to contact the bridge owners of the 100 randomly sampled bridges so the latest inspection report could be obtained. The jurisdictions of states, counties, cities/towns, and federal forests constituted the bridge owners of the 100 sampled bridges. In the end, a total of 62 reports for single-span, solid-sawn girder timber bridges with a bituminous wearing surface were obtained from various jurisdictions throughout the United States.

Although sampling procedures were random, it was not feasible to continue the investigation into all those bridges. Due to budgetary and time restraints and bridge owner

Table 3— χ^2 comparison for sample size of 20

NBI category	χ^2	$\chi^2_{0.05, n-1}$
Sufficiency rating	1	5.99
Span length	1.67	5.99
Bridge width	4.97	5.99
Skew angle	0.23	3.84
Average daily traffic	0.69	3.84
Age	0.16	3.84

cooperation, a group of 23 bridges was selected based primarily on geographic location and general compatibility with the study. Sixteen bridges located in western North Carolina, four bridges in western Montana, and three bridges in Colorado were selected for visual inspection. All but eight of the bridges in western North Carolina were also selected for static load testing. Although the inspected and tested bridges were not completely chosen at random, the authors felt that the goals could still be met.

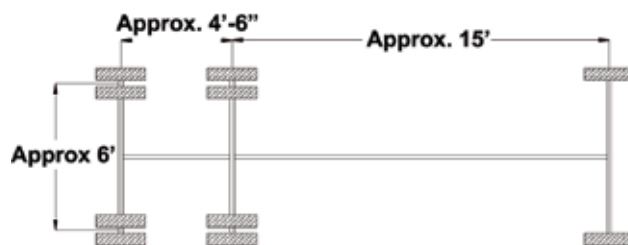
3.1.2 Visual Inspection

The process of visual inspection was formalized by a single report format developed after Ritter (1990) and White and others (1992). The form consisted of eight major sections: (1) General Information, (2) Bridge Geometry, (3) Overall Structure Inspection, (4) Deck Inspection, (5) Superstructure Inspection, (6) Substructure Inspection, (7) Moisture Content, and (8) Comments/Remarks. Much of the form was created to identify deterioration modes specific to timber bridges.

The general information and bridge geometry portions of the form simply identify the bridge and the necessary geometry information. The overall structure inspection portion was intended to note symptoms of deterioration that were present in more than one of the subcategories that followed. The subcategories of deck inspection, superstructure inspection, and substructure inspection were intended to specify deterioration particular to those elements. Moisture readings from various locations throughout the deck and girder elements were noted in the moisture content portion, and any other applicable comments not previously noted followed in the comments/remarks portion.

Aside from the general information and bridge geometry portions of the form, much of the form is directed towards identifying deterioration. For example, staining or discoloration, vegetation, and odor may all be signs of biotic growth infiltration. Sagging, crushing, holes, frass, and powder posting may be signs of insect inhabitation and wood rot. Generally, knots, sloped grains, or cracks are the result of poor wood grade, excessive service loads, fluctuating moisture contents, or aging.

With timber bridges, the origin of deterioration is often a direct result of deterioration in the wearing surface and deck.

**Figure 2. Typical load truck.****Figure 3. Typical truck dimensions.**

Emphasis was placed on these elements in the inspection form as it was thought this would best capture the current condition. Wearing surface cracking and deck board detachment are examples of deterioration that can vastly affect the condition of the entire bridge.

A number of moisture content readings were taken at several locations throughout the bridge structure using a two-prong electric resistance moisture meter. Moisture content of timber can significantly alter bridge performance under load. An increase or decrease in moisture content can result in fluctuations in the modulus of elasticity, cause shrinkage and swelling, and provide a catalyst for rotting and other deterioration.

3.1.3 Static Load Testing

To determine the behavior of selected bridges under live load, static load tests were performed on a total of 15 bridges, including 8 bridges in western North Carolina, 4 in western Montana, and 3 in Colorado. The test procedures and instrumentation methods were identical for all load tests. Static loading of each bridge was completed using a tandem-axle dump truck provided by the bridge owner, and the dimensions of each truck, though not identical, were approximately the same. A typical truck and truck dimensions are shown in Figures 2 and 3, respectively.

The total combined weight of the truck and the load varied for each test; 45,000 to 55,000 lb was typical. Deflection



Figure 4. Typical deflection gage instrumentation.

data were collected through the use of ratiometric potentiometers manufactured by Celesco Transducer Products, Inc. (Chatsworth, California), and the signals from these instruments were collected using an Optim Megadac 3415AC (Germantown, Maryland) data acquisition system running TCS (Dublin, Ohio) windows software. A typical set up of deflection gages is shown in Figure 4. Because of the

relatively short span and the need for only maximum deflection data, deflection gages were attached only at the center of the clear span of each girder (see Fig. 5).

Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics, Inc. (BDI, Boulder, Colorado), using WinSTS software. The intention of equipping the bridge with strain transducers was to be able to calculate the maximum compression and tension stresses achieved during the load test. Typical strain transducer locations were at midspan and near one of the abutments of select girders (Fig. 5). The transducers near the abutment were located a distance equal to the depth of the girder measured from the centerline of the timber bearing sill in the longitudinal direction. Because of the assumed symmetrical behavior, only one abutment was instrumented. Two strain transducers were placed at each location; one was placed at the bottom of the girder and the other 2 in. from the top of the girder (Fig. 6).

To provide consistency and for comparison, three load paths were considered for each load test. Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along

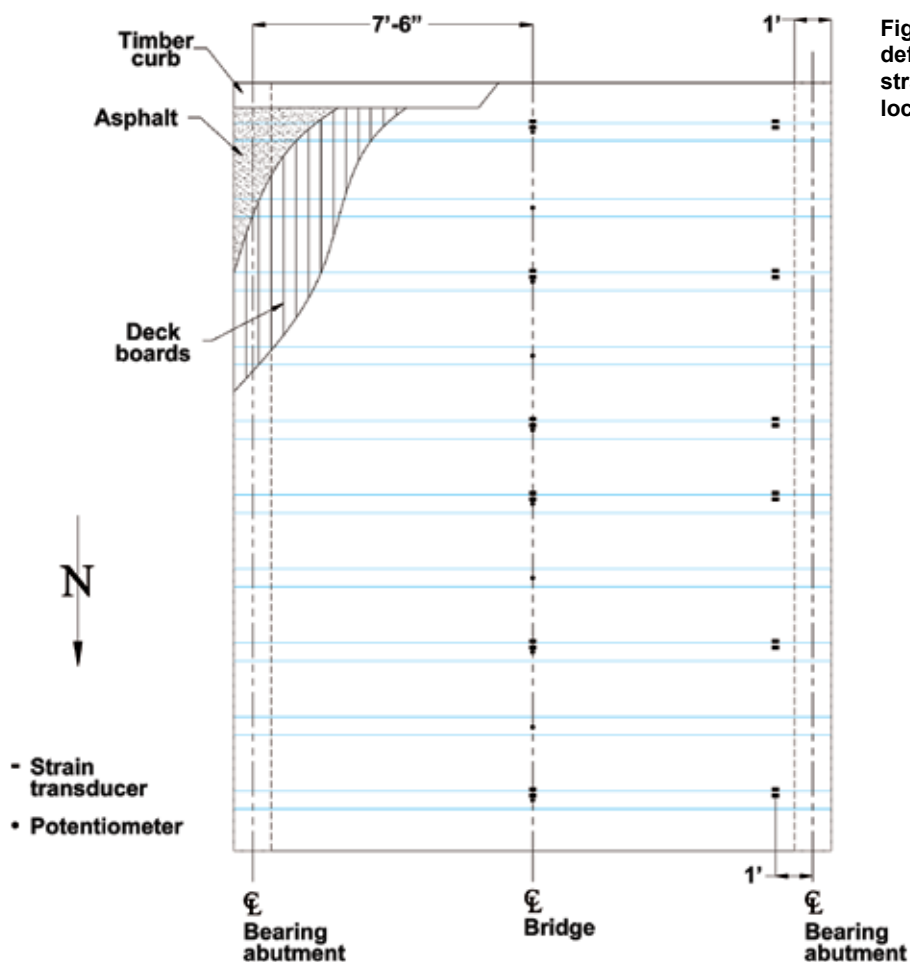


Figure 5. Typical deflection gage and strain transducer locations.



Figure 6. Typical strain transducer placement.

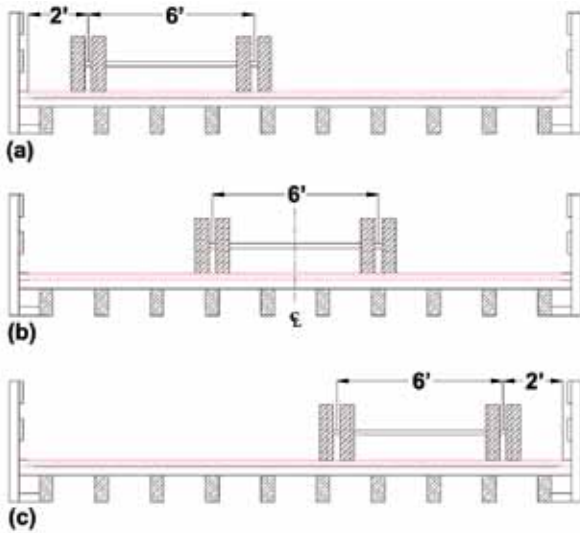


Figure 7. Typical load paths.

each side and the center of the bridge while keeping with typical traffic patterns.

For the first load path, the right wheel line of the truck was driven 2 ft from the inside of the right side curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the left wheel line of the truck was driven 2 ft from the inside of the left side curb. For each load path, the truck was driven at a crawl speed and multiple passes were made to ensure that the collected data were repeatable. Figure 7 illustrates the typical load



Figure 8. North Carolina bridge used for finite element modeling.

paths. Note that typical experimental results are given in the following sections.

3.1.4 Finite Element Modeling

To predict the performance of timber bridges, three finite element models of bridges subjected to static load testing were developed. Three models were chosen so that each location of static load testing would be represented and so that each of the models could be compared and contrasted for accuracy of analysis. The three models developed were of North Carolina bridge number 560510, Colorado bridge number P-19-AS, and Montana bridge number L25003009+09001. Detailed descriptions of the developed models can be found in the Appendixes E, M, and O, respectively.

The objective was to create a model that nearly replicated the field results obtained from static live load testing. If this could be achieved, then hypothetically the model could be subjected to various load cases and the results could be obtained without actually performing a full-scale test. One can clearly see the advantages of creating this model. Essentially, an infinite number of load tests could be performed through the use of the model, and deterioration and other conditions could be simulated.

The North Carolina bridge is a single-span, two-lane, solid-sawn timber girder bridge with a bituminous wearing surface located in Madison County, North Carolina (Fig. 8).

This bridge was subjected to static live load testing during summer 2006. From the load test, deflection results were obtained from which a finite element model could be calibrated. Figure 9 shows the finite element model developed. This model consists of solid elements with anisotropic capabilities representing the girders, deck boards, and rails.

Unlike steel or concrete, for which modulus of elasticity is well known, the timber modulus of elasticity varies considerably due to a number of factors including, but not limited to, species, grade, and moisture content. Consequently, a

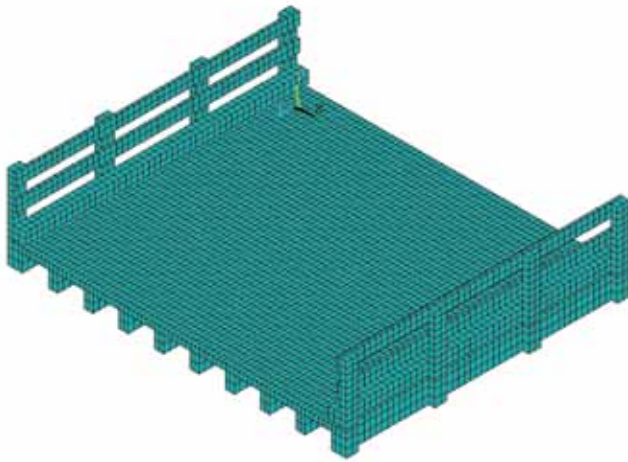


Figure 9. Finite element model of North Carolina bridge.

typical range for the modulus was determined and a value was arbitrarily selected to first model the bridge. To calibrate the model it was decided that the midspan deflections should have nearly the same shape and magnitude of the load test midspan deflection results, thereby demonstrating that the load distribution and the flexural rigidity of the model were similar to those of the actual bridge. Loads replicating those of the test vehicle were applied to the model, and midspan deflection results were obtained. If necessary, the effective modulus of elasticity was adjusted to better match the field results. Figures 10 to 12 show the calibration results when the field and finite element results were nearly equal when subjected to the same loading.

The Montana bridge is a single-span, two-lane, solid-sawn timber girder bridge with a bituminous wearing surface located near Wolf Creek, Montana (Fig. 13). This bridge was subjected to static live load testing during summer 2006. From the load test, deflection results were obtained from which a finite element model could be calibrated. Figure 14 shows the finite element model developed; much like the previous model, this model consists of solid elements with anisotropic capabilities representing the girders, deck boards, and rails.

Similar to the approach used for the North Carolina Bridge, a typical range for the modulus of elasticity was determined and a value was arbitrarily selected to first model the bridge. To calibrate the model, it was decided that the midspan deflections should have nearly the same shape and magnitude of the load test midspan deflection results, thereby demonstrating that the load distribution and the flexural rigidity of the model was similar to that of the actual bridge. Loads replicating those of the test vehicle were applied to the model and midspan deflection results were obtained. If necessary, the effective modulus of elasticity was adjusted to better match the field results. Figures 15 to 17 show the calibration results from when the field and finite element results were nearly equal when subjected to the same loading.

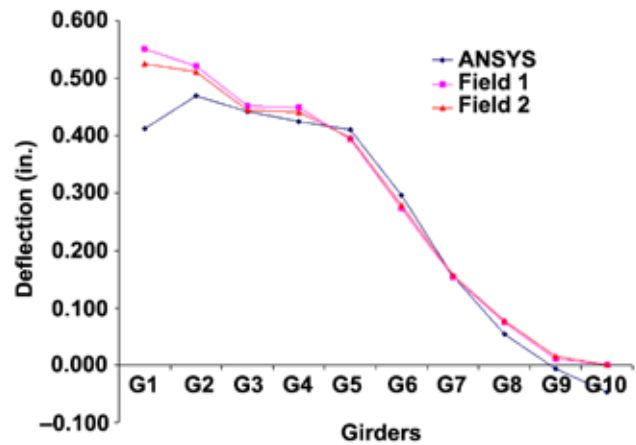


Figure 10. North Carolina finite element calibration results for path 1.

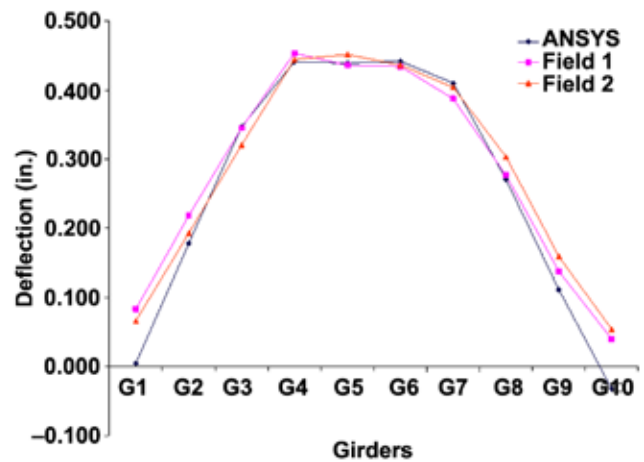


Figure 11. North Carolina finite element calibration results for path 2.

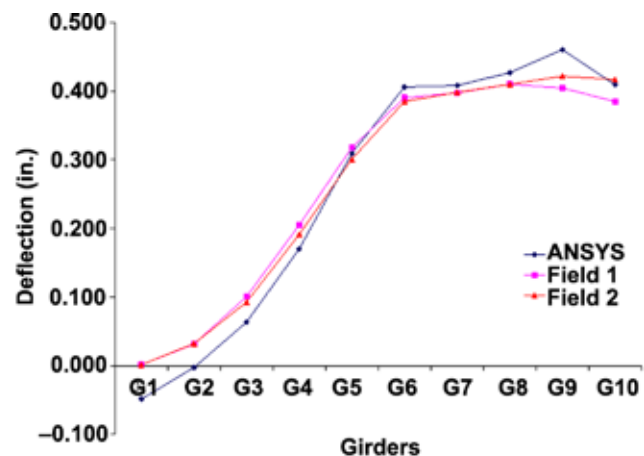


Figure 12. North Carolina finite element calibration results for path 3.



Figure 13. Montana Bridge used for finite element modeling.

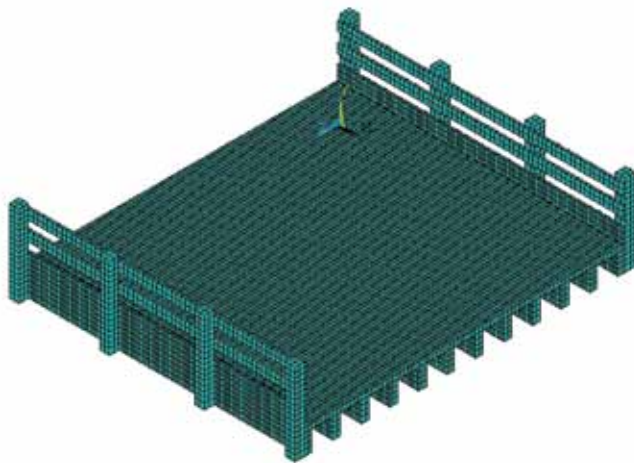


Figure 14. Finite element model of Montana bridge.

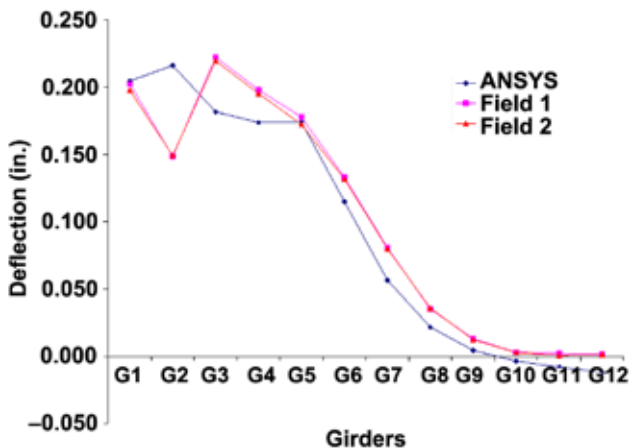


Figure 15. Montana finite element calibration results for path 1.

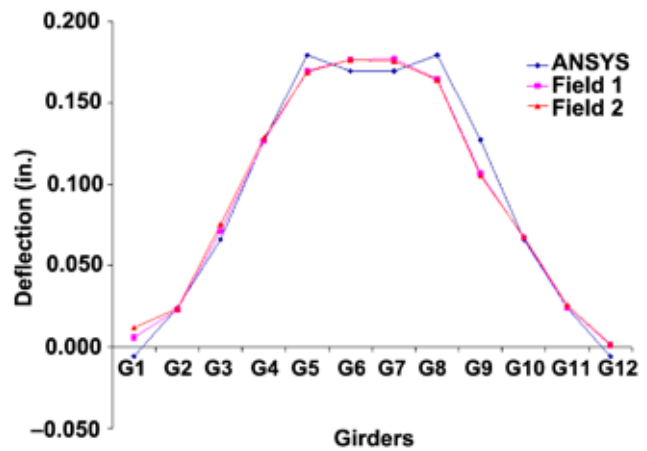


Figure 16. Montana finite element calibration results for path 2.

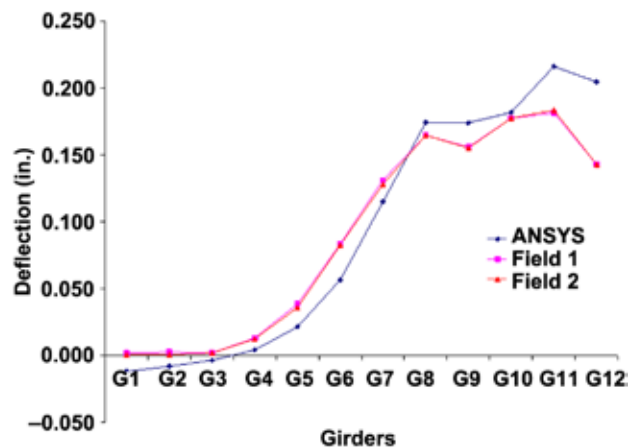


Figure 17. Montana finite element calibration results for path 3.

The Colorado Bridge is a single-span, two-lane, solid-sawn timber girder bridge with a bituminous wearing surface located near Trinidad, Colorado (Fig. 18). This bridge was subjected to static live load testing during summer 2006. From the load test, deflection results were obtained from which a finite element model could be calibrated. Figure 19 shows the finite element model developed; much like the previous two models, this model consists of solid elements with anisotropic capabilities representing the girders and deck boards. One difference between the Colorado model and the previous two models is the absence of a railing. After running identical load cases with and without the railing in the North Carolina and Montana models, it was observed that very little, if any, difference occurred in the maximum midspan deflection results. Consequently, the inclusion of a railing in the model was neglected.

Much like the approach used in the previous two models, a typical range for modulus of elasticity was determined, and



Figure 18. Colorado bridge used for finite element modeling.

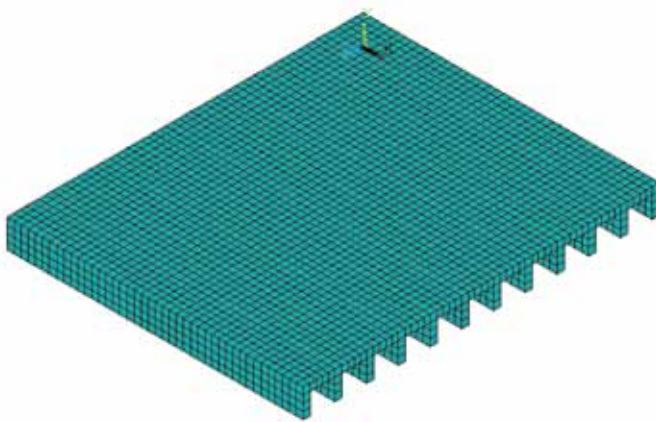


Figure 19. Finite element model of Colorado bridge.

a value was arbitrarily selected to first model the bridge. Again, to calibrate the model it was decided that the mid-span deflections should have nearly the same shape and magnitude of the load test midspan deflection results, thereby demonstrating that the load distribution and the flexural rigidity of the model was similar to that of the actual bridge. Loads replicating those of the test vehicle were applied to the model, and midspan deflection results were obtained. If necessary, the effective modulus of elasticity was adjusted to better match the field results. Figures 20 to 22 show the calibration results where the field results and finite element results were nearly equal when subjected to the same loading.

3.2 Fleet Management of Timber Bridges Concept

3.2.1 Development of Performance Parameter

A four-level parameter was developed to describe the performance of the fleet of single-span timber girder bridges with an asphalt wearing surface selected and described previously. Each level corresponds to the level of information obtained for each bridge in the fleet; the first level is based

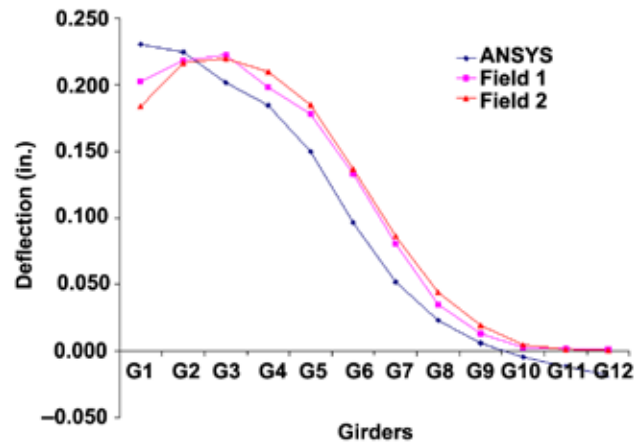


Figure 20. Colorado finite element calibration results for path 1.

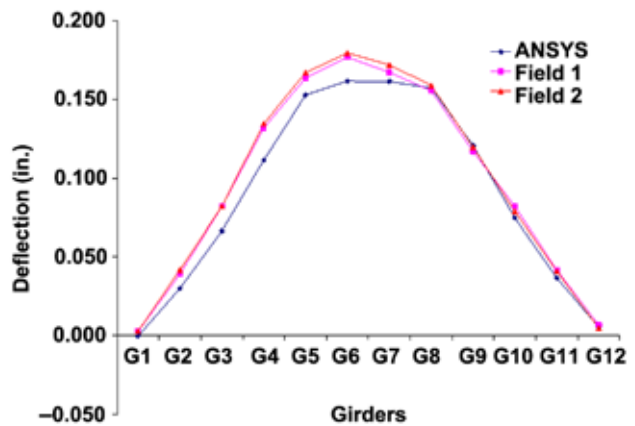


Figure 21. Colorado finite element calibration results for path 2.

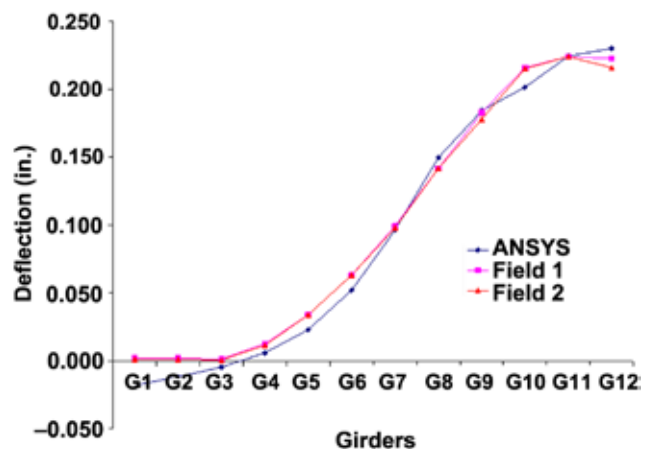


Figure 22. Colorado finite element calibration results for path 3.

Table 4—Rating by comparison of ADT^a and inventory rating

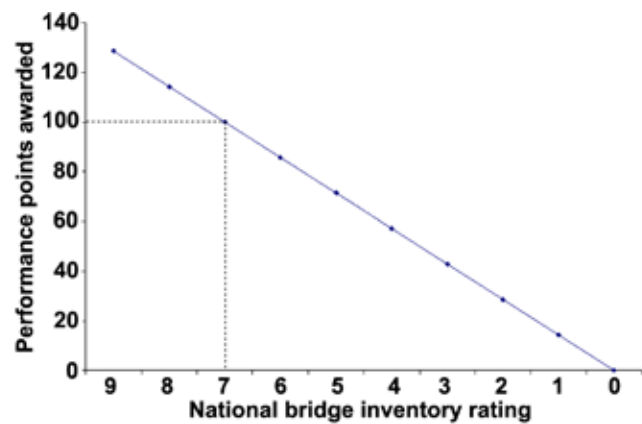
Structural evaluation rating code	Inventory rating (tons) for three ranges of ADT ^b		
	0–500	501–5,000	>5,000
9	>32.4 (MS18)	>32.4 (MS18)	>32.4 (MS18)
8	32.4 (MS18)	32.4 (MS18)	32.4 (MS18)
7	27.9 (MS15.5)	27.9 (MS15.5)	27.9 (MS15.5)
6	20.7 (MS11.5)	22.5 (MS12.5)	24.3 (MS13.5)
5	16.2 (MS9)	18.0 (MS10)	19.8 (MS11)
4	10.8 (MS6)	12.6 (MS7)	16.2 (MS9)
3	Inventory rating less than value in rating code of 4 and requiring corrective action		
2	Inventory rating less than value in rating code of 4 and requiring replacement		
0	Bridge closed due to structural condition		

^aADT, average daily traffic.^bMS load ratings.

on scores assigned and documented in the National Bridge Inventory for the deck condition rating, superstructure condition rating, inventory rating, and structural evaluation; the second level is based on information obtained from visual inspection of 23 bridges completed by the research team; the third level is based on the static load tests of 15 bridges performed by the research team; the fourth level is based on the finite element analysis of three bridges subjected to static live load testing.

3.2.1.1 Level 1—The research team’s desire was to best describe the condition and performance of the bridges in the selected fleet by the information provided in the National Bridge Inventory. Extensive investigation into the correlation of deck and superstructure rating to age, geometrical, and service descriptors showed that very little correlation existed with each descriptor alone. This was somewhat surprising. So, a performance parameter was developed that incorporated all the items that were thought to be of greatest influence on bridge performance. These items were the deck and superstructure rating obtained from visual inspection, the inventory rating, and the structural evaluation. These National Bridge Inventory items they are summarized below.

The deck and superstructure ratings describe the overall condition of the respective element. Ratings are on a scale of 0 to 9, where 9 represents excellent condition and 0 represents failed condition, out of service, or beyond corrective action. One should note that the condition of the wearing surface, curbs, and railings are not considered in the overall deck evaluation. The inventory rating is the capacity rating that will result in a load level that can safely utilize an existing structure for an indefinite period of time. The structural evaluation is the lowest of the codes obtained from the superstructure rating or the substructure rating or developed by incorporating inventory rating and average daily traffic (ADT) (Table 4).

**Figure 23. Level 1 awarded performance parameter points for deck rating, superstructure rating, and structural evaluation rating.**

For all bridges in the fleet for which the information for these four items was complete, the performance parameter could be calculated. The deck and superstructure condition ratings and the structural evaluation could have a maximum value of 9, corresponding to an excellent condition. Logically, one would expect that a condition rating of 8 or 9 would be assigned to very new bridges, yet if one were to assign a maximum value of performance points to a bridge with a rating of 9, then immediately the performance of some very good bridges would be reduced and the actual performance of bridges would not be reflected. Therefore, it was decided to assign 100 points to a bridge rating of 7 where 100 points is considered superior performance and 0 points to a bridge rating of 0. Subsequently, if a bridge is assigned ratings of 8 or 9, the potential of earning more points than the 100 points exists. Figure 23 shows the scale by which performance points were assigned.

Because the inventory rating is an indication of the maximum permissible load level that can use the bridge for an indefinite period of time, the assigned rating is not 0 through 9 as for the other items in this level. Rather, a scale was developed that gave 100 points to a rating that corresponded to the vehicle weight of the HS20 AASHTO design vehicle (32.7 metric tons). If a bridge is rated for at least this vehicle, it was thought that the bridge should be awarded at least 100 points. Therefore, potential for more than the 100 points exists if the inventory rating is greater than the HS20 design vehicle. Figure 24 shows the scale by which performance points were assigned. Subsequent to all bridges in the fleet receiving performance parameter scores for the four National Bridge Inventory items described above, each item was assigned a total percentage of the total Level 1 score. This percentage was found by completing an analysis that maximized the correlation value (r) of the performance parameter against the age of the bridge. The percentage of the total score determined by the deck condition rating, superstructure condition rating, inventory rating, and structural

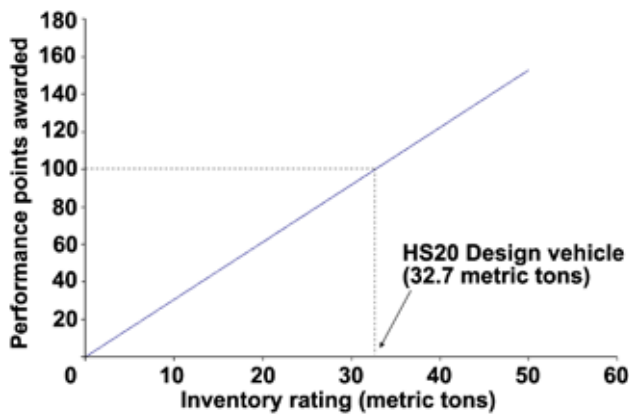


Figure 24. Level 1 performance parameter points awarded for inventory rating.

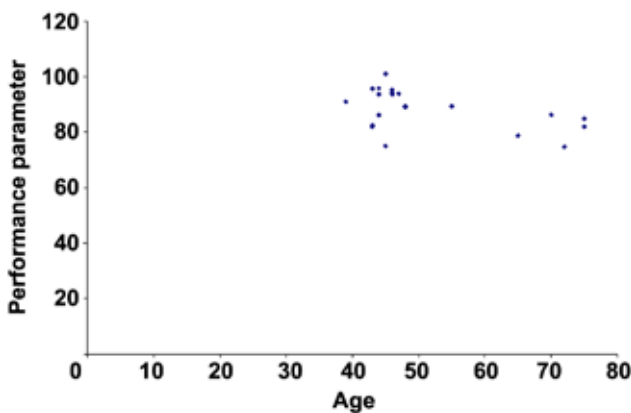


Figure 25. Total scores after Level 1 performance determination.

evaluation were 30%, 20%, 40%, and 10%, respectively. Final Level 1 scores for each bridge were evaluated by summing the factor of each item score and its respective percentage. The final scores are shown in Figure 25. Individual contributions to the performance parameters can be found in Appendix A.

3.2.1.2 Level 2—Level 2 performance parameter scores were developed using the visual inspection information gathered by the research team during 23 bridge visits in summer 2006. Three major categories compose the Level 2 scores, including (1) deck and railing, (2) superstructure, and (3) wearing surface. The total score after Level 2 scores had been figured was a combination of the Level 1 and Level 2 scores. Level 1 and Level 2 composed 40% and 60% of the total score, respectively.

Within the deck and railing category, moisture content and seven subsections were used to quantify the condition state of the category. These subsections included (1) color change, (2) knots, sloped grains, cracks, (3) holes, frass, posting, (4) ultraviolet degradation, (5) mechanical damage, (6) sagging and crushing, and (7) detachment of deck.

Following the model used to develop the California Bridge Health Index, the condition of the deck and railing was quantified. Five possible states of condition make up the entire condition state, including States 1 to 5. These states could be described as Excellent, Good, Fair, Poor, and Unacceptable, respectively. For any subsection, the portion of the whole that is described by a particular state was multiplied by the respective weighting of that state (State 1 = 100%, State 2 = 75%, State 3 = 50%, State 4 = 25%, and State 5 = 0%). For example, if 65% of the deck and railing was in excellent condition with respect to color change, 30% in good condition, and 5% in fair condition, then the total score would be the sum of the product of 65 and 1.0, 30 and 0.75, and 5 and 0.5, for a total score of 90. Note that this score makes up one of seven subsections for the deck and railing. Performing the same process for the other six subsections and then summing the scores for each gives the total earned points. Dividing the total earned points by the number of subsections gives a total score out of 100. This portion of the deck and railing make up 60% of the total deck and railing score. The other 40% is attributed to the deck moisture content measurements. Figure 26 illustrates the spreadsheet used for the deck and railing portion of the Level 2 score.

To determine the score for deck moisture content, it was necessary to develop a scale that apportioned points to reflect the moisture-affected properties of wood. After reviewing a number of sources (Bodig and Jayne 1982, Ritter 1990), it was determined that timber at 19% moisture content was still apt to achieve desirable strength performance and was thus awarded 100 points. If it is assumed that timber loses 25% of its strength by increasing the moisture content to 25%, the assumed fiber saturation point, then the scale for apportioning points is shown in Figure 27. Note that timber does not continue to lose strength beyond the fiber saturation point. The average of all deck moisture content measurements for a particular bridge was used to find the apportioned points. The final deck and railing score is the product of 0.4 and the moisture content points plus the product of 0.6 and the condition state points.

Similarly, the same process was performed on the second of the three major categories, the superstructure. The only difference lies in the subsection classifications, where misalignment and insufficient bearing were used.

The last major category, wearing surface, does not have a moisture content portion of the score. Rather, the entirety of the score is made up of the condition state portion, with only three subsections, including (1) uneven wearing surface, (2) cracks, holes, and delamination, and (3) pavement approach.

The Level 2 score is composed of the three major category scores: (1) The deck and railing and (2) the superstructure portions make up 45% of the total score, whereas the (3) wearing surface portion makes up only 10% of the total

Points	
Awarded	Possible
	100

Deck Moisture Percentage	
--------------------------	--

	State 1	State 2	State 3	State 4	State 5		
	1	0.75	0.5	0.25	0		
Color Change							100
Knots, Sloped Grains, Cracks							100
Holes, Frass, Posting							100
Ultraviolet Degradation							100
Mechanical Damage							100
Sagging and Crushing							100
Detachment of Deck							100

Total	0	700
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Deck and Railing Score	
------------------------	--

Figure 26. Deck and railing scoring example.

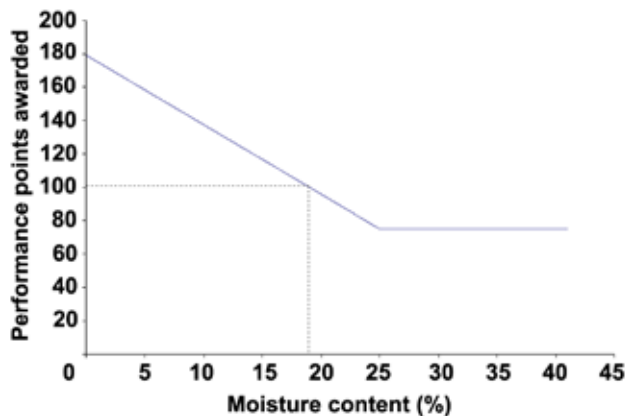


Figure 27. Level 2 moisture content performance parameter points.

score. It was thought that this weighting reflected the importance of each category to the structural integrity of the bridge. Total scores after the determination of Level 2 scores are shown in Figure 28. The individual contributions to the performance parameters can be found in Appendix A

3.2.1.3 Level 3—Level 3 scoring utilized the data acquired from load testing a bridge. Much as for Level 2, three major categories make up the final Level 3 scoring, including (1) midspan deflection, (2) differential deflection, and (3) distribution factor. Each category score composed one-third of the Level 3 score. The total score after Level 3 scores had been figured was a combination of the Level 1, Level 2, and Level 3 scores, which composed 20%, 35%, and 45% of the total score, respectively.

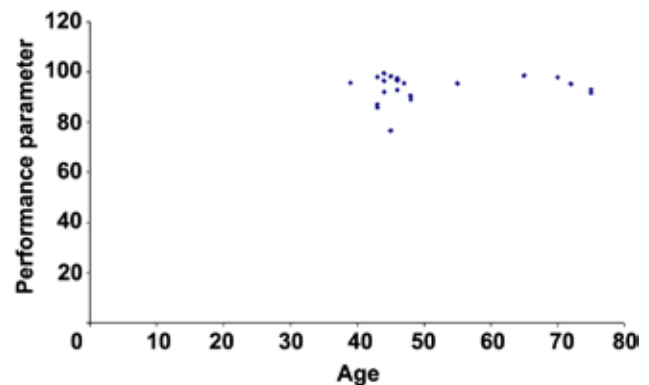


Figure 28. Total scores after Level 2 performance determination.

The recommended midspan deflection limit for timber girders is $L/360$ (Ritter 1990), where L equals the clear span distance of the girders. For each load path, the maximum normalized midspan deflection was identified and compared with the recommended deflection limit, and the awarded points reflected this comparison. If the maximum midspan deflection was equal to or greater than two times the suggested limit, 0 points were awarded. Conversely, if the maximum midspan deflection was 0, a maximum of 200 points was awarded. The scale by which points were awarded for midspan deflection is shown in Figure 29.

Little information regarding differential deflection between adjacent girders is published, so unlike the midspan deflection limits, recommended differential deflection limits do

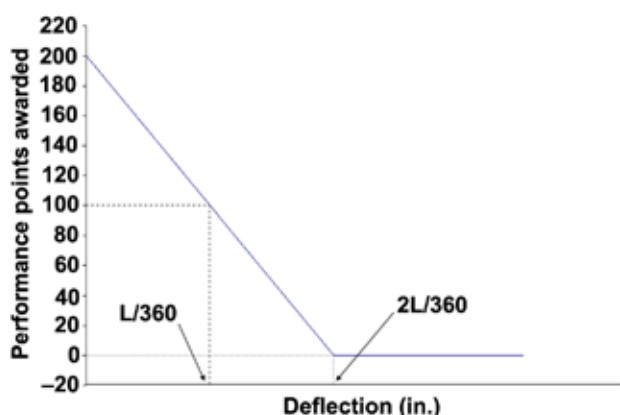


Figure 29. Level 3 performance points awarded for maximum midspan deflection.

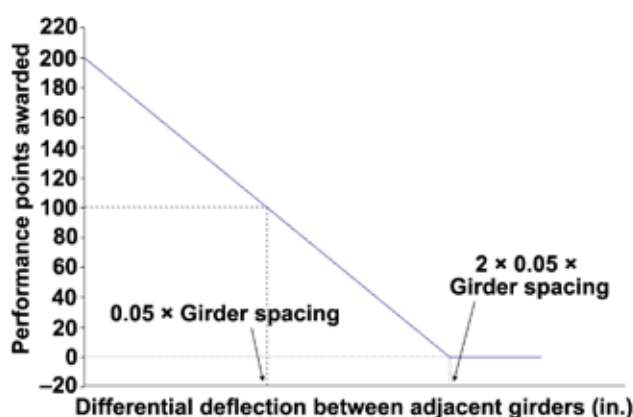


Figure 30. Level 3 performance points awarded for differential deflection.

not exist. Nevertheless, it is known that large differential deflections between adjacent girders can have adverse effects on the wearing surface of the bridge. After referencing other timber bridge studies where differential deflection was addressed, it was decided that a maximum recommended differential deflection between adjacent girders should be no more than 0.05 in. per foot of girder spacing. For example, if the girder spacing was 24 in., then 0.10 in. would be the recommended maximum differential deflection between adjacent girders. It is thought that with this recommended limit, wearing surface cracking due to differential deflection can be minimized or avoided. As a result, the points awarded to the differential deflection portion of Level 3 took into account the girder spacing and maximum normalized differential deflection for each load path. If the maximum normalized differential deflection exceeded two times the maximum recommended differential deflection, then 0 points were awarded. If the maximum differential deflection was 0, then 200 points were awarded; 100 points were awarded to a differential deflection equaling the product of 0.05 in. and the girder spacing. Figure 30 shows the scale by which differential deflection points were awarded.

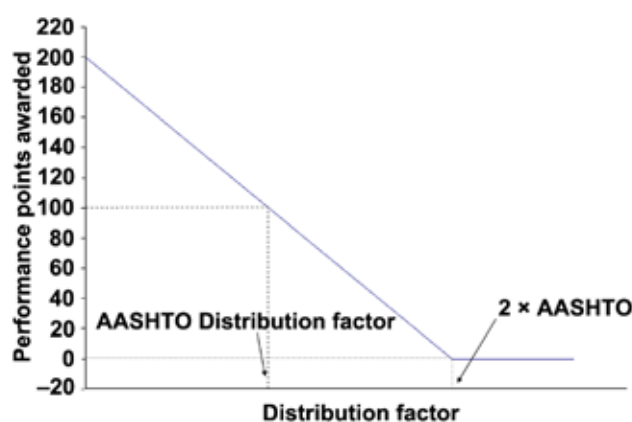


Figure 31. Level 3 performance parameter points awarded for distribution factor.

AASHTO code provisions specify a single-lane loading distribution factor of $S/6.7$; for two-lane loading, a distribution factor of $S/7.5$ is specified, where S is the spacing between girders in feet. For simplicity and because of the relatively short distance between adjacent girders, interior live load distribution factor limits were also applied to exterior girders. One should note that if the clear roadway width is less than 20 ft, the bridge is considered only a one-lane bridge and the two-lane distribution factor is not applicable, as was the case with several of the bridges in this study. That is, the distribution results obtained from the normalized vehicle were compared only with the single-lane live load distribution factors set by AASHTO. For bridges where the clear roadway width is greater than 20 ft, load paths 1 and 3 were superimposed and compared with the two-lane live load distribution factors set by AASHTO. Because of the position of load path 2, only the single-lane live load distribution factors were considered for this load path. Where the bridge clear width is greater than 20 ft, only two load cases were considered when awarding points: (1) load paths 1 and 3 and (2) load path 2. Otherwise, all three load cases were considered individually when awarding points.

Points were awarded based on the relationship between the maximum normalized distribution factor and the AASHTO code specified distribution factor. For situations where the distribution factor was equal to the AASHTO code distribution factor, 100 points were awarded. Any distribution factors that exceeded two times the code provisions were awarded 0 points, and in the case where the distribution factor was 0, 200 points were awarded. Figure 31 shows the scale by which the distribution factor points were awarded. Figure 32 shows the total score after the Level 3 scores were determined. The individual contributions to the performance parameters can be found in Appendix A.

3.2.1.4 Level 4—Level 4 performance is based on finite element modeling. Each of the models was of a bridge that was also inspected and subjected to live load testing. Results from the models were used to award points, much as for

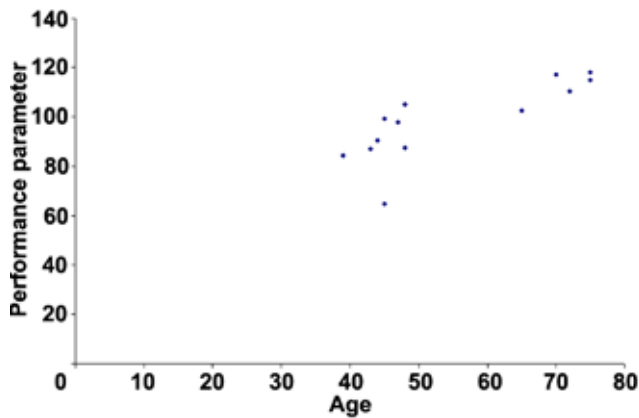


Figure 32. Total scores after Level 3 performance determination.

previous levels. Besides one added major category, several similarities exist between Levels 3 and 4—the three major categories that are present in Level 3 also exist in Level 4. To reiterate, these categories include (1) midspan deflection, (2) differential deflection, and (3) distribution factor. Allowable bending stress is the added major category. Each of the four categories will make up 25% of the final Level 4 score. The total score after Level 4 scores had been figured was a combination of the Level 1, Level 2, Level 3, and Level 4 scores, which composed 10%, 20%, 30%, and 40% of the total score, respectively.

The scales by which points were awarded for the maximum midspan deflection, differential deflection, and distribution factor portions of this level were identical to those of Level 3. The only difference between both levels was how the results were obtained. Therefore, Figures 29 to 31 are also applicable to those three major categories in Level 4.

The points awarded for allowable stress, the added major category, is as follows. According to the National Design Specification Design Values for Wood Construction (AF&PA 2001), allowable bending stress for the girder timber can range from approximately 1,150 to 1,750 lb/in² with respect to the wood grade. Because the exact grade of the girder timber is unknown, an average value of 1,450 lb/in² was selected as the maximum allowable stress. Analytical stress results at midspan were obtained and compared with this maximum allowable stress. If the maximum midspan stress was equal to the allowable stress, 100 points were awarded. If the maximum midspan stress was equal to or exceeded two times the allowable stress, then 0 points were awarded. Conversely, if the maximum midspan stress was 0, the maximum 200 points were awarded. Figure 33 shows the scale by which allowable bending stress points were awarded. Combining the allowable bending stress score with the scores from the other Level 4 major categories gives a final Level 4 score. The total scores after Level 4 are shown in Figure 34. The individual contributions to the performance parameters can be found in Appendix A.

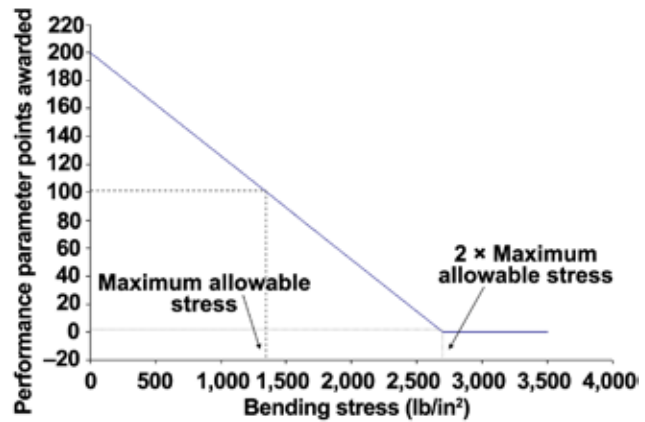


Figure 33. Level 4 performance parameter points awarded for bending stress.

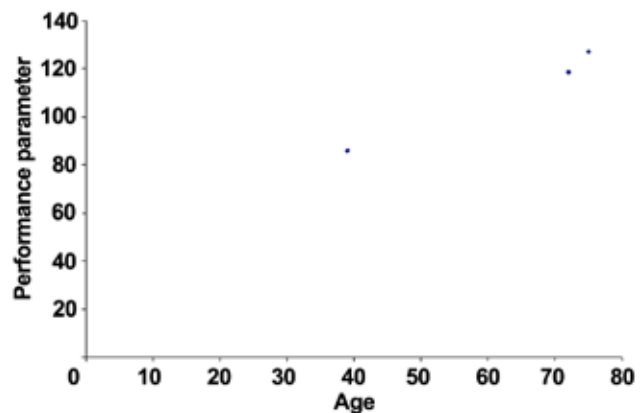


Figure 34. Total scores after Level 4 performance determination.

3.2.2 Application of Performance Parameter

The four-level performance parameter was developed with the intention of transforming the performance of any given level to Level 4, so that the state of the bridge fleet or any individual bridge may more accurately be reflected. If one could determine how bridge performance scores change between Level 4 and other levels for a smaller number of bridges, then the performance of all bridges at each Level 4 may be determined.

For all bridges within a certain level, performance scores were determined. Then the difference in scores between Level 4 and each level was determined and plotted against the age of the bridge to identify trends in score differences. Identification of trends enables one to effectively transform the score of a bridge in Level 1 to that of another level. Figures 35 to 37 show the change in performance with respect to age between Levels 1 and 4, Levels 2 and 4, and Levels 3 and 4, respectively.

The following equations describe the trend lines shown in the figures. If one applies the respective transforming

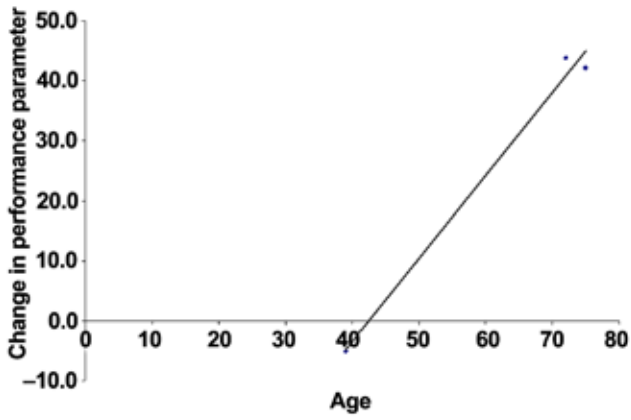


Figure 35. Change in performance between Levels 1 and 4.

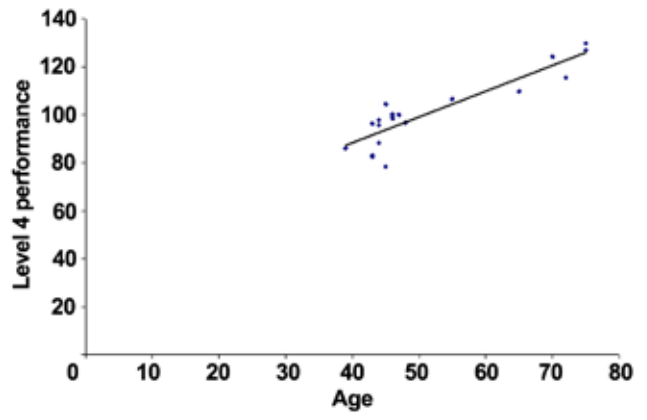


Figure 38. Level 4 performance from Level 1.

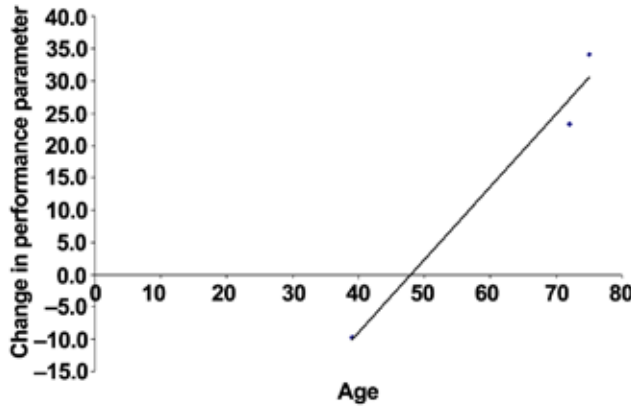


Figure 36. Change in performance between Levels 2 and 4.

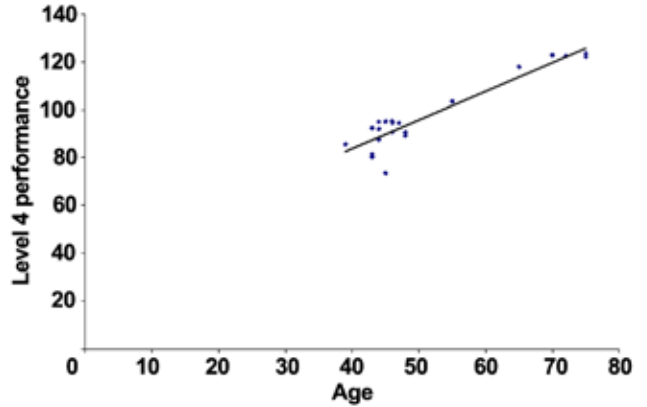


Figure 39. Level 4 performance from Level 2.

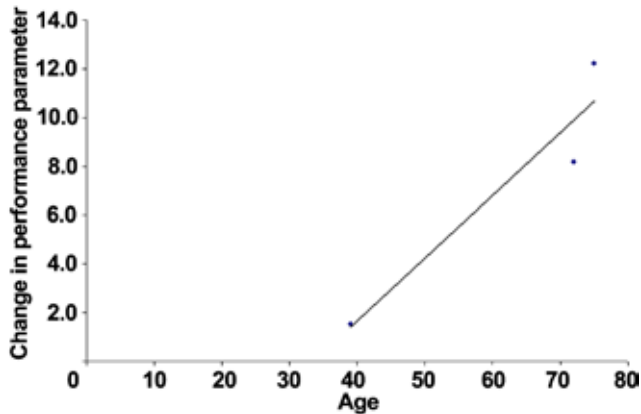


Figure 37. Change in performance between Levels 3 and 4.

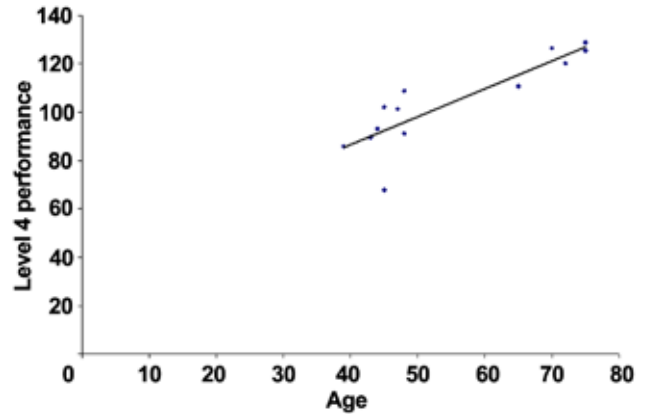


Figure 40. Level 4 performance from Level 3.

function to the Levels 1, 2, and 3 results, the plots shown in Figures 38 to 40 are obtained.

Level 4 fleet performance derived from Level 1:

$$L4 = L1 + f(L4 - L1) \quad (3)$$

where $f(L4 - L1) = 1.382 \times \text{Age} - 58.69$.

Level 4 fleet performance derived from Level 2:

$$L4 = L2 + f(L4 - L2) \quad (4)$$

where $f(L4 - L2) = 1.128 \times \text{Age} - 54.04$.

Level 4 fleet performance derived from Level 3:

$$L4 = L3 + f(L4 - L3) \quad (5)$$

where $f(L4 - L3) = 0.257 \times \text{Age} - 8.60$.

One would expect the Level 4 values for a particular bridge to be nearly equal regardless of the level from which the Level 4 scores were obtained. That is, Level 4 scores are nearly the same no matter which level the score for Level 4 score was determined. Figure 41 illustrates this idea by

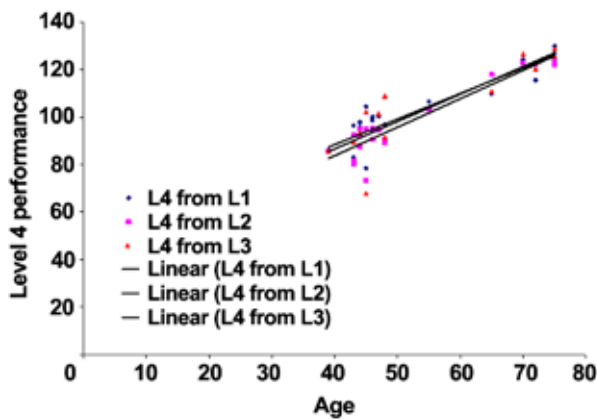


Figure 41. Level 4 performance from each level.

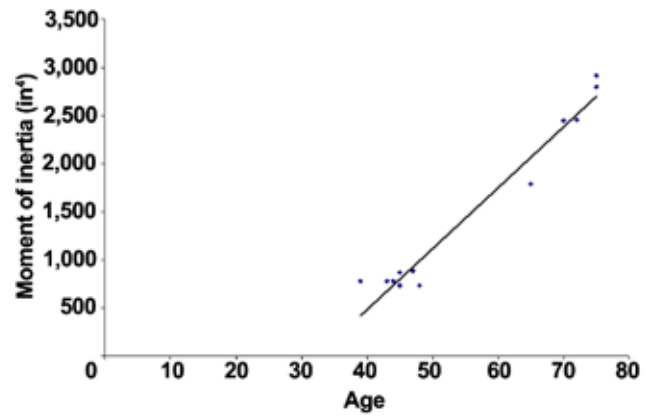


Figure 43. Relationship between moment of inertia and age.

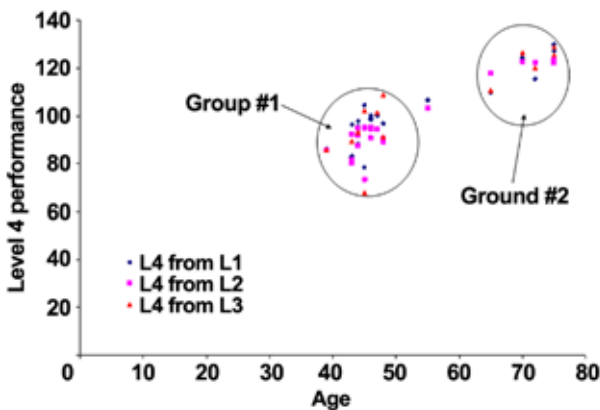


Figure 42. Data groups within Level 4 performance.

overlaying the Level 4 performance scores of Figures 38 to 40. Engineering judgment says that performance should not increase over time, yet Figures 38 to 40 show exactly that phenomenon. It is necessary to determine if confounding variables are actually affecting the bridge performance. Looking closely at Figure 41, it appears that two separate groups of data are present along the trend lines. These groups are shown below in Figure 42. A confounding variable and common factor found within each group is the moment of inertia of the girders, or more simply put, the size of the girders. When the moment of inertia of the girders was plotted against bridge age, a pattern was revealed. Coincidentally, the older bridges of group 2 had girders of a greater moment of inertia than the younger bridges of group 1 (Fig. 43), even though age and girder size are independent parameters. That is, the moment of inertia of the girders of bridges in group 1 tends to be smaller than that of group 2.

Figure 44 shows the Level 4 performance plotted against the moment of inertia of the girders for each respective bridge. It appears that overall bridge performance may be better in bridges with girders of greater moment of inertia. Another confounding variable and common factor found between groups 1 and 2 of Figure 42 was average superstructure moisture content. When average superstructure moisture

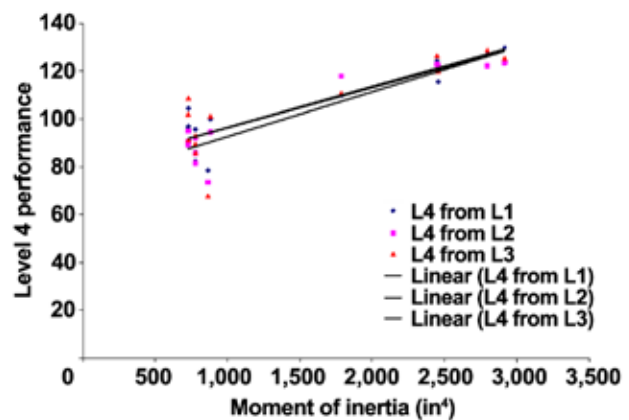


Figure 44. Level 4 performance plotted against girder moment of inertia.

content was plotted against bridge age, a trend was identified (Fig. 45). Seemingly, as age of the bridge increases, moisture content decreases, even though these parameters are independent of each other. As a result, the Level 4 performance was plotted against the average moisture content of the superstructure and another trend was identified (Fig. 46). The Level 4 performance may increase with decreasing average superstructure moisture content. One should note the strong correlation between moisture content and geographic location, as would be expected. This fact could prove useful in management practices, because it is much less difficult to determine geographic location of a bridge than average superstructure moisture content.

Girder size and moisture content appear to be two factors governing the Level 4 performance. Although a good correlation was found between Level 4 performance and age of the bridges, it did not seem correct that the performance of a bridge would increase with increasing age. However, it does seem correct to say that performance should increase with a greater moment of inertia and lower moisture, as shown in Figures 44 and 46. The aging of a bridge most certainly is still a factor among bridges with very similar girder sizes

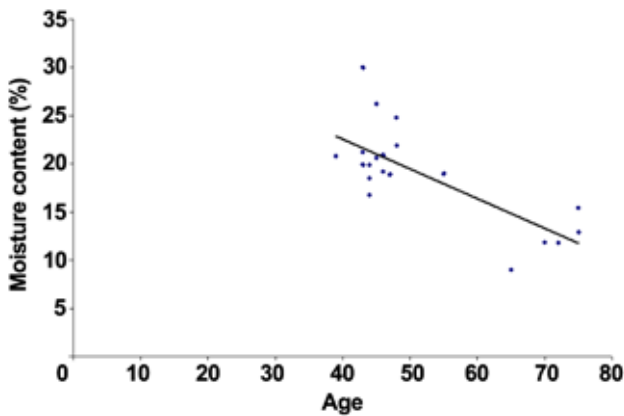


Figure 45. Average superstructure moisture content plotted against age.

and moisture contents, so age should not be discarded as a governing factor.

One could conclude from these figures that bridges with girders of lower moment of inertia will not perform as well as bridges with girders of greater moment of inertia, and bridges with greater average superstructure moisture content will not perform as well as bridges with lower moisture content. Preventive maintenance could be administered more frequently to bridges with higher superstructure moisture contents and lower girder moments of inertia to equalize the effects of lower performance. For preventive maintenance practices, one should conform to the methods outlined previously. Note that moisture contents are not as readily available as are girder dimensions for each bridge; therefore, bridge owners may need to predict average moisture content through climatology reports.

4 Observations, Conclusions, and Recommendations of Fleet Management Concept

4.1 When To Use Fleet Management Concept

When using any deterioration model, one must determine the level at which it is efficient and/or necessary to take corrective action. From the previous section, it is evident that the deterioration model will be dependent upon the fleet of bridges within a single bridge owner's jurisdiction. A fleet manager or bridge owner will need to find the most efficient balance between preventive and nonscheduled maintenance after the performance of the fleet has been determined. The performance level at which to take corrective action will be different for each jurisdiction, though the procedures for fleet management outlined within this report are recommended. The economic implications will most likely determine the performance level at which corrective action takes place.

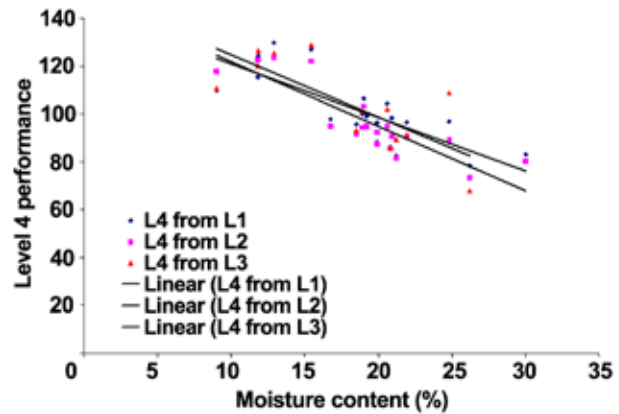


Figure 46. Level 4 performance plotted against average superstructure moisture content.

4.2 Limitations of Fleet Management Concept

A number of limitations for applying this specific fleet management concept to timber bridges exist, and one should consider these limitations before using this concept:

- The performance parameter was developed using only single-span, sawn-timber girder bridges with a bituminous wearing surface. If a bridge does not meet this description, it is recommended that the concept developed in this report not be used.
- The bridges subjected to visual inspection and static load testing all had relatively short spans. Though the applicability of this concept has not been invalidated for longer spans, one should consider the length of the bridge when using this concept.
- One should note the number of bridges within the fleet that were actually subjected to visual inspection and static load testing. Though procedures for obtaining this sample of bridges were provided, it is beneficial for bridge owners to carefully judge the applicability to their own fleet.
- A completely random sample was not used for the visual inspection and testing because budgetary and time constraints limited the sample to bridges in a few geographic locations. Therefore, though a vast amount of good information was obtained through this research, bridge owners should be aware of the sample when determining the applicability to their own fleet.
- A significant amount of variability exists within timber products. Unlike steel or concrete, for which material properties are almost exactly known, timber properties can vary even within a single bridge.
- The material properties of timber products can vary considerably with geographic location due to differences in climate. One should note the possible change in performance for bridges subjected to certain environmental conditions. This point may be negligible because most

bridge owners maintain bridges within a region that is environmentally consistent.

- Along with different regions comes different maintenance procedures and bridge management philosophies. Current maintenance practices may differ between fleets of bridges, thereby creating different initial fleet states.

If a bridge owner must manage a number of bridges that meet the recommended criteria, this concept could prove to be quite beneficial. Rather than treating each bridge on an individual basis, maintenance scheduling can be determined by fleet behavior patterns.

4.3 Conclusions

The objectives of this research were to optimize system preservation activities and enhance management approaches by taking advantage of structural similarities and better performance indicators; changing the basis of management decisions from code-evaluated individual bridges to behavior-based evaluations of a bridge fleet; determining the concepts of and information needed to adopt and implement fleet management strategies; and illustrating the viability of fleet management strategies.

To achieve these objectives, a number of tasks were completed: investigation of fleet management strategies within profit-driven industries such as trucking, busing, or airline; fleet identification; information collection, visual inspection and static live load testing on a sample of bridges within the predetermined fleet; and development and application of a performance parameter.

A fleet was selected from the nation's timber bridges and was restricted to only single-span, timber girder bridges with a bituminous wearing surface. From this fleet, 23 bridges were subjected to a thorough visual inspection and condition assessment. Of the 23, 15 were subjected to static live load testing, of which three were eventually modeled by finite element method. From these tasks, a performance parameter was developed.

In an attempt to achieve the objectives and through performing the tasks associated with this research, the following conclusions were made:

- System preservation can be enhanced by taking advantage of structural and behavioral similarities.
- Behavioral similarities exist to a point at which behavior-based evaluations of a bridge fleet could be warranted.
- The concepts of and information needed to implement fleet management strategies are well outlined in profit-driven industries and could be applied to the management of bridges.
- Though further studies should be conducted to refine the fleet management of bridges, positive evidence of the possible applicability of the concept was generated.

4.4 Recommendations

Initially this fleet management concept was intended for nationwide use; that is, a fleet could consist of bridges from anywhere in the nation as long as the "fleet" criteria are met. After investigating bridges from various states, it was found that significant differences exist between locations. These differences include, but are not limited to, bridge geometry, construction practices, condition, and maintenance practices. It is recommended that the fleet management concept may be more useful if the criteria defining a fleet were more distinct. Namely, a fleet could take into account location and maintenance jurisdiction and possibly other features not examined in this research.

For example, of all the bridges investigated and tested in western North Carolina, the methods of construction and materials used were consistent and each of the bridges was owned and maintained by the State of North Carolina. It follows that a level of uniformity with respect to condition and performance existed. Similarly, those bridges in Colorado and Montana were more alike than unlike in condition and performance.

Along with the previous recommendations, a more comprehensive list of recommendations for further research follows:

- Generally speaking, a greater emphasis should be placed on identifying a fleet from which a statistically significant sample can be taken. That is, more definite criteria should be investigated that create a more similar fleet and eliminate a number of variables that are difficult to account for.
- More specifically, one may need to limit the fleets to within a single maintenance jurisdiction, thereby attempting to eliminate variability in maintenance practices.
- More specifically, one may need to limit the fleets to a single climate region, thereby attempting to eliminate timber behavior differences due to environmental issues.
- Investigation and inclusion of preservative treatments and the effects thereof should be addressed to determine the effects of degradation prevention.
- One may consider conducting similar research that investigates fleet management of specific bridge elements (such as wearing surface, girders).
- One may consider a sample of bridges with greater age variance to identify a better performance over time curve.

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Appendix A—Summary of Performance Indicators

Level 1

- NBIS deck rating (30%)
- NBIS superstructure rating (20%)
- Inventory rating (40%)
- Structural evaluation (10%)

Level 2

- Level 1 score (40%)
- Deck rating index (27%)
 - Color change (2.3%)
 - Knots, sloped grains, cracks (2.3%)
 - Holes, frass, posting (2.3%)
 - UV degradation (2.3%)
 - Mechanical damage (2.3%)
 - Sagging and crushing (2.3%)
 - Detachment of deck (2.3%)
 - Moisture content (11%)
- Superstructure rating index (27%)
 - Misalignment (1.8%)
 - Insufficient bearing (1.8%)
 - Color change (1.8%)
 - Knots, sloped grains, cracks (1.8%)
 - Holes, frass, posting (1.8%)
 - UV degradation (1.8%)
 - Mechanical damage (1.8%)
 - Sagging and crushing (1.8%)
 - Detachment of deck (1.8%)
 - Moisture content (11%)
- Wearing surface index (6%)
 - Uneven wearing surface (2%)
 - Cracks, holes, and delamination (2%)
 - Pavement approach (2%)

Level 3

- Level 1 score (20%)
- Level 2 score (35%)
- Measured midspan deflection (15%)
- Measured differential deflection (15%)
- Measured distribution factor (15%)

Level 4

- Level 1 score (10%)
- Level 2 score (20%)

- Level 3 score (30%)
- Analytical midspan deflection (10%)
- Analytical differential deflection (10%)
- Analytical distribution factor (10%)
- Analytical bending stress (10%)

Appendix B—Individual Bridge Performance Reports

A drastic change in bridge construction practices occurred during the past century. Advancements of steel and concrete as construction materials nearly eliminated the use of timber in bridge projects. Before that, timber was the most frequently used material for bridge building.

As traffic loads increased, the use of high-strength materials such as steel and concrete became necessary. As a result, a vast amount of research and development revolved around steel and concrete. It follows that most university coursework emphasized the use of these materials. Even more, heavy competition between the steel and concrete industries maintained low prices. Clearly, advancements in bridge construction were being made, yet timber was neglected as a bridge building material and timber research and innovation were relatively idle due to the lack of interest and capital base, thus impeding the use of timber in bridge projects.

Using timber as a primary bridge construction material offers a number of benefits. Among these benefits are timber's strength, light weight, and energy-absorption capabilities. Minimal sensitivity to weather conditions and de-icing agents are also desirable properties, and constructability is often better than that of materials such as steel and concrete. Timber bridge construction costs are competitive with other materials and offer a number of economic benefits over the lifetime of the bridge.

Though a number of great qualities exist in timber bridge construction, timber bridge inspection and maintenance is an unresolved issue. Inspections are typically conducted visually, which often does not thoroughly detect deterioration in timber members. Inspection and maintenance practices are still in the early stages of development, and more efficient practices are desired. With future advancements in timber bridge construction, inspection practices and maintenance inefficiencies could be reformed and minimized.

An attempt to restore the use of timber in highway bridge construction was made when the U.S. Congress passed legislation known as the Timber Bridge Initiative in 1988. The USDA Forest Service was assigned the task of administering the timber bridge program. Part of the USDA Forest Service, the Forest Products Laboratory was assigned the research portion of the Timber Bridge Initiative. In 1992, as part of the Intermodal Surface Transportation Efficiency Act, the Forest Products Laboratory joined with the Federal Highway Administration (FHWA) Turner–Fairbanks Highway Research Center to implement the FHWA timber bridge research program. As part of this program, university researchers have been employed to conduct research advancing timber bridge construction.

A research study intended to develop maintenance schedules for similar timber bridges was conducted at Iowa State

University. During the summer of 2006, the study afforded the opportunity to perform static load tests on a number of timber bridges throughout the United States, thereby increasing the knowledge of timber bridge performance and deterioration modes. The following 15 reports summarize the results of bridge tests intended to gather and analyze information on timber bridge performance under load.

Objective and Scope

Objectives of this research were to develop and demonstrate fleet management strategies for timber bridges of similar geometry, material, and performance behavior. The project scope includes a preliminary investigation of timber bridges of a certain fleet (that is, single-span, timber girder bridges with a bituminous wearing surface), data collection and analysis under static loading, and computer modeling of loaded bridges. Results of the project will be used to develop and prove the viability of a maintenance schedule for bridges of a certain fleet.

Evaluation Methodology

Bridge evaluation consisted of investigating the bridge condition through visual inspection, moisture content measurement, and deflection and strain data collection under static load.

Moisture measurements were taken using a two-prong electric resistance moisture meter. Measurements were taken at several locations on the underside of the deck and the girders. Deflection data were collected through the use of ratio-metric potentiometers manufactured by Celesco Transducer Products, Inc. (Chatsworth, California). The signals from these instruments were collected using an Optim Megadac 3415AC (Armonk, New York) data acquisition system running TCS (New York, New York) windows software. Strain data were collected using the Structural Testing System manufactured by Bridge Diagnostics Inc. (BDI) (Boulder, Colorado) using WinSTS (Neuchatel, Switzerland) software.

Moisture Content

The moisture content of timber can significantly alter the bridge performance under load. An increase or decrease in moisture content can result in fluctuations in the modulus of elasticity and cause shrinkage and swelling, and provides a catalyst for rotting and other deterioration. Therefore, moisture content measurements were taken at several locations throughout the girder and deck elements.

Condition Assessment

A condition assessment was conducted as part of the bridge investigation by the Iowa State University (ISU) research team. In particular, the wearing surface, deck, and superstructure were thoroughly assessed. In addition, the substructure was viewed, though due to concealing conditions much of the substructure was not visible.

As part of the visual inspection, the bridge wood components were checked for discoloration, vegetation, splits, cracks, checks, absorption of water, odor, sagging, crushing, holes, frass, powder posting, knots, mechanical damage, ultraviolet degradation, lightening or darkening, water staining, and sunken faces.

The wearing surface was viewed for cracking, delamination, holes, debris accumulation, and transitional problems between the deck and approaches.

The superstructure was inspected for abrasion and deterioration between the deck and girders, drainage of surface materials through the floor system, sufficient bearing area for the girders on the sill, misalignment in the girders, looseness of fasteners, and any other abnormal superstructure behavior.

Deflection Criteria

Several sources recommend a live load deflection limit state for timber bridges (see Table B1). These recommendations are primarily derived from the effects of deflection on the wearing surface of the bridge and are given in the form L/n , where L is the clear span length of the girder in inches. If the deflection exceeds the length divided by the n value, a stronger likelihood of cracking and deterioration of the wearing surface exists.

Moreover, the n value can be calculated given the deflection under live load and the length of the bridge. To more easily compare n values between bridges, the deflection was normalized by the ratio of actual truck weight to the weight specified for the AASHTO standard HS-20 tandem axle loading, which is most like the trucks used in this study. The equation for the n value is

$$n = \frac{\text{Length}}{\text{Deflection} \times \frac{\text{HS-20 Load}}{\text{Actual Load}}} \quad (\text{B1})$$

where deflection and length are in inches.

Distribution Factors

As the load traverses the bridge, the load is distributed transversely to the girders by the deck system. Assuming that each of the girders is of equal stiffness, the deflection achieved at the midspan of all the girders should be proportional to the percentage of load distributed to that girder. Subsequently, the load fractions were computed using

$$E_i = \frac{\Delta_i}{\sum_{i=1}^n \Delta_i} \quad (\text{B2})$$

where LF_i is load fraction of the i th girder, Δ_i the deflection of the i th girder, $\sum \Delta_i$ the sum of all girder deflections, and n the number of girders.

Table B1—Live load deflection limit states

Source	n value
Timber bridges ^a	$L/360$
Highway bridges ^b	$L/425$
AASHTO ^c	$L/500$

^aRitter (1990).

^bBarker and Puckett (2007).

^cAASHTO (2006).

Differential Deflections

It was shown that the overall deflections should not exceed a recommended value with respect to the length of the bridge primarily due to possible degrading effects on the wearing surface. Another deflection criterion worth consideration is the differential deflection between adjacent girders. Though design considerations regarding differential deflections have not been published, a significant amount of differential deflection can also have adverse effects on the wearing surface. After investigating other timber bridge studies where differential deflection was addressed, the authors of this report thought that a maximum recommended differential deflection between adjacent girders should be no more than 0.05 inches per foot of girder spacing to inhibit wearing surface cracking.

Bridge Performance Reports

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Appendix C—North Carolina Bridge No. 860131, Chestnut Cove Creek Bridge

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Abstract

The Chestnut Cove Creek Bridge is a single-span, timber girder bridge with a bituminous wearing surface located in Swain County, North Carolina. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented here.

Acknowledgments

We express our appreciation to those who were of assistance to this project and those without whom we would not have completed this research project: Henry Black, North Carolina Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing; Chris Lee, North Carolina Department of Transportation employee, who organized the load testing; and Donny Warren, North Carolina Department of Transportation employee, who operated the load truck during testing.

Background

The location of North Carolina state bridge number 860131, hereinafter referred to as the Chestnut Cove Creek Bridge, is shown in Figure C1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Chestnut Cove Creek Bridge was built in 1962 and is located in Swain County in western North Carolina, 0.1 miles south of junction SR1177 across Chestnut Cove



Figure C1. Location of Chestnut Cove Creek Bridge in North Carolina.



Figure C2. Chestnut Cove Creek parapet support.

Creek. SR1122 is carried by the structure. Currently, the bridge is posted for 15 tons (single vehicle) and 21 tons (type S3 truck).

Bridge Description

The Chestnut Cove Creek Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface. The bridge length measures 16 ft–3 in. from the west face of the west backwall to the east face of the east backwall. The bridge width measures 19 ft–0 in. from inside of curb to inside of curb and 20 ft–9 in. from outside of rail to outside of rail. The substructure consists of solid timber posts and sills seated on concrete.

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by timber blocks and bolts into the exterior girders along with bolts into the curb which is seated and bolted to the top of the deck, as shown in Figures C2 and C3.



Figure C3. Chestnut Cove Creek parapet.

Girders measure 16 ft–0 in. from end to end and have a clear span of 14 ft–0 in. A total of 10 girders, spaced 2 ft center-to-center, measuring 5-3/4 by 11-3/4 in. in cross-section, are present and are seated and toe-nailed to the 12- by 12-in. timber sills with spikes. The deck consists of individual 4- by 8-in. nominal boards laid transverse to the longitudinal girder direction, which are fastened to the girders with spikes. Overlaying the deck is a 3-in.-thick layer of asphalt wearing surface. Figure C4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure C5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 10 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure C6.

Strain transducers were attached to girder numbers 1, 3, 5, 6, 8, and 10, with 1 being the outside girder on the south side of the bridge and 10 being the outside girder on the north side of the bridge. The midspan and one abutment were instrumented (see Fig. C5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder (see Fig. C7). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the North Carolina Department

of Transportation–Division 14. Dimensions of the truck are shown in Figure C8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–6 in.; the distance between the forward-most rear axle and the front axle hubs measured 15 ft–3 in. Exact weight of the truck was unknown because a scale was not accessible during testing. The driver approximated the total weight of the truck and load to be 45,000 lb. Typically, 70% of the weight on a loaded tandem-axle truck is distributed to the rear axles. Using this assumption, the total weight on each rear axle and the front axle may be 15,750 lb and 13,500 lb, respectively. A truck similar to the one shown in Figure C9 was used for the load test.

Three load paths were considered when testing the bridge (see Figs. C10 to C12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the right wheel line of the truck was driven 2 ft from the inside of the north curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the left wheel line of the truck was driven 2 ft from the inside of the south curb. For all load paths, the dump truck was driven at a crawl speed from east to west and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on October 15, 2005, was obtained from the North Carolina DOT (NC–DOT). This report was reviewed and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed.

Wearing Surface

According to the NC–DOT 2005 report, scattered transverse cracks were present at the floor board seams. Though determined to be relatively minor, these cracks were verified by the ISU team during testing in 2006. The asphalt pavement generally looked to be in good condition; however, at the transition between the gravel approach and the bridge wearing surface at the west end of the bridge, the asphalt was chipping away and revealed the timber decking beneath (see Fig. C13). Besides this local case, the transition between the roadway and asphalt does not appear to be problematic for the bridge. An uneven transition could subject the bridge to unnecessary effects from dynamic loads even though slow

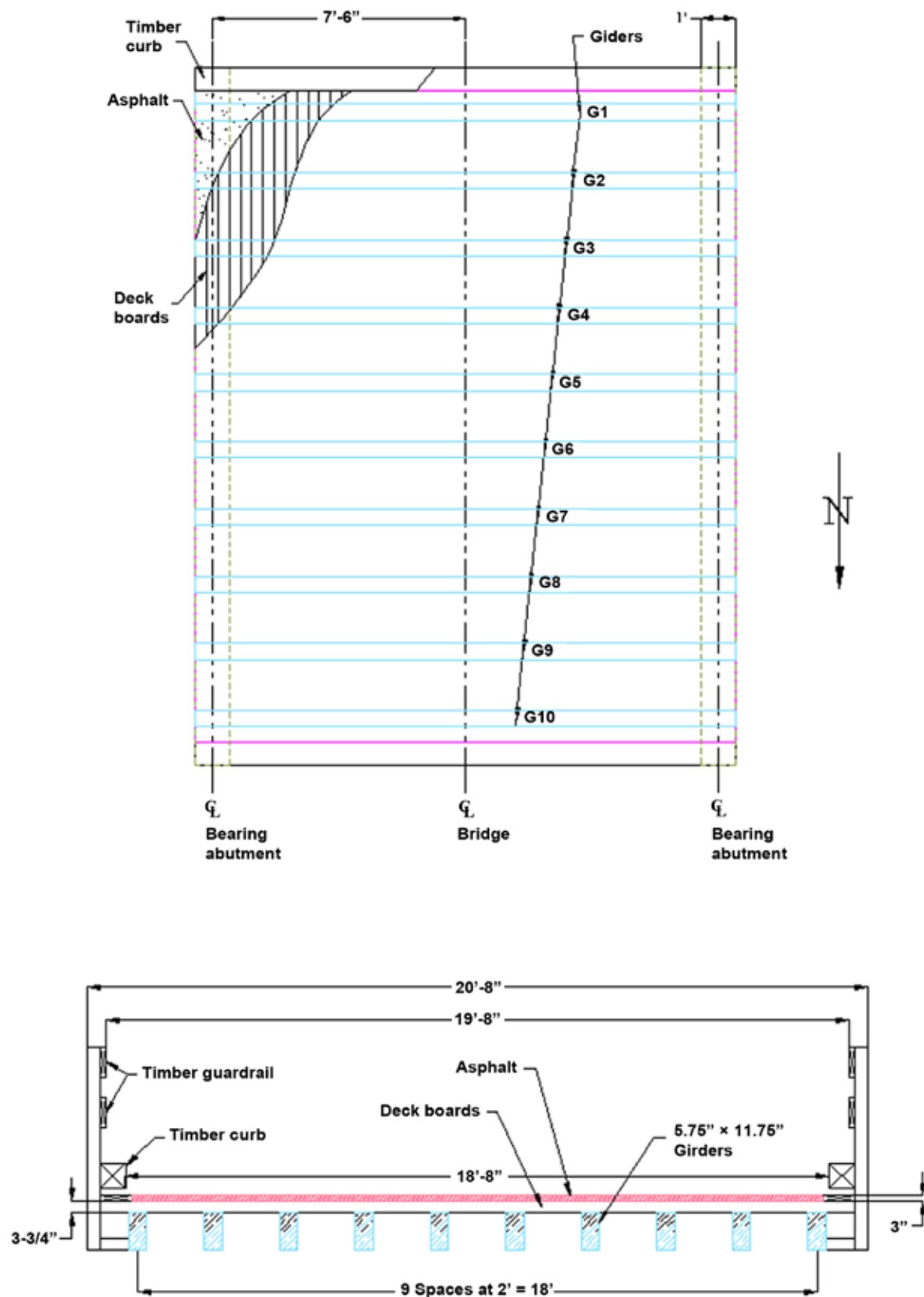


Figure C4. Plan and profile layout of Chestnut Cove Creek Bridge.

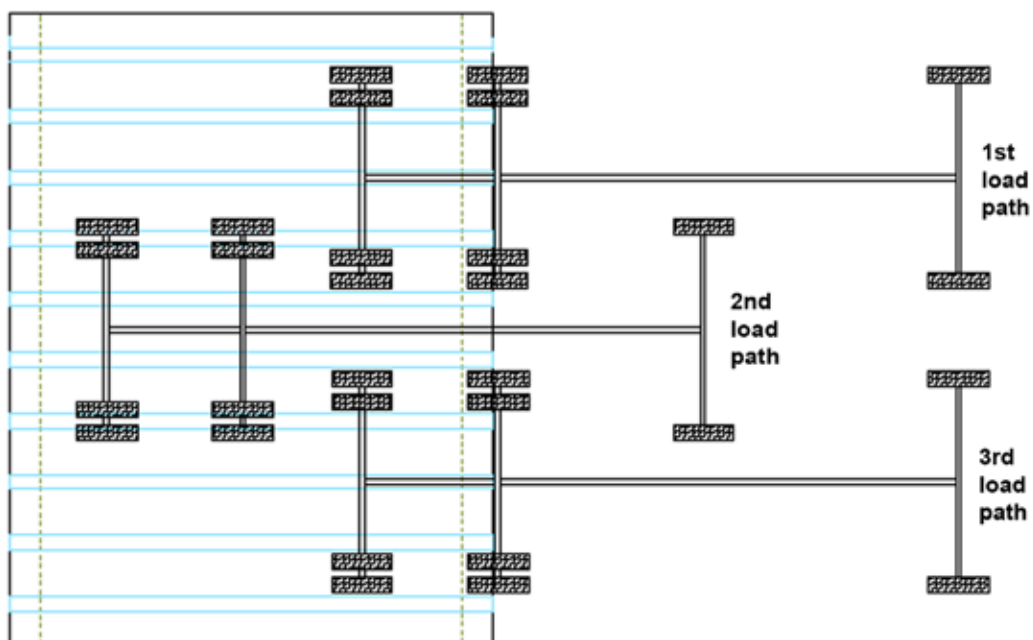
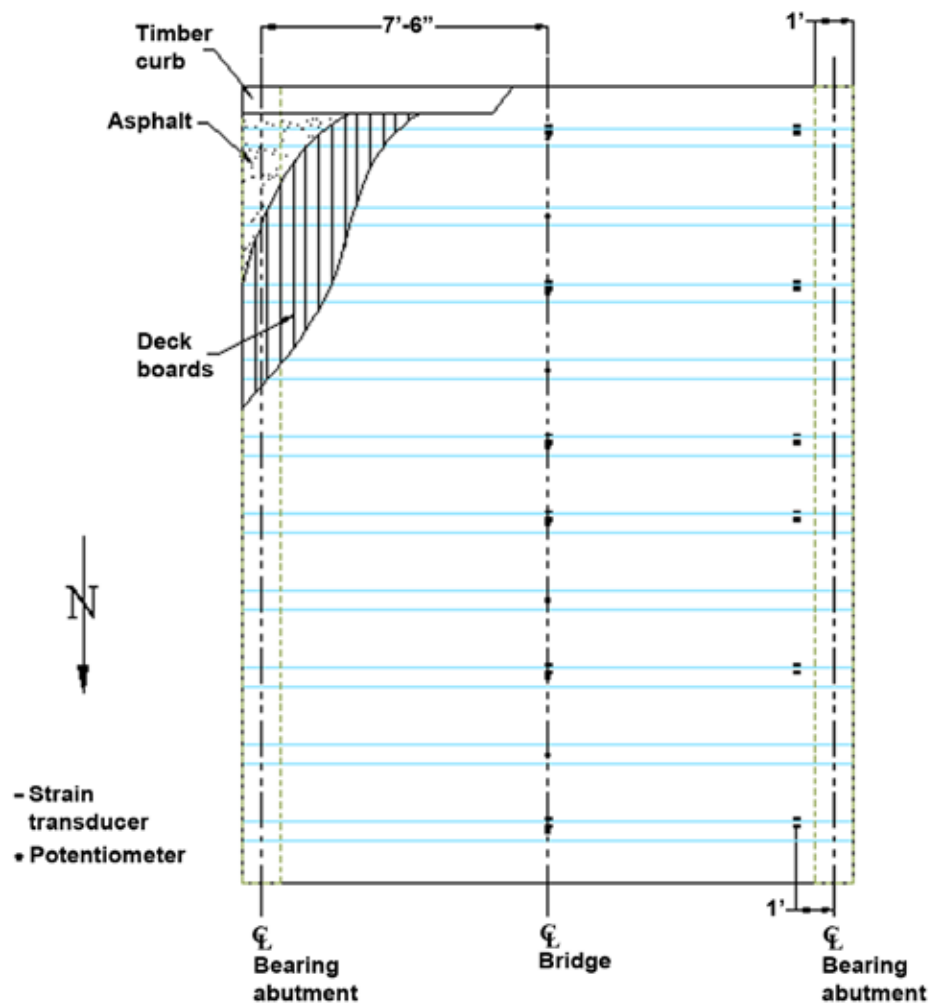


Figure C5. Instrumentation and load paths of Chestnut Cove Creek Bridge.



Figure C6. Deflection instrumentation.



Figure C7. Strain transducers.

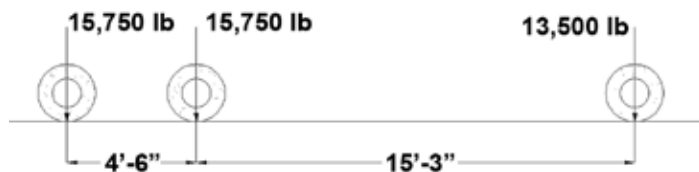


Figure C8. Truck configuration and axle loads.

vehicle speeds on this roadway make this unlikely. In addition, a significant amount of debris had collected on top of the bridge hindering drainage and promoting seepage through the wearing surface to the decking and superstructure.

Deck

The deck appeared to be in good condition, there was no visible detachment of the deck boards from the girders, and all deck boards were securely fastened. Minor water staining from seepage through the wearing surface was present throughout, although there were no signs of imminent decay. Some moss growth was present at the ends of the deck boards.

Superstructure

The interface between the deck and the girders was wetter than other areas of the girders. Seepage through the wearing surface was soaking into at least the girder surface at this location, and most of the girders showed signs of water seepage and staining throughout. White residue had also formed on the faces of many of the girders and is assumed to be biotic growth from high moisture conditions. The residue did not appear to permeate the girder, however. Figure C14 shows a typical example of white residue.

The girder bearing on the sill was sufficient, and there was no misalignment. The only noticeable degradation was a check at the bottom of girder number 10.

Overall Structure

The overall structure was in satisfactory condition, and structurally the bridge was sound. No odor like anise or wintergreen, signifying fungal growth, was present. There was no evidence of insect, mechanical, or ultraviolet degradation. Minor issues of concern in addition those already stated include the presence of filtering at the abutments where various locations on the sill and backwalls were very wet. Moss growth was prevalent at the water line at the bottom of the northeast substructure posts (see Fig. C15). There were minor checks in the parapet and parapet curb.

Results

The following presents the results of the static load testing of the Chestnut Cove Creek Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.



Figure C9. Tandem-axle load truck.

Time-History Deflections

Figures C16 to C18 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck, one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges, where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures C16 to C18, and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table C1. Each passing of the three load paths is illustrated in Figures C19 to C21. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table C2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths is 177. This value is less than the minimum recommended value for timber girders. In fact, all the n values are below the recommended n values stated in Table C2. The possible reasons for deflections greater than those recommended will be discussed later.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure C22 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure C23 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures C24 to C26 show differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table C3.

The maximum differential deflection of 0.125 in. occurs in load path 2. This is nearly 38% of the maximum deflection resulting from that load path and 0.063 in. per ft of girder spacing. Among other potential reasons for large differential deflections, the possibility exists that the load is not well distributed transversely between these two girders or the

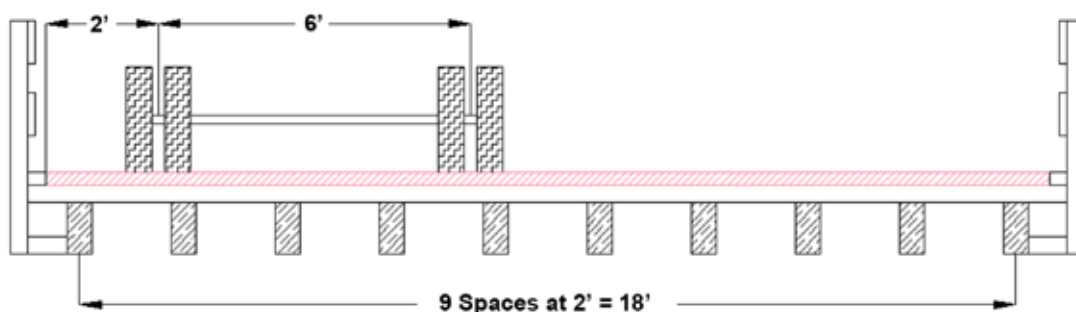


Figure C10. Transverse truck position—load path 1.

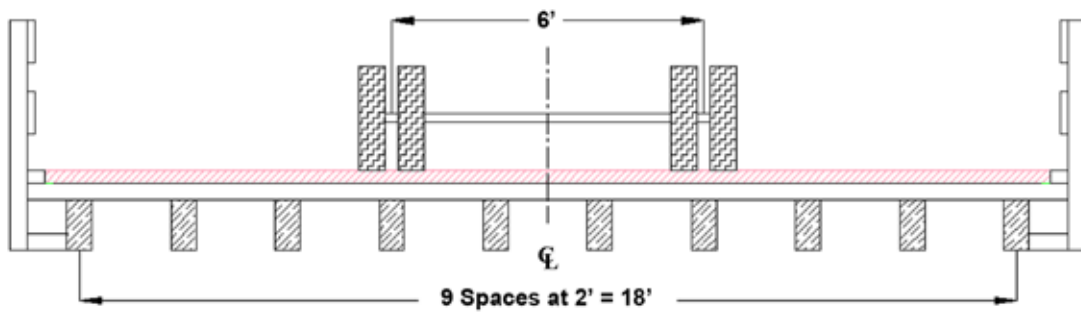


Figure C11. Transverse truck position—load path 2.

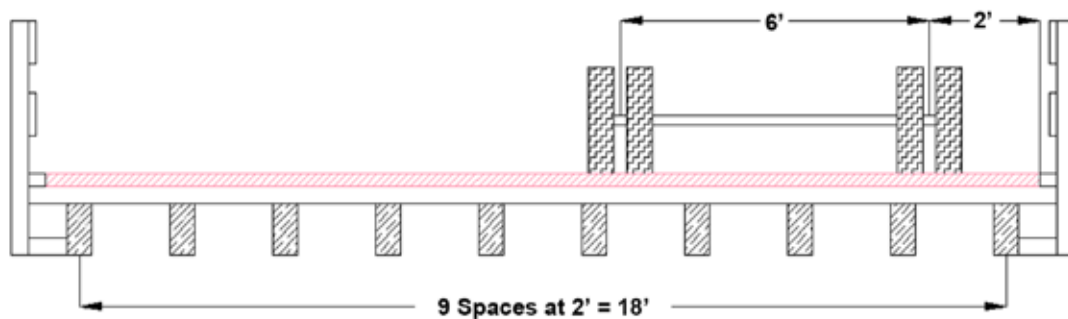


Figure C12. Transverse truck position—load path 3.



Figure C13. Pavement chipping at west end.



Figure C15. Moss growth at base of substructure posts.



Figure C14. Biotic growth on girders.

assumption that both girders are of equal stiffness is false. The same is true for load paths 1 and 3 as the maximum differential deflections are both around 0.1 in.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan, with compression and tensile strains of and 403 and 437 microstrain, respectively. The strain plot at midspan is shown in Figures C27 to C29 for load paths 1, 2, and 3,

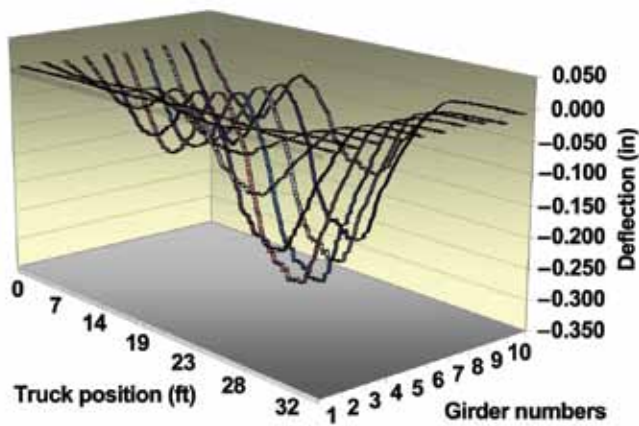


Figure C16. Deflections vs. truck position for load path 1.

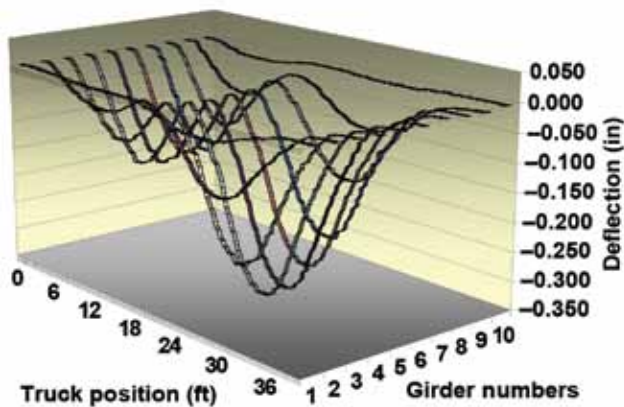


Figure C17. Deflections vs. truck position for load path 2.

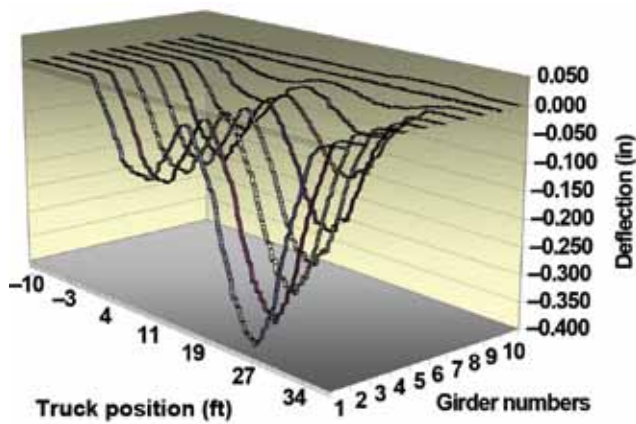


Figure C18. Deflections vs. truck position for load path 3.

respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus

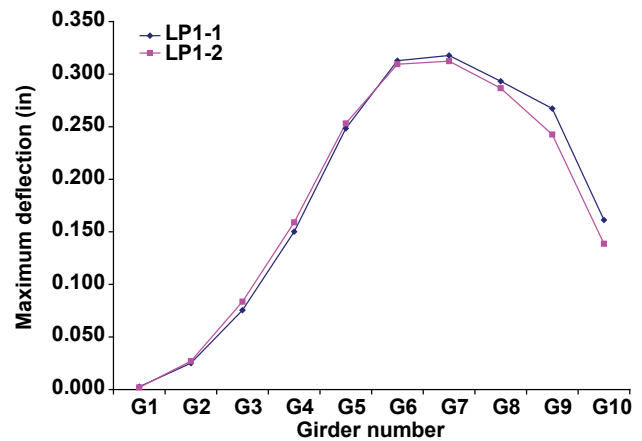


Figure C19. Maximum deflections for load path 1.

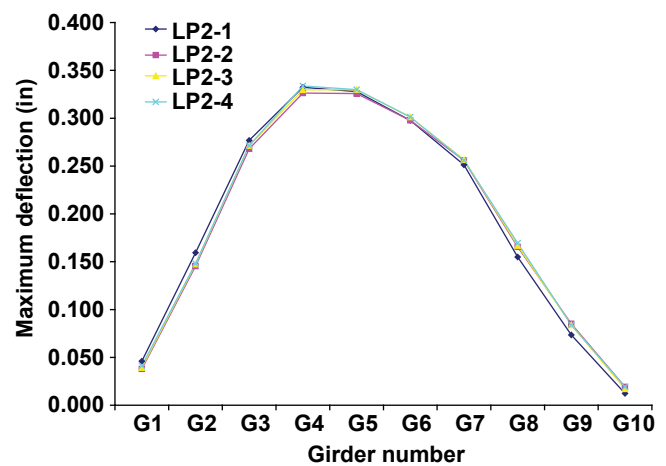


Figure C20. Maximum deflections for load path 2.

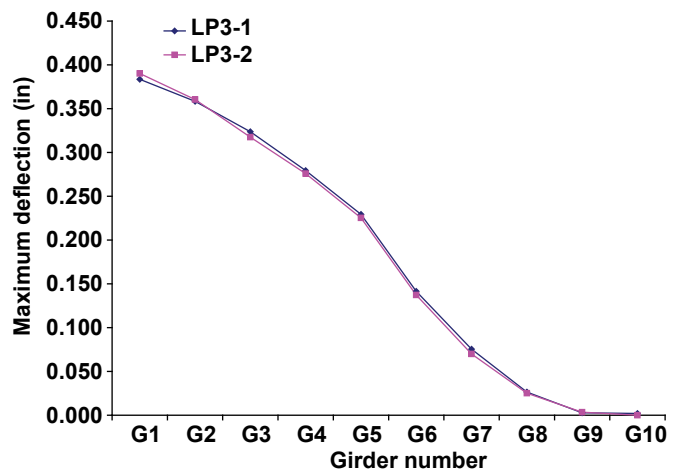


Figure C21. Maximum deflections for load path 3.

of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

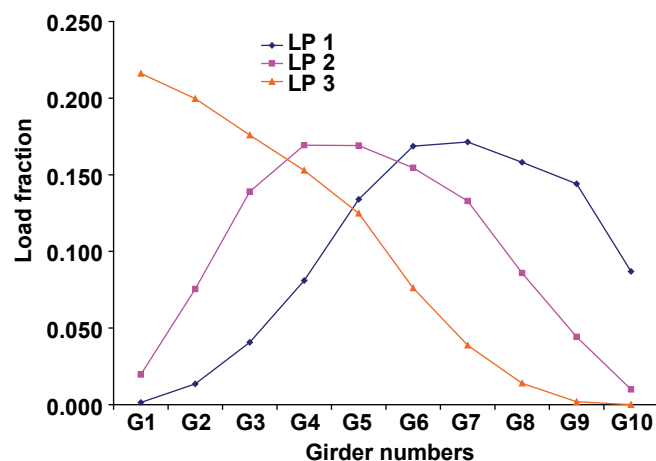
Figures C27 to C29 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data

Table C1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.318
Load path 2	0.334
Load path 3	0.390

Table C2—Most critical *n* value

	<i>n</i> value for girder of most deflection for each load path
Load path 1	185
Load path 2	177
Load path 3	178

**Figure C22. Load fractions for each load path.**

signifies that there is very little if any composite action with the deck; that is, the girders act independently of the deck when subjected to bending.

Moisture Content

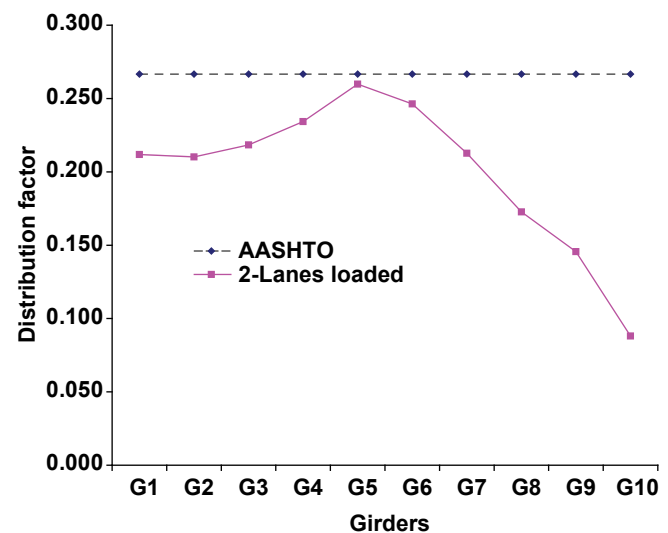
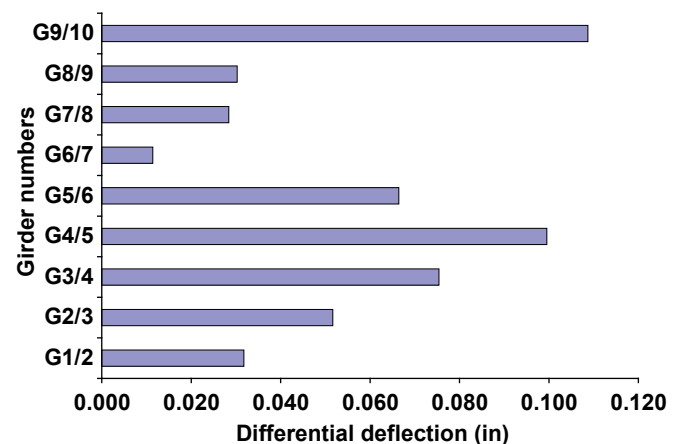
Moisture content measurements were taken at 10 locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 4, 7, and 10 at midspan and girders 1, 5, 7 and 10 at the west abutment. The bottom of the deck outside girder 10 and between girders 6 and 7 was measured at midspan. Measurements ranged from 20.2% to 23.0%. Overall, significant moisture content differences were not found throughout the bridge. The moisture content measurements are summarized in Table C4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because timber strength is not critically affected until deflections become excessive. However, the girder deflections do exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability.

Exceeding the live load deflection recommendations can have adverse affects on structure fasteners, wearing surface, and aesthetics, among other features. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasten-

**Figure C23. AASHTO design live load distribution.****Figure C24. Differential deflections for load path 1.**

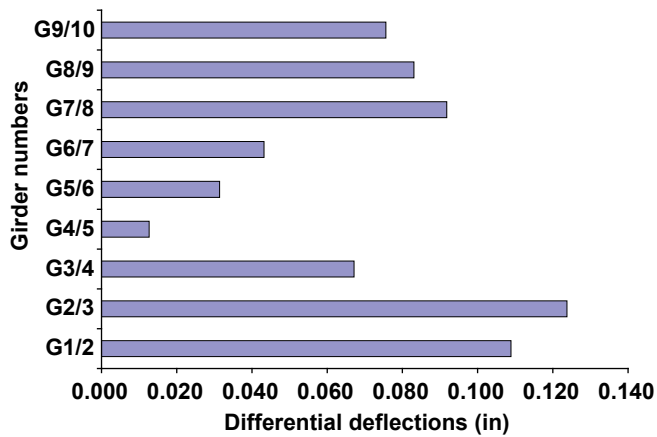


Figure C25. Differential deflections for load path 2.

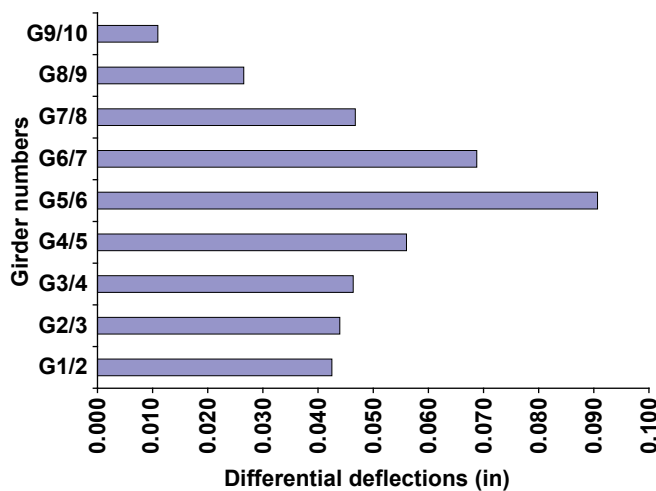


Figure C26. Differential deflections for load path 3.

Table C3—Maximum differential deflection

	Maximum differential deflection at midspan between adjacent girders (in.)
Load path 1	0.109
Load path 2	0.125
Load path 3	0.092

ers and wearing surface cracking produce a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded because asphalt has very little fatigue resistance. Numerous problems associated with cracking exist including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders

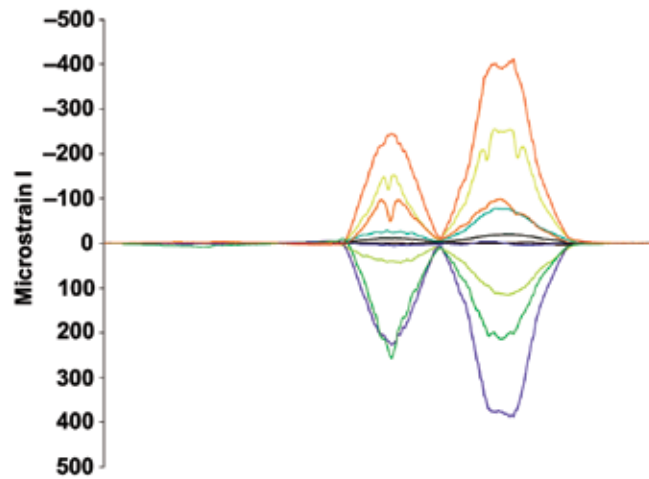


Figure C27. Strain at midspan for load path 1.

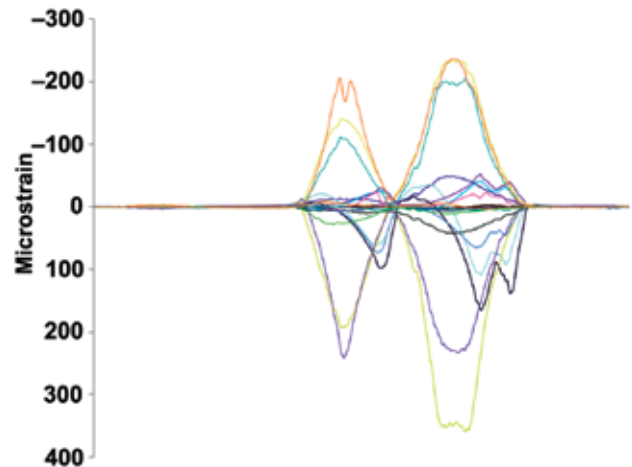


Figure C28. Strain at midspan for load path 2.

is also often a result of decay. Conditions are not ideal for seepage to quickly evaporate, because western North Carolina typically has a very humid climate. As a result, any water seepage through the deck will be prone to permeate the girders.

Through visual inspection, transverse cracks in the wearing surface were found. Deflections exceeding the recommended live load limit state would suggest that the wearing surface may show transverse cracking. The wearing surface of this particular bridge is in satisfactory condition, though close attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report, having investigated other timber

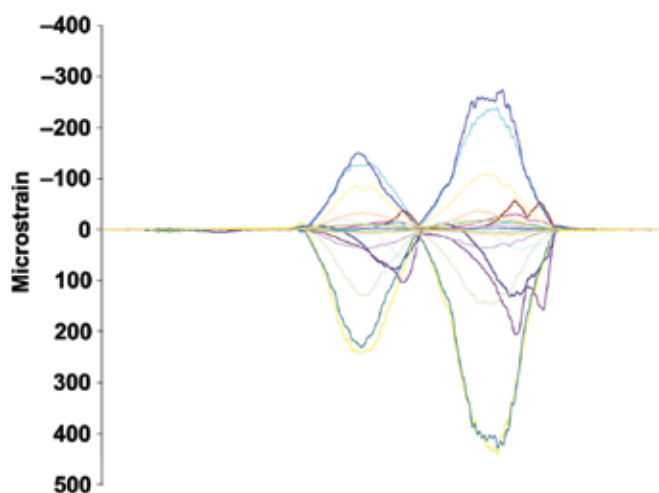


Figure C29. Strain at midspan for load path 3.

Table C4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, west abutment	21.8
Girder 1, midspan	20.5
Girder 4, midspan	21.6
Girder 5, west abutment	21.2
Girder 7, west abutment	20.2
Girder 7, midspan	21.0
Girder 10, west abutment	22.0
Girder 10, midspan	21.3
Bottom of deck outside girder 10	23.0
Bottom of deck between girders 6 and 7	22.8

bridge research, have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was over that limit by 0.023 in. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limit their use for primary analysis. Nevertheless, Figures C27 to C29 do show a reasonable relationship between truck position and

strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 403 and 437 microstrain, respectively. These strains equate to maximum stresses of 463 and 503 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 736 and 798 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses to anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the humid climate in North Carolina, higher moisture contents were expected and also found. The amount of water present in wood can modify its physical properties. With increasing moisture content, the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

The moisture content values were all within a couple percentage points of one another. This shows that none of the tested areas are subjected to vastly different amounts of moisture.

Conclusions

Several methods of condition and performance investigation were performed on the Chestnut Cove Creek Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; and finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of notable concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Minor transverse cracking in the wearing surface was observed. Seepage through the wearing surface and into the deck boards and girders was also evident. Some biotic growth was apparent on the underside of the deck and the faces of the girders, and seemed consistent with the moisture content measurements throughout the bridge. However, the growth appeared to be limited to the surfaces of these elements.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Deflection values at midspan, however, failed to meet recommended values.

Appendix D—North Carolina Bridge No. 560460, Madison County Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Figure D1. Madison County Bridge in North Carolina.

Abstract

The Madison County Bridge is a single-span timber girder bridge with a bituminous wearing surface located in Madison County, North Carolina. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and those of whom we, without their participation, would not have completed this research project: Henry Black, North Carolina Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing; Gary Moore, North Carolina Department of Transportation employee, who organized the load testing; Garney Rice, North Carolina Department of Transportation employee, who operated the load truck during testing; and Brett Rhindhart, North Carolina Department of Transportation employee, who assisted during the load testing.

Background

The location of North Carolina state bridge number 860131, hereinafter referred to as the Madison County Bridge, is shown in Figure D1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.



Figure D2. Substructure sill and columns.

The Madison County Bridge was built in 1957 and is located in Madison County in western North Carolina, 0.2 miles south of junction NC-251. SR1592 is carried by the structure. Currently, the bridge is posted for 16 tons (single vehicle) and 24 tons (type S3 truck).

Bridge Description

The Madison County Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface. The bridge length measures 14 ft–5 in. from the north backwall to the south backwall. The bridge width measures 17 ft–1 in. from inside of curb to inside of curb and 18 ft–8 in. from outside of rail to outside of rail. The substructure consists of solid timber posts and sills seated on concrete (Fig. D2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by



Figure D3. Madison County parapet.

timber blocks and bolts into the exterior girders along with bolts into the curb which is seated and bolted to the top of the deck, as shown in Figure D3.

Girders measure 14 ft–5 in. from end to end and have a clear span of 12 ft–5 in. A total of nine girders, spaced 2 ft center-to-center, measuring 5-3/4 in. by 11-1/2 in. in cross section, are present and are seated and toe-nailed to the 12- by 12-in. timber sills with spikes. The deck consists of individual 4- by 8-in. nominal boards laid transverse to the longitudinal girder direction, which are fastened to the girders with spikes. Overlaying the deck is a 2-in.-thick layer of

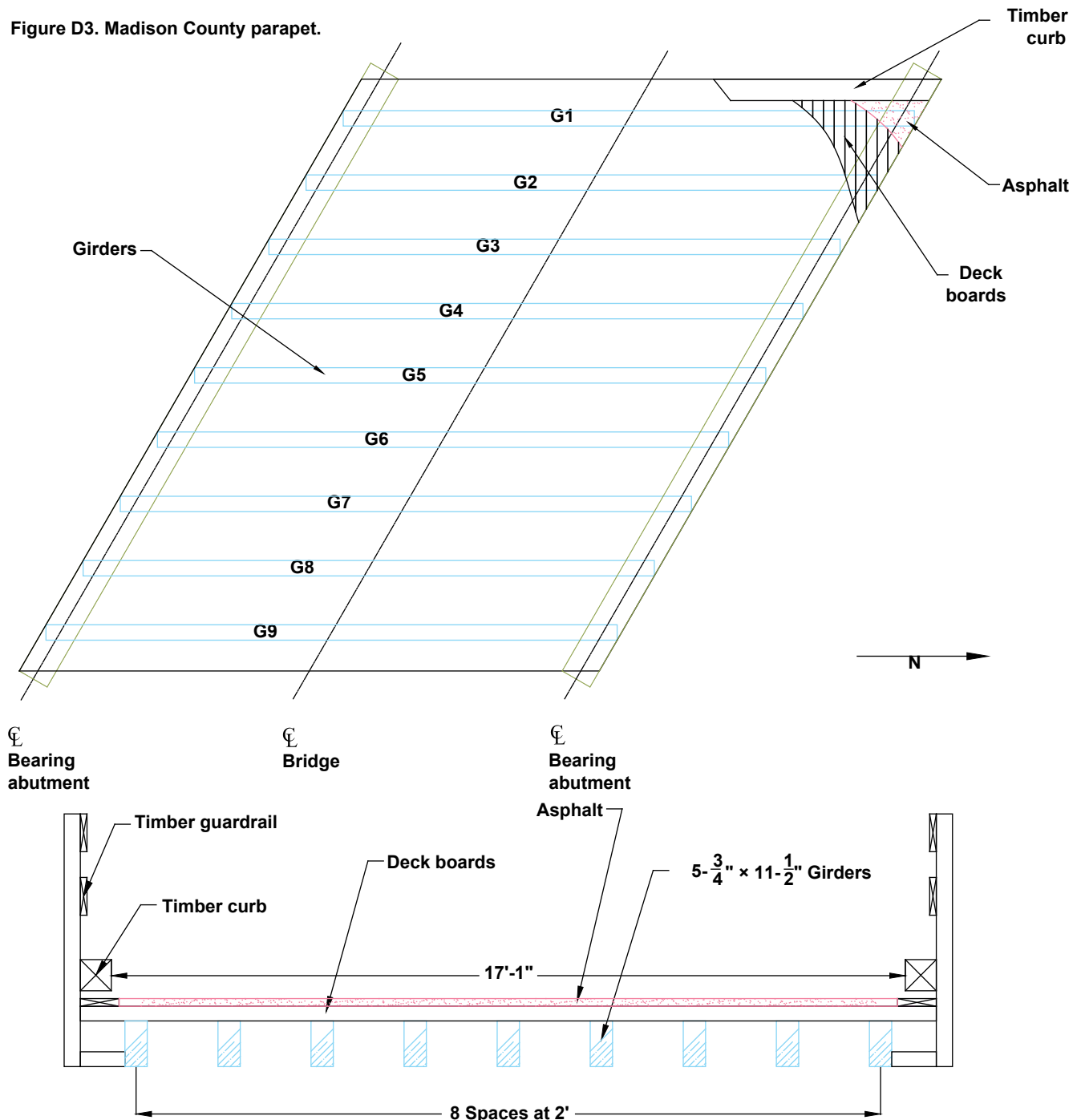


Figure D4. Plan and profile layout of Madison County Bridge.

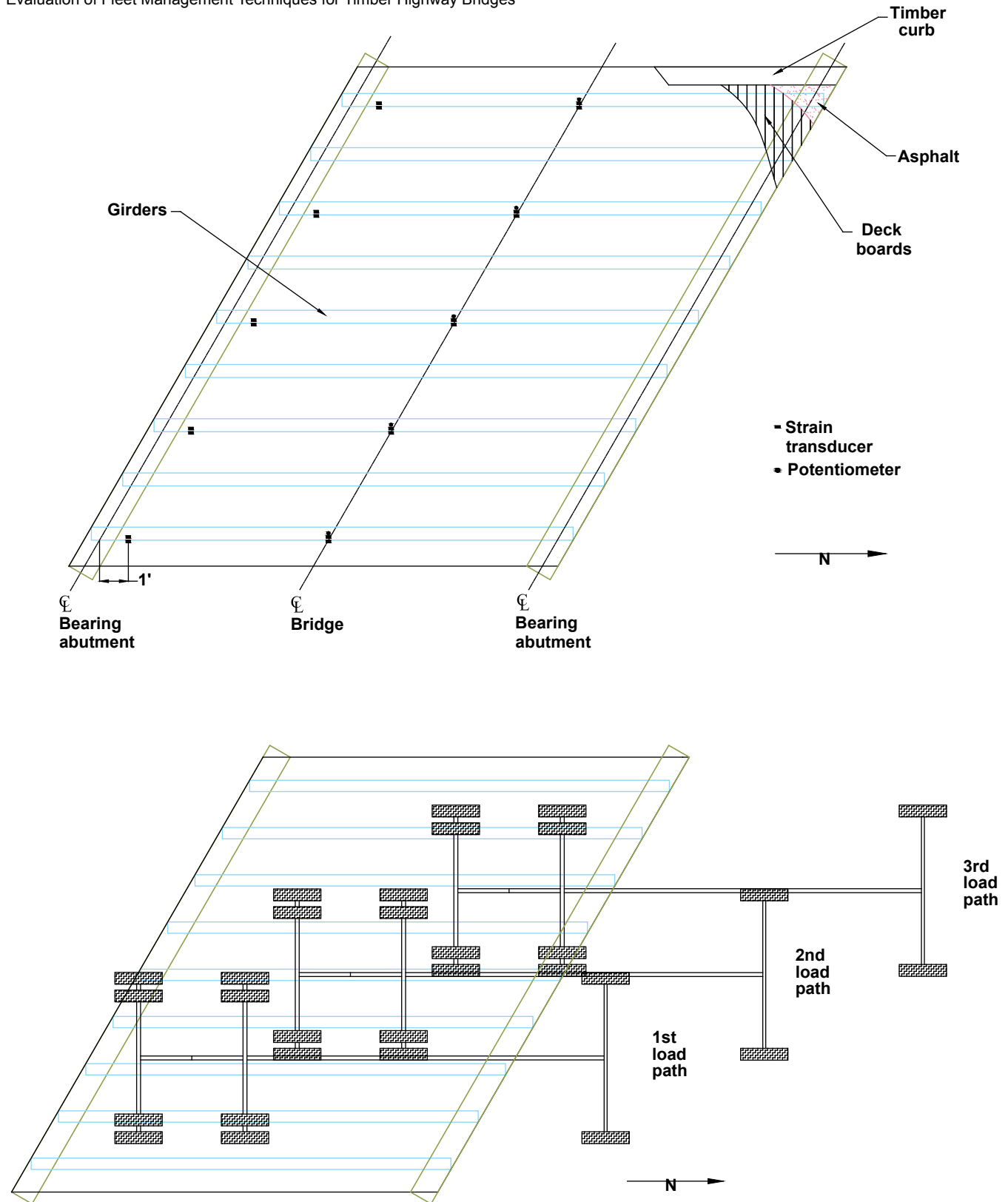


Figure D5. Instrumentation and load paths of Madison County Bridge.

asphalt wearing surface. Figure D4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain

transducers, and the truck position for each load path are shown in Figure D5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the nine girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-



Figure D6. Deflection instrumentation.



Figure D7. Strain transducers.

measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure D6.

Strain transducers were attached to girder numbers 1, 3, 5, 7, and 9 with 1 being the outside girder on the west side of the bridge and 9 being the outside girder on the east side of the

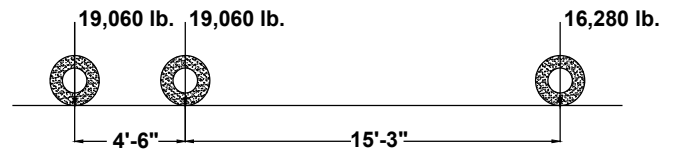


Figure D8. Truck configuration and axle loads.



Figure D9. Tandem-axle load truck.

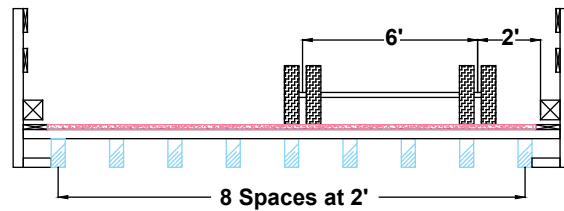


Figure D10. Transverse truck position—load path 1.

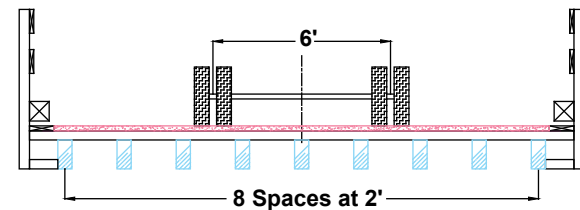


Figure D11. Transverse truck position—load path 2.

bridge. The midspan and one abutment were instrumented (see Fig. D5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder (see Fig. D7). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

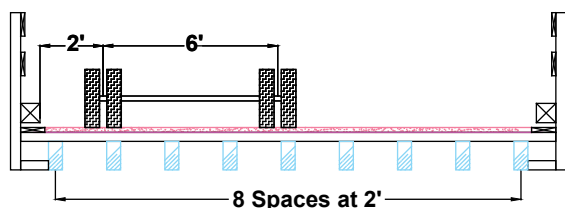


Figure D12. Transverse truck position—load path 3.



Figure D13. Pavement chipping at west end.



Figure D14. Transition between roadway and bridge.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the North Carolina Department of Transportation—Division 13. Dimensions of the truck are shown in Figure D8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–6 in.; the distance between the forward most rear axle and the front axle hubs measured 15 ft–3 in. Exact weight of the truck was 54,280 lb. The load over the front axle was 16,280 lb, and assuming that the load over each rear axle

was equal, the load was 19,060 lb over each rear axle.

Figure D9 shows the truck used for the load testing.

Three load paths were considered when testing the bridge (see Figs. D10 to D12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the right wheel line of the truck was driven 2 ft from the inside of the east curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the left wheel line of the truck was driven 2 ft from the inside of the west curb. For all load paths, the dump truck was driven at a crawl speed from south to north and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on April 22, 2004, was obtained from the North Carolina DOT (NC–DOT). This report was reviewed and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

Several transverse cracks were observed throughout the bridge wearing surface. These cracks aligned with the transverse floor boards (see Fig. D13). Aside from the cracks, the wearing surface appeared to be in good condition. At the transitions between the roadway and the bridge, however, larger cracks have formed that mirror the abutment backwalls, which leaves the bridge vulnerable to filtering at the abutments (see Fig. D14). Imminent decay is not likely, though these cracks could be cause for concern in the future.

Deck

The deck appeared to be in good condition, and all deck boards were securely fastened. At the ends of the deck boards, detachment of approximately 1/4 in. was observed. Very minor water staining from seepage through the wearing surface was present throughout, though there were no signs of imminent decay.

Superstructure

The superstructure appeared to be well protected from moisture, as only very minor water staining was observed. The most noteworthy observation was the checking found along the centerline of several girders (Figs. D15 and D16).



Figure D15. Checking at centerline of girder.



Figure D16. Checking at centerline near abutment.

Though the checking appeared to be minor, the condition should be watched with future inspections. The girder bearing on the sill was sufficient, and there is no misalignment.

Overall Structure

The overall structure is in satisfactory condition, and structurally the bridge is sound. No odor like anise or winter-green, signifying fungal growth, was present. There was no evidence of insect, mechanical, or ultraviolet degradation. Minor issues of concern in addition to those already stated include the presence of filtering at the abutments where various locations on the sill and backwalls were very wet. Moderate rot was observed at the base of the backwalls.

Results

The following presents the results of the static load testing of the Madison County Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

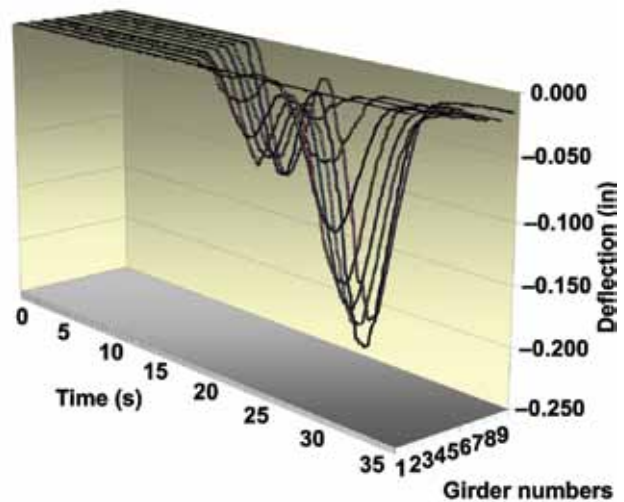


Figure D17. Deflections load path 1.

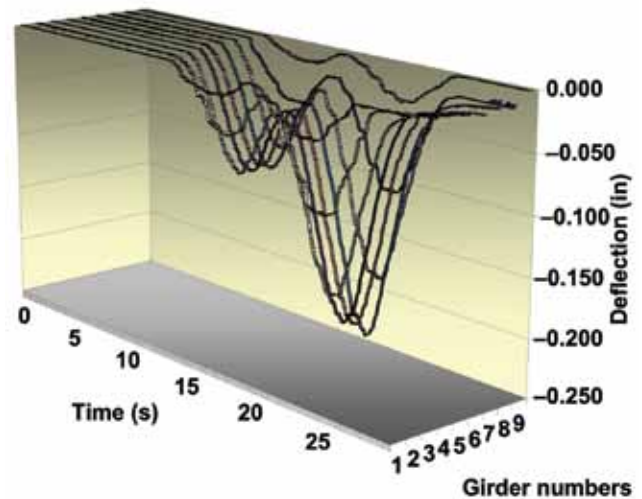


Figure D18. Deflections load path 2.

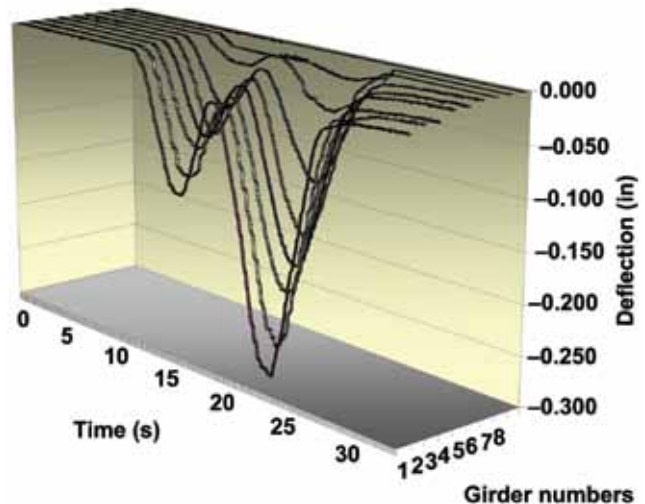


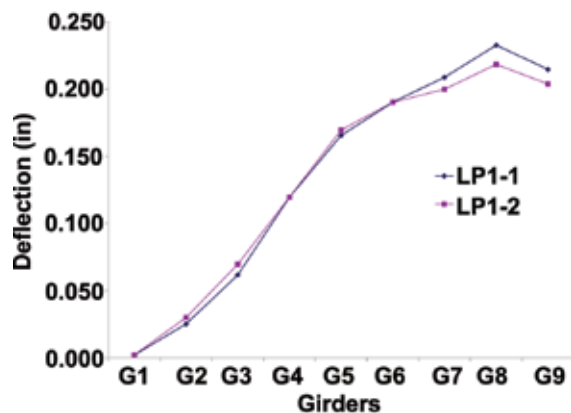
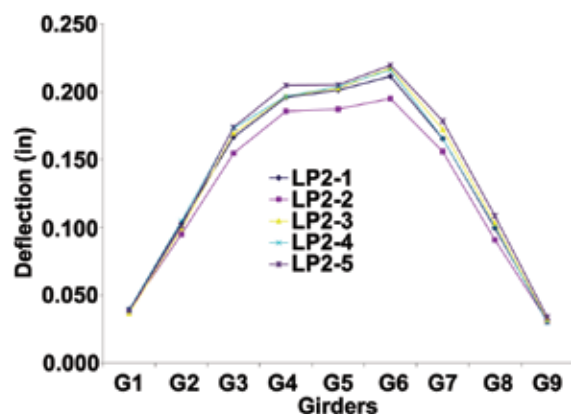
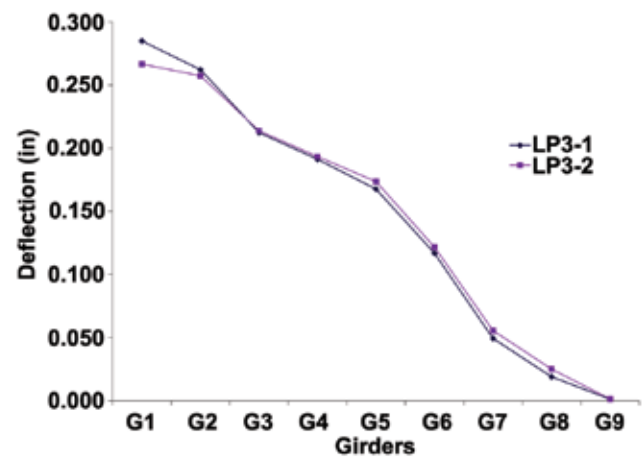
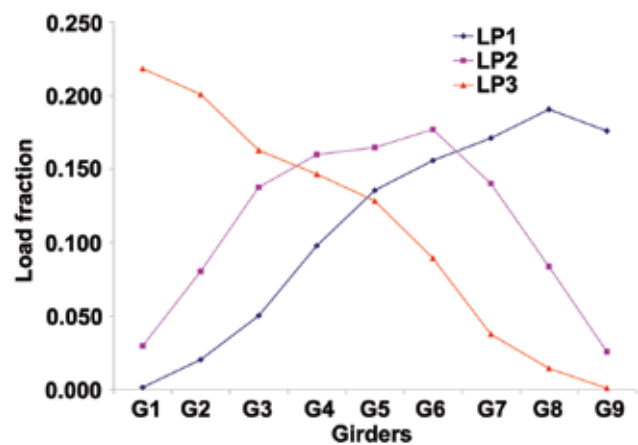
Figure D19. Deflections load path 3.

Table D1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.233
Load path 2	0.218
Load path 3	0.285

Table D2—Most critical *n* value

	<i>n</i> value for girder of most deflection for each load path
Load path 1	504
Load path 2	539
Load path 3	412

**Figure D20. Maximum deflections for load path 1.****Figure D21. Maximum deflections for load path 2.****Figure D22. Maximum deflections for load path 3.****Figure D23. Load fractions for each load path.**

Time-History Deflections

Figures D17 to D19 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck, one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges where one wave is typically present as the entire truck is supported by the girders at the same time. This two-wave relationship is quite evident in the figures, and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table D1. Each passing of the three load paths is illustrated in Figures D20 to D22. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for

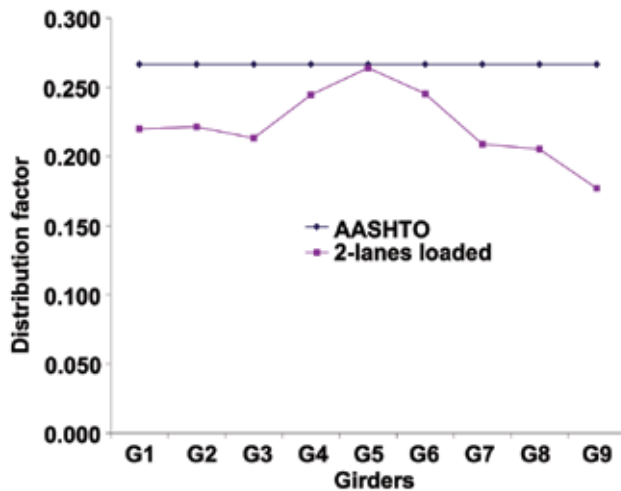


Figure D24. AASHTO design live load distribution.

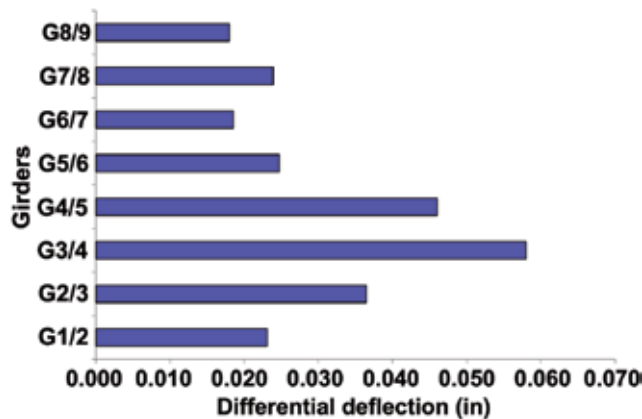


Figure D25. Differential deflections for load path 1.

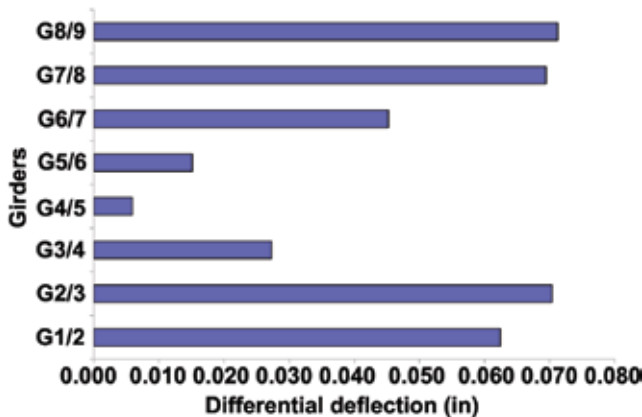


Figure D26. Differential deflections for load path 2.

each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

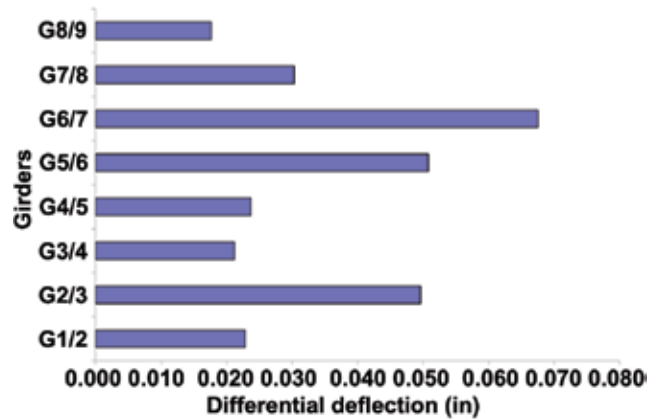


Figure D27. Differential deflections for load path 3.

Table D3—Maximum differential deflection

	Maximum differential deflection at midspan between adjacent girders (in.)
Load path 1	0.058
Load path 2	0.071
Load path 3	0.067

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table D2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths is 412. This value is at least greater than the minimum recommended value for timber girders from Ritter (1990). Load paths 1 and 2 minimum n values are greater than the recommended value from all three sources.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure D23 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure D24 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential

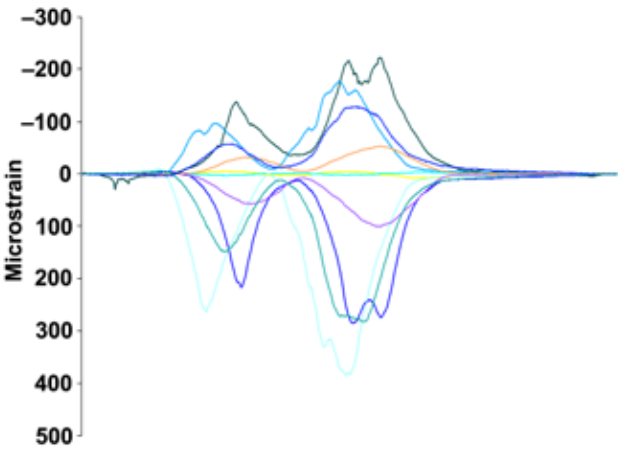


Figure D28. Strain at midspan for load path 1.

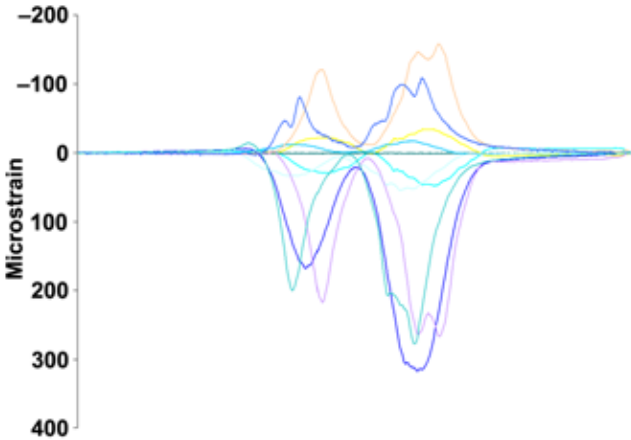


Figure D29. Strain at midspan for load path 2.

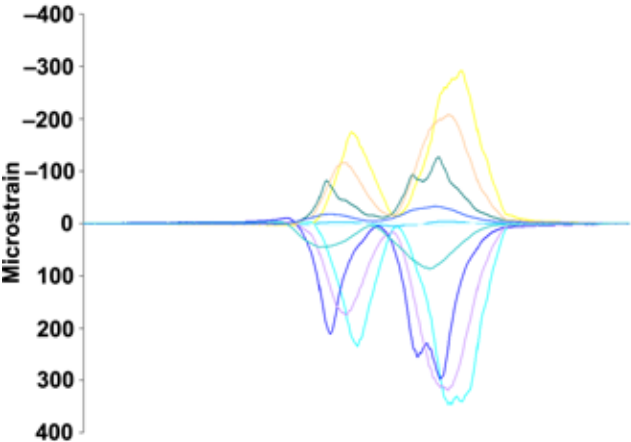


Figure D30. Strain at midspan for load path 3.

Table D4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, south abutment	25.1
Girder 1, midspan	24.2
Girder 5, south abutment	20.0
Girder 5, midspan	22.4
Girder 9, south abutment	29.8
Girder 9, midspan	27.8
Bottom of deck between girders 1 and 2	21.0
Bottom of deck between girders 5 and 6	26.2
Bottom of deck between girders 8 and 9	23.9

deflection between adjacent girders. Figures D25 to D27 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table D3.

The maximum differential deflection of 0.071 in. occurs in load path 2. This is nearly 33% of the maximum deflection resulting from that load path and 0.036 in. per ft of girder spacing. This value is within the recommended limit for differential deflection. The same is true for load paths 1 and 3 as the maximum differential deflections are both around 0.6 in.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 222 and 384 microstrain, respectively. The strain plot at midspan is shown in Figures D28 to D30 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures D28 to D30 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck; that is, the girders act independently of the deck when subjected to bending.

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 5, and 9 at midspan and girders 1, 5, and 9 at the south abutment. The bottom of the deck between girders 1 and 2, 5 and 6, and 8 and 9 was measured at midspan. Measurements ranged from 20.2% to 23.0%. Overall, though higher moisture contents were observed, significant moisture content differences were not found throughout the bridge. The moisture content measurements are summarized in Table D4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because timber strength is not critically affected until deflections become excessive. All recommended live load deflection limits derived primarily from wearing surface degradation and maintainability were satisfied except for load path 3, where one recommended limit was exceeded.

Exceeding the live load deflection recommendations can have adverse effects on structure fasteners, wearing surface, and aesthetics, among other features. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produce a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded because asphalt has very little fatigue resistance. Numerous problems associated with cracking exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. Conditions are not ideal for seepage to quickly evaporate as western North Carolina typically has a very humid climate. As a result, any water seepage through the deck will be prone to permeate the girders.

Through visual inspection, transverse cracks in the wearing surface were found. Deflections exceeding the recommended live load limit state would suggest that the wearing surface may show transverse cracking; however, as previously noted, the recommended limits were satisfied. Transverse cracking may be influenced by other factors besides live load deflection. The wearing surface of this particular bridge is in satisfactory condition, though close attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so,

the authors of this report having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was within that limit. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders. The differential deflections observed during testing of this bridge were not large in comparison with the results from load tests of similar bridges. It follows that longitudinal cracking was not found during visual inspection.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary because the intrinsic properties of wood limit their use for primary analysis. Nevertheless, Figures D28 to D30 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 222 and 384 microstrain, respectively. These strains equate to maximum stresses of 255 and 442 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 335 and 580 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the humid climate in North Carolina, higher moisture contents were expected and also found. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

The moisture content percentages were all within a couple percentage points of one another. This shows that none of the tested areas are subjected to vastly different amounts of moisture.

Conclusions

Several methods of condition and performance investigation were performed on the Madison County Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team

to verify prior inspection report comments and to more fully investigate element level condition; lastly, using a loaded tandem axle dump truck a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of notable concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Minor transverse cracking in the wearing surface was observed. Some very minor seepage through the wearing surface and into the deck boards and girders was also evident. Larger cracks are forming at the transition between the roadway and bridge wearing surface.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Also, deflection values at midspan mostly met recommended values.

Appendix E—North Carolina Bridge No. 560510, Red Horse Creek Bridge

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Abstract

The Red Horse Creek Bridge is a single-span timber girder bridge with a bituminous wearing surface located in Madison County, North Carolina. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and those without whom we would not have completed this research project: Henry Black, North Carolina Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing; Gary Moore, North Carolina Department of Transportation employee, who organized the load testing; Garney Rice, North Carolina Department of Transportation employee, who operated the load truck during testing; and Brett Rhindhart, North Carolina Department of Transportation employee, who assisted in the static load testing.

Background

The location of North Carolina state bridge number 560510, hereinafter referred to as the Red Horse Creek Bridge, is



Figure E1. Bridge location.



Figure E2. Red Horse Creek Bridge.

shown in Figure E1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Red Horse Creek Bridge was constructed in 1966 and is located in Madison County in western North Carolina, 0.1 miles west of junction SR1109 across Red Horse Creek. SR1110 is carried by the structure. Currently, the bridge is posted for 16 tons (single vehicle) and 24 tons (type S3 truck).

Bridge Description

The Red Horse Creek Bridge, shown in Figure E2, is a single-span, two-lane, timber girder bridge with a bituminous wearing surface. The bridge length measures 16 ft–2 in. from the north face of the north backwall to the south face of the south backwall. The bridge width measures 19 ft–1 in. from inside of curb to inside of curb and 20 ft–9 in. from outside of rail to outside of rail. The substructure consists of a timber sill seated on solid timber posts (see Fig. E3).



Figure E3. Bridge substructure.

Girders measure 16 ft–0 in. from end to end and have a clear span of 14 ft–0 in. A total of 10 girders, spaced 2 ft center-to-center, measuring 5-3/4 by 11-3/4 in. in cross-section are present and are seated and toe-nailed to the 12- by 12-in. timber sill with spikes. The deck consists of individual 4- by 8-in. nominal boards laid transverse to the longitudinal girder direction, which are fastened to the girders with spikes. Overlaying the deck is a 2-in.-deep layer of asphalt wearing surface. Figure E4 illustrates the layout of the bridge.

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by timber blocks and bolts into the exterior girders along with bolts into the curb, which is seated and bolted to the top of the deck.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure E5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 10 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured center line of the clear span. Non-stretch piano wire was used to connect the deflection gage string to the eye hook, and the base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure E6.

Strain transducers were attached to girder numbers 1, 4, 7, and 10, with 1 being the outside girder on the west side of the bridge and 10 being the outside girder on the east side of the bridge. Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder (see Fig. E7). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the North Carolina Department of Transportation–Division 13. Dimensions of the truck are shown in Figure E8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–6 in.; the distance between the forward most rear axle and the front axle hubs measured 15 ft–3 in. Exact weight of the truck was 54,280 lb. The load over the front axle was 16,280 lb, and assuming that the load over each rear axle was equal, the load was 19,060 lb over each rear axle. Figure E9 shows the truck used for load testing.

Three load paths were considered when testing the bridge (see Figs. E10 to E12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the west curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the left wheel line of the truck was driven 2 ft from the inside of the east curb. For all load paths, the truck was driven at a crawl speed from the south to the north and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on April 20, 2004, was obtained from the North Carolina DOT (NC–DOT). This report was reviewed and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU research team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

According to the NC–DOT 2004 report, the structure was still in good condition. No wearing surface problems were noted. The ISU inspection did note, however, that a number of transverse cracks had developed. Though determined to be relatively minor, it was decided that these cracks should be noted nonetheless. The asphalt pavement generally looked to be in good condition; however, at the transition between the roadway and the bridge wearing surface at the north end, a large crack has formed. The pavement condition and transition crack are shown in Figure E13. Large transition cracks leave the bridge susceptible to water seepage at the abutments and girder ends. Another notable, though minor, problem is the overgrowth along the east side of

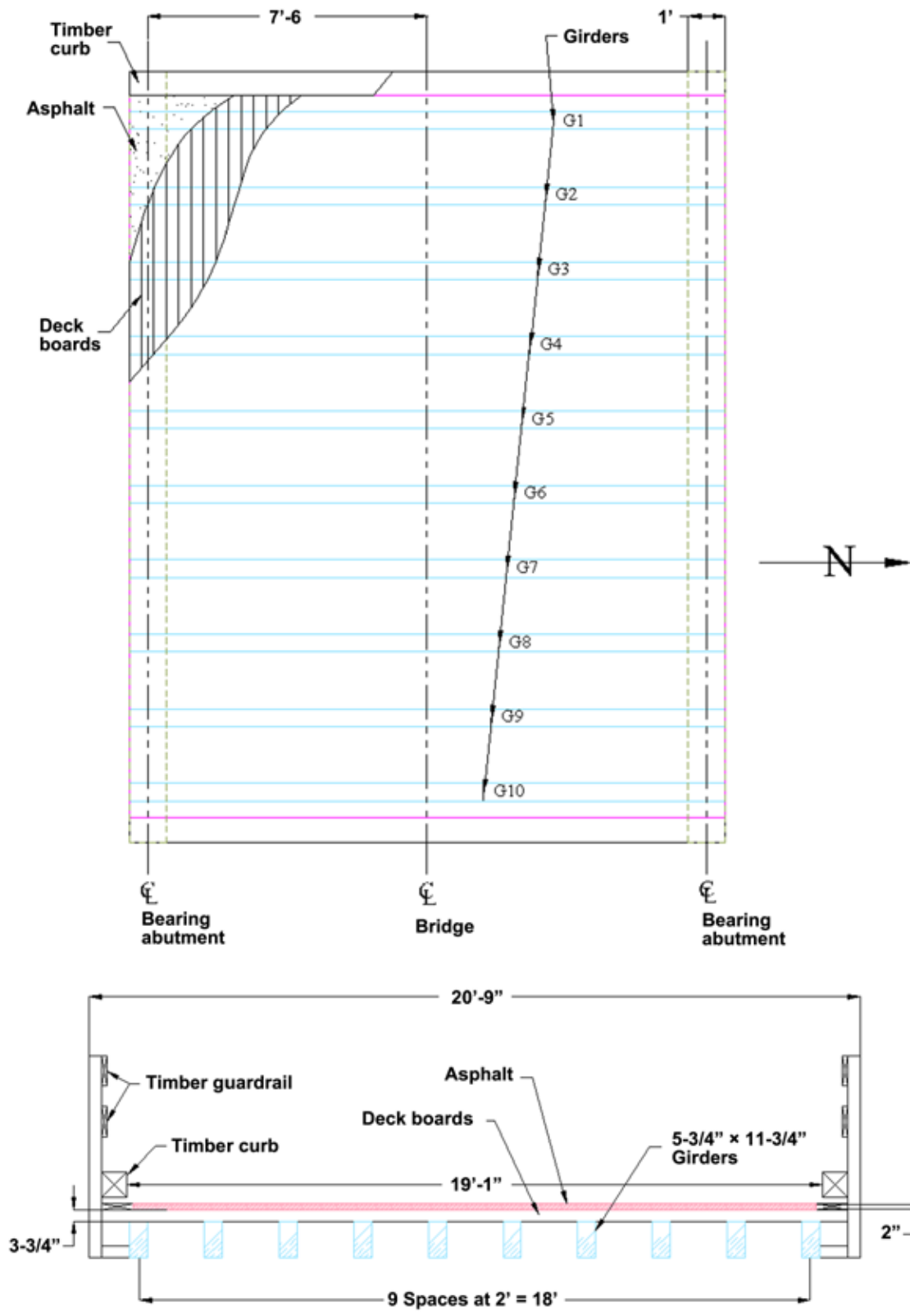


Figure E4. Plan and Profile layout of Red Horse Creek Bridge.

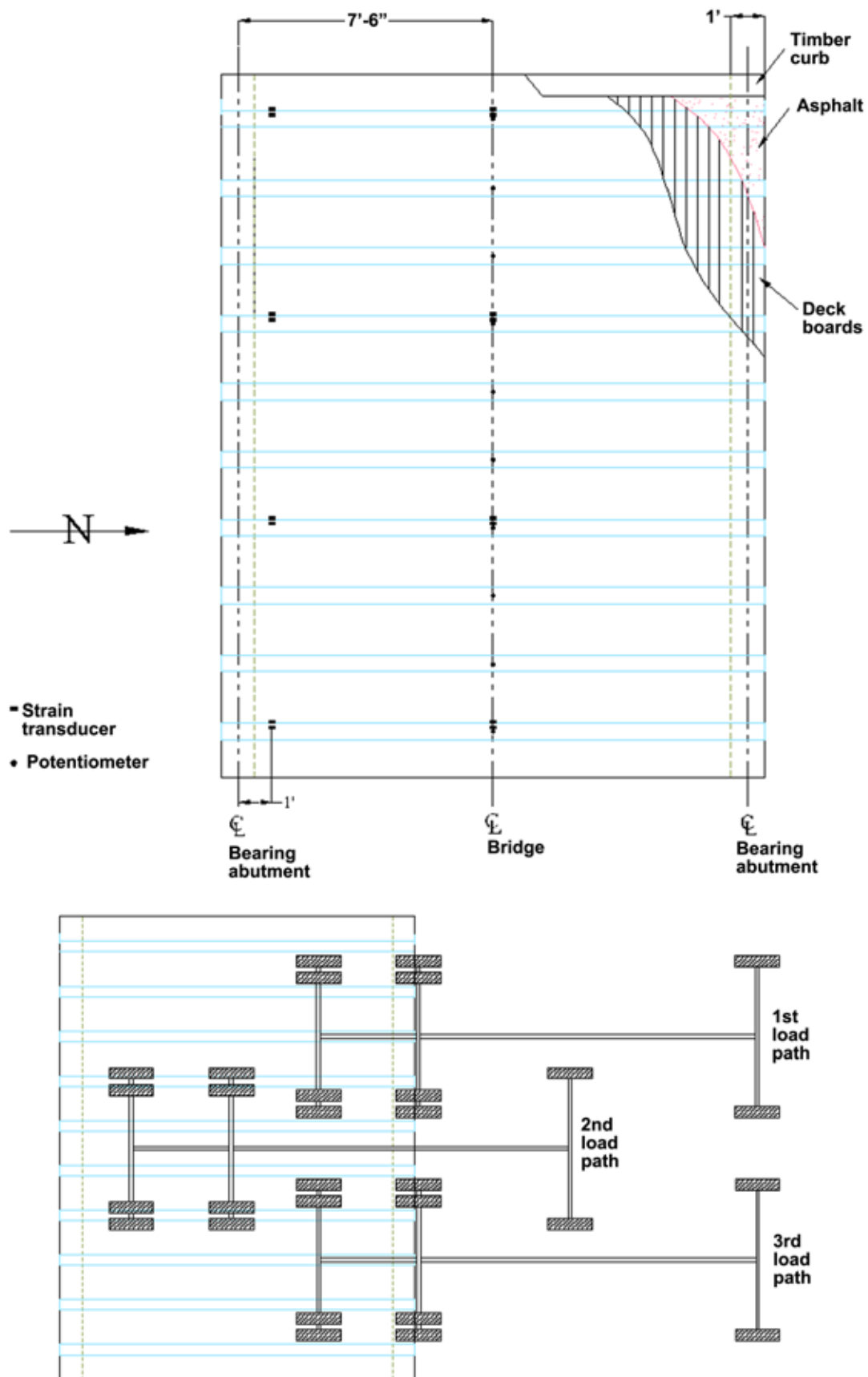


Figure E5. Instrumentation and load paths of Red Horse Creek Bridge.



Figure E6. Deflection instrumentation.



Figure E7. Strain transducers.

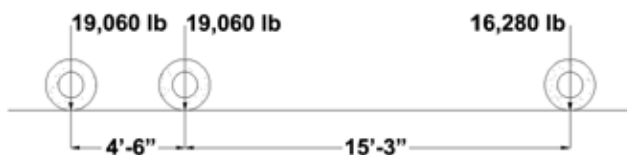


Figure E8. Truck configuration and axle loads.

the bridge. Excessive vegetation can promote pavement deterioration.

Deck

Overall, the deck appeared to be in good condition. The deck boards were securely fastened to the girders except at the outermost girders, where some detachment has occurred (see Fig. E14). Minor water staining from seepage through



Figure E9. Tandem-axle load truck.

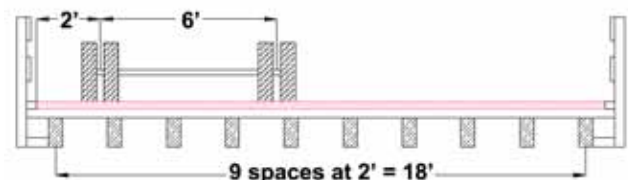


Figure E10. Transverse truck position—load path 1.

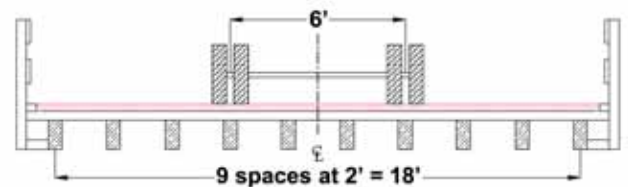


Figure E11. Transverse truck position—load path 2.

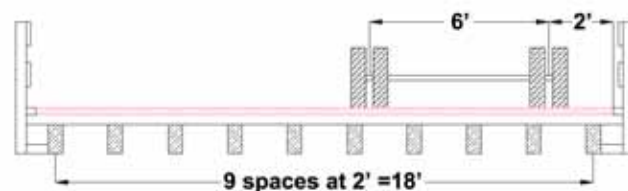


Figure E12. Transverse truck position—load path 3.

the wearing surface was present throughout, and white residue was present in a number of locations, though there were no signs of imminent decay.

Superstructure

The interface between the deck boards and girders showed that only minor seepage through the wearing surface has occurred. White residue was present in some areas, though this issue was considered very minor.

A number of checks were observed at or near the centerline of the girders throughout the bridge superstructure. These



Figure E13. Pavement condition.



Figure E14. Deck board detachment.

checks are not detrimental to the structural integrity of the bridge in the current state. Advances in degradation may present future problems at these locations, however. Considering the age of the bridge, these checks are not thought to be abnormal. Figure E15 shows an example of observed checks. The girder bearing on the sill was sufficient and there is no misalignment.

Overall Structure

The overall structure is in satisfactory condition, and structurally the bridge is sound. No odor like anise or winter-green, signifying fungal growth, was present. There was no evidence of insect, mechanical, or ultraviolet degradation. Minor issues of concern besides those already stated include the presence of filtering at the abutments, where various locations on the sill and backwalls were very wet. There were minor checks in the parapet and considerable checks in the parapet curb.

Results

The following presents the results of the static load testing and finite element modeling of the Red Horse Creek Bridge.



Figure E15. Observed checking in girders.

These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures E16 to E18 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck, one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in the figures, and clearly the deflections reflect the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table E1. Each passing of the three load paths is illustrated in Figures E19 to E21. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table E2 lists the n value for the girder of most deflection for each load path.

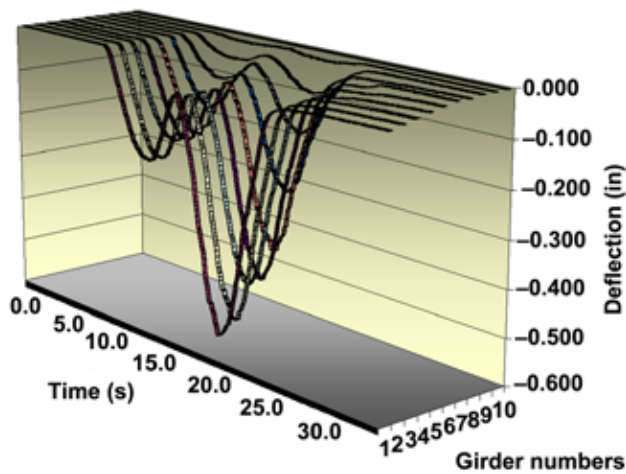


Figure E16. Deflections load path 1.

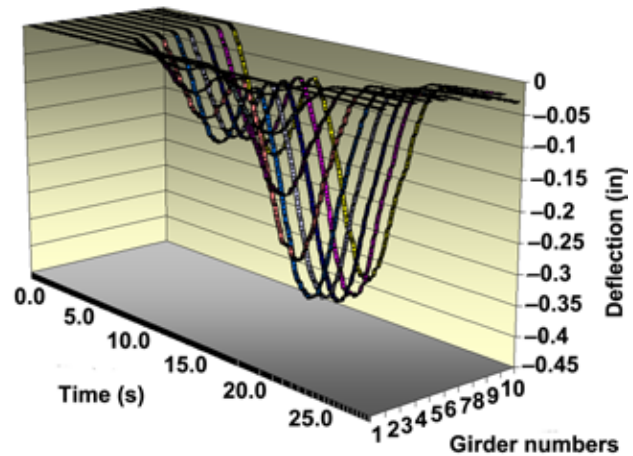


Figure E18. Deflections load path 3.

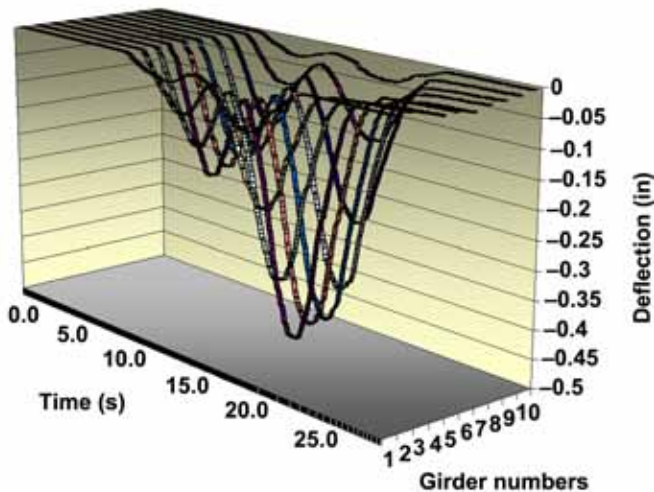


Figure E17. Deflections load path 2.

The minimum n value of the three load paths is 238. This value is less than the minimum recommended value for timber girders. In fact, all these n values are below the recommended n values stated in Table E2. The possible reasons for deflections greater than those recommended will be discussed later.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure E22 shows the distribution factors for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this case, the exterior lane live load distribution factors equal those of the interior lanes. Shown in Figure E23 is the comparison of design live

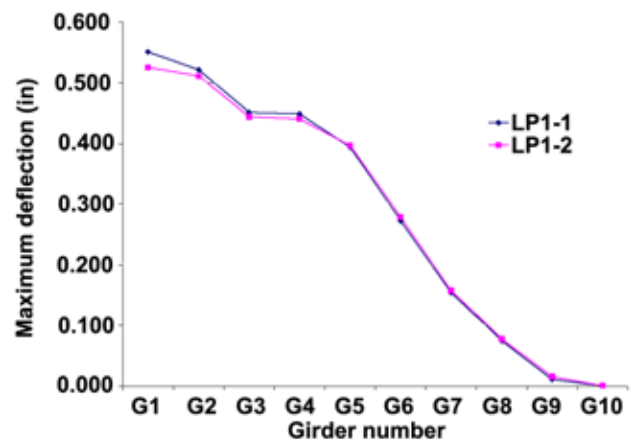


Figure E19. Maximum deflections for load path 1.

load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures E24 to E26 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table E3.

The maximum differential deflection of 0.144 in. occurs in load path 2. This is nearly 32% of the maximum deflection resulting from that load path and 0.072 in. per ft of girder spacing. Among other potential reasons for large differential deflections, the possibility exists that the load is not well distributed transversely between these two girders or the assumption that both girders are of equal stiffness is false. The same is true for load paths 1 and 3 as the maximum differential deflections are both around 0.12 in.

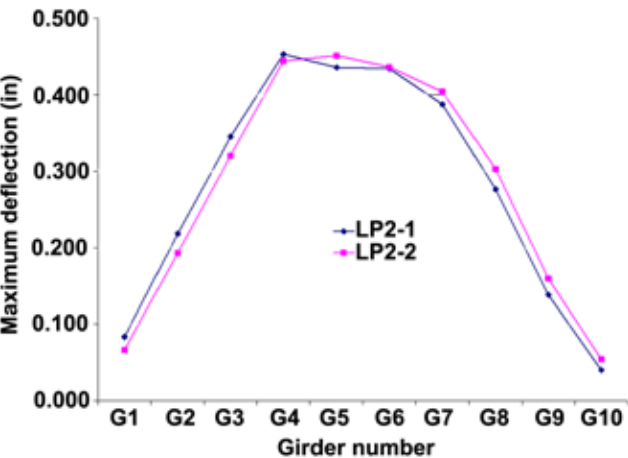


Figure E20. Maximum deflections for load path 2.

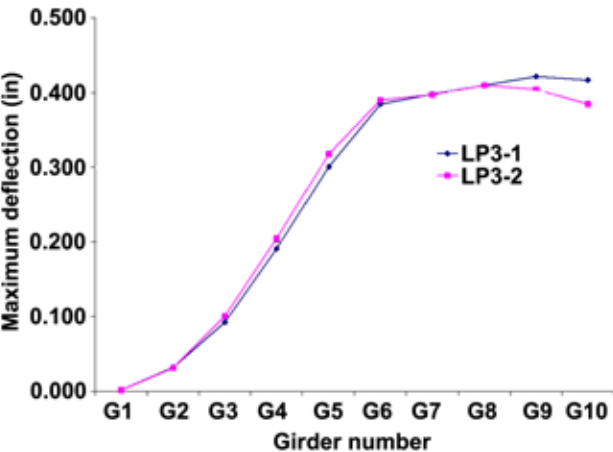


Figure E21. Maximum deflections for load path 3.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 619 and 581 microstrain, respectively. The strain plot at midspan is shown in Figures E27 to E29 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Table E2—Most critical *n* value

	<i>n</i> value for girder of most deflection for each load path
Load path 1	238
Load path 2	283
Load path 3	308

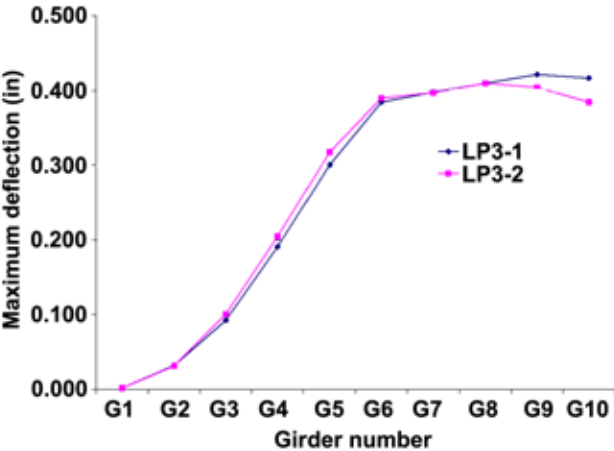


Figure E22. Load fractions for each load path.

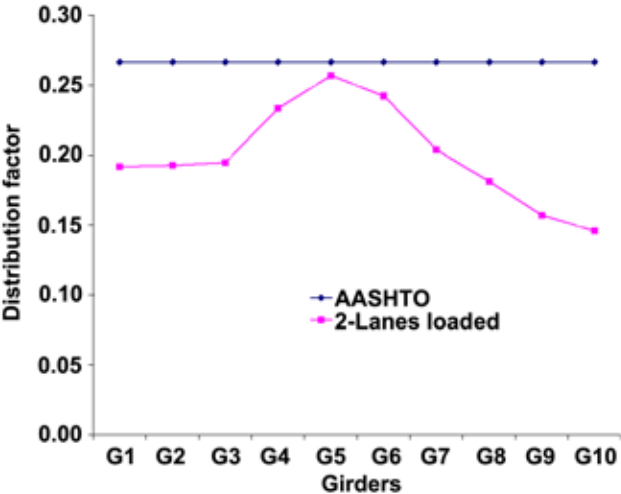


Figure E23. AASHTO design live load distribution.

Figures E27 to E29 also illustrate the proportion about the neutral axis at midspan. That is, the tensile strains nearly reflect the compression strains about the neutral axis. The proportional pattern of the data signifies that there is very little if any composite action with the deck; that is, the beams act independently of the deck when subjected to bending.

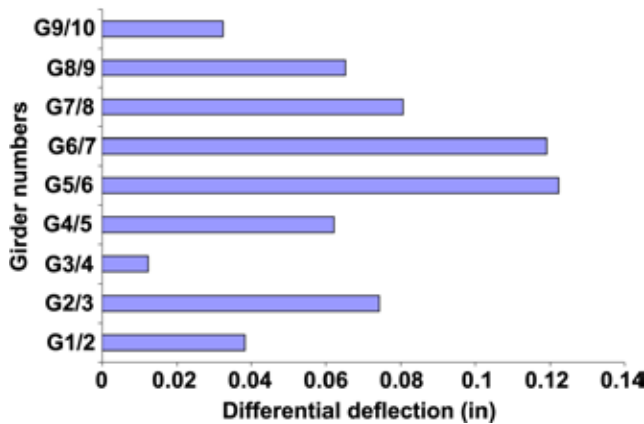


Figure E24. Differential deflections for load path 1.

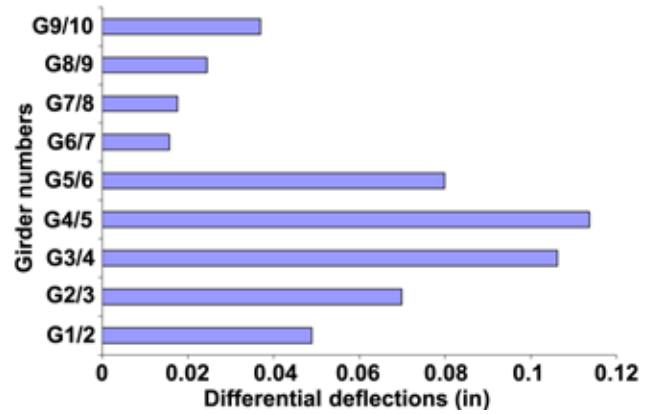


Figure E26. Differential deflections for load path 3.

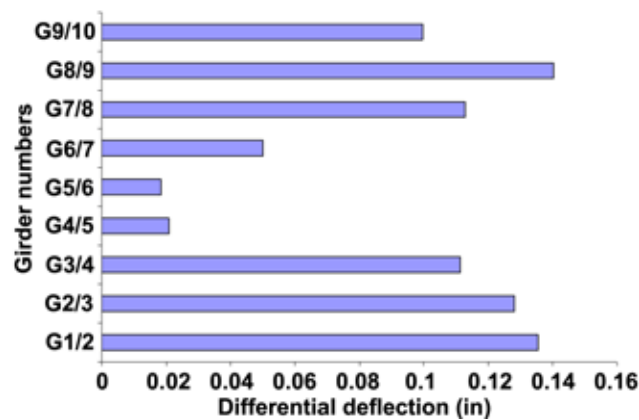


Figure E25. Differential deflections for load path 2.

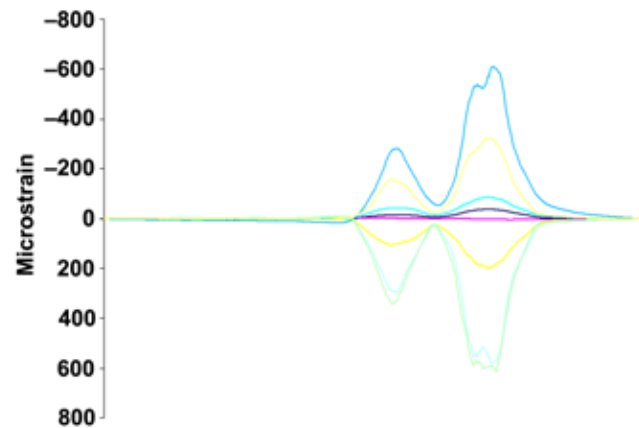


Figure E27. Strain at midspan for load path 1.

Table E3—Maximum differential deflection

	Maximum differential deflection at midspan between adjacent girders (in.)
Load path 1	0.123
Load path 2	0.144
Load path 3	0.114

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 5, and 10 at midspan and at the south abutment. The bottom of the deck between girders 1 and 2, 5 and 6, and 9 and 10 was measured at midspan. Measurements ranged from 13.0% to 25.9%. The moisture content measurements are summarized in Table E4.

Finite Element Analysis

A finite element model was developed (see Fig. E30) for the Red Horse Creek Bridge using ANSYS, a well known

finite element software. The objective was to create a model that would replicate field results when subjected to the same loading. After calibrating the model to the midspan deflection results obtained from the static load test, it was decided that the model would be subjected to a load simulating the AASHTO HS20 tandem-axle design vehicle. Deflection and tensile strain results at midspan were obtained from the model.

Figures E31 to E33 show the calibrated model results when subjected to the same load as that during the static load test. Notice the similarities between the ANSYS and field results.

Figure E34 shows the maximum deflections at midspan after subjecting the finite element model to the load of the AASHTO HS20 tandem-axle design vehicle traveled along each load path.

Figure E35 shows the maximum tensile stresses at midspan due to the AASHTO HS20 tandem-axle design vehicle traveled along each load path.

Discussion of Results

The following discussion is based on the results presented, including deflections at midspan, distribution factors,

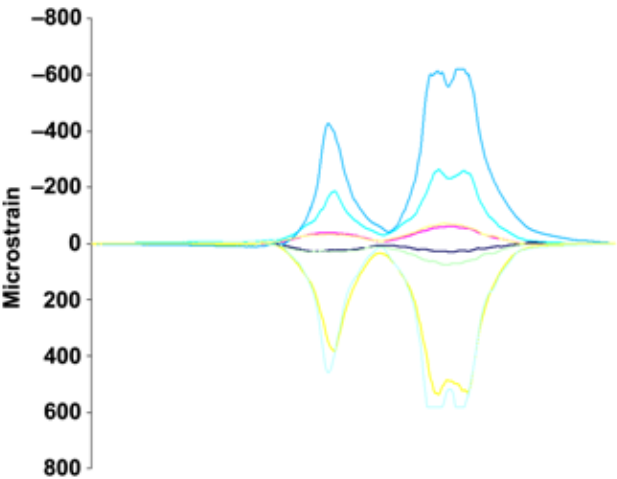


Figure E28. Strain at midspan for load path 2.

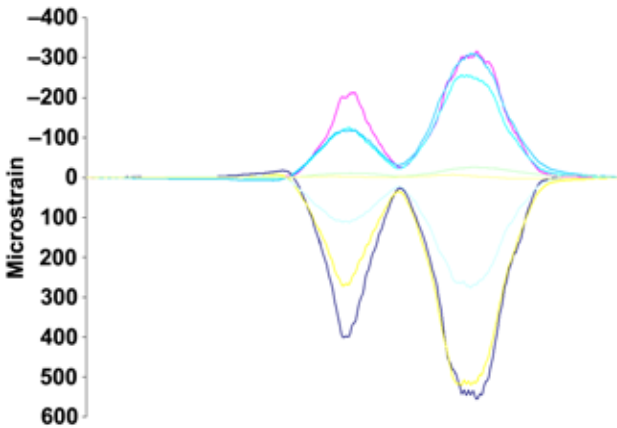


Figure E29. Strain at midspan for load path 3.

differential deflections, girder strain, moisture content, and finite element results.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because only until deflections become excessive is strength critically affected. However, the girder deflections do exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability, and also user comfort.

Exceeding the live load deflection recommendations can have adverse affects on structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produces a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded because asphalt has very little fatigue resistance. Numerous problems associated with

Table E4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, south abutment	23.0
Girder 1, midspan	18.8
Girder 5, south abutment	25.9
Girder 5, midspan	24.8
Girder 10, south abutment	19.0
Girder 10, midspan	13.5
Bottom of deck between girders 1 and 2	14.3
Bottom of deck between girders 5 and 6	13.0
Bottom of deck between girders 9 and 10	13.3

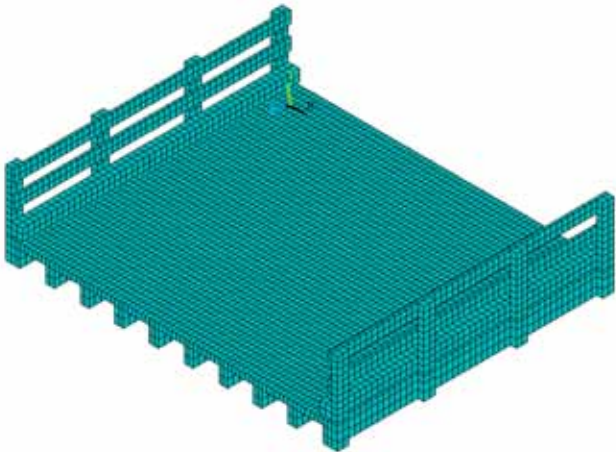


Figure E30. Finite element model.

cracking exist including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. Conditions are not ideal for seepage to quickly evaporate because western North Carolina typically has a very humid climate. As a result, any water seepage through the deck will be prone to permeate the girders.

Through visual inspection, transverse cracks in the wearing surface were found. Deflections exceeding the recommended live load limit state would suggest that the wearing surface may show transverse cracking. The wearing surface of this particular bridge is in satisfactory condition, though close attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This

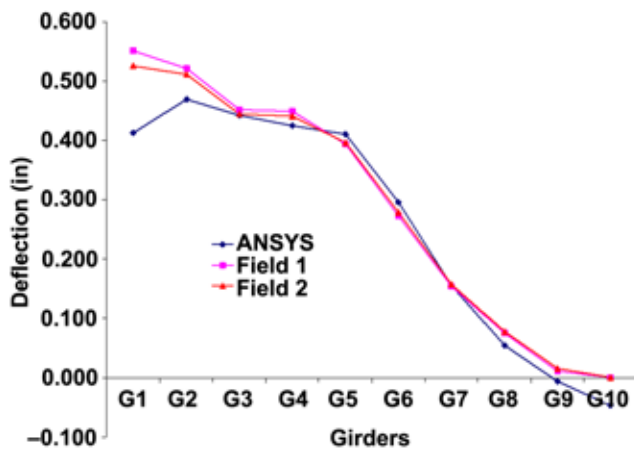


Figure E31. ANSYS calibration results load path 1.

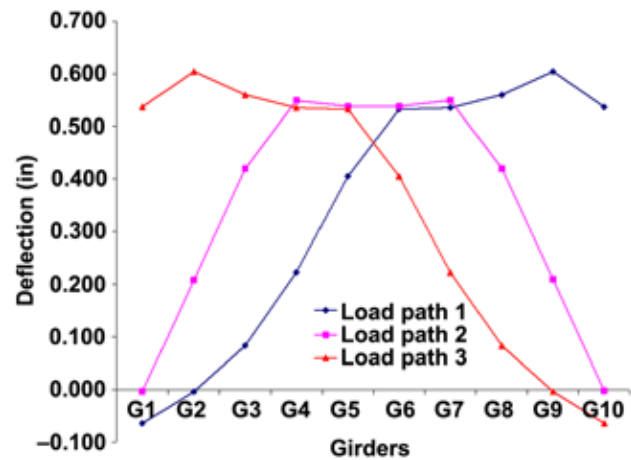


Figure E34. ANSYS deflection results for each load path when subjected to HS20 tandem-axle design vehicle.

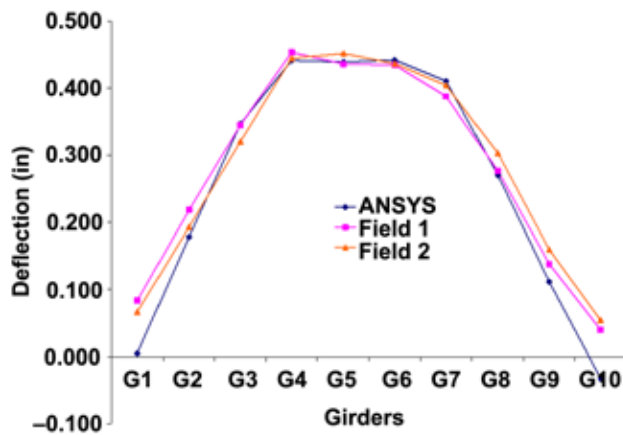


Figure E32. ANSYS calibration results load path 2.

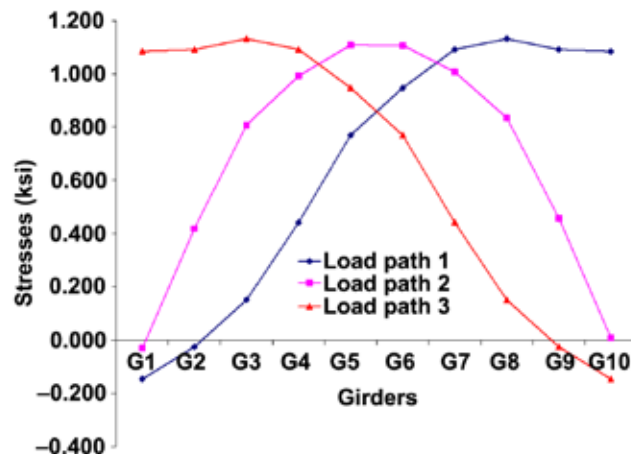


Figure E35. ANSYS tensile stress for each load path when subjected to HS20 tandem-axle design vehicle.

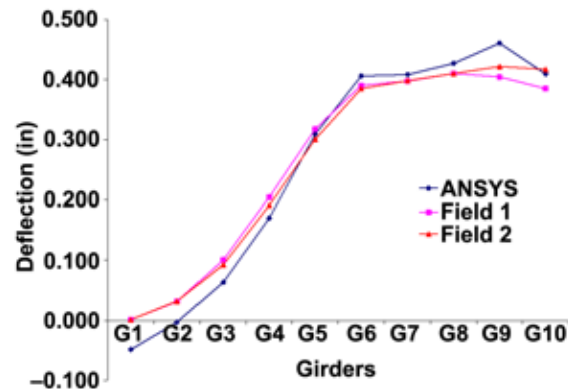


Figure E33. ANSYS calibration results load path 3.

bridge was over that limit by 0.022 in. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction

(that is, the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limit their use for primary analysis. Nevertheless, Figures E27 to E29 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained

from the three load paths are 619 and 581 microstrain, respectively. These strains equate to maximum stresses of 712 and 667 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 934 and 875 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard trucks.

Due to the humid climate in North Carolina, higher moisture contents were expected and also found in some locations of the bridge. Generally, the girders had higher moisture content than the deck. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength. Strengths of the girders should be consistent with respect to moisture content as each of the girders had approximately the same moisture content.

Maximum midspan stresses and deflections were obtained from the finite element model. The maximum deflection was 0.604 in. from load paths 1 and 3, and 0.550 in. from load path 2. Much like the normalized vehicle loading, these results exceeded the recommended limit states for live load deflection. The maximum stresses at midspan for load paths 1 and 3, and 2 were 1,131 and 1,108 lb/in², respectively. Much like the stresses obtained from the normalized vehicle loading, these values were within the values set by allowable stress design. The finite model is consistent with the results discussed previously; recommended live load deflection limits were exceeded and allowable stresses were not exceeded.

Conclusions

Several methods of condition and performance investigation were performed on the Red Horse Creek Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of notable concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Minor transverse cracking in the wearing surface and a larger crack at the bridge/roadway transition at the north end was observed. Some seepage through the wearing surface and into the deck boards and girders was also evident. Though considered minor, some white residue was present and appeared to be the result of high moisture conditions.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel along the same load paths. Live load distribution factors were within AASHTO's prescribed design live load distribution. Deflection values at midspan, however, failed to meet recommended values.

The finite element model yielded results that were consistent with the bridge performance under live load. Recommended live load deflection limits at midspan were exceeded, while allowable stresses at midspan were not.

Appendix F—North Carolina Bridge No. 560385, Madison County Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Abstract

The Madison County Bridge is a single-span, timber girder bridge with a bituminous wearing surface located in Madison County, North Carolina. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: Henry Black, North Carolina Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing; Gary Moore, North Carolina Department of Transportation employee, who organized the load testing; Garney Rice, North Carolina Department of Transportation employee, who operated the load truck during testing; and Brett Rhindhart, North Carolina Department of Transportation employee, who assisted in the static load testing.

Background

The location of North Carolina state bridge number 560385, hereinafter referred to as the Madison County Bridge, is shown in Figure F1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Madison County Bridge was built in 1962 and is located in Madison County in western North Carolina,



Figure F1. Madison County Bridge in North Carolina.



Figure F2. Madison County Bridge.

0.1 mile south of junction SR1177. SR1122 is carried by the structure. Currently, the bridge is posted for 15 tons (single vehicle) and 21 tons (type S3 truck).

Bridge Description

The Madison County Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface (see Fig. F2). The bridge length measures 16 ft–0 in. from the north face of the north backwall to the south face of the south backwall. The bridge width measures 19 ft–3 in. from inside of curb to inside of curb and 20 ft–10 in. from outside of rail to outside of rail. The substructure consists of solid timber posts and sills (see Fig. F3).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by timber blocks and bolts into the exterior girders along with bolts into the curb, which is seated and bolted to the top of the deck.

Girders measure 16 ft–0 in. from end to end and have a clear span of 14 ft–0 in. A total of 16 girders, spaced 14 in. center-to-center, measuring 5 by 11-1/2 in. in cross-section, are present and are seated and toe-nailed to the 12- by 12-in. timber sills with spikes. The deck consists of individual 4- by 8-in. nominal boards laid transverse to the longitudinal girder direction, which are fastened to the girders



Figure F3. Madison County Bridge substructure.

with spikes. Overlaying the deck is a 2-in.-thick layer of asphalt wearing surface. Figure F4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure F5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 16 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. A typical setup of deflection instrumentation is shown in Figure F6.

Strain transducers were attached to girder numbers 1, 4, 7, 10, 13, and 16, with 1 being the outside girder on the west side of the bridge and 16 being the outside girder on the east side of the bridge. The midspan and one abutment were instrumented (see Fig. 5). Transducers were placed near only one abutment because of the symmetry of the bridge. Due to the proximity of the girders, only the exterior girders were equipped with strain gages in the compression zone. All strain instrumented girders were equipped with tensile strain gages. A typical setup at the outside girders is shown in Figure F7. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the North Carolina Department of Transportation—Division 13. Dimensions of the truck are shown in Figure F8. The rear wheel base was 6 ft–0 in.; the

distance between the hubs of the two rear axles measured 4 ft–6 in.; the distance between the forward most rear axle and the front axle hubs measured 15 ft–3 in. Exact weight of the truck was 54,280 lb. The load over the front axle was 16,280 lb, and assuming that the load over each rear axle was equal, the load was 19,060 lb over each rear axle.

Figure F9 shows the truck used for the load testing.

Three load paths were considered when testing the bridge. Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb, in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the west curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the east curb. For all load paths, the dump truck was driven at a crawl speed from east to west and multiple passes were made on each path to ensure the collected data were repeatable. Each load path is illustrated in Figures F10 to F12.

Condition Assessment

The report for the bridge inspection conducted on June 23, 2004, was obtained from the North Carolina DOT (NC–DOT). This report was reviewed, and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed.

Wearing Surface

A number of transverse cracks were present throughout the bridge wearing surface, and these cracks are consistent with the direction of the floorboards. Figure F13 shows the transverse cracking.

At the transition between the roadway and the bridge and at approximately 5 ft before the bridge, large cracks have formed (see Fig. F14). An uneven transition could subject the bridge to unnecessary effects from dynamic loads even though slow vehicle speeds on this roadway make this unlikely.

Deck

The deck appeared to be in overall good condition yet some detachment and warping was evident at the ends of the deck boards. Even so, all deck boards appeared securely fastened. Figure F15 shows this detachment. Minor water staining from seepage through the wearing surface was present throughout, although there were no signs of imminent decay.

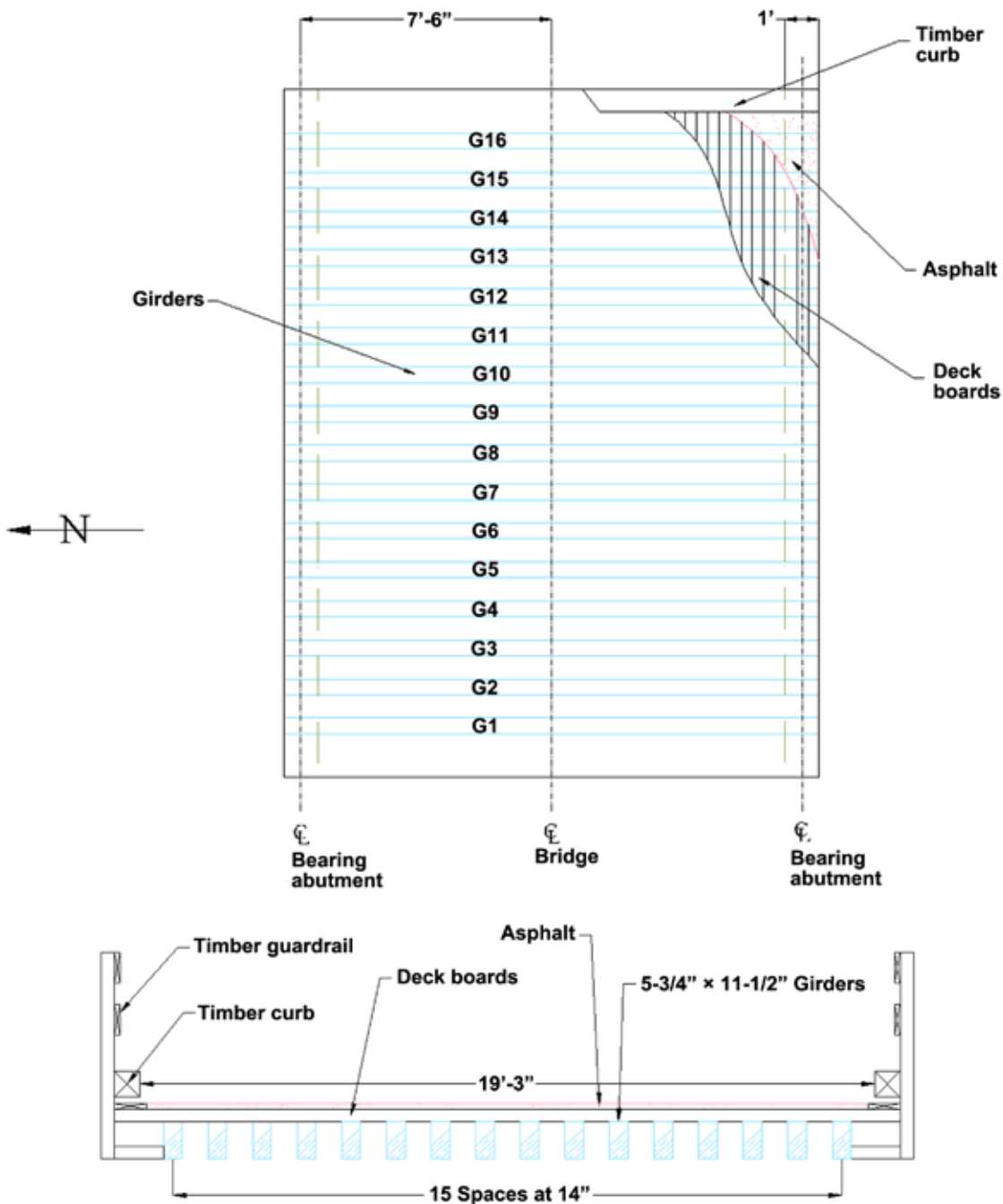


Figure F4. Plan and profile layout of Madison County Bridge.

Superstructure

It was noted in the NC–DOT 2004 report that the bridge superstructure was recently rehabilitated. This was evident to the ISU research team. The superstructure appeared in overall good condition, although some very minor areas of water seepage were observed. A more notable issue is the checking seen in some of the bridge girders, though the checks are not considered severe. This checking typically was along the centerline of the girder and a typical case is shown in Figure F16. The girder bearing on the sill was sufficient and there is no misalignment.

Overall Structure

The overall structure is in good condition, and structurally the bridge is sound. No odor like anise or wintergreen, signifying fungal growth, was present. There was no evidence of insect, mechanical, or ultraviolet degradation. The transverse cracking throughout the wearing surface was the issue most noteworthy as this can lead to future problems and possibly promote decay in other elements of the bridge.

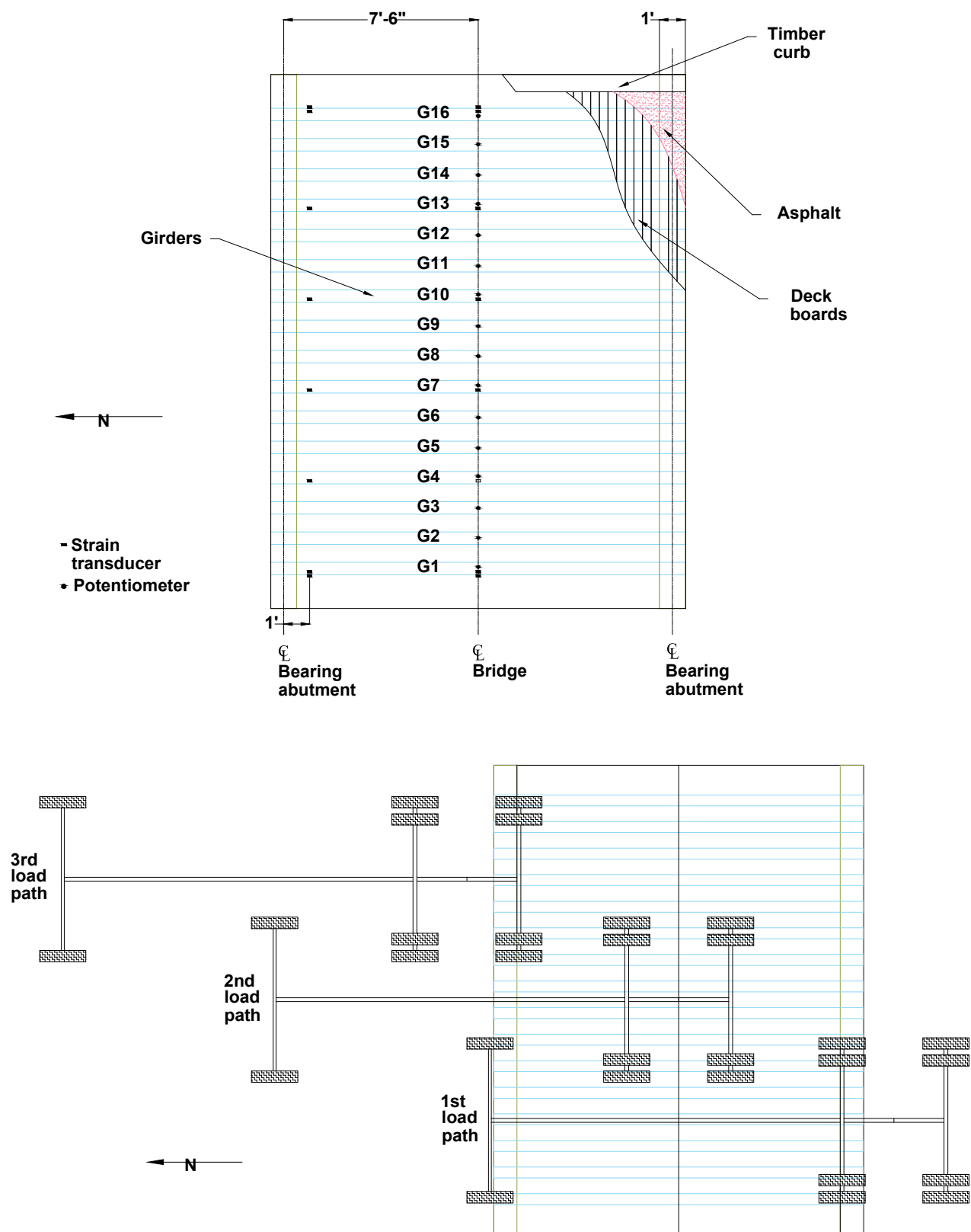


Figure F5. Instrumentation and load paths of Madison County Bridge.



Figure F6. Typical deflection instrumentation.



Figure F7. Outside girder strain transducer setup.



Figure F8. Truck Configuration and Axle Loads

Results

The following presents the results of the static load testing of the Madison County Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum



Figure F9. Tandem-axle load truck.

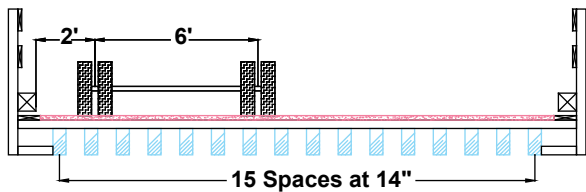


Figure F10. Transverse truck position—load path 1.

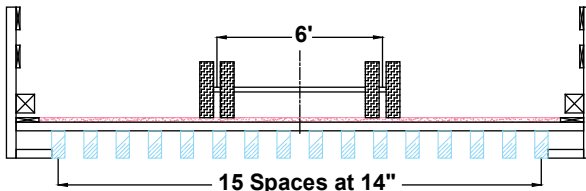


Figure F11. Transverse truck position—load path 2.

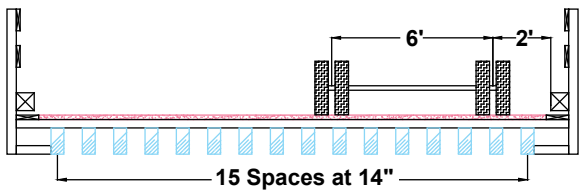


Figure F12. Transverse truck position—load path 3.

differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures F17 to F19 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is



Figure F13. Presence of transverse cracks.



Figure F16. Checking at girder centerline.



Figure F14. Transverse cracks at bridge approach.



Figure F15. Deck board detachment.

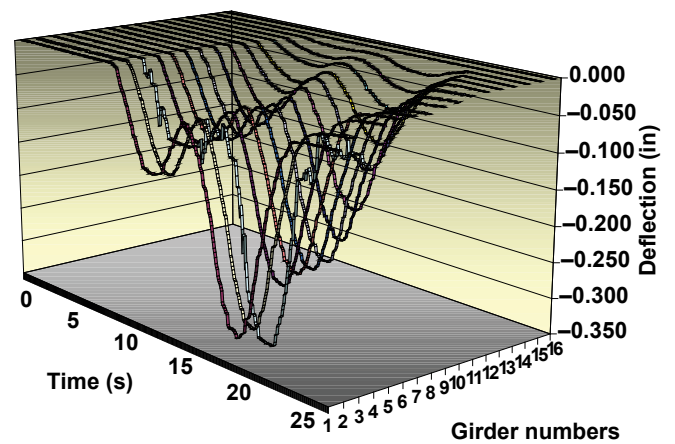


Figure F17. Deflections for Load Path 1

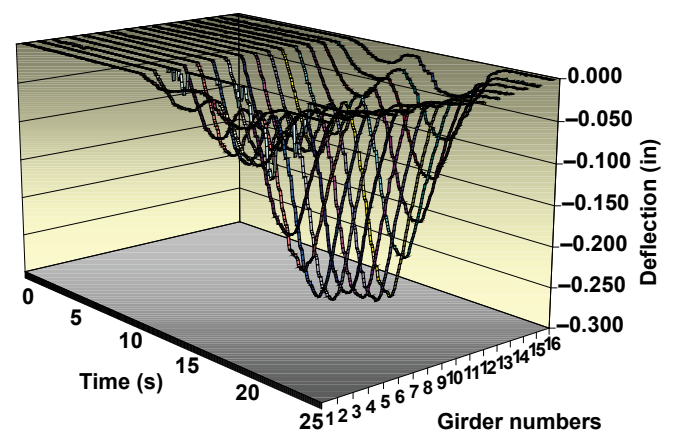


Figure F18. Deflections for Load Path 2

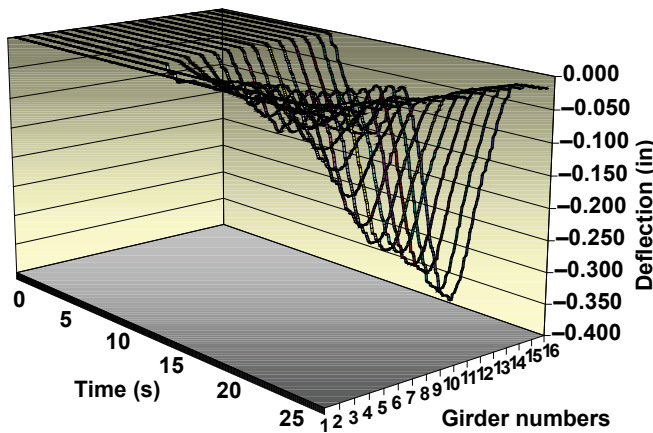


Figure F19. Deflections for Load Path 3

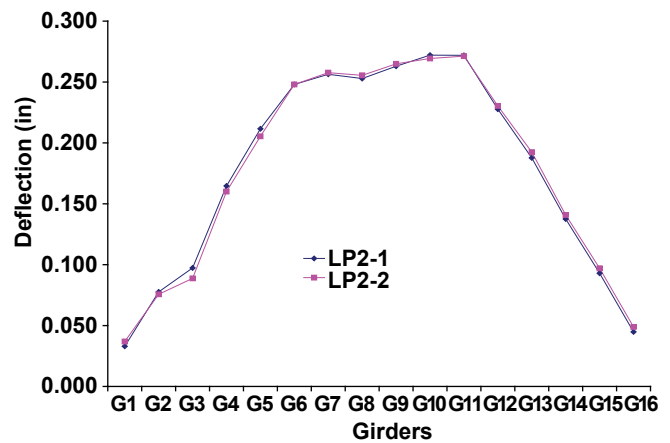


Figure F21. Maximum deflections for load path 2.

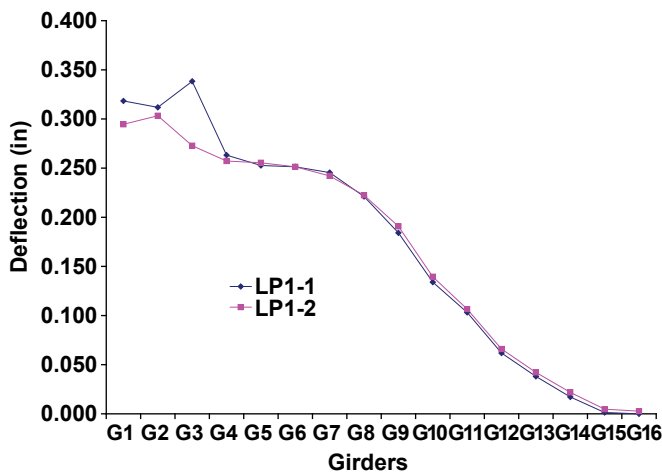


Figure F20. Maximum deflections for load path 1.

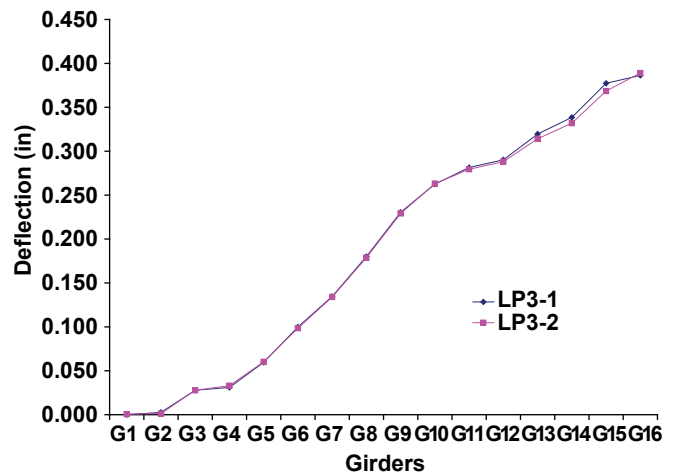


Figure F22. Maximum deflections for load path 3.

opposed to the loading patterns of longer bridges where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures F17 to F19, and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table F1. Each passing of the three load paths is illustrated in Figures F20 to F22. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n

(Table B1). Table F2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths is 322. This value is less than the minimum recommended value for timber girders. Values for the other two load paths exceed at least one of the recommended live load deflection limits stated in Table F2. The possible reasons for deflections greater than those recommended will be discussed later.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure F23 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed

Table F1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.338
Load path 2	0.272
Load path 3	0.389

Table F2—Most critical *n* value

	<i>n</i> value for girder of most deflection for each load path
Load path 1	379
Load path 2	471
Load path 3	322

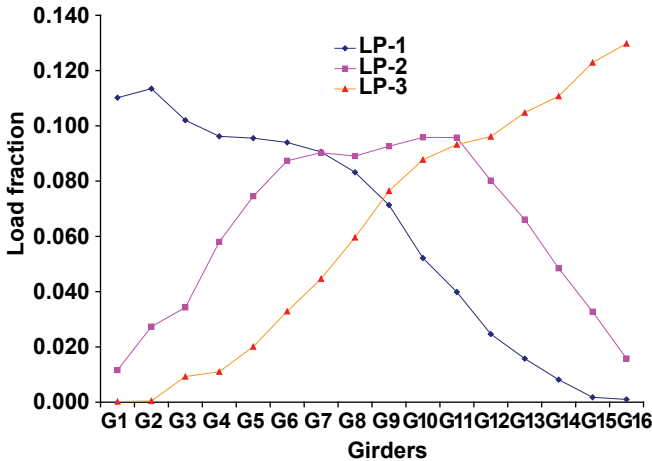


Figure F24. AASHTO design live load distribution.

equal to that of the interior lanes. Shown in Figure F24 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures F25 to F27 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table F3.

The maximum differential deflection of 0.067 in. occurs in load path 2. This is nearly 25% of the maximum deflection

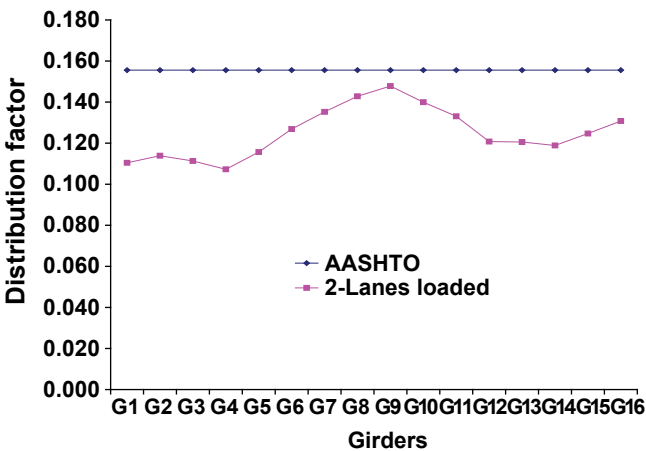


Figure F25. Differential deflections for load path 1.

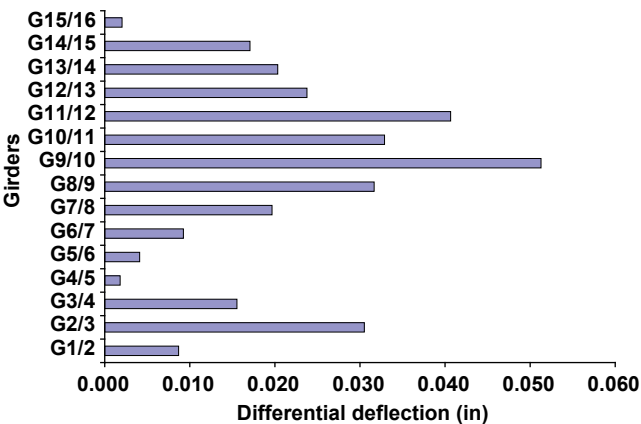


Figure F26. Differential deflections for load path 2.

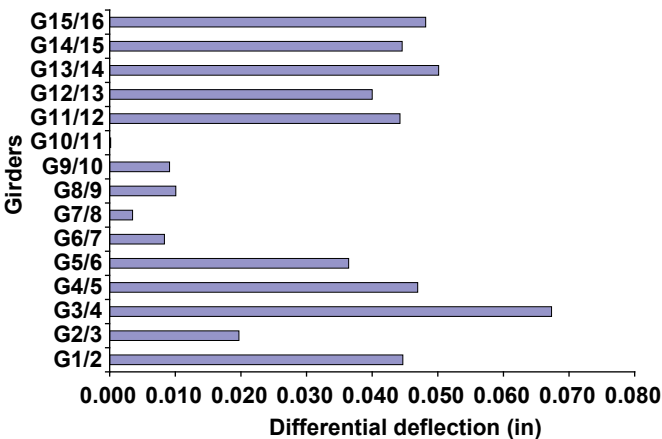
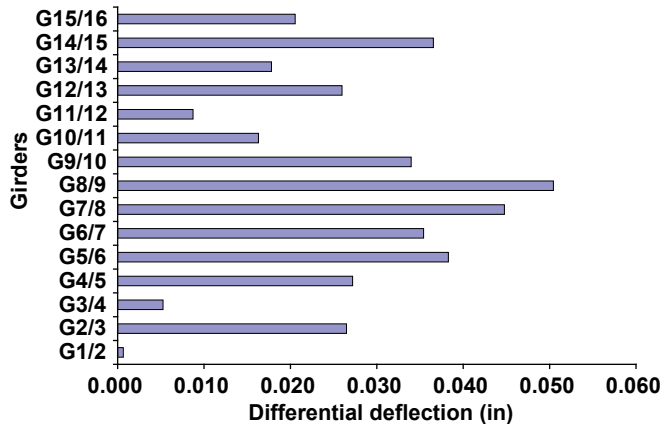
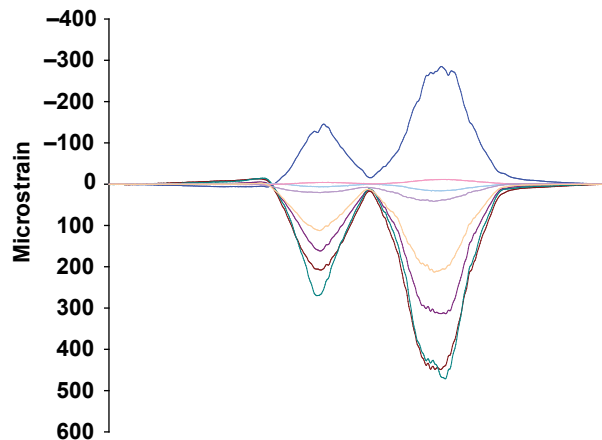


Figure F27. Differential deflections for load path 3.

resulting from that load path and is equal to 0.057 in. of deflection per ft of girder spacing. These results are consistent with other differential deflection results of similar bridges. The same is true for load paths 1 and 3 as the maximum differential deflections are both around 0.05 in. Among

Table F3—Maximum differential deflection

	Maximum differential deflection at midspan between adjacent girders (in.)
Load path 1	0.051
Load path 2	0.067
Load path 3	0.050

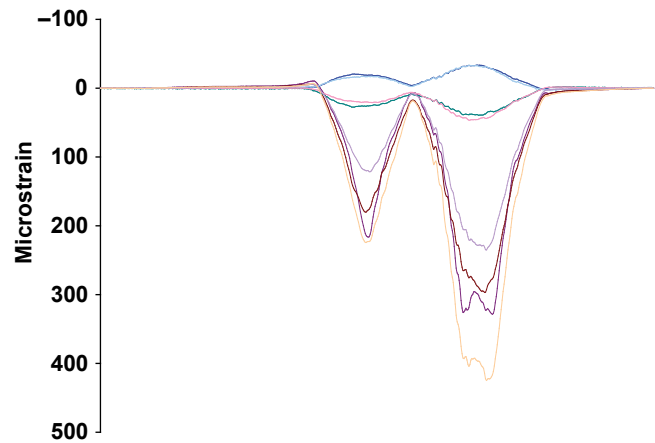
**Figure F28. Strain at midspan for load path 1.****Figure F29. Strain at midspan for load path 2.**

potential reasons for large differential deflections, the possibility exists that the load is not well distributed transversely between the two respective girders or the assumption that both girders are of equal stiffness is false.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the

**Figure F30. Strain at midspan for load path 3.****Table F4—Moisture content reading summary**

Location	Moisture content value (%)
Girder 1, north abutment	18.1
Girder 1, midspan	17.6
Girder 8, north abutment	17.2
Girder 8, midspan	17.0
Girder 16, north abutment	29.8
Girder 16, midspan	24.0
Bottom of deck between girders 1 and 2	15.1
Bottom of deck between girders 8 and 9	23.1
Bottom of deck between girders 15 and 16	17.1

girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of and 285 and 471 microstrain, respectively. The strain plot at midspan is shown in Figures F28 to F30 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It should be repeated that only the two outside girders were equipped with strain transducers in the compressive zones. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures F28 to F30 also illustrate the proportion about the neutral axis at midspan for load paths 1 and 3. The proportional pattern of the data signifies that there is very little if any composite action with the deck; that is, the girders act independently of the deck when subjected to bending.

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken

at the bottom of girders 1, 8, and 16 at midspan and north abutment. The bottom of the deck between girders 1 and 2, 8 and 9, and 15 and 16 was measured at midspan. Measurements ranged from 15.1% to 29.8%. The moisture content measurements are summarized in Table F4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength, because timber strength is not critically affected until deflections become excessive. However, at least one of the load paths included girder deflections that exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability. The deflections from the other two load paths were at least within the recommended limits of one or more sources.

Exceeding the live load deflection recommendations can have adverse affects on structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produces a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded as asphalt has very little fatigue resistance. Numerous problems associated with cracking exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. Conditions are not ideal for seepage to quickly evaporate as western North Carolina typically has a very humid climate. As a result, any water seepage through the deck will be prone to permeate the girders.

Through visual inspection, transverse cracks in the wearing surface were found. Deflections exceeding the recommended live load limit state would suggest that the wearing surface may show transverse cracking. The wearing surface of this particular bridge is in satisfactory condition, though close attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This

bridge was very near though just over that limit. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limit their use for primary analysis. Nevertheless, Figures F28 to F30 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 285 and 471 microstrain, respectively. These strains equate to maximum stresses of 328 and 542 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 430 and 711 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the humid climate in North Carolina, higher moisture contents were expected and also found. Girder 16 showed higher moisture contents than the other girders. This could be a sign that girder 16 is being exposed to more moisture than the other elements of the bridge. The amount of water present in wood can modify its physical properties. With increasing moisture content, the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

Conclusions

Several methods of condition and performance investigation were performed on the Madison County Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of severe degradation. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

The transverse cracks in the wearing surface and the effects thereof should be monitored with future inspections. Numerous transverse cracks in the wearing surface were observed. Some minor seepage through the wearing surface and into the deck boards and girders was also evident.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. All of the deflection values at midspan, however, failed to meet recommended values.

Appendix G—North Carolina Bridge No. 430351, Dix Creek Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Abstract

The Dix Creek Bridge is a single-span timber girder bridge with a bituminous wearing surface located in Haywood County, North Carolina. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: Henry Black, North Carolina Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing; Chris Lee, North Carolina Department of Transportation employee, who organized the load testing; Dean Smith, North Carolina Department of Transportation employee, who assisted during the load testing; Jeremy Turner, North Carolina Department of Transportation employee, who assisted during the load testing; and Rick Arrington, North Carolina Department of Transportation employee, who assisted during the load testing.

Background

The location of North Carolina state bridge number 430351, hereinafter referred to as the Dix Creek Bridge, is shown in Figure G1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Dix Creek Bridge was built in 1957 and is located in Haywood County in western North Carolina, 100 ft west of



Figure G1. Dix Creek Bridge in North Carolina.



Figure G2. Dix Creek Bridge substructure.

junction SR1106 across Dix Creek. SR1107 is carried by the structure. Currently, the bridge is posted for 11 tons (single vehicle) and 20 tons (type S3 truck).

Bridge Description

The Dix Creek Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface. The bridge length measures 17 ft–9 in. from the west backwall to the east backwall. The bridge width measures 17 ft–1 in. from inside of curb to inside of curb and 18 ft–0 in. from outside to outside of deck. The substructure consists of solid timber posts and sills seated on concrete (see Fig. G2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by timber blocks and bolts into the exterior girders along with bolts into the curb, which is seated and bolted to the top of the deck, as shown in Figure G3.

Girders measure 17 ft–9 in. from end to end and have a clear span of 16 ft–1 in. A total of nine girders, spaced 24-3/4 in. center-to-center, measuring 5-3/4 by 11-1/2 in. in cross-section are present and are seated and toe-nailed to the 10- by 10-in. timber sills with spikes. The deck consists of individual 4- by 8-in. nominal boards laid transverse to the



Figure G3. Dix Creek parapet.

longitudinal girder direction, which are fastened to the girders with spikes. Overlaying the deck is a 2-in.-thick layer of asphalt wearing surface. Figure G4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure G5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the nine girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure G6.

Strain transducers were attached to girder numbers 1, 3, 5, 7, and 9, with 1 being the outside girder on the north side of the bridge and 9 being the outside girder on the south side of the bridge. The midspan and one abutment were instrumented (see Fig. G5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder (see Fig. G7). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Static Loading

Static loading of the bridge was completed using a tandem axle dump truck provided by the North Carolina Department of Transportation–Division 14. Dimensions of the truck are shown in Figure G8. The rear wheel base was 6 ft–0 in.; the

distance between the hubs of the two rear axles measured 4 ft–6 in.; the distance between the forward-most rear axle and the front axle hubs measured 14 ft–11 in. The weight of the vehicle was 53,320 lb, and it is typical that 70% of the weight of a loaded tandem-axle truck is distributed to the rear axles. Using this assumption, the total weight on each rear axle and the front axle may be 18,662 lb and 15,996 lb, respectively. Figure G9 shows the truck used for load testing.

Three load paths were considered when testing the bridge. Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the right wheel line of the truck was driven 2 ft from the inside of the north curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the left wheel line of the truck was driven 2 ft from the inside of the south curb. For all load paths, the dump truck was driven at a crawl speed from west to east and multiple passes were made on each path to ensure the collected data were repeatable. Figures G10 to G12 illustrate each load path.

Condition Assessment

The report for the bridge inspection conducted on July 7, 2005, was obtained from the North Carolina DOT (NC–DOT). This report was reviewed and certain portions are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

Transverse cracks were observed in the asphalt pavement at the floor board seams and are shown in Figure G13. Though determined to be relatively minor, these cracks should be noted. Overall, the asphalt pavement generally looked to be in satisfactory condition. At the transitions between the approach and the bridge wearing surface, transverse cracking begins, though the transition between the roadway and asphalt does not appear to be problematic for the bridge. An uneven transition could subject the bridge to unnecessary effects from dynamic loads, although slow vehicle speeds on this roadway make this unlikely. In addition, a significant amount of debris had collected on top of the bridge near the curbs, which could hinder drainage and promote seepage through the wearing surface to the decking and superstructure.

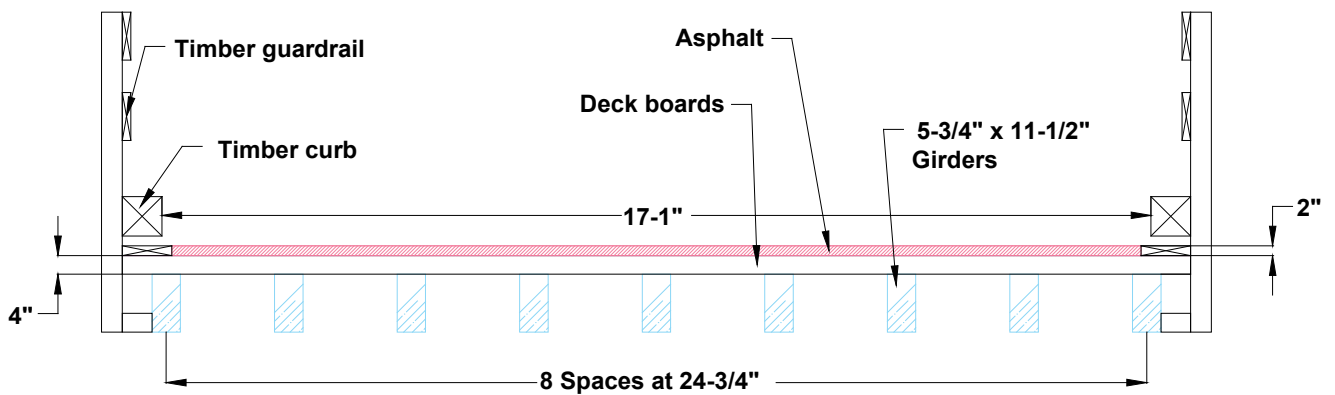
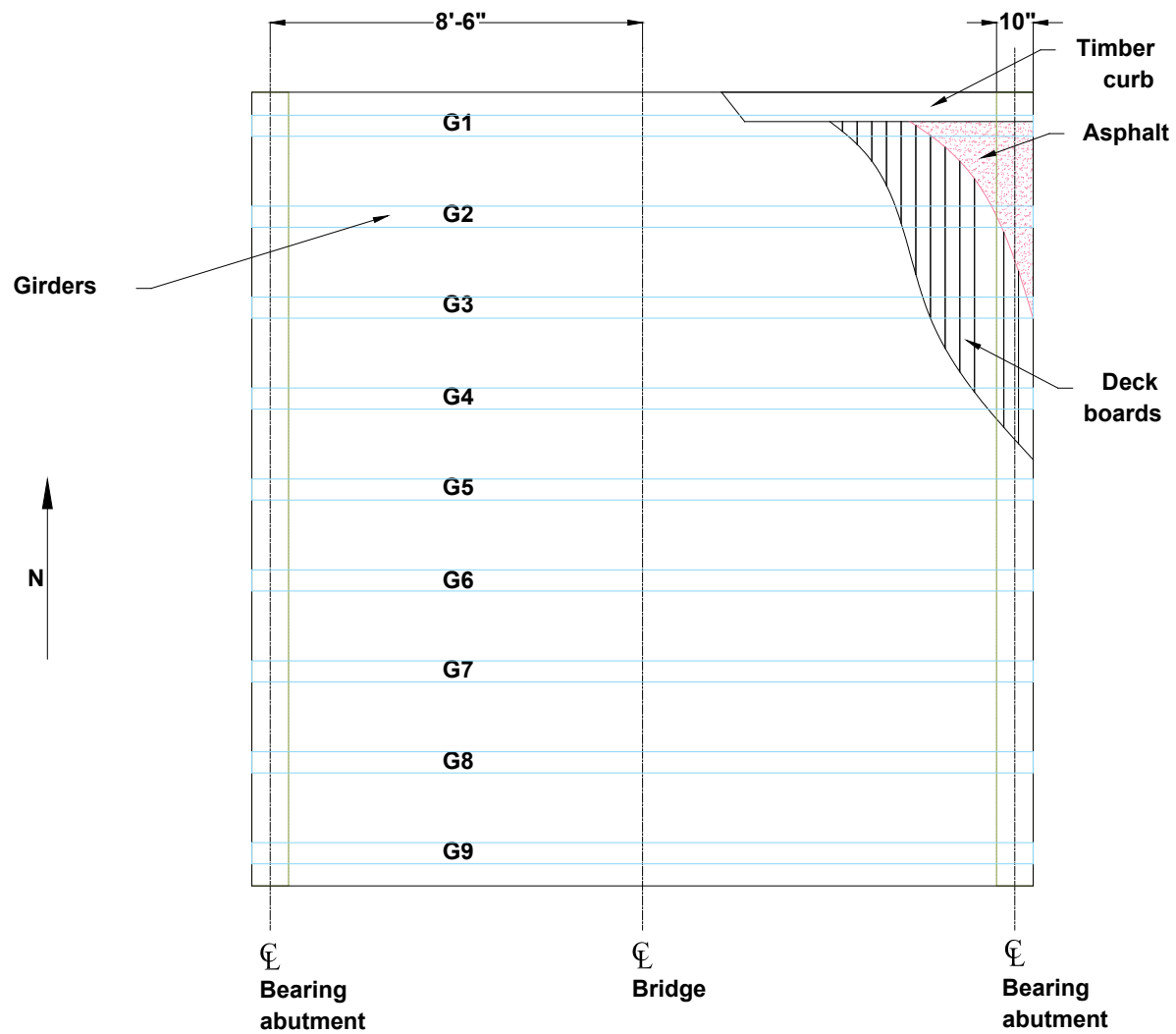


Figure G4. Plan and profile layout of Dix Creek Bridge.

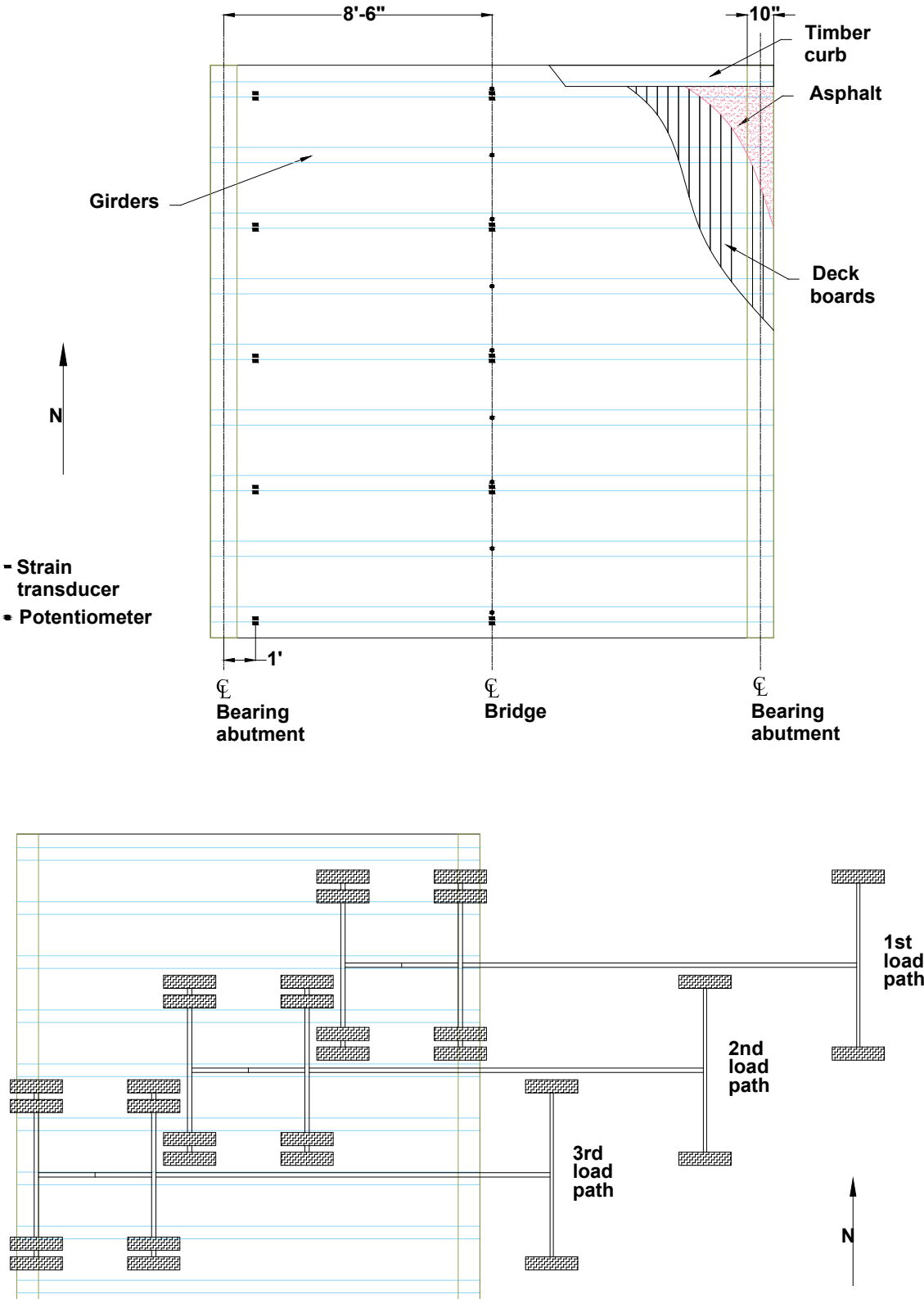


Figure G5. Instrumentation and load paths of Dix Creek Bridge.



Figure G6. Deflection instrumentation.



Figure G7. Strain transducers.

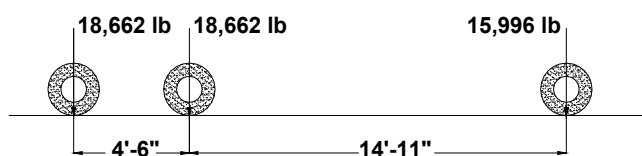


Figure G8. Truck configuration and axle loads.

Deck

The deck appeared to be in good condition; there was no visible detachment of the deck boards from the girders, and all deck boards were securely fastened. Minor water staining from seepage through the wearing surface was present throughout, though there were no signs of imminent decay.



Figure G9. Tandem-axle load truck.

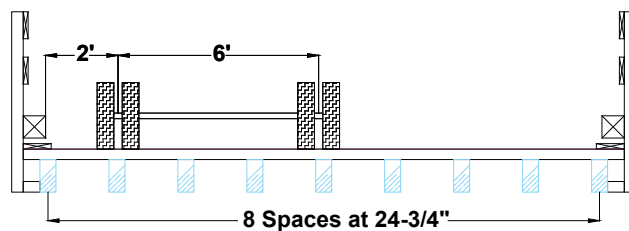


Figure G10. Transverse truck position—load path 1.

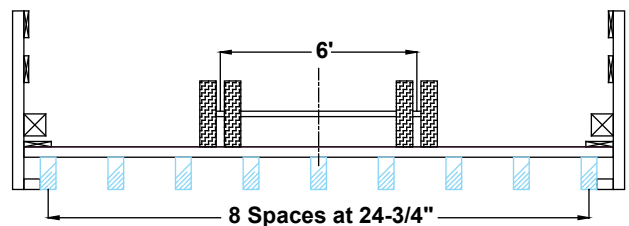


Figure G11. Transverse truck position—load path 2.

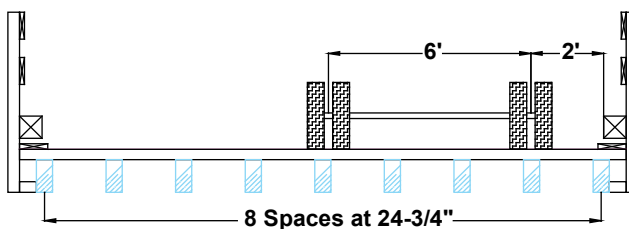


Figure G12. Transverse truck position—load path 3.

Superstructure

In the NC-DOT 2005 report, the only decay noted was some scattered light decay in the bulkhead timbers. This decay was verified by the ISU team during testing in 2006. Seepage through the wearing surface was permeating at



Figure G13. Transverse cracks in the wearing surface.



Figure G14. Girder end conditions.

least the girder surface at a number of locations as most of the girders showed signs of water seepage and staining throughout. Water staining appeared to be more of a problem at several girder ends. This could be the result of larger transverse cracks at the approach transition. Typical girder end conditions are shown in Figure G14.

The girder bearing on the sill was sufficient, and there is no misalignment. The only noticeable degradation is minor checking at the centerline of most of the girders. Figure G15 shows a typical case of checking found throughout the bridge.

Overall Structure

The overall structure is in satisfactory condition and structurally the bridge is sound. No odor like anise or



Figure G15. Checking at girder centerline.



Figure G16. Parapet condition.

wintergreen signifying fungal growth was present. There was no evidence of insect, mechanical, or ultraviolet degradation. Minor issues of concern include the presence of filtering at the abutments, where various locations on the sill and backwalls were very wet. There were minor checks in the parapet and parapet curb (see Fig. G16).

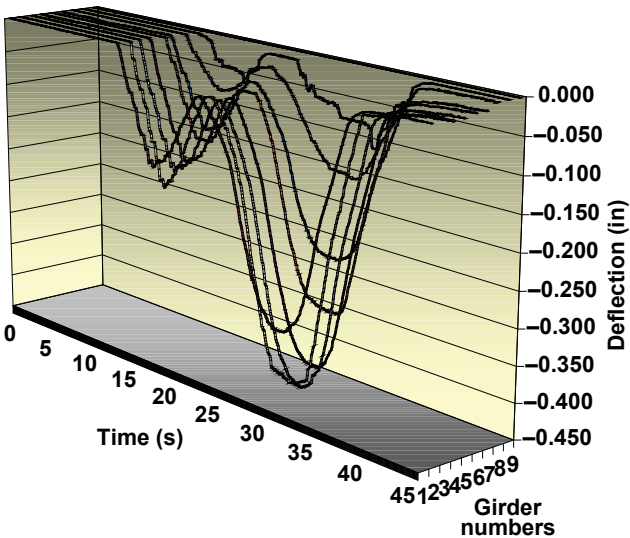


Figure G17. Deflections load path 1.

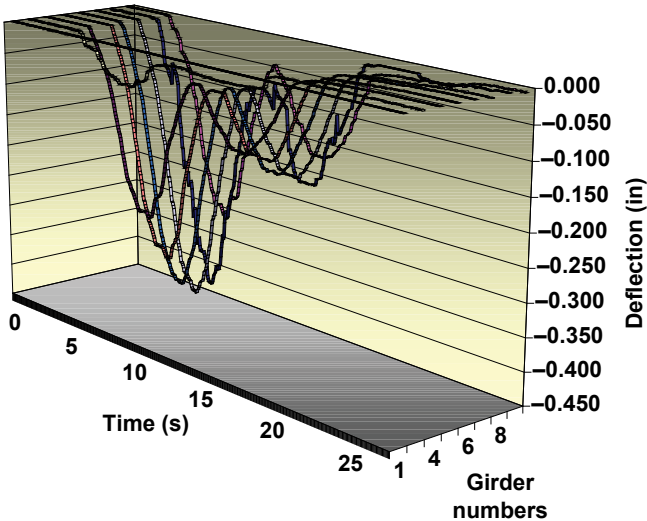


Figure G20. Maximum deflections for load path 1.

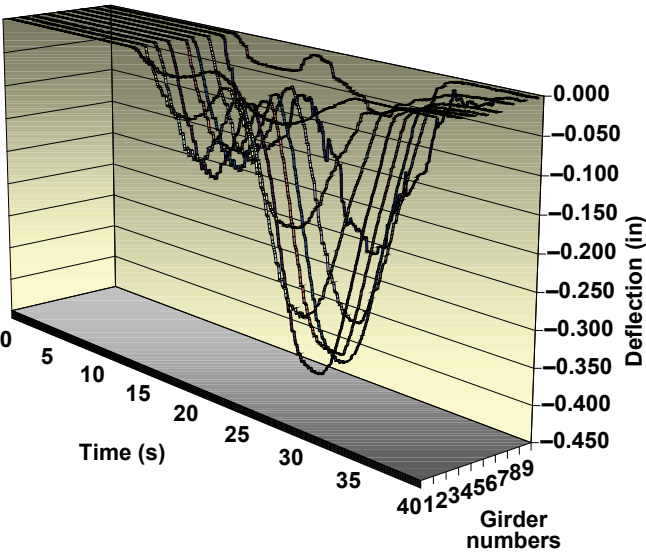


Figure G18. Deflections load path 2.

Table G1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.446
Load path 2	0.406
Load path 3	0.456

the truck one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures G17 to G19 and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table G1. Each passing of the three load paths is illustrated in Figures G20 to G22. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n

Results

The following presents the results of the static load testing of the Dix Creek Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures G17 to G19 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of

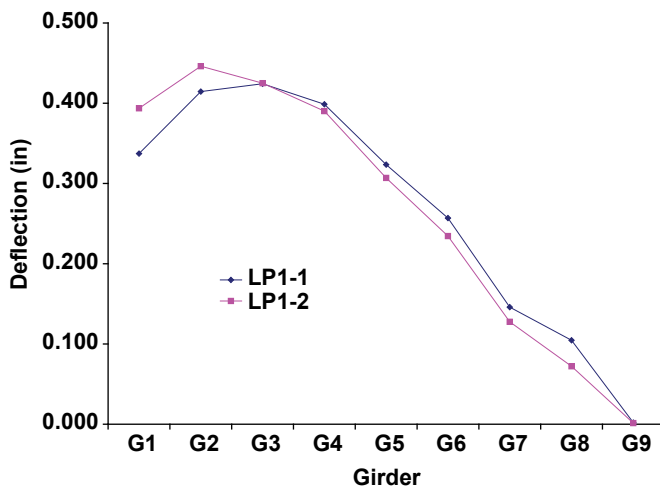


Figure G21. Maximum deflections for load path 2.

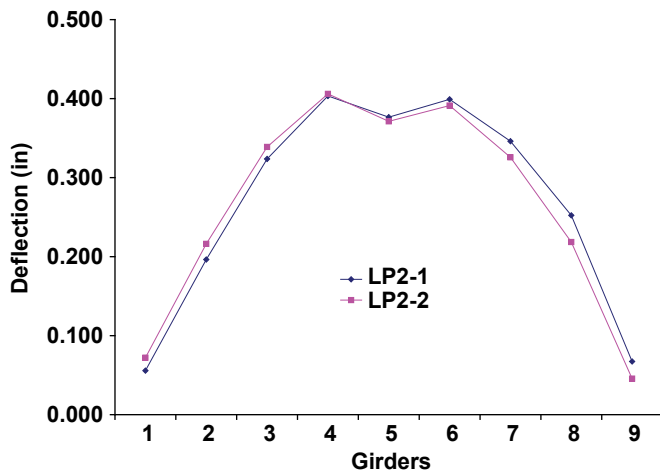


Figure G22. Maximum deflections for load path 3.

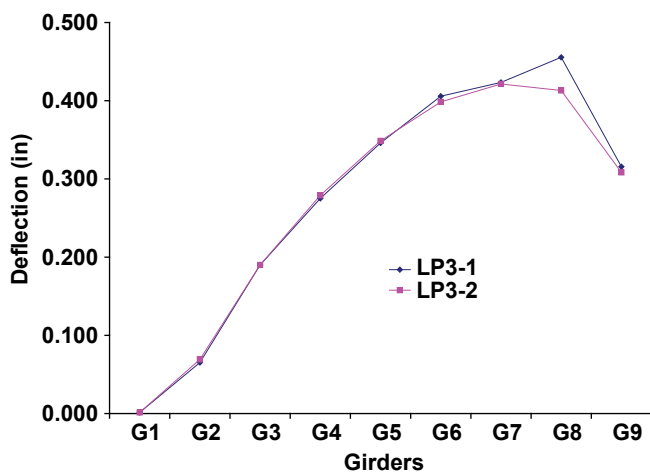


Figure G23. Load fractions for each load path.

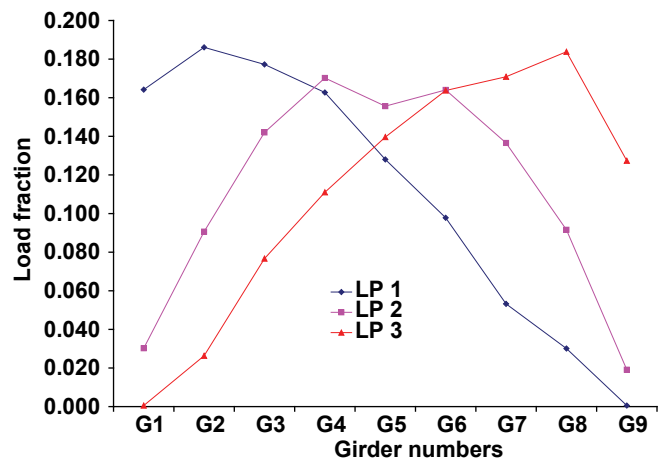


Figure G24. AASHTO design live load distribution.

(Table B1). Table G2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths is 316. This value is less than the minimum recommended value for timber girders. In fact, all the n values are below the recommended n values stated in Table G2. The possible reasons for deflections greater than those recommended will be discussed later.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure G23 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure G24 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures G25 to G27 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table G3.

The maximum differential deflection of 0.185 in. occurs in load path 2. This is nearly 46% of the maximum deflection resulting from that load path and 0.090 in. per ft of girder spacing. Among other potential reasons for large differential deflections, the possibility exists that the load is not well

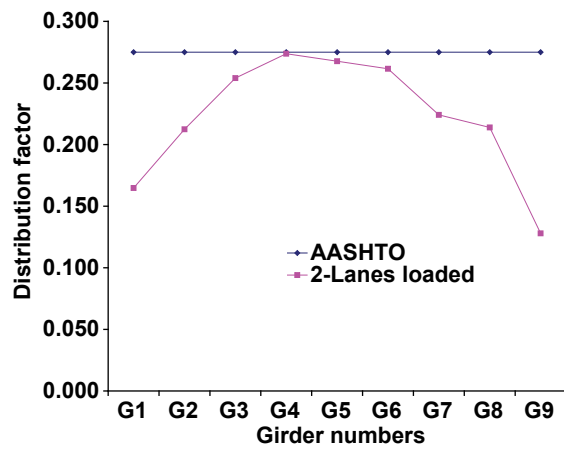


Figure G25. Differential deflections for load path 1.

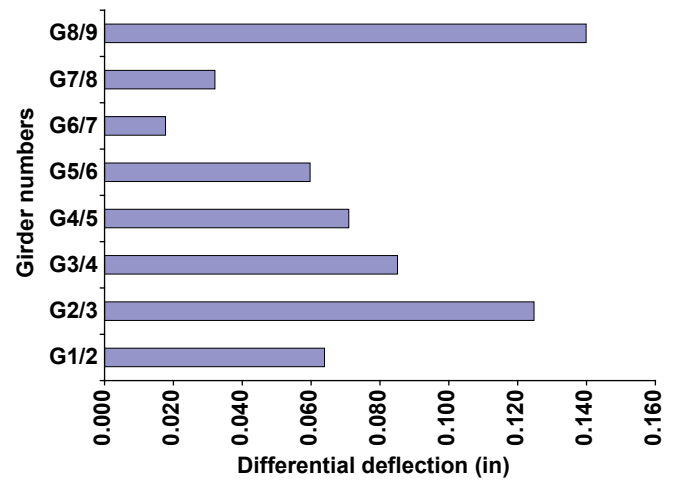


Figure G28. Strain at midspan for load path 1.

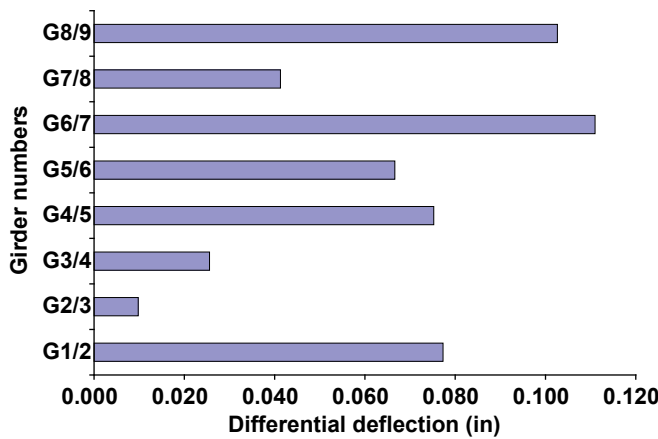


Figure G26. Differential deflections for load path 2.

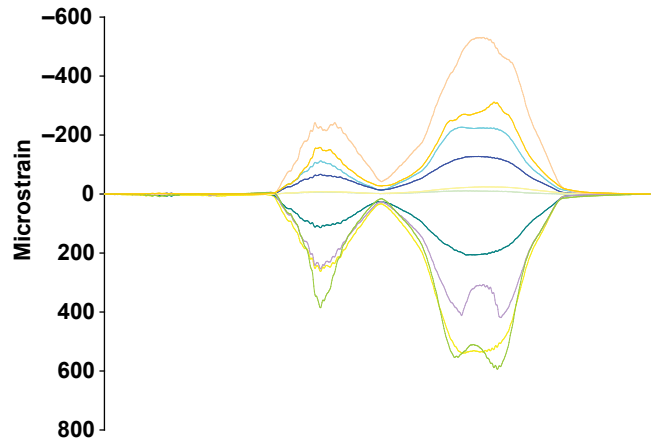


Figure G29. Strain at midspan for load path 2.

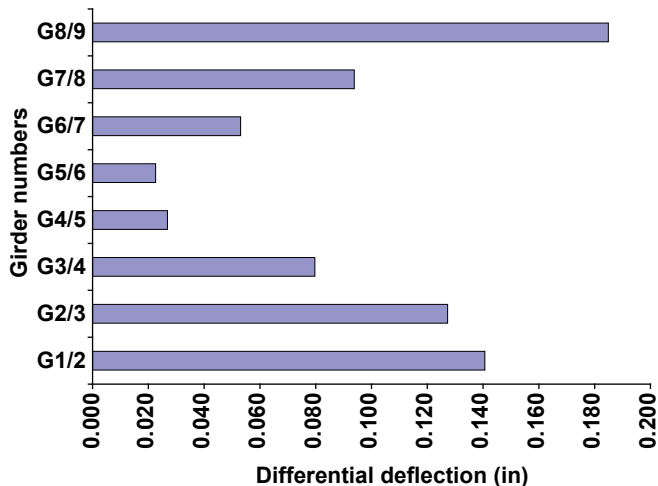


Figure G27. Differential deflections for load path 3.

distributed transversely between these two girders or the assumption that both girders are of equal stiffness is false. The same is true for load paths 1 and 3 as the maximum differential deflections are both greater than 0.1 in.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 530 and 636 microstrain, respectively. The strain plot at midspan is shown in Figures G28 to G30 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during

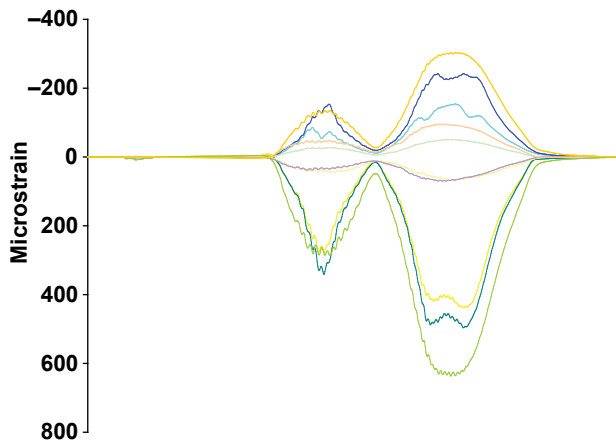


Figure G30. Strain at midspan for load path 3.

Table G4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, west abutment	21.9
Girder 1, midspan	20.5
Girder 5, west abutment	24.0
Girder 5, midspan	23.1
Girder 9, west abutment	19.0
Girder 9, midspan	23.0
Bottom of deck between girders 1 and 2	18.2
Bottom of deck between girders 5 and 6	26.0
Bottom of deck between girders 8 and 9	30 +

loading; therefore, a direct relationship exists between stress and strain, and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures G28 to G30 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck; that is, the girders act independently of the deck when subjected to bending.

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 5, and 9 at midspan and at the west abutment. The bottom of the deck between girders 1 and 2, 5 and 6, and 8 and 9 was measured at midspan. Measurements ranged from 18.2% to greater than 30%. Overall, significant moisture content differences were not found throughout the bridge except between girders 8 and 9. The moisture content measurements are summarized in Table G4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength, because timber strength is not critically affected until deflections become excessive. However, the girder deflections do exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability.

Exceeding the live load deflection recommendations can have adverse affects on, but not limited to, the structure fasteners, wearing surface, and aesthetics. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produces a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded as asphalt has very little fatigue resistance. Numerous problems associated with cracking exist including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. Conditions are not ideal for seepage to quickly evaporate as western North Carolina typically has a very humid climate. As a result, any water seepage through the deck will be prone to permeate the girders.

Through visual inspection, transverse cracks in the wearing surface were found. Deflections exceeding the recommended live load limit state would suggest that the wearing surface may show transverse cracking. The wearing surface of this particular bridge is in satisfactory condition, though close attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore, a defined limit does not exist. Even so, the authors of this report having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was over that limit by 0.040 in. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limit their use for primary analysis. Nevertheless, Figures G28 to G30 do show a reasonable relationship between truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 530 and 636 microstrain, respectively. These strains equate to maximum stresses of 610 and 731 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 817 and 980 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in² depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the humid climate in North Carolina, higher moisture contents were expected and also found. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

The moisture content measurements were all within a couple percentage points of one another except for one measurement. This shows that none of the tested areas are subjected to vastly different amounts of moisture. The single measurement that was of greater moisture content could be disregarded as it was only one of a series of measurements.

Conclusions

Several methods of condition and performance investigation were performed on the Dix Creek Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of notable concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Minor transverse cracking in the wearing surface was observed. Some seepage through the wearing surface and into the deck boards and girders was also evident. The most notable areas are at the girder ends and backwalls, where it appears more seepage than average has taken place.

Overall the bridge appeared in good condition. Minor centerline checking has occurred in a number of girders. This should be noted and observed with future inspections.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Deflection values at midspan, however, failed to meet recommended values.

Appendix H—North Carolina Bridge No. 430306, Bald Creek Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Abstract

The Bald Creek Bridge is a single-span timber girder bridge with a bituminous wearing surface located in Swain County, North Carolina. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and those without whom we would not have completed this research project. : Henry Black, North Carolina Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing; Chris Lee, North Carolina Department of Transportation employee, who organized the load testing; Dean Smith, North Carolina Department of Transportation employee, who assisted during the load testing; Jeremy Turner, North Carolina Department of Transportation employee, who assisted during the load testing; and Rick Arrington, North Carolina Department of Transportation employee, who assisted during the load testing.

Background

The location of North Carolina state bridge number 460306, hereinafter referred to as the Bald Creek Bridge, is shown in Figure H1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.



Figure H1. Bald Creek Bridge in North Carolina.



Figure H2. Bald Creek Bridge substructure.

The Bald Creek Bridge was built in 1961 and is located in Haywood County in western North Carolina, 0.1 miles east of junction SR1505 across Bald Creek. SR1506 is carried by the structure. Currently, the bridge is not posted.

Bridge Description

The Bald Creek Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface set on a 20° skew. The bridge length measures 17 ft–9 in. from the west backwall to the east backwall. The bridge width measures 19 ft–2 in. from inside of curb to inside of curb and 20 ft–11 in. from outside of rail to outside of rail. The substructure consists of solid timber posts and sills (see Fig. H2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by timber blocks and bolts into the exterior girders along with bolts into the curb, which is seated and bolted to the top of the deck. The bridge parapet is shown in Figure H3.

Girders measure 17 ft–9 in. from end to end and have a clear span of 16 ft–1 in. A total of 18 girders, spaced 13-1/2 in. center-to-center, measuring 5-3/4 by 11-1/2 in. in cross-section are present and are seated and toe-nailed to the 10- by 10-in. timber sills with spikes. The deck consists of individual 4- by 8-in. nominal boards laid transverse to the longitudinal girder direction, which are fastened to the



Figure H3. Bald Creek Bridge parapet.

girders with spikes. Overlaying the deck is a 2-in.-thick layer of asphalt wearing surface. Figure H4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure H5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 18 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure H6.

Strain transducers were attached to the bottom of girder numbers 1, 3, 5, 7, 9, 10, 12, 14, 16, and 18, with 1 being the outside girder on the north side of the bridge and 18 being the outside girder on the south side of the bridge. The midspan and one abutment were instrumented (see Fig. H5). Transducers were placed near only one abutment because of the symmetry of the bridge. Because of limited accessibility, only the two outside girders were instrumented with a strain transducer 2 in. from the top of the girder. Figure H7 shows the strain transducer setup for the outside girders. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the North Carolina Department of Transportation—Division 14. Dimensions of the truck are shown in Figure H8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–6 in.; the distance between the forward-most rear axle and the front axle hubs measured 14 ft–11 in. The weight of

the vehicle was 53,320 lb, and typically, 70% of the weight on a loaded tandem-axle truck is distributed to the rear axles. Using this assumption, the total weight on each rear axle and the front axle may be 18,662 lb and 15,996 lb, respectively. Figure H9 shows the truck used for the load testing.

Three load paths were considered when testing the bridge. Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the south curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the north curb. For all load paths, the dump truck was driven at a crawl speed from east to west and multiple passes were made on each path to ensure the collected data were repeatable. Figures H10 to H12 illustrate each load path.

Condition Assessment

The report for the bridge inspection conducted on August 31, 2005, was obtained from the North Carolina DOT (NC–DOT). This report was reviewed, and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

According to the NC–DOT 2005 report, the bridge had been recently rehabilitated. The wearing surface appeared to be in above average condition, though several small scattered transverse cracks were present at the floor board seams (see Fig. H13). Though determined to be relatively minor, these cracks were noted by the ISU team during testing in 2006. Besides this local case, the transition between the roadway and asphalt does not appear to be problematic for the bridge. An uneven transition could subject the bridge to unnecessary effects from dynamic loads even though slow vehicle speeds on this roadway make this unlikely. In addition, some debris had collected on top of the bridge which could hinder drainage and promote seepage through the wearing surface to the decking and superstructure.

Deck

The deck appeared to be in good condition, there was no visible detachment of the deck boards from the girders, and all deck boards were securely fastened. Minor water staining from seepage through the wearing surface was present throughout, though there were no signs of imminent decay.

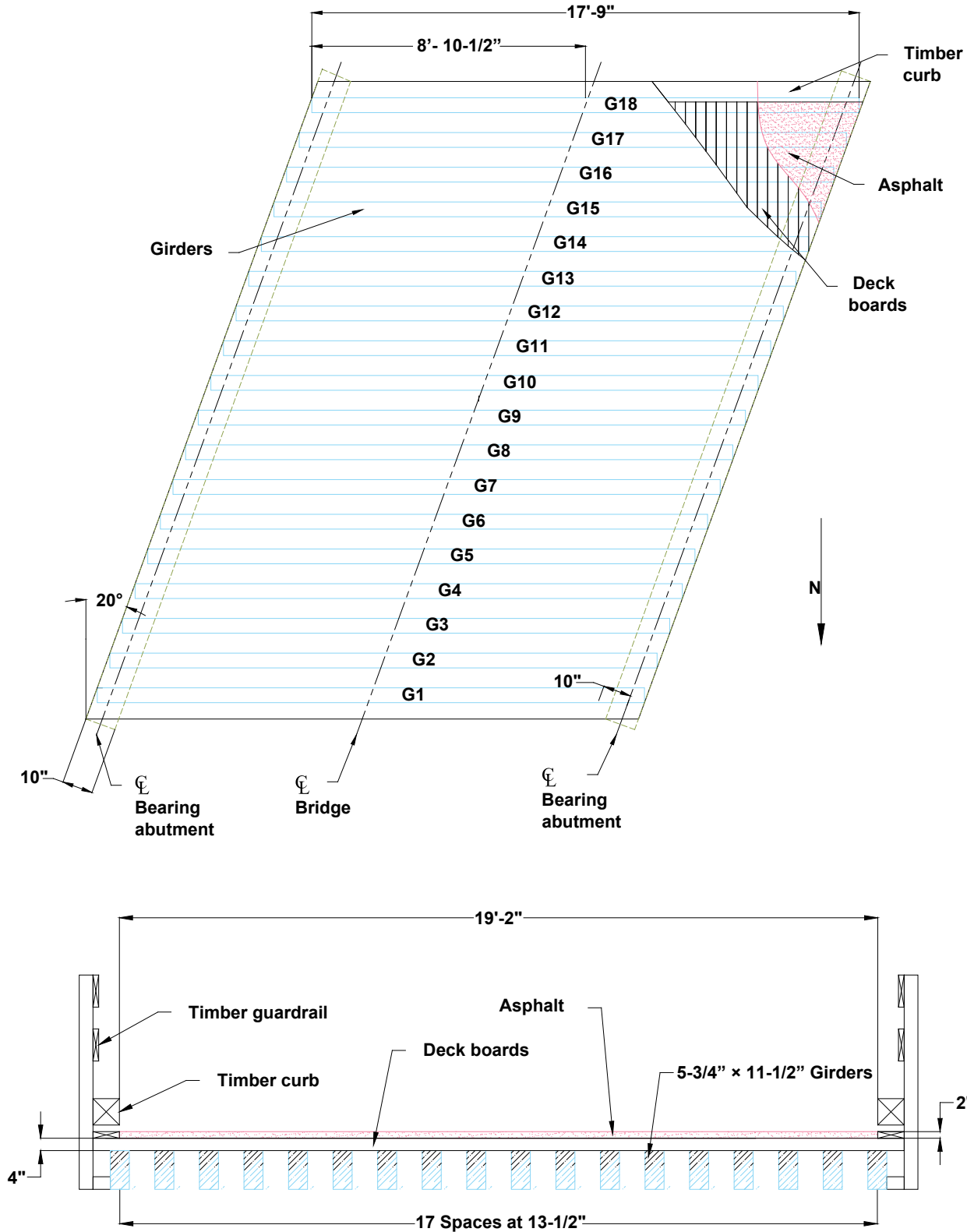


Figure H4. Plan and profile layout of Bald Creek Bridge.

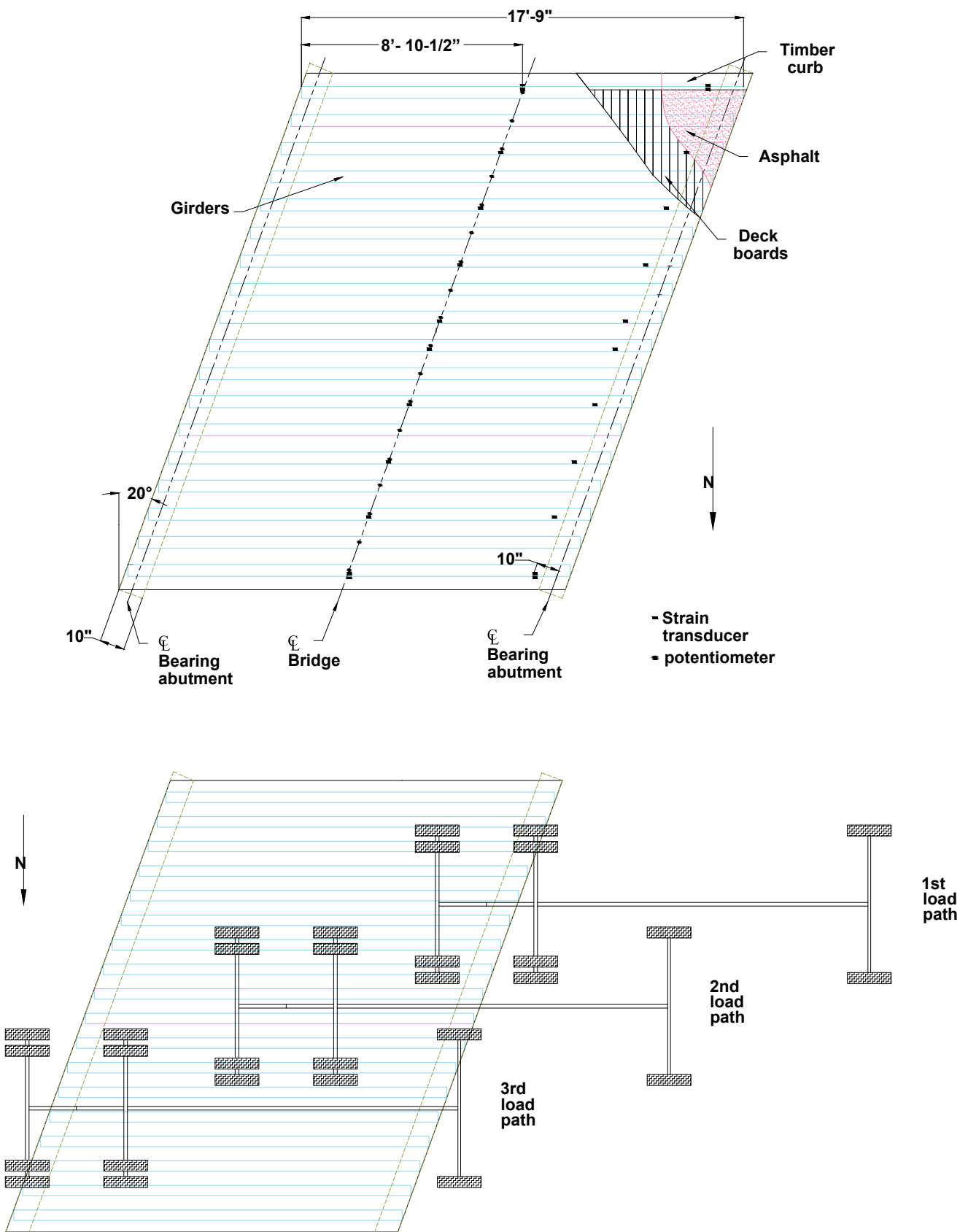


Figure H5. Instrumentation and load paths of Bald Creek Bridge.



Figure H6. Deflection instrumentation.



Figure H7. Outside girder strain transducer setup.

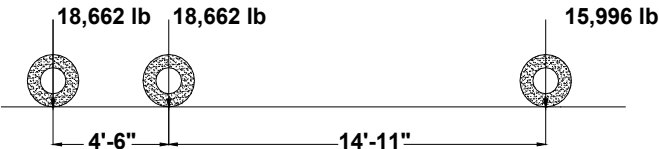


Figure H8. Truck configuration and axle loads.



Figure H9. Tandem-axle load truck.

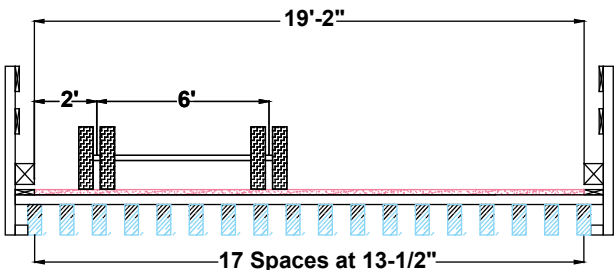


Figure H10. Transverse truck position—load path 1.

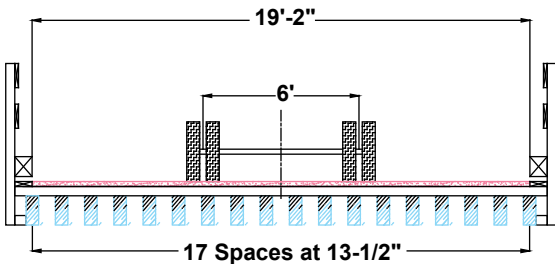


Figure H11. Transverse truck position—load path 2.

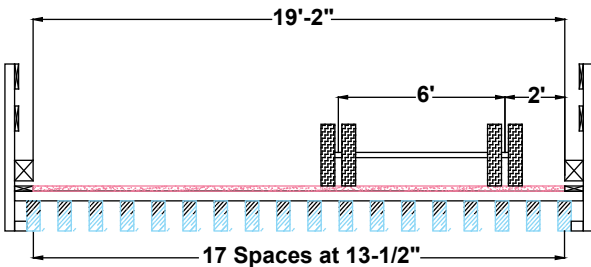


Figure H12. Transverse truck position—load path 3.



Figure H13. Minor transverse cracks.



Figure H14. Typical superstructure condition.

Superstructure

It is evident that the bridge had been recently rehabilitated as the superstructure appeared in excellent condition. Minimal evidence of moisture was present throughout the girders, though some moss growth was observed on the north face of girder number 1. Figure H14 shows the typical condition of the superstructure.

The girder bearing on the sill was sufficient, and there is no misalignment. The only noticeable degradation is some minor checking throughout the girders

Overall Structure

The overall structure is in good condition, and structurally the bridge is sound. No odor like anise or wintergreen signifying fungal growth was present. There was no evidence of insect, mechanical, or ultraviolet degradation. A minor issue of concern in addition to those already stated includes some collection of debris between the sill and girders at the abutment, which appears to be the result of nesting birds or other animals. This could be an area where water retention is higher, promoting degradation. Also, though the curbs are not structural members, advanced degradation was observed and should be noted (see Fig. H15).

Results

The following presents the results of the static load testing of the Bald Creek Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and



Figure H15. Advanced degradation throughout curbs.

the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures H16 to H18 present the time-history deflections for each girder as the truck traveled across the bridge. Given

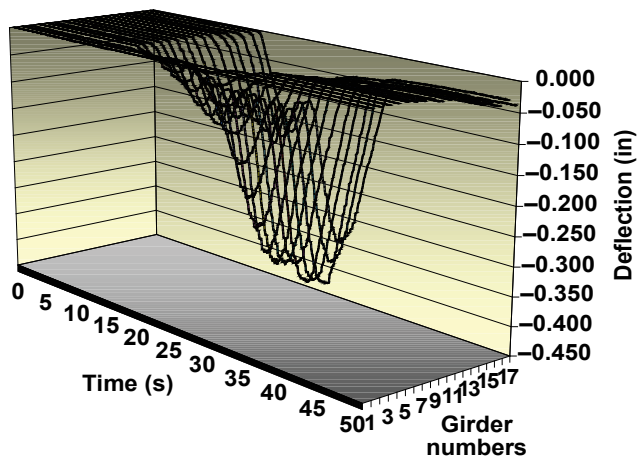


Figure H16. Deflections load path 1.

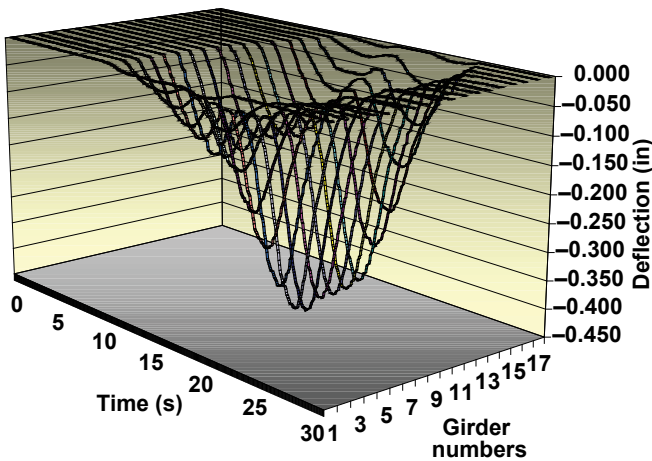


Figure H17. Deflections load path 2.

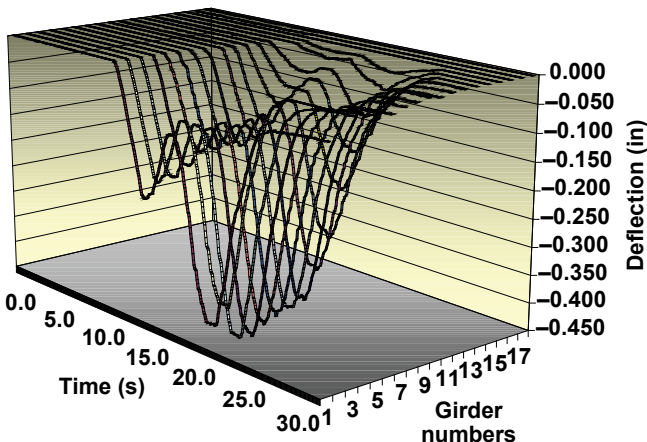


Figure H18. Deflections load path 3.

Table H1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.411
Load path 2	0.412
Load path 3	0.448

the relationship of the length of the bridge to the length of the truck one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures H16 to H18 and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table H1. Each passing of the three load paths is illustrated in Figures H19 to H21. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table H2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths is 321. This value is less than the minimum recommended value for timber girders. In fact, all the n values are below the recommended n values stated in Table H2. The possible reasons for deflections greater than those recommended will be discussed later.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure H22 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure H23 is the

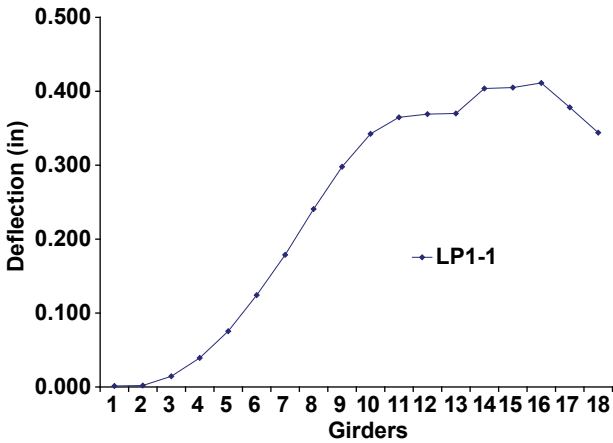


Figure H19. Maximum deflections for load path 1.

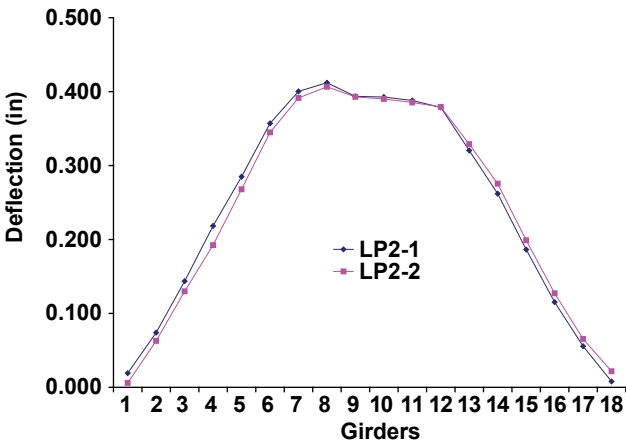


Figure H20. Maximum deflections for load path 2.

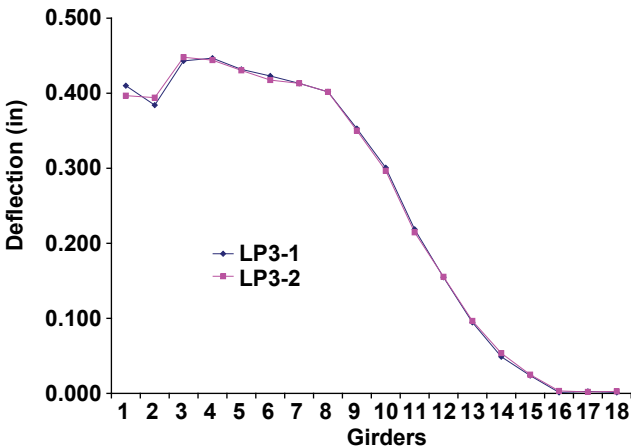


Figure H21. Maximum deflections for load path 3.

Table H2—Most critical <i>n</i> value	
	<i>n</i> value for girder of most deflection for each load path
Load path 1	349
Load path 2	349
Load path 3	321

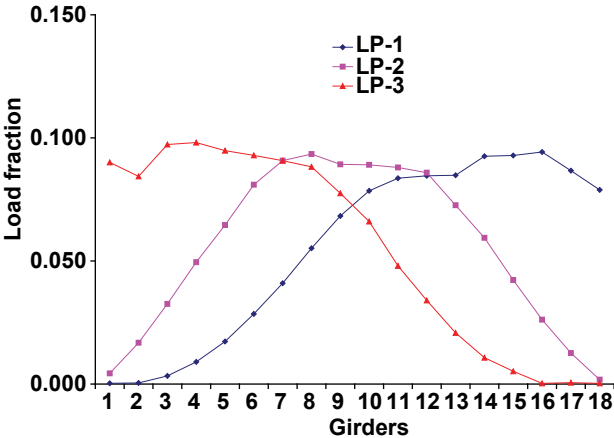


Figure H22. Load fractions for each load path.

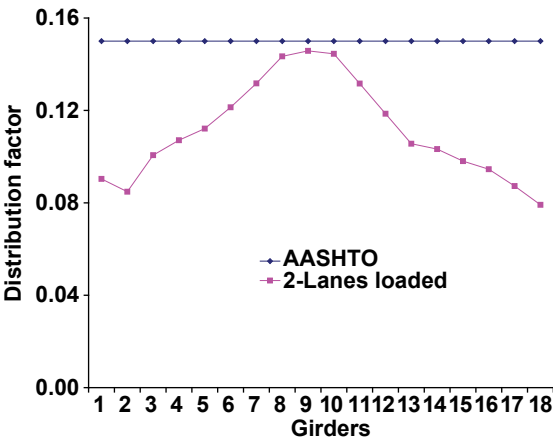


Figure H23. AASHTO design live load distribution.

comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures H24 to H26 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum

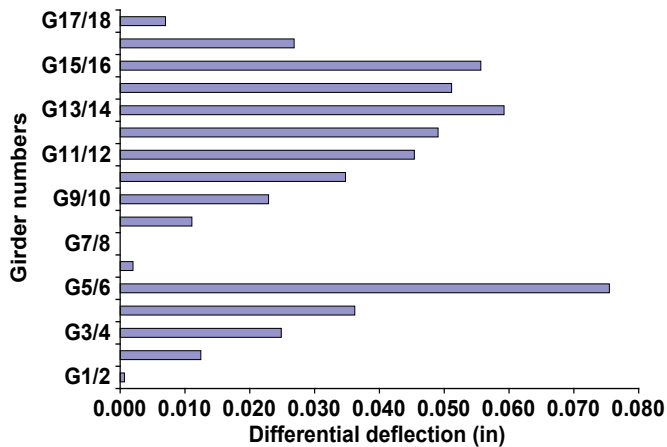


Figure H24. Differential deflections for load path 1.

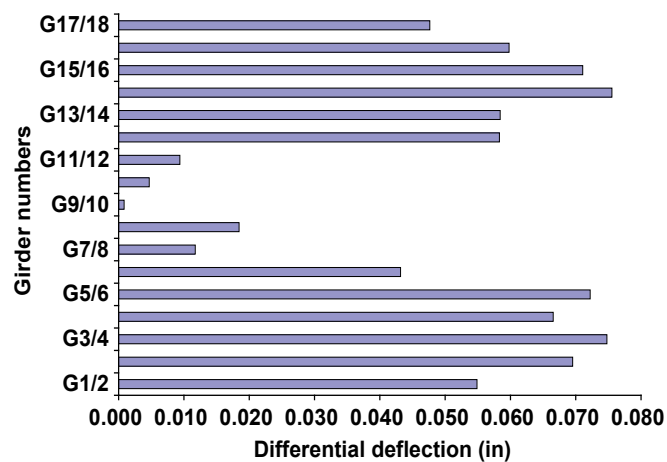


Figure H25. Differential deflections for load path 2.

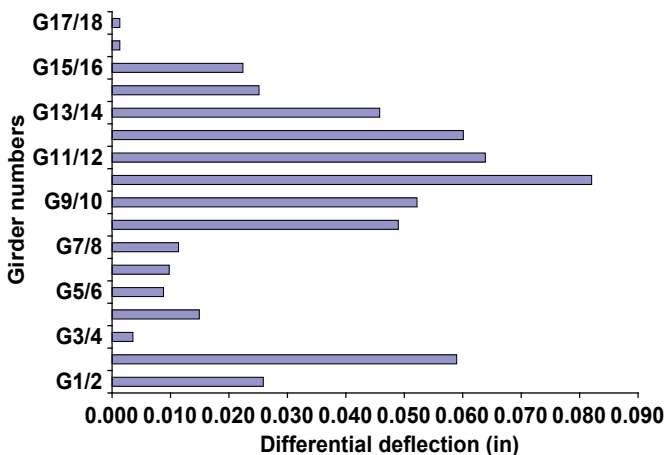


Figure H26. Differential deflections for load path 3.

Table H3—Maximum differential deflection

	Maximum differential deflection at midspan between adjacent girders (in.)
Load path 1	0.075
Load path 2	0.077
Load path 3	0.082

differential deflections between adjacent girders are presented in Table H3.

The maximum differential deflection of 0.082 in. occurs in load path 3. This is approximately 18% of the maximum deflection resulting from that load path and 0.073 in. per ft of girder spacing. Among other potential reasons for large differential deflections, the possibility exists that the load is not well distributed transversely between these two girders or the assumption that both girders are of equal stiffness is false. The same is true for load paths 1 and 2 as the maximum differential deflections are both around 0.08 in.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 631 and 648 microstrain, respectively. The strain plot at midspan is shown in Figures H27 to H29 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. One should note that only two girders were equipped with strain transducers in the compressive region. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures H27 to H29 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck; that is, the girders act independently of the deck when subjected to bending.

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 9, and 18 at midspan and at the east abutment. The bottom of the deck between girders 1 and 2,

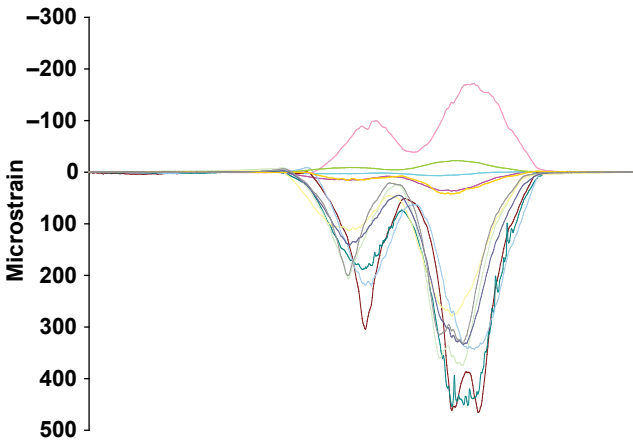


Figure H27. Strain at midspan for load path 1.

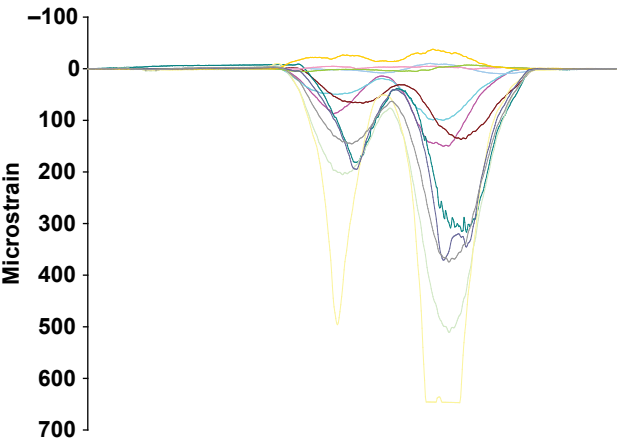


Figure H28. Strain at midspan for load path 2.

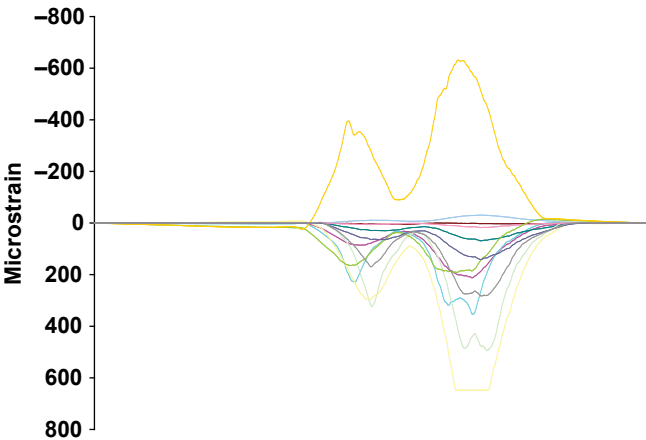


Figure H29. Strain at midspan for load path 3.

Table H4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, east abutment	22.0
Girder 1, midspan	19.0
Girder 9, east abutment	16.9
Girder 9, midspan	17.9
Girder 18, east abutment	18.2
Girder 18, midspan	17.0
Bottom of deck between girders 1 and 2	22.2
Bottom of deck between girders 9 and 10	14.5
Bottom of deck between girders 17 and 18	21.2

9 and 10, and 17 and 18 was measured at midspan. Measurements ranged from 14.5% to 22.2%. Overall, significant moisture content differences were not found throughout the girders, and the deck measurement of 14.5% was the exception. The moisture content measurements are summarized in Table H4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because timber strength is not critically affected until deflections become excessive. However, the girder deflections do exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability.

Exceeding the live load deflection recommendations can have adverse affects on structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produces a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded as asphalt has very little fatigue resistance. Numerous problems associated with cracking exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. Conditions are not ideal for seepage to quickly evaporate as western North Carolina typically has a very humid climate. As a result, any water seepage through the deck will be prone to permeate the girders.

Through visual inspection, some minor transverse cracks in the wearing surface were found. Deflections exceeding the recommended live load limit state would suggest that the wearing surface may show transverse cracking. The wearing surface of this particular bridge is in good condition, though close attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was over that limit by 0.023 in. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limit their use for primary analysis. Nevertheless, Figures H27 to H29 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 631 and 648 microstrain, respectively. These strains equate to maximum stresses of 726 and 745 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 973 and 998 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the humid climate in North Carolina, higher moisture contents were expected and also found. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

The moisture content percentages were all within a couple percentage points of one another. This shows that none of

the tested areas are subjected to vastly different amounts of moisture.

Conclusions

Several methods of condition and performance investigation were performed on the Bald Creek Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of notable concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Minor transverse cracking in the wearing surface was observed. It is evident, however, that the bridge has been recently rehabilitated as the overall wearing surface, deck, and superstructure appear in good condition. The static live load test also showed that the performance under load is consistent with similar bridges.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Deflection values at midspan, however, failed to meet recommended values.

Appendix I—North Carolina Bridge No. 990163, Doebag Creek Bridge

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Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Abstract

The Doebag Creek Bridge is a single-span timber girder bridge located in Yancey County, North Carolina. The bridge was load tested and visually assessed as part of a research project through the United States Department of Agriculture (USDA) – Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: Henry Black, North Carolina Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing; Gary Moore, North Carolina Department of Transportation employee, who organized the load testing; and Dennis Burleson, North Carolina Department of Transportation employee, who operated the load truck during testing.

Background

The location of North Carolina state bridge number 990163, hereinafter referred to as the Doebag Creek Bridge, is shown in Figure I1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Doebag Creek Bridge was built in 1960 and is located in Yancey County in western North Carolina, 0.4 miles east of junction SR1308 across Doebag Creek. SR1311 is carried



Figure I1. Bridge location.



Figure I2. Substructure sill and posts.

by the structure. Currently, the bridge is posted for 16 tons (single vehicle) and 23 tons (type S3 truck).

Bridge Description

The Doebag Creek Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface set on a 30° skew. The bridge length measures 17 ft–7 in. from the west backwall to the east backwall. The bridge width measures 16 ft–1 in. from inside of curb to inside of curb and 19 ft–3 in. from outside of rail to outside of rail. The substructure consists of solid timber posts and sills (see Fig. I2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by timber blocks and bolts into the exterior girders along with bolts into the curb, which is seated and bolted to the top of the deck, as shown in Figure I3.

Girders measure 17 ft–7 in. from end to end and have a clear span of 15 ft–11 in. A total of 10 girders and 1 helper, spaced 1 ft–9 in. center-to-center, measuring 6 by 12 in. in cross-section, are present and are seated and toe-nailed to



Figure I3. Doebag Creek Bridge parapet.

the 10- by 10-in. timber sills with spikes. The deck consists of individual 4- by 8-in. nominal boards laid transverse to the longitudinal girder direction, which are fastened to the girders with spikes. No asphalt overlay is present on the bridge. Figure I4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure I5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 10 girders and 1 helper. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured center line of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure I6.

Strain transducers were attached to girder numbers 1, 5, 8, and 10, with 1 being the outside girder on the south side of the bridge and 10 being the outside girder on the north side of the bridge. The midspan and one abutment were instrumented (see Fig. I5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder (see Fig. I7). The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the North Carolina Department of Transportation–Division 13. Dimensions of the truck are shown in Figure I8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured

4 ft–6 in.; the distance between the forward-most rear axle and the front axle hubs measured 14 ft–8 in. Exact weight of the truck was 54,420 lb, and the total weight on each rear axle and the front axle is 19,330 and 16,660 lb, respectively. Figure I9 shows the truck used for the load testing.

Three load paths were considered when testing the bridge (see Figs. I10 to I12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the south curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the north curb. For all load paths, the dump truck was driven at a crawl speed from east to west and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on July 16, 2003, was obtained from the North Carolina DOT (NC–DOT). This report was reviewed, and certain aspects are included in here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU research team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

The roadway was covered with dirt and gravel, and no asphalt wearing surface was present. The lack of wearing surface is evident by the condition of the remainder of the bridge.

Deck

According to the NC–DOT 2003 report, moderate decay in the ends of several deck boards (see Fig. I13) and scattered fungus on the underside of the deck were present. These remarks were verified by the ISU team in 2006. Much of the deck was covered with fungal growth and was considerably wet throughout in comparison with other observed bridges of the same type in North Carolina.

Superstructure

The NC–DOT 2003 report notes fungus staining over sections of several girders, primarily on the top and ends of the girders. At these locations the girders sounded dull, were soft, and light to moderate decay was noted. No crushing, bulging, or signs of failure were observed, however. In

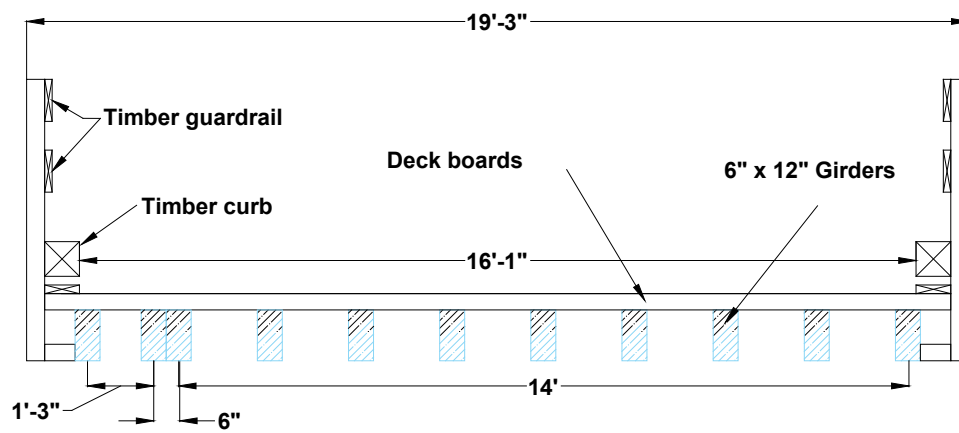
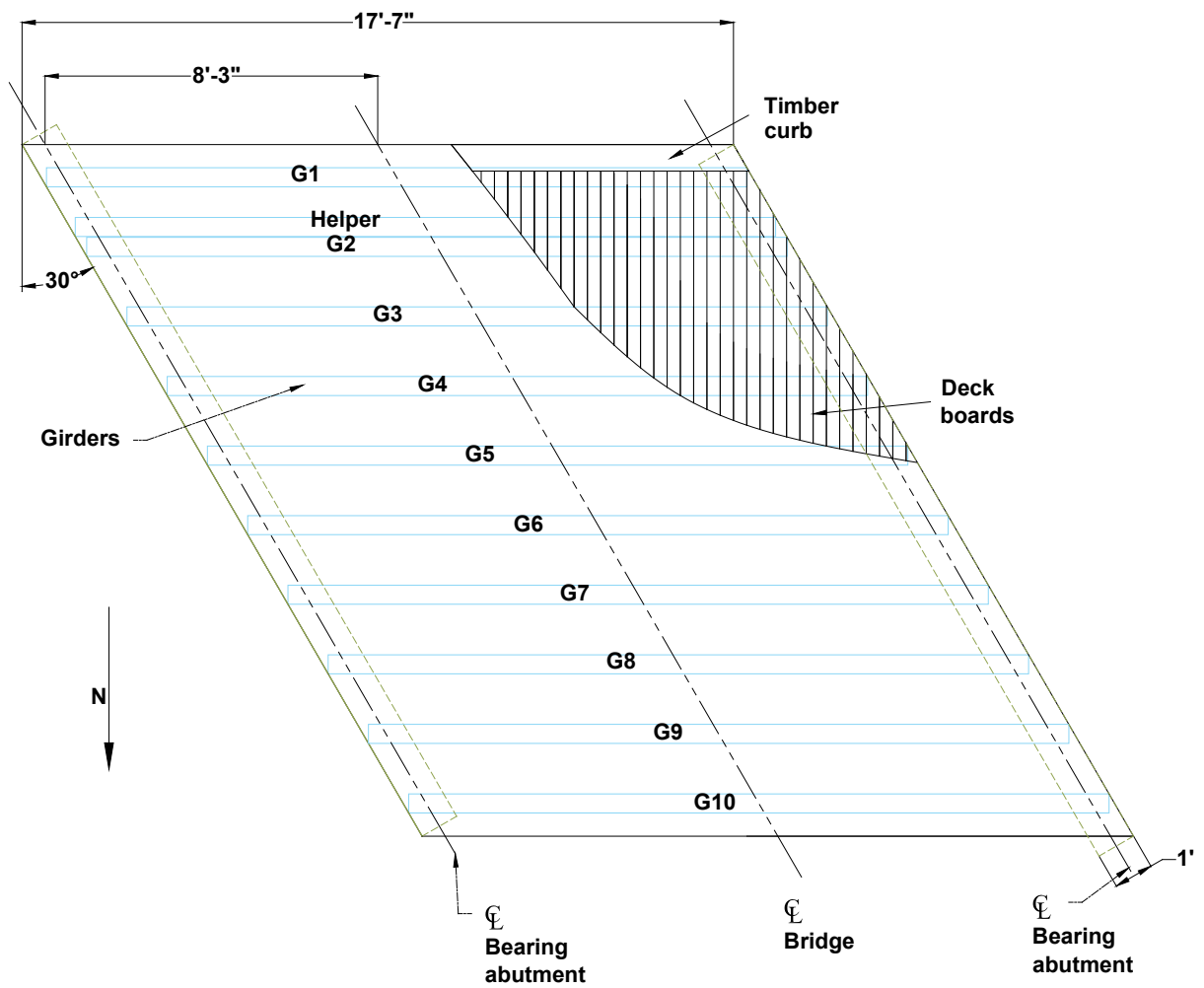


Figure 14. Plan and profile layout of Doebag Creek Bridge.

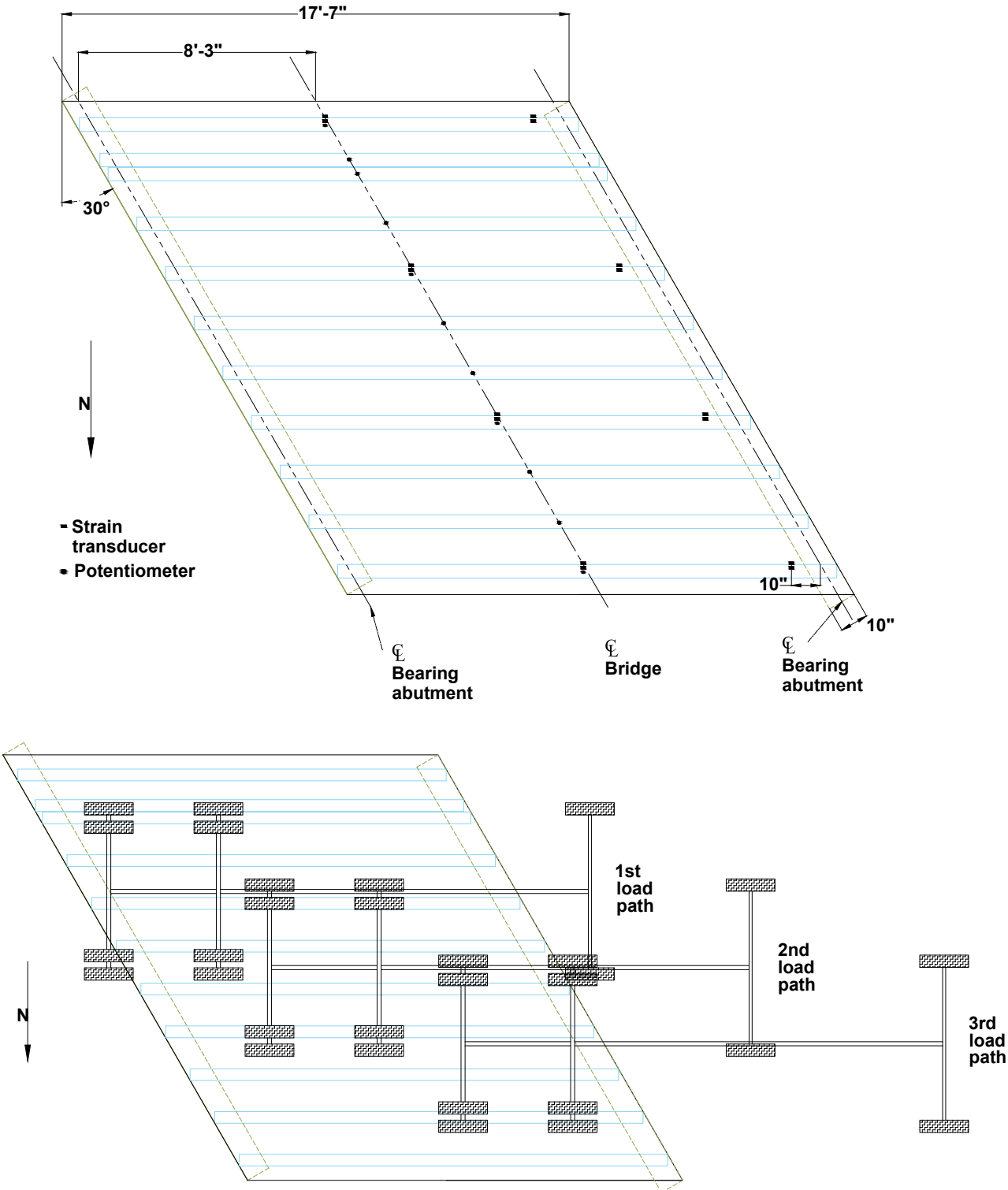


Figure 15. Instrumentation and load paths of Doebag Creek Bridge.



Figure I6. Deflection instrumentation.



Figure I7. Strain transducers.

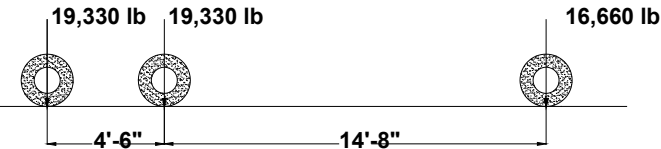


Figure I8. Truck configuration and axle loads.



Figure I9. Tandem-axle load truck.

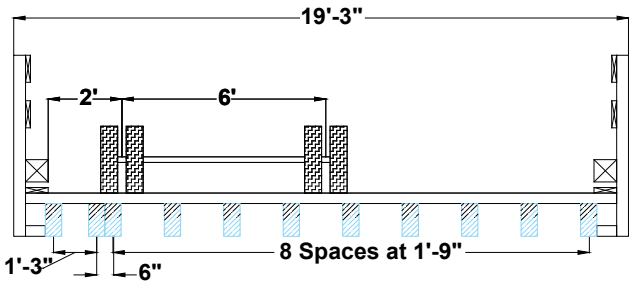


Figure I10. Transverse truck position—load path 1.

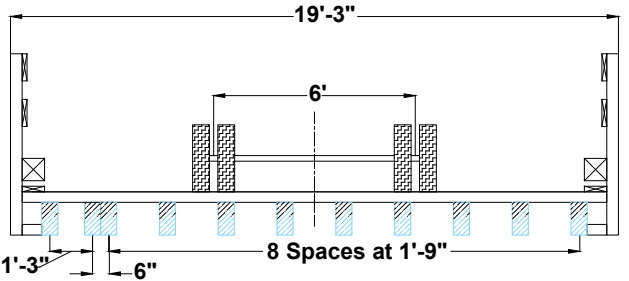


Figure I11. Transverse truck position—load path 2.

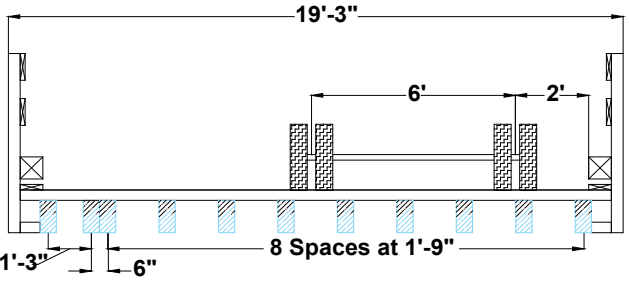


Figure I12. Transverse truck position—load path 3.



Figure I13. Decay in the ends of the deck boards.



Figure I16. Mechanical damage to girder.



Figure I14. Girder and deck condition.



Figure I17. Checking in the girders.



Figure I15. Fungal growth throughout superstructure.

addition to the previous remarks, the ISU team noted what appeared to be some mechanical damage possibly done by stream debris and a number of checks in the girders. Figures I14 to I17 show the condition of the superstructure. The bearing of the girders on the sill was sufficient and there is no misalignment.

Overall Structure

The overall structure is in a state of advanced degradation. Fungal growth and wood decay is scattered throughout, and environmental conditions promote the advancement of each. A plus, however, was that there was no evidence of insect or ultraviolet degradation. Issues of concern besides those already stated in previous sections include the presence of filtering at the abutments, where various locations on the sill and backwalls were very wet and considerable overgrowth of the surrounding foliage was observed (see Fig. I18). There were minor checks in the parapet and parapet curb.



Figure I18. Foliage overgrowth.

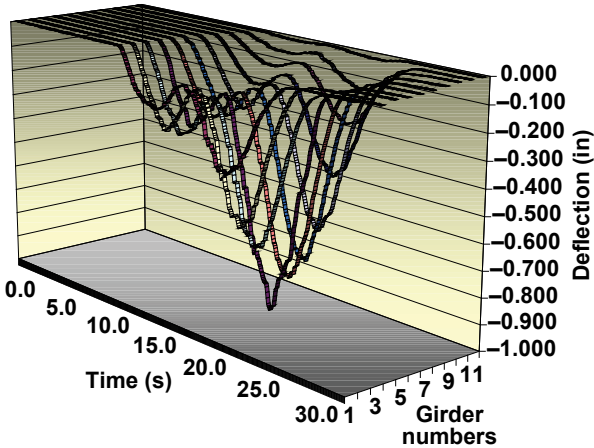


Figure I19. Deflections load path 1.

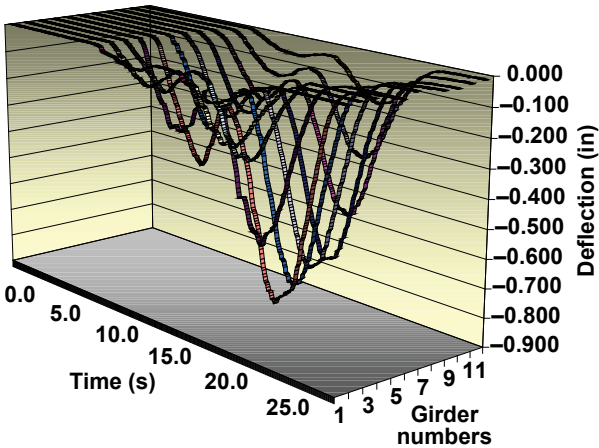


Figure I20. Deflections load path 2.

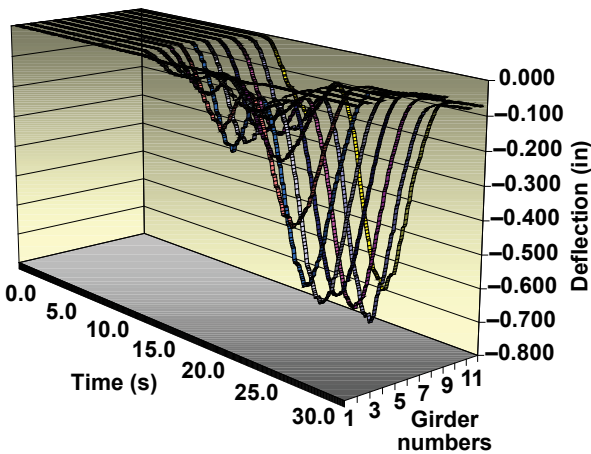


Figure I21. Deflections load path 3.

Table I1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.908
Load path 2	0.814
Load path 3	0.784

Results

The following presents the results of the static load testing of the Doebag Creek Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures I19 to I21 present the time-history deflections for each bridge girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck, one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures I19 to I21, and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table I1. Each passing of the three load paths

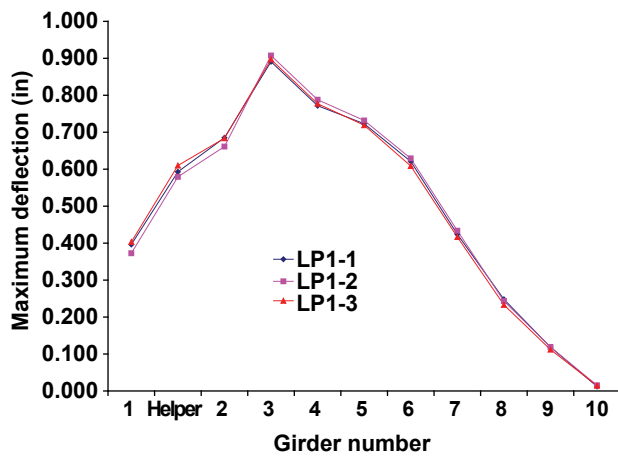


Figure I22. Maximum deflections for load path 1.

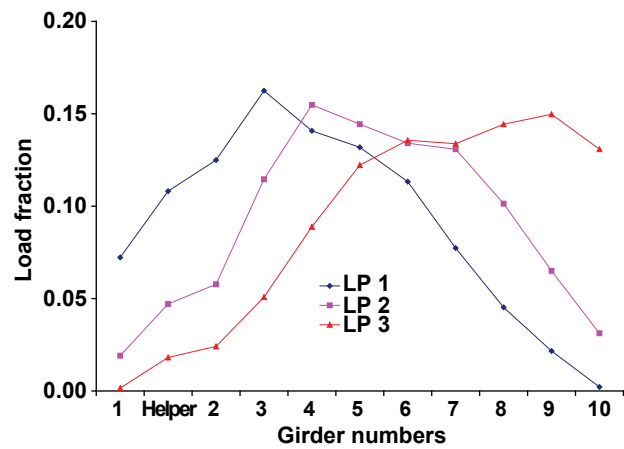


Figure I25. Load fractions for each load path.

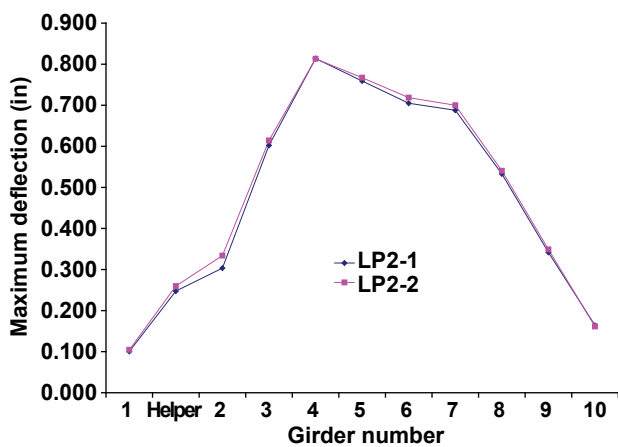


Figure I23. Maximum deflections for load path 2.

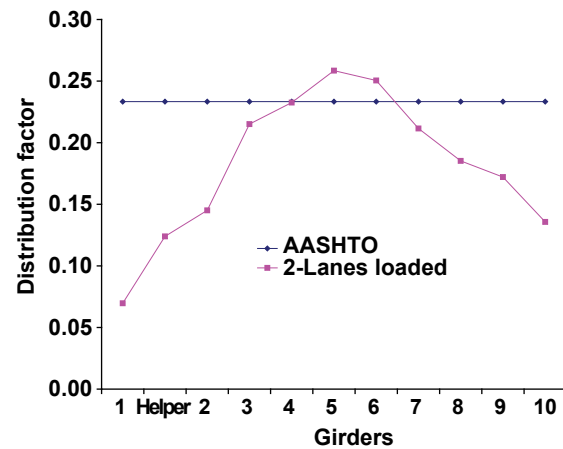


Figure I26. AASHTO design live load distribution.

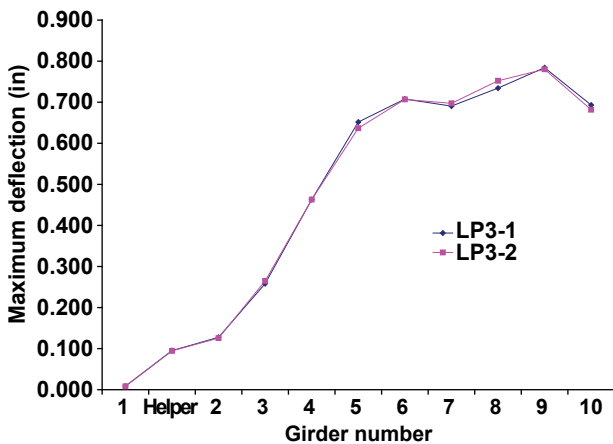


Figure I24. Maximum deflections for load path 3.

is illustrated in Figures I22 to I24. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table I2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths is 163. This value is less than the minimum recommended value for timber girders. In fact, all the n values are below the recommended n values stated in Table I2. The possible reasons for deflections greater than those recommended will be discussed later.

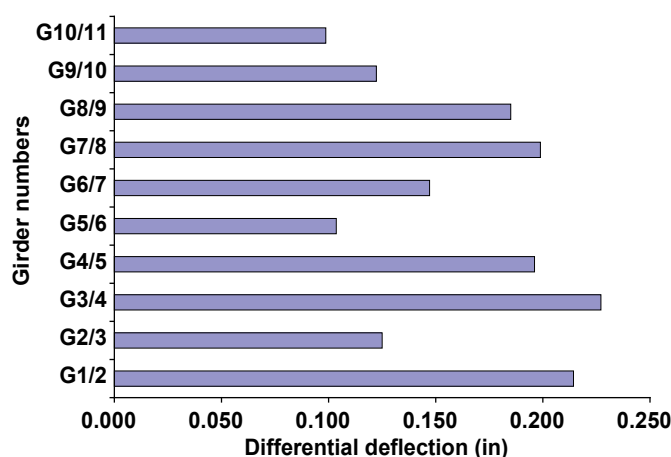


Figure I27. Differential deflections for load path 1.

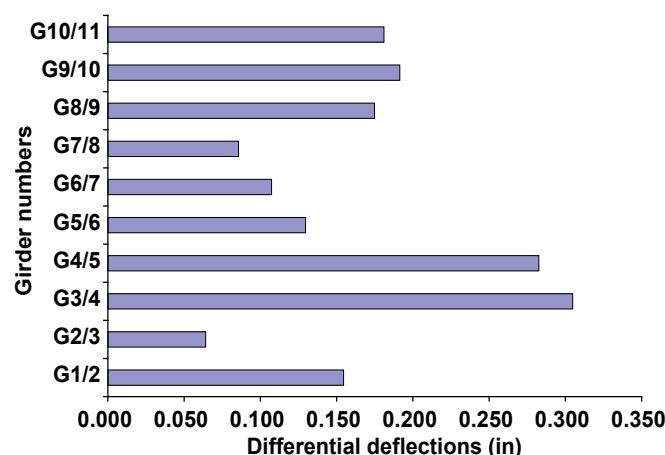


Figure I28. Differential deflections for load path 2.

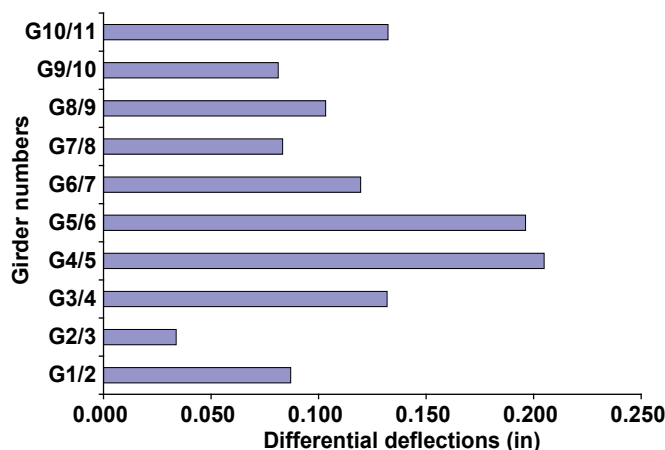


Figure I29. Differential deflections for load path 3.

Table I2—Most critical *n* value

	<i>n</i> value for girder of most deflection for each load path
Load path 1	163
Load path 2	181
Load path 3	188

Table I3—Maximum differential deflection

	Maximum differential deflection at midspan between adjacent girder (in.)
Load path 1	0.227
Load path 2	0.305
Load path 3	0.205

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure I25 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure I26 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all of the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. It should be noted that both of the criteria discussed in Appendix B are primarily based on wearing surface degradation, and for this particular bridge both are not applicable. Regardless, the results are provided for discussion purposes. Figures I27 to I29 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table I3.

The maximum differential deflection of 0.305 in. occurs in load path 2. This is nearly 38% of the maximum deflection resulting from that load path and 0.174 in. per ft of girders spacing. Among other potential reasons for large differential

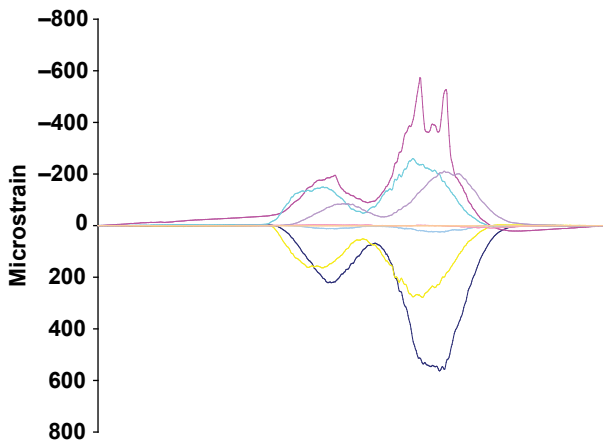


Figure I30. Truck position versus strain at midspan for load path 1.

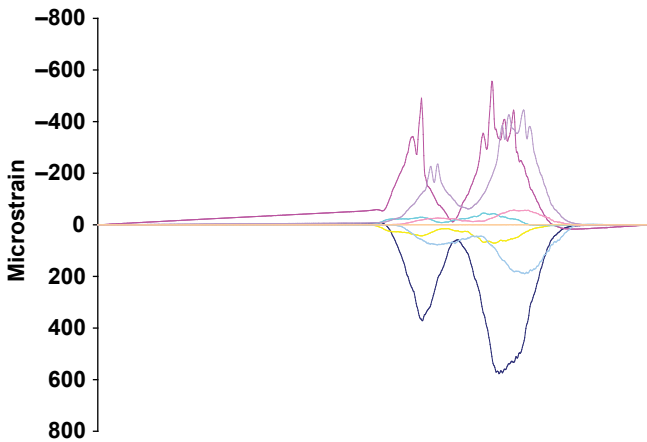


Figure I31. Truck position versus strain at midspan for load path 2.

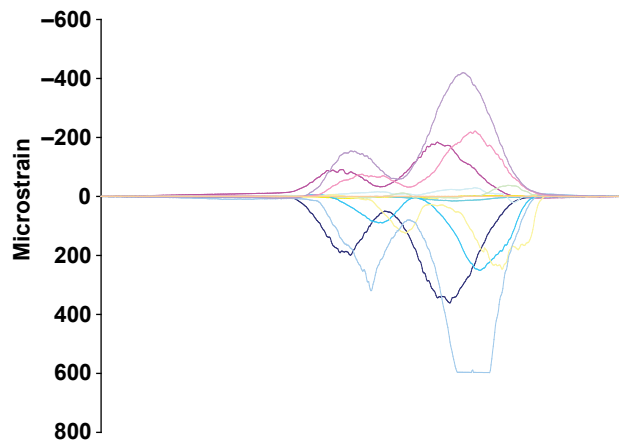


Figure I32. Truck position versus strain at midspan for load path 3.

deflections, the possibility exists that the load is not well distributed transversely between these two girders or the assumption that both girders are of equal stiffness is false. The same is true for load paths 1 and 3 as the maximum differential deflections are both around 0.2 in.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 572 and 597 microstrain, respectively. The strain plot at midspan is shown in Figures I30 to I32 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures I30 to I32 also illustrate the proportion about the neutral axis at midspan. The symmetrical pattern of the data signifies that there is very little if any composite action with the deck; that is, the girders act independently of the deck when subjected to bending.

Moisture Content

Moisture content measurements were taken at 9 locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 5, and 10 at midspan and at the west abutment. The bottom of the deck between girders 1 and 2, 5 and 6, and 9 and 10 was measured at midspan. All measurements were at least 20.2%. Overall, significant moisture content was found throughout the bridge. The moisture content measurements are summarized in Table I4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because timber strength is not critically affected until deflections become excessive. However, the girder deflections do exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability. Recommended limit states for wearing surface degradation do not apply for this bridge because no wearing surface exists.

Table 14—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, west abutment	22.1
Girder 1, midspan	22.0
Girder 5, west abutment	30 +
Girder 5, midspan	30 +
Girder 10, west abutment	23.0
Girder 10, midspan	20.2
Bottom of deck between girders 1 and 2	23.4
Bottom of deck between girders 5 and 6	30 +
Bottom of deck between girders 9 and 10	25.2

Exceeding the live load deflection recommendations can have adverse affects on structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produces a displeasing sight and perception of an unsafe bridge.

Numerous problems typically associated with wearing surface cracking exist, including seepage, decay, and corrosion. Because a wearing surface does not exist for this bridge, these problems are only magnified. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. Conditions are not ideal for seepage to quickly evaporate as western North Carolina is typically a very humid climate. Any water seepage through the deck will be susceptible to permeation of the girders.

If the Doebag Creek Bridge had a wearing surface, differential deflections between adjacent girders should be considered. The results achieved for differential deflection by static load testing showed that some higher differential deflections exist.

The distribution factor of each girder exceeded the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limit their use for primary analysis. Nevertheless, Figures I30 to I32 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 572 and 597 microstrain, respectively. These strains equate to maximum stresses of 658 and 687 lb/in², respectively. If the strains are

normalized to the AASHTO tandem load design, stresses of 851 and 888 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore allowable stresses are not exceeded by standard trucks.

Due to the lack of wearing surface and humid climate in North Carolina, higher moisture contents were expected and also found. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

The moisture content percentages were all relatively high, as one would expect by visual inspection of this bridge. The conditions the bridge is subjected to promote the advancement of decay at a much quicker rate than bridges with a wearing surface.

Conclusions

Several methods of condition and performance investigation were performed on the Doebag Creek Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection revealed moderate deterioration of concern. The condition of the bridge was inconsistent with other bridges similarly aged and subjected to similar weathering and loading conditions primarily because of the lack of wearing surface.

Seepage into the deck boards and girders was also evident. Significant biotic growth was apparent on the underside of the deck and the faces of the girders and seemed consistent with the moisture content measurements throughout the bridge.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel along the same load paths. Live load distribution factors exceeded AASHTO's prescribed design live load distribution. Deflection values at midspan failed to meet recommended values if a wearing surface existed.

Appendix J—North Carolina Bridge No. 580125, McDowell County Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Abstract

The McDowell County Bridge is a single-span timber girder bridge with a bituminous wearing surface located in McDowell County, North Carolina. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: Henry Black, North Carolina Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing; Gary Moore, North Carolina Department of Transportation employee, who organized the load testing; and Jerry Smith, Marion Bridge Maintenance employee, who operated the load truck during testing.

Background

The location of North Carolina state bridge number 580125, hereinafter referred to as the McDowell County Bridge, is shown in Figure J1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The McDowell County Bridge was built in 1958 and is located in McDowell County in western North Carolina



Figure J1. McDowell County Bridge in North Carolina.



Figure J2. McDowell County Bridge substructure.

0.8 miles south of junction US221. SR560 is carried by the structure. Currently, the bridge is not posted.

Bridge Description

The McDowell County Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface set on a 23° skew. The bridge length measures 17ft–9 in. from the west backwall to the east backwall. The bridge width measures 19 ft–1 in. from inside of curb to inside of curb and 20 ft–9 in. from outside of rail to outside of rail. The substructure consists of solid timber posts and sills (see Fig. J2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by timber blocks and bolts into the exterior girders along with bolts into the curb, which is seated and bolted to the top of the deck, as shown in Figure J3.



Figure J3. McDowell County Bridge parapet support.

Girders measure 17 ft–9 in. from end to end and have a clear span of 15 ft–9 in. A total of 19 girders, spaced 11-1/2 in. center-to-center, measuring 5-3/4 by 12-1/4 in. in cross-section, are present and are seated and toe-nailed to the 12- by 12-in. timber sills with spikes. The deck consists of individual 4-by 8-in. nominal boards laid transverse to the longitudinal girder direction, which are fastened to the girders with spikes. Overlaying the deck is a 4-in.-thick layer of asphalt wearing surface. Figure J4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure J5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 19 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure J6.

Strain transducers were attached to girder numbers 1, 4, 7, 10, 13, 16, and 19, with 1 being the outside girder on the south side of the bridge and 19 being the outside girder on the north side of the bridge. The midspan and one abutment were instrumented (see Fig. J5). Transducers were placed near only one abutment because of the symmetry of the bridge. Due to the proximity of the girders, only the exterior girders were equipped with strain gages in the compression zone. All strain instrumented girders were equipped with tensile strain gages. A typical setup at the outside girders is shown in Figure J7. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the North Carolina Department of Transportation–Division 13. Dimensions of the truck are shown in Figure J8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–6 in.; the distance between the forward-most rear axle and the front axle hubs measured 14 ft–10 in. Exact weight of the truck was 57,060 lb. Typically, 70% of the weight on a loaded tandem-axle truck is distributed to the rear axles. Using this assumption, the total weight on each rear axle and the front axle may be 19,971 lb and 17,118 lb, respectively. The truck used for load testing is shown in Figure J9.

Three load paths were considered when testing the bridge (see Figs. J10 to J12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the south curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the north curb. For all load paths, the dump truck was driven at a crawl speed from east to west and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on January 23, 2004, was obtained from the North Carolina DOT (NC–DOT). This report was reviewed, and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

According to the NC–DOT 2004 report, the wearing surface was cracked along several deck timbers. Some transverse cracking was observed by the ISU research team during testing in 2006. Though considered minor, the presence of cracks was verified, and they are shown in Figure J13. The asphalt pavement generally looked to be in good condition aside from the transverse cracking.

Deck

The deck appeared to be in good condition, there was no visible detachment of the deck boards from the girders, and all deck boards were securely fastened. Some very minor

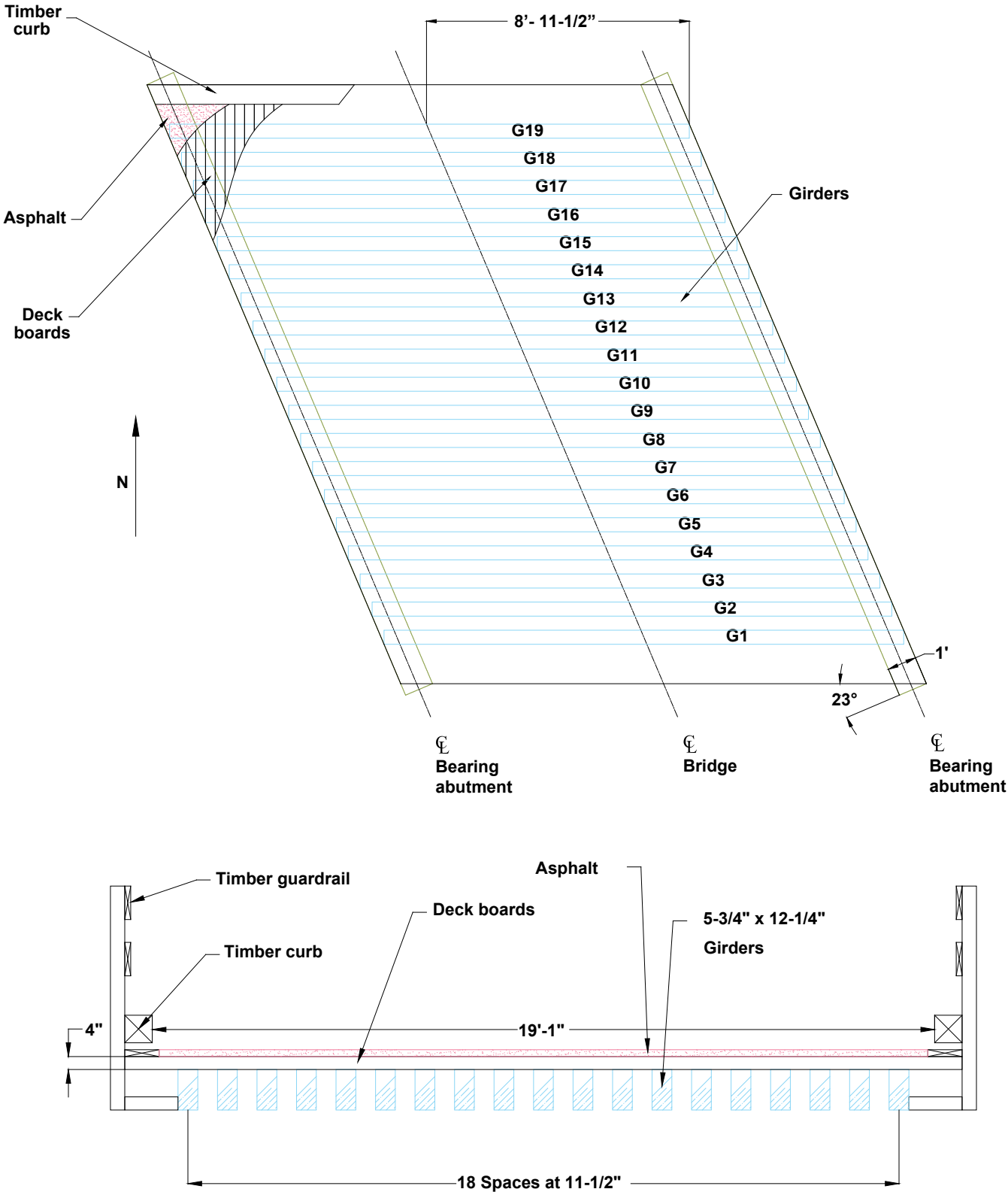


Figure J4. Plan and profile layout of McDowell County Bridge.

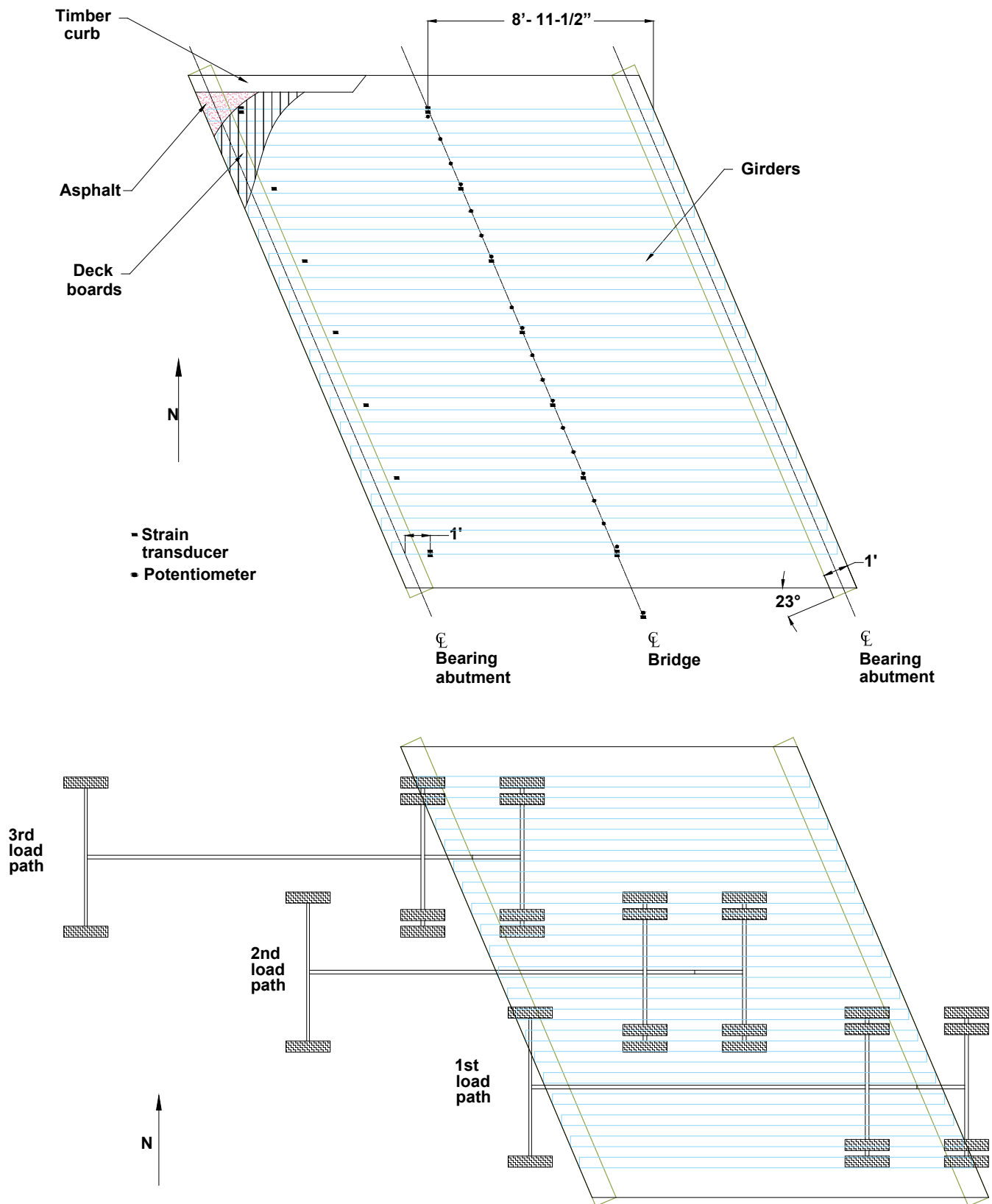


Figure J5. Instrumentation and load paths of McDowell County Bridge.



Figure J6. Deflection instrumentation.



Figure J9. Tandem axle load truck.



Figure J7. Strain transducers.

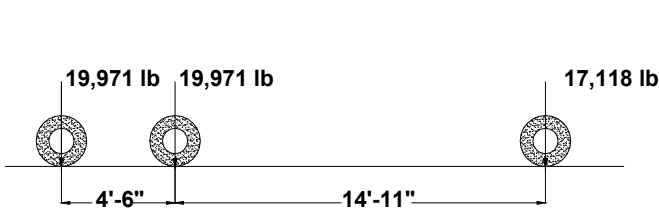


Figure J8. Truck configuration and axle loads.

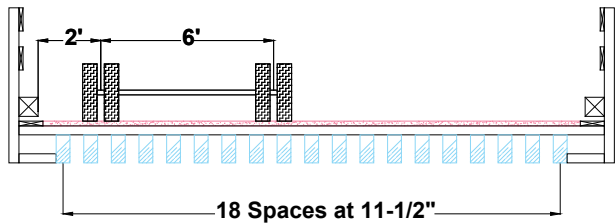


Figure J10. Transverse truck position—load path 1.

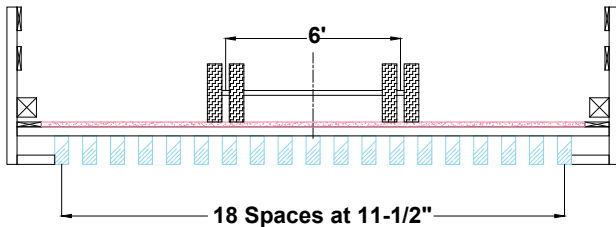


Figure J11. Transverse truck position—load path 2.

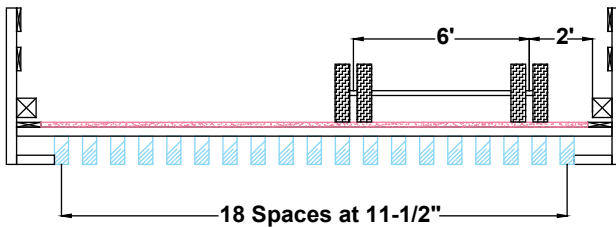


Figure J12. Transverse truck position—load path 3.



Figure J13. Observed transverse cracks.



Figure J14. Moss growth on girder 19.



Figure J15. Wingwall weathering and decay.

water staining from seepage through the wearing surface was present throughout, though there were no signs of imminent decay.

Superstructure

The girders appeared in good condition. It was apparent that the girders are well protected from weathering conditions. Even so, some moss growth was observed on north face of girder 19. This growth does not look to permeate past the girder face (see Fig. J14). Very minor checking was present near the centerline of some girders, and it appears that some mechanical damage was done to the girders on the upstream side, possibly the result of high water debris. The girder bearing on the sill was sufficient and there is no misalignment.

Overall Structure

The overall structure is in satisfactory condition, and structurally the bridge is sound. No odor like anise or winter-green signifying fungal growth was present. There was no evidence of insect or ultraviolet degradation. Issues of concern besides those already stated include significant weathering and scattered decay in the wingwalls, as noted in the NC-DOT report (see Fig. J15). The visible substructure and backwalls showed signs of decay and weathering, and the condition should be closely monitored with future inspections. There were minor checks in the parapet and parapet curb.

Results

The following presents the results of the static load testing of the McDowell County Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures J16 to J18 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck, one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges, where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures J16 to J18, and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

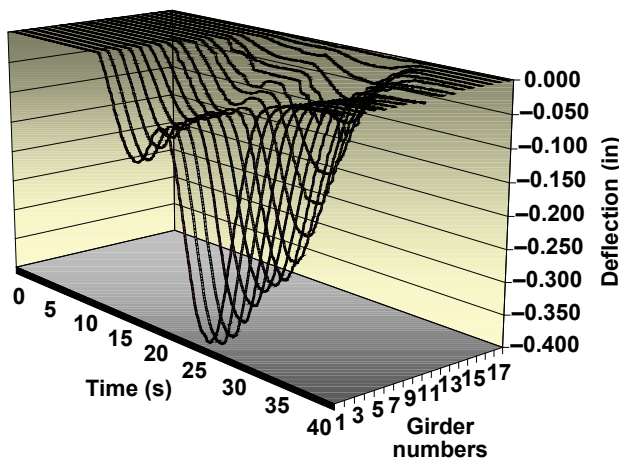
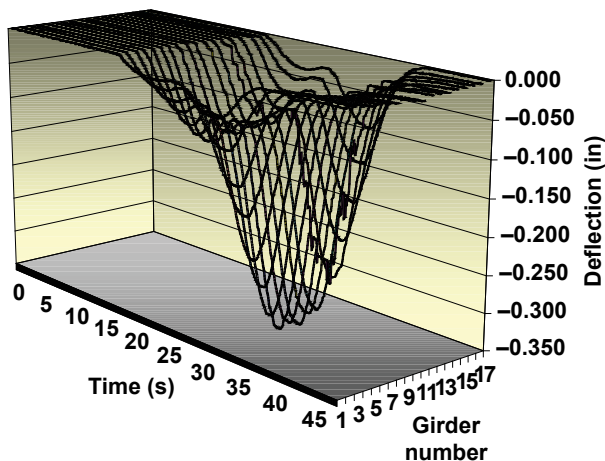
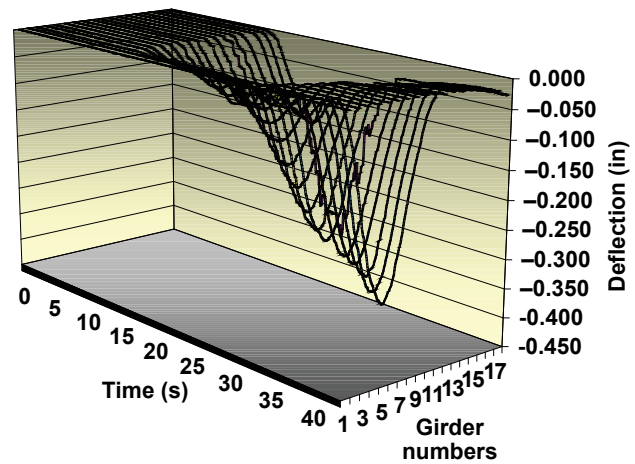
The maximum deflections achieved for each load path are presented in Table J1. Each passing of the three load paths is illustrated in Figures J19 to J21. One can notice the similar

Table J1—Maximum girder deflections

	Maximum midspan deflection for each passing (in.)
Load path 1	0.403
Load path 2	0.347
Load path 3	0.448

Table J2—Most critical n value

	n value for girder of most deflection for each load path
Load path 1	379
Load path 2	435
Load path 3	337

**Figure J16. Deflections load path 1.****Figure J17. Deflections load path 2.****Figure J18. Deflections load path 3.**

trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table J2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths is 337. This value is less than the minimum recommended value for timber girders. Values for the other two load paths exceed at least one of the recommended live load deflection limits stated in Table J2. The possible reasons for deflections greater than those recommended will be discussed later.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure J22 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure J23 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all of the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures J24 to J26 show

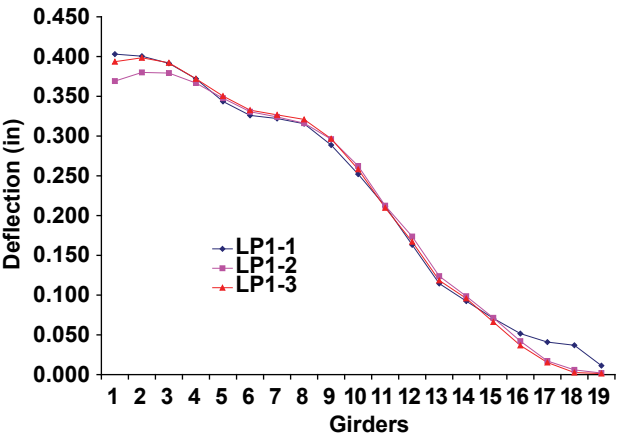


Figure J19. Maximum deflections for load path 1.

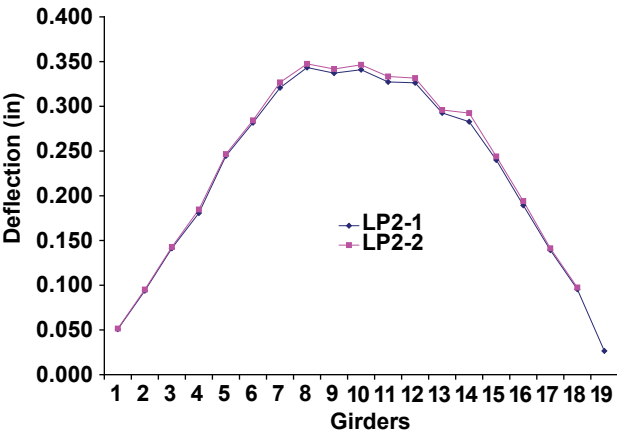


Figure J20. Maximum deflections for load path 2.

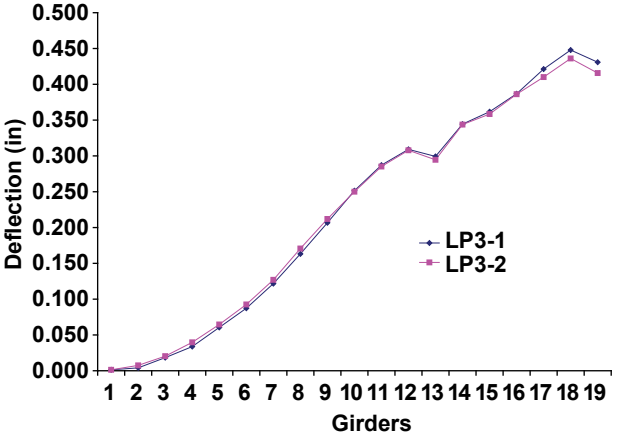


Figure J21. Maximum deflections for load path 3.

the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table J3.

Table J3—Maximum differential deflection

Maximum differential deflection at midspan between adjacent girders (in.)	
Load path 1	0.049
Load path 2	0.068
Load path 3	0.045

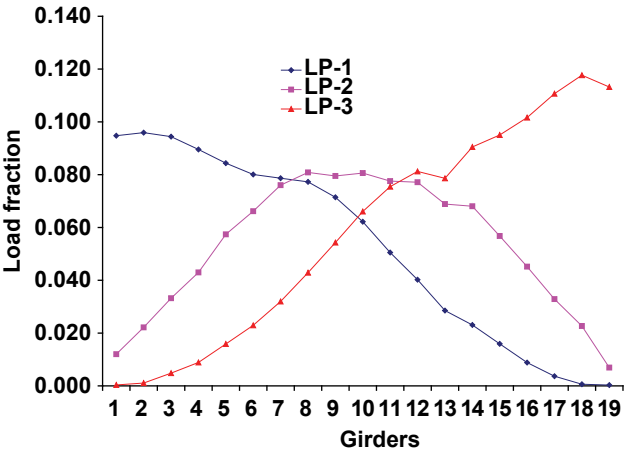


Figure J22. Load fractions for each load path.

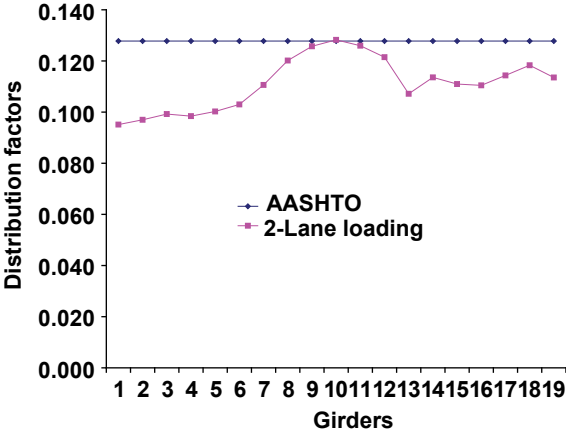


Figure J23. AASHTO design live load distribution.

The maximum differential deflection of 0.068 in. occurs in load path 2. This is nearly 20% of the maximum deflection resulting from that load path and 0.071 in. per ft of girder spacing. These results are consistent with other differential deflection results of similar bridges. The same is true for load paths 1 and 3 as the maximum differential deflections are both around 0.05 in. Among potential reasons for large differential deflections, the possibility exists that the load is not well distributed transversely between these two girders or the assumption that both girders are of equal stiffness is false.

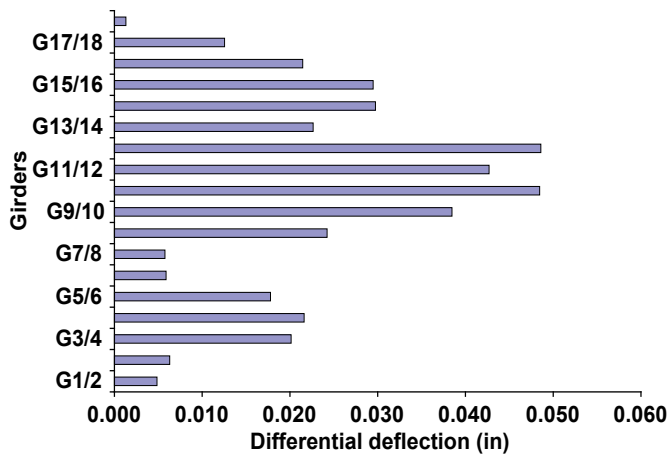


Figure J24. Differential deflections for load path 1.

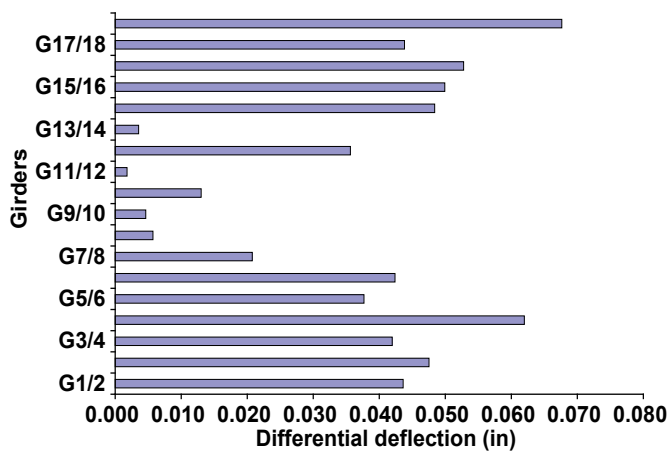


Figure J25. Differential deflections for load path 2.

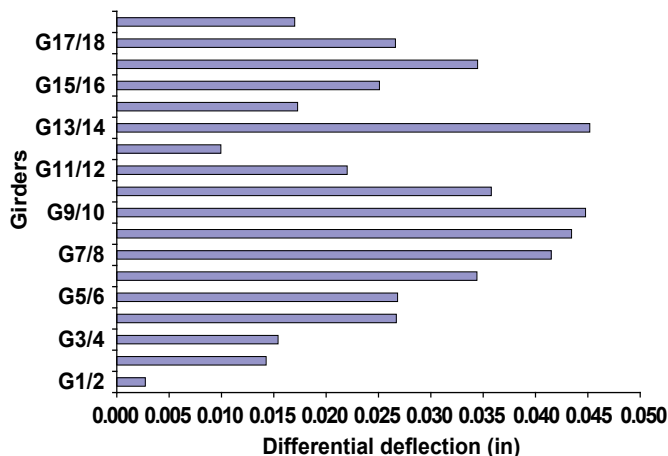


Figure J26. Differential deflections for load path 3.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 250 and 516 microstrain, respectively. The strain plot at midspan is shown in Figures J27 to J29 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures J27 to J29 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck; that is, the girders act independently of the deck when subjected to bending.

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 10, and 19 at midspan and at the west abutment. The bottom of the deck between girders 1 and 2, 9 and 10, and 18 and 19 was measured at midspan. Measurements ranged from 16.1% to 24.2%. The moisture content measurements are summarized in Table J4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength, because timber strength is not critically affected until deflections become excessive. However, at least one of the load paths included girder deflections that exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability. The deflections from the other two load paths were at least within the recommended limits of one or more sources.

Exceeding the live load deflection recommendations can have adverse affects on structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners

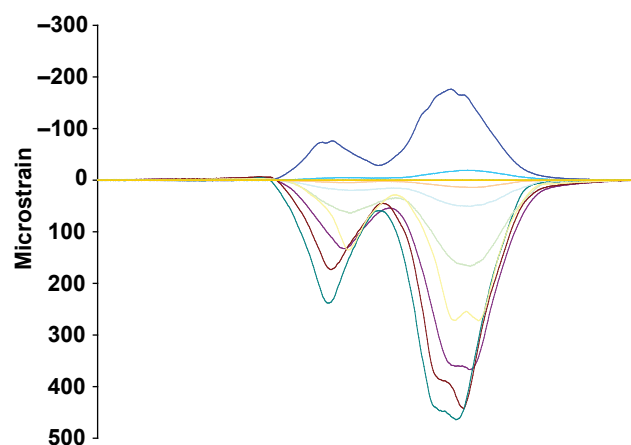


Figure J27. Strain at midspan for load path 1.

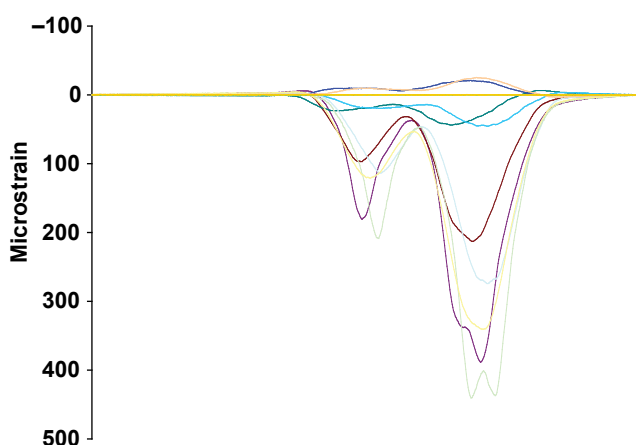


Figure J28. Strain at midspan for load path 2.

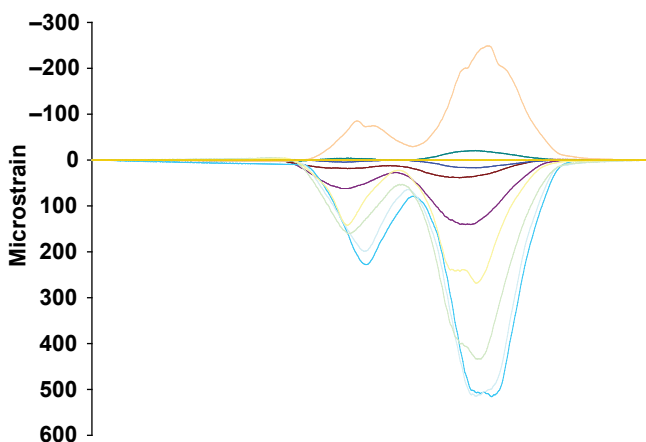


Figure J29. Strain at midspan for load path 3.

such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produces a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded as asphalt has very little fatigue resistance. Numerous problems associated with cracking exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. Conditions are not ideal for seepage to quickly evaporate as western North Carolina typically has a very humid climate. As a result, any water seepage through the deck will be prone to permeate the girders.

Through visual inspection, transverse cracks in the wearing surface were found. Deflections exceeding the recommended live load limit state would suggest that the wearing surface may show transverse cracking. The wearing surface of this particular bridge is in good condition, though attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was over that limit by 0.021 in. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limit their use for primary analysis. Nevertheless, Figures J27 to J29 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 250 and 516 microstrain,

respectively. These strains equate to maximum stresses of 289 and 593 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 362 and 742 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the humid climate in North Carolina, higher moisture contents were expected and also found. The amount of water present in wood can modify its physical properties. With increasing moisture content, the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

Excessive moisture contents were not observed therefore one could conclude that none of the tested areas are subjected to vastly different amounts of moisture.

Conclusions

Several methods of condition and performance investigation were performed on the McDowell County Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of notable concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Minor transverse cracking in the wearing surface was observed. As a result, very minor seepage through the wearing surface and into the deck boards and girders was evident. Some biotic growth was apparent on the north face of girder 19, though the growth appeared to be limited to the surface of this element.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Deflection values at midspan met at least one recommended live load deflection limit except for the maximum deflection value of load path 3.

Appendix K—Larimer County, Colorado, Bridge No. LR5J-0.2-70, Larimer County Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Abstract

The Larimer County Bridge is a single-span timber girder bridge with a bituminous wearing surface located in Larimer County, Colorado. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: Dale Miller, Larimer County Road and Bridge employee, who organized the load testing; and Jim Graves, Larimer County employee, who operated the load vehicle during live load testing.

Background

The location of Colorado state bridge LR5J-0.2-70, hereinafter referred to as the Larimer County Bridge, is shown in Figure K1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Larimer County Bridge was built in 1940 and is located in Larimer County in northern Colorado, approximately 13 miles north of Fort Collins on county road 5J. Currently, the bridge is posted for 14 tons (type 3 truck), 23 tons (type 3S2 truck), and 23 tons (type 3-2 truck).



Figure K1. Larimer County Bridge location.



Figure K2. Bridge substructure.

Bridge Description

The Larimer County Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface. The bridge length measures 24 ft-3 in. from the southwest backwall to the northeast backwall. The bridge width measures 25 ft-1 in. from inside of curb to inside of curb and 25 ft-9 in. from inside of rail to inside of rail. The substructure consists of solid timber posts and sills (see Fig. K2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by bolts into the exterior girders along with bolts into the curb, which is seated and bolted to the top of the deck, as shown in Figure K3.

Girders measure 24 ft-3 in. from end to end and have a clear span of 22 ft-3 in. A total of 15 girders, spaced an average of 22 in. center-to-center, measuring 4 by 17-1/2 in. in cross section are present and are seated and toe-nailed to



Figure K3. Larimer County Bridge parapet support.

the 12- by 12-in. timber sills with spikes. The deck consists of individual 2- by 4-in. nominal boards laid upon the short face transverse to the longitudinal girder direction. Overlaying the deck is a 2-in.-thick layer of asphalt wearing surface. Figure K4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure K5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 15 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure K6.

Strain transducers were attached to girder numbers 1, 6, 10, and 15, with 1 being the outside girder on the northwest side of the bridge and 15 being the outside girder on the southeast side of the bridge. The midspan and one abutment were instrumented (see Fig. K5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill. Figure K7 shows a typical setup of strain transducers near the girder ends.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by Larimer County. Dimensions

of the truck are shown in Figure K8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–7 in.; the distance between the forward-most rear axle and the front axle hubs measured 15 ft–1 in. The truck weighed approximately 54,000 lb. Assuming that 70% of the vehicle weight was distributed over the rear axles, the weight over the front axle and back axles was 16,200 lb and 18,900 lb, respectively. The assumed axle weights are shown in Figure K8, and the truck used for load testing is shown in Figure K9.

Three load paths were considered when testing the bridge (see Figs. K10 to K12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the northwest curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the southeast curb. For all load paths, the dump truck was driven at a crawl speed from southwest to northeast and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on April 6, 2005, was obtained from Larimer County. This report was reviewed, and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

Transverse cracking in the wearing surface in line with the transverse deck boards beneath was observed by the ISU research team. Crack formation started at the transition between the roadway and the bridge, and these cracks could pose future problems if not monitored and repaired. An uneven transition could subject the bridge to unnecessary effects from dynamic loads, which can be magnified by higher vehicle speeds on this roadway. Figure K13 shows some of the cracking observed.

Except for the transverse cracking, the wearing surface looked to be in satisfactory condition, although much of the bridge wearing surface was uneven.

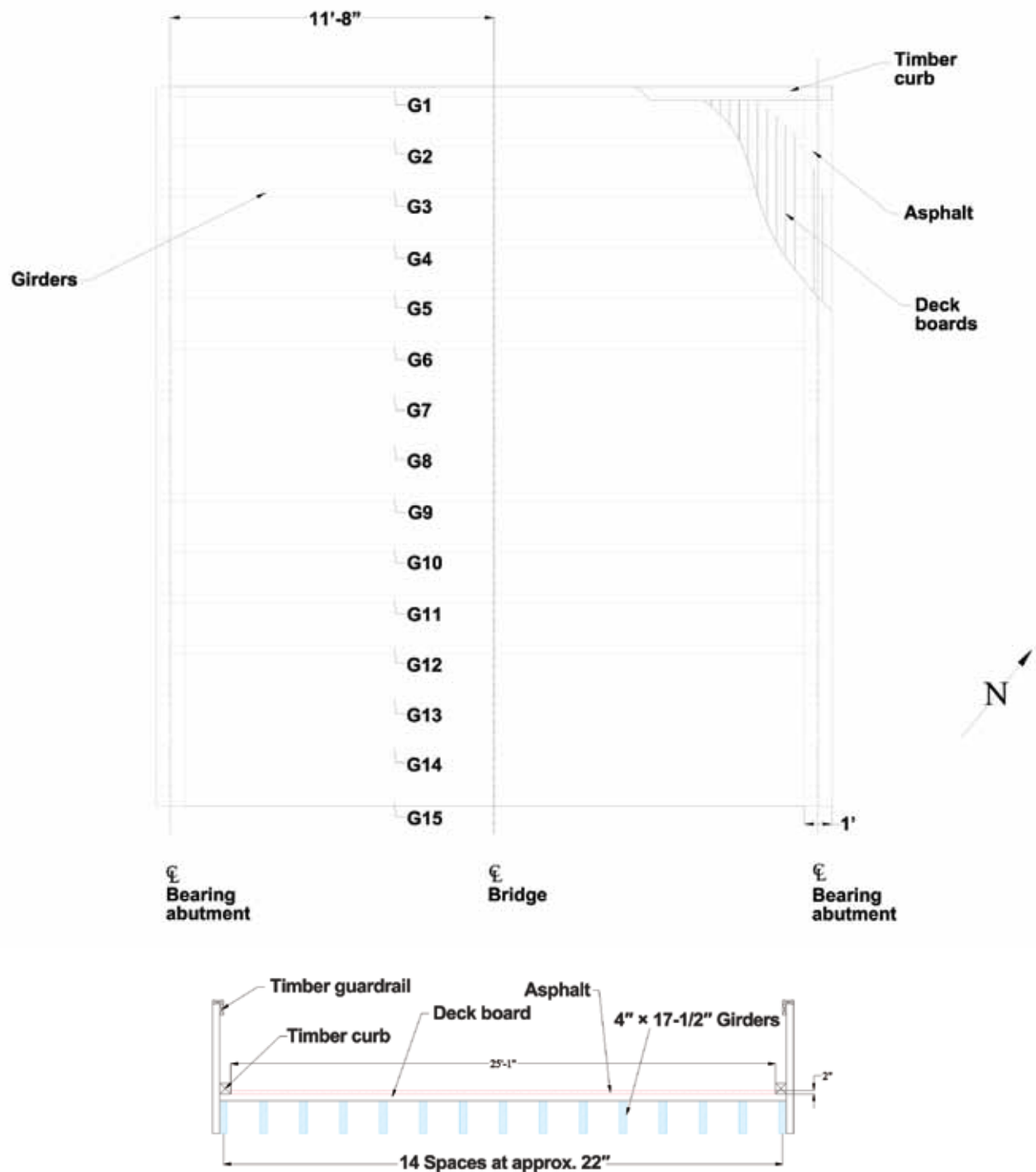


Figure K4. Plan and profile layout of Larimer County Bridge.

Deck

According to the Larimer County 2005 report, numerous water stains were present in random locations. This condition was verified by the ISU team, though much of the deck was found to be dry. Some cracking in the exposed ends of the deck boards was present. Even so, the deck appeared to be in good condition, there was no visible detachment of

the deck boards from the girders, and all deck boards were securely fastened.

Superstructure

The girders appear in good condition, yet some minor degradation was present throughout. The exterior girders were in poorer condition than the rest, presumably a result of more exposure to weathering conditions. Even so, the condition

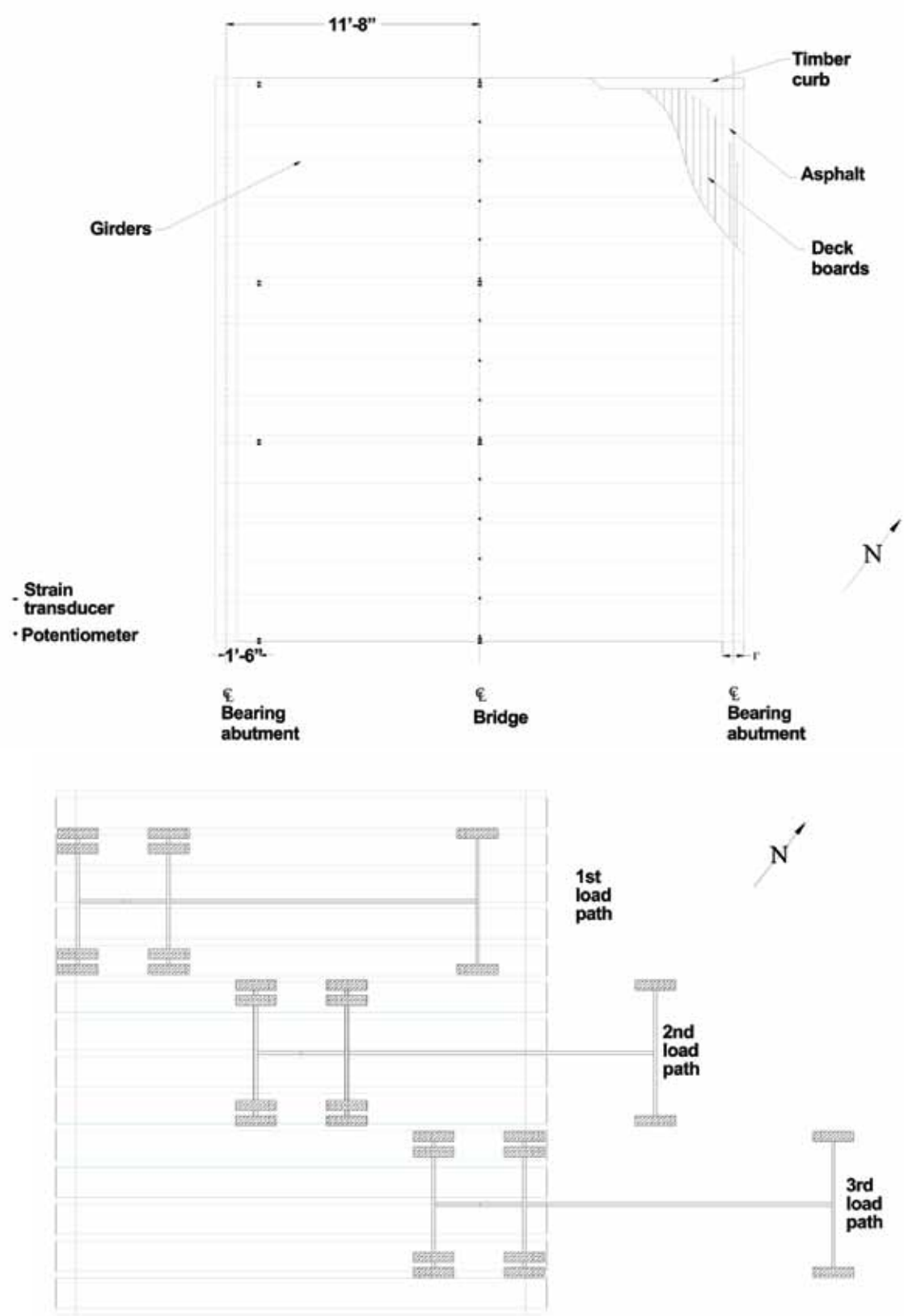


Figure K5. Instrumentation and load paths of Larimer County Bridge.



Figure K6. Deflection instrumentation.



Figure K7. Strain transducers.

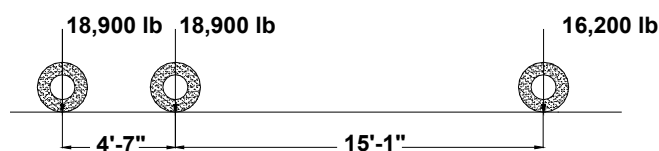


Figure K8. Truck configuration and axle loads.

is not cause for concern. These girders should be monitored closely with future inspections because if checking becomes severe, degradation effects can be accelerated further and the structural integrity of the girder could be compromised. Figure K14 shows the general superstructure condition, and Figure K15 shows the exterior girder condition.

Minor water staining was present in random locations throughout the girder elements, much like the deck. The girder bearing on the sill was sufficient, and no misalignment was observed. A large amount of dirt has accumulated



Figure K9. Tandem-axle load truck.

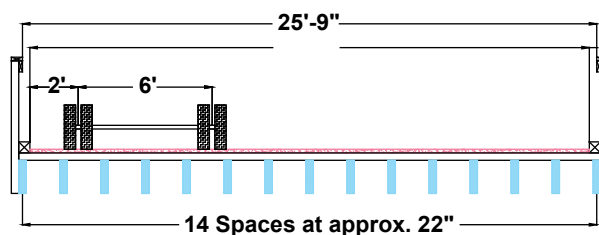


Figure K10. Transverse truck position—load path 1.

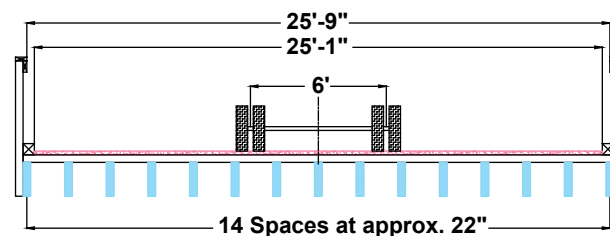


Figure K11. Transverse truck position—load path 2.

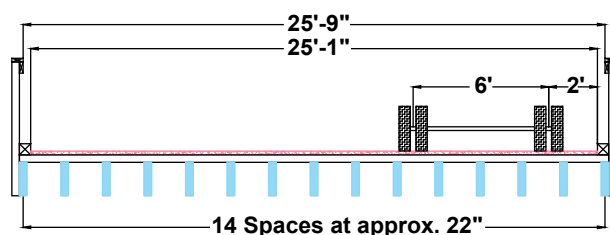


Figure K12. Transverse truck position—load path 3.

on the bearing seats between the girders in a number of locations at both abutments.

Overall Structure

The overall structure is in satisfactory condition, and structurally the bridge is sound. No odor like anise or wintergreen signifying fungal growth was present. There was no evidence of insect or mechanical degradation. Exposed timber members looked to be weathered and subjected to



Figure K13. Transverse wearing surface cracking.



Figure K14. General superstructure condition.



Figure K15. Exterior girder condition.

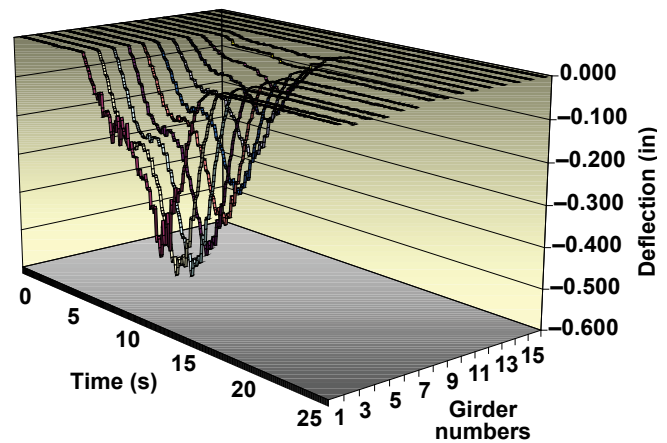


Figure K16. Deflections for load path 1.

ultraviolet degradation. The substructure also showed signs of minor checking. The timber railing should be watched for further degradation because some of the posts are split.

Results

The following presents the results of the static load testing of the Larimer County Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures K16 to K18 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck, one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges, where one wave is typically present. This two wave relationship is evident in Figures K16 to K18, and the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table K1. Each passing of the three load paths is illustrated in Figures K19 to K21. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

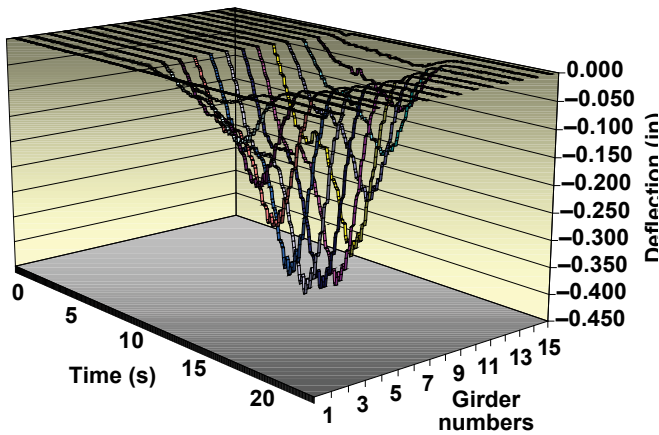


Figure K17. Deflections for load path 2.

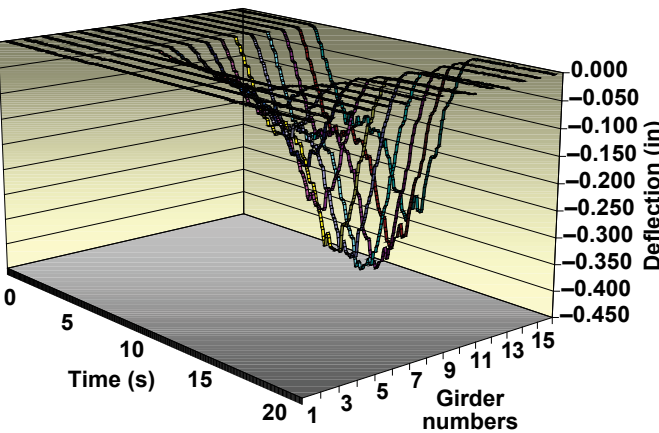


Figure K18. Deflections for load path 3.

Table K1—Maximum girder deflection	
	Maximum midspan deflection for each passing (in.)
Load path 1	0.505
Load path 2	0.407
Load path 3	0.425

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table K2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths is 400. This value is greater than one of the minimum recommended values for timber girders. The other two load paths yield values

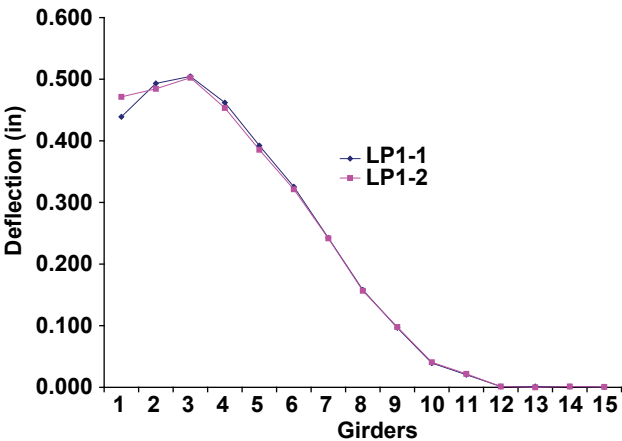


Figure K19. Maximum deflections for load path 1.

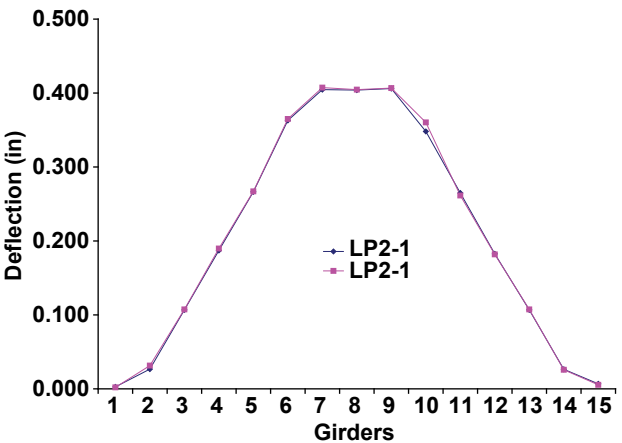


Figure K20. Maximum deflections for load path 2.

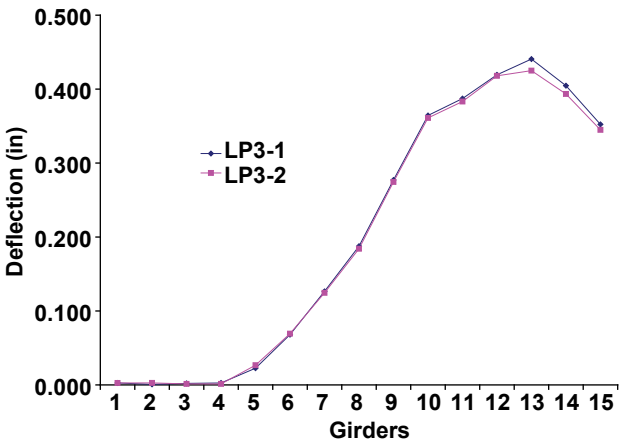
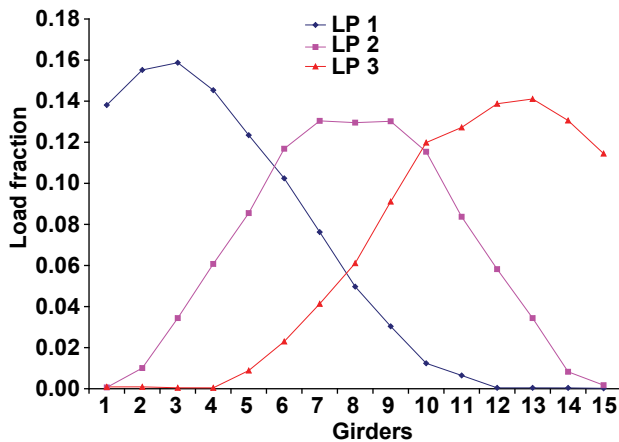
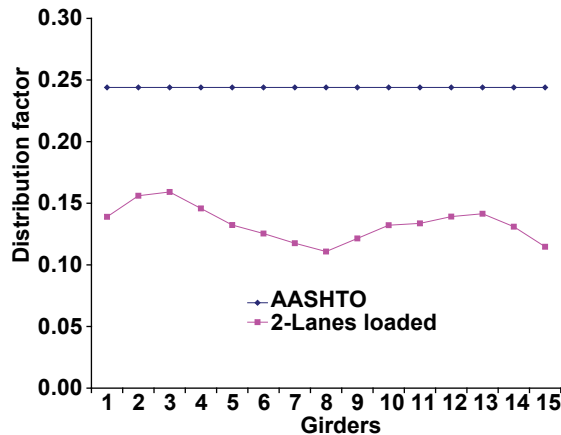


Figure K21. Maximum deflections for load path 3.

Table K2—Most critical n value

	n value for girder of most deflection for each load path
Load path 1	400
Load path 2	496
Load path 3	475

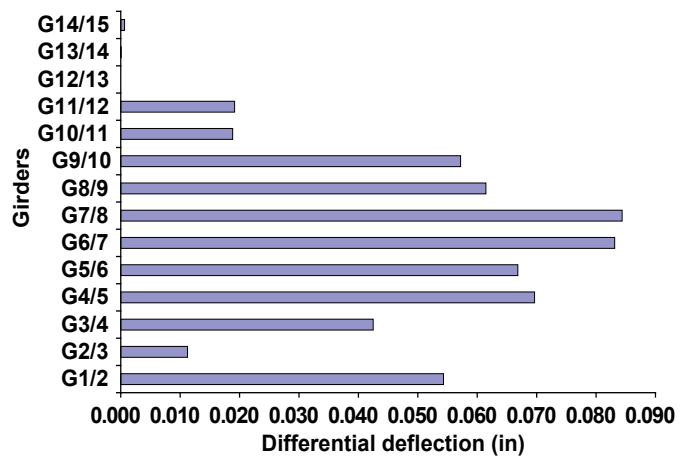
**Figure K22. Load fractions for each load path.****Figure K23. AASHTO design live load distribution.**

greater than the two of the recommended n values stated in Table K2.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure K22 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges

**Figure K24. Differential deflections for load path 1.**

are $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure K23 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures K24 to K26 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table K3.

The maximum differential deflection of 0.099 in. occurs in load path 2, and this is nearly 25% of the maximum deflection for this load path and 0.054 in. per ft of girder spacing. This differential deflection may cause future problems with longitudinal cracking. The same is true for load paths 1 and 3 as the maximum differential deflections are both around 0.09 in. With larger differential deflections, the possibility exists that the load was not well distributed transversely between the adjacent girders or the assumption that both girders are of equal stiffness was false.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 288 and 601 microstrain, respectively. The strain plot at midspan is

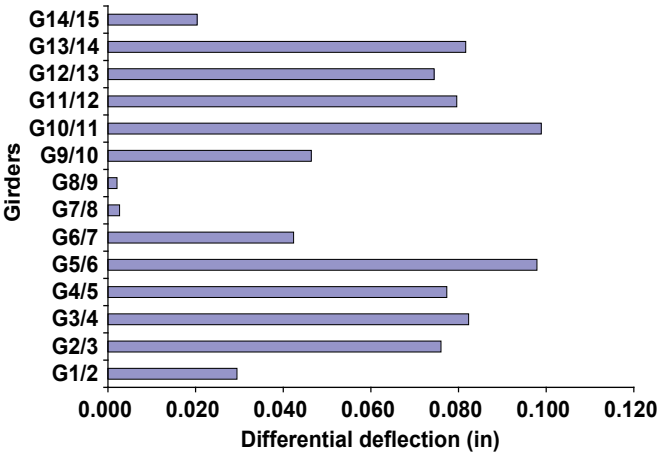


Figure K25. Differential deflections for load path 2.

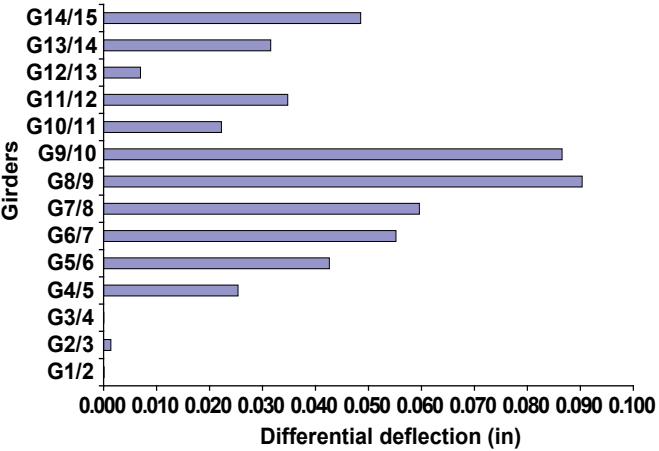


Figure K26. Differential deflections for load path 3.

Table K3—Maximum differential deflection	
	Maximum differential deflection at midspan between adjacent girders (in.)
Load path 1	0.084
Load path 2	0.099
Load path 3	0.090

shown in Figures K27 to K29 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

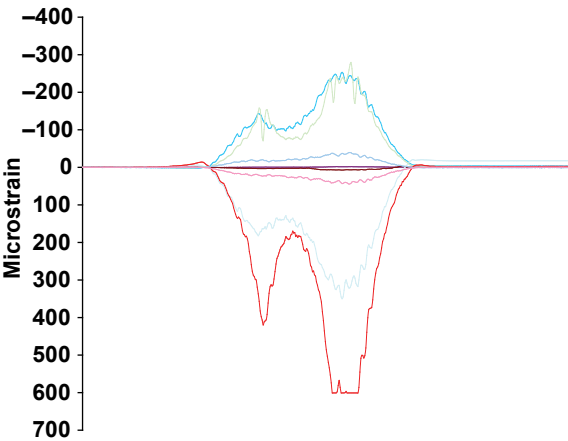


Figure K27. Strain at midspan for load path 1.

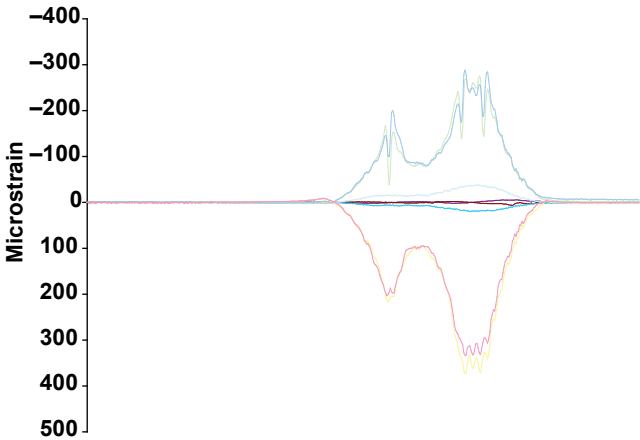


Figure K28. Strain at midspan for load path 2.

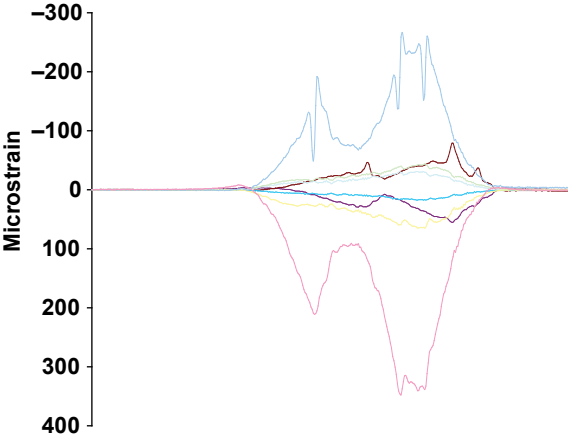


Figure K29. Strain at midspan for load path 3.

Figures K27 to K29 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck; that is, the girders act independently of the deck when subjected to bending.

Table K4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, northeast abutment	8.6
Girder 1, midspan	9.3
Girder 6, northeast abutment	9.4
Girder 6, midspan	9.5
Girder 10, northeast abutment	9.1
Girder 10, midspan	8.8
Girder 15, northeast abutment	8.3
Girder 15, midspan	9.2
Bottom of deck between girders 1 and 2	10.6
Bottom of deck between girders 7 and 8	8.6
Bottom of deck between girders 14 and 15	10.4

Moisture Content

Moisture content measurements were taken at 11 locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 6, 10, and 15 at the midspan and northeast abutment. The bottom of the deck between girders 1 and 2, 7 and 8, and 14 and 15 was measured at midspan. Measurements ranged from 8.6% to 10.6%. The moisture content measurements are summarized in Table K4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength, because timber strength is not critically affected until deflections become excessive. Also, the girder deflections do not exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability.

Exceeding the live load deflection recommendations can have adverse affects on structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produces a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded because asphalt has very little fatigue resistance. Numerous problems associated with cracking exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. A benefit of the bridge location is that conditions are ideal for seepage to quickly

evaporate because of the more arid climate. As a result, any water seepage through the deck will be prone to evaporation before permeation of the girders.

It could seem that the wearing surface may show transverse cracking only if deflections exceeded the recommended live load limit state. Even so, through visual inspection, transverse cracks in the wearing surface were found. The wearing surface of this particular bridge is in satisfactory condition, though close attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report, having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was very nearly at that limit. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary because the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures 27 to 29 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 288 and 601 microstrain, respectively. These strains equate to maximum stresses of 331 and 691 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 438 and 914 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the climate in northern Colorado, lower moisture contents were expected and also found. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

The moisture content percentages were all within a couple percentage points of one another. This shows that none of the tested areas are subjected to vastly different amounts of moisture.

Conclusions

Several methods of condition and performance investigation were performed on the Larimer County Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of immediate concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Some transverse cracking in the wearing surface was observed. A number of random locations throughout the deck and girder elements showed water staining, presumably from seepage through the wearing surface.

Though the superstructure and deck look to be in overall good condition, the affects of the northern Colorado climate and weathering is apparent in most exposed timber elements.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions, though the load is not particularly well distributed across the bridge. All deflection values at midspan were within at least one of the recommended maximum values.

Appendix L—Colorado Bridge No. P-19-F Minor, Colorado Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Abstract

The Colorado Bridge is a single-span timber girder bridge with a bituminous wearing surface located in Las Animas County, Colorado. The bridge was load tested and visually assessed as part of a research project through the United States Department of Agriculture (USDA) – Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: Mark Nord, Colorado Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing.

Background

The location of Colorado state bridge P-19-F Minor, hereinafter referred to as the Colorado Bridge, is shown in Figure L1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Colorado Bridge was built in 1930 and is located in Las Animas County in southern Colorado, approximately 3 miles east of Trinidad on State Highway 160. Currently, the bridge is not posted.



Figure L1. Colorado Bridge location.

Bridge Description

The Colorado Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface. The bridge length measures 17 ft–0 in. from the west backwall to the east backwall. The bridge width measures 24 ft–6 in. from inside of curb to inside of curb and 25 ft–4 in. from outside of rail to outside of rail. The substructure consists of solid timber posts and sills (see Fig. L2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by bolts into the exterior girders along with bolts into the curb, which is seated and bolted to the top of the deck, as shown in Figure L3.

Girders measure 17 ft–0 in. from end to end and have a clear span of 15 ft–0 in. A total of 11 girders, spaced 29-3/4 in. center-to-center, measuring 5-3/4 by 18 in. in cross section are present and are seated and toe-nailed to the 12- by 12-in. timber sills with spikes. The deck consists of individual 3- by 6-in. nominal boards laid upon the short face transverse to the longitudinal girder direction. Overlaying the deck is a 10-1/2 in.-thick layer of asphalt wearing surface. Figure L4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure L5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 11 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was



Figure L2. Bridge substructure.



Figure L3. Colorado Bridge parapet support.

used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure L6.

Strain transducers were attached to girder numbers 1, 5, 8, and 11, with 1 being the outside girder on the north side of the bridge and 11 being the outside girder on the south side of the bridge. The midspan and one abutment were instrumented (see Fig. L5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill. Figure L7 shows a typical setup of strain transducers near the girder ends.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the Colorado Department of

Transportation. Dimensions of the truck are shown in Figure L8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–6 in.; the distance between the forward-most rear axle and the front axle hubs measured 14 ft–11 in. Exact weight of the truck was 49,120 lb; the total rear weight equaled 32,620 lb and the front axle weight was 16,500 lb. Assuming equal weights on each rear axle, the rear axles weighed 16,810 lb. The axle weights are shown in Figure L8, and the truck used in the load testing is shown in Figure L9.

Three load paths were considered when testing the bridge (see Figs. L10 to L12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the south curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the north curb. For all load paths, the dump truck was driven at a crawl speed from east to west, and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on July 1, 2004, was obtained from the Colorado DOT (CDOT). This report was reviewed, and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

Some transverse cracking in the wearing surface reflecting the transverse deck boards beneath was observed by the ISU research team. Larger cracks have formed at the transition between the roadway and the bridge, and these cracks could pose future problems if not monitored and repaired. An uneven transition could subject the bridge to unnecessary effects from dynamic loads, which can be magnified by higher vehicle speeds on this roadway. Figure L13 shows some of the cracking observed.

Aside from the transverse cracking, the wearing surface looked to be in good condition. One should note that the wearing surface thickness of 10-1/2 in. was greater than that typically found on timber bridges.

Deck

According to the CDOT 2004 report, white fungus was growing on the untreated bottom of the deck. This condition

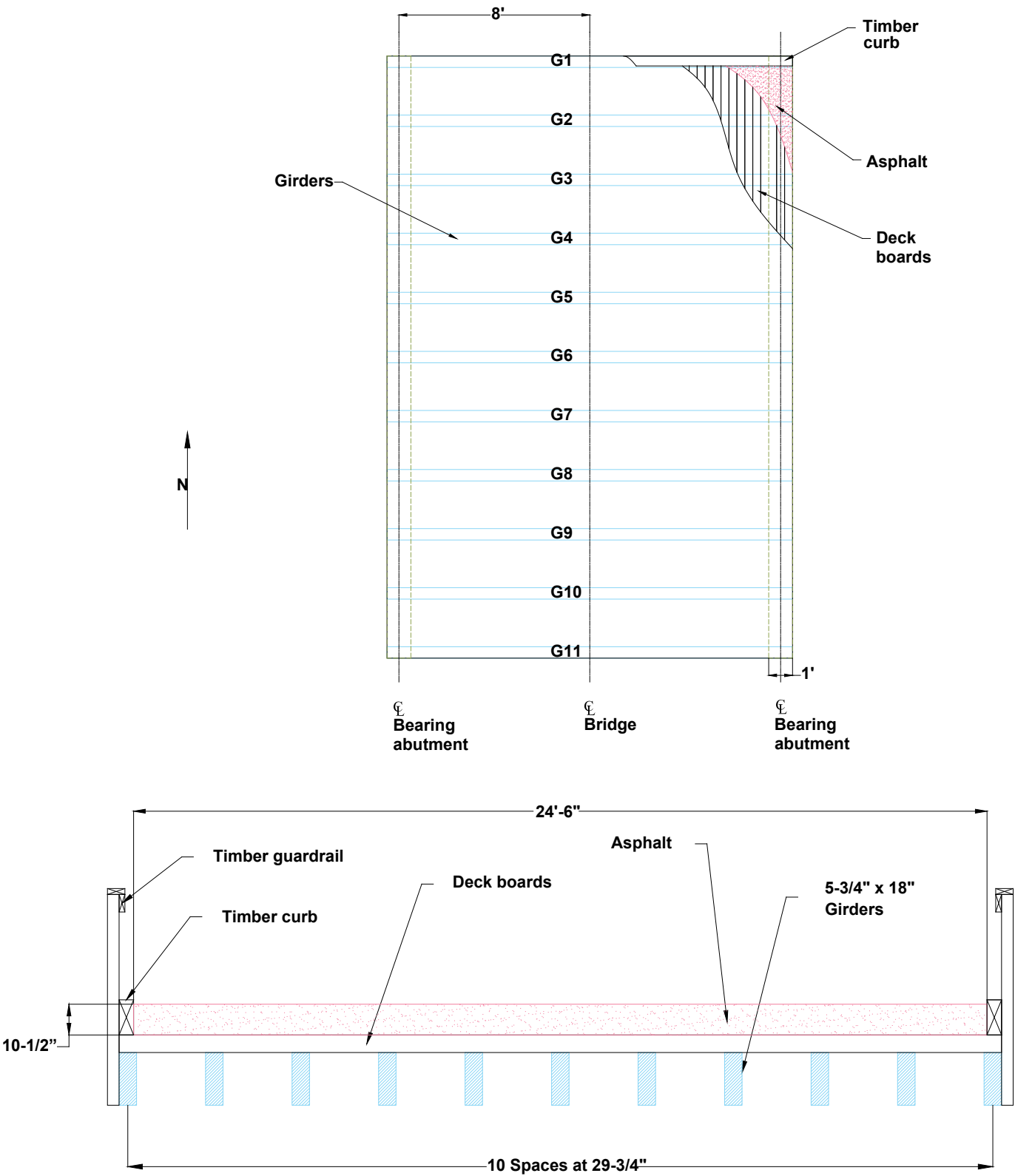


Figure L4. Plan and profile layout of Colorado Bridge.

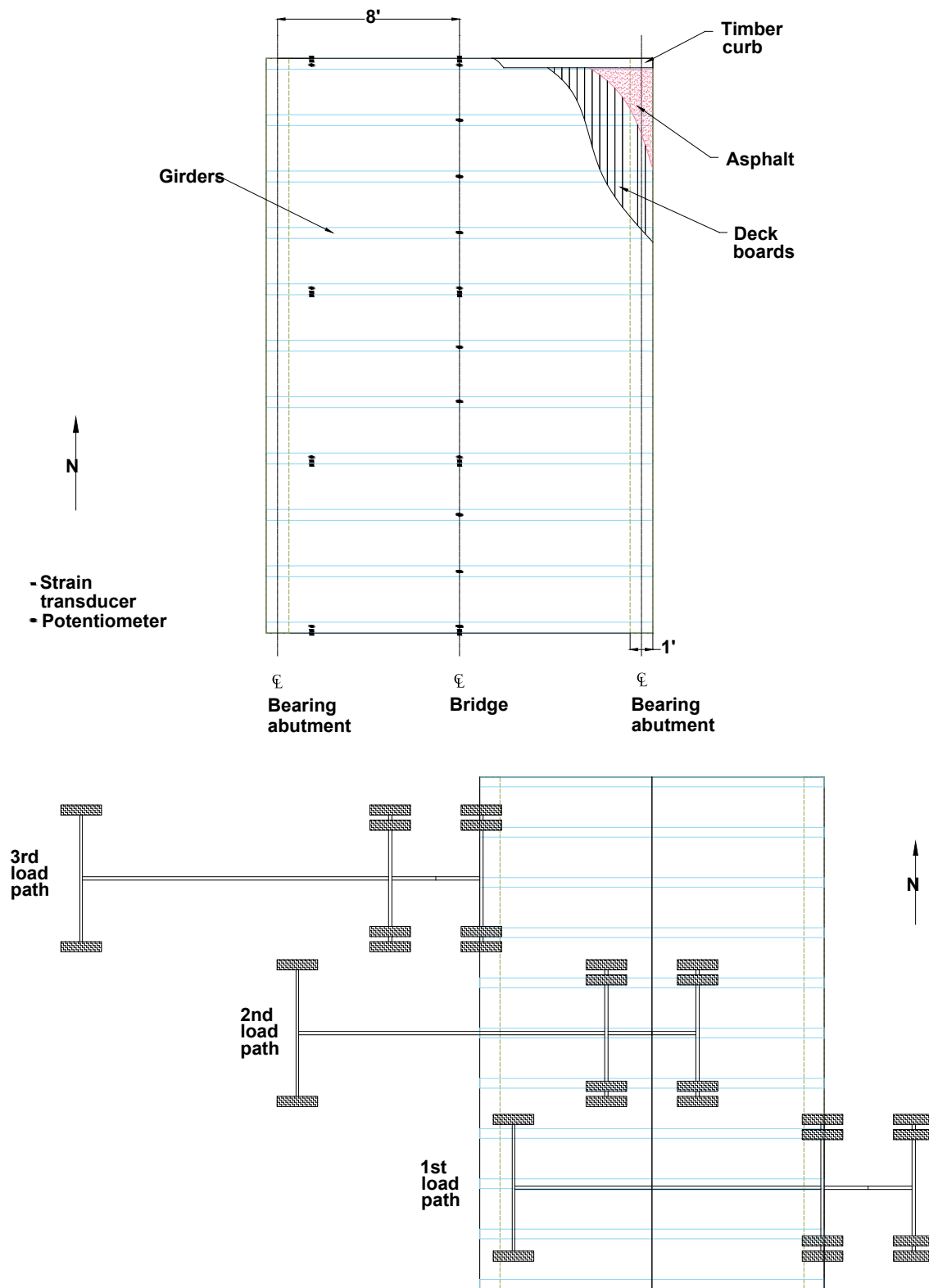


Figure L5. Instrumentation and load paths of Colorado Bridge.



Figure L6. Deflection instrumentation.



Figure L7. Strain transducers.

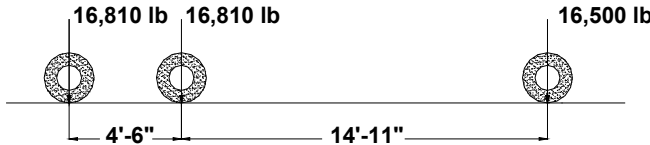


Figure L8. Truck configuration and axle loads.



Figure L9. Tandem-axle load truck.

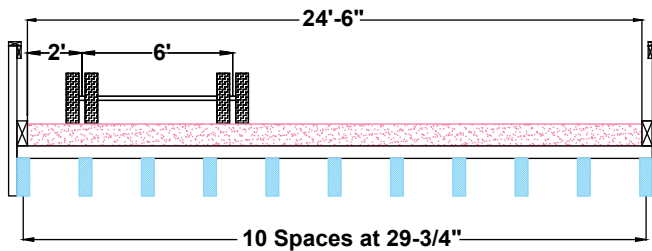


Figure L10. Transverse truck position—load path 1.

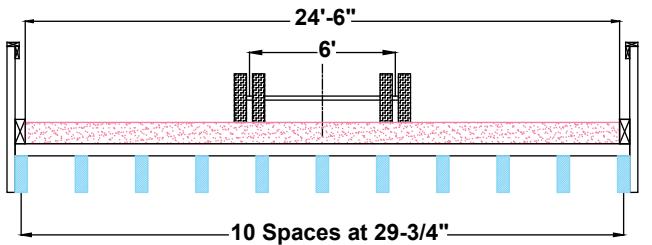


Figure L11. Transverse truck position—load path 2.

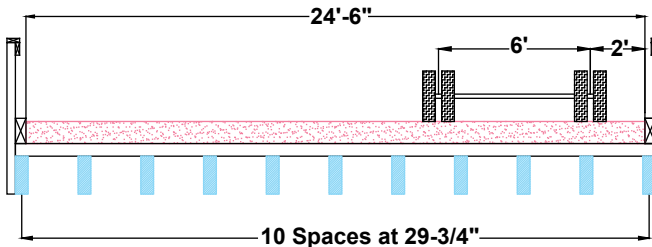


Figure L12. Transverse truck position—load path 3.



Figure L13. Transverse wearing surface cracking.



Figure L14. Checking in exterior girders.

could not be verified by the ISU team. Some cracking in the exposed ends of the deck boards was present, though the deck appeared to be in good condition, there was no visible detachment of the deck boards from the girders, and all deck boards were securely fastened.

Superstructure

Moderate checking was present in most girders, and the exterior girders were in worse condition than the rest, presumably a result of more exposure to weathering conditions. The checks in the exterior girders were deep in some locations and should be closely monitored with future inspections. If

checking becomes severe, degradation effects can be accelerated further and the structural integrity of the girder could be compromised. Figure L14 shows the typical checking in the exterior girders. The girder bearing on the sill was sufficient, and no misalignment was observed.

Overall Structure

The overall structure is in satisfactory condition, and structurally the bridge is sound. No odor like anise or wintergreen signifying fungal growth was present. There was no evidence of insect or mechanical degradation. Exposed timber members looked to be weathered and subjected to ultraviolet degradation. The substructure also showed signs of moderate checking even though much of the substructure was not visible due to a significant amount soil deposited beneath the bridge. The timber railing should be watched for further degradation because some of the posts are split.

Results

The following presents the results of the static load testing of the Colorado Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures L15 to L17 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck, one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures L15 to L17, and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table L1. Each passing of the three load paths is illustrated in Figures L18 to L20. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table L2 lists the n value for the girder of most deflection for each load path.

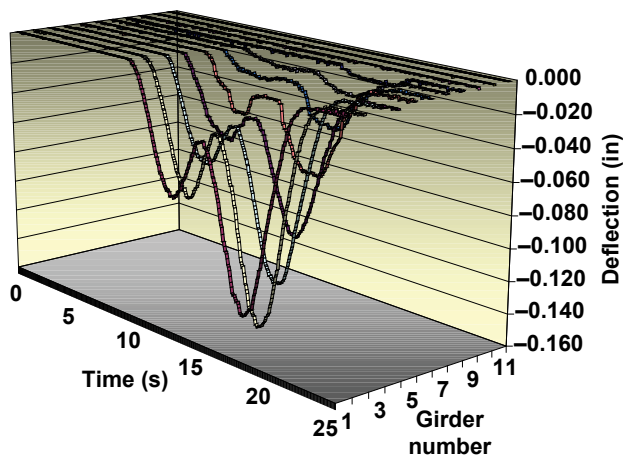


Figure L15. Deflections for load path 1.

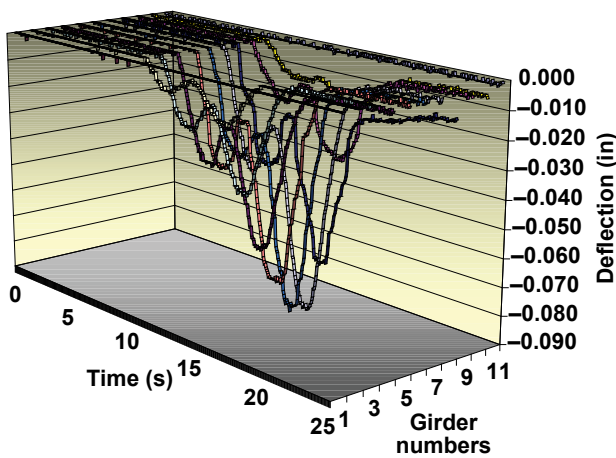


Figure L16. Deflections for load path 2.

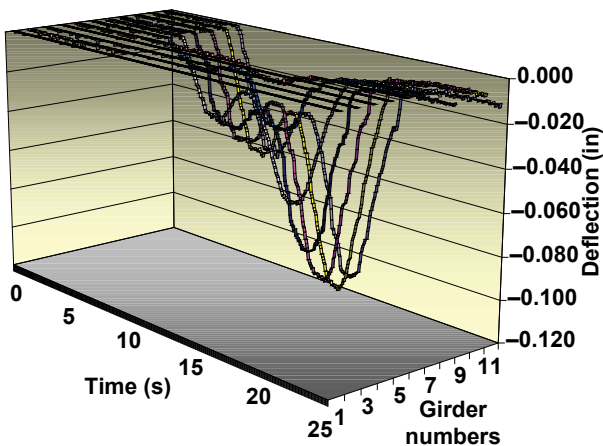


Figure L17. Deflections for load path 3.

Table L1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.145
Load path 2	0.087
Load path 3	0.115

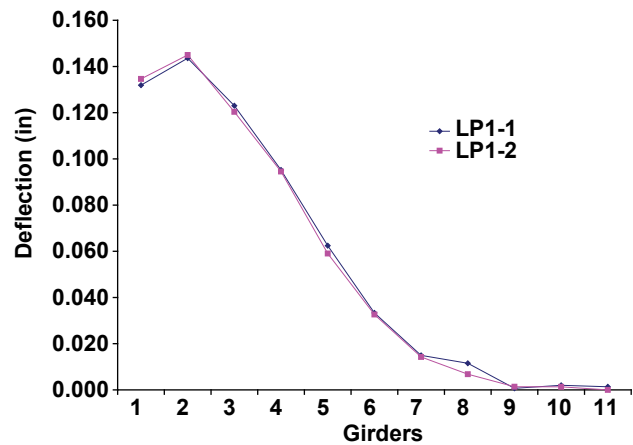


Figure L18. Maximum deflections for load path 1.

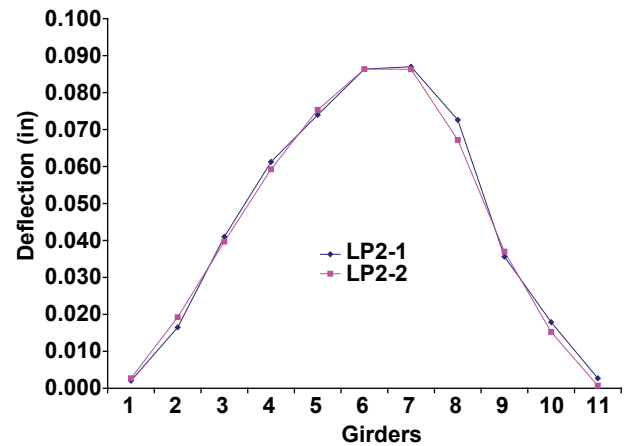


Figure L19. Maximum deflections for load path 2.

The minimum n value of the three load paths is 810. This value is greater than the minimum recommended value for timber girders. In fact, all the n values are greater than the recommended n values stated in Table L2.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure L21 shows the load fractions for each girder for each load path.

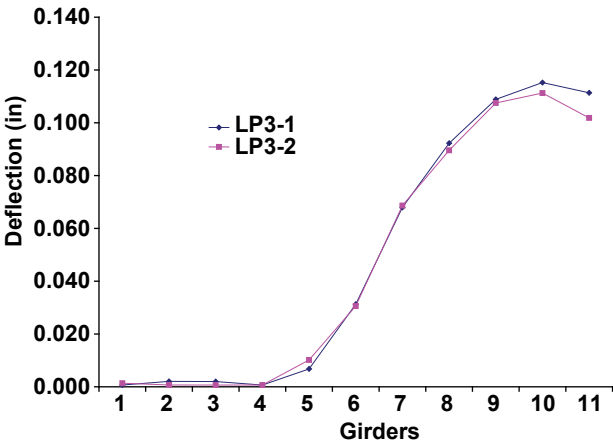


Figure L20. Maximum deflections for load path 3.

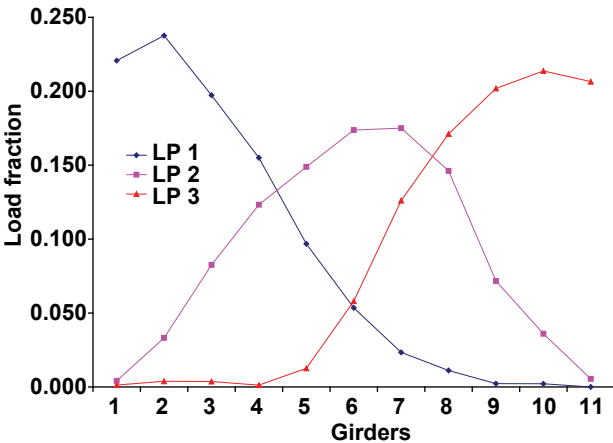


Figure L21. Load fractions for each load path.

Table L2—Most critical *n* value

	<i>n</i> value for girder of most deflection for each load path
Load path 1	810
Load path 2	1,350
Load path 3	1,019

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges are $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure L22 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures L23 to L25 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table L3.

The maximum differential deflection of 0.037 in. occurs in load path 3 and equals 0.015 in. per ft of girder spacing. It does not appear to be an issue, because it is a relatively small value. The same is true for load paths 1 and 3, for which the maximum differential deflections are both around 0.04 in. If the differential deflections were large, the possibility exists that the load was not well distributed

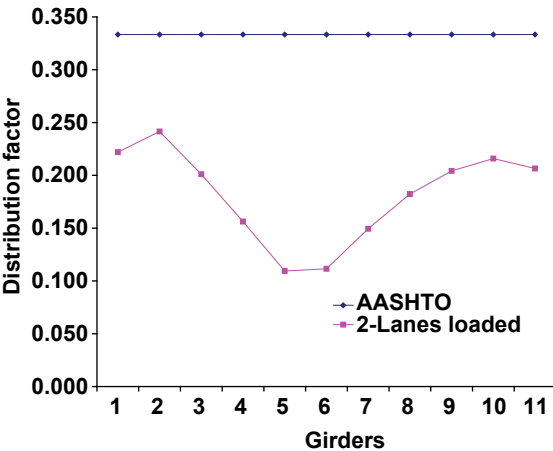


Figure L22. AASHTO design live load distribution.

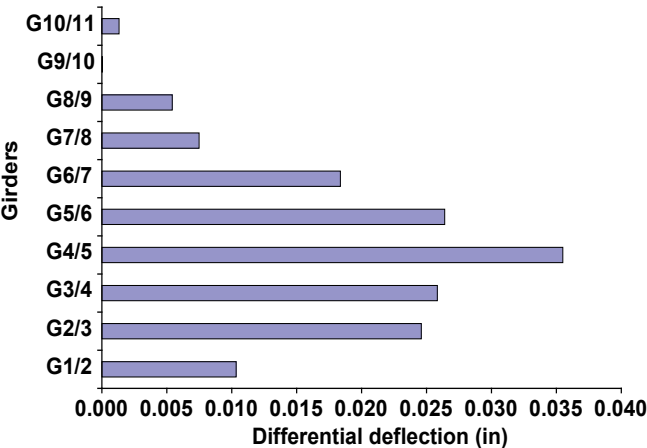


Figure L23. Differential deflections for load path 1.

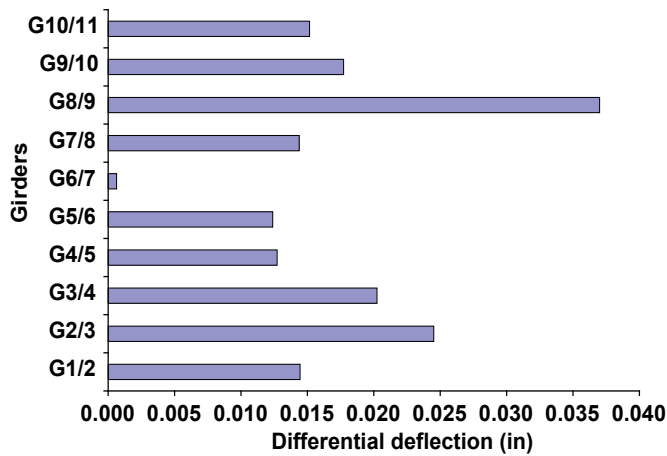


Figure L24. Differential deflections for load path 2.

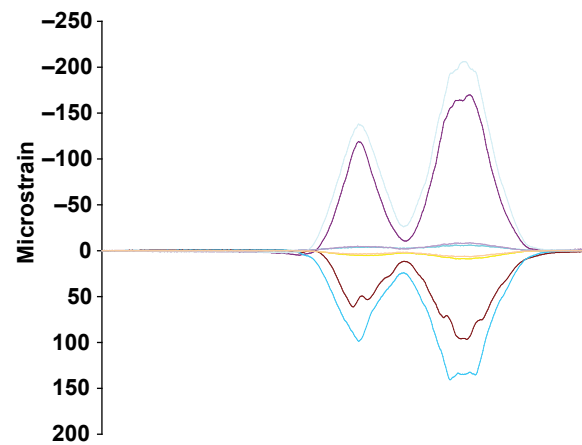


Figure L26. Strain at midspan for load path 1.

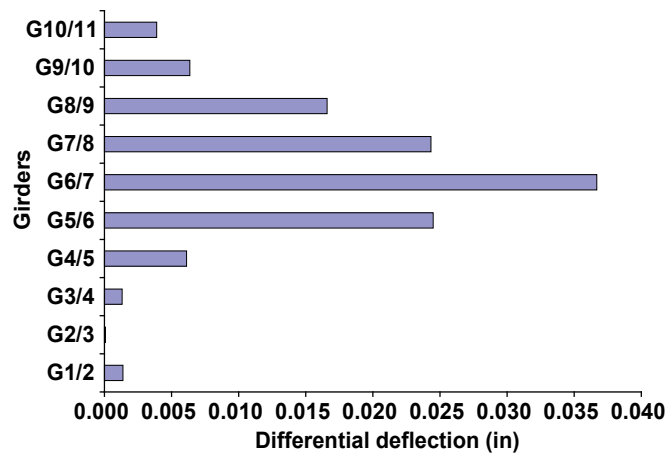


Figure L25. Differential deflections for load path 3.

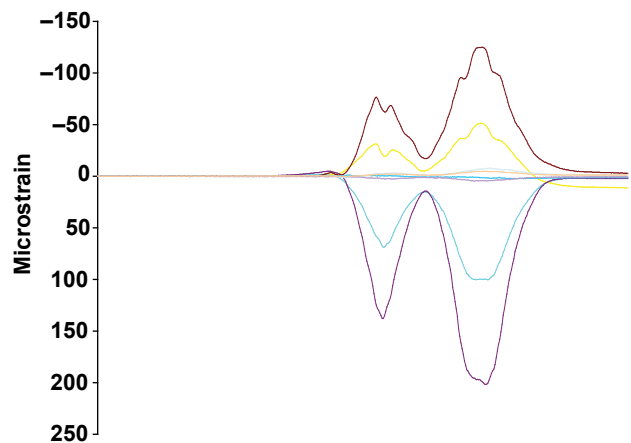


Figure L27. Strain at midspan for load path 2.

Table L3—Maximum differential deflection

	Maximum differential deflections at midspan between adjacent girders (in.)
Load path 1	0.035
Load path 2	0.036
Load path 3	0.037

transversely between these two girders or the assumption that both girders are of equal stiffness was false.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at

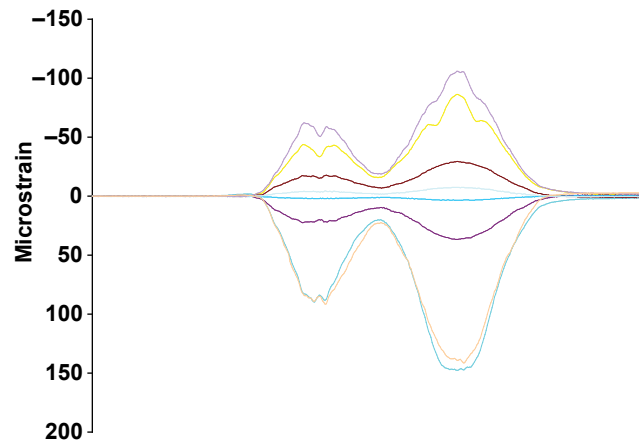


Figure L28. Strain at midspan for load path 3.

midspan, with compression and tensile strains of 206 and 202 microstrain, respectively. The strain plot at midspan is shown in Figures L26 to L28 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains,

constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures L26 to L28 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck; that is, the girders act independently of the deck when subjected to bending.

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 5, and 11 at the midspan and west abutment. The bottom of the deck between girders 1 and 2, 5 and 6, and 10 and 11 was measured at midspan. Measurements ranged from 10.6% to 25.7%. The moisture content measurements are summarized in Table L4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because timber strength is not critically affected until deflections become excessive. Also, the girder deflections do not exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability.

Exceeding the live load deflection recommendations can have adverse affects on the structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produces a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded as asphalt has very little fatigue resistance. Numerous problems associated with cracking exist including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. A benefit of the bridge location is that conditions are ideal for seepage to quickly evaporate because of the more arid climate. As a result, any water seepage through the deck will be prone to evaporation before permeation of the girders.

Table L4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, west abutment	17.2
Girder 1, midspan	15.4
Girder 5, west abutment	13.0
Girder 5, midspan	10.6
Girder 11, west abutment	19.3
Girder 11, midspan	17.2
Bottom of deck between girders 1 and 2	19.1
Bottom of deck between girders 5 and 6	25.7
Bottom of deck between girders 10 and 11	18.2

It would suggest that the wearing surface may show transverse cracking if deflections exceeded the recommended live load limit state. Even so, through visual inspection, transverse cracks in the wearing surface were found. The wearing surface of this particular bridge is in satisfactory condition, though close attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was within that limit. It could be argued that the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges. Despite that, one should note that the load was not well distributed across the bridge as evident by Figure L22, where it is seen that the girders opposite the truck path carry minimal load.

Strain data for timber bridges should be considered supplementary because the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures L26 to L28 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile

strains obtained from the three load paths are 206 and 203 microstrain, respectively. These strains equate to maximum stresses of 237 and 234 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 353 and 348 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the climate in southern Colorado, lower moisture contents were expected and also found, except for one deck location. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

Aside from the higher measurement in one deck location, the moisture content values were all within a couple percentage points of one another. This shows that none of the tested areas are subjected to vastly different amounts of moisture.

Conclusions

Several methods of condition and performance investigation were performed on the Colorado Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of immediate concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Some transverse cracking in the wearing surface was observed. Even so, seepage through the wearing surface and into the deck boards and girders was not evident.

A fair amount of checking is occurring throughout the bridge structure. The affects of the southern Colorado climate, and weathering is apparent in most exposed timber elements.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed

code provisions, though the load is not particularly well distributed across the bridge. Deflection values at midspan were within all of the recommended maximum values.

Appendix M—Colorado Bridge No. P-19-AS, Colorado Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Figure M1. Colorado Bridge location.

Abstract

The Colorado Bridge is a single-span timber girder bridge with a bituminous wearing surface located in Las Animas County, Colorado. The bridge was load tested and visually assessed as part of a research project through the United States Department of Agriculture (USDA) – Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: Mark Nord, Colorado Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing.

Background

The location of Colorado state bridge P-19-AS, hereinafter referred to as the Colorado Bridge, is shown in Figure M1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Colorado Bridge was built in 1930 and is located in Las Animas County in southern Colorado, approximately 3 miles east of Trinidad on State Highway 160. Currently, the bridge is not posted.



Figure M2. Bridge substructure.

Bridge Description

The Colorado Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface. The bridge length measures 21 ft–0 in. from the west backwall to the east backwall. The bridge width measures 24 ft–2 in. from inside of curb to inside of curb and 25 ft–3 in. from outside of rail to outside of rail. The substructure consists of solid timber posts and sills seated on concrete (see Fig. M2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by bolts into the exterior girders along with bolts into the curb, which is seated and bolted to the top of the deck, as shown in Figure M3.

Girders measure 21 ft–0 in. from end to end and have a clear span of 19 ft–0 in. A total of 12 girders, spaced 26-3/4 in.



Figure M3. Colorado Bridge parapet support.

center-to-center, measuring 6 by 18 in. in cross-section, are present and are seated and toe-nailed to the 12- by 12-in. timber sills with spikes. The deck consists of individual 3- by 6-in. nominal boards laid upon the short face transverse to the longitudinal girder direction. Overlaying the deck is a 5-in.-thick layer of asphalt wearing surface. Figure M4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure M5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 12 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure M6.

Strain transducers were attached to girder numbers 1, 3, 5, 7, 9, and 11 with 1 being the outside girder on the north side of the bridge and 11 being the outside girder on the south side of the bridge. The midspan and one abutment were instrumented (see Fig. M5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill. Figure M7 shows a typical setup of strain transducers near the girder ends.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the Colorado Department of Transportation. Dimensions of the truck are shown in Figure M8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–6 in.; the distance between the forward most rear axle and the front axle hubs measured 14 ft–11 in. Exact weight of the truck was 49,120 lb, where the total rear weight equaled 32,620 lb and the front axle weight was 16,500 lb. Assuming equal weights on each rear axle, the rear axles weighed 16,810 lb. The axle weights are shown in Figure M8. The truck used for the load testing is shown in Figure M9.

Three load paths were considered when testing the bridge (see Figs. M10 to M12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the north curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the south curb. For all load paths, the dump truck was driven at a crawl speed from west to east and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on July 1, 2004 was obtained from the Colorado DOT (CDOT). This report was reviewed and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

Some transverse cracking in the wearing surface reflecting the transverse deck boards beneath was observed by the ISU research team. Cracks have also formed at the transition between the roadway and the bridge, and these cracks could pose future problems if not monitored and repaired. Moisture could seep through these cracks and advance degradation in the backwalls, girder ends, and substructure. Also, if the wearing surface becomes uneven at the transition, the bridge could be subjected to unnecessary effects from dynamic loads which can be magnified by higher vehicle speeds on this roadway. Figure M13 shows some of the cracking observed. Aside from the transverse cracking, the wearing surface looked to be in good condition.

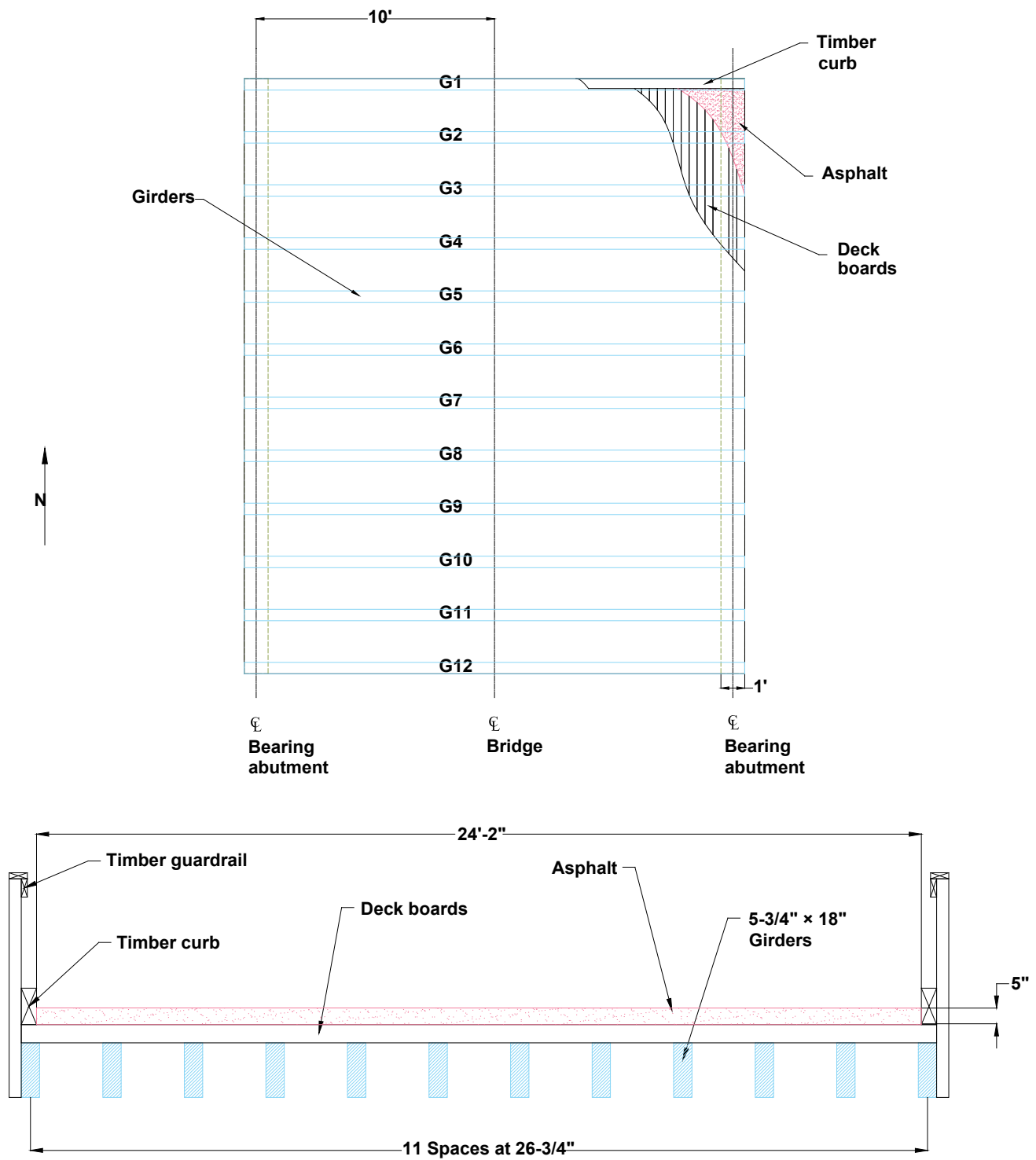


Figure M4. Plan and profile layout of Colorado Bridge.

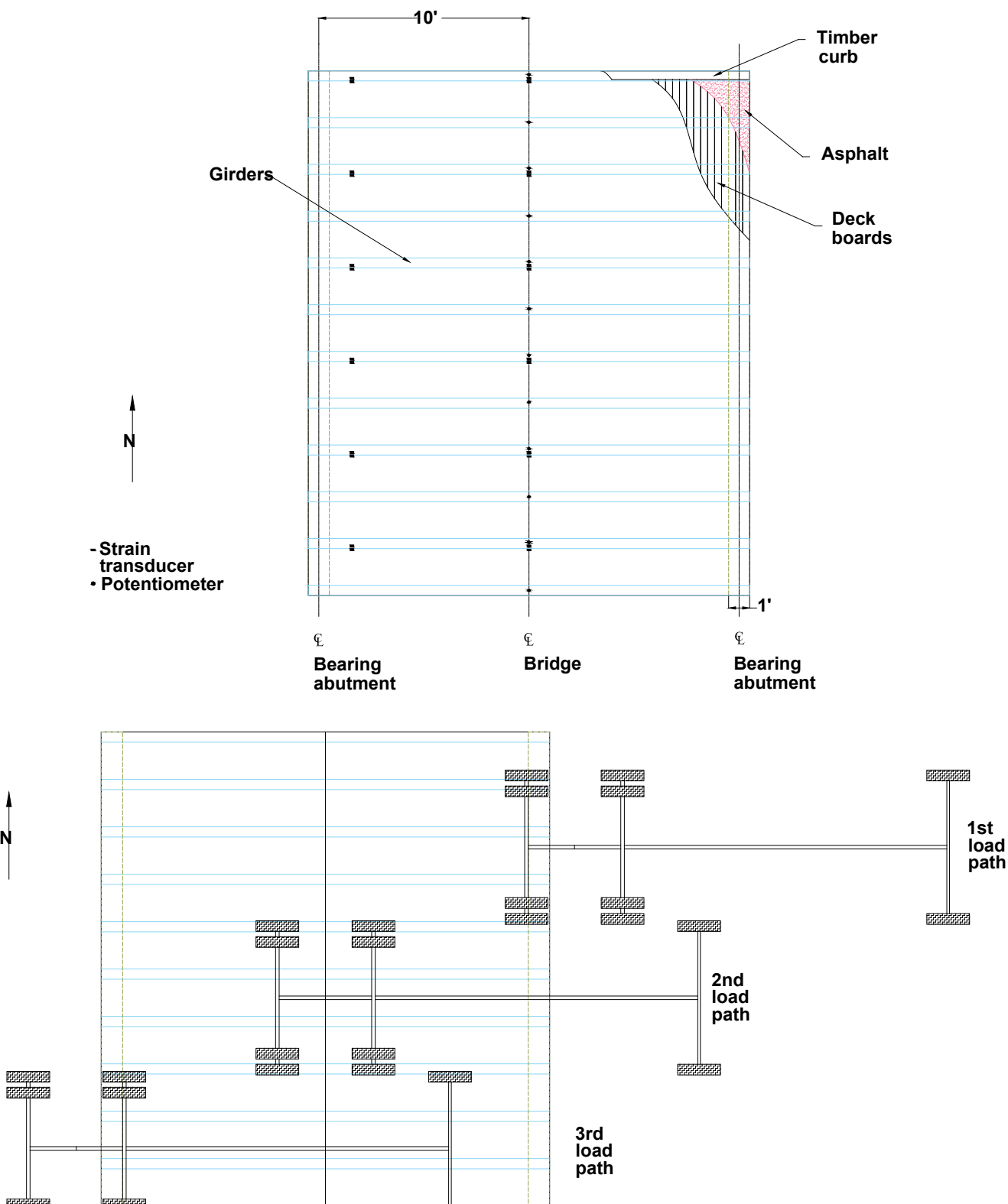


Figure M5. Instrumentation and load paths of Colorado Bridge.



Figure M6. Deflection instrumentation.



Figure M7. Strain transducers.

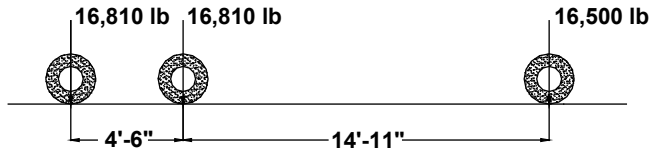


Figure M8. Truck configuration and axle loads.



Figure M9. Tandem-axle load truck.

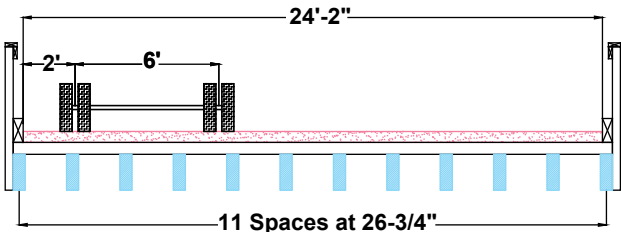


Figure M10. Transverse truck position—load path 1.

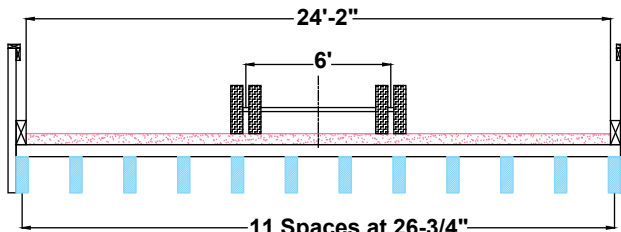


Figure M11. Transverse truck position—load path 2.

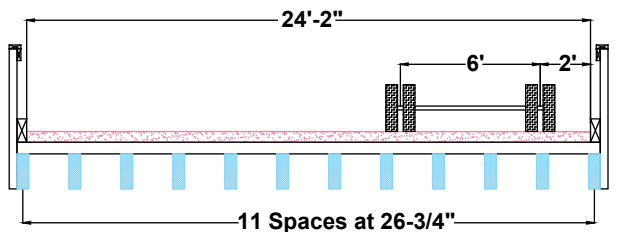


Figure M12. Transverse truck position—load path 3.



Figure M13. Transverse wearing surface cracking.



Figure M14. Checking in exterior girders.

Deck

According to the CDOT 2004 report, minor water stains were present on the untreated bottom of the deck. This condition was verified by the ISU team. The evidence of moisture was consistent with the moisture measurements obtained throughout the deck, which will be discussed later. Some cracking in the exposed ends of the deck boards was present, though the deck appeared to be in good condition, there was no visible detachment of the deck boards from the girders, and all deck boards were securely fastened.

Superstructure

Slight checking was present in most girders, and the exterior girders were in worse condition than the rest, presumably a result of more exposure to weathering conditions. The checks in the exterior girders were deep in some locations and should be closely monitored with future inspections. If checking becomes severe, degradation effects can be accelerated further and the structural integrity of the girder could

be compromised. Figure M14 shows the typical checking in the exterior girders. The girder bearing on the sill was sufficient, and no misalignment was observed.

Overall Structure

The overall structure is in satisfactory condition and structurally the bridge is sound. No odor like anise or winter-green signifying fungal growth was present. There was no evidence of insect or mechanical degradation. Exposed timber members looked to be weathered and subjected to ultraviolet degradation. The substructure also showed signs of moderate checking even though much of the substructure was not visible due to a significant amount of soil deposited beneath the bridge. Signs of rot are present at the base of the backwalls and timber columns. The timber railing and curb should be watched for further degradation because some of the posts are split and there is checking in the curb.

Results

The following presents the results of the static load testing and finite element modeling of the Colorado Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures M15 to M17 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures M15 to M17, and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table M1. Each passing of the three load paths is illustrated in Figures M18 to M20. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table M2 lists the n value for the girder of most deflection for each load path.

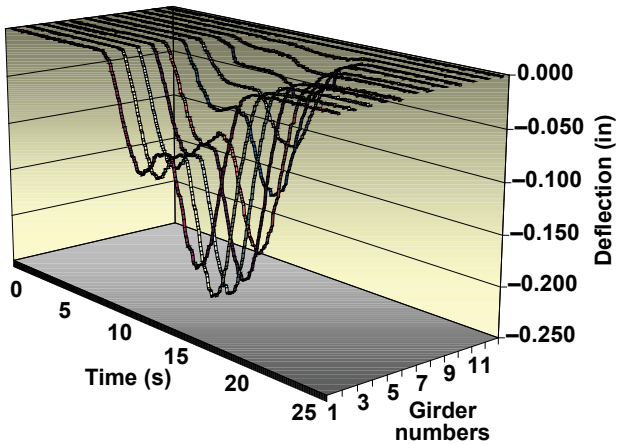


Figure M15. Deflections for load path 1.

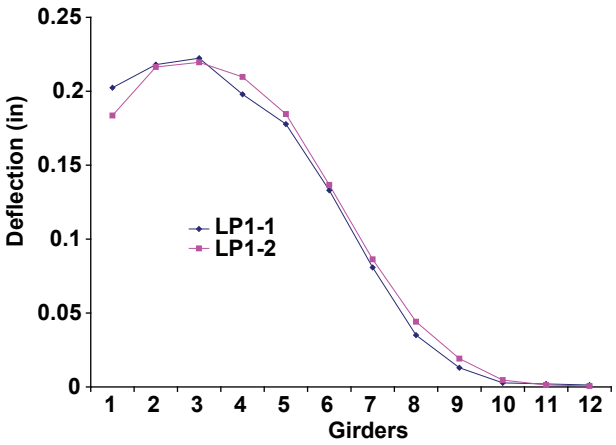


Figure M18. Maximum deflections for load path 1.

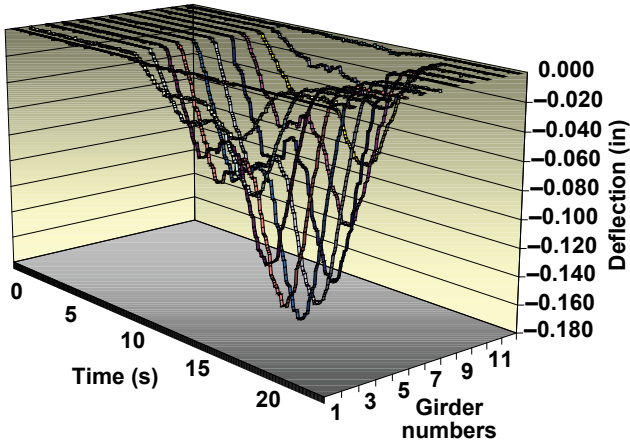


Figure M16. Deflections for load path 2.

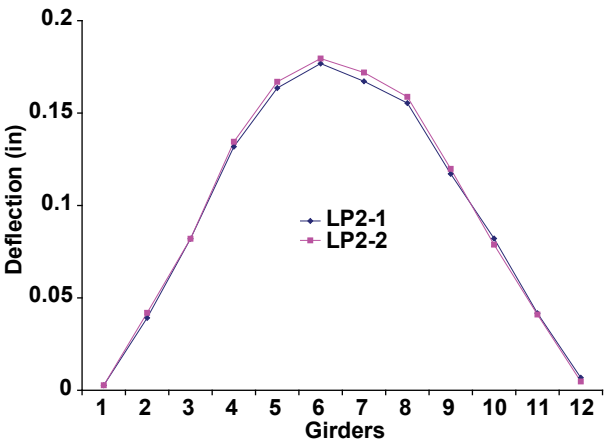


Figure M19. Maximum deflections for load path 2.

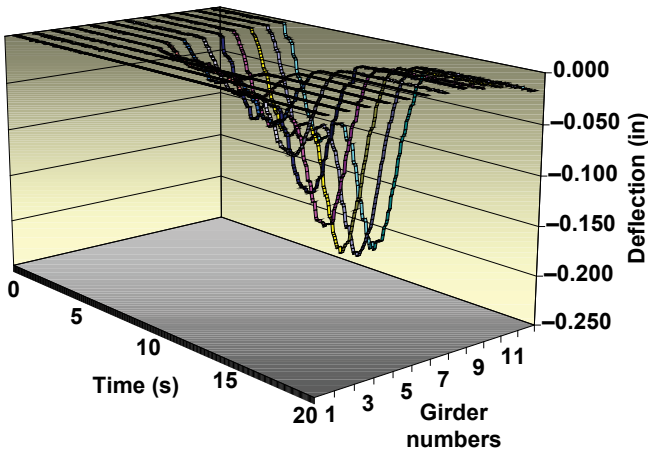


Figure M17. Deflections for load path 3.

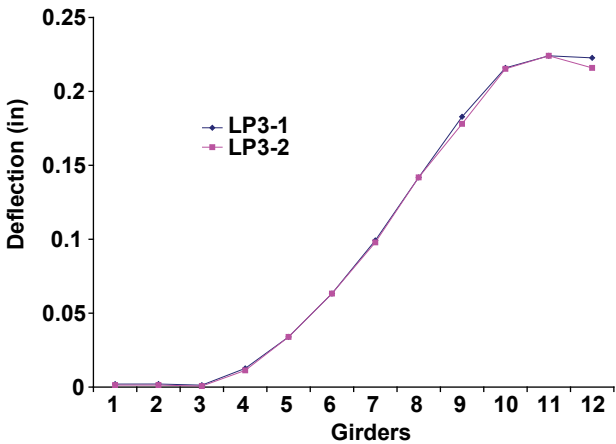


Figure M20. Maximum deflections for load path 3.

Table M1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.222
Load path 2	0.180
Load path 3	0.224

Table M2—Most critical n value

	n value for girder of most deflection for each load path
Load path 1	669
Load path 2	829
Load path 3	664

The minimum n value of the three load paths is 664. This value is greater than the minimum recommended value for timber girders. In fact, all the n values are greater than the recommended n values stated in Table M2.

Distribution Factors

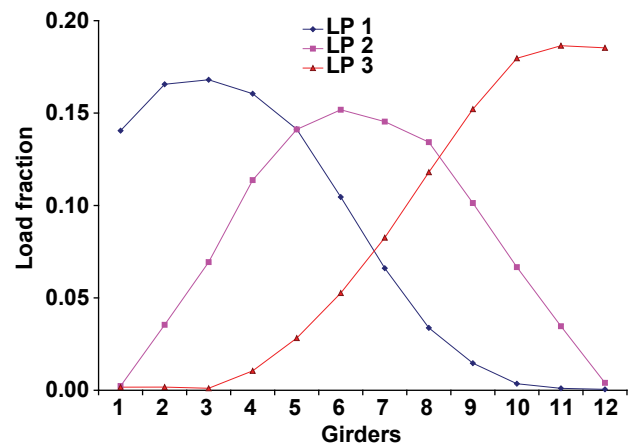
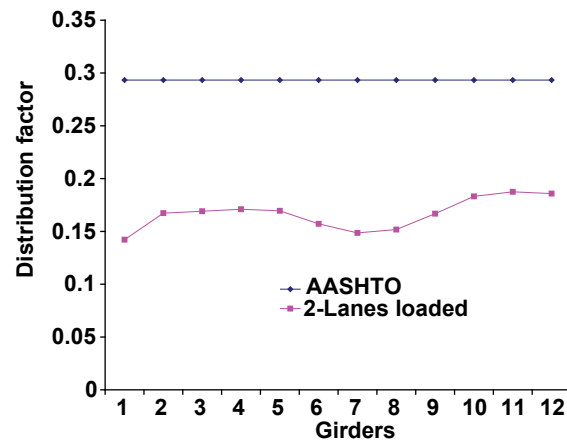
Appendix B discusses load distribution factors and the calculation of load fractions. Figure M21 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges are $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, anywhere S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure M22 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures M23 to M25 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table M3.

The maximum differential deflection of 0.052 in. occurs in load paths 1 and 2, equaling 0.023 in. per ft of girder spacing. It does not appear to be an issue as it is a relatively

**Figure M21. Load fractions for each load path.****Figure M22. AASHTO design live load distribution.**

small amount. The same is true for load paths 2 and 3 as the maximum differential deflections are both around 0.05 in. If the differential deflections were large, the possibility exists that the load was not well distributed transversely between these two girders or the assumption that both girders are of equal stiffness was false.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 189 and 262 microstrain, respectively. The strain plot at midspan is shown in Figures M26 to M28 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph.

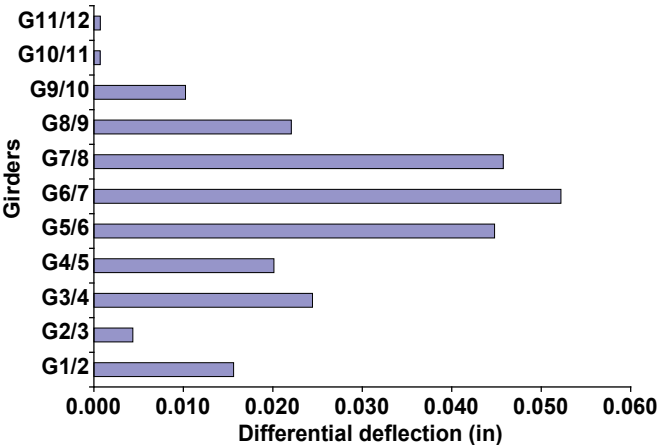


Figure M23. Differential deflections for load path 1.

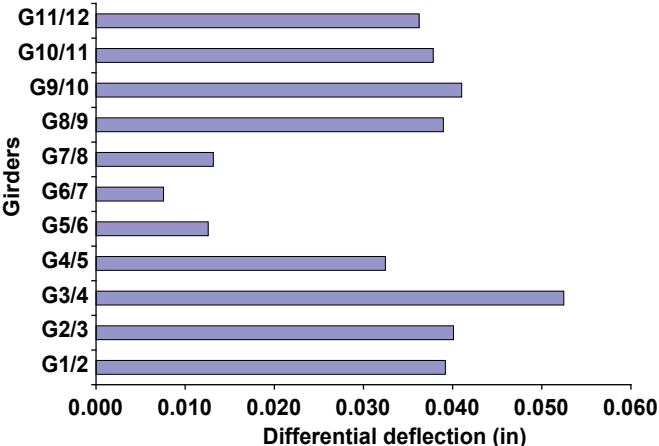


Figure M24. Differential deflections for load path 2.

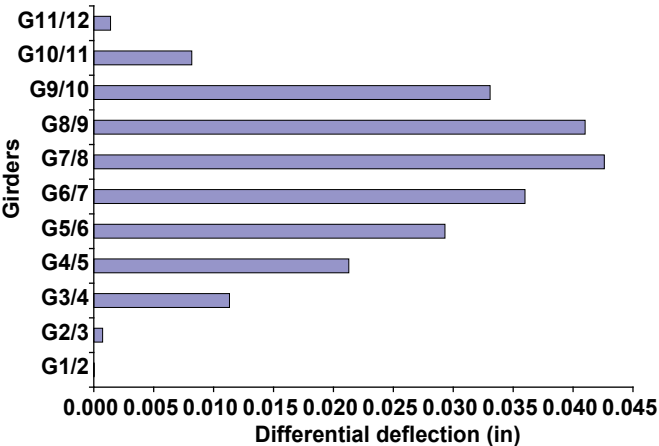


Figure M25. Differential deflections for load path 3.

It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be

Table M3—Maximum differential deflection	
Maximum differential deflections at midspan between adjacent girders (in.)	
Load path 1	0.052
Load path 2	0.052
Load path 3	0.043

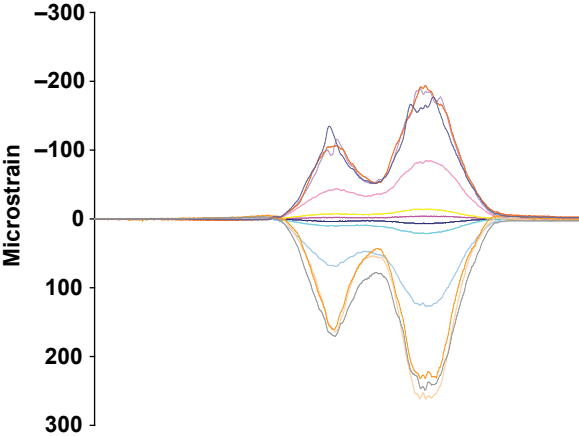


Figure M26. Strain at midspan for load path 1.

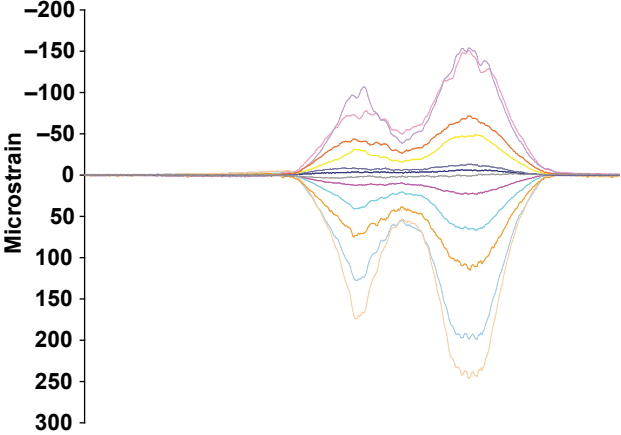


Figure M27. Strain at midspan for load path 2.

used to determine the stress. The resulting stresses are discussed in the following section.

Figures M26 to M28 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck (that is, the girders act independently of the deck when subjected to bending).

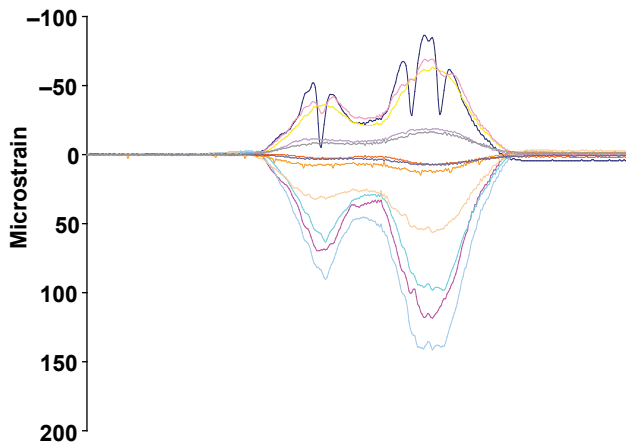


Figure M28. Strain at midspan for load path 3.

Table M4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, west abutment	9.6
Girder 1, midspan	9.9
Girder 6, west abutment	10.1
Girder 6, midspan	10.4
Girder 12, west abutment	16.6
Girder 12, midspan	20.9
Bottom of deck between girders 1 and 2	25.4
Bottom of deck between girders 5 and 6	24.1
Bottom of deck between girders 11 and 12	24.1

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 6, and 12 at midspan and west abutment. The bottom of the deck between girders 1 and 2, 5 and 6, and 11 and 12 was measured at midspan. Measurements ranged from 9.6% to 25.4%. The moisture content measurements are summarized in Table M4.

Finite Element Analysis

A finite element model was developed (see Fig. M29) for the Colorado Bridge using ANSYS, a well-known finite element software. The objective was to create a model that would replicate field results when subjected to the same loading. After calibrating the model to the midspan deflection results obtained from the static load test, it was decided that the model would be subjected to a load simulating the AASHTO HS20 tandem-axle design vehicle. Deflection and tensile strain results at midspan were obtained from the model.

Figures M30 to M32 show the calibrated model results when subjected to the same load as that during the static load test. Notice the similarities between each plot.

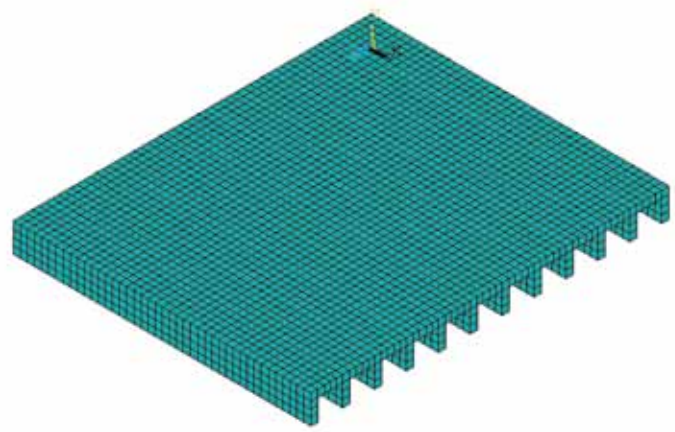


Figure M29. Finite element model.

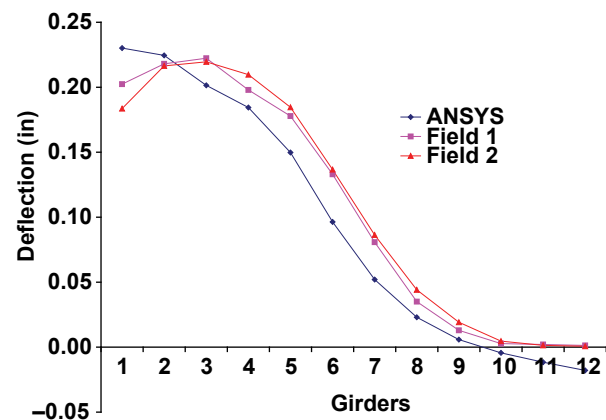


Figure M30. ANSYS calibration results load path 1.

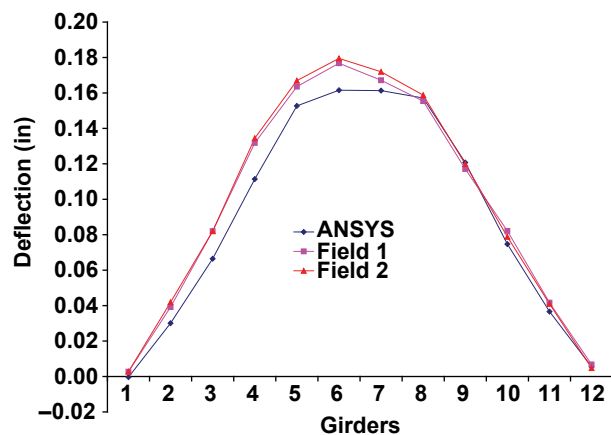


Figure M31. ANSYS calibration results load path 2.

Figure M33 shows the maximum deflections at midspan after subjecting the finite element model to the load of the AASHTO HS20 tandem-axle design vehicle traveled along each load path.

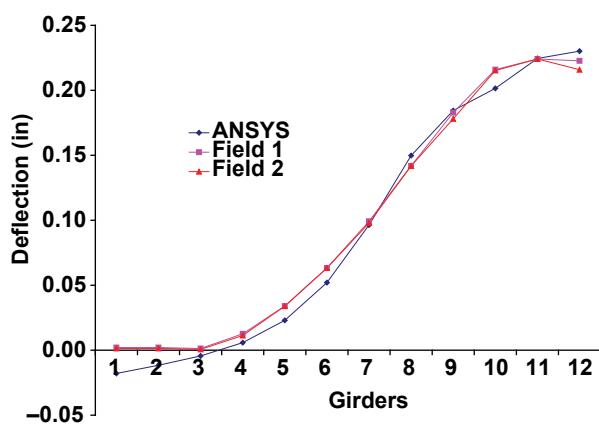


Figure M32. ANSYS calibration results load path 3.

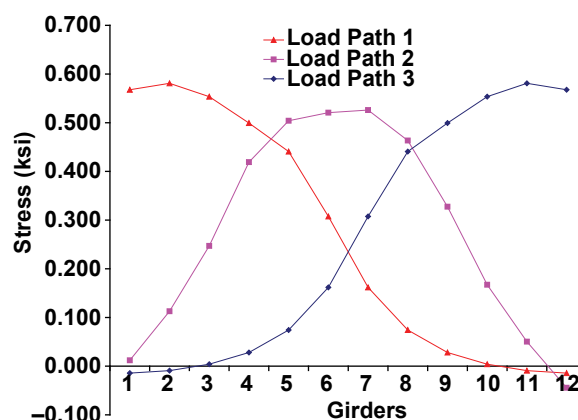


Figure M34. ANSYS tensile stress for each load path when subjected to HS20 tandem-axle design vehicle.

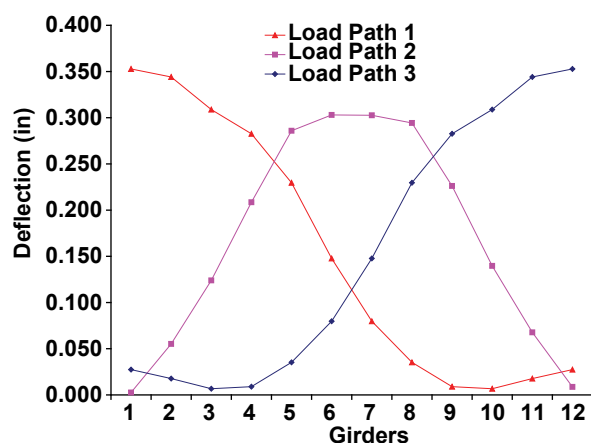


Figure M33. ANSYS deflection results for each load path when subjected to HS20 tandem-axle design vehicle.

Figure M34 shows the maximum tensile stresses at midspan due to the AASHTO HS20 tandem axle design vehicle traveled along each load path.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, moisture content, and finite element modeling.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because timber strength is not critically affected until deflections become excessive. Also, the girder deflections do not exceed the values necessary to meet recommended limit states for live load deflection derived primarily from wearing surface degradation and maintainability.

Exceeding the live load deflection recommendations can have adverse affects on structure fasteners, wearing surface,

and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produce a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded because asphalt has very little fatigue resistance. Numerous problems associated with cracking exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay

The wearing surface may show transverse cracking if deflections exceeded the recommended live load limit state. Even so, through visual inspection, transverse cracks in the wearing surface were found. The wearing surface of this particular bridge is in satisfactory condition, though close attention should be paid to the existing transverse cracks and the effects thereof.

Differential deflections between adjacent girders could also result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report, having investigated other timber bridge research, have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was within that limit. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges. Despite that, one should note that the load was not well distributed across the bridge, as is evident in Figure M22, where it is seen that the girders opposite the truck path carry minimal load.

Strain data for timber bridges should be considered supplementary because the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures M26 to M28 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 189 and 262 microstrain, respectively. These strains equate to maximum stresses of 217 and 301 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 281 and 389 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the climate in southern Colorado, lower moisture contents were expected and also found except for measurements obtained from the deck boards and girder 12. The deck boards consistently had higher moisture content measurements, and this could be the result of transverse cracking and seepage through the wearing surface. Girder 12 measurements were not as elevated, though an increase was observed between girder 12 and the other girders.

The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

Aside from the higher measurements in the deck boards and girder 12, the moisture content percentages were all within a couple percentage points of one another. This would suggest that the other tested areas are not subjected to vastly different amounts of moisture.

Maximum midspan stresses and deflections were obtained from the finite element model. The maximum deflection was 0.353 in. from load paths 1 and 3, and 0.303 in. from load path 2. Much like the normalized vehicle loading, the results met the recommended limit states for live load deflection. The maximum stresses at midspan for load paths 1 and 3 and load path 2 were 581 and 526 lb/in², respectively. Much like the stresses obtained from the normalized vehicle loading, these values were within the values set by allowable stress design. The finite element model is consistent with the

results discussed previously; recommended live load deflection limits and allowable stresses were not exceeded.

Conclusions

Several methods of condition and performance investigation were performed on the Colorado Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of immediate concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Some transverse cracking in the wearing surface was observed. Some seepage through the wearing surface and into the deck boards and girders was evident. Moisture contents were elevated within the deck board elements.

A fair amount of slight checking is occurring throughout the bridge structure, and more severe checking is occurring on the exterior faces of the outermost girders. The affects of the southern Colorado climate and weathering is apparent in most exposed timber elements.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Deflection values at midspan were within all of the recommended maximum values.

The finite element model yielded results that were consistent with the bridge performance under live load. Recommended live load deflection limits and allowable stresses at midspan were not exceeded.

Appendix N—Montana Bridge No. P00009040+01001, Montana Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Figure N1. Montana Bridge location.

Abstract

The Montana Bridge is a single-span timber girder bridge with a bituminous wearing surface located near Augusta, Montana. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: William Lay, Montana Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing.

Background

The location of Montana state bridge P00009040+01001, hereinafter referred to as the Montana Bridge, is shown in Figure N1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Montana Bridge was built in 1935 and is located approximately 1 mile northeast of Augusta, Montana, on US 287. Currently, the bridge is not posted.



Figure N2. Bridge substructure.

Bridge Description

The Montana Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface. The bridge length measures 20 ft–0 in. from the south backwall to the north backwall. The bridge width measures 21 ft–1 in. from inside of curb to inside of curb and 22 ft–0 in. from inside of rail to inside of rail. The substructure consists of solid timber posts and sills (see Fig. N2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by bolts into the exterior girders along with bolts into the curb, which is seated on top of the deck, as shown in Figure N3.



Figure N3. Montana Bridge parapet support.

Girders measure 20 ft–0 in. from end to end and have a clear span of 18 ft–0 in. A total of 12 girders, spaced 23-1/2 in. on average center-to-center, measuring 6 by 16-3/4 in. in cross-section, are present and are seated and toe-nailed to the 12- by 12-in. timber sills with spikes. The deck consists of individual 2- by 4-in. nominal boards laid upon the short face transverse to the longitudinal girder direction. Overlaying the deck is a 15-in.-thick layer of asphalt wearing surface. Figure N4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure N5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 12 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure N6.

Strain transducers were attached to girder numbers 1, 3, 5, 7, 9, and 11, with 1 being the outside girder on the west side of the bridge and 12 being the outside girder on the east side of the bridge. The midspan and one abutment were instrumented (see Fig. N5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill. Figure N7 shows a typical setup of strain transducers near the girder ends.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the Montana Department of Transportation. Dimensions of the truck are shown in Figure N8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–5 in.; the distance between the forward-most rear axle and the front axle hubs measured 13 ft–4 in. Exact weight of the truck was 39,020 lb, where the total rear weight equaled 28,080 lb and the front axle weight was 10,940 lb. Assuming equal weights on each rear axle, the rear axles weighed 14,040 lb. The axle weights are shown in Figure N8, and the load truck is shown in Figure N9.

Three load paths were considered when testing the bridge (see Figs. N10 to N12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the west curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the east curb. For all load paths, the dump truck was driven at a crawl speed from south to north, and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on August 20, 2004, was obtained from the Montana DOT (MDT). This report was reviewed, and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

Overall, the wearing surface looked to be in good condition. No cracking was observed in the wearing surface as a new chip seal had just been laid upon the time of the ISU team visual inspection. The wearing surface did have a significant thickness of 15 in., which effectively reduced the parapet height and added a large amount of dead weight to the bridge structure. This is evident in Figure N13 as the wearing surface is shown above the curb.

Deck

According to the MDT 2004 report, water appeared to be leaking through the decking. Some water staining was verified by the ISU research team, though this may have been

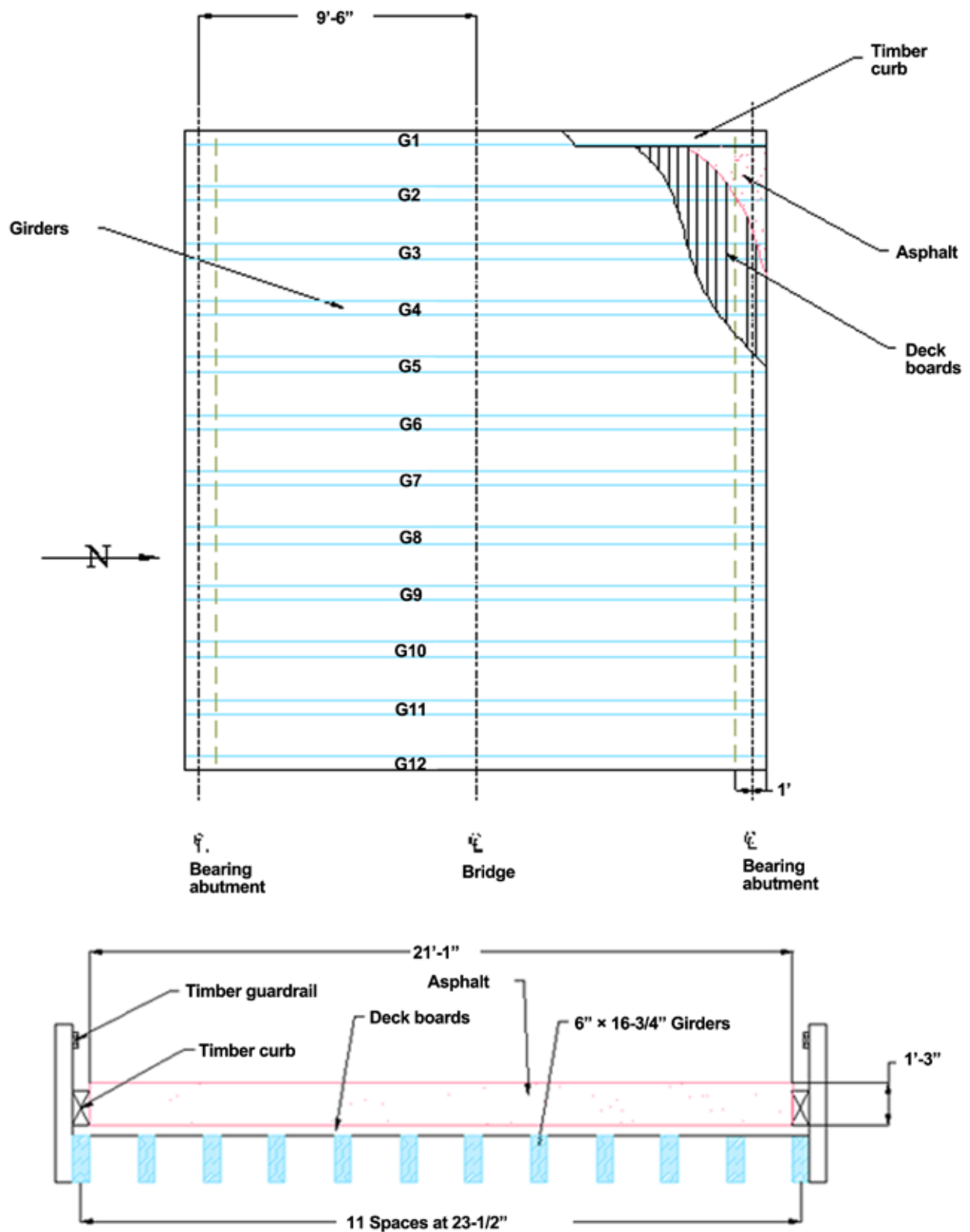


Figure N4. Plan and profile layout of Montana Bridge.

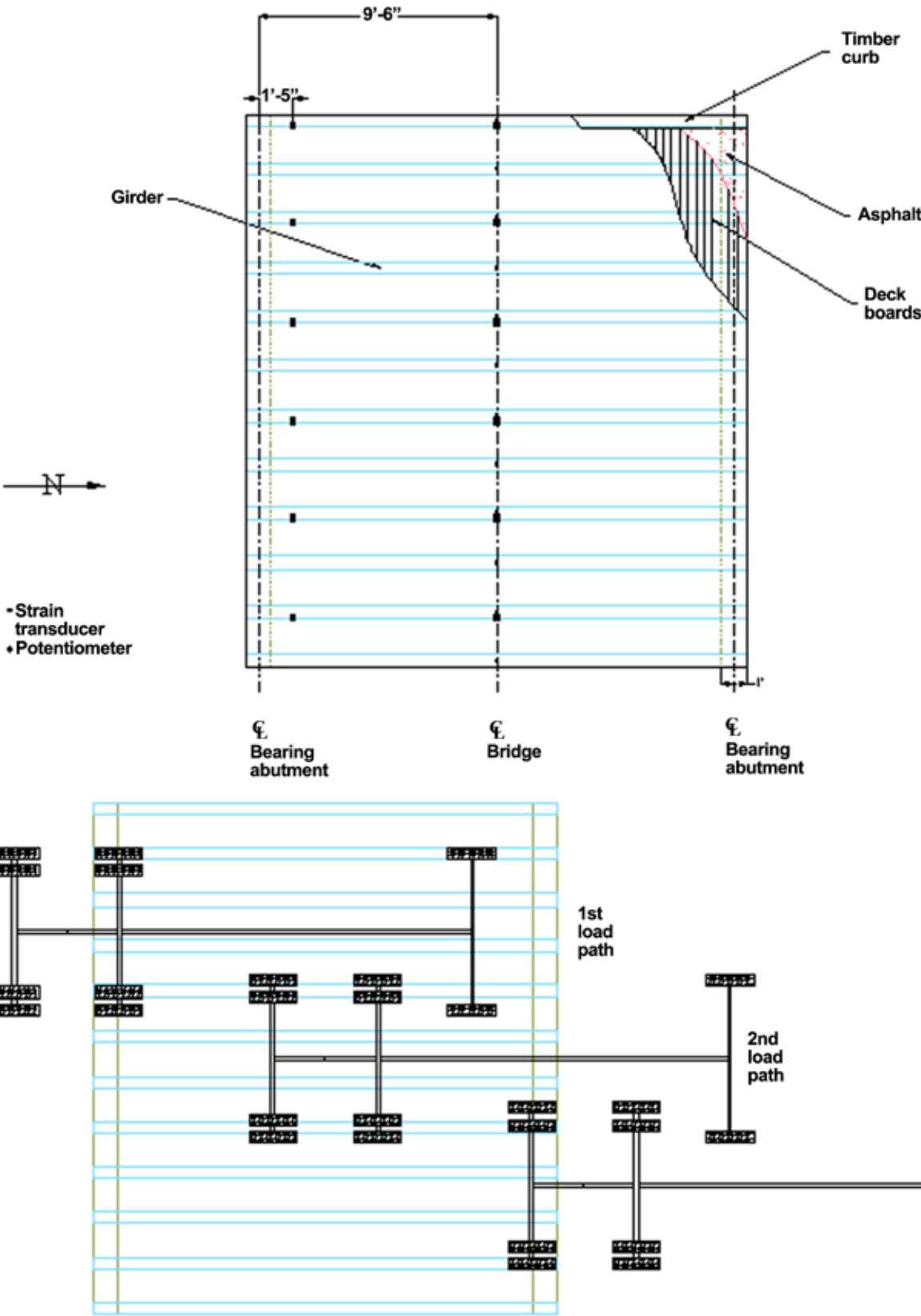


Figure N5. Instrumentation and load paths of Montana Bridge.



Figure N6. Deflection instrumentation.



Figure N7. Strain transducers.

from prior to the new chip seal. The ISU team could not verify if seepage continues to be a problem, though some white residue was forming between girders 1 and 2 and this may be a condition of continued seepage (see Fig. N14). The ends of the deck boards appeared in good condition, there was no visible detachment of the deck boards from the girders, and all deck boards were securely fastened.

Superstructure

The interior girders looked in good condition; no visual degradation was observed. Conversely, the exterior girders were in worse condition, presumably a result of more exposure to weathering conditions (see Fig. N15). The checks in the

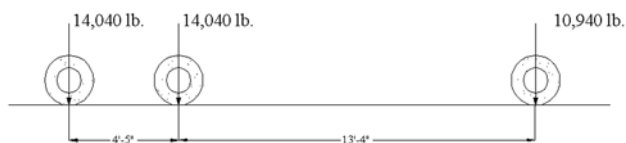


Figure N8. Truck configuration and axle loads.



Figure N9. Tandem-axle load truck.

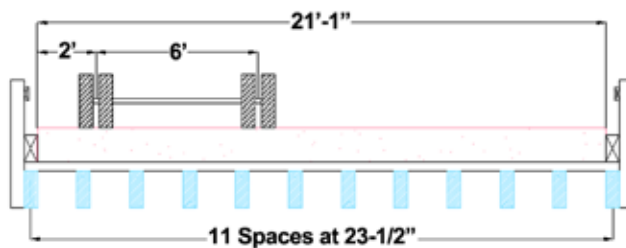


Figure N10. Transverse truck position—load path 1.

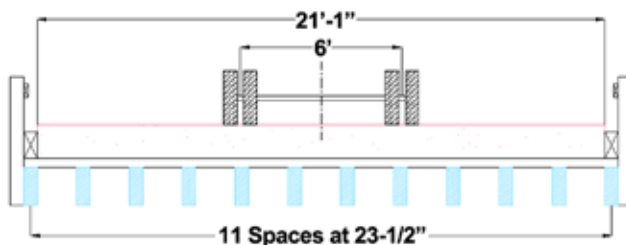


Figure N11. Transverse truck position—load path 2.

exterior girders were deep in some locations and should be closely monitored with future inspections. If checking becomes severe, degradation effects can be accelerated further and the structural integrity of the girder could be compromised. The girder bearing on the sill was sufficient, and no misalignment was observed.

Overall Structure

The overall structure was in satisfactory condition, and structurally the bridge was sound. No odor like anise or wintergreen signifying fungal growth was present. There was no

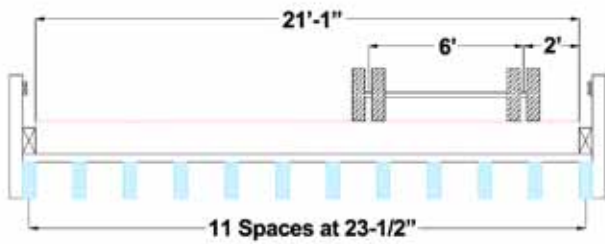


Figure N12. Transverse truck position—load path 3.



Figure N13. Wearing surface thickness.



Figure N14. White residue formation on deck.



Figure N15. Checking in exterior girders.

evidence of insect damage. Exposed timber members looked to be weathered and subjected to ultraviolet degradation, and the substructure also showed signs of moderate checking. It appeared that something hit the railing on the east side as it was significantly askew from the assumed original position. Also, the timber railing should be watched for further degradation as some of the posts are split.

Results

The following presents the results of the static load testing of the Montana Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures N16 to N18 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck, one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges, where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures N16 to N18 and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table N1. Each passing of the three load paths is illustrated in Figure N19 to N21. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for

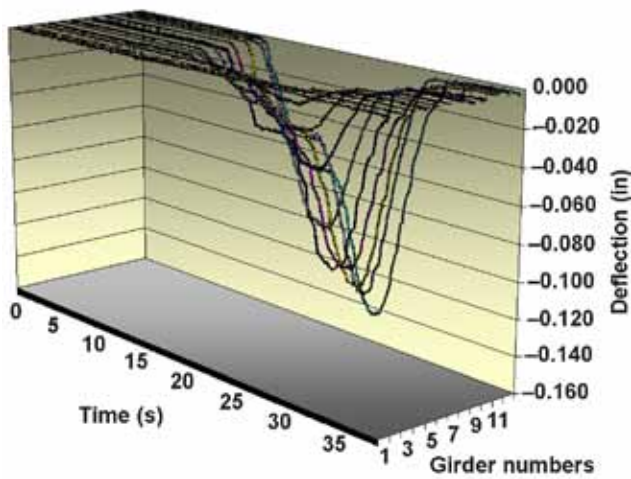


Figure N16. Deflections for load path 1.

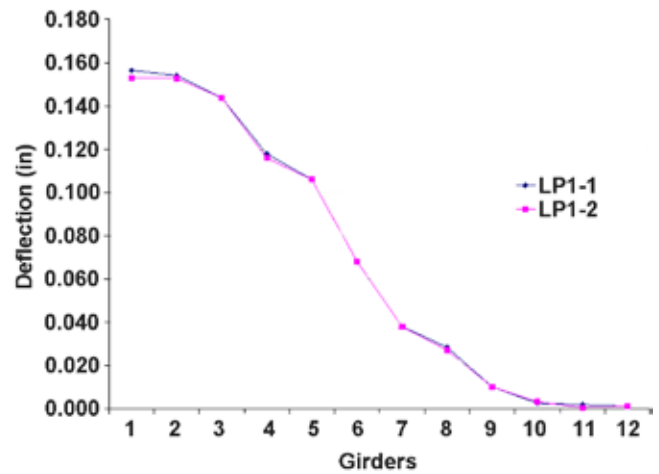


Figure N19. Maximum deflections for load path 1.

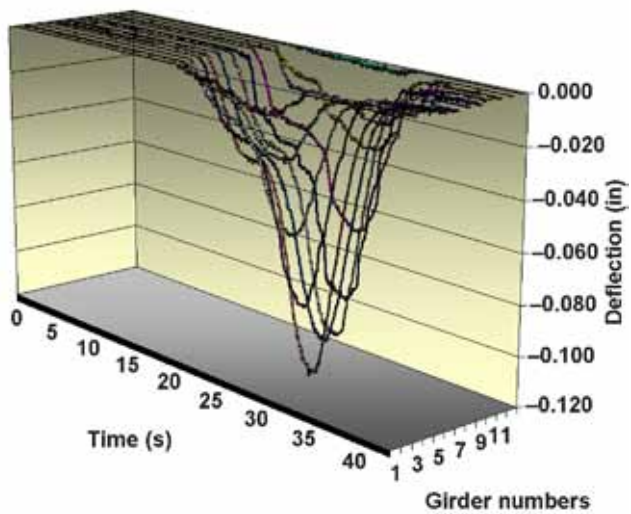


Figure N17. Deflections for load path 2.

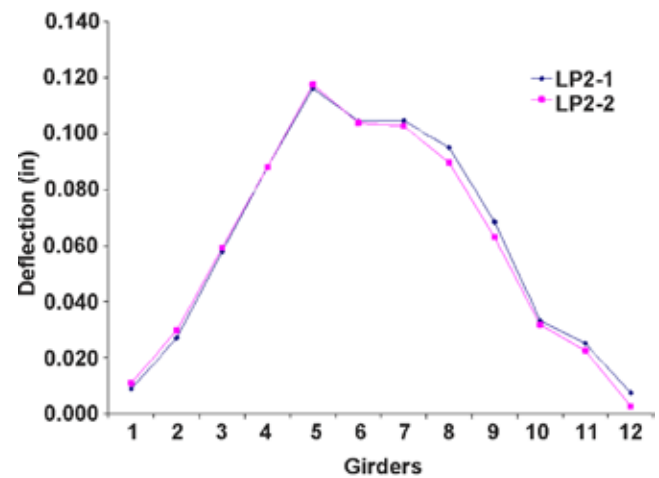


Figure N20. Maximum deflections for load path 2.

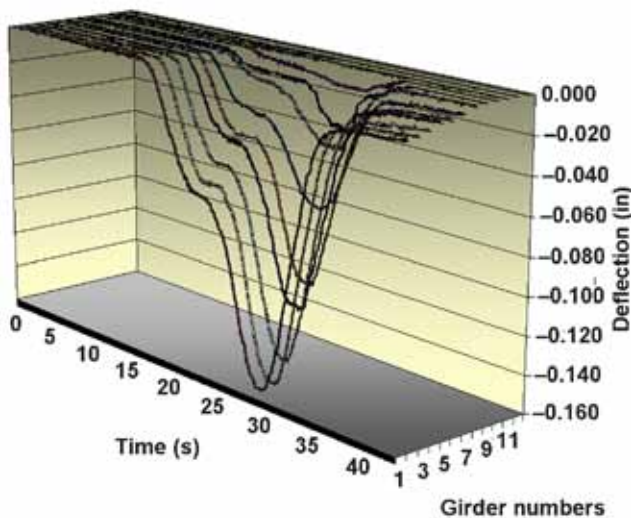


Figure N18. Deflections for load path 3.

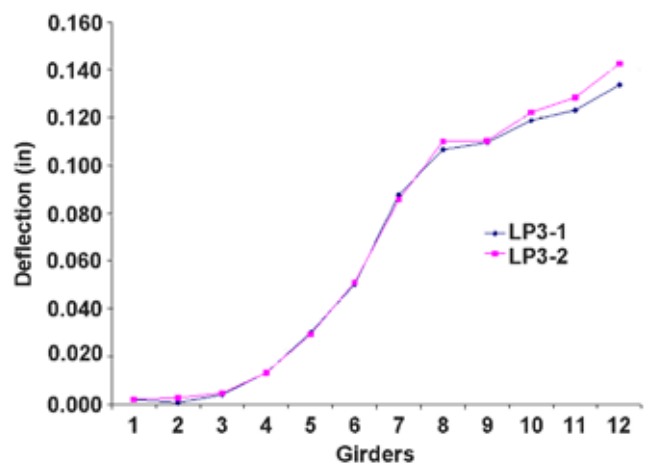


Figure N21. Maximum deflections for load path 3.

Table N1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.156
Load path 2	0.118
Load path 3	0.143

Table N2—Most critical *n* value

	<i>n</i> value for girder of most deflection for each load path
Load path 1	400
Load path 2	496
Load path 3	475

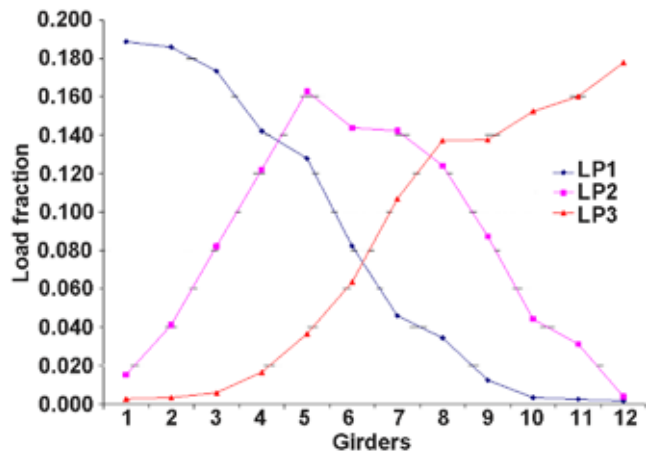


Figure N22. Load fractions for each load path.

each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table N2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths was 400. This value was greater than at least one of the minimum recommended values for timber girders. Each of the other load paths were greater than at least two of the recommended values stated in Table N2.

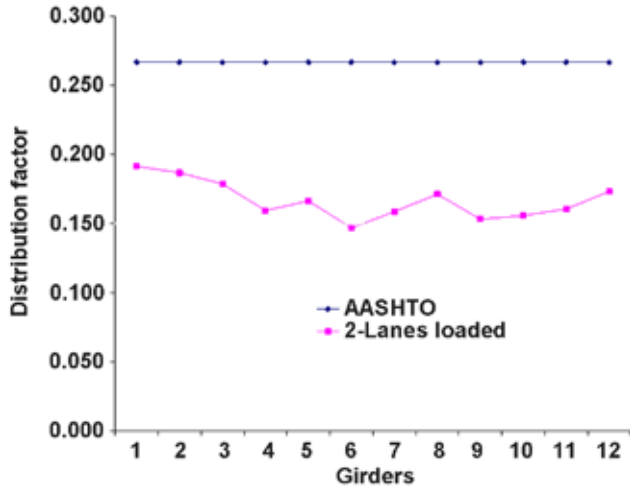


Figure N23. AASHTO design live load distribution.

Table N3—Maximum differential deflection

	Maximum differential deflection at midspan between adjacent girders (in.)
Load path 1	0.038
Load path 2	0.031
Load path 3	0.035

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure N22 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure N23 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures N24 to N26 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table N3.

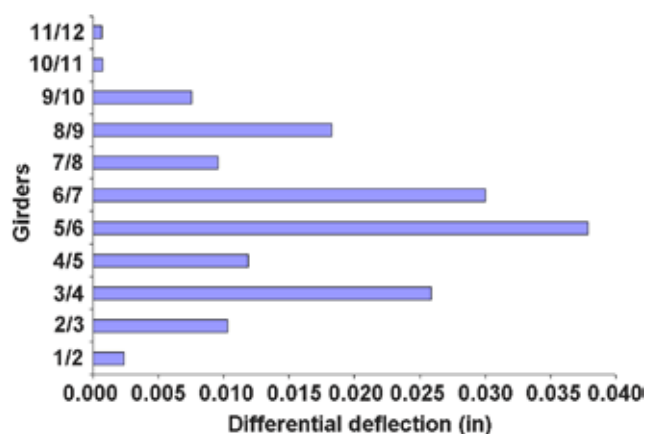


Figure N24. Differential deflections for load path 1.

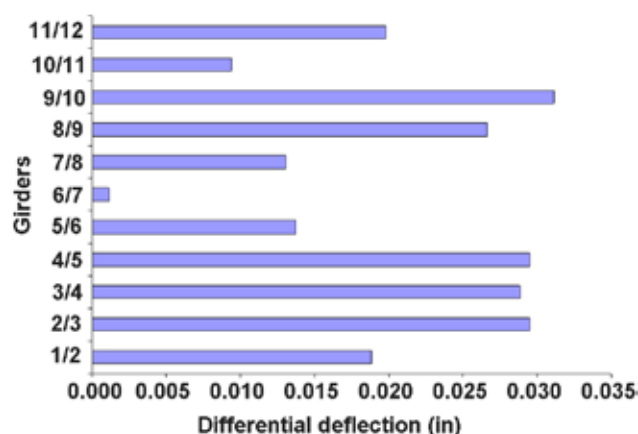


Figure N25. Differential deflections for load path 2.

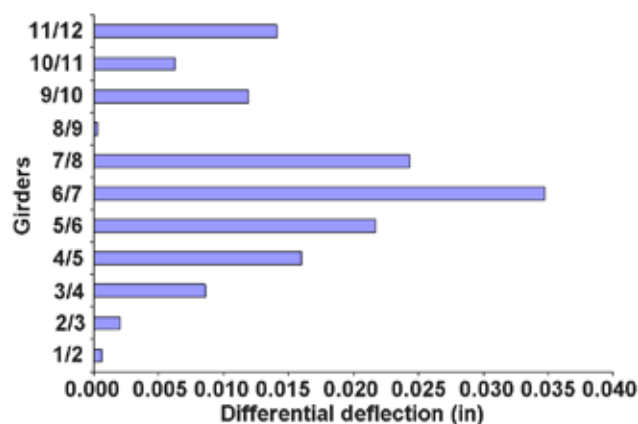


Figure N26. Differential deflections for load path 3.

The maximum differential deflection of 0.038 in. occurs in load path 1 and this equals 0.019 in. per ft of girder spacing. This does not appear to be an issue as it is a relatively small value. The same is true for load paths 1 and 3, as the maximum differential deflections are both around 0.03 in. If the differential deflections were large, the possibility exists that

the load was not well distributed transversely between these two girders or the assumption that both girders are of equal stiffness was false.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan, with compression and tensile strains of 145 and 211 microstrain, respectively. The strain plot at midspan is shown in Figures N27 to N29 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures 27 to 29 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little, if any, composite action with the deck (that is, the girders act independently of the deck when subjected to bending).

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 6, and 12 at the midspan and south abutment. The bottom of the deck between girders 1 and 2, 6 and 7, and 11 and 12 was measured at midspan. Measurements ranged from 9.9% to 15.0%. The moisture content measurements are summarized in Table N4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because timber strength is not critically affected until deflections become excessive. Also, each of the maximum girder deflections for each load path meets at least one recommended limit state for live load deflection derived primarily from wearing surface degradation and maintainability.

Exceeding the live load deflection recommendations can have adverse affects on the structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed

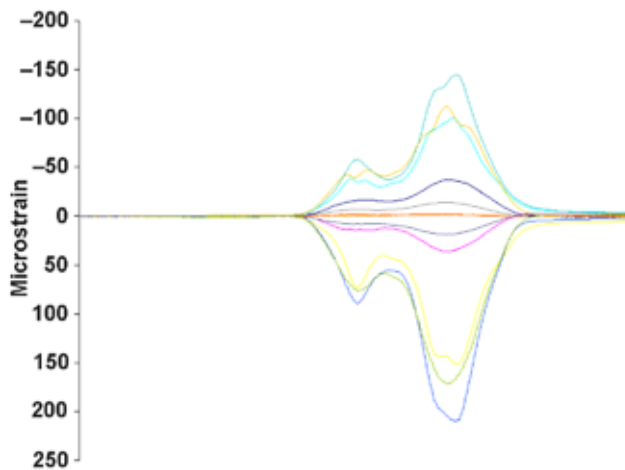


Figure N27. Strain at midspan for load path 1.

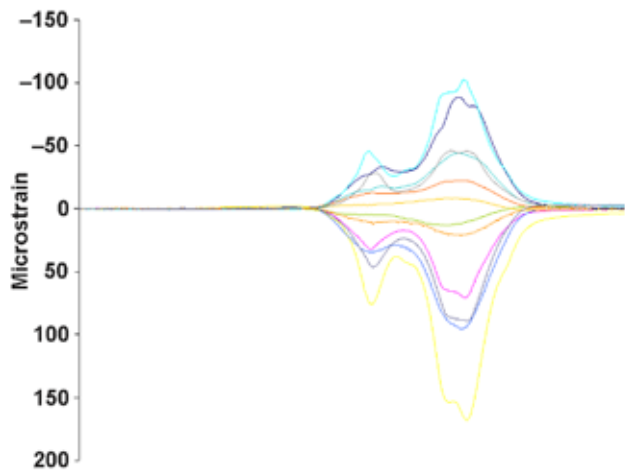


Figure N28. Strain at midspan for load path 2.

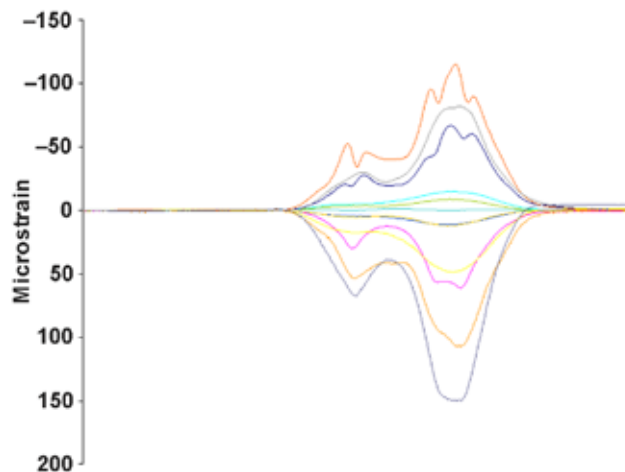


Figure N29. Strain at midspan for load path 3.

Table N4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, south abutment	11.3
Girder 1, midspan	10.2
Girder 6, south abutment	10.3
Girder 6, midspan	9.9
Girder 12, south abutment	13.7
Girder 12, midspan	13.9
Bottom of deck between girders 1 and 2	15.0
Bottom of deck between girders 6 and 7	11.6
Bottom of deck between girders 11 and 12	12.0

fasteners and wearing surface cracking produce a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded as asphalt has very little fatigue resistance. Numerous problems associated with cracking exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. A benefit of the bridge location is that conditions are ideal for seepage to quickly evaporate because of the more arid climate. As a result, any water seepage through the deck should be prone to evaporation before permeation of the girders occurs.

Through visual inspection, transverse cracks in the wearing surface were not found and the wearing surface was in good condition.

Differential deflections between adjacent girders could result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report, having investigated other timber bridge research, have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was within that limit. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures N27 to N29 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 145 and 211 microstrain, respectively. These strains equate to maximum stresses of 167 and 243 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 297 and 433 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the climate in western Montana, lower moisture contents were expected and also found except for one deck location. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

The moisture content percentages were all within a couple percentage points of one another. This shows that none of the tested areas are subjected to vastly different amounts of moisture.

Conclusions

Several methods of condition and performance investigation were performed on the Montana Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of immediate concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

There was no cracking in the wearing surface observed as a new chip seal was recently applied to the bridge wearing surface. Even so, some water staining into the deck boards and girders was evident from prior seepage and some white residue has formed between girders 1 and 2.

A fair amount of checking is occurring throughout the exterior girders. The affects of the western Montana climate and weathering is apparent in most exposed timber elements.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Deflection values at midspan were within at least one of the recommended maximum values.

Appendix O—Montana State Bridge No. L25003009+09001, Montana Bridge

Terry Wipf
Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Abstract

The Montana Bridge is a single-span timber girder bridge with a bituminous wearing surface located near Wolf Creek, Montana. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: William Lay, Montana Department of Transportation employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing.

Background

The location of Montana State Bridge L25003009+09001, hereinafter referred to as the Montana Bridge, is shown in Figure O1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Montana Bridge was built in 1933 and is located in approximately 2 miles east of Wolf Creek, Montana, on Craig Frontage Road. Currently, the bridge is not posted.



Figure O1. Montana Bridge location.



Figure O2. Bridge substructure.

Bridge Description

The Montana Bridge is a single-span, two-lane, timber girder bridge with a bituminous wearing surface. The bridge length measures 20 ft–7 in. from the east backwall to the west backwall. The bridge width measures 22 ft–0 in. from inside of curb to inside of curb and 22 ft–5 in. from outside of rail to outside of rail. The substructure consists of solid timber posts and sills (see Fig. O2).

The parapet consists of solid timber posts and timber rails with a timber curb. Support for the parapet is provided by bolts into the exterior girders along with bolts into the curb, which is seated on top of the deck, as shown in Figure O3.

Girders measure 20 ft–7 in. from end to end and have a clear span of 17 ft–11 in. A total of 12 girders, spaced 24 in.



Figure O3. Montana Bridge parapet support.

center-to-center, measuring 6 by 17 in. in cross section, are present and are seated and toe-nailed to the 12- by 12-in. timber sills with spikes. The deck consists of individual 2- by 4-in. nominal boards laid upon the short face transverse to the longitudinal girder direction. Overlaying the deck is an 11-in.-thick layer of asphalt wearing surface. Figure O4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure O5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the 12 girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure O6.

Strain transducers were attached to girder numbers 1, 3, 5, 7, 9, and 11, with 1 being the outside girder on the north side of the bridge and 12 being the outside girder on the south side of the bridge. The midspan and one abutment were instrumented (see Fig. O5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder and another was placed 2 in. from the top of the girder. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill. Figure O7 shows a typical setup of strain transducers near the girder ends.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the Montana Department of

Transportation. Dimensions of the truck are shown in Figure O8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–5 in.; the distance between the forward most rear axle and the front axle hubs measured 13 ft–4 in. Exact weight of the truck was 39,020 lb, where the total rear weight was 28,080 lb and the front axle weight was 10,940 lb. Assuming equal weights on each rear axle, the rear axles weighed 14,040 lb. The axle weights are shown in Figure O8, and the load truck used for testing is shown in Figure O9.

Three load paths were considered when testing the bridge (see Figs. O10 to O12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the north curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the south curb. For all load paths, the dump truck was driven at a crawl speed from west to east and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on November 17, 2004, was obtained from the Montana DOT (MDT). This report was reviewed, and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

Overall, the wearing surface looked to be in good condition as no cracking was observed. According to the MDT 2004 report, a new asphalt patch was placed on the structure between 2001 and 2002. This patch repaired rutting and impending pot holes in the asphalt. The wearing surface did, however, have a significant thickness of 11 in., which can effectively reduce the parapet height and add a large amount of dead weight to the bridge structure.

Deck

According to the MDT 2004 report, minor water staining was present on the underside of the decking. This water staining was verified by the ISU research team during the visual inspection of summer 2006. Some white residue was also forming on the underside of the decking, and this may be a condition of continued seepage (see Fig. O13). The

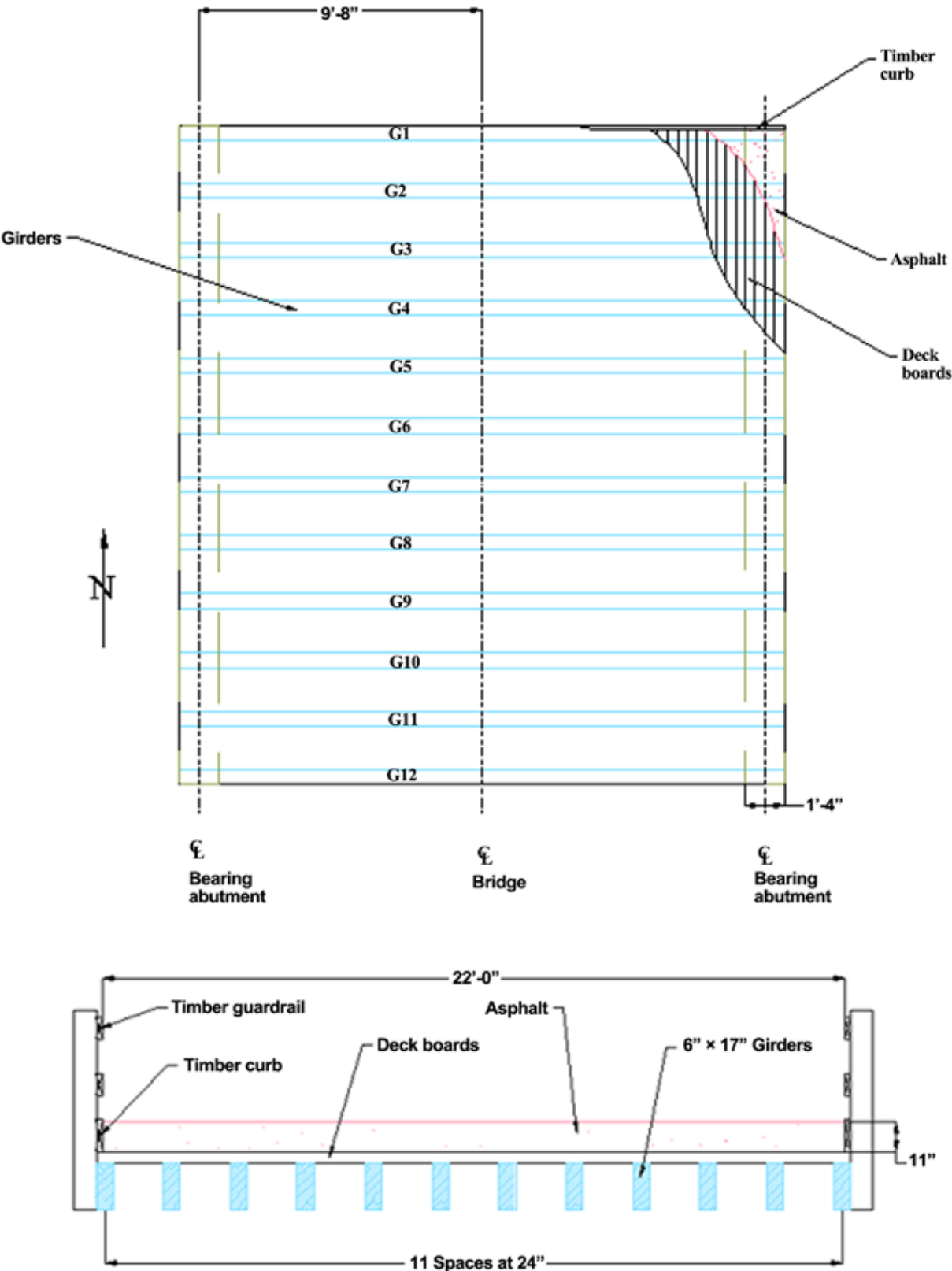


Figure O4. Plan and profile layout of Montana Bridge.

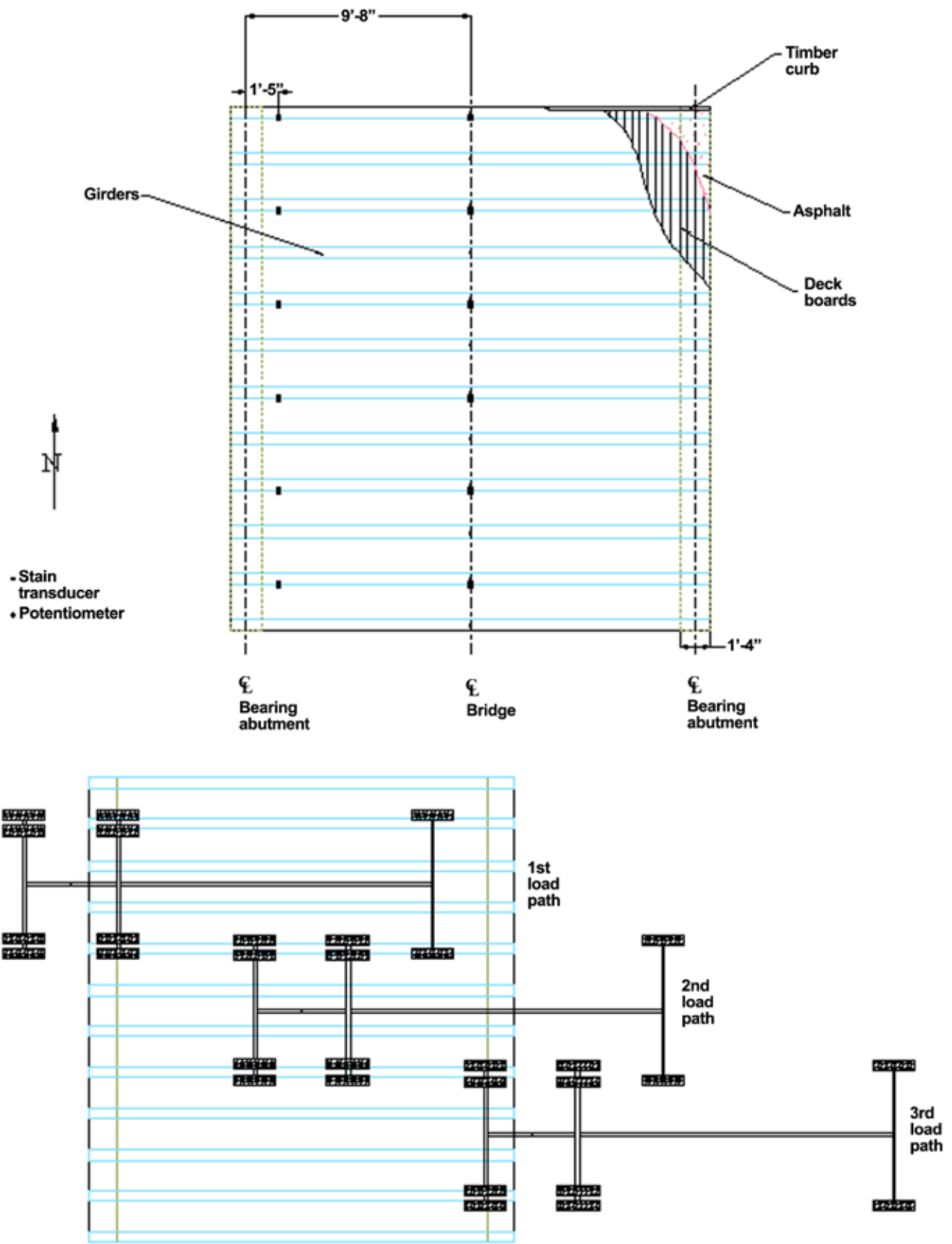


Figure O5. Instrumentation and load paths of Montana Bridge.



Figure O6. Deflection instrumentation.



Figure O7. Strain transducers.

ends of the deck boards appeared to be in good condition; there was no visible detachment of the deck boards from the girders except at girder number 2, where a gap of approximately 1/4 to 1/2 in. was present between the deck boards and girder. All deck boards were securely fastened, however.

Superstructure

Moderate checking was present along the midline in approximately half the girders. These checks should be monitored with future inspections as a possibility exists that these checks could worsen and the structural integrity of the girder could be compromised. Spotting was present throughout

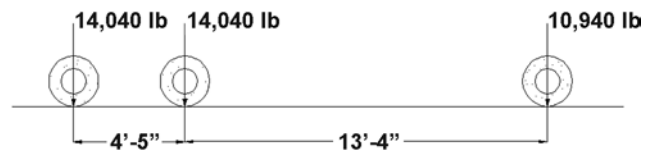


Figure O8. Truck configuration and axle loads.



Figure O9. Tandem axle load truck.

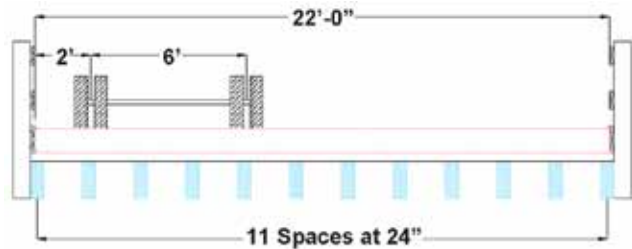


Figure O10. Transverse truck position—load path 1.

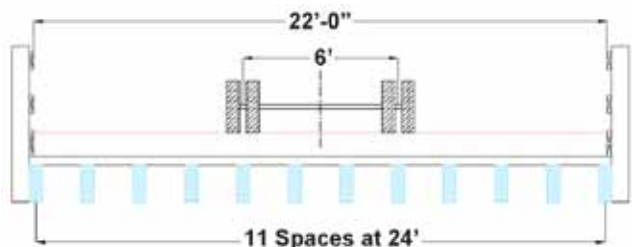


Figure O11. Transverse truck position—load path 2.

the superstructure, presumably from water seepage. Exterior girders were in worse condition than interior girders, presumably a result of more exposure to weathering conditions (see Fig. O14). The checks in the exterior girders were deep in some locations. The girder bearing on the sill was sufficient, and no misalignment was observed except for girder number 2, where some twisting has taken place.

Overall Structure

The overall structure was in satisfactory condition, and structurally the bridge was sound. No odor like anise or

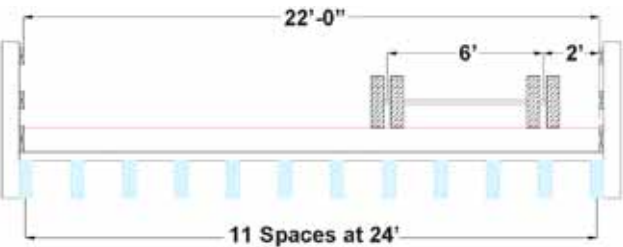


Figure O12. Transverse truck position—load path 3.



Figure O13. White residue formation on deck.



Figure O14. Checking in exterior girders.

Table O1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.222
Load path 2	0.177
Load path 3	0.183

wintergreen signifying fungal growth was present. There was no evidence of insect damage. Exposed timber members looked to be weathered and subjected to ultraviolet degradation. The substructure appeared in good condition, although vertical checking is present in most columns.

Results

The following presents the results of the static load testing of the Montana Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures O15 to O17 present the time-history deflections for each girder as the truck traveled across the bridge. Given the relationship of the length of the bridge to the length of the truck, one would expect to see two waves of loading as the front axle and back axles traverse the bridge. This is opposed to the loading patterns of longer bridges, where one wave is typically present as the entire truck is supported by the girders at the same time. This two wave relationship is quite evident in Figures O15 to O17, and clearly the deflections represent the difference in load from the front axle to the back axles.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table O1. Each passing of the three load paths is illustrated in Figures O18 to O20. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table O2 lists the n value for the girder of most deflection for each load path.

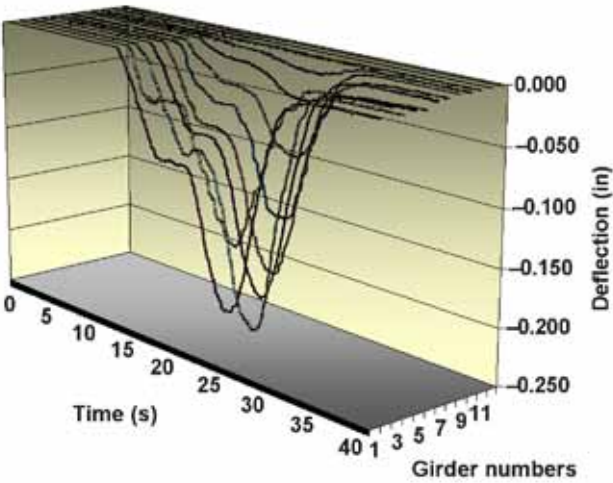


Figure O15. Deflections for load path 1.

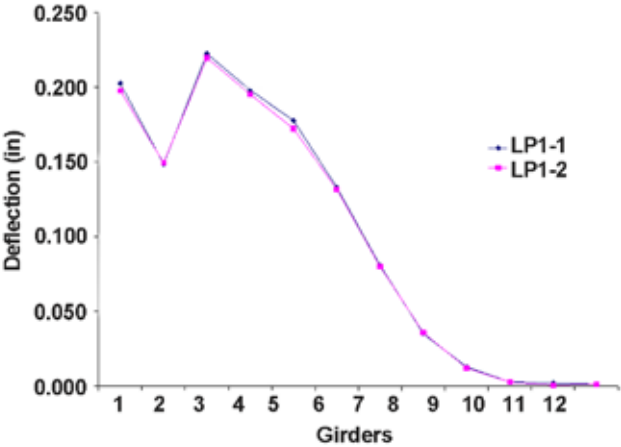


Figure O18. Maximum deflections for load path 1.

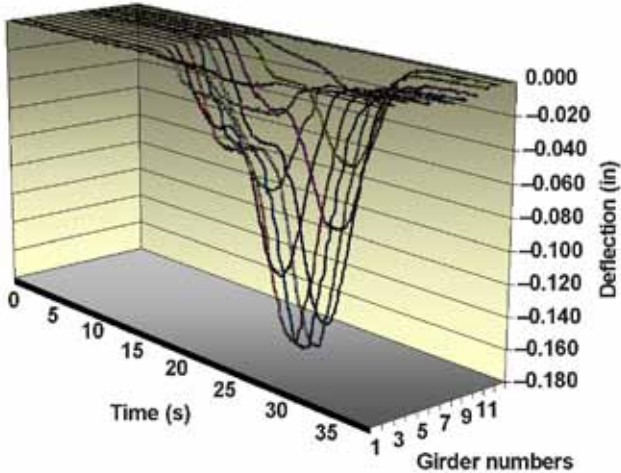


Figure O16. Deflections for load path 2.

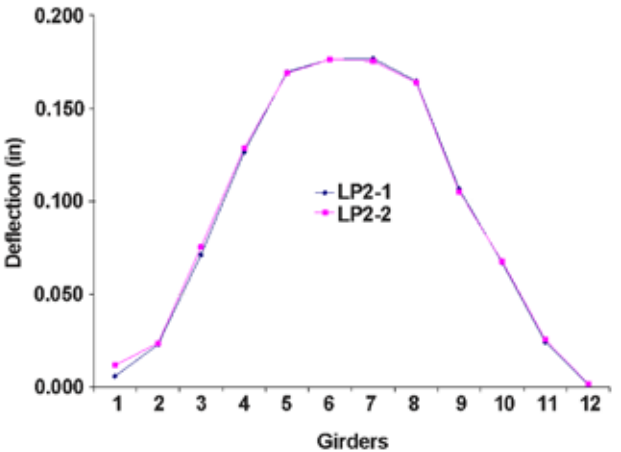


Figure O19. Maximum deflections for load path 2.

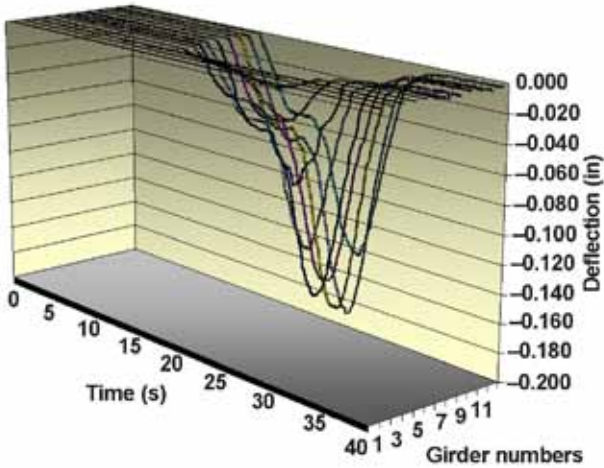


Figure O17. Deflections for load path 3.

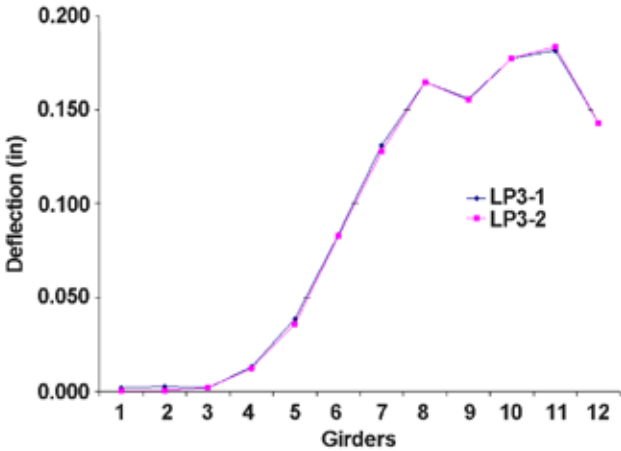


Figure O20. Maximum deflections for load path 3.

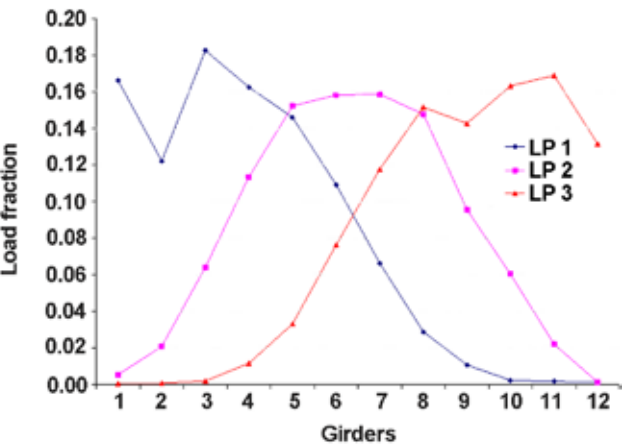


Figure O21. Load fractions for each load path.

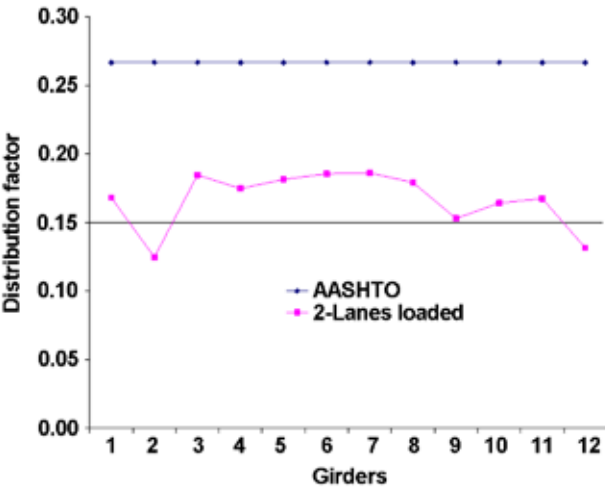


Figure O22. AASHTO design live load distribution.

The minimum n value of the three load paths was 543. This value was greater than all the minimum recommended values for timber girders in Table O2. Therefore, the maximum midspan deflections are more favorable for no wearing surface cracking to develop.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure O21 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure O22 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load

Table O2—Most critical n value

n value for girder of most deflection for each load path	
Load path 1	543
Load path 2	683
Load path 3	658

Table O3—Maximum differential deflection

Maximum differential deflection at midspan between adjacent girders (in.)	
Load path 1	0.074
Load path 2	0.058
Load path 3	0.047

distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures O23 to O25 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table O3.

The maximum differential deflection of 0.074 in. occurs in load path 1 and equals 0.037 in. per ft of girder spacing. This does not appear to be an issue as it is a relatively small amount. The same is true for load paths 1 and 3 as the maximum differential deflections are both around 0.05 in. If the differential deflections were large, the possibility would exist that the load was not well distributed transversely between these two girders or the assumption that both girders are of equal stiffness was false.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 207 and 244 microstrain, respectively. The strain plot at midspan is shown in Figures O26 to O28 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or

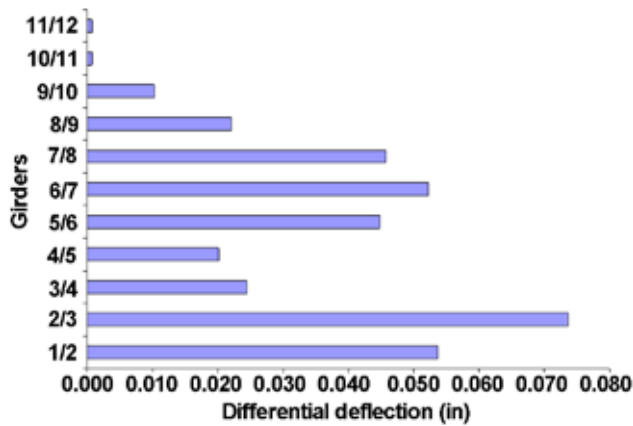


Figure O23. Differential deflections for load path 1.

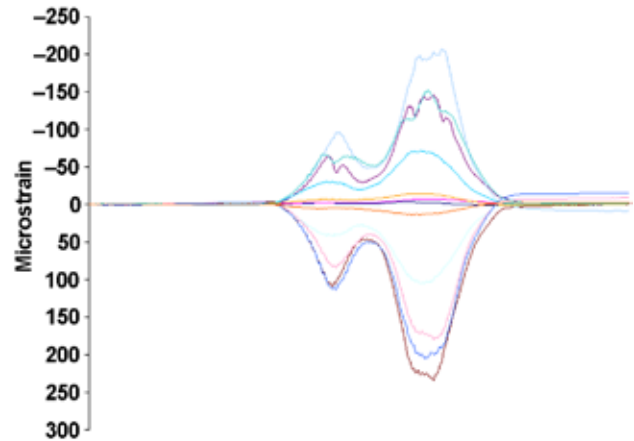


Figure O26. Strain at midspan for load path 1.

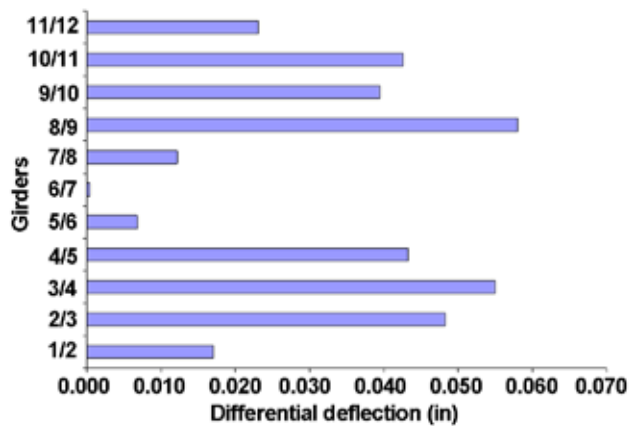


Figure O24. Differential deflections for load path 2.

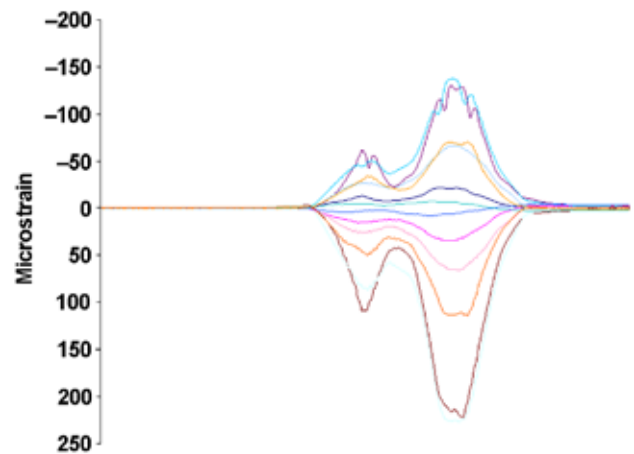


Figure O27. Strain at midspan for load path 2.

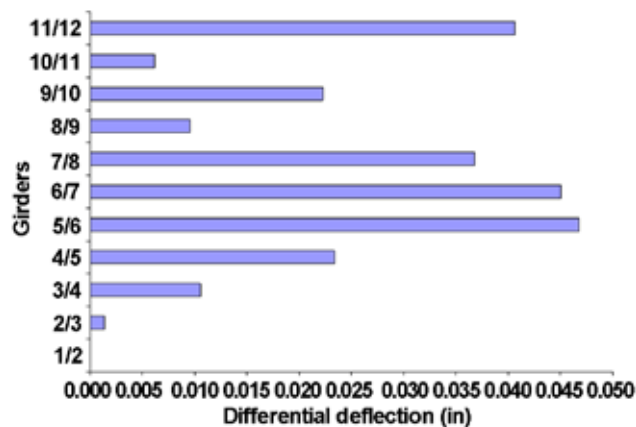


Figure O25. Differential deflections for load path 3.

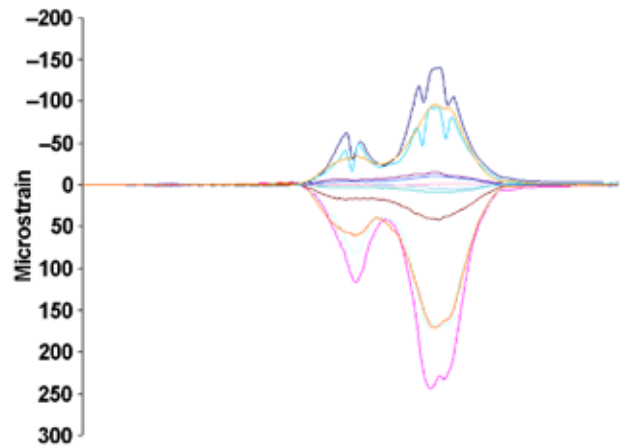


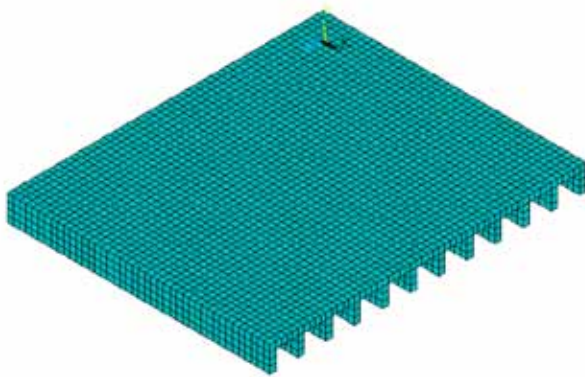
Figure O28. Strain at midspan for load path 3.

positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures O26 to O28 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck (that is, the girders act independently of the deck when subjected to bending).

Table O4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, west abutment	11.1
Girder 1, midspan	9.7
Girder 6, west abutment	8.3
Girder 6, midspan	9.8
Girder 12, west abutment	14.5
Girder 12, midspan	17.1
Bottom of deck between girders 1 and 2	9.8
Bottom of deck between girders 6 and 7	8.5
Bottom of deck between girders 11 and 12	11.0

**Figure O29. Finite element model.**

Moisture Content

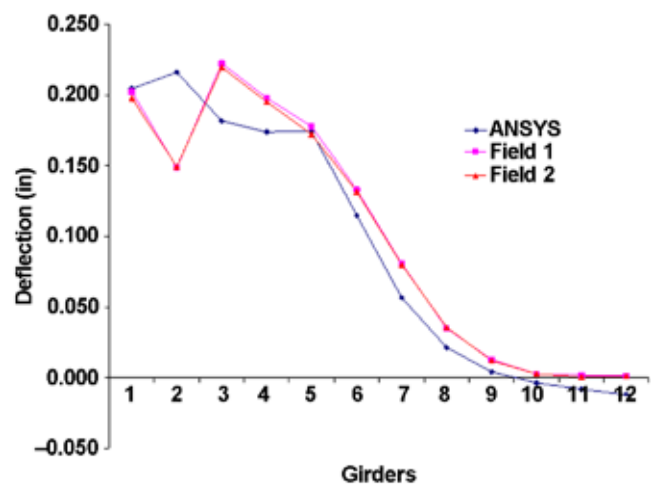
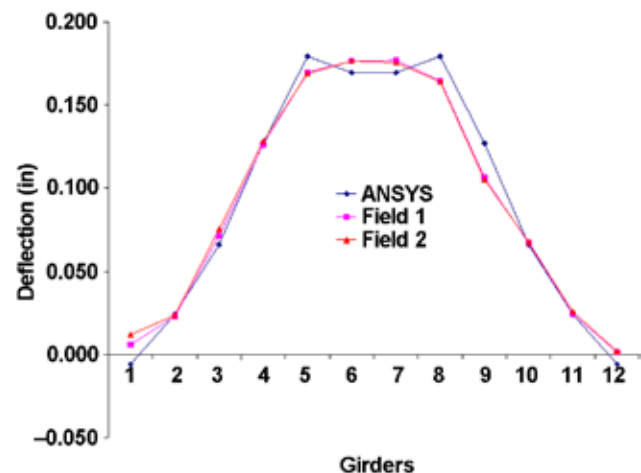
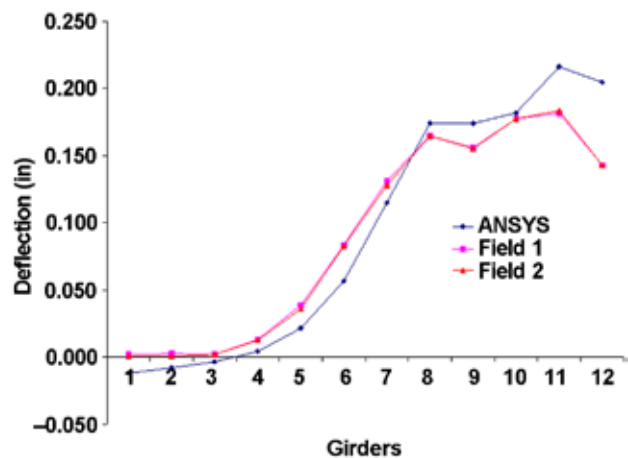
Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 6, and 12 at the midspan and west abutment. The bottom of the deck between girders 1 and 2, 6 and 7, and 11 and 12 was measured at midspan. Measurements ranged from 8.3% to 17.1%. The moisture content measurements are summarized in Table O4.

Finite Element Analysis

A finite element model was developed (see Fig. O29) for the Montana Bridge using ANSYS, a well-known finite element software. The objective was to create a model that would replicate field results when subjected to the same loading. After calibrating the model to the midspan deflection results obtained from the static load test, it was decided that the model would be subjected to a load simulating the AASH-T0 HS20 tandem-axle design vehicle. Deflection and tensile strain results at midspan were obtained from the model.

Figures O30 to O32 show the calibrated model results when subjected to the same load as that during the static load test. Notice the similarities between each plot.

Figure O33 shows the maximum deflections at midspan after subjecting the finite element model to the load of the

**Figure O30. ANSYS calibration results load path 1.****Figure O31. ANSYS calibration results load path 2.****Figure O32. ANSYS calibration results load path 3.**

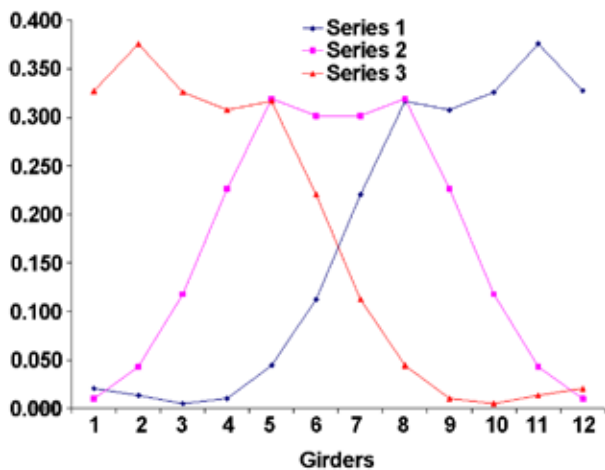


Figure O33. ANSYS deflection results for each load path when subjected to HS20 tandem-axle design vehicle.

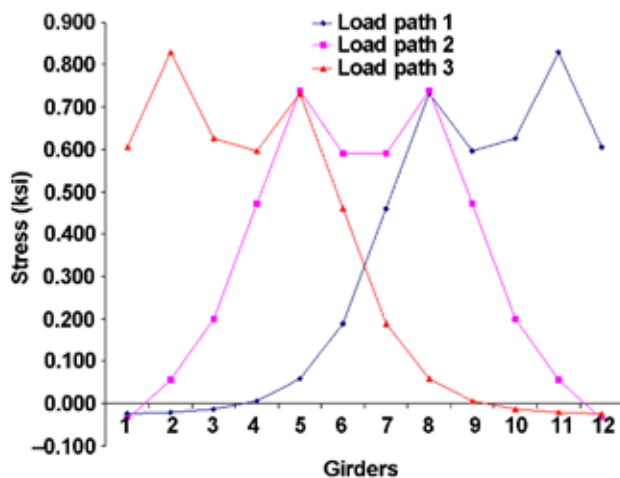


Figure O34. ANSYS tensile stress for each load path when subjected to HS20 tandem-axle design vehicle.

AASHTO HS20 tandem-axle design vehicle traveled along each load path.

Figure O34 shows the maximum tensile stresses at midspan due to the AASHTO HS20 tandem-axle design vehicle traveled along each load path.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because timber strength is not critically affected until deflections become excessive. Also, each of the maximum girder deflections for each load path meets the recommended limit state for live load deflection derived primarily from wearing surface degradation and maintainability.

The maximum deflection of girder number 2 at midspan appeared inconsistent with the maximum midspan deflection of the adjacent girders. This inconsistency was presumably a result of the gap between the deck boards and girder. Load is not immediately transferred to the girder so immediate deflection does not occur, thereby reducing the maximum deflection.

Exceeding the live load deflection recommendations can have adverse affects on the structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produce a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded as asphalt has very little fatigue resistance. Numerous problems associated with cracking exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. A benefit of the bridge location is that conditions are ideal for seepage to quickly evaporate because of the more arid climate. As a result, any water seepage through the deck should be prone to evaporation before permeation of the girders occurs.

Through visual inspection, transverse cracks in the wearing surface were not found and the wearing surface was in good condition as the bridge has been recently resurfaced.

Differential deflections between adjacent girders could result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was within that limit. It could be argued the transverse layout of the deck boards would appear to oppose longitudinal cracking because a longitudinal plane of weakness does not exist as it does in the transverse direction (that is, the discontinuity of adjacent deck boards). Even so, it could also be argued that the proximity of girders would appear to increase the chances of longitudinal cracking because any differential deflection is magnified by the short span between adjacent girders.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures O26 to O28 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of

compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 207 and 244 microstrain, respectively. These strains equate to maximum stresses of 238 and 281 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 424 and 500 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 lb/in², depending on the wood grade and moisture content. Therefore, it appears that allowable stresses are not exceeded by standard load trucks.

Due to the climate in western Montana, lower moisture contents were expected and also found, except for one measurement at the midspan of girder no. 12. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

Aside from the measurement at the midspan of girder no. 12, the moisture content percentages were all within a couple percentage points of one another. This shows that none of the measured areas are subjected to vastly different amounts of moisture.

Maximum midspan stresses and deflections were obtained from the finite element model. The maximum deflection was 0.376 in. from load paths 1 and 3, and 0.319 in. from load path 2. Much like the normalized vehicle loading, the results met the recommended limit states for live load deflection. The maximum stresses at midspan for load paths 1 and 3, and 2 were 829 and 738 lb/in², respectively. Much like the stresses obtained from the normalized vehicle loading, these values were within the values set by allowable stress design. The finite element model is consistent with the results discussed previously; recommended live load deflection limits and allowable stresses were not exceeded.

Conclusions

Several methods of condition and performance investigation were performed on the Montana Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of immediate concern. The condition of the bridge was consistent with other

bridges similarly aged and subjected to similar weathering and loading conditions.

There was no cracking in the wearing surface observed, as a new chip seal was recently applied to the bridge wearing surface. Even so, some water staining into the deck boards and girders was evident from prior seepage, and some white residue has formed between girders 1 and 2.

A fair amount of checking is occurring throughout the exterior girders. The affects of the western Montana climate and weathering is apparent in most exposed timber elements.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Deflection values at midspan were within at least one of the recommended maximum values.

The finite element model yielded results that were consistent with the bridge performance under live load. Recommended live load deflection limits and allowable stresses at midspan were not exceeded.

Appendix P—USDA Forest Service Burnt Fork Bridge

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Brent Phares
Travis Hosteng
Doug Wood
Michael Ritter
Justin Dahlberg



Abstract

The Burnt Fork Bridge is a single-span timber girder bridge located in the Bitterroot National Forest in western Montana. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgment

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: John Kettel, USDA Forest Service employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing.

Background

The location of the Burnt Fork Bridge is shown in Figure P1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Burnt Fork Bridge was built in 1967 and is located approximately 35 miles south of Missoula, Montana. Currently, the bridge is not posted.

Bridge Description

The Burnt Fork Bridge is a single-span, single-lane, timber girder bridge with a timber runner wearing surface. The



Figure P1. Burnt Fork Bridge location.



Figure P2. Bridge substructure.

bridge length measures 28 ft–3 in. from the east backwall to the west backwall. The bridge width measures 14 ft–2 in. from inside of curb to inside of curb and 15 ft–5 in. from inside of rail to inside of rail. The substructure consists of solid timber posts and sills (see Fig. P2).

The parapet consists of solid timber posts and metal rails with a timber curb. Support for the parapet is provided by bolts into the exterior girders along with bolts into the curb, which is seated on top of the deck, as shown in Figure P3.

Girders measure 28 ft–3 in. from end to end and have a clear span of 26 ft–3 in. A total of seven girders, spaced 25-1/2 in. center-to-center, measuring 8 by 23-1/2 in. in cross-section, are present and are seated and toe-nailed to the 12- by 12-in. timber sills with spikes. The deck consists of individual 2- by 6-in. nominal boards laid upon the short face transverse to the longitudinal girder direction. Overlaying the deck are 10 3- by 12-in. timber runners totaling 10 ft wide along the center of the bridge. Figure P4 illustrates the layout of the bridge.



Figure P3. Burnt Fork Bridge parapet support.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain transducers, and the truck position for each load path are shown in Figure P5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the seven girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure P6.

Strain transducers were attached to girder numbers 4, 5, 6, and 7, with 4 being the center girder of the bridge and 7 being the outside girder on the south side of the bridge. The midspan and one abutment were instrumented (see Fig. P5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder, another was placed at the midline of the girder, and another was placed 2 in. from the top of the girder. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill. Figure P7 shows a typical setup of strain transducers near the girder ends.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the USDA Forest Service. Dimensions of the truck are shown in Figure P8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–7 in.; the distance between the forward-most rear axle and the front axle hubs measured 14 ft–8 in. Exact weight of the truck was 55,180 lb, where the total rear weight equaled 38,626 lb and the front axle weight was 16,554 lb. Assuming equal weights on each rear

axle, the rear axles weighed 19,313 lb. The axle weights are shown in Figure P8, and the load truck used for the testing is shown in Figure P9.

Three load paths were considered when testing the bridge (see Figs. P10 to P12). Each load path was selected based on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the north curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the south curb. For all load paths, the dump truck was driven at a crawl speed from west to east and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on October 20, 2000, was obtained from the USDA Forest Service. This report was reviewed, and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

Overall, the wearing surface appeared in good condition. No checking or cracking was observed in the timber running planks. The wearing surface is shown in Figure P13.

Deck

Overall the deck appeared in good condition, and all deck boards were securely fastened, though there was minor detachment and twisting of the deck boards at the ends (see Fig. P14). Water appeared to be leaking through the decking and water staining was observed by the ISU research team.

Superstructure

Water staining was also present on the girders in various locations throughout the superstructure and was presumably a result of seepage through the deck boards. The interior girders looked in good condition as no visual degradation such as checking was observed. Conversely, checking was observed on the two exterior girders at the girder midline (see Fig. P15). The checks in the exterior girders were deep in some locations and should be closely monitored with future inspections. If checking becomes severe, degradation effects can be accelerated and the structural integrity of the girder could be compromised. The girder bearing on the sill was sufficient and no misalignment was observed.

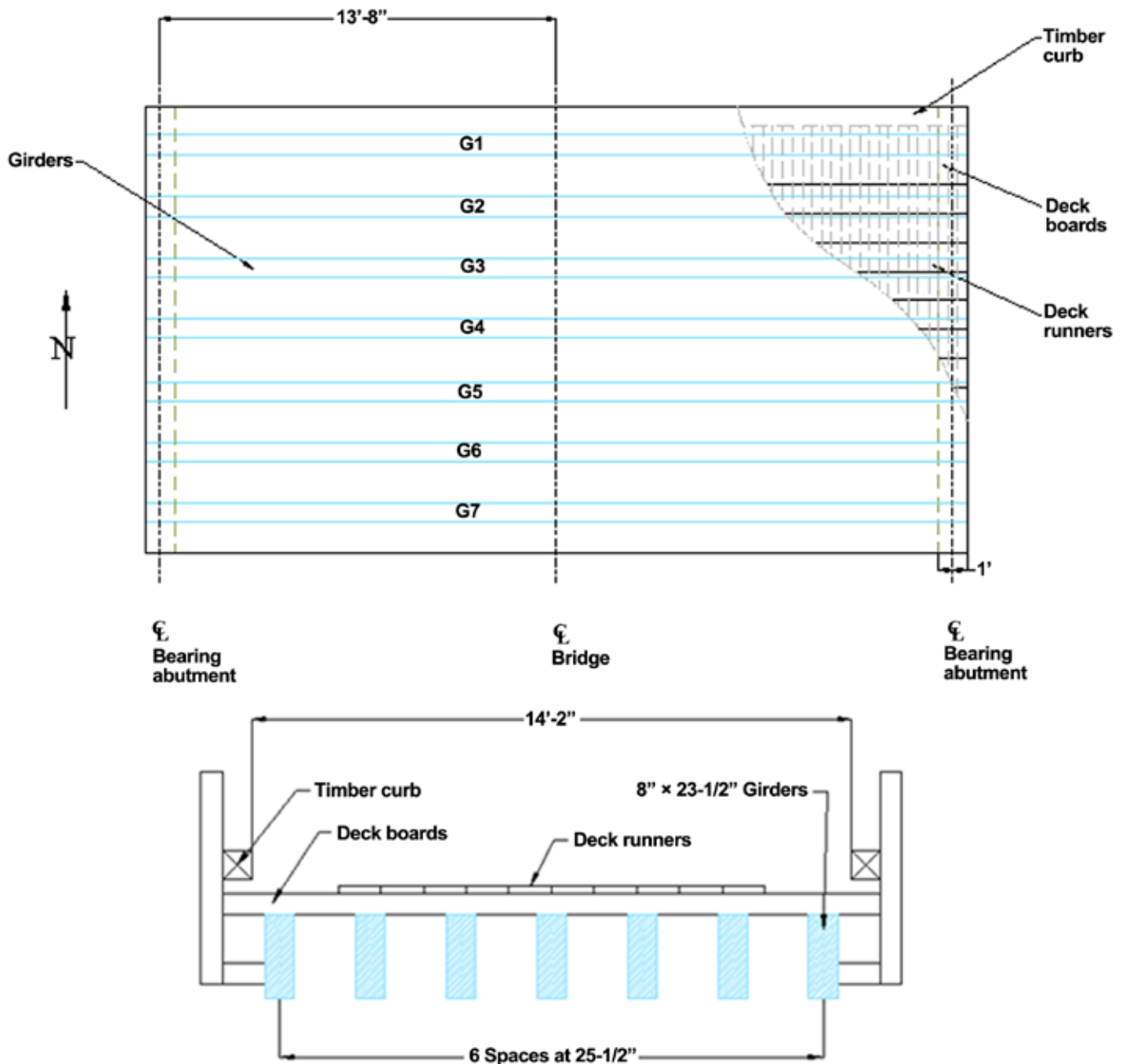


Figure P4. Plan and profile layout of Burnt Fork Bridge.

Overall Structure

The overall structure was in satisfactory condition, and structurally the bridge was sound. No odor like anise or wintergreen signifying fungal growth was present. There was no evidence of insect damage. Exposed timber members looked to be weathered and subjected to some ultraviolet degradation, and the substructure also showed signs of moderate checking. Vertical checks were observed in the substructure columns.

Results

The following presents the results of the static load testing of the Burnt Fork Bridge. These results include, for each

load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures P16 to P18 present the time-history deflections for each girder as the truck traveled across the bridge. Notice how the deflection pattern changes as the truck changes transverse locations.

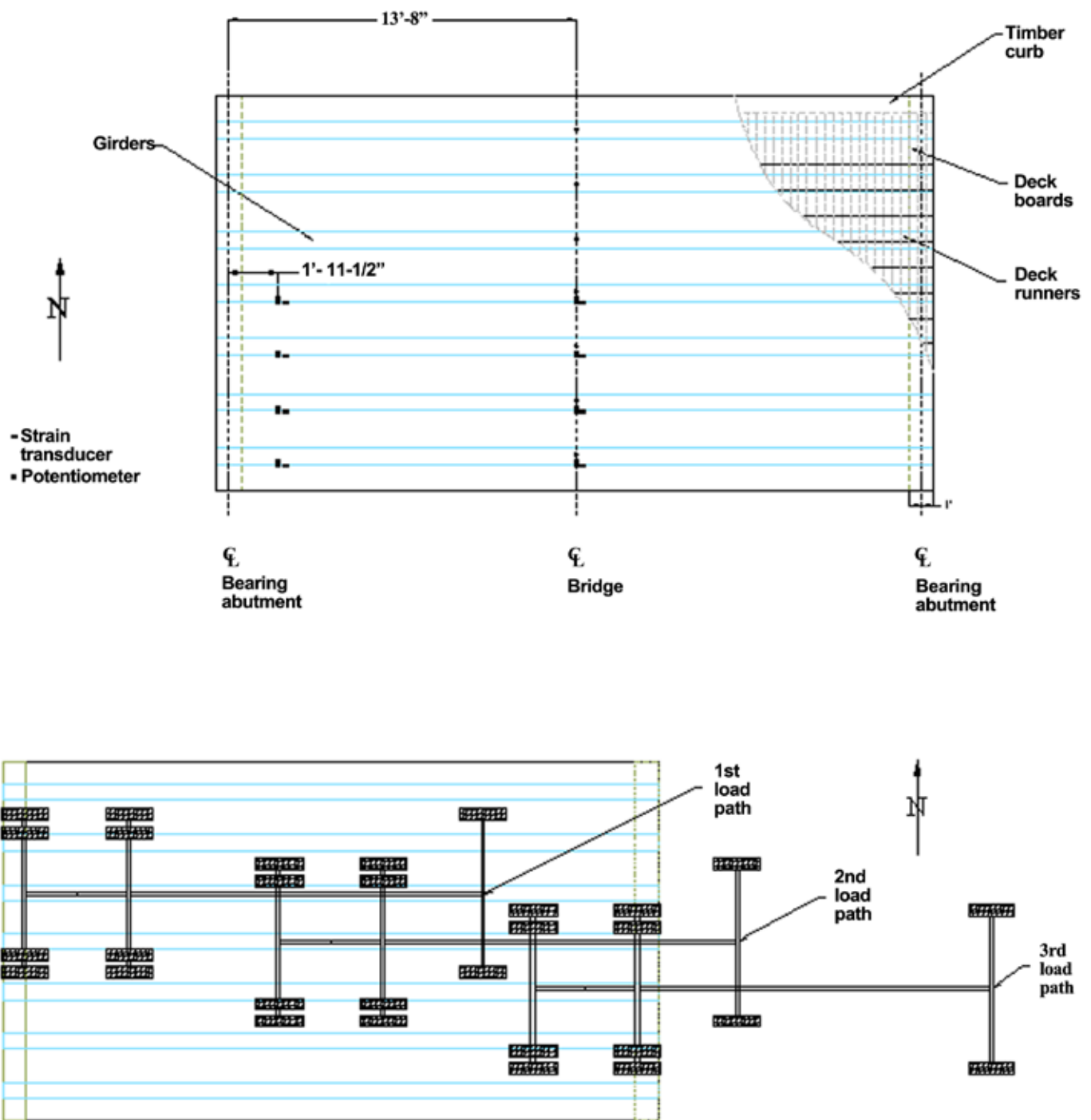


Figure P5. Instrumentation and load paths of Burnt Fork Bridge.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table P1. Each passing of the three load paths is illustrated in Figures P19 to P21. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the girders is repeatable. Consequently, only one passing for

each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table P2 lists the n value for the girder of most deflection for each load path.



Figure P6. Deflection instrumentation.



Figure P7. Strain transducers.

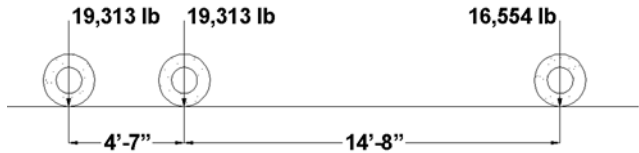


Figure P8. Truck configuration and axle loads.



Figure P9. Tandem-axle load truck.

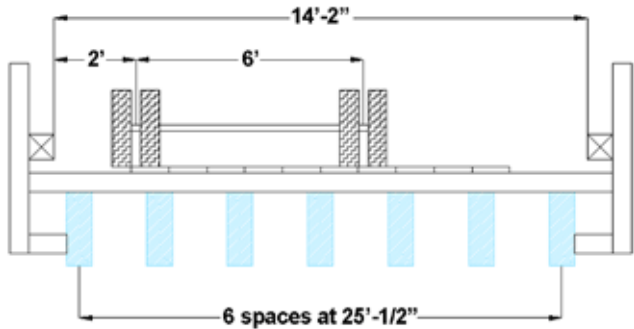


Figure P10. Transverse truck position—load path 1.

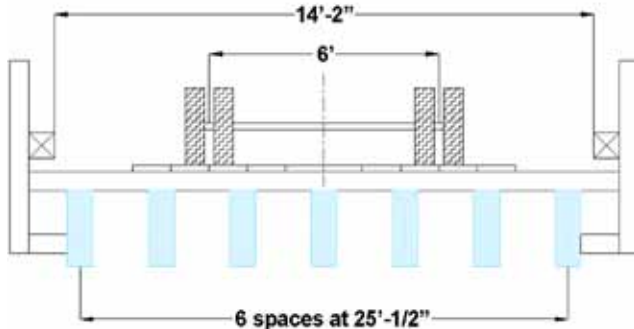


Figure P11. Transverse truck position—load path 2.

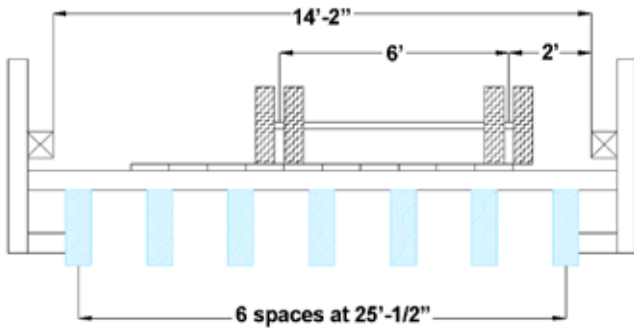


Figure P12. Transverse truck position—load path 3.



Figure P13. Wearing surface.



Figure P14. Twisting of deck boards.



Figure P15. Checking in exterior girders.

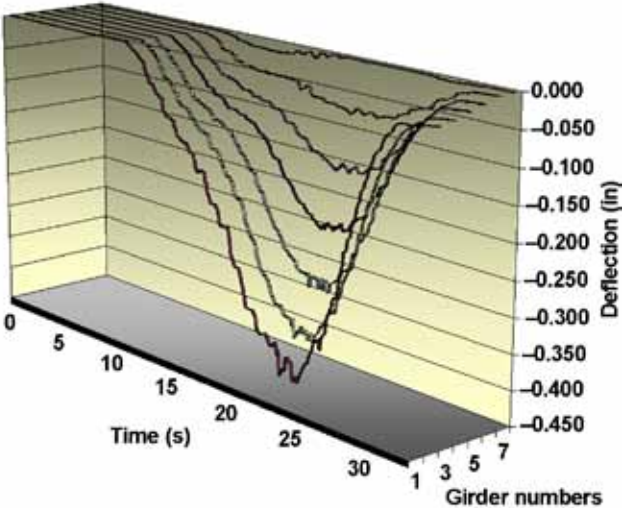


Figure P16. Deflections for load path 1.

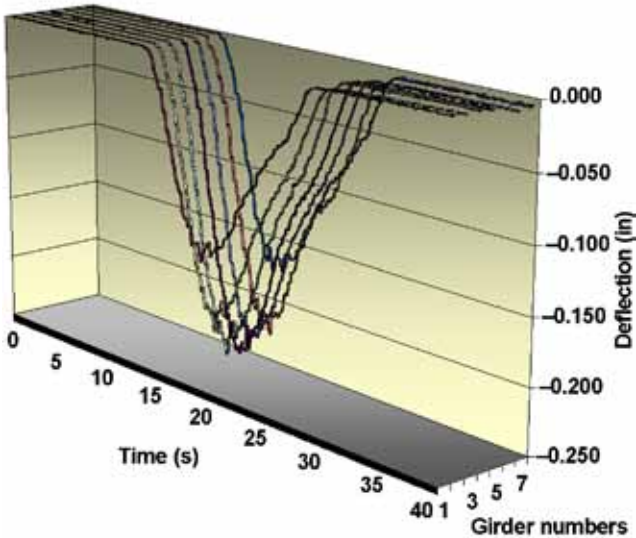


Figure P17. Deflections for load path 2.

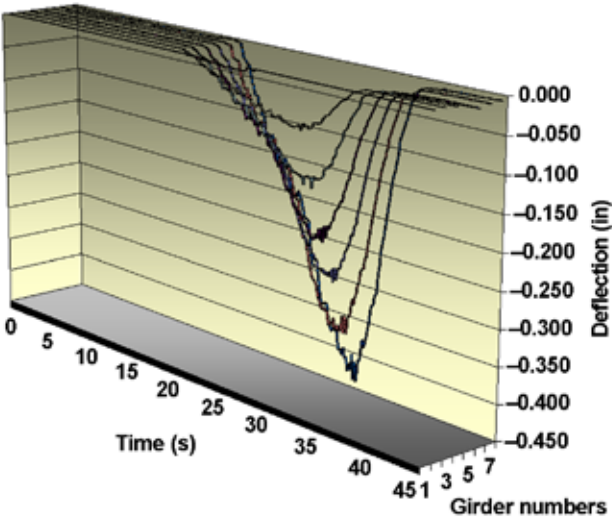


Figure P18. Deflections for load path 3.

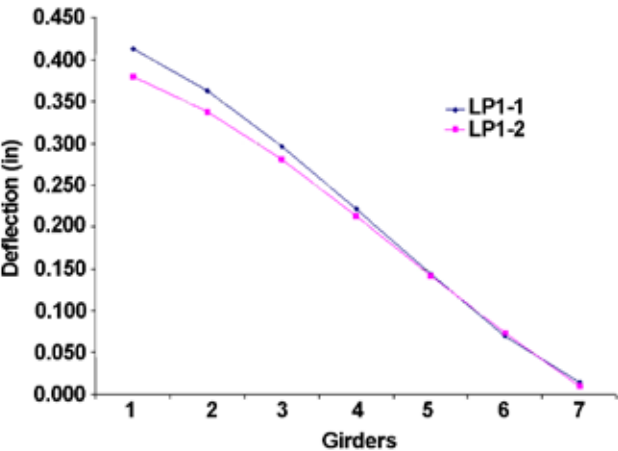


Figure P19. Maximum deflections for load path 1.

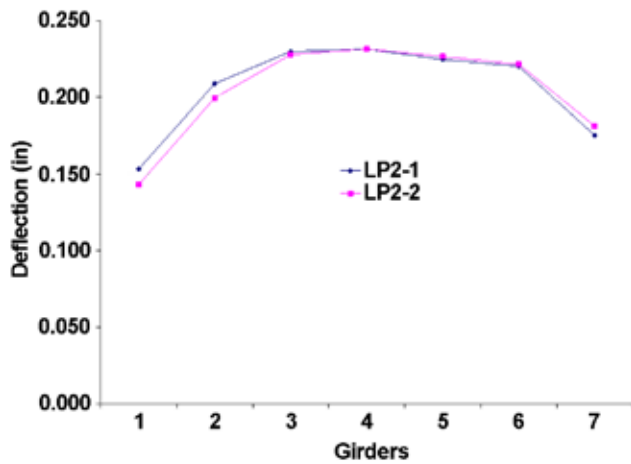


Figure P20. Maximum deflections for load path 2.

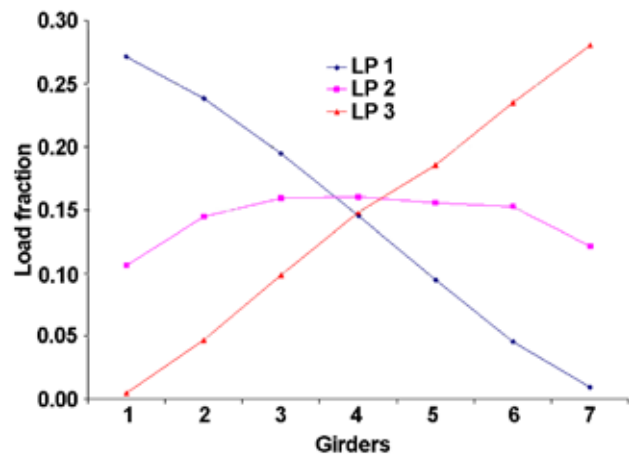


Figure P22. Load fractions for each load path.

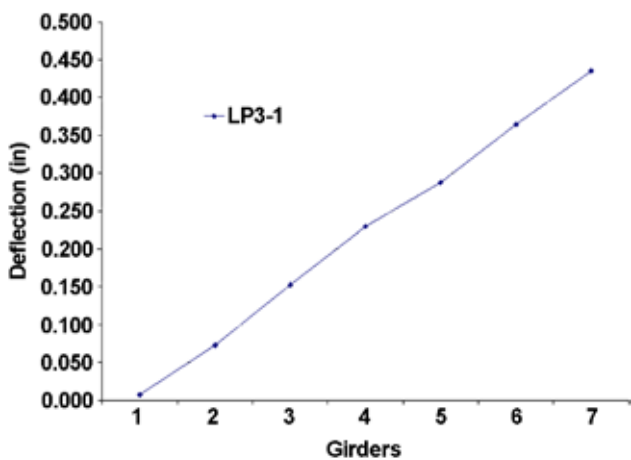


Figure P21. Maximum deflections for load path 3.

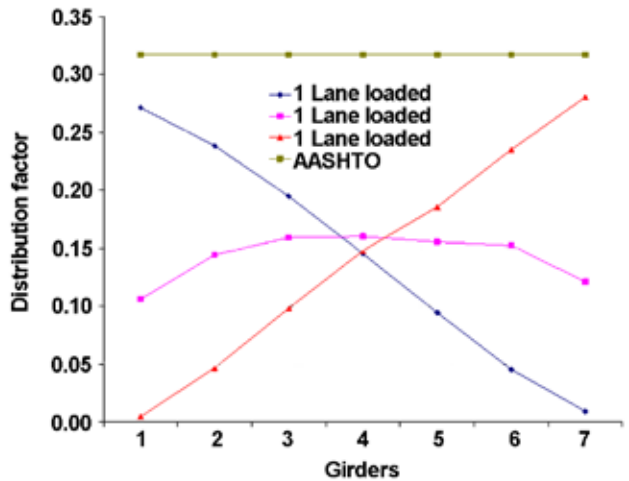


Figure P23. AASHTO design live load distribution.

Table P1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.413
Load path 2	0.232
Load path 3	0.435

Table P2—Most critical n value

	n value for girder of most deflection for each load path
Load path 1	589
Load path 2	1,051
Load path 3	559

The minimum n value of the three load paths was 559. This value was greater than all the minimum recommended values for timber girders stated in Table P2.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure P22 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for plank deck timber bridges is $S/6.7$ and $S/7.5$ for one design lane loaded and two or more design lanes loaded, respectively, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure P23 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all of the actual live load distribution factors.

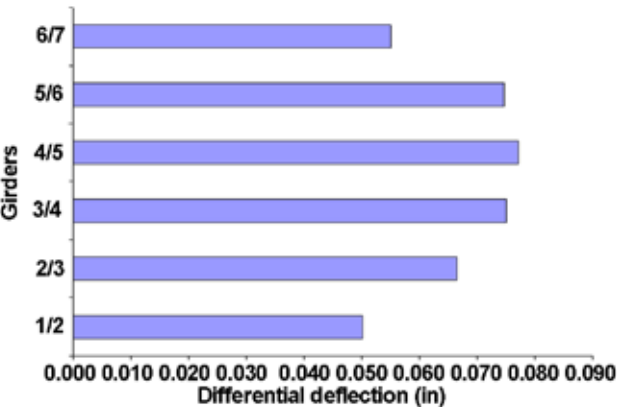


Figure P24. Differential deflections for load path 1.

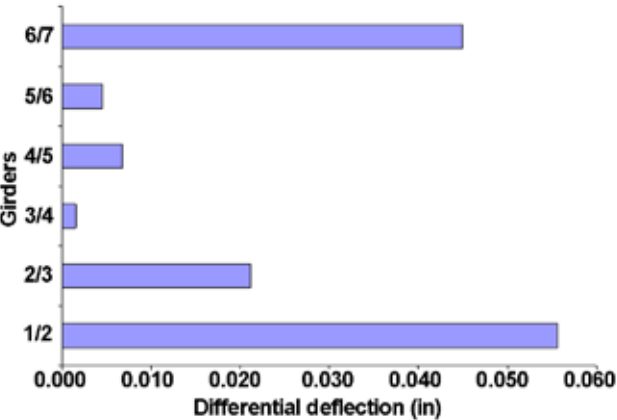


Figure P25. Differential deflections for load path 2.

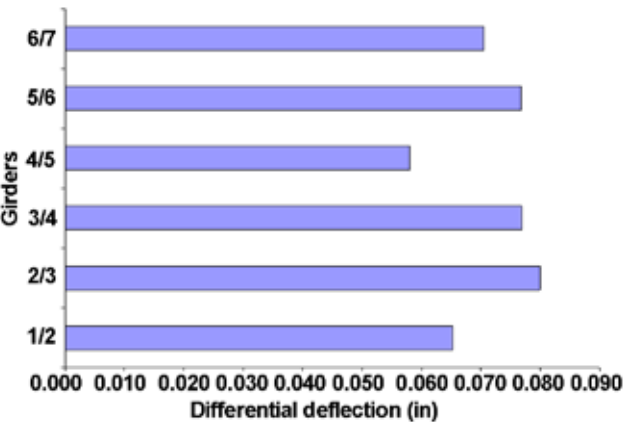


Figure P26. Differential deflections for load path 3.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures P24 to P26 show the differential deflections between adjacent girders for load paths 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table P3.

Table P3—Maximum differential deflection	
Maximum differential deflection at midspan between adjacent girders (in.)	
Load path 1	0.077
Load path 2	0.056
Load path 3	0.080

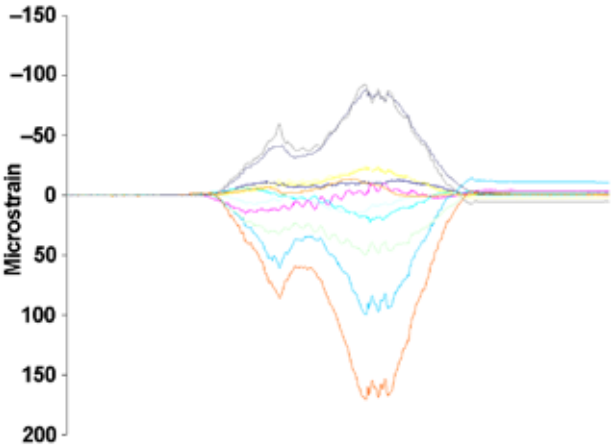


Figure P27. Strain at midspan for load path 1.

The maximum differential deflection of 0.080 in. occurs in load path 3 and does not appear to be an issue as it is a relatively small amount. The same is true for load paths 1 and 2 as the maximum differential deflections are both below 0.08 in. If the differential deflections were large, the possibility exists that the load was not well distributed transversely between these two girders or the assumption that both girders are of equal stiffness was false.

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present. Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Only two strain plots are presented because the placement of the strain gages did not warrant a third strain test. Assuming the bridge behavior is symmetrical, a third pass would have yielded results much like those shown in Figure P27. Maximum strain achieved in the girders was at midspan, with compression and tensile strains of 145 and 188 microstrain, respectively. The strain plots at midspan are shown in Figures P27 and P28 for load paths 1 and 2, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress

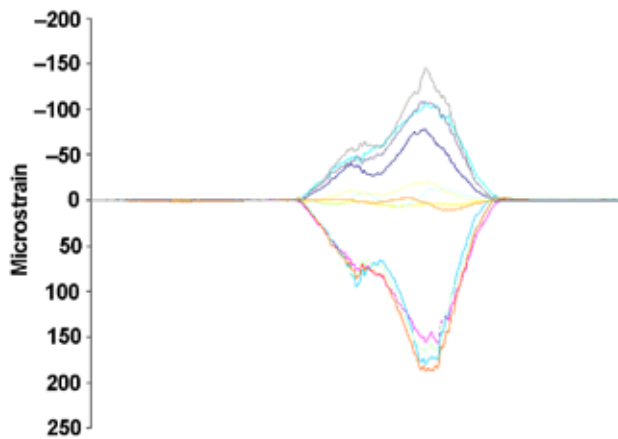


Figure P28. Strain at midspan for load path 2.

Table P4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, west abutment	10.7
Girder 1, midspan	12.1
Girder 4, west abutment	13.5
Girder 4, midspan	12.1
Girder 7, west abutment	13.8
Girder 7, midspan	14.9
Bottom of deck between girders 1 and 2	16.1
Bottom of deck between girders 4 and 5	17.8
Bottom of deck between girders 6 and 7	14.4

and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures P27 and P28 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck (that is, the girders act independently of the deck when subjected to bending).

Moisture Content

Moisture content measurements were taken at nine locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 4, and 7 at the midspan and south abutment. The bottom of the deck between girders 1 and 2, 4 and 5, and 6 and 7 was measured at midspan. Measurements ranged from 10.7% to 17.8% and are summarized in Table P4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength, because timber strength is not critically affected until deflections become excessive. Also, each of the maximum girder deflections for each load path meets the recommended limit state for live load deflection derived primarily from degradation and maintainability.

Exceeding the live load deflection recommendations can have adverse affects on the structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface degradation produces a displeasing sight and perception of an unsafe bridge.

A number of problems associated with a timber runner wearing surface could exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay. A benefit of the bridge location is that conditions are ideal for seepage to quickly evaporate because of the more arid climate. As a result, any water seepage through the deck should be prone to evaporation before permeation of the girders occurs.

Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, differential deflections between adjacent girders could result in undue degradation if those deflections are large. Large differential deflections may be signs of poor load distribution or other degradation.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for plank deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures P27 and P28 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 145 and 188 microstrain, respectively. These strains equate to maximum stresses of 167 and 216 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 216 and 280 lb/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1150 to 1750 lb/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the climate in western Montana, lower moisture contents were expected and also found, except for

measurements obtained from the deck. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

The moisture content percentages in the girders were all within a couple percentage points of one another. This shows that none of the tested areas are subjected to vastly different amounts of moisture.

Conclusions

Several methods of condition and performance investigation were performed on the Burnt Fork Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of immediate concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

The timber runners wearing surface appeared in good condition. Even so, this type of wearing surface leaves the bridge vulnerable to water seepage into and through the deck. Evidence of seepage was observed in various locations throughout the underside of the bridge.

Checking is occurring throughout the exterior girders, and weathering effects are apparent in most exposed timber elements.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 lb/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Deflection values at midspan were within all of the recommended maximum values.

Appendix Q—USDA Forest Service Trapper Creek Bridge

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Abstract

The Trapper Creek Bridge is a single-span timber girder bridge with a bituminous wearing surface located in the Bitterroot National Forest in western Montana. The bridge was load tested and visually assessed as part of a research project through the U.S. Department of Agriculture (USDA) Forest Products Laboratory, the Federal Highway Administration (FHWA), and the Bridge Engineering Center at Iowa State University. The results of the testing and assessment are presented in this report.

Acknowledgments

We express our appreciation to those who were of assistance to this project and without whom we would not have completed this research project: John Kettell, USDA Forest Service employee, who initially sent the latest inspection report for this bridge and who gave permission to pursue load testing.

Background

The location of Trapper Creek Bridge is shown in Figure Q1. The static load test data and visual inspection assessments are the basis for discussion throughout the remainder of this report.

The Trapper Creek Bridge was built in 1979 and is located approximately 5 miles southwest of Trapper Peak in Ravalli County, Montana. Currently, the bridge is not posted.

Bridge Description

The Trapper Creek Bridge is a single-span, single-lane, glue laminated timber girder bridge with a bituminous wearing



Figure Q1. Trapper Creek Bridge location.



Figure Q2. Bridge substructure.

surface set on a 10° skew. The bridge length measures 36 ft–0 in. from the east backwall to the west backwall. The bridge width measures 14 ft–0 in. from inside of curb to inside of curb and 16 ft–0 in. from outside of curb to outside of curb. The substructure consists of solid timber posts and sills (see Fig. Q2).

The parapet consists only of a timber 8- by 12-in. timber curb bolted to the deck boards through an 8- by 12-in. board. The curb is shown in Figure Q3.

Girders measure 35 ft–10 in. from end to end and have a clear span of 33 ft–10 in. A total of four girders, spaced 46 in. center-to-center, measuring 9 by 27 in. in cross-section, are present and are seated and bolted to the 12- by 12-in. timber sills. The deck consists of individual 4-ft-wide by 5-3/4-in.-thick glue laminated deck panels. Overlaying the deck is a 3-in.-thick layer of asphalt wearing surface. Figure Q4 illustrates the layout of the bridge.

Instrumentation

Instrumentation consisted of deflection gages and strain transducers. Locations of the deflection gages, strain



Figure Q3. Trapper Creek Bridge parapet support.

transducers, and the truck position for each load path are shown in Figure Q5. Because of the relatively short span and the need for only the maximum deflection data, deflection gages were attached at the center of the clear span at each of the four girders. To attach the gages, a small eye hook was inserted into the bottom of the girder at the pre-measured centerline of the clear span. Non-stretchable piano wire was used to connect the deflection gage string to the eye hook. The base of the deflection gage was attached to a stationary platform constructed from 2- by 6-in. planks and tripods. Deflection instrumentation is shown in Figure Q6.

Strain transducers were attached to each girder. Girder 1 was the outside girder on the south side of the bridge and 4 was the outside girder on the north side of the bridge. The midspan and one abutment were instrumented (see Fig. Q5). Transducers were placed near only one abutment because of the symmetry of the bridge. At each location, one transducer was placed on the bottom of the girder, another at the girder midline, and another was placed 2 in. from the top of the girder. The transducers near the abutment were placed a distance equal to the girder depth from the centerline of the sill. Figure Q7 shows a typical setup of strain transducers near midspan.

Static Loading

Static loading of the bridge was completed using a tandem-axle dump truck provided by the USDA Forest Service. Dimensions of the truck are shown in Figure Q8. The rear wheel base was 6 ft–0 in.; the distance between the hubs of the two rear axles measured 4 ft–7 in.; the distance between the forward-most rear axle and the front axle hubs measured 14 ft–8 in. Exact weight of the truck was 55,180 lb, where the total rear weight equaled 38,626 lb and the front axle weight was 16,554 lb. Assuming equal weights on each rear axle, the rear axles weighed 19,313 lb. The axle weights are shown in Figure Q8, and the load truck used for testing is shown in Figure Q9.

Three load paths were considered when testing the bridge (see Figs. Q10 to Q12). Each load path was selected based

on typical traffic paths and the objective of the project to standardize load conditions for all tested bridges. That is, maximum strains and deflections were desired along each side and the center of the bridge while keeping with typical traffic patterns. The outermost wheel line was centered on a line 2 ft from the inner face of the curb in accordance with AASHTO code provisions.

For the first load path, the left wheel line of the truck was driven 2 ft from the inside of the south curb. For the second load path, the truck was centered along the centerline of the bridge. For the third load path, the right wheel line of the truck was driven 2 ft from the inside of the north curb. For all load paths, the dump truck was driven at a crawl speed from east to west and multiple passes were made on each path to ensure the collected data were repeatable.

Condition Assessment

The report for the bridge inspection conducted on November 8, 2005, was obtained from the USDA Forest Service. This report was reviewed, and certain aspects are included here. A visual inspection of the bridge wearing surface, deck, superstructure, and overall structure was conducted by the ISU team upon completion of the static loading. The findings of both visual inspection reports are discussed ensuing.

Wearing Surface

Overall, the wearing surface looked to be in good condition. Only single cracks were observed in the wearing surface at midspan and at the transitions between the roadway and ends of the bridge. In addition to the cracking, a large amount of gravel debris was observed on the wearing surface, which was noted in the 2005 Forest Service report. The cracking and accumulation of debris is shown in Figure Q13.

Deck

The deck appeared to be in good condition as the underside of the deck was mostly uniform in color, signifying that the creosote treatment had not been washed away. Some washing was present at the outside edges of the deck panels, however (see Fig. Q14). Each of the deck panels was in good condition as no wear between panels was observed and the panels were securely fastened to the girders.

Superstructure

Water appeared to be leaking through the decking as the creosote treatment on the girders was streaking (see Fig. Q15). Washing was more prevalent near the girder ends near the abutments. Even so, the timber looked to be in good condition. At some locations on the faces of the girders small holes were present. The girder bearing on the sill was sufficient and no misalignment was observed.

Overall Structure

The overall structure was in good condition, and structurally the bridge was sound. No odor like anise or wintergreen

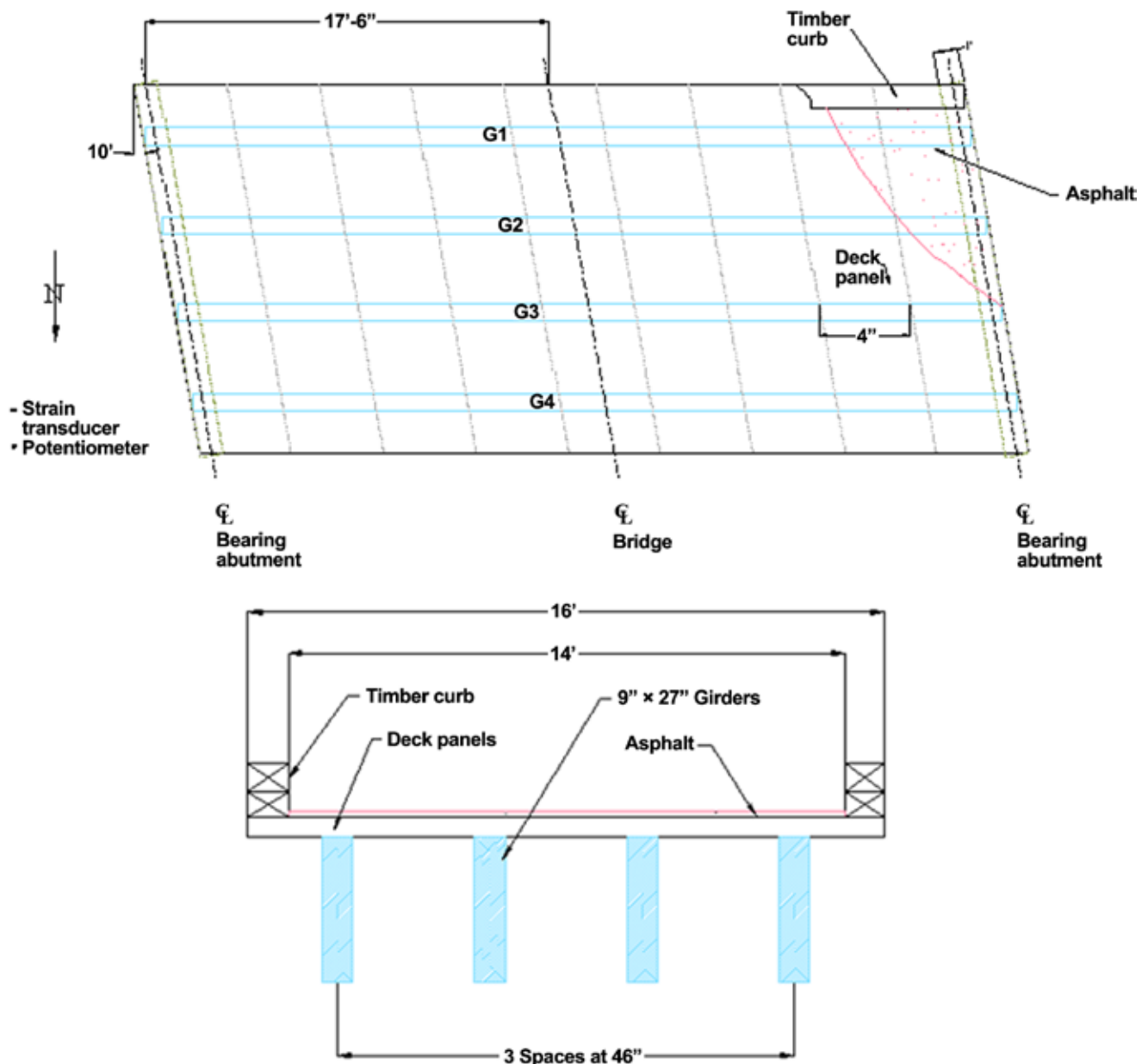


Figure Q4. Plan and profile layout of Trapper Creek Bridge.

signifying fungal growth was present. There was no evidence of insect damage. Columns, sills, and backwalls appear more weathered than the rest of the bridge, though still in good condition. Checking is present in all substructure columns and both sills at midline. Both timber curbs have large checks at the midline running the length of the curb in the longitudinal direction.

Results

The following presents the results of the static load testing of the Trapper Creek Bridge. These results include, for each load path, the time-history deflections of all girders, the maximum deflection of the bridge girders at midspan and the relation to published deflection criteria, the maximum differential deflection between adjacent girders, the

distribution factors for individual girders, and strain results for instrumented girders.

Time-History Deflections

Figures Q16 to Q18 present the time-history deflections for each girder as the truck traveled across the bridge. Notice the difference in girder deflections as the transverse truck position changes.

Maximum Deflections

The maximum deflections achieved for each load path are presented in Table Q1. Each passing of the three load paths is illustrated in Figures Q19 to Q21. One can notice the similar trend of the data for each passing of a particular load path. By achieving the same or near same deflections for each passing, one can be sure the deflection behavior of the

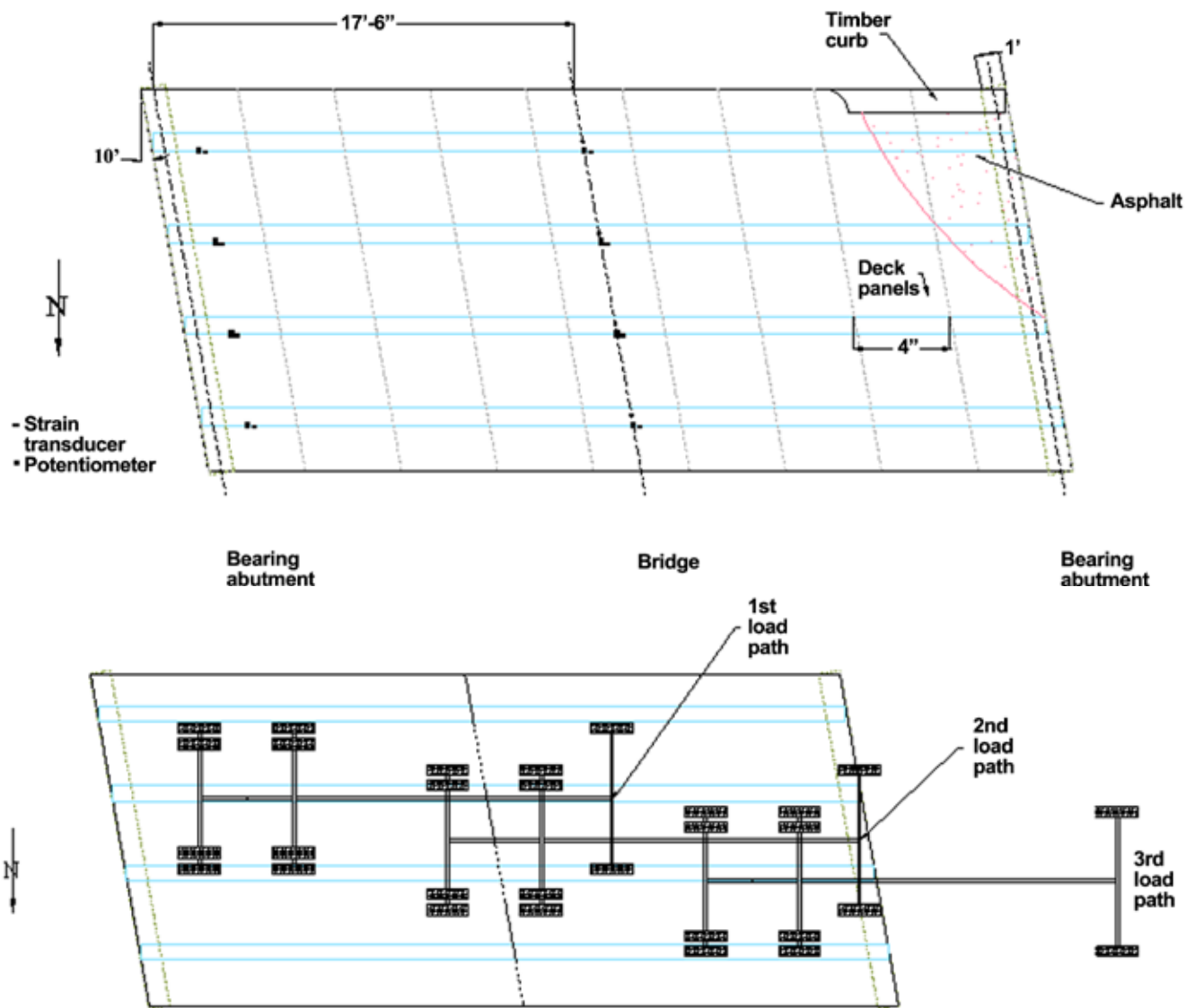


Figure Q5. Instrumentation and load paths of Trapper Creek Bridge.



Figure Q6. Deflection instrumentation.



Figure Q7. Strain transducers.

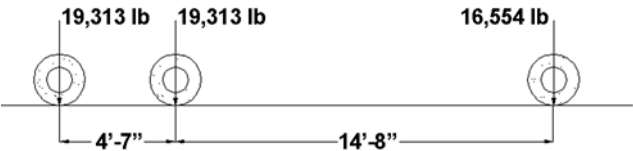


Figure Q8. Truck configuration and axle loads.



Figure Q9. Tandem-axle load truck.

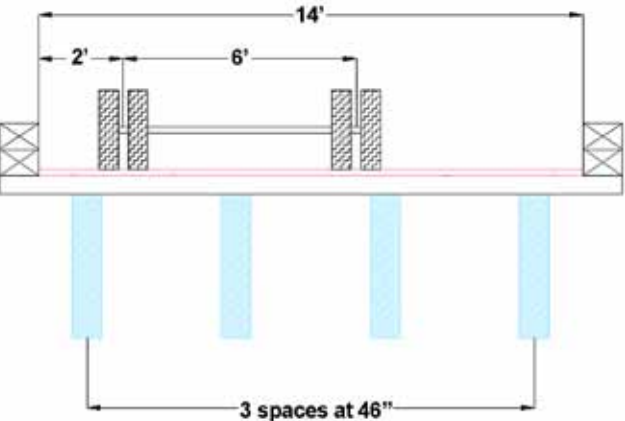


Figure Q10. Transverse truck position—load path 1.

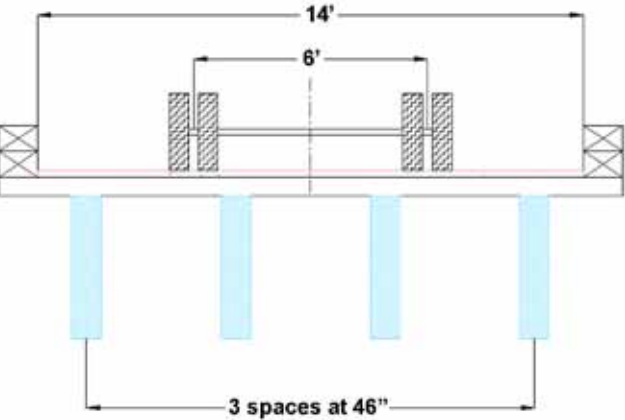


Figure Q11. Transverse truck position—load path 2.

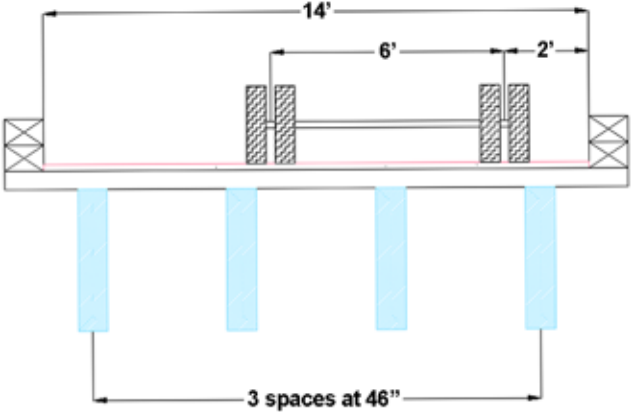


Figure Q12. Transverse truck position—load path 3.



Figure Q13. Wearing surface cracking and debris.



Figure Q14. Washing of creosote at panels ends.



Figure Q15. Girder streaking and holes.

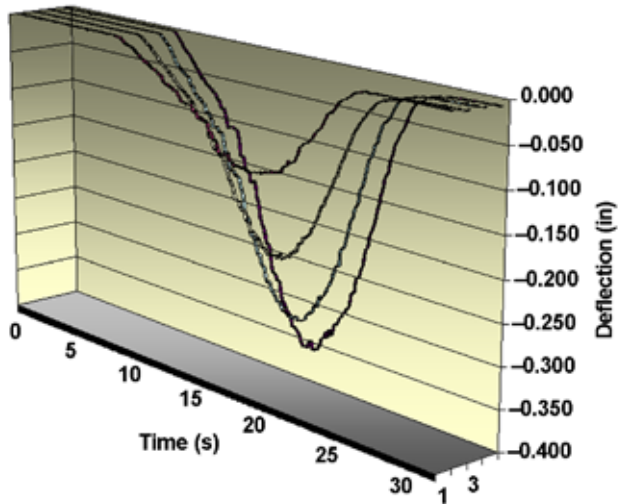


Figure Q18. Deflections for load path 3.

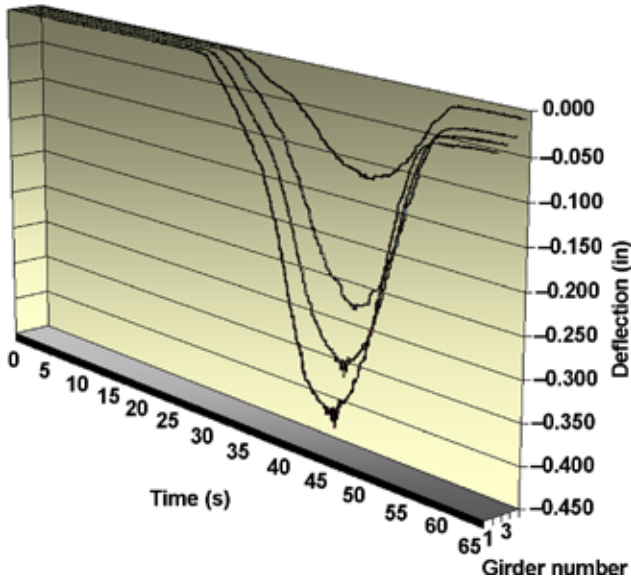


Figure Q16. Deflections for load path 1.

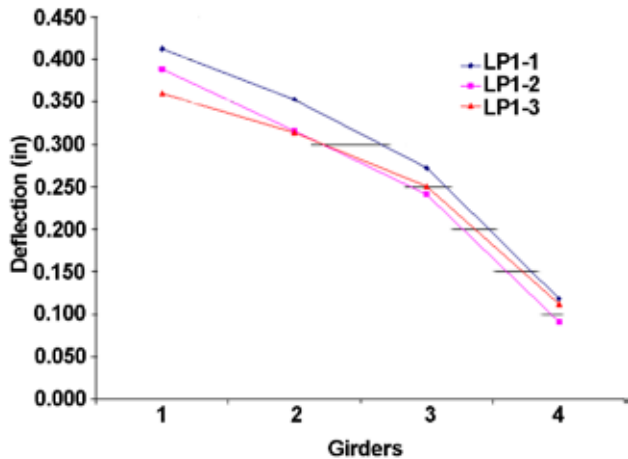


Figure Q19. Maximum deflections for load path 1.

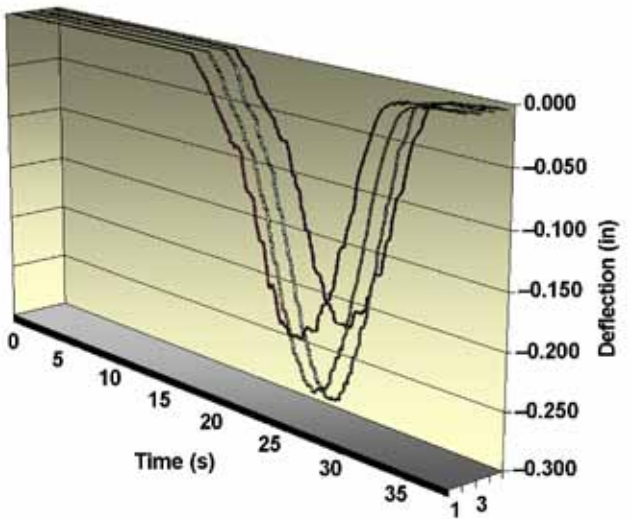


Figure Q17. Deflections for load path 2.

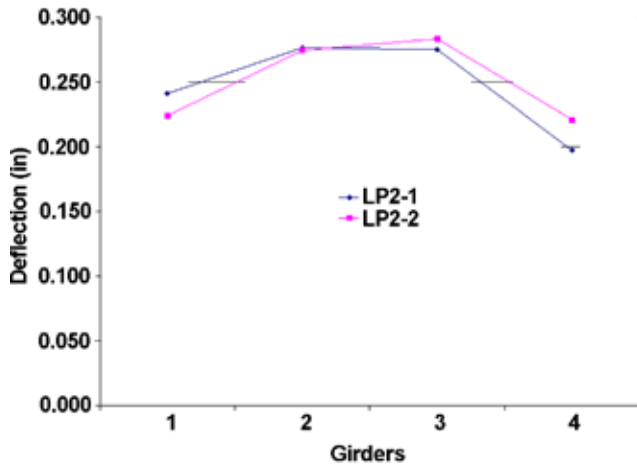


Figure Q20. Maximum deflections for load path 2.

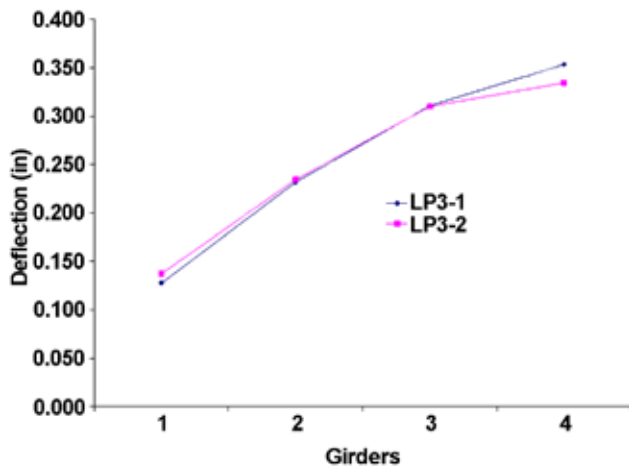


Figure Q21 Maximum deflections for load path 3.

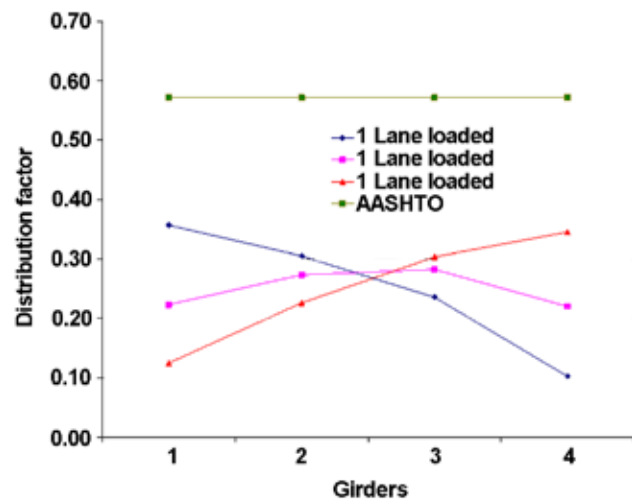


Figure Q23. AASHTO design live load distribution.

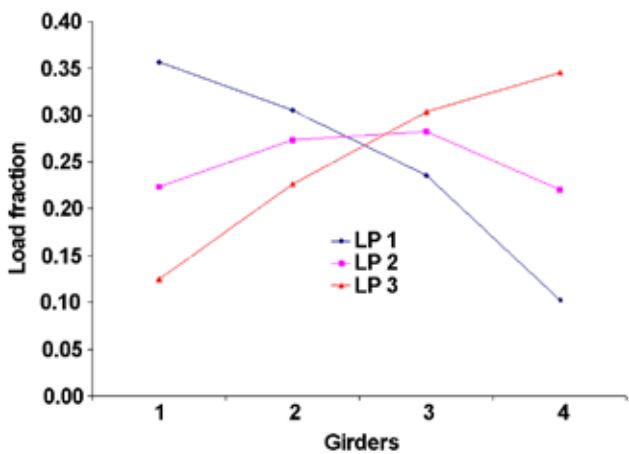


Figure Q22. Load fractions for each load path.

Table Q1—Maximum girder deflection

	Maximum midspan deflection for each passing (in.)
Load path 1	0.413
Load path 2	0.284
Load path 3	0.354

Table Q2—Most critical n value

	n value for girder of most deflection for each load path
Load path 1	760
Load path 2	1,106
Load path 3	887

girders is repeatable. Consequently, only one passing for each load path will be included in the results following this section.

Deflection Criteria

Appendix B discusses recommendations for live load deflection limit states for timber bridges in the form L/n (Table B1). Table Q2 lists the n value for the girder of most deflection for each load path.

The minimum n value of the three load paths was 760. This value is greater than all the minimum recommended values for timber girders stated in Table Q2.

Distribution Factors

Appendix B discusses load distribution factors and the calculation of load fractions. Figure Q22 shows the load fractions for each girder for each load path.

The design live load distribution factors for interior girders as prescribed by AASHTO for glued-laminated panel deck timber bridges is $S/10.0$ for one design lane loaded and two or more design lanes loaded, where S is the transverse spacing between adjacent girders. For this bridge, the exterior lane live load distribution factors were assumed equal to that of the interior lanes. Shown in Figure Q23 is the comparison of design live load distribution values and actual live load distribution. Notice how the design live load distribution factors exceed all the actual live load distribution factors.

Differential Deflections

Appendix B discusses maximum recommended differential deflection between adjacent girders. Figures Q24 to Q26 show the differential deflections between adjacent girders for load path 1, 2, and 3, respectively. The maximum differential deflections between adjacent girders are presented in Table Q3.

The maximum differential deflection of 0.154 in. occurs in load path 1 and equals 0.040 in. per ft of girders spacing.

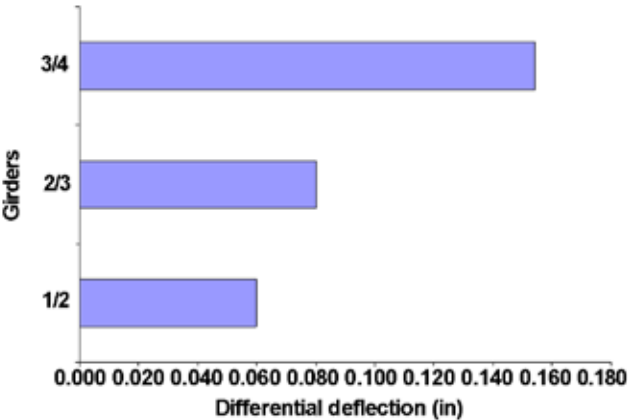


Figure Q24. Differential deflections for load path 1.

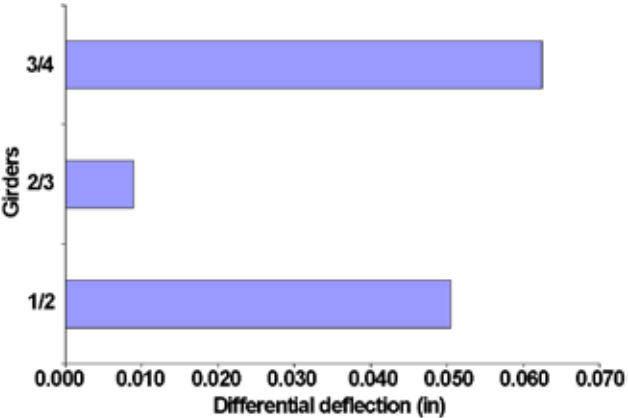


Figure Q25. Differential deflections for load path 2.

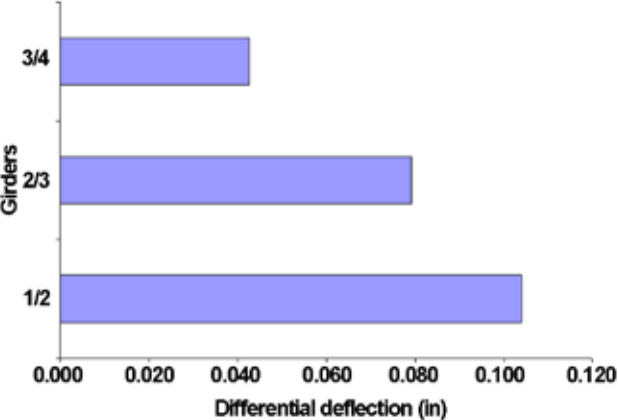


Figure Q26. Differential deflections for load path 3.

This does not appear to be an issue as the amount is relatively small. The same is true for load paths 1 and 3 as the maximum differential deflections are also both small. If the differential deflections were large, the possibility exists that the load was not well distributed transversely between these two girders or the assumption that both girders are of equal stiffness was false.

Table Q3—Maximum differential deflection

	Maximum differential deflection at midspan between adjacent girders (in.)
Load path 1	0.154
Load path 2	0.063
Load path 3	0.104

Strain

The intent of collecting strain data was to estimate maximum stresses in the girders and to determine if composite action between the deck and girders was present.

Maximum stresses are determined using the maximum strain values and an estimated modulus of elasticity of the girder. Maximum strain achieved in the girders was at midspan with compression and tensile strains of 342 and 241 microstrain, respectively. The strain plot at midspan is shown in Figures Q27 to Q29 for load paths 1, 2, and 3, respectively. The compressive strains, or negative strains, constitute the top portion of the graph, and the tensile strains, or positive strains, constitute the bottom portion of the graph. It is assumed that all girders remain linearly elastic during loading, therefore a direct relationship exists between stress and strain and the estimated modulus of elasticity can be used to determine the stress. The resulting stresses are discussed in the following section.

Figures Q27 to Q29 also illustrate the proportion about the neutral axis at midspan. The proportional pattern of the data signifies that there is very little if any composite action with the deck (that is, the girders act independently of the deck when subjected to bending).

Moisture Content

Moisture content measurements were taken at 11 locations on the underside of the bridge. Measurements were taken at the bottom of girders 1, 2, 3, and 4 at the midspan and east abutment. The bottom of the deck between girders 2 and 3, 3 and 4, and outside of girder 1 was measured at midspan. Measurements ranged from 10.5% to 21.2%. The moisture content measurements are summarized in Table Q4.

Discussion of Results

The following discussion is based on the results previously presented, including deflections at midspan, distribution factors, differential deflections, girder strain, and moisture content.

The deflection of the girders in and of itself does not exceed the deflection that would critically affect strength because timber strength is not critically affected until deflections become excessive. Also, each of the maximum girder deflections for each load path meets all recommended limit states

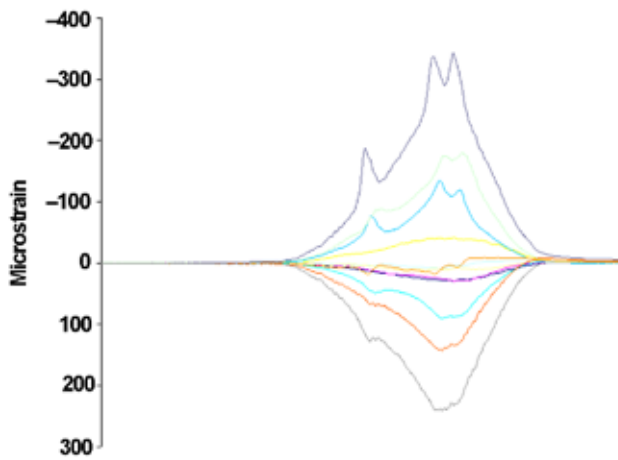


Figure Q27. Strain at midspan for load path 1.

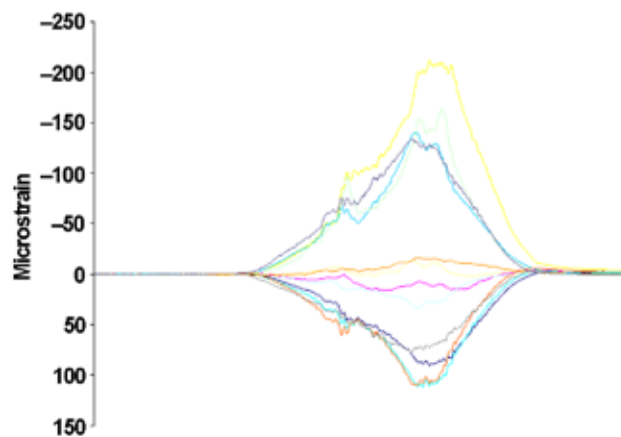


Figure Q28. Strain at midspan for load path 2.

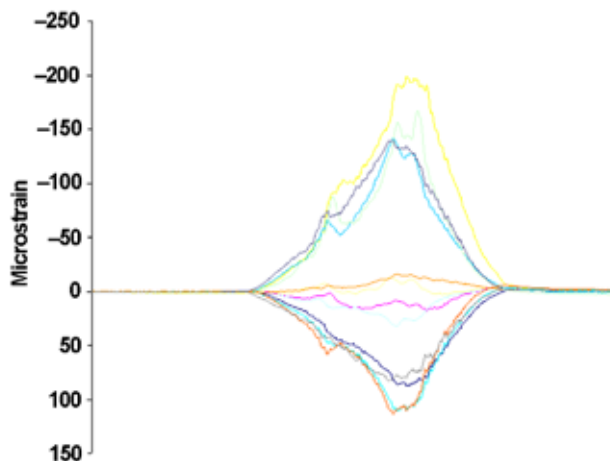


Figure Q29. Strain at midspan for load path 3.

Table Q4—Moisture content reading summary

Location	Moisture content value (%)
Girder 1, east abutment	11.9
Girder 1, midspan	12.1
Girder 2, east abutment	11.9
Girder 2, midspan	10.5
Girder 3, east abutment	13.4
Girder 3, midspan	11.6
Girder 4 east abutment	12.0
Girder 4 midspan	11.4
Bottom of deck between girders 2 and 3	21.2
Bottom of deck between girders 3 and 4	21.0
Outside of girder 1	18.1

for live load deflection derived primarily from wearing surface degradation and maintainability.

Exceeding the live load deflection recommendations can have adverse affects on the structure fasteners, wearing surface, and aesthetics, among other factors. Mechanical fasteners such as bolts or nails could become loose or even fail if excessive girder deflections exist. Aesthetically, failed fasteners and wearing surface cracking produces a displeasing sight and perception of an unsafe bridge.

The wearing surface is susceptible to cracking when live load deflection limits are exceeded as asphalt has very little fatigue resistance. Numerous problems associated with cracking exist, including seepage, decay, and corrosion. Water seepage through the deck can create conditions ideal for wood decay and corrosion of fasteners, reducing the lifetime of the bridge. In addition, reduced strength in the girders is also often a result of decay.

Through visual inspection, transverse cracks in the wearing surface were found only at midspan and the roadway and bridge transitions. Overall, the wearing surface was in good condition.

Differential deflections between adjacent girders could result in wearing surface cracking if those deflections are large. Recommended values of differential deflection are not published; therefore a defined limit does not exist. Even so, the authors of this report having investigated other timber bridge research have advised that a differential deflection limit of 0.05 in. per ft of girder spacing could be used. This bridge was within that limit.

The distribution factor of each girder is within the design live load distribution factors prescribed by AASHTO for glued-laminated panel deck timber bridges.

Strain data for timber bridges should be considered supplementary as the intrinsic properties of wood limits their use for primary analysis. Nevertheless, Figures Q27 to Q29 do show a reasonable relationship between the truck position and strain pattern. Assuming that the maximum values of compressive and tensile strain are in fact correct, the maximum compressive and tensile stresses can be obtained. The maximum overall compressive and tensile strains obtained from the three load paths are 342 and 241 microstrain, respectively. These strains equate to maximum stresses of 393 and 277 lb/in², respectively. If the strains are normalized to the AASHTO tandem load design, stresses of 509 and 359 b/in² are obtained. Allowable stress design limits the total compressive and tensile stresses anywhere from 1,150 to 1,750 b/in², depending on the wood grade and moisture content. Therefore it appears that allowable stresses are not exceeded by standard load trucks.

Due to the climate in western Montana, lower moisture contents were expected and also found, except for in the deck locations. The amount of water present in wood can modify its physical properties. With increasing moisture content the strength of the wood decreases until the moisture content reaches the point of fiber saturation. At this point, the wood no longer continues to lose strength with increasing moisture content, nor does wood regain any lost strength.

Aside from the deck locations, the moisture content percentages were all within a couple percentage points of one another. This shows that those tested areas are not subjected to vastly different amounts of moisture.

Conclusions

Several methods of condition and performance investigation were performed on the Trapper Creek Bridge: Past inspection reports were reviewed; an onsite visual inspection was performed by Iowa State University's Research Team to verify prior inspection report comments and to more fully investigate element level condition; finally, using a loaded tandem-axle dump truck, a static load test was performed to gather performance data. The bridge was subjected to three load cases: a single pass 2 ft from each curb and another over the centerline of the bridge. Deflection and strain data were acquired at locations of interest.

Review of past inspection reports and the performed visual inspection did not reveal any areas of immediate concern. The condition of the bridge was consistent with other bridges similarly aged and subjected to similar weathering and loading conditions.

Cracking of the asphalt was only visible at midspan and the bridge ends, and overall the deck was in good condition. Even so, washing was visible throughout the girders and appears to be the result of seepage through the deck.

Checking and weathering was more evident in the abutment columns, sills, and backwalls than in the superstructure of the bridge.

The bridge performance under live load was within design criteria for allowable stresses and live load distribution. The design value of allowable stress is approximately 1,500 b/in², which exceeds the applied stress if the design vehicle were to travel the same load paths. Live load distribution factors were within AASHTO's prescribed code provisions. Deflection values at midspan met all the recommended maximum values.